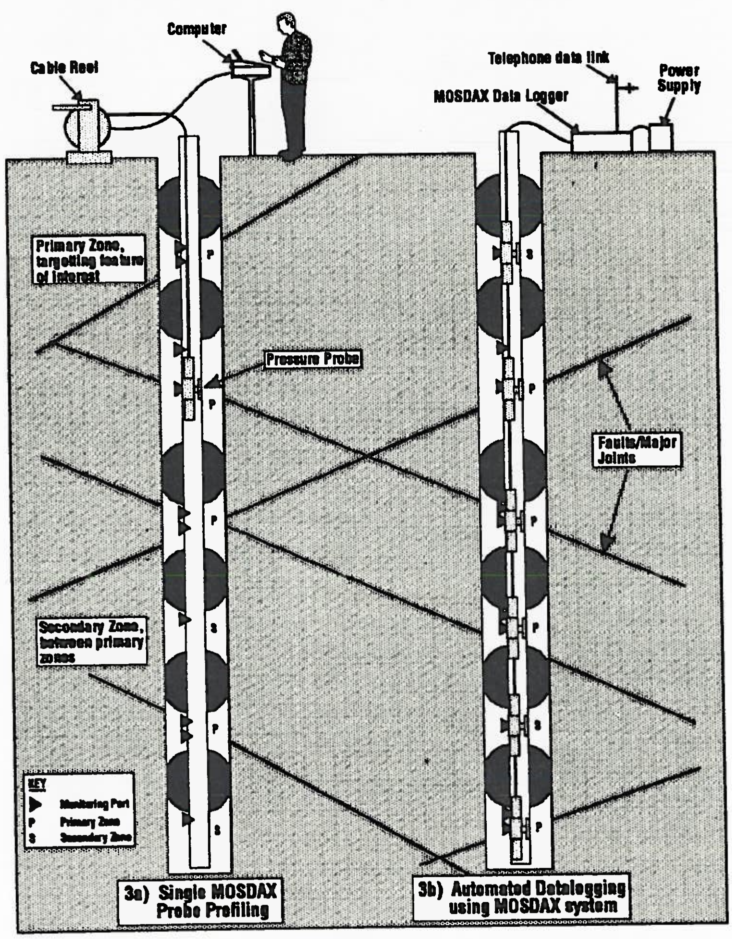
OHIO RIVER VALLEY SOILS SEMINAR XXVI

SITE INVESTIGATIONS

Geotechnical and Environmental



Clarksville, Indiana
October 20, 1995

ORVSS XXVI
SITE INVESTIGATIONS
Geotechnical and Environmental

8:45 am	**Welcome**
9:00 am	**Microgravity Techniques for Detection of Karst Subsurface Features**, by Nicholas C. Crawford
9:45 am	**Geotechnical and Environmental Investigation Methods for New Tunnel Construction: A Case Study**, by Daniel J. Hurst, James W. Martin, and Bernard H. Voor III
10:15 am	**Accelerated Site Characterization Techniques Implemented at U. S. Army Corps of Engineers Contaminated Sites**, by James D. Dzubay and Mark S. Meyers
10:35 am	**Break**
11:05 am	**Use of the Ground Penetrating Radar and Seismic Sounding System for Geotechnical Investigation**, by M. Zoghi, P. J. Wolfe, B. H. Richard, G. F. Mitchell, and T. Nogami
11:35 am	**Investigation and Instrumentation of Deep Lacustrine Clay Deposits in the Cuyahoga River Valley**, by Paul H. Anderson and Stuart P. Ravary
11:55 am	**Laboratory and Field Measurements of Coal Refuse Properties**, by Mohamed M. Nofal and R. Michael Holbrook
12:15 pm	**Lunch**
1:30 pm	**Announcements**
1:40 pm	**Design and Implementation of a Multipurpose Groundwater Monitoring System at Sellafield, U. K.**, by Chris D. Eldred and John A. Scarrow
2:25 pm	**Site Characterization Methods for the Design of a Groundwater Extraction System in a Bedrock Aquifer**, by D. Joseph Wessley, Richard H. Weber, and Daniel A. Otzelberger
2:55 pm	**Use of Existing Geotechnical Data to Supplement Site Investigations**, by William J. Pfalzer
3:15 pm	**Break**
3:45 pm	**Site Characterization Aided by Evaluation of Pumping Test Data on Environmental Remediation Projects**, by Barry K. Thacker
4:15 pm	**Geotechnical Characterization of the Waste Pit Material for the Fernald Environmental Management Project**, by Mark T. Bowers and Dale A. Lutz
4:35 pm	**Geotechnical Engineering in Environmental Site Characterization and Restoration Projects**, by Michael J. Saffran and Chris R. Karem
5:00 pm	**Social Hour**

Proceedings of the Twenty-Sixth
Ohio River Valley Soils Seminar

SITE INVESTIGATIONS
Geotechnical and Environmental

October 20, 1995
Holiday Inn Lakeview
Clarksville, Indiana

Sponsored by

Kentucky Geotechnical Engineering Group, ASCE

University of Louisville
Department of Civil Engineering

University of Kentucky
Department of Civil Engineering
and Office of Continuing Education

and

Cincinnati Geotechnical Engineering Group, ASCE

University of Cincinnati
Department of Civil Engineering

ORVSS XXVI
ORGANIZING COMMITTEE

Milton M. Greenbaum, P.E.
Greenbaum Associates, Inc.

D. Joseph Hagerty, Ph.D., P.E.
University of Louisville

Michael J. Hagerty
PSI, Inc.

Steven A. Hite, E.I.T.
U. S. Army Corps of Engineers

Bashar B. Masri, P.E.
Law Engineering

Kevin McIlvaine, P.E.
PSI, Inc.

Randy L. Roberts, P.E.
Fuller, Mossbarger, Scott & May Engineers, Inc.

Michael Ronayne, P.E.
ATEC Associates, Inc.

Jane C. Ruhl, P.E.
U. S. Army Corps of Engineers

C. Robert Ullrich, Ph.D., P.E.
University of Louisville

Gerald T. Vandevelde, P.E.
Ground Engineering & Testing Service, Inc.

ORVSS XXVI
EXHIBITORS

Central Mine Equipment Company
Earth City, Missouri

A. B. Chance Company
Centralia, Missouri

Contech Construction Products, Inc.
Indianapolis, Indiana

Deidrich Drilling Equipment, Inc.
LaPorte, Indiana

Dravo Lime Company
Butler, Kentucky

Elastizell Systems, Inc.
Dayton, Ohio

Global Drilling Suppliers, Inc.
Cincinnati, Ohio

GRL and Associates, Inc.
Cleveland, Ohio

Irrigation Supply Company, Inc.
Louisville, Kentucky

Midwest Construction Products Corporation
Louisville, Kentucky

United Dynamics, Inc.
Jeffersonville, Indiana

Vibra-Tech Engineers, Inc.
Louisville, Kentucky

TABLE OF CONTENTS

Reuboal
— John Fisk instruments
Austin Tx

MICROGRAVITY TECHNIQUES FOR DETECTION OF KARST SUBSURFACE FEATURES

Nicholas C. Crawford, Ph.D.
Center for Cave and Karst Studies
Department of Geography and Geology
Western Kentucky University
Bowling Green, Kentucky 42101

ABSTRACT

Bouguer gravity can identify locations on the earth's surface that have relatively higher or lower gravity caused by lateral variations in subsurface density. The density contrasts of -1.0 to -2.5 g/cm^3 between air, water or clay filled cave passages and limestone bedrock can often be detected from the surface by the use of microgravity. Although gravity surveys usually employ a grid pattern, this research has demonstrated the effectiveness of using traverses established perpendicular to linear subsurface features and groundwater flow paths in karst terrain. The direction of these features is determined by a combination of lineament analysis, dye traces and potentiometric surface mapping. Once a low-gravity anomaly has been confirmed to be a cave, usually by exploratory boring, the general route of the cave passage can be determined by proceeding in a "leap frog" fashion with short parallel traverses. If the caves are relatively large and shallow (less than 100 feet deep), this technique is particularly useful for locating cave streams for determining groundwater flow direction and for installing monitoring or recovery wells directly into the karst conduit that drains a hazardous material site. Crawford has drilled over 60 such wells into caves detected by low-gravity anomalies. Microgravity can also detect voids in the regolith above bedrock and also depth to bedrock. It is therefore a useful tool for identifying potential sites for sinkhole collapses and for investigating existing collapses.

INTRODUCTION

Gravity surveys are used to detect variation in the density of subsurface materials. Variations in the earth's gravitational force higher than normal indicate underlying material of higher density while areas of low gravity indicate areas of lower density. In order to detect voids or cavities, very high precision is required. Accurate gravity readings to 10 microgals (1 gal = 1 m/s^2) are necessary. This is equal to 1 part in 1,000,000 of the earth's normal gravity. A LaCoste and Romberg Model D Microgal Gravity Meter which has a 1 microgal sensitivity was used for all the investigations discussed in this paper.

MICROGRAVITY RESEARCH PROCEDURE

A base station is established near the site to be surveyed. Gravity is measured at this base station at approximately a one-hour interval while the meter is being used in order to derive instrument drift. A base station derived instrument drift curve is interpolated to the time of each survey station reading and each station reading is then corrected for instrument drift. Earth tide corrections are made for each gravity reading and differences in elevation between the survey station and the base station are then compensated for using free-air correction procedures. The free-air effect compensates for the decrease in gravity with elevation due to increasing distance from the center of the earth. Free-air gravity indicates that the reference ellipsoid (gravity on a sea-level, rotation ellipsoidal mode of the earth--a function of latitude) and free-air effect are included in calculating the theoretical gravity. The free-air gravity is modified to obtain simple Bouguer gravity by applying the Bouguer slab effect correction. This correction refers to the attraction of the slab of material, which is caused by variation in density, between the station elevation and sea-level. Terrain corrections of the microgravity data are often not necessary in order to measure relative differences in gravity when measured along a short traverse.

In the karst areas discussed in this paper, the following density values were assumed:

- air = 0 gm/cm^3
- water = 1.0 gm/cm^3
- clay = 1.5 gm/cm^3
- limestone = 2.5 gm/cm^3

Therefore, density contrasts of -1.0 to -2.5 g/cm^3 were anticipated for any subsurface cavity, depending on whether the cavity was filled with air, water, or clay.

Although microgravity subsurface investigations usually consist of measuring gravity at stations established in a grid pattern, Crawford, Webster, and Winter (1989) have demonstrated the effectiveness of using traverses established perpendicular to linear subsurface features and groundwater flow paths in karst terrain.

DETECTION OF SUBSURFACE FEATURES IN KARST TERRAIN

Bouguer gravity can identify locations on the earth's surface that have relatively higher or lower gravity caused by lateral variations in subsurface density. Crawford has used microgravity extensively to locate bedrock caves from the ground surface. The lower density of the air, water, or mud within a cave compared to the surrounding carbonate rock results in a low-gravity anomaly. He has also used microgravity to locate voids in the regolith (unconsolidated material above bedrock) which are potential sinkhole collapses. Since regolith is less dense than limestone bedrock, Bouguer gravity can also identify variations in depth to bedrock. In limestone areas, depth to bedrock is often very irregular with limestone pinnacles that protrude upward and cutters that extend downward. Cutters are V-shaped regolith-filled crevices formed by solution of the limestone by soil water as it percolates down to the karst aquifer. Regolith arches form as regolith spalls into solutionally enlarged voids in the bedrock. In some cases regolith may then be carried off by a cave stream. For these reasons, low-gravity anomalies indicate bedrock caves, voids in the regolith or places where depth to bedrock is abruptly greater which is often indicative of places where regolith may be descending along regolith arches into bedrock crevices.

MICROGRAVITY USED TO DETECT BEDROCK CAVES

Several researchers have demonstrated that gravity can be used to detect large bedrock caves (Omnes G. 1976; Kirk K.G. and Werner, E. 1981; Butler, D.K. 1983 and Smith, D.L. and Smith, G.L. 1987). Figures 1, 2, and 3 are examples where relatively large low gravity anomalies along level traverses reveal the location of the underlying cave passages. Although some studies had identified anomalies that were hypothesized to be caves, few, if any, wells had been drilled into these anomalies to conform that they did in fact indicate cave passages previous to research performed by Crawford in 1985. Toxic and explosive vapors rising from contaminated cave passages into homes under Bowling Green, Kentucky in 1984 and 1985 resulted in an intensive effort to find the cave passages over a relatively

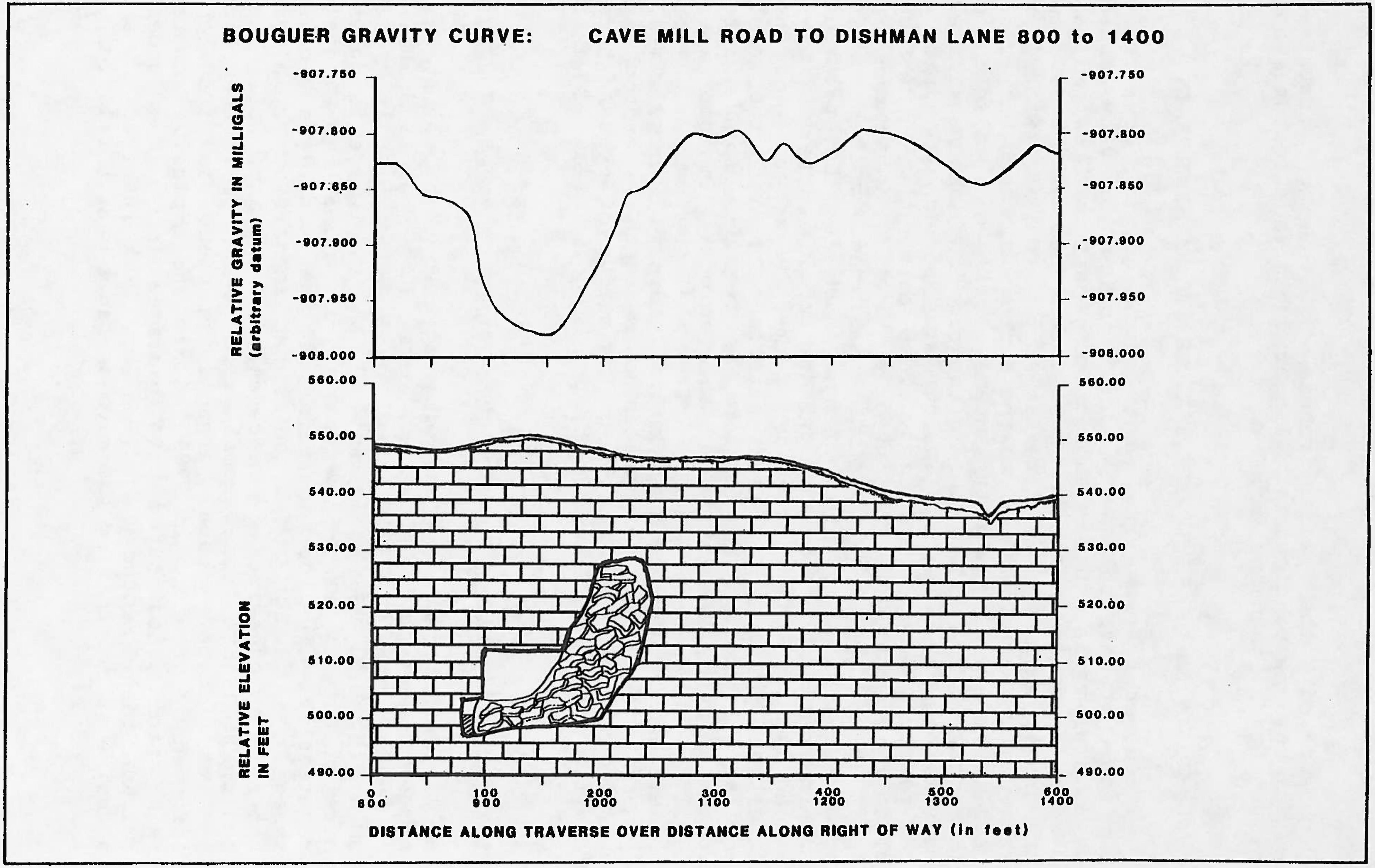


FIGURE 1. Microgravity measurements were taken at a 10 foot interval along a traverse perpendicular to State Trooper Cave in Bowling Green, Kentucky. Microgravity measurements were taken along the proposed route of the Dishman Lane Extension. The route was modified to avoid crossing over the cave at a place were the roof had collapsed.

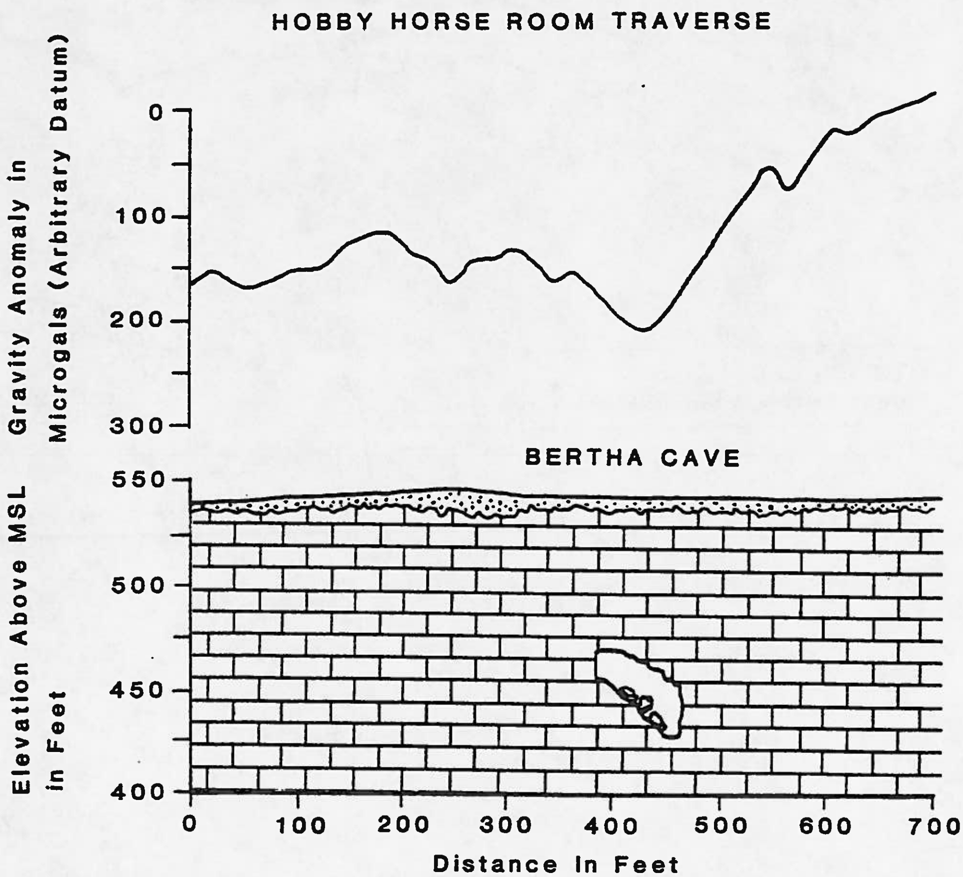


FIGURE 2. Microgravity traverse over Hobby Horse Room in Bertha Cave in Bowling Green, Kentucky. The cave room is over 70 feet below the surface.

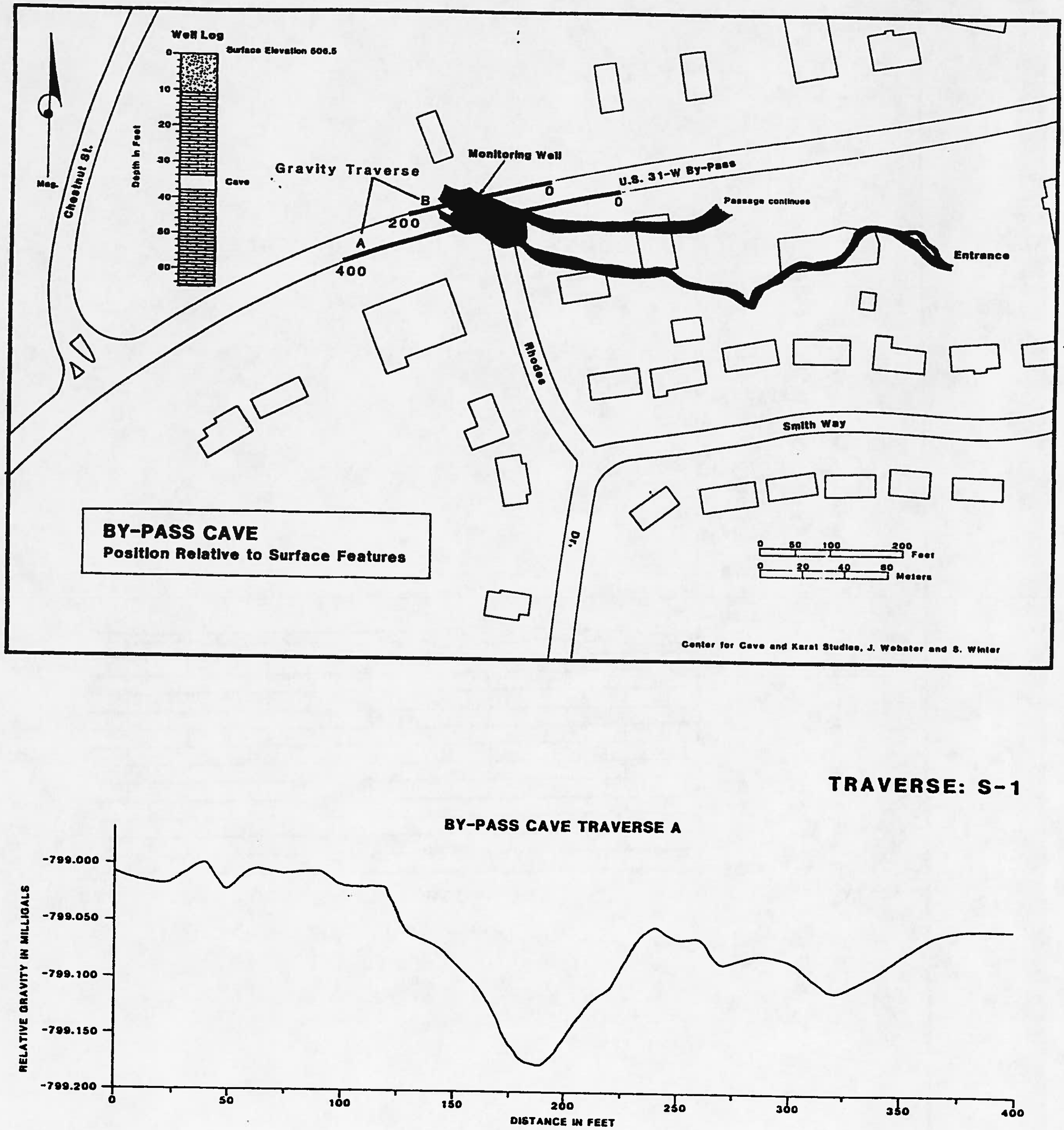


FIGURE 3. Microgravity low anomaly along Traverse S-1 where it crosses over By-Pass Cave, Bowling Green, Kentucky.

large area (Crawford, 1989; Crawford, Webster, and Winter, 1989).

Although several geophysical techniques for locating caves were considered and a few tried, the most successful was microgravity. The best results were obtained by taking microgravity measurements with a LaCoste and Romberg Model D Microgal Gravity Meter at ten-foot intervals along traverses perpendicular to a hypothesized route of a cave stream. The hypothesized route was derived from topographic analysis, knowledge of local hydrogeology, dye traces and a detailed water table map. Voids existing beneath low-gravity anomalies were confirmed by exploratory drilling (Figures 4, 5, and 6). By proceeding in a "leap frog" fashion with short parallel traverses, the route of the cave was established. The precise location of Creason Cave, a paleosection of the large Lost River Cave, was determined in this manner. After confirming its existence with three small-diameter exploratory holes and with a downhole camera, a 30 inch diameter well was drilled into the cave to provide access by cavers for mapping. Crawford has used this microgravity traverse technique on several occasions for locating sites for downgradient monitoring wells or recovery wells into cave streams. For example, lineament analyses with perpendicular microgravity traverses were used to locate a sediment-filled cave about 200 feet downgradient from a train derailment site near Lewisburg, Tennessee (Figure 7). Approximately 15,000 gallons of chloroform, a DNAPL, sank into the karst aquifer. Sediment was excavated at a place where the cave roof had collapsed, and a large-diameter recovery well was installed to pump chloroform from the aquifer (Crawford and Ulmer, 1994).

MICROGRAVITY USED FOR SINKHOLE COLLAPSE INVESTIGATIONS

Bouguer gravity has been used at several locations by Crawford to investigate subsurface conditions in the vicinity of sinkhole collapses. Virtually all sinkhole collapses in karst areas are regolith collapses (Figures 8 and 9). Although bedrock cave roofs do occasionally collapse, these collapses are so rare that they do not constitute a serious threat. However, regolith collapses occur frequently in many karst areas.

Regolith collapses occur due to the formation and the eventual collapse of regolith arches (domes). Urban development in karst areas will almost always produce an increase in the collapse or regolith arches. Regolith arches form by the downward movement of unconsolidated

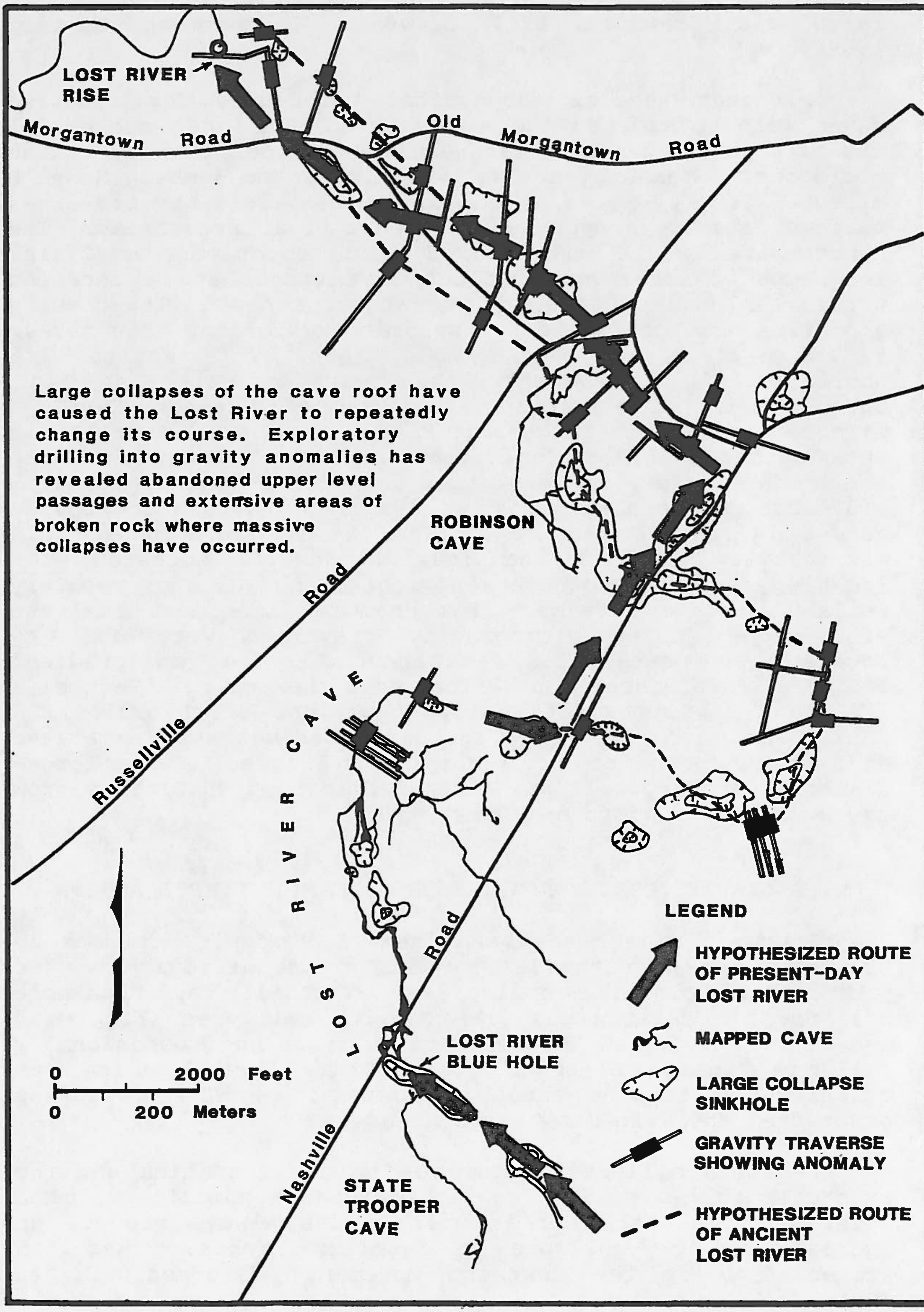


FIGURE 4. Mapped portions of Lost River Cave and hypothesized present day and ancient routes as determined by microgravity (Crawford, 1986).

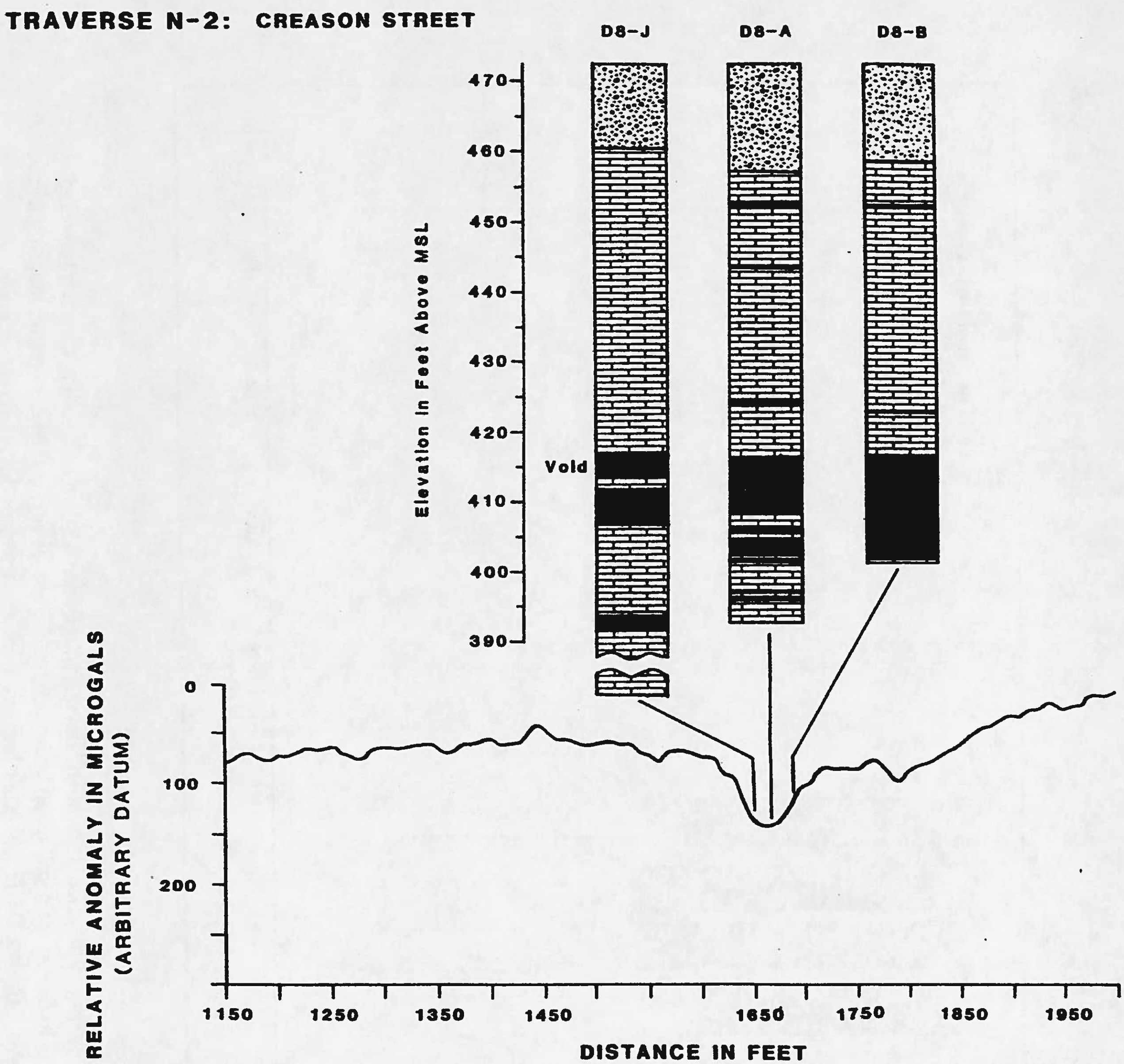


FIGURE 5. Three borings into this microgravity anomaly intersected a water-filled cave over 15 feet high and 50 feet wide.

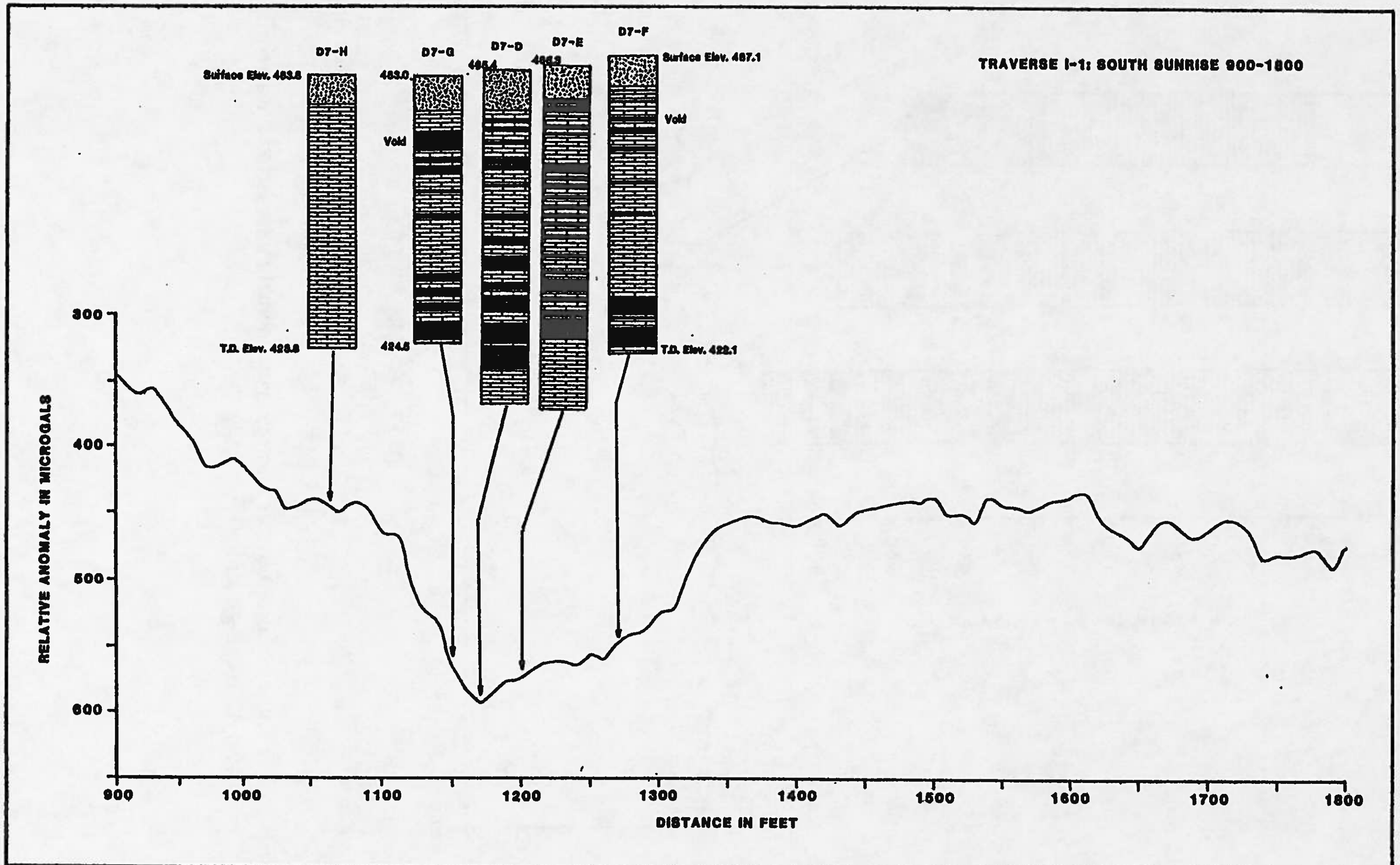


FIGURE 6. Four borings into this large low-gravity anomaly along South Sunrise Street in Bowling Green intersected numerous voids and boulders indicative of a collapsed bedrock cave. There is no surface expression that might reveal the presence of the collapsed cave. Boring D7-H and other borings along the traverse did not intersect voids.

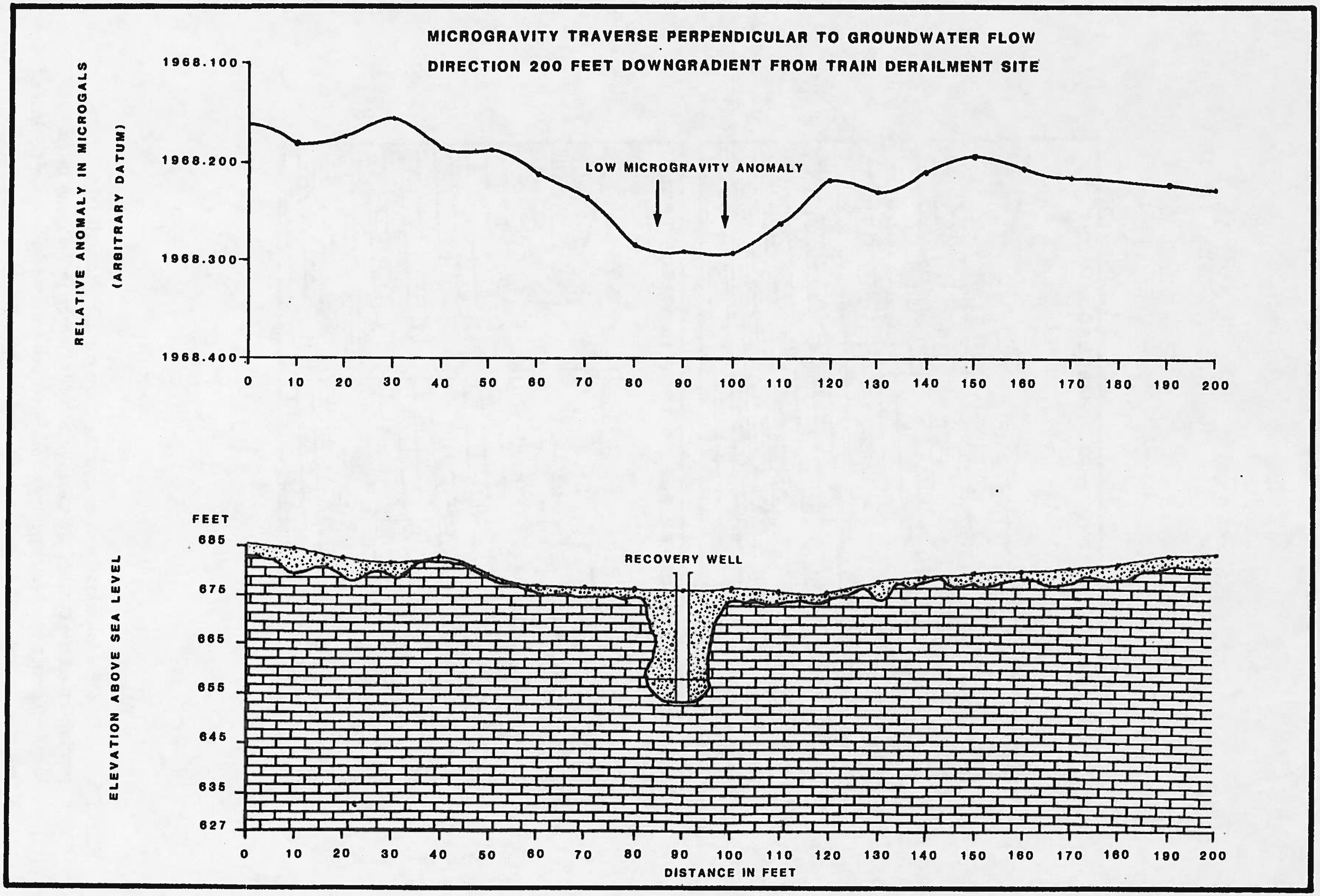


FIGURE 7. Lineament analysis and microgravity traverses were used to locate a sediment-filled cave about 200 feet downgradient from a train derailment site near Lewisburg, Tennessee. Approximately 15,000 gallons of chloroform, a DNAPL, sank into the karst aquifer. Sediment was excavated at a place where the cave roof had collapsed and a large-diameter recovery well installed.

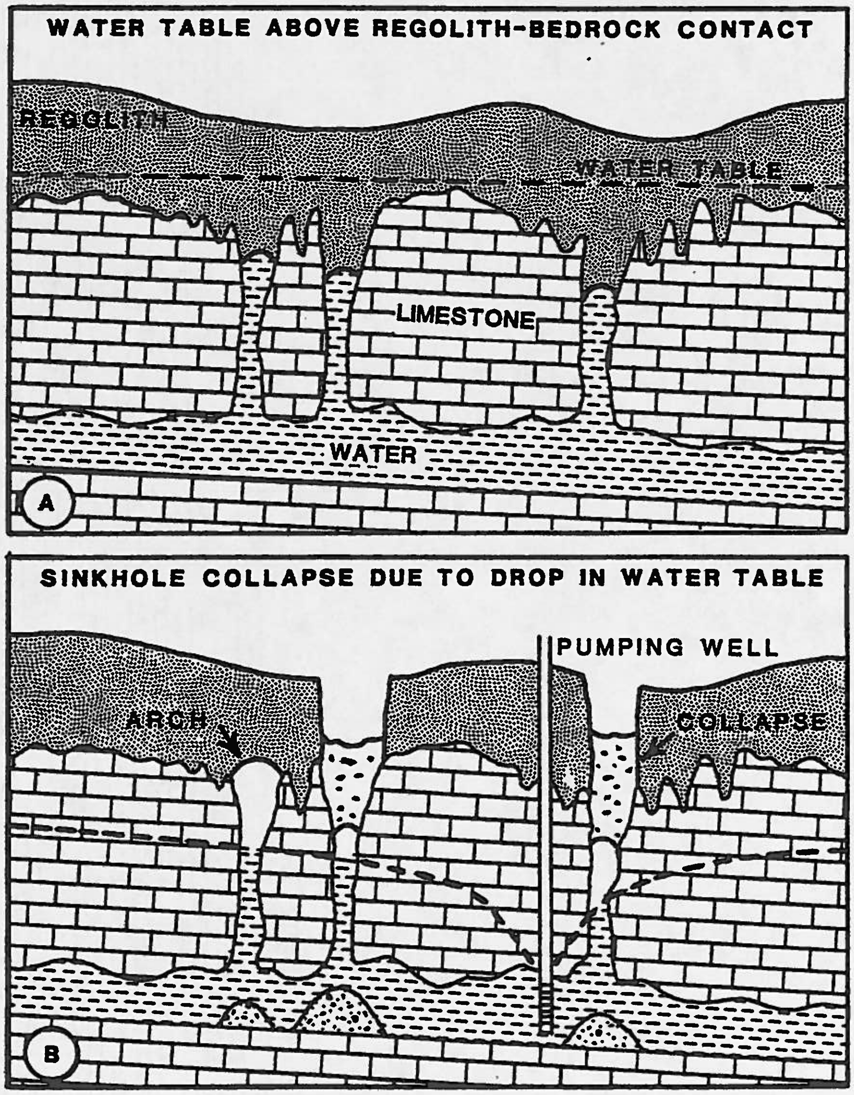


FIGURE 8. Sinkhole collapses in areas were the water table is above the regolith-limestone contact are usually caused by a drop in the water table. Regolith arches spanning openings in the bedrock collapse because of the loss of buoyant support and because of downward moving surface water.

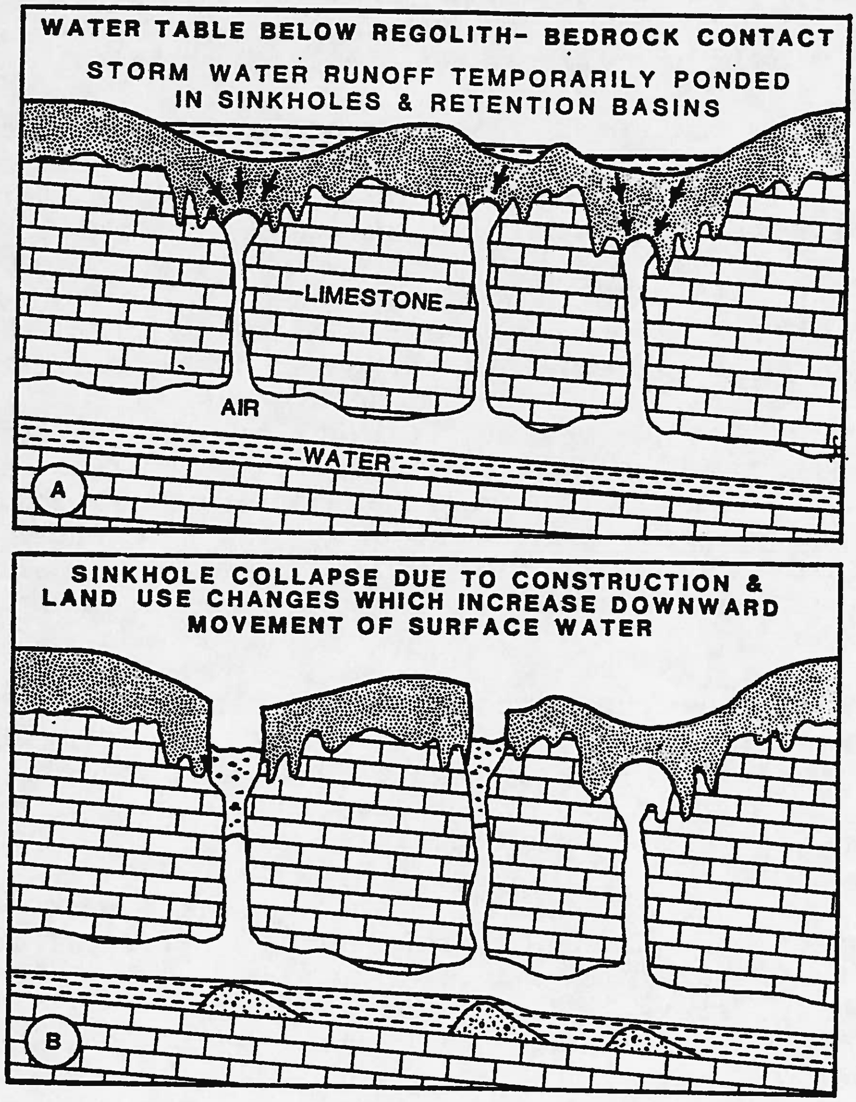


FIGURE 9. Sinkhole collapses in areas where the water table is below the regolith-bedrock contact are usually caused by an increase in the downward movement of surface water. Land use changes and construction activities that concentrate surface water in drains, sinkholes, and impoundments may locally increase the downward movement of surface water and induce the collapse of regolith arches.

sediments into voids in the bedrock. In areas where the water table is usually above the regolith-bedrock contact, collapses often occur when the water table drops during droughts or high-volume pumping (Figure 8). Physically, the collapses in these cases are caused by loss of buoyant support for the regolith arches which span openings in the limestone. Collapses are also caused by spalling of saturated regolith down the opening, enlarging the arch, and eventually causing collapse at the land surface.

Regolith collapses also may occur in situations where the water table is usually below the regolith-bedrock contact (Figure 9). Construction and land use changes that concentrate surface runoff in sinkholes, retention basins, ditches, and ponds may locally increase the downward movement of water resulting in the piping of saturated regolith into openings in the limestone (Figure 10). Most of the sinkhole collapses investigated in the Warren County, Kentucky area by the Center for Cave and Karst Studies at Western Kentucky University (Sinkhole Collapse Inventory, Vols. 1-4) are of this type (Crawford, Webster, and Veni, 1990). An estimated 70 percent are man-induced collapses of existing regolith arches (Figure 11). Changes in the surface drainage associated with farming, and particularly urban development, are believed to be the primary cause of most collapses.

Both of the preceding collapse mechanisms often work together. The regolith-bedrock contact is often within the zone of the storm elevated water table so regolith arches can be both stopped from below and have sediment spalls due to descending surface waters.

Microgravity will often reveal the existence of a void in the regolith above bedrock and therefore, identify a potential sinkhole collapse. Figure 12 is an example of a microgravity traverse over a regolith void that had collapsed all the way to the surface so that a small hole was actually visible.

Microgravity traverses can be made along foundations previous to building construction to identify low-gravity anomalies. However, depth to bedrock borings into the anomalies are usually needed to establish if they are regolith voids, bedrock caves or cutters (areas of deep regolith between pinnacles).

Crawford has used microgravity at several sites to investigate subsurface conditions in the vicinity of sinkhole collapses. It provides useful information

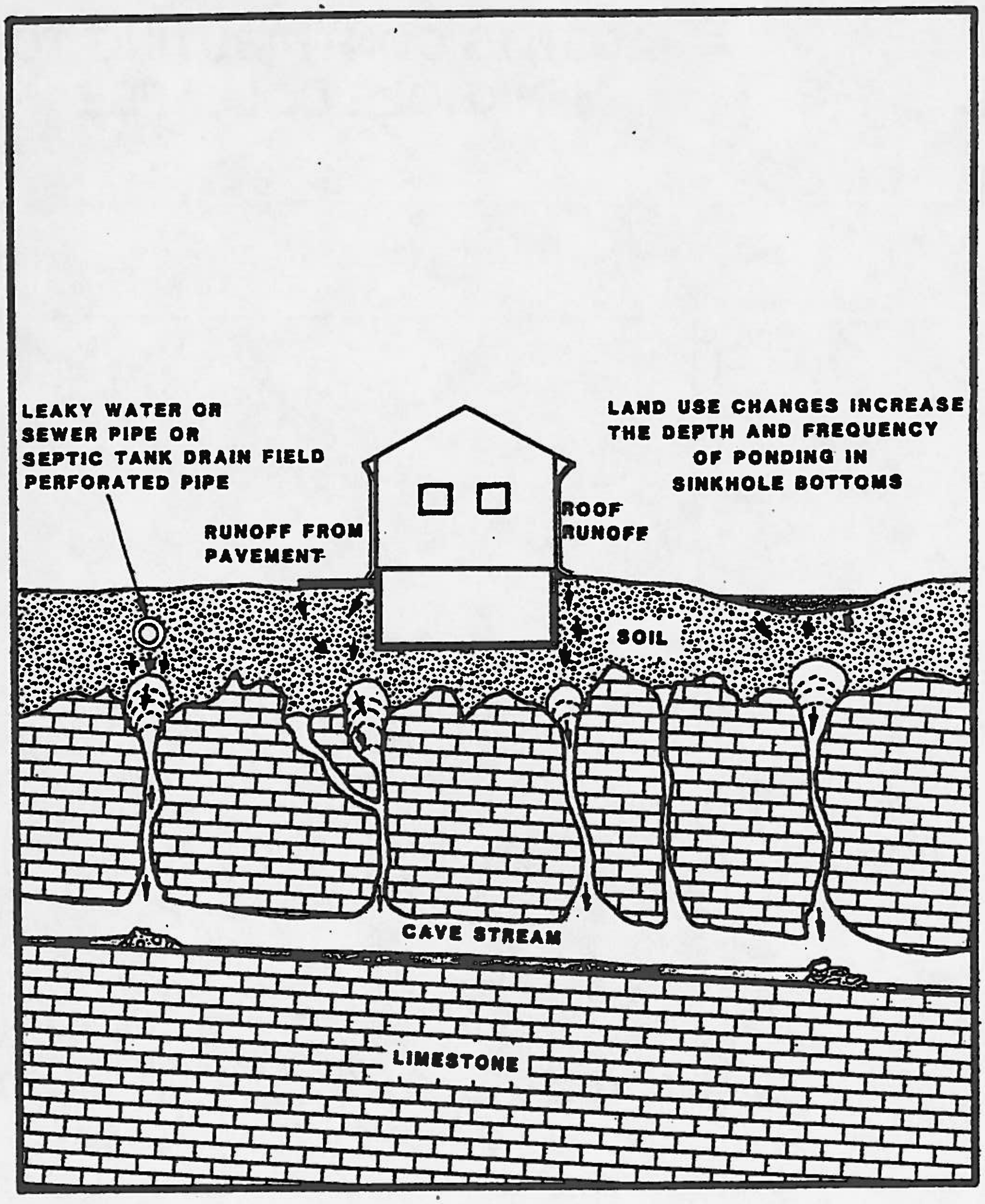


FIGURE 10. Growth of regolith arches toward the surface induced by modification of natural runoff and infiltration conditions.

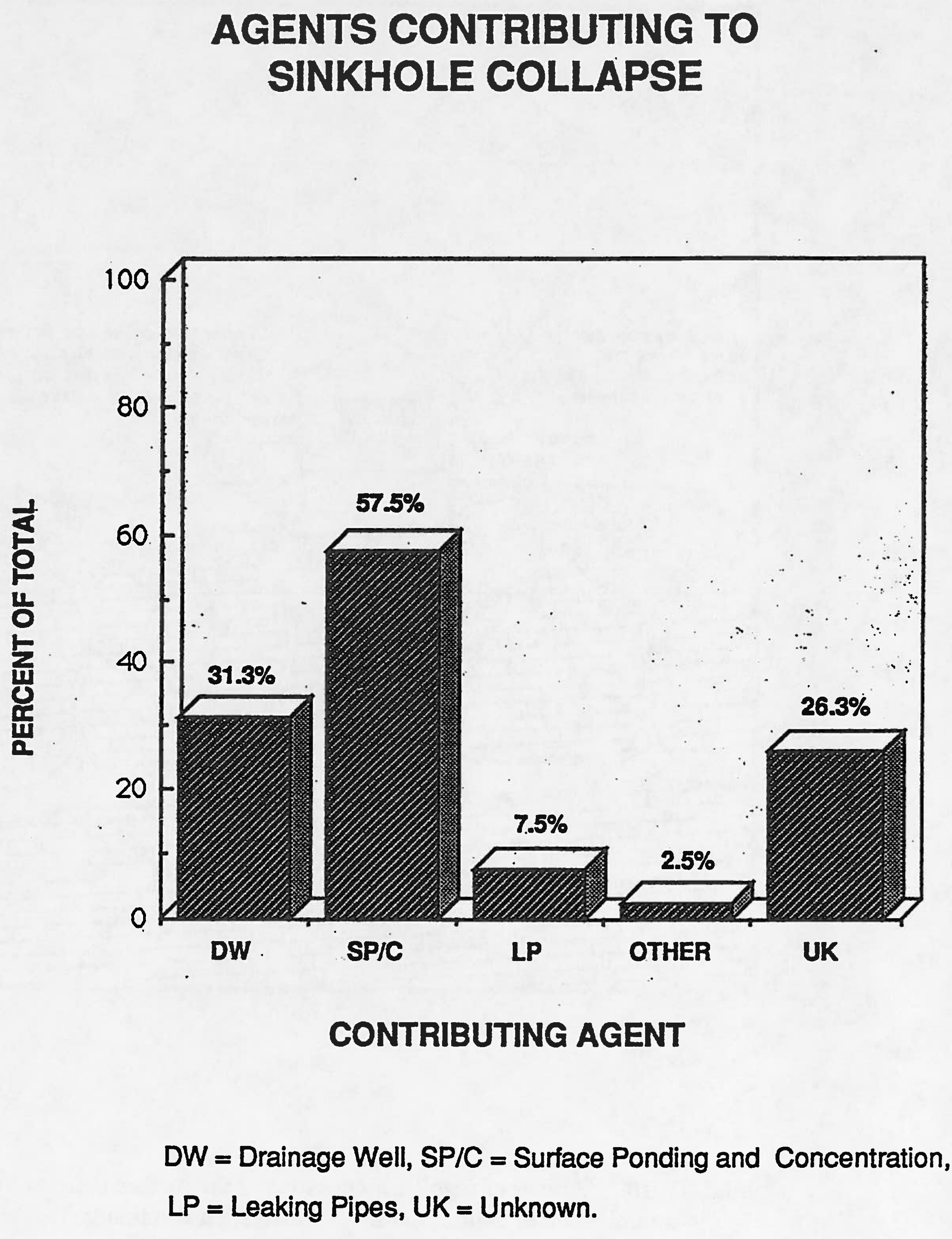


FIGURE 11. Primary agents contributing to sinkhole collapses at 80 inventoried sinkhole collapses within the city limits of Bowling Green, Kentucky (Crawford, Webster, and Veni, 1989).

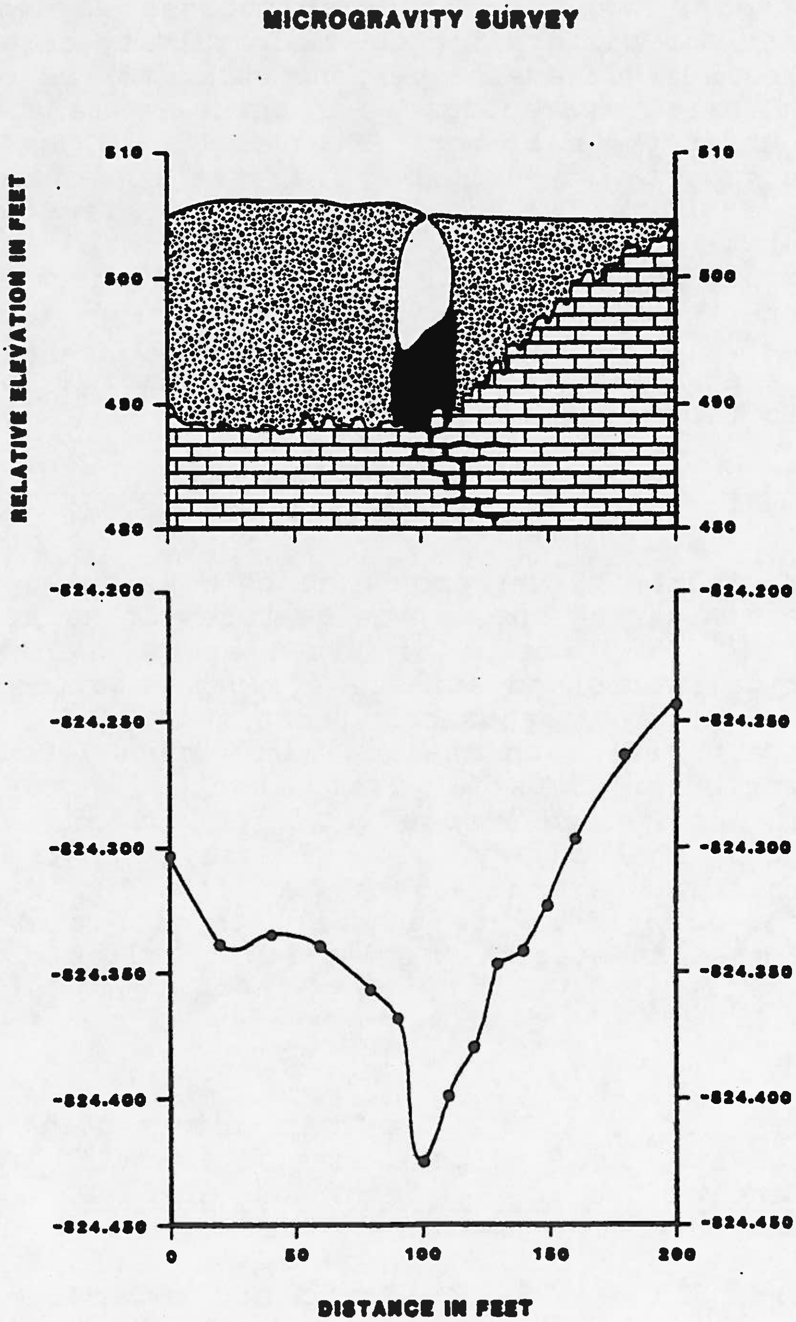


FIGURE 12. Low-gravity anomaly along a traverse over a regolith void that had collapsed all the way to the surface so that a small hole was actually visible.

concerning: a) depth to bedrock, b) extent and shape of the collapse area below the surface, c) location of the crevice, or crevices, into which the regolith is collapsing and d) locations of additional regolith voids in the vicinity of the initial collapse. Figures 13 and 14 are microgravity traverses taken at the site of a sinkhole collapse under a large two-story building. Figure 13 is a traverse taken along the side of the building, primarily to establish depth to bedrock. Depth to bedrock borings were made along the traverse for calibrating the microgravity data. Figure 14 is a microgravity traverse perpendicular to the building. Some of the gravity readings were actually taken in the crawl space under the building. Figures 15, 16, and 17 illustrate a microgravity investigation made at the site of a sinkhole collapse under the shoulder and one lane of a six-lane interstate highway. The investigation was made to determine: a) locations of additional regolith voids that might be ready to collapse and b) shape of existing collapse and depth to bedrock so that a repair strategy could be formulated that would result in minimal obstruction to traffic flow.

CONCLUSIONS

Microgravity has proven to be a useful geophysical tool for investigating subsurface features in karst terrains. It is particularly useful in urban areas where buried pipes, electrical wires and surface structures often limit the use of some other geophysical techniques. If the caves are relatively large and shallow (less than 100 feet deep), it is particularly useful for locating cave streams for determining groundwater flow direction and for installing monitoring or recovery wells directly into the karst conduit that drains a hazardous material site. It is also a useful tool for identifying potential sites for sinkhole collapses and in the investigation of existing collapses.

REFERENCES

Butler, D.K. (1983). "Microgravimetric and magnetic surveys, Medford Cave Site, Florida," Cavity Detection Research Report 1, Tech, Rep GL-83-1. Vicksburg, MS: Army Engineer Waterways Station p.92.

Crawford, N.C. (1989). Toxic and explosive fumes rising from contaminated groundwater flowing through caves under Bowling Green, Kentucky, prepared for City of Bowling Green, Municipal Order No.1 85-83, 54 p.

Crawford, N.C. and Ulmer, C.S. (1994). Hydrogeologic investigations of contaminant movement in karst aquifers in the vicinity of a train derailment near Lewisburg, Tennessee. Environmental Geology, V.23, pp. 41-52.

Crawford, N.C. and Webster, J.W. (1988). Microgravity investigation of the proposed Dishman Lane Extension, Bowling Green, Kentucky, report for Law Engineering, Nashville, Tennessee, 20p.

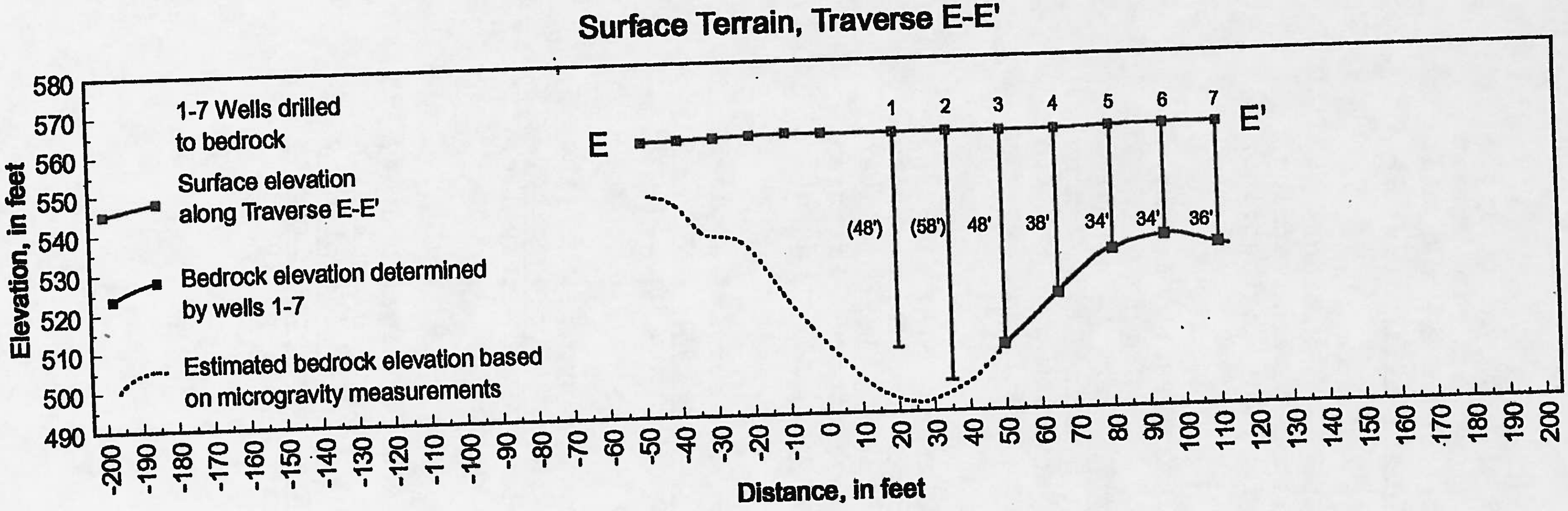
Crawford, N.C., Webster, J.W. and Veni, G. (1980). Sinkhole collapse problems in Warren County, in Crawford, N.C. (ed.) The Karst Landscape of Warren County, prepared for Bowling Green-Warren County Planning and Zoning Commission, Center for Local Government, Western Kentucky University, Bowling Green, Kentucky, pp.71-115.

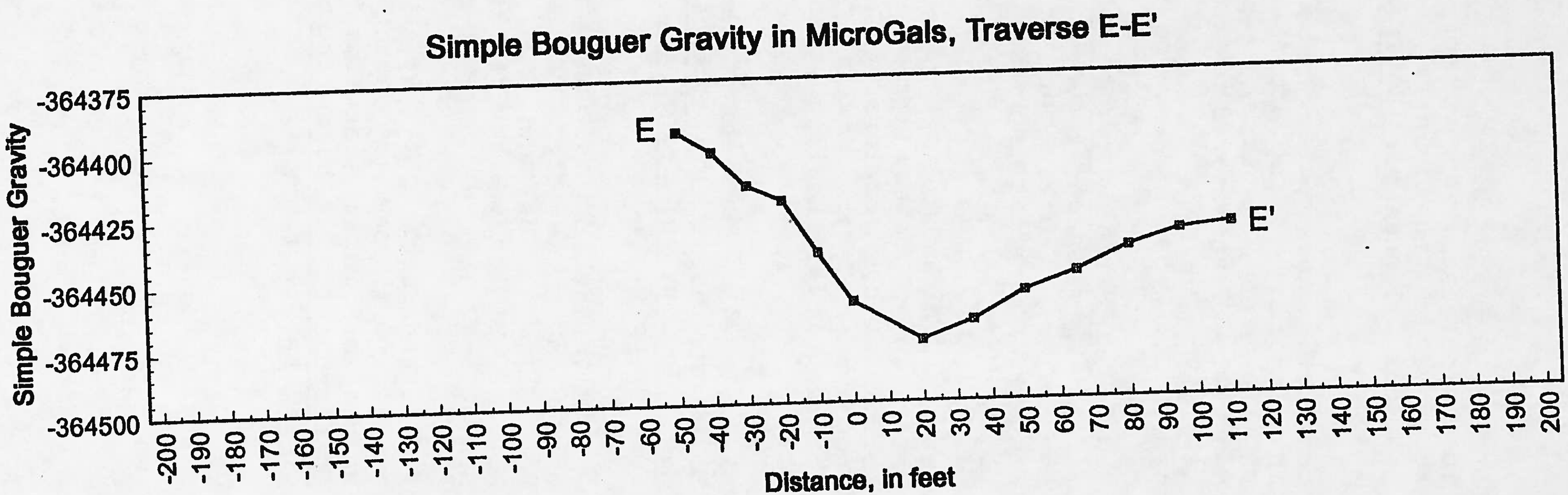
Crawford, N.C., Webster, J.W. and Winter, S.A. (1989). Detection of caves from the surface by microgravity followed by exploratory drilling: Lost River Groundwater Basin, Bowling Green, Kentucky, Prepared for the City of Bowling Green, Municipal Order No. 85-83.

Kirk, K.G. (1974). "Resistivity and gravity surveys applied to karst research," Proceedings of the 4th Conf. on Karst Geology and Hydrology. West Virginia Geological Survey, pp.61-71.

Omnes, G. (1976). "High accuracy gravity applied to the detection of karstic cavities". Karst Hydrogeology, ed. J.S. Tolson and F.L. Doyle UAH, Huntsville, Alabama, USA or W.B. White, Internal. Assoc. Hydrogeol. Mem. 12, 273-284.

Smith, D.L. and Smith G.L. (1987). "Use of vertical gravity gradient analyses to detect near-surface dissolution voids in karst terrains," 2nd Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst, Ed. by Beck B.F. and Wilson, W.L. p. 205-209.





slab density = 2.5

FIGURE 13. Gravity traverse in the vicinity of a sinkhole under a two-story building. Bedrock elevation was determined by borings 1-7.

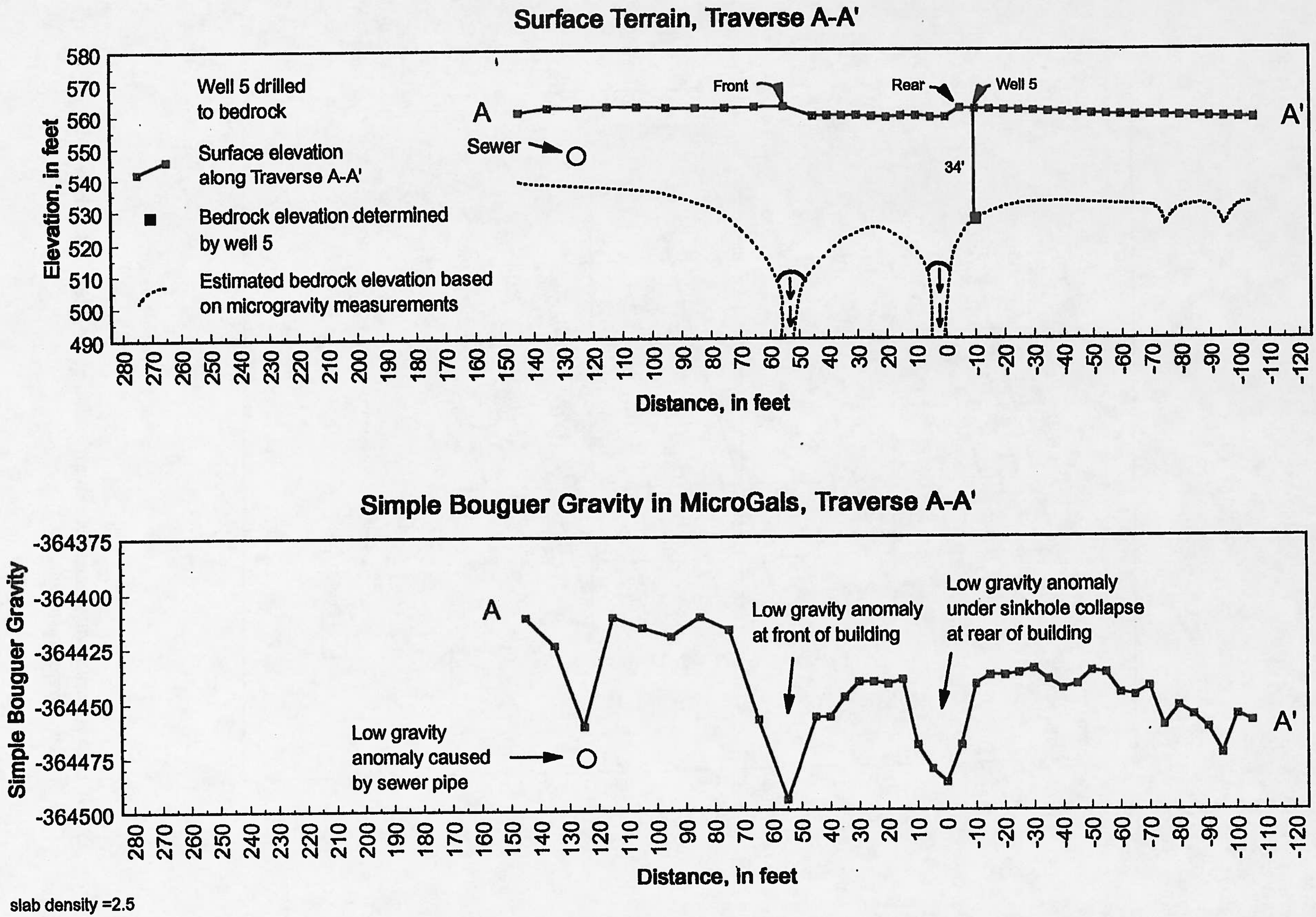


FIGURE 14. Microgravity traverse perpendicular to building. Some of the gravity measurements were taken in the crawl space under the building.

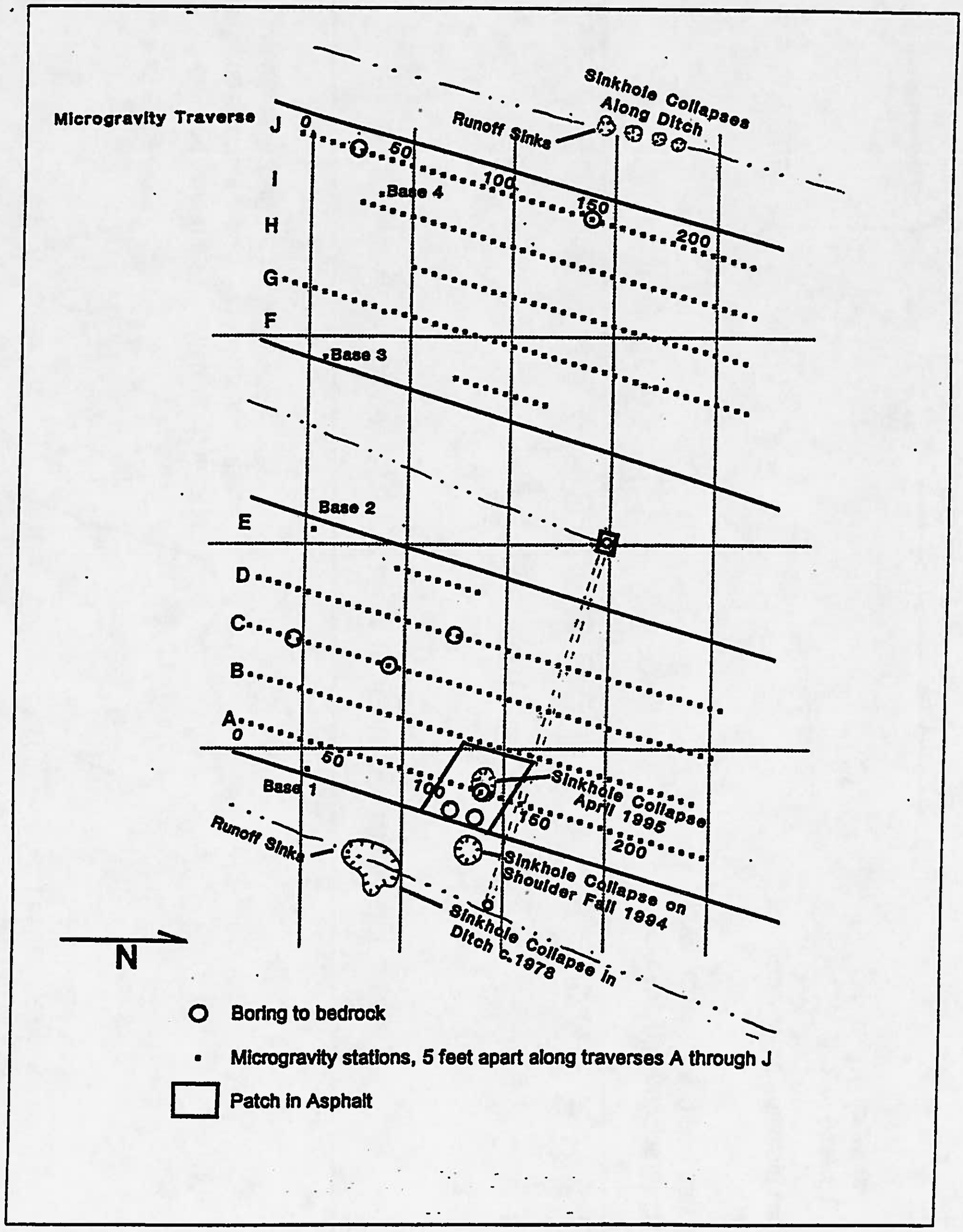


FIGURE 15.Microgravity subsurface investigation at the site of a sinkhole collapse under an interstate highway.

Traverse A

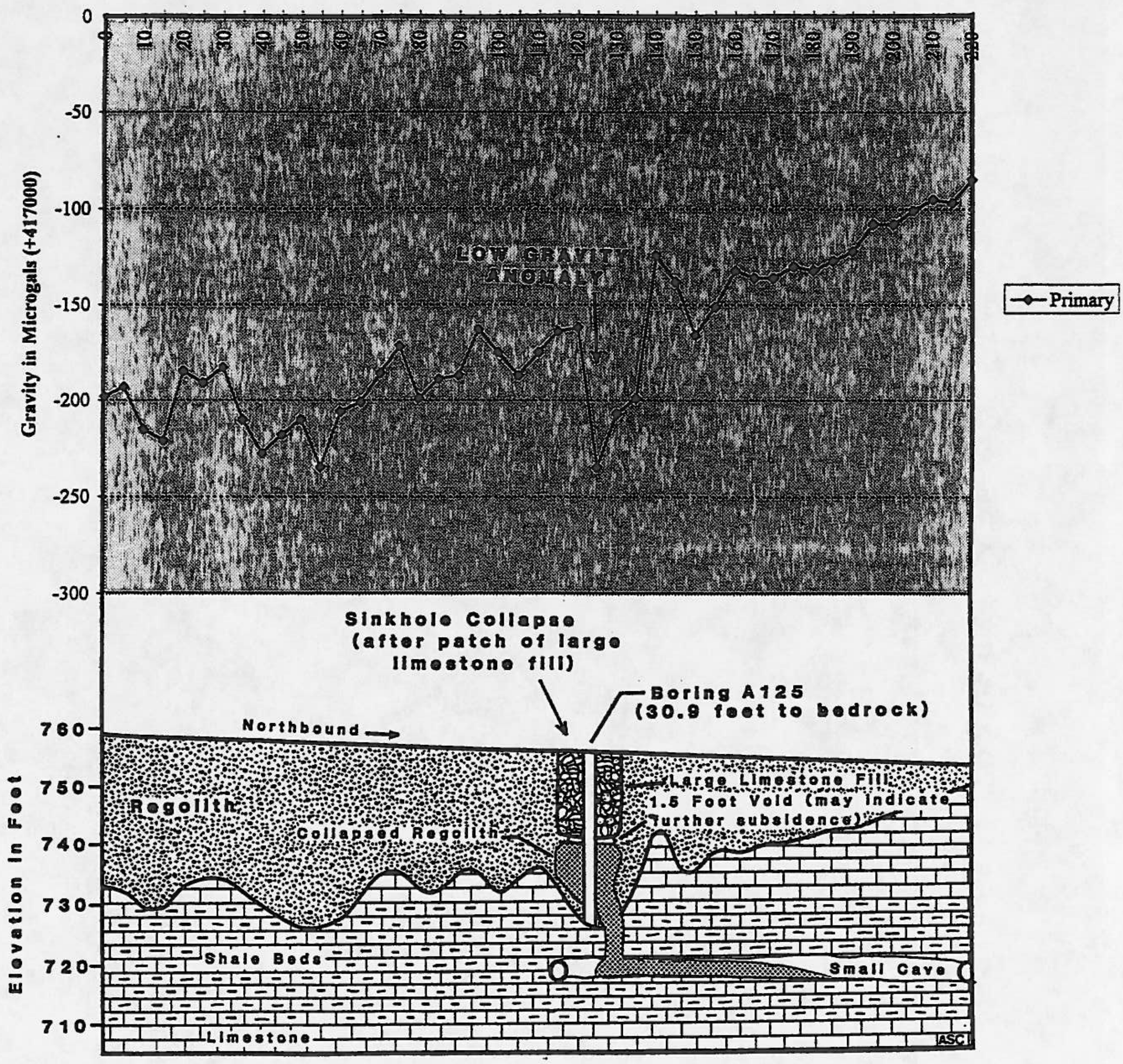


FIGURE 16. Depth to bedrock and other subsurface features estimated from microgravity readings and Boring A125. Stormwater sinking into a sinkhole collapse in the east ditch, located about 60 feet southwest of the site, probably induced the collapse of the regolith arch over the bedrock crevice. The collapse grew upward until it reached the surface.

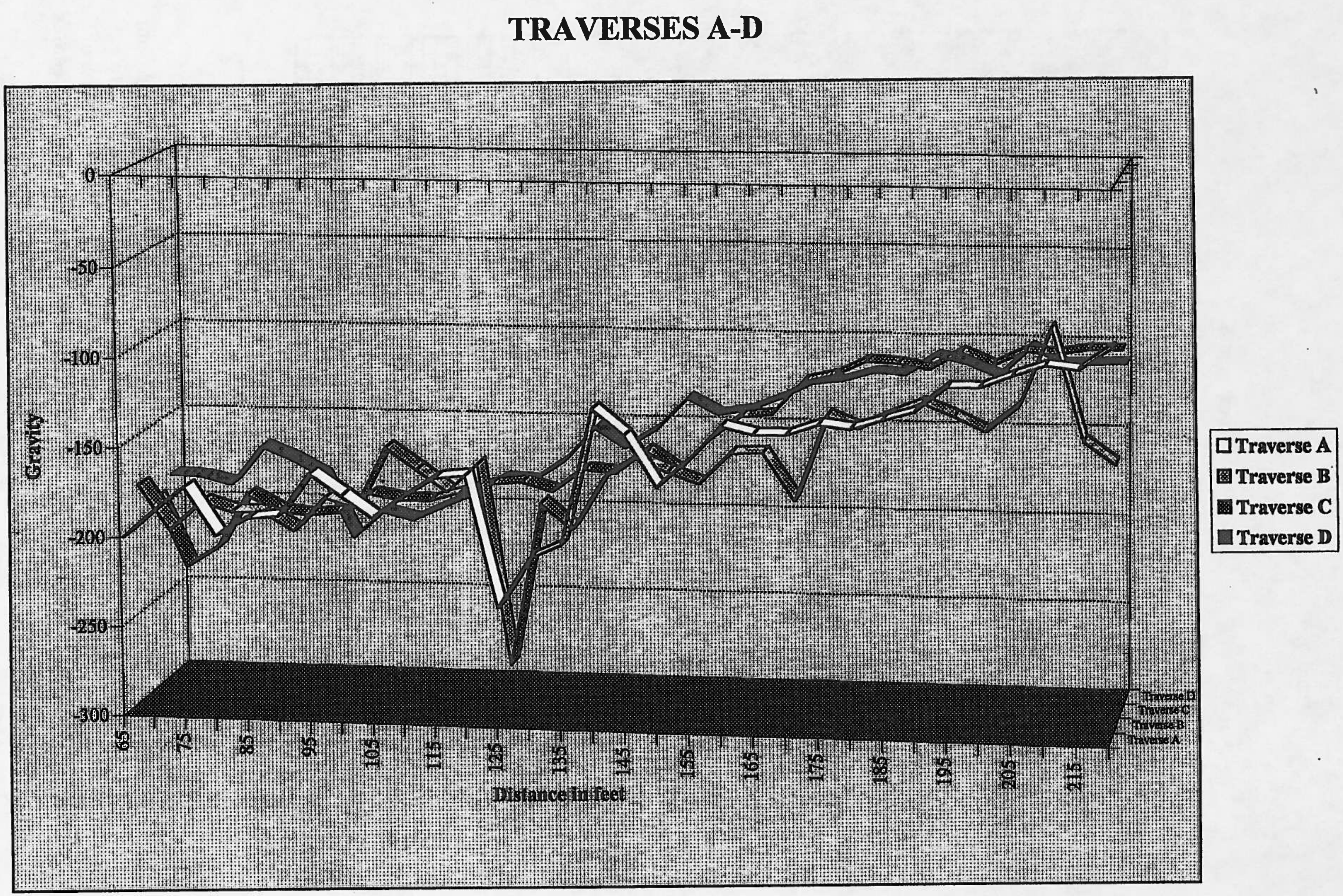


FIGURE 17. Four parallel gravity traverses along the paved shoulder and three northbound lanes of the interstate highway. Low-gravity anomalies were detected along Traverse A (paved shoulder) and B (outside lane) in the vicinity of the collapse. Anomalies were not detected along traverse C or D or on any of the southbound lanes.

GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION METHODS FOR NEW TUNNEL CONSTRUCTION: A CASE STUDY

BY
DANIEL J. HURST, PG.
SENIOR ENGINEERING GEOLOGIST
OGDEN ENVIRONMENTAL AND ENERGY SERVICES CO., NASHVILLE, TN

JAMES WM. MARTIN, P.E.
SENIOR GEOTECHNICAL ENGINEER
OGDEN ENVIRONMENTAL AND ENERGY SERVICES CO., NASHVILLE, TN

BERNARD H. VOOR, III, P.E.
SENIOR GEOTECHNICAL ENGINEER
OGDEN ENVIRONMENTAL AND ENERGY SERVICES CO., NASHVILLE, TN

ABSTRACT

Part of a combined sewer overflow abatement program in Nashville, Tennessee, involves construction of a new tunnel through limestone and soft flood-plain sediments near the Cumberland River. In order to better manage construction risks and reduce bid contingencies, the Owner elected to provide prospective contractors with a contractually binding baseline description of anticipated subsurface conditions, both geotechnical and environmental, and an interpretation of how those conditions could affect construction. Potential claims for additional compensation during construction will be the based on the difference between actual conditions and the baseline interpretation.

This paper describes the methods used to help characterize subsurface conditions and concerns along the project alignment. The investigation included: geotechnical borings; environmental borings; and, a combination thereof. Drilling techniques included: vertical hollow-stem augering; wash boring; and, diamond-bit coring. It also included angled borings and a 500-foot long directionally drilled boring along the tunnel profile. Field tests included standard penetration testing, continuous sampling, in-situ vane-shear testing, pump tests, pressure tests, slug tests, piezometer and well monitoring, and headspace (vapor) testing. Laboratory testing consisted of routine soil and rock mechanics testing. Soil samples were subjected to: standard index tests, triaxial tests, consolidation tests, permeability tests, and chemical analytical tests. Bedrock samples were subjected to: unconfined compressive strength testing, sodium sulphate soundness testing, slake durability testing, chemical analysis, rock hardness (Schmidt, Shore and Total Hardness), and modified Taber Abrasion testing.

INTRODUCTION

Expanded awareness of the problems and liabilities associated with both the geotechnical and environmental conditions that might be encountered during underground construction projects has created a need for very comprehensive site investigations for those types of projects. This paper describes the impact that such issues can have on tunnel construction and gives an overview of the geotechnical and environmental study methods recently used during the site investigation for a 5,450 foot long, 12 foot diameter bored tunnel in Nashville, Tennessee.

RISK AND DISPUTE REDUCTION IN UNDERGROUND CONSTRUCTION

Before describing the investigation methods used for the Lewis Street CSO Tunnel project, it is worthwhile to briefly describe the framework for what is rapidly becoming a routine approach for complex projects where geologic conditions are a major obstacle which must be overcome during construction. The approach discussed here, and which was utilized on the subject project, is also applicable to many construction projects other than tunnels. It is a method for reducing risk to all parties. Such an approach is in direct contrast to what many consultants face in their day-to-day practice, where fierce competition and aggressive pricing requirements often reward those who do minimal site investigations and then make conservative recommendations because of a poor understanding of actual subsurface conditions.

Subsurface conditions cause more construction problems than any other factor (ASFE, 1992). When unexpected conditions are encountered, contractors often resort to litigation for relief from the effects of what they perceive as differing site conditions. Such litigation increasingly intrudes into the practice of both geotechnical and environmental consultants, who must defend such claims either on the owner's behalf or, more likely, their own behalf. Studies indicate that in a majority of cases where contractors filed suit, they convinced the courts that costs related to unexpected site conditions were, in part, the owner's responsibility (Gould, 1995). Owners may then involve the geotechnical or environmental consultant in the litigation, even turning a differing site condition claim by a contractor into a professional negligence or errors and omissions claim against the consultant. All too often, site investigations are tailored to only assist in the design process with little attention given to issues of constructability and how to provide potential contractors with the information they need.

Because of the unique nature of underground construction projects and the large number of projects involving litigation, the underground construction industry has been in the forefront of efforts to improve the usefulness of site investigations and contracting practices. Tunnel design engineers often face an uphill battle in convincing an owner that a geotechnical study that may cost several hundred thousand dollars is worth the expense. However, it is also true that "a thorough job of exploration is always made on every tunnel completed; however, on some projects, it is accomplished by the actual driving of the tunnel when it can no longer be an economic benefit to the Owner" (Bickel and Kuesel, 1982). The Underground Technology Research Council (UTRC) has issued a series of recommendations which were initially aimed specifically at underground construction but now are recognized by many as appropriate whenever subsurface conditions may affect the success of a construction project. Avoiding and Resolving Disputes During Construction, 1991, details the UTRC'S recommendations and urges owners to accept an equitable allocation of risk associated with subsurface conditions. Contractors should not, and will not, absorb all the costs associated with unanticipated subsurface conditions when completion of the project is for the owner's benefit. Therefore, owners should be prepared to pay either the cost of determining conditions prior to construction or the extra claims when those conditions are discovered during construction. Among the UTRC's recommendations are:

1. Disclose fully all subsurface data and interpretations.

2. Eliminate disclaimers regarding the contractors ability to rely on the subsurface data provided.

3. In addition to the geotechnical data, include a Geotechnical Design Summary Report (GDSR).

The UTRC 1991 guidelines contain a description of what a GDSR should contain. It should be specific and brief, not repeating factual information contained in the data report, but concisely describing the design teams conclusions in regard to subsurface conditions.

Although the UTRC's recommendations include contractual and other issues, much emphasis is placed on providing a through subsurface investigation and interpretation. Gould (1995) notes that when the GDSR concept is utilized, the subsurface consultants role is enlarged and enhanced, "they must carry out exploration to inform both designer and builder." The GDSR is part of a multi-phase exploration process in which factual data such as borings, laboratory results, environmental assessments, and other site and project specific information are provided in a subsurface data report, only, without interpretation or speculation. The consultant then evaluates the subsurface data and proposed construction and authors the GDSR, which "sets forth the designer's anticipated subsurface conditions and their impact on design and construction. Thus the engineer and owner establish the baseline for all anticipated conditions,...if conditions are materially different from the baseline, and the contractor can demonstrate a financial impact, he is entitled to additional compensation. Thus, the owner accepts risk for conditions more difficult than the baseline" (UTRC, 1991). In order to emphasize that the document establishes the "baseline" conditions, revised guidelines will recommend changing names from GDSR to "Geotechnical Baseline Report" (GBR) (Austin, 1994).

Site investigations should be designed to obtain data in phases, with existing information utilized to help determine what is already known and what new information is needed. Each subsequent phase utilizes and builds on previously acquired knowledge until the owner and engineer decide that an adequate understanding of subsurface conditions has been obtained. Therefore, it is typically not possible to develop a complete exploration program (or cost for services) at the beginning of a project. Owners that insist on such an approach and are inflexible with regard to changes in study scope and cost should be educated or avoided. Exploration programs completed with no modifications often are poorly conceived, poorly conducted and/or wasteful (Bickel and Kuesel, 1982). Also, Dodds (Bickel and Kuesel, 1982) points out that..."the amount of exploration required to provide the needed answers is in no way related to the funds available for the work. The funds available may control the work accomplished, but they do not delineate the work actually required."

Readers are urged to further educate themselves on the benefits of the UTRC's guidelines, and to view site investigations as a tool for helping designers, owners and contractors avoid disputes regarding subsurface conditions. Innovative methods of obtaining data may be necessary in order to collect the required data, even if the cost of the subsurface study must be increased to utilize unique methods. Owners should be educated as to the benefits of a very thorough subsurface study which, although it may cost thousands of dollars more than a "cursory" study, might avoid much larger costs in extras claims, litigation or conservative designs.

LEWIS STREET PROJECT DESCRIPTION

The Lewis Street Combined Sewer Overflow (CSO) Tunnel is one of several projects in the Nashville Overflow Abatement Program which is being implemented to abate sanitary and combined sewer overflows in Nashville. The Lewis Street CSO Tunnel and an adjacent detention basin will temporarily store a portion of storm event runoff and slowly decant this runoff to a treatment facility prior to discharge. The program will reduce uncontrolled overflows to the river from approximately 70 per year to less than one per year. The entire combined sewer system is maintained by the Metropolitan Nashville-Davidson County Department of Water and Sewerage Services.

The project is located within the southeast quadrant of downtown Nashville within a highly urbanized area consisting of: medium to heavy industry along the northern portion of the alignment; residential and public housing along the middle portion of the alignment and, mixed light industry and residential housing along the southern one-third of the alignment. Figure 1 shows a location map of the site.

The northern 350 foot reach of the tunnel will lie within the flood plain of the Cumberland River. The balance of the alignment is within an upland area with residual clay overlying limestone bedrock. The southern end of the tunnel will lie within the flood plain of Browns Creek (a tributary of the Cumberland River). The project will include three vertical diversion structures to collect and divert stormwater runoff to the Lewis Street CSO Tunnel. The proposed tunnel invert will vary from about 25 to 65 feet below the existing ground surface, with an average depth of about 40 feet. Figures 2 and 3 show a plan view and profile, respectively, of the tunnel.

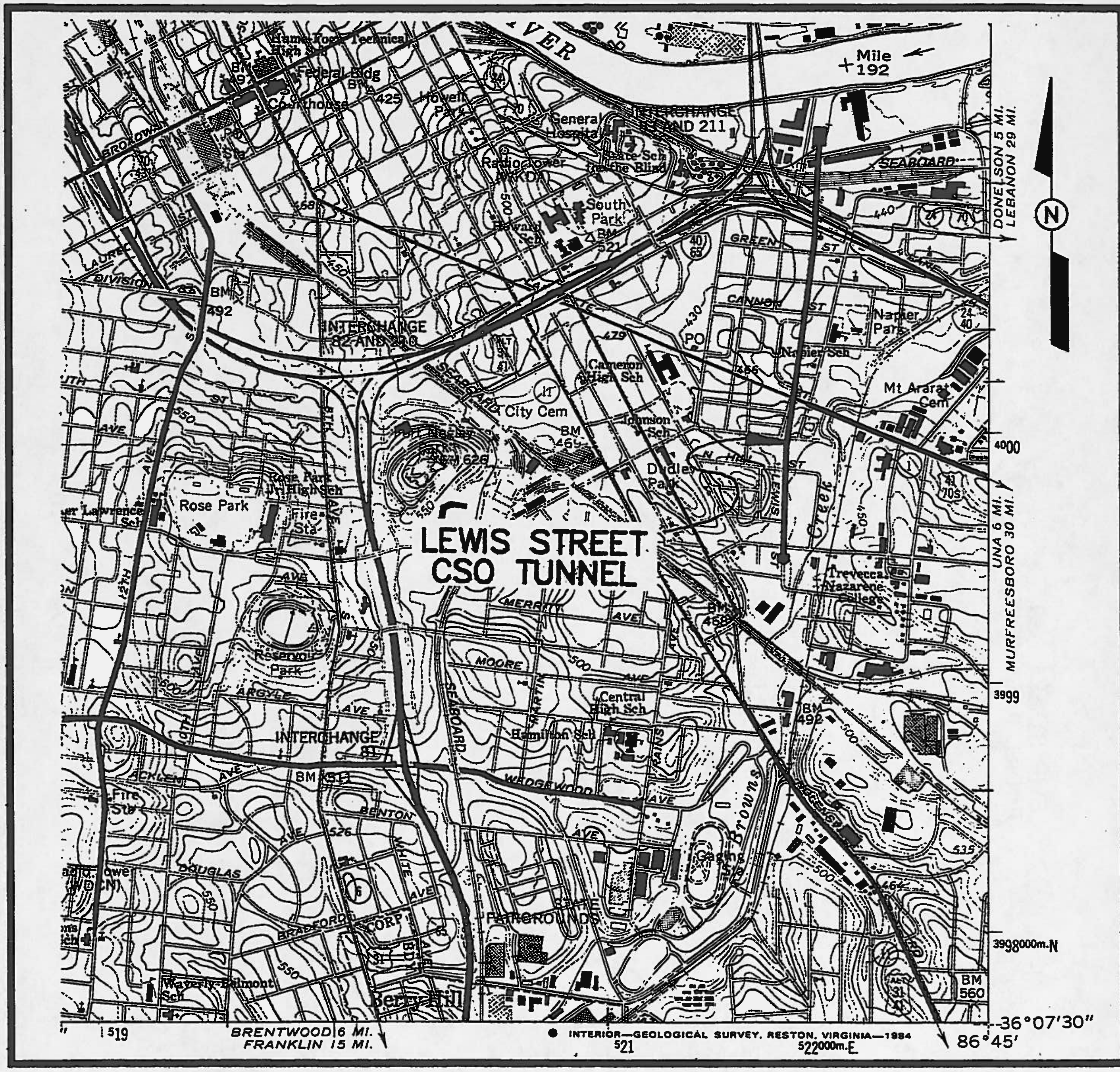


FIGURE 1 – LOCATION MAP

SCALE: 1"=2000'

MAP ADAPTED FROM U.S.G.S. 7 1/2 minute TOPOGRAPHIC QUADRANGLE OF NASHVILLE WEST, DATED 1968, REV. 1983

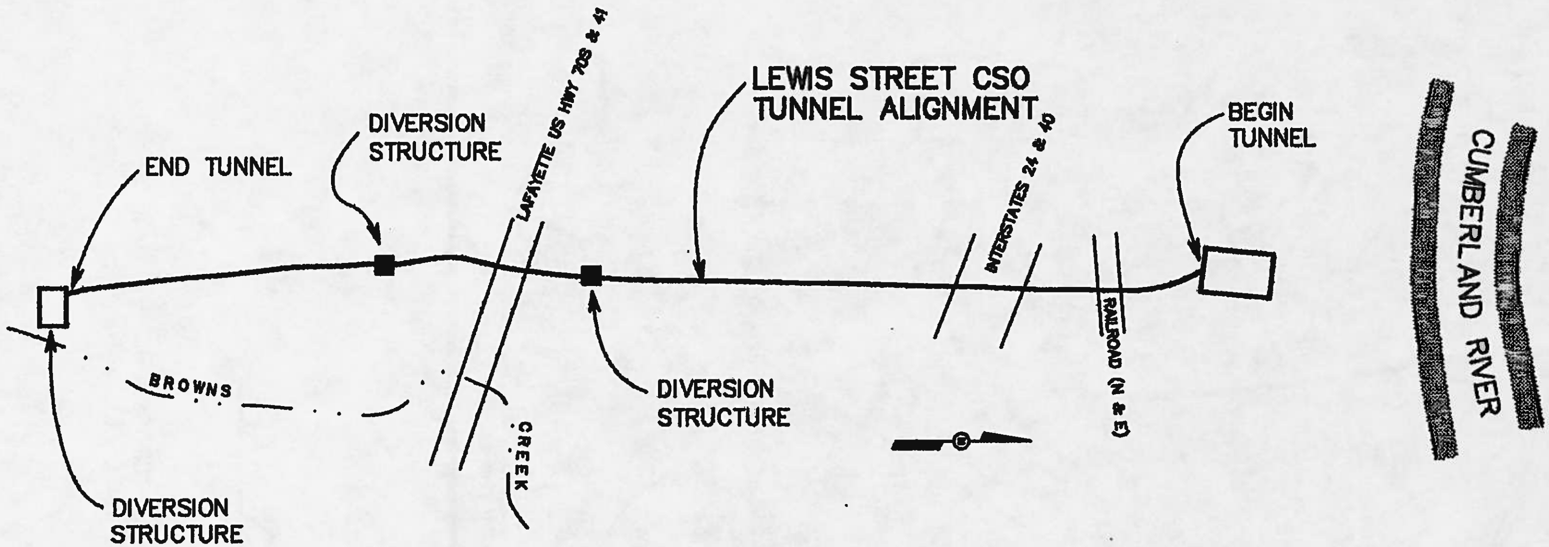


FIGURE 2 – TUNNEL ALIGNMENT PLAN

NOT TO SCALE

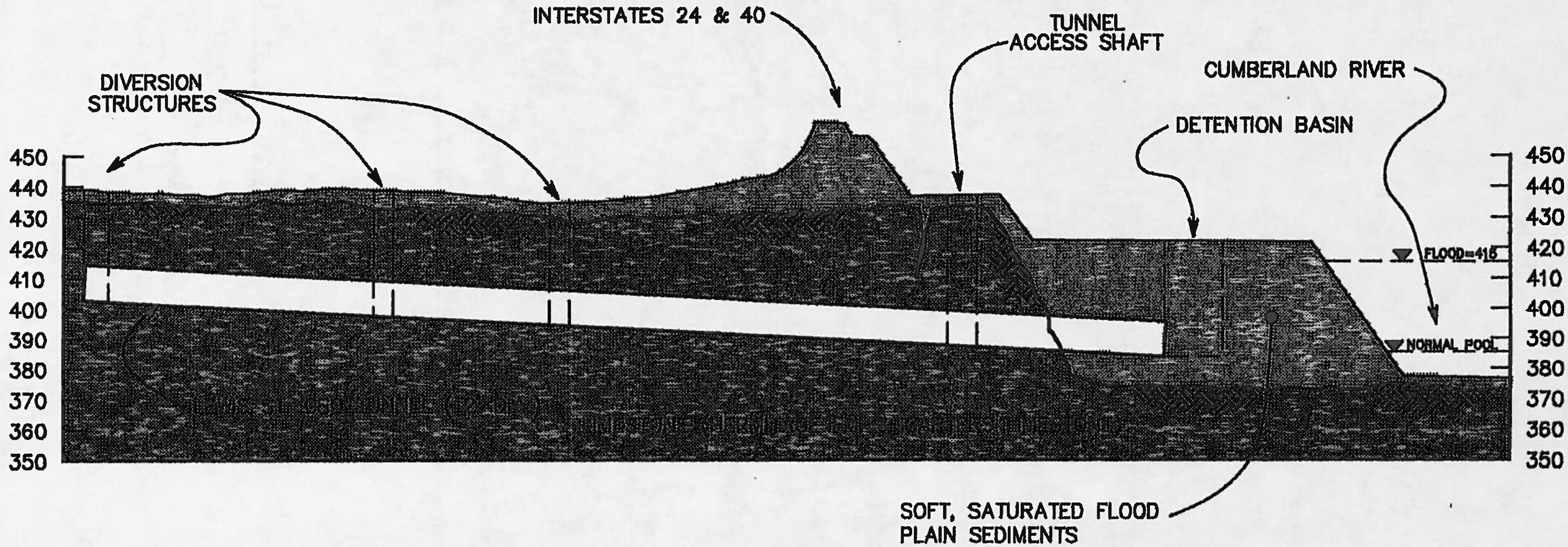


FIGURE 3 – GENERALIZED TUNNEL PROFILE

NOT TO SCALE

LEWIS STREET CSO TUNNEL - SITE INVESTIGATION AND FIELD TESTING

General

The subsurface study for the Lewis Street CSO Tunnel was performed using a multi-phase, multi-task approach in conjunction with the design process. Initial alignment studies were conducted by Consoer Townsend and Associates (CTA), working as program manager for the Metropolitan Nashville-Davidson County Department of Water and Sewerage Services. Prior to finalizing the alignment and grade, CTA recognized that tunneling might be an attractive construction option because of the sewer depth and the heavily developed nature of the area. CTA retained Ogden Environmental and Energy Services Co. (Ogden) to perform a preliminarily subsurface investigation to determine if tunneling was feasible and if geologic conditions might have an affect on the preferred alignment or grade. Based on the initial results, tunneling was considered to be an appropriate method of construction.

The Owner noted major concerns regarding the construction schedule, which was dictated by a State order including penalties if the project was not completed on time. The Owner also expressed concerns regarding the ability to accurately budget for the project so as to avoid the need to seek additional public funds during construction. Therefore, it was decided that the geotechnical study should be as thorough as possible to help identify those geologic conditions which might have an adverse affect on schedule or budget.

In evaluating the needed work scope, it quickly became apparent that environmentally sensitive materials, if encountered unexpectedly, would have a major impact. As a result, part of the work scope included drilling selected borings using an environmental protocol and splitting sampling from some holes between geotechnical and environmental analysis. The use of a single boring to obtain both geotechnical and environmental data can be a cost-effective method of obtaining the needed data. In addition, a Level I Environmental Assessment was conducted to identify environmental concerns associated with properties along the alignment.

After the geotechnical study and environmental assessment were completed, two supplemental studies were conducted to help answer questions generated during the previous study phases. It was decided the project would be best served by following guidelines published by the UTRC, including separating the data in one series of reports, with baseline interpretations provided in a GDSR. The cost of the geotechnical/environmental study and the techniques used to gain an understanding of the subsurface conditions resulted in a benefit to the Owner by lowering construction bids and decreasing the potential for disputes and litigation.

Literature Review

Published geologic literature and maps were initially reviewed to gain an understanding of the regional geology, groundwater regime, structural orientation and stratigraphic units across the site. Geologic mapping of exposed rock outcrops was also performed. That data, along with information derived from published literature was used to assist in planning the exploration program.

In addition to the initial geologic data review, a Level I Environmental Assessment was conducted. The purpose of the environmental assessment was to assess whether or not any sites located along the tunnel alignment or within the study area could potentially have adverse environmental impacts on the proposed tunnel construction project.

The initial phase of the environmental assessment consisted of a visual reconnaissance to identify the locations and addresses of all parcels within the study area. Upon identifying all parcels, historical City Directories were reviewed for each parcel to determine past uses. Concurrent with the City Directories review, historical fire insurance maps and aerial photographs were reviewed to aid in determining historical land uses. A review was also conducted of federal and state environmental regulatory databases to identify sites of known or potential environmental problems. The regulatory records for those sites identified in the databases or which appeared suspect based on the field reconnaissance were then reviewed.

The City Directories review indicted that, of the 183 total parcels within the study area, 85 were used for residential purposes for the past 50 years. Additionally, 53 parcels were found to have been commercial/industrial at some point within the past 50 years and 45 parcels were currently vacant. The parcels which were listed as being historically residential were verified as being residential during the field reconnaissance as were those parcels identified as vacant. No additional investigation or assessment of these parcels was conducted.

Several sites along the alignment were considered to be of potential environmental concern based on the assessment. One site was occupied by the Tennessee Central Railyard Repair Shop and Round House from at least 1914 to 1963. The proposed tunnel passes beneath this site and an access shaft is within the old railroad yard. One diversion structure is located very near a former city dump which was closed in the 1960's. A manufacturer of insecticides was previously located adjacent to another proposed diversion structure. Additionally, a former oil distributor site is located east of this property. The oil distributorship suffered a large spill in March 1982. Also the records indicated that several former service stations in close proximity to the alignment had experienced unremediated spills or leaks of fuel products.

Several additional sites of potential environmental concern were identified within the study area and within approximately 0.25 mile of the proposed tunnel. However, based upon the records search and field reconnaissance, these sites were not as significant as those previously discussed. As such, the Owner elected to conduct no further study of these sites because of the comparatively low risk.

Geotechnical Drilling and Sampling

The preliminary study included 29 exploratory borings along the approximate sewer alignment. The geotechnical study included an additional 63 borings. Additional borings were drilled for a possible alternate alignment and where data gaps existed after the initial studies. A total of 102 borings were made for this project. A detailed boring log for each hole drilled was provided in the data report.

The majority of the borings were drive-sampled in general accordance with ASTM D 1586 (*Penetration Test and Split-Barrel Sampling of Soils*). At selected locations in some borings, relatively undisturbed samples were obtained using thin-walled (Shelby) tubes, in general conformance with ASTM D 1587 (*Thin-Wall Tube Sampling of Soil*). Those borings not sampled were power augered to refusal to provide information about general soil type and thickness.

Over half of the borings were advanced into bedrock by means of diamond core drilling in general conformance with ASTM D 2113. The overall bedrock coring program extended below the elevation of the proposed tunnel. In general, borings were located at about 200 feet on center along the entire alignment. Additional borings were completed wherever the initial exploration indicated unusual conditions such as severe bedrock weathering, environmental concerns, high groundwater or significant variations in depth to bedrock.

The initial exploration indicated most of the proposed tunnel alignment would encounter Ordivician age limestone at a depth of less than 10 feet below the ground surface. However, the depth to bedrock increased to over 40 feet within the Cumberland River flood plain such that the northernmost portion of the tunnel would be founded in soft soil below the water table. Within the saturated flood plain sediments, selected borings were drilled using wash-boring techniques. In order to accurately locate the transition from soil to bedrock at proposed tunnel elevation, a series of tightly spaced borings were located in the area of the expected transition. In addition to vertical borings, five angled borings were cored in this area to help define the reach over which both soil and rock would be exposed in the tunnel face and the degree of weathering at the soil/bedrock interface.

In one area, an 8-lane interstate highway and an adjacent, high-volume, secondary highway precluded surface access for over 500 linear feet of the proposed alignment. Because of concern over the potential for encountering caves or significant karst-related weathering beneath the roadways, horizontal coring was performed in this area. The cost of utilizing this technique was believed to be worthwhile, considering the risk of not defining conditions and encountering a major anomaly. Initially, a 6-inch diameter rock bit at 19° off horizonal was used to produce a starter hole into bedrock. Then a 3-inch diameter drill bit was used to drill the curved portion until the hole was horizontal, after which a diamond core bit was used to retrieve over 300 linear feet of 1.755 inch diameter core samples. Almost 200 linear feet of drilling was required to curve from 19° at 10 feet below ground surface to horizontal in the tunnel zone at about 45 feet below the surface. The curved portion was controlled by a hydraulically driven, down-hole motor located adjacent to the drill bit. The motor housing applies force to one side of the drill wall to provide change of direction for the drill hole. Direction was determined with a down-hole survey using a single-shot camera to record the magnetic declination (bearing) and vertical angle as measured at the drill bit by an internal compass. The down-hole, internal compass method of survey control is less expensive than an alternative, above-ground tracking system, and is fairly accurate as along as no magnetic fields interfere. Completion of 300 linear feet of continuous coring with excellent core recovery, provided valuable data which could not have otherwise been obtained.

The bedrock cores were logged for lithology and physical weaknesses by a registered professional geologist. Rock Quality Design (RQD) values were assigned to each core run in general conformance with RQD After Twenty Years, (Deere, 1989). Logs of the cored holes were shown on Profiles and Logs provided in the geotechnical data report. All of the bedrock cores were photographed prior to extracting samples for laboratory testing; those photographic records were also provided in the geotechnical report and all core samples were made available to contractors during a pre-bid meeting.

In addition to Standard Penetration Testing (SPT), geotechnical field testing included in-situ vane shear testing. A vane borer instrument was used at three locations in order to evaluate the in-situ undrained shear strength of the soft, saturated clay within the flood plain portion of the alignment. The vane borer instrument was lowered through hollow stem augers and advanced at least one foot past the end of the augers. The vanes were torqued using a calibrated vane bore gearbox until either shear failure of the soil occurred, or the maximum capacity of the equipment was reached. After failure occurred, the vanes were again torqued with the vane gearbox to measure the residual shear strength of the in-situ soil. Because of the soft, saturated nature of the soils, the vane shear testing was thought to provided a more accurate shear strength value than conventional laboratory testing of undisturbed samples.

Environmental Drilling and Sampling

In selected borings, monitoring wells were installed after completion of soil sampling operations in order to obtain water samples for environmental analysis. The monitoring wells were constructed of 2-inch diameter plastic pipe with 10 feet of screen. In some borings, a rubber Tri-seal™ was placed 2 feet above the top of the screen and the remainder of the borehole was sealed with a cement-bentonite grout. No sand pack was placed in these wells because they were installed in bedrock core holes. All downhole tools used in the drilling of the monitoring well boreholes were decontaminated prior to use. Each well was developed using an air purging technique until the water was visually clear. Well sampling was conducted using cleaned Teflon™ bailers for both purging and sampling. Each well was purged of three well volumes prior to sampling. Sample bottles were labeled then placed on ice for transport.

Each of the 14 exploratory borings in which environmental soil sampling was conducted were sampled continuously in the unsaturated overburden using split spoon samplers in general accordance with ASTM D 1586. All downhole equipment and sampling tools were decontaminated prior to use and, in the case of split spoon samplers, between each sample. Sampler decontamination consisted of a wash with a detergent/potable water mix, a potable water rinse, a deionized water rinse, an isopropyl alcohol rinse, and air drying. Auger decontamination consisted of steam cleaning using a water/detergent mix, rinsing with potable water, and air drying. All site personnel who were involved in handling samples or sampling equipment donned new disposable latex gloves between each sample.

Upon removal of the sample from the split spoon sampler, approximately equal portions were placed in a quart mason jar and a laboratory-cleaned glass sample jar. The quart mason jar was then placed in sunlight where the sample was allowed to volatilize. An organic vapor meter (OVM) was then used to measure the level of organic vapor inside the jar. This method of headspace screening provides a relative indication of the degree of possible organic contamination of the sample. In this case, the sample from each hole which exhibited the highest OVM reading was submitted to the laboratory for quantitative analysis. If all samples from a particular hole exhibited the same OVM readings, or if evidence of possible contamination was present, the sample was selected from the horizon most likely to have been impacted.

The presence of toxic or explosive gases can cause major disruptions to tunneling and dramatically increase construction costs if encountered unexpectedly. An example is the Baltimore Metro Northeast Extension tunneling project, completed in 1993. The unexpected discovery of gasoline and dry-cleaning fluids during soft ground tunneling contributed to major project delays, and an extras claim of $36 million was paid by the Owner on a project that was originally bid at $70 million (Edwards and Merrill, 1995). Although naturally occurring gases such as methane and hydrogen sulfide are not common in Middle Tennessee, the environmental portion of the study raised concerns regarding man-made gases from former garbage dumps and petroleum leaks within the vicinity of the project. Therefore, in addition to chemical analysis of soil and groundwater, the headspace within selected borings and monitoring wells were periodically tested with down-hole monitors designed to detect hydrogen sulfide, methane or other organic gases.

Groundwater

Water flow into tunnels during construction can be a major problem, but there is no method of accurately determining the flow quantity ahead of construction. However, various methods can be used to estimate the possible magnitude of water inflow. In-situ methods used during this study included well pumping tests within the flood plain sediments and within bedrock, pressure testing (using two different techniques), slug testing and piezometer surveys.

Simple standpipe piezometers and monitoring wells were installed in several borings to provide information regarding ground water levels, so that gradients and flow directions could be evaluated. Simple piezometers were installed in twelve borings. Monitoring wells were installed in another twelve borings. Water levels in all available piezometers and monitoring wells were measured periodically over a nine month period to develop data on the potentiometric conditions during both dry and wet periods.

Pressure (Packer) tests were conducted in four borings in general conformance to the procedures outlined in ASTM D 4630. The tests were performed using a constant-head method to assess the hydraulic conductivity of the tested interval. Each pressure test was conducted in the open bore hole using a double packer arrangement. Pressure transducers were attached to allow measurement of pressure above, between, and below the two packers. After inflation of the packers, potable water was pumped into the interval between the packers and the water pressure and flow rate were monitored. A constant water pressure was maintained throughout the 15-20 minutes of the test.

In addition to the constant pressure, double packer testing described above, three borings were tested using a single packer, step-pressure test. That testing was conducted following the protocol for the modified Lugeon water-test (Houlsby, 1976). The Lugeon water test was developed to assess the need for foundation grouting at dam sites, but is also useful in determining the ability of fractures to transmit fluids. After sealing the bore hole with a single packer, water was pumped in at five different pressures, with rising then falling pressures. Each step was held for 10 minutes, volumes were recorded and a Lugeon value was determined. Inspection of the five Lugeon valves and comparison to published literature allowed an interpretation of the flow regime as either: 1) laminar flow, due to small fractures; 2) turbulent flow due to large fractures; 3) dilation due to temporary opening of fissures at high pressure; 4) wash-out due to permanent removal of fracture filling materials; or 5) void-filling of empty voids.

Four slug tests were conducted, two rising head and two falling head tests. The tests involved first inserting, then withdrawing, a stainless-steel slug into the water in the piezometer, thereby displacing the water and causing a near-instantaneous rise or fall in the water level. The recovery of the water to its static level was measured using a pressure transducer connected to an automatic data logger. The data collected were then analyzed to determine the hydraulic conductivity of the tested interval.

Aquifer pumping tests were conducted at two locations to provide data necessary to assess potential construction dewatering of the flood plain sediments. Each pumping well consisted of a 4-inch diameter well with 30 feet of screen. The screen extended over the entire saturated section of the silty sand aquifer and rested on bedrock. A 2-foot-thick bentonite seal and bentonite/cement grout to the surface prevent infiltration of surface water through the borehole. A 1.5-inch diameter observation well, also screened over the entire saturated section, was installed 25 feet from the pumping well. Natural (background) water level fluctuations in the aquifer were evaluated prior to performing the long-term pumping tests. An automatic data logger recorded water levels on an hourly basis over a 65-hour period. Initially, step-draw down pumping tests were conducted. The purpose of the step drawdown test was to determine the optimal pumping rate at which a constant-discharge pumping test should be performed and to permit a calculation of the specific capacity of the pumping well. The optimal pumping rate for a constant-discharge test is the rate which will stress the aquifer without causing the well to go dry. Constant-discharge pumping tests were also conducted. After completion of the step drawdown test, the well was pumped at a constant rate of flow while monitoring drawdown in the pumping well and observation wells. Pressure transducers connected to an automatic data logger were used to monitor drawdown in observation wells. The tests were originally scheduled to run 72 hours, but were stopped whenever water levels in the pumping and observation wells stabilized.

LABORATORY TESTING

Geotechnical Testing

After visually reviewing and classifying the recovered soil samples from within the soft ground portion of the alignment, selected specimens were subjected to routine geotechnical laboratory testing. Soil testing included unit weight determinations; specific gravity; Atterberg limits; grain-size; natural moisture content; unconfined compressive strength; undrained shear strength; one-dimensional consolidation; and, vertical and horizontal permeability.

In terms of constructability, rock hardness and chemical composition are critical in evaluating probable tunneling rates. Selected intervals of bedrock core from each rock type likely to be encountered during tunneling were tested for unconfined compressive strength, unit weight determinations, and specific gravity. In addition, chemical analyses were performed on selected core samples to determine percent silica, calcium carbonate and magnesium carbonate. Bulk samples of the core were processed by crushing in order to perform sodium sulfate soundness tests according to ASTM D-4644. Selected rock cores were also subjected to Brazilian tensile testing. Brazilian splitting tensile strength tests were performed according to ASTM D 3967. The specimens were prepared from full core diameters by sawing perpendicular to the core axis and grinding to meet ASTM requirements.

In addition to the routine rock testing discussed above, other testing was conducted to offer contractors data they needed to evaluate the tunneling characteristics of the rock. Those tests included Slake Durability, Elastic Moduli, Rebound Hardness, Scleroscope Hardness, and Abrasive Hardness testing.

Elastic moduli were obtained from one inch diameter cylindrical specimens cored from the full-size cores obtained in the field. The sample preparation was performed to provide specimens meeting the requirements of ASTM D-4543, and the specimens were tested according to ASTM D-3148. The elastic parameters were derived from linear regression of the stress-strain curves between axial stresses of 500 and 5000 pounds per square inch.

Rebound hardness values were obtained using a recently calibrated Type L Schmidt hammer. There is not yet an ASTM standard for rebound hardness of rock. Core samples were impacted 10 times, the 5 lowest readings discarded, and the 5 highest readings averaged to obtain the reported value.

Scleroscope hardness values were obtained using a recently calibrated Type D Shore scleroscope. There is no ASTM standard for scleroscope hardness of rock. Core samples were impacted 20 times, the 10 lowest ratings discarded, and the 10 highest readings averaged to obtain the reported value.

Abrasive hardness values were obtained using a modified Taber Model 503 abraser with a 250 gram weight. There is no ASTM standard for abrasive hardness. Core samples were weighed, abraded for 400 revolutions, turned over, abraded for another 400 revolutions, weighed, and the abrasive hardness was reported as the reciprocal of the average weight loss in grams.

Total hardness values were calculated using the results of the rebound hardness and abrasive hardness tests according to the methods developed by Tarkoy (1985). Typical total hardness values may vary from about 200 for the hardest of rocks to less than 5 for the softest of rocks. Total hardness results indicate that the seven tested samples are of relatively low total hardness, which implies relatively high penetration rates and relatively low cutter wear.

Table 1 summarizes the geotechnical laboratory testing conducted for this project.

TABLE 1
SUMMARY OF GEOTECHNICAL LABORATORY TESTING
LEWIS STREET CSO TUNNEL PROJECT

Test Type	Number of Tests Completed		
	Flood Plain Soils	Upland Soils	Bedrock
Natural Moisture(ASTM D 2216)	42	48	NA
Wet Unit Weight	14	6	0
Dry Unit Weight	8	6	18
Specific Gravity (ASTM D 854/C 127)	7	1	11
Atterberg Limits (ASTM D 4318)	21	19	NA
Sieve/Grain Size (ASTM D 422/D 1140)	22	16	NA
Direct Shear (ASTM D 3080)	1	0	0
Triaxial (Consolidated, Undrained)	3	1	0
Triaxial (Consolidated, Undrained) (ASTM D 2850)	1	0	0
Unconfined Compressive Strength (ASTM D 2166)	3	2	18
Splitting Tensile Strength (Brazilian) (ASTM D 3967)	NA	NA	3
Hardness (Shore, Schmidt and Total) ASTM C 886, ASTM C 805 and ASTM Committee D 18.21.01	NA	NA	7
Sodium Sulfate Soundness (ASTM C 88)	NA	NA	4
Slake Durability (ASTM D 4644)	NA	NA	3
Modulus of Elasticity	NA	NA	7
Chemical Composition	NA	NA	15
Modified Taber Abrasion	NA	NA	7

Analytical Testing

During drilling, soil samples from randomly selected boreholes were initially screened using an Organic Vapor Meter (OVM). Afterwards, the sample with the highest OVM reading in each sampled boring was analyzed for target compound list/target analyte list (TCL/TAL) parameters by Specialized Assays of Nashville, Tennessee. In addition, at certain borings near sites of previous petroleum contamination, testing for various petroleum-based products was conducted. Other samples were tested for metals, particularly lead, because of lead contamination encountered at another construction site less than 1/2 mile away. Also, ground water samples from twelve monitoring wells were submitted for laboratory analysis. In addition to that testing, ground water samples from randomly selected monitoring wells were tested for pH and corrosivity parameters.

Table 2 below summarizes the analytical testing conducted for this project.

TABLE 2
SUMMARY OF ANALYTICAL TESTING
LEWIS STREET CSO TUNNEL PROJECT

Test Type	Number of Tests Completed	
	Soil	Groundwater
BTEX	2	3
TRPH	0	3
TCL/TAL	15	9
DRO/GRO	1	1
PAHs	1	1
Total Cadmium and Lead	25	4
pH, Sulfide, Corrosivity	0	5

BTEX - Benzone, Toluene, Ethylbenzene and Xylene
TRPH - Total Recoverable Petroleum Hydrocarbons
TCL/TAL - Target Compound List/Target Analyte List
DRO - Diesel Range Organics
GRO - Gasoline Range Organics
PAH - Polynuclear Aromatic Hydrocarbons

INFLUENCE OF SUBSURFACE INVESTIGATION ON CONTRACT DOCUMENTS

The GDSR established baseline conditions--known conditions. It also discussed probable but undetected conditions--known unknowns--and remotely possible conditions--unknown unknowns. Based on that information, the contract documents were tailored to produce the lowest base bid with other likely conditions covered by budget allowances. The following items were included in the contract documents as a result of the GDSR:

- Bidders were told to expect groundwater inflow totaling up to 1500 gallons per minute. Inflows above that amount will be subject to additional compensation and a budget allowance of $500,000 was included in all bids to cover this potential condition.

- Bidders were to expect to encounter a specific quantity of special and hazardous wastes from surface excavations. A Health and Safety Plan to be developed by the contractor, including OSHA 40 hour hazardous waste operations training and air quality monitoring, were required as part of the base bid. During construction, analytical testing and monitoring of water and spoil is required in the event that additional wastes are discovered. All bidders had to include a $800,000 budget allowance in the event that additional material beyond the established quantity is encountered.

- The contract documents (which include the GDSR as a binding interpretation) discuss the likelihood that during construction the tunnel will be declared "potentially gassy," which is an OSHA definition requiring that certain equipment and procedures be followed. Bidders were instructed to provide equipment and training for working in potentially gassy conditions as part of their base bid. An allowance of $500,000 was also included in all bids, for stand-by time, if needed due to evacuation of the tunnel when unacceptable levels of gases and vapors are present.

- Approximately 1,800 linear feet of the tunnel has poor rock conditions. Bidders were informed at these conditions and required to furnish primary tunnel support as part of the base bid. Unit prices were required for an additional 500 linear feet of primary support and for 4,800 linear feet of rock bolts.

CONSTRUCTION

Construction of the Lewis St. CSO Tunnel began in March of 1995 with over 1,500 linear feet of tunnel completed at the time of this writing. Several bidders, including the successful bidder, W. L. Hailey & Company, Inc., reported that the data reports and GDSR allowed them to remove contingencies they would have otherwise been forced to include, resulting in lower bids. The winning bid for the project totaled just over $13 million, which was below the engineer's construction estimate. Even bidders who normally do not include contingencies in their proposal felt more comfortable bidding this project as compared to projects where only modest subsurface data is available.

To date, the contractor has encountered several areas of poor rock quality and several significant water inflows which affected progress. However, the GDSR accurately described these conditions and even the locations of most of the problem areas encountered thus far. Less than $50,000 from the various contract allowances has been used in order to install extra pumps, holding tanks and diversion systems to decrease response time in the event that excess water flow and/or containments are encountered. This precaution was taken so that the overall project schedule would be less affected given the stiff penalties possible for late completion.

Another benefit to the level of data provided with the contract documents became apparent when the contractor proposed to tunnel the soft ground portion of the alignment rather than perform a braced, cut and cover excavation as designed. After reviewing the geotechnical data, the contractor offered a price savings of over $50,000 if allowed to tunnel this reach and agreed to assume all risks associated with the soft ground tunneling operation.

CONCLUSIONS

Subsurface exploration programs which focus only on design concerns may allow contractors to bid low and then rely on a changed-condition claim to make up for the low bid. Such an approach not only promotes confrontation between the owner and contractor, but also increases the risk of litigation against the geotechnical/environmental consultant. On projects where knowledge of subsurface conditions is critical in determining constructability and rate of progress, both a basic subsurface data report and a GDSR are prudent. Typically, it is not difficult ground conditions that produce construction claims, but rather the failure to properly define and account for the impact that such conditions will have during construction that causes disputes.

The following points are considered by the authors to be important factors in a successful subsurface exploration for tunnel projects.

- Before the exploration begins, conduct a thorough review of existing geologic literature, including aerial photographs and environmental records to identify potential major geologic anomalies and sources of pollution.

- Use field reconnaissance to confirm the literature review and to develop a sense of the project setting particularly when the site may have a history of potential environmental issues.

- Perform a preliminary exploration of the overall alignment in order to evaluate where detailed exploration is warranted.

- Include environmental analysis of soil and groundwater samples. Use headspace monitoring if gassy conditions are possible. Combine geotechnical and environmental sampling if possible to reduce exploration costs.

- Where possible, provide some redundancy in exploration including laboratory tests, in-situ tests and geophysical tests. Perform additional exploration if there are inconsistencies in the results.

- Have an experienced geologist in the field during exploration to log samples and adjust the program as needed.

- Describe each soil sample and each bedrock weakness on detailed boring logs and provide color photographs of all core. Also, make all samples and core available for viewing by prospective contractors.

- Perform complete groundwater studies, including background readings and seasonal variations. Include at least a modest pumping test as well as pressure and slug testing where ground water inflow or dewatering are expected during construction.

- Conduct sufficient laboratory testing of each different type of soil and rock so that the properties are clearly known. Include data that may be useful to the Contractor in order to reduce or eliminate bid contingencies.

- The average value of a series of laboratory testing does not indicate the extreme conditions that might be encountered. Identify maximum and minimum properties that are anticipated.

REFERENCES

ASCE (1974)
Subsurface Exploration for Underground Excavation and Heavy Construction, ASCE, New York, NY, 440 pp.

ASFE (1992)
"Important Information about Your Geotechnical Report"

AUSTIN, T., (1994)
"Geotechnical Baseline Report." Civil Engineering, ASCE, Vol. 64, No. 12, pp. 56-57.

BICKEL, J.O., AND KUESEL, T.R. (Ed.) (1982)
Tunnel Engineering Handbook. Krieger Publishing Company, Malabar, Florida, 670 pp.

DENNING, J. (1993)
"More Than an Underground Success." Civil Engineering, ASCE, Vol. 63, No. 12, pp. 42-45.

DEERE, D.U. and DEERE, D.W. (1989)
Rock Quality Designation (RQD) After Twenty Years, Contract Report GL-89-1- for U.S. Army Corps of Engineers, 67 pp.

EDWARDS, C.A. AND MERRILL, K.D. (1995)
"Baltimore Metro Northeast Extension," Proceedings - 1995 Rapid Excavation and Tunneling Conference, SME, Littleton, Colorado, pp. 93-110.

GOULD, J.P., (1995)
"Geotechnology in Dispute Resolution." Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 7, pp. 523-534.

HOULSBY, A.C. (1976)
"Routine Interpretation of the Lugeon Water-Test", Quarterly Journal of Engineering Geology, Vol. 9, pp. 303-313.

OGDEN ENVIRONMENTAL AND ENERGY SERVICES COMPANY, Nashville, Tennessee, Report of Geotechnical Investigation for Proposed Lewis Street Combined Sewer Overflow Project, Volumes 1 and 2, March 8, 1994.

OGDEN ENVIRONMENTAL AND ENERGY SERVICES COMPANY, Nashville, Tennessee, Supplement #1 - Report of Geotechnical Investigation for Possible Alternate Alignment, Proposed Lewis Street CSO Project, March 9, 1994.

OGDEN ENVIRONMENTAL AND ENERGY SERVICES COMPANY, Nashville, Tennessee, Supplement #2 to Report of Geotechnical Investigation for Lewis Street CSO Project, March 5, 1994.

OGDEN ENVIRONMENTAL AND ENERGY SERVICES COMPANY, Nashville, Tennessee, Level 1 Environmental Assessment for Proposed Lewis Street CSO Tunnel Project, Metro Project No. 90-SC-168B, February 25, 1994.

OGDEN ENVIRONMENTAL AND ENERGY SERVICES COMPANY, Nashville, Tennessee, Geotechnical Design Summary Report, Lewis Street CSO Project, June 3, 1994.

TARKOY, P.J. (1985)
"Proposed Standard for Sampling and Determining Total Hardness of Intact Rock Core Specimens" presented to ASTM Committee D-18.12.01, January 1985, Parts 1 through V.

UNDERGROUND TECHNOLOGY RESEARCH COUNCIL (UTRC), (1991)
Avoiding and Resolving Disputes During Construction. ASCE, New York, NY, 82 pp.

ACCELERATED SITE CHARACTERIZATION TECHNIQUES IMPLEMENTED AT U.S. ARMY CORPS OF ENGINEERS CONTAMINATED SITES

James D. Dzubay, M.S.
President
MATRIX Technologies, Inc.
Columbus, Ohio and Osseo, Minnesota

and

Mark S. Meyers, PhD, PE
Geotechnical Engineer
U.S. Army Corps of Engineers
St. Paul District
St. Paul, MN

ABSTRACT

The development of direct push sampling techniques and the use of laboratory grade equipment in mobile laboratories has allowed the St. Paul District, U.S. Army Corps of Engineers, to provide more comprehensive site characterization information at contaminated sites for both in-house clients and for other Federal and State agencies. The more comprehensive information is realized at a reduced cost and in a more rapid timeframe than conventional site characterization techniques, the end result being site closure with less investigative phases and the related site, sampling, and fixed-base analytical laboratory costs.

This paper will discuss the direct push sampling techniques and capabilities, including soil sampling, macro-core soil sampling, groundwater sampling, and soil gas survey sampling; laboratory grade analytical instrumentation techniques and capabilities, including purge and trap sample concentration, gas chromatography, the use of a photoionization detector and flame ionization detector in series, modified methods developed specifically for field screening and approved by regulators, and the various compounds which can be analyzed for in the field.

Two case histories are discussed where these techniques were implemented to minimize cost and time a Corps of Engineers boatyard and dredge service base on the Mississippi River and a site included in the Defense Environmental Restoration Program for Formerly Used Defense Sites (DERP/FUDS). The case histories include a cost comparison to conventional drilling, sampling, and fixed-base laboratory methods which would have been used in the past.

INTRODUCTION

Traditionally, subsurface investigations to define the extent and magnitude of contamination on site characterization projects have been conducted using conventional drilling techniques to sample site soils and to install permanent monitoring points. Samples are then collected and sent to a fixed-base laboratory for analysis. It may then take from two to four weeks to receive the results. Based on the data, some sites may require multiple mobilizations to complete definition of the contaminants. Rising project costs and long investigation time-frames using conventional methods have resulted in increased use of integrated direct push sampling techniques and on-site sample analysis.

DIRECT PUSH SAMPLING TECHNIQUES

Introduction to the Basics of Direct Push Technology

Direct push sampling techniques can be employed by a variety of mechanisms. The most efficient technique utilizes hydraulic probing equipment mounted in the back of a carrier vehicle such as a pick-up, cargo van, or all-terrain vehicle (Figure 1). The vehicles shown in Figure 1 use hydraulic pumps belt-driven by the vehicle engine to provide power to the probing unit. Auxiliary engine type probe units are also in use. Several factors including site terrain, sample collection technique, sampling depths, and field analytical requirements should be considered in selecting the proper carrier vehicle.

The small diameter sampling tools are advanced by the static weight of the carrier vehicle and hydraulic hammer percussion. Although various hammer configurations have been used in the field, the most common in use today applies an impulse force of 600 to 1200 pounds to the top of the probing tool string at a frequency of 30 Hz (Christy 1987). Utilizing these forces the narrow diameter sampling tools can be consistently driven to depths of greater than 50-feet. In some instances depths of greater than 110-feet have been achieved.

Soil Gas Sample Collection

Soil gas samples can be collected utilizing a variety of sampling techniques. The most commonly used technique involves driving the probe rods to the targeted sampling depth, the rods are then pulled back, disengaging the drive point. This provides an open passage for vapor flow up the rods in response to a vacuum being applied at the top of the sampling train. Disadvantages of this sampling method include: potential for sample dilution due to leakage at rod joints; large purge volumes for representative sample collection are required; and decontamination of sampling equipment must be thorough to prevent cross contamination.

A more effective and reproducible sampling system allows the attachment of sample tubing to a holder on the bottom of the sampling train (Figure 2). This tubing is added to the sampling rods after the rods have been driven to the targeted sampling depth and pulled back to remove the drive point. Tubing materials for use in this application include Teflon®, stainless steel, and polyethylene. The sorption characteristics of certain tubing may not permit their application in all circumstances (Christy 1987).

The benefits of this inner tubing system include: low purge volumes (2.5 to 15.1 ml/ft); a continuous, non-leaking conduit which assures that the vapor sample originated at the targeted depth; and elimination of possible contaminant carry-over by not collecting the sample through the probe rods.

Soil Core Sample Collection

Important to any site characterization project is the collection of discrete soil samples for classification, field screening, and lab analysis. Unlike conventional drilling techniques, probing tools do not offer an open borehole into which open soil samplers can be inserted to obtain soil samples at depth. Therefore, special tools have been developed in order to push a closed sampler to depth, open the sampler, and obtain a discrete soil sample (Figure 3).

Operators can take samples at several different depths at the same location by re-entering the borehole each time a new sample is to be collected and driving the sampler to the next sampling interval. Vertical profiling of contamination is routinely accomplished in this manner.

Ground Water Sample Collection

With today's concerns about ground water contamination, it is important to be able to collect discrete, representative ground water samples. Up until recently, the only way to do this was to install expensive, permanent monitoring points. Now, utilizing direct push probing equipment and specially designed sampling tools, discrete ground water samples can be collected in a rapid fashion.

Two commonly employed methods used to sample groundwater are shown in Figure 4. Configuration A shows a mill slotted sampler. This open slotted tool is driven from the ground surface into the water table where an inner tubing is inserted and samples collected. The main disadvantage of this tool is that the open slots for water sampling cannot be closed during probe driving, it is therefore subject to cross contamination by impacted soils in the vadose zone. Configuration B shows a screened point sampler. This tool is driven from the ground surface to the desired sampling depth with the screen protected in the drive sheath. The probe rods are then retracted and the screen pushed out into the aquifer formation and water allowed to collect in the sampler.

Water samples collected at shallow depths (< 20 feet) can be collected using a peristaltic pump. Tubing with a bottom check valve can also be inserted down the bore of the probe rods. This inner tubing is oscillated up and down to produce a momentum pumping action. The pumping action of the inner tubing may be sufficient to lift the sample to the ground surface.

Soil Conductivity Profiling

Environmental professionals are always searching for new ways to be able to distinguish different stratigraphic units (i.e. sand zones from finer grained silt or clay zones) by some method other than direct sampling. Soil conductivity measurements and logs of soil conductivity profiles have long been used as a tool by agricultural scientists to measure soil alkalinity (Rhoades et al, 1976). Unlike borehole geophysical logging tools, the probes in this application have direct contact with the soil.

More recently, techniques such as cone penetration testing and direct push/hydraulic hammer percussion employ advancing small diameter tools into the subsurface for collecting soil resistivity measurements. A typical conductivity probe construction and probe system set-up are shown in Figures 5 and 6.

Utilizing a conductivity probe system can improve your site characterization projects in the following ways:

- Provides detailed characterization of stratigraphic profiles for use in geologic interpretation.
- The ease and rate of data collection translates to more subsurface information for use in the preparation of RI/RAP reports.
- Defines relative degree of permeability and distinguishes units which may promote or inhibit contaminant transport.
- Identifies permeable units for vertical placement of well screens.

ON-SITE MOBILE LABORATORY ANALYSIS

Environmental sample analysis is a major portion of a project budget in today's hazardous waste site clean-up. The objective of environmental sampling is to obtain information about the extent of contamination at the site from representative samples and, thus, provide guidance for appropriate monitoring and remediation options. Reliable representative samples and accurate analytical results play an important role in reducing monitoring and treatment costs. One way to increase sampling quality is to increase the number of analytical samples. However, this will only increase project costs. Therefore, obtaining the most information about a site at the lowest cost within the shortest time frame, while still satisfying regulatory constraints becomes a primary concern for many environmental project managers.

One method being employed to increase sampling quality and reduce overall project costs is to integrate sample collection and analysis on site. Providing on-site analysis of sample matrices can be a valuable tool, allowing the field professional to get the whole "picture" in the field. Recent development of portable and lab grade analytical instrumentation has made on-site testing for environmental samples effective and inexpensive, while producing data that correlates with reference methodologies.

Due to its separation capability and high sensitivity, gas chromatography has become one of the most widely used analytical instrument techniques for on-site analysis of environmental samples. The gas chromatograph can be set up with a wide variety of detectors and columns to perform analysis of a wide range of analytes (Table 1).

TABLE 1

TYPICAL ON-SITE LAB ANALYTES

- Volatile Organic Compounds
- Petroleum Hydrocarbons
- Polynuclear Aromatic Hydrocarbons
- PolyChlorinated Biphenyls
- Chlorinated Pesticides

Practical quantitation limits for on-site gas chromatograph results for the compounds listed in Table 1 vary greatly (1ppb to 1ppm), and depend on the sample matrix, sample preparation technique, and specific analyte properties. An example chromatogram from a ground water sample analyzed on-site is shown in Figure 7.

As with any other sampling program, proper quality assurance/quality control (QA/QC) measures must be implemented. At a minimum, the following should be implemented to maintain proper QA/QC when performing on-site gas chromatography analysis:

♦ Complete at least a three point calibration curve for the target analytes.

♦ Run a check standard at least once a day to confirm the calibration curve.

♦ Run a reagent water blank prior to any sample analyses and after "hot" samples to confirm that the system is free of interferences.

♦ A surrogate standard should be run with every sample to monitor retention time and concentration accuracy.

♦ Duplicate samples should be run about 1 out of every 20-samples to confirm results.

- Matrix spike and matrix spike duplicate samples should be run to confirm precision and accuracy of the analytical system and to identify possible matrix effects.

CASE HISTORIES

The St. Paul District Corps of Engineers has utilized the hydraulic push sampling and on-site laboratory grade analytical equipment on several sites in the past two to three years. The sites ranged from a simple LUST contamination extent investigation at a Defense Environmental Restoration for Formerly Used Defense Sites (DERP/FUDS) Site to a Remedial Investigation at a Corps of Engineers Service Base located on the Upper Mississippi River. Since these two sites represent the ends of the spectrum of Corps of Engineers site characterization studies, they will be discussed further.

DERP LUST Site

Background

Removal of four underground storage tanks (USTs) at the University of Minnesota-Duluth (UMD) Natural Resources Research Institute (NRRI) indicated the tanks had leaked and product was released to the surrounding soils and/or groundwater, requiring a Preliminary Remedial Investigative (PRI) effort. The project site was formerly owned by the Department of Defense (DOD), making it eligible for cleanup under the DERP/FUDS Program. The UMD NRRI is located southwest of Duluth, Minnesota International Airport, immediately adjacent to the Duluth Air National Guard Base. An oil above ground storage (AST) tank farm consisting of three large ASTs exists on the property immediately east of the site.

The site was formerly owned by the DOD and was used as a navigational facility for missile launch sites. In 1955, three 30,000 gallon USTs and one 15,000 gallon UST were installed to store fuel oil for heating and other purposes. In May 1990, s soil boring near the tank basin revealed elevated jar headspace readings, indicating a petroleum release had occurred. The tanks and associated piping were removed in October 1992 and were found to be in good condition.

Site Geology

The site is underlain by approximately 5 to 15 feet of glacial till consisting of red to brown lean sandy clay with various amounts of gravel and cobbles. The till is underlain by a basement complex of igneous Precambrian rocks referred to locally as Duluth Gabbro, which is a heavy, dense, crystalline rock with few to no pore spaces.

Site Hydrogeology

The site is underlain by two hydrostratigraphic units: the Duluth Gabbro and the overlying glacial till. The Duluth Gabbro contains fracture zones which may be closely spaced, but are more frequently miles to tens of miles apart. These fracture zones can yield water in low to moderate quantities. Previous studies in the area have shown that the hydraulic head in the bedrock is similar to that of the overlying glacial drift, indicating that the two hydrostratigraphic units are hydraulically connected. Although the Duluth Gabbro and overlying glacial till are connected hydraulically, vertical flow from the till to the bedrock is low (except in areas where fractures intersect the bedrock surface) due to the lower permeability of the massive Duluth Gabbro.

Three samples were obtained when the monitoring wells were installed at the site. Laboratory permeabilities ranged from 2.7×10^{-8} cm/s to 5.4×10^{-8} cm/s. The principal flow path of ground water at the site is direct recharge from the ground surface to the shallow water table in the glacial till, then horizontal flow to discharge in local streams and drainages. The groundwater flow direction at the site ranges from northeast to east with an average gradient of 0.008 feet/feet. The groundwater flow velocity at the site ranges from 1.5×10^{-6} to 3.2×10^{-6} feet/day.

Sampling and Analysis Program

In November 1993, a 3-day sampling program was conducted at the site utilizing the hydraulic push sampling techniques discussed above. Fourteen probes were advanced to depths ranging from 2 to 12 feet below the ground surface (bgs). Two feet soil cores were collected, analyzed with a PID, and logged. Nineteen soils samples were analyzed for total petroleum hydrocarbons (TPH) as fuel oil, for petroleum volatile organic compounds (PVOCs), and for TPH as gasoline range organics (GRO) using the Wisconsin Department of Natural Resources (WDNR) modified GRO Method. In addition, 17 probes were advanced to depths ranging from 5 to 14 feet bgs for the collection of groundwater samples. The 17 groundwater samples were analyzed for PVOC/GRO.

Soil and groundwater samples were concentrated with an OI-Analytical Model 4560 purge and trap sample concentrator. The purge and trap sample concentrator is directly connected to a Hewlett Packard 5890 Series II Gas Chromatograph (GC). The soil samples were analyzed by PID and FID detectors in series. QA/QC consisted of: running a system blank prior to any sample analysis; running a prepared standard to verify the calibration curve; running an internal surrogate standard (4-bromofluorobenzene) with each purge and trap sample to confirm retention time accuracy and concentration efficiency; and running a matrix spike and a matrix spike duplicate to confirm the precision and accuracy of the analytical system.

Results of the Site Investigation

It was determined that the majority of the soil contamination was limited to the tank basin and was removed when the USTs were removed, with the exception of some soil contamination to the east and downslope of the tank basin. The latter soil contamination was attributed to the percolation of overflows of fuel which occurred during periodic filling of the tanks.

Groundwater contamination was limited to the area between the site and the tank farm. Since the tank farm ASTs also contain fuel oil, the source of the groundwater contamination was difficult to determine. Four monitoring wells were installed at the site. The wells were sampled and tested quarterly for a one year period and Groundwater Monitoring Report was prepared.

It was determined that no significant amount of groundwater contamination occurred due to the release of fuel oil at the site. Groundwater contamination in the easterly most well was attributed to the adjacent tank farm. The site was closed by the Minnesota Pollution Control Agency in 1995.

Costs

The total cost for the hydraulic push sampling and on-site analytical testing program amounted to $7,000. A comparative cost for a standard drilling rig and fixed-based analytical testing is difficult to determine, but a standard rig for three days with two drillers would have cost approximately $6,000 and the analytical testing would have cost approximately $7,400 for a total cost of $13,400. It should be noted that the drilling and analytical costs are minimum costs because a standard grid pattern would have had to be set up if a standard rig were used, resulting in more drilling time and more samples to be tested for each contaminant of interest.

Benefits of Hydraulic Push Sampling and On-Site Analytical Testing

The benefits derived from the sampling and testing program included reduced time in the field, reduced costs, and the determination of an efficient network of four monitoring wells (MWs) on the site. The MW locations were based on the gradient at the site, which was determined from the groundwater elevations obtained during the sampling process. Approximate soil and groundwater contaminant levels and aerial extents were determined from the on-site analytical testing. This information was also used to assist in locating the MWs. This information was obtained at a much lower cost than using a standard drilling rig and fixed-based laboratory testing. The on-site analytical, although not able to be used to close a site, proved to be an excellent tool, providing information real-time to assist in actively "chasing" the plume at the site.

Corps of Engineers Service Base

Background

The Corps of Engineers (COE) operates a maintenance facility on the Upper Mississippi River at the north end of Fountain City, Wisconsin. The Fountain City Service Base is the COE's principal maintenance facility along the Upper Mississippi River. The site is the location of releases of wastes generated from maintenance and operation activities carried out at the facility between 1950 and 1975. Maintenance activities include changing lubricating fluids; cleaning, changing, or repairing mechanical parts; and cosmetic maintenance such as sandblasting and painting. Prior to 1975, some of the wastes generated by these activities were disposed of on-site. The normal procedure for the on-site disposal was to form a depression in the sand, pour the material into the depression, and allow the porous soil to absorb the material.

Previous Investigations

The numerous on-site disposal areas described above were located through employee inquiries and review of operation records. Soils contaminated with the residual wastes from past facility operations were removed from the site in the early 1980s and shipped off-site for treatment and disposal. An internal audit in 1993 identified an area which had not been previously tested for the presence of contaminants. Several potential disposal pits were identified, soils samples were collected, and the samples were analyzed for PCBs, MEK, MIBK, and lead. Elevated levels of all contaminants were found to exist, resulting in the WDNR requiring a Remedial Investigation to be performed at the site.

Site Geology

The site geology is dominated by the presence of 10 to 15 feet of sandy dredged alluvium fill with discontinuous silty and clayey seams. This material was deposited during construction of the facility. The origin of the fill material was sand dredged from main channel maintenance of the Mississippi River. Underlying the dredge fill material to a depth of at least 40 feet are interbedded alluvial sand, gravel, clay, and silt.

Site Hydrogeology

The sandy soil at the site provides direct communication between the saturated aquifer and the River. Groundwater flow is toward the River during normal River flow conditions. Groundwater levels fluctuate with fluctuations in the River. The River elevation during the investigation was 653.2. The regulated normal pool at the site is elevation 651. Seasonal or periodic variations recorded over the last 10 years indicate a low pool elevation of 649.5 and a high pool elevation of 658.5.

Sampling and Testing Program

In April 1994, a 4-day sampling program was conducted at the site utilizing the hydraulic push sampling techniques discussed above. The initial sampling locations were based on a 20-foot by 20-foot sampling grid. Borings were placed on 20-foot centers beginning at one of the previously identified disposal areas or "hot spots". Rows of borings were added until the limits of contamination identified through field screening methods were determined.

Thirty probes were advanced to depths ranging from 2 to 16 feet bgs for a total probe depth of 228 feet. Two of the probes were manually advanced from a floating barge. Probes were advanced to a depth two feet below the water table at the time of the investigation. Soil cores two feet in length were collected, classified according to the Unified Soil Classification System, analyzed with a PID, and logged on Corps of Engineers field boring log forms. Thirty six soils samples were analyzed for volatile organic compounds (VOCs) in accordance with USEPA Method 601/602. Since groundwater contamination was a given at the site, only one grab sample was collected and tested for VOCs. Sampling, testing, and QA/QC were as discussed for the LUST site.

In addition to the on-site analytical testing, immuno assay kits were used to determine PCB and PAH levels at the site. Interferences produced by the heavy hydrocarbon contaminants at the site produced many false positives when the immuno assay results were compared to fixed-base analytical results on duplicate samples. Therefore, the results of the immuno assay testing program were inconclusive.

The initial round of sampling and on-site analytical testing identified major "hot spots" on the site. The hydraulic push sampling was then used at these locations to obtain samples for verification testing at a fixed-base analytical laboratory. Six samples were collected and analyzed for VOCs, DRO, PAHs, metals, and PCBs. The four holes from which the samples were obtained amounted to an additional 52 feet of probing. The total probe length at the site was 280 vertical feet.

Results of the Site Investigation

PCBs and DROs were found to exist at both shallow depths (2 to 6 feet bgs) and in a smear zone at the elevation of the water table. The smear zone is consistent with the known fluctuations in the River levels. VOCs and PAHs were not found at the site.

The presence of PCBs at levels greater than 100 ppm was indicated by the immuno assay testing. The fixed-base analytical testing did not verify these levels as discussed above. Based on information provided by those associated with the previous investigations, the on-site analytical equipment was calibrated for VOCs with the ability to note the presence of heavy hydrocarbons (fuel oil remnants). The on-site testing did not detect VOCs, but all samples indicated the presence of heavy hydrocarbons.

Based on the PCB false positives and the uncertainty in the DRO contaminant levels, a Phase II Remedial Investigation was completed in June 1995. The goal of the Phase II work was to quantify the PCB contamination levels and extents to develop an accurate remediation cost estimate, the most costly variable being the amount of material with PCB levels greater than 50 PPM.

Borings from the initial sampling program were resampled and tested in the on-site laboratory, which was calibrated for TPH as fuel oil. Samples were also collected for fixed-base PCB testing. The scope of the investigation was expanded to include a larger portion of the site based on the detection of heavy hydrocarbons in an area that was thought to be previously remediated. This work resulted in 4 days of hydraulic push sampling (approximately 300 vertical feet) and on-site analytical testing. Samples were collected using the 2-inch by 4 feet long Macro Cores to obtain the required amount of sample for DRO and PCB fixed-base laboratory testing.

Costs

The total cost for the hydraulic push sampling and on-site analytical testing program amounted to $10,200. A comparative cost for a standard drilling rig and fixed-based analytical testing is difficult to determine, but a standard rig for four days with two drillers would have cost approximately $8,000 and the analytical testing for VOCs and DRO would have cost approximately $9,500 for a total cost of $17,500. It should again be noted that the drilling and analytical costs are minimum costs because a standard drilling rig would have required more drilling time and more samples would have had to be collected and tested for each contaminant of interest. The hydraulic push sampling and on-site testing portion of the Phase II field work was completed at a cost of approximately $11,500.

Benefits of Hydraulic Push Sampling and On-Site Analytical Testing

The benefits derived from the sampling and testing program included reduced time in the field and reduced costs for sampling and testing for the purposes of delineating the extent and approximate levels of contamination at the site. This information was obtained at a much lower cost than using a standard drilling rig and fixed-based laboratory testing. The on-site analytical testing, although not able to be used to close a site, proved to be an excellent tool, providing information real-time to assist in actively "chasing" the plume at the site.

CONCLUSIONS

Applications

Integrated direct push sampling and on-site mobile laboratory analysis can be a valuable tool for a variety of site characterization projects. It can be applied to:

- ♦ Provide collection and analysis of sample matrices to confirm suspected releases, identify release source areas, and evaluate the vertical and horizontal extent and magnitude of the contaminant plume.
- ♦ Produce subsurface and analytical results for environmental property assessments.
- ♦ Landfill gas (i.e. methane and/or VOCs) delineation and monitoring.
- ♦ Continuous conductivity profiling to delineate subsurface stratigraphy of unconsolidated materials.
- ♦ Provide sampling and on-site analytical data to confirm remediation system effectiveness or confirm that specified closure criteria have been met.

Value and Advantages

The value and advantages of utilizing these techniques on site characterization projects include:

- ♦ Increasing the number of data points per unit time over that of traditional methods.
- ♦ On-site data allows you to make decisions in the field.
- ♦ Limit the number of monitoring wells by knowing the plume configuration prior to well installation.
- ♦ Lower overall project costs by reducing the number of fixed-base laboratory analytical samples and permanent monitoring points with resulting shorter project investigation time frames.
- ♦ Minimal disruption to the site, sample collection is discrete and rapid. No drill cuttings or development water to dispose of.
- ♦ Sample areas with limited site access.

REFERENCES

CHRISTY, T. (1987)
The Use of Small Diameter Probing Equipment for Contaminated Site Investigation. Geoprobe Systems. 5pp.

RHOADES, J.D. and J. van SCHILFGAARDE. (1976)
An Electrical Conductivity Probe for Determining Soil Salinity. Soil Sci. Soc. Am. J. 40:647-651.

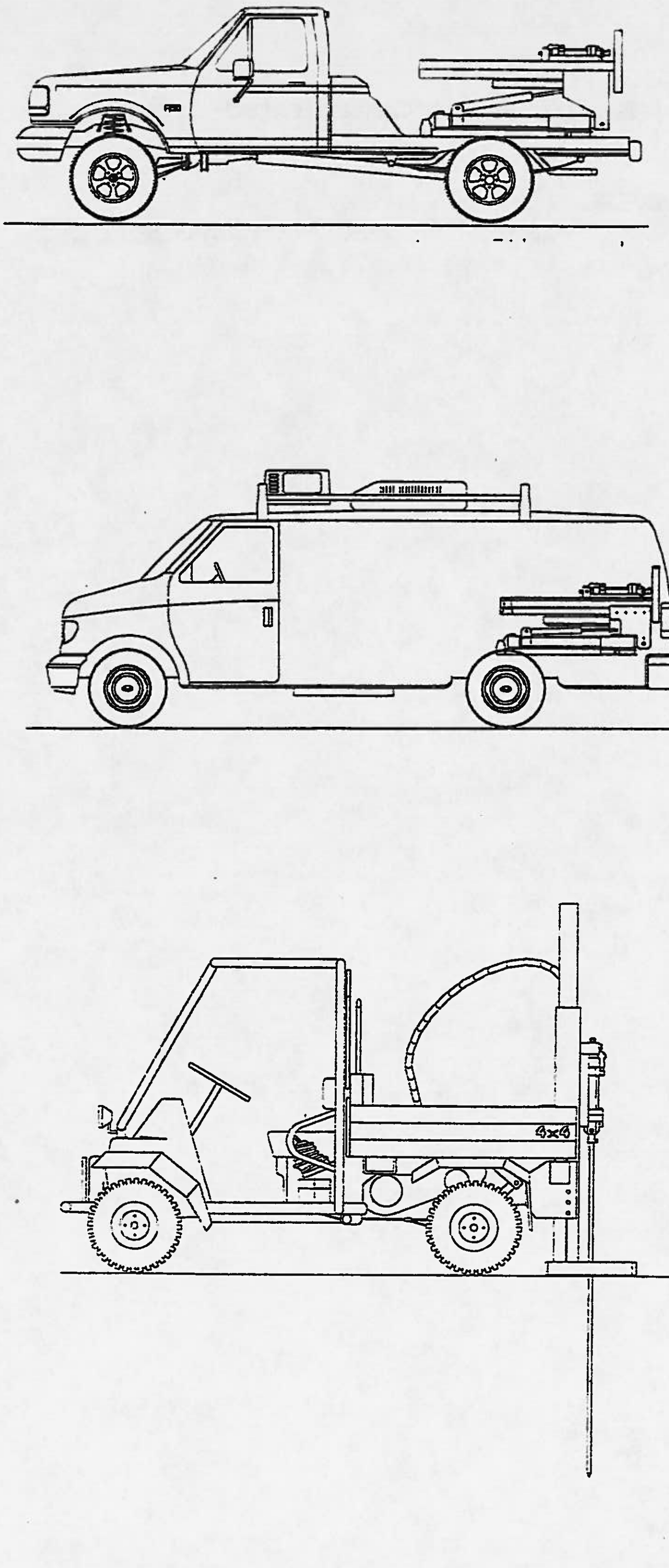


Figure 1:Various direct push sampling carrier vehicles including; 4X4 pick-up, cargo van, and all-terrain vehicle.

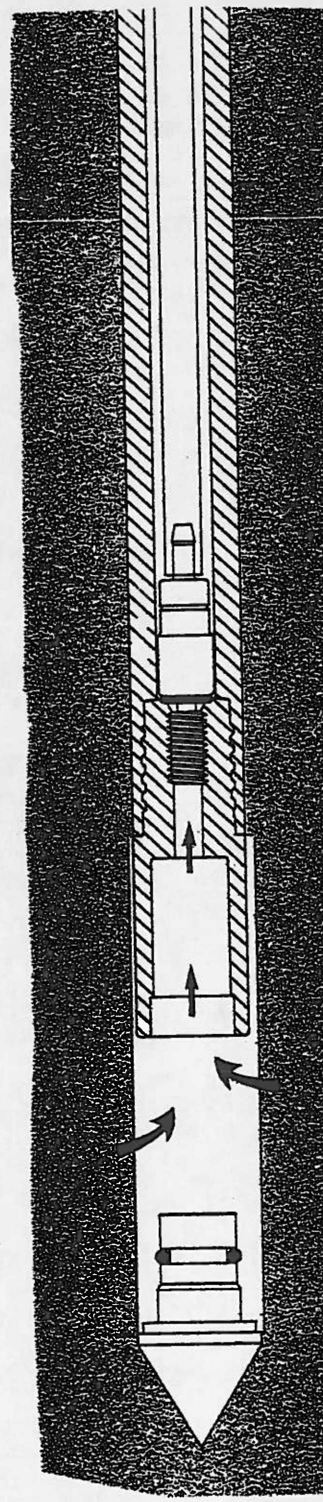


Figure 2: A cross section showing how soil gas is drawn through the inner tubing system.

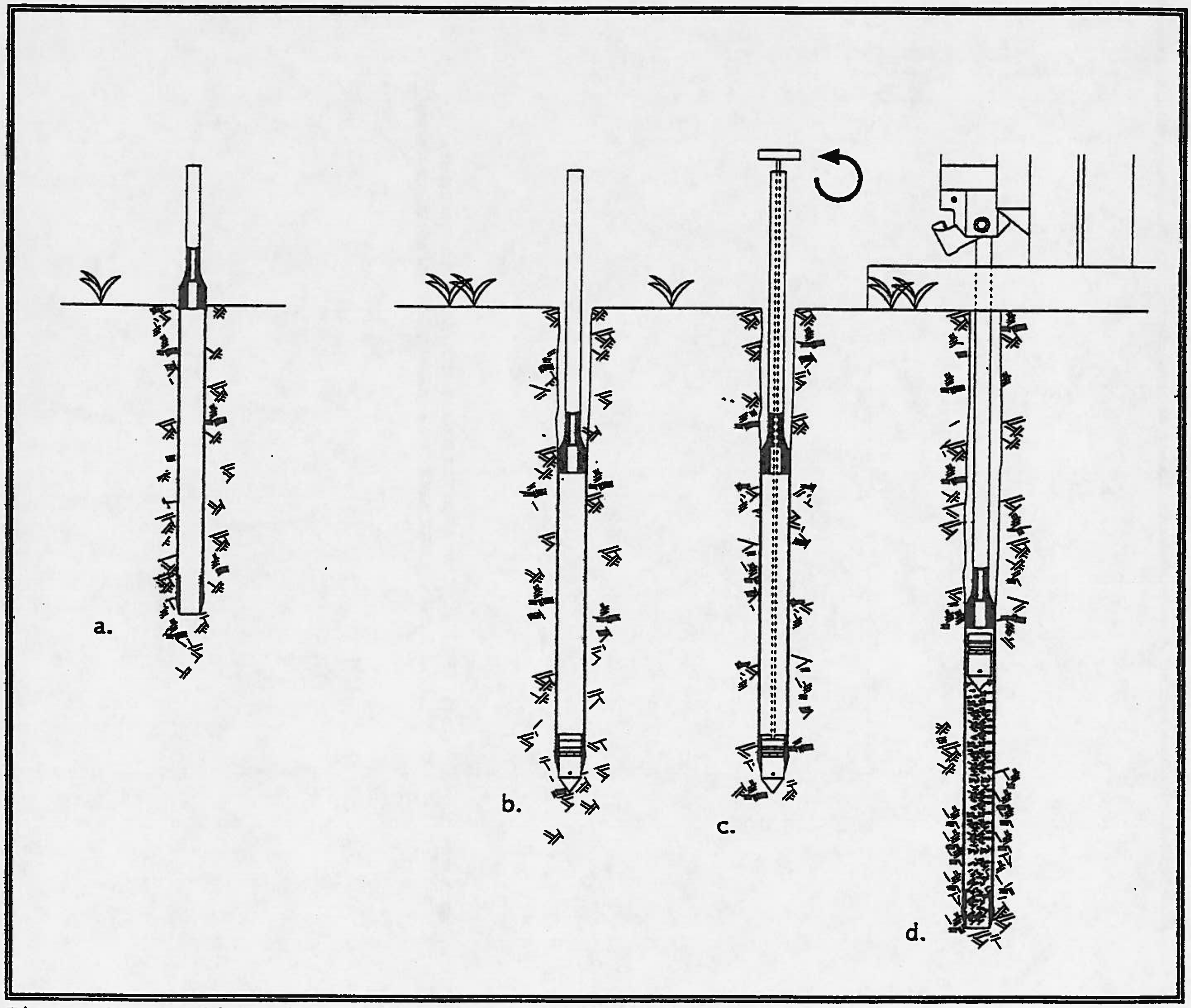


Figure 3: Sampling with soil sampler. (a) Sampling from ground surface (b) Closed sampler reaches top of sample depth (c) Sampler is unlocked (d) Sampler is advanced to recover new core

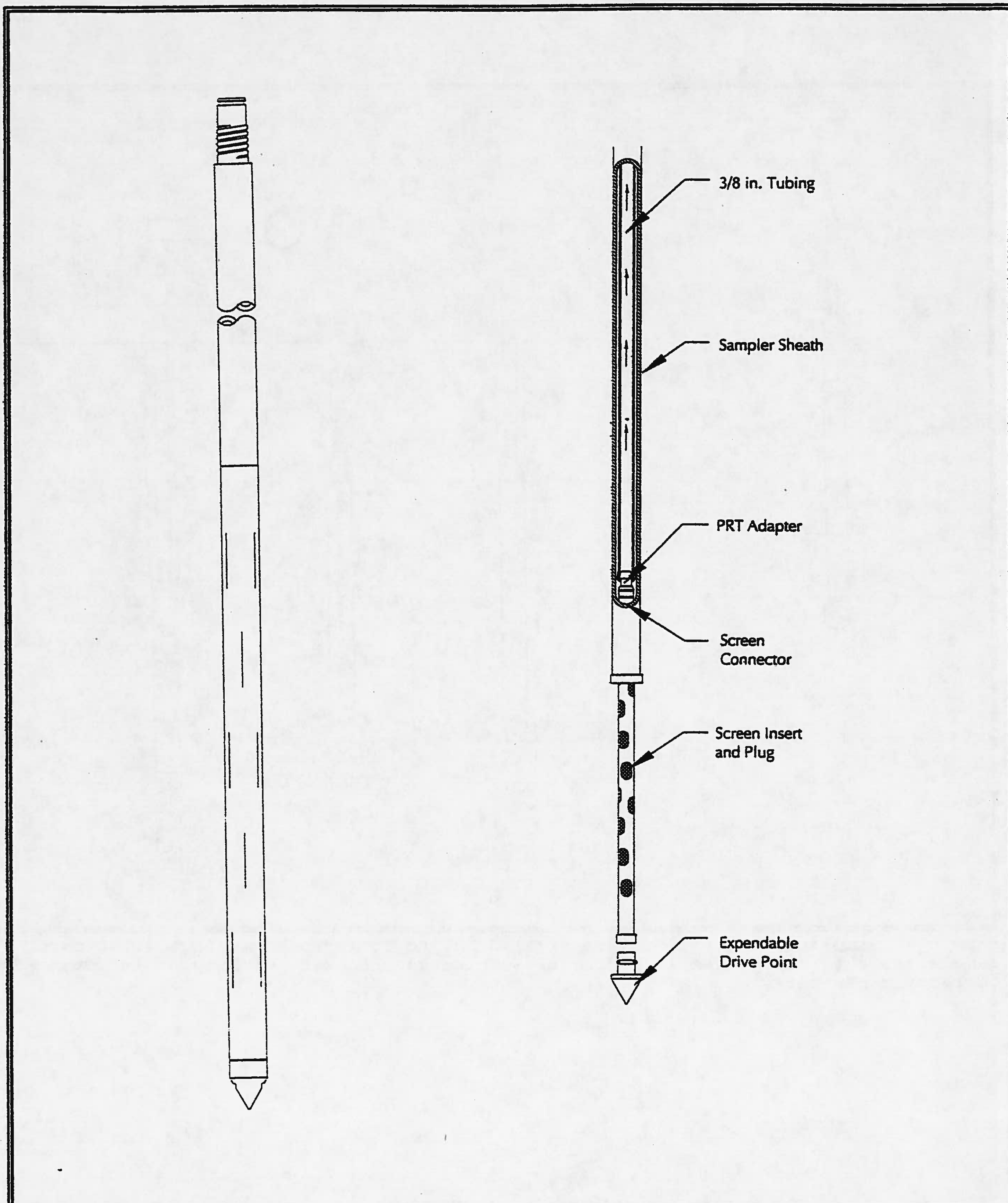


Figure 4: (Left) Mill Slotted Sampler (Right) Screened Point Ground Water Sampler

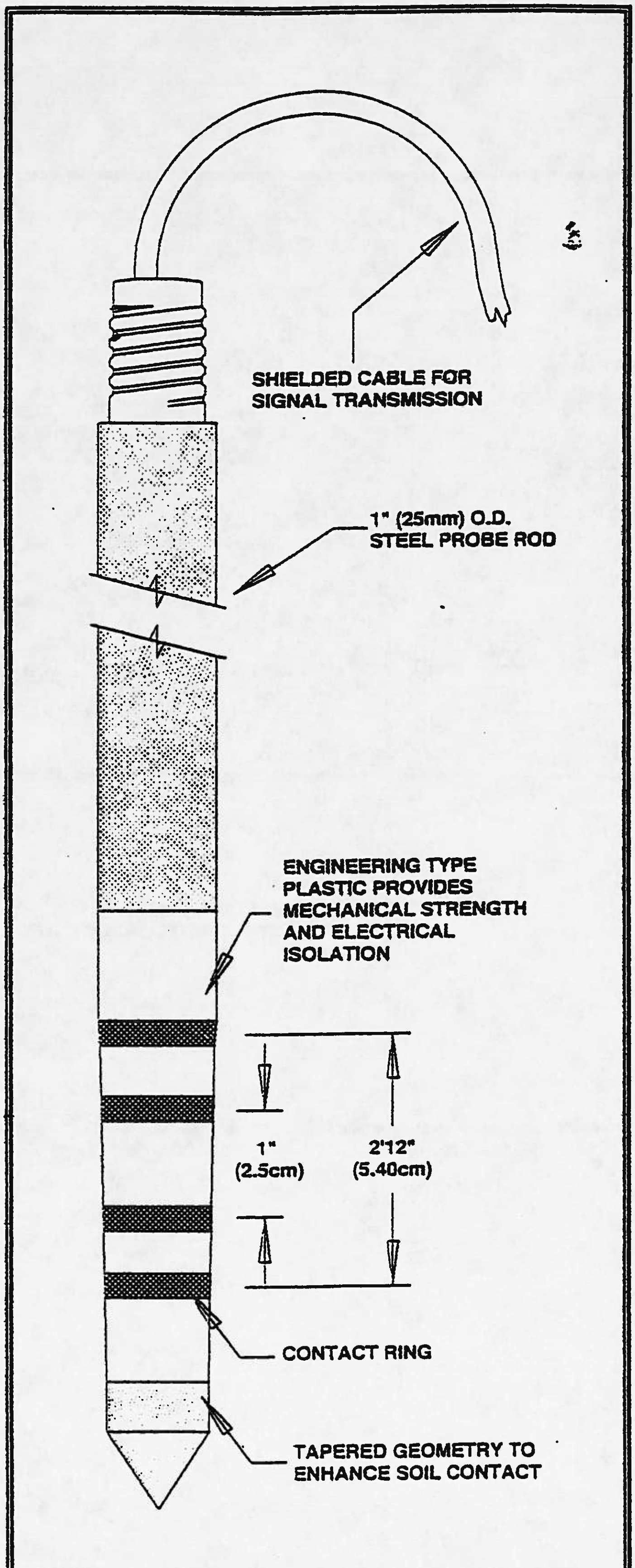


Figure 5: Conductivity probe construction

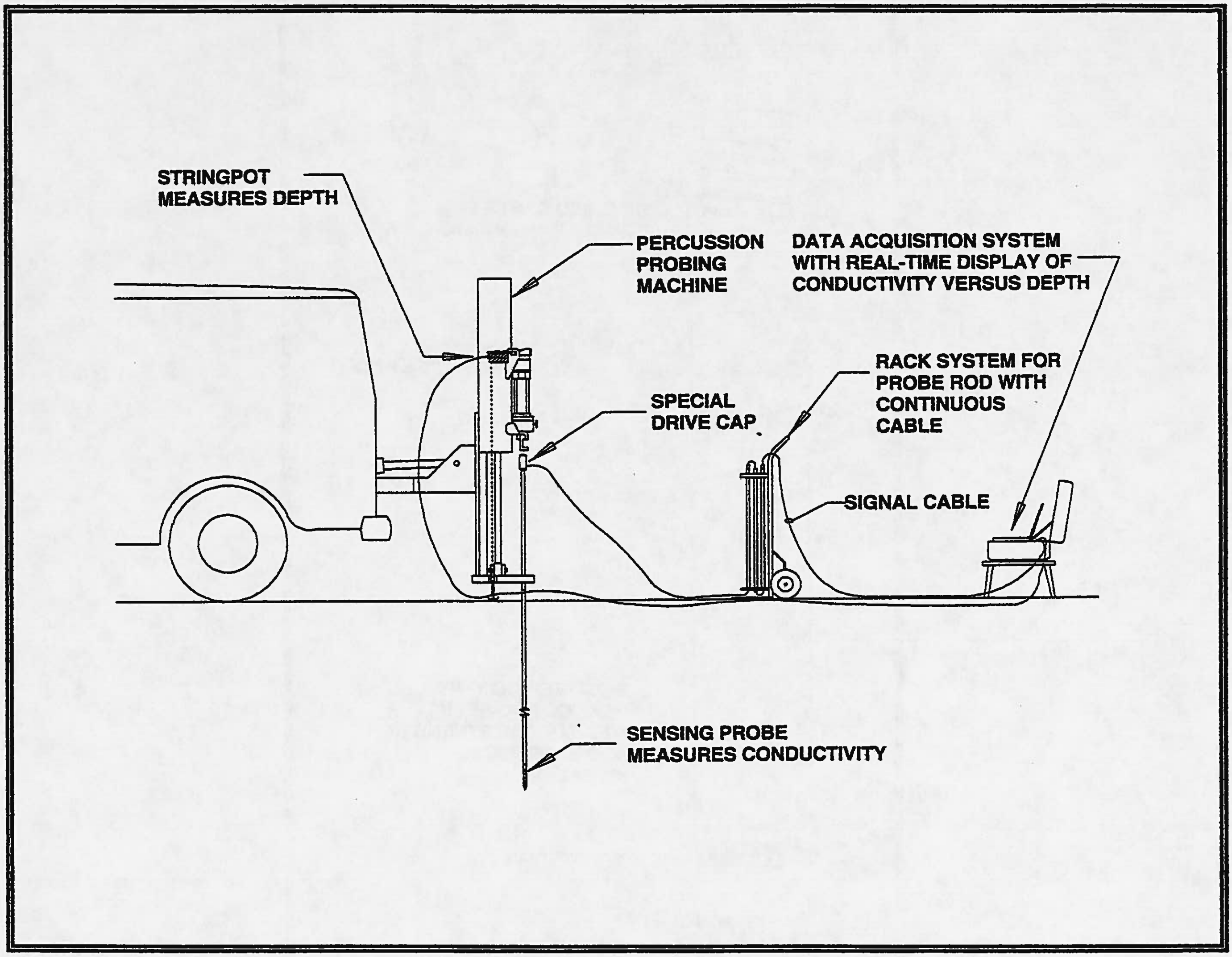


Figure 6: Major components of the conductivity probe system

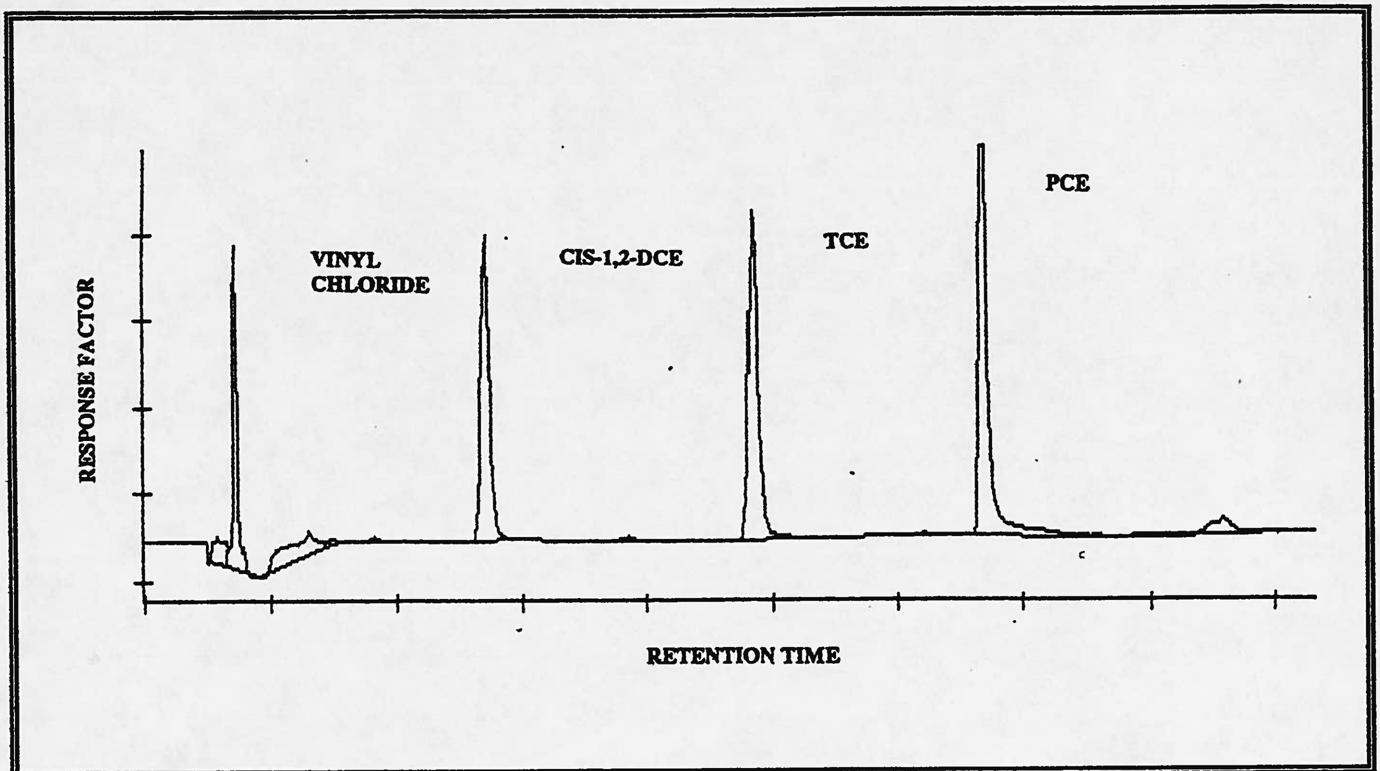


Figure 7: Chromatogram of a ground water sample showing the target contaminant PCE, and some of its common breakdown products vinyl chloride, cis-1,2-dichloroethylene, and trichloroethylene

USE OF THE GROUND PENETRATING RADAR AND SEISMIC SOUNDING SYSTEM FOR GEOTECHNICAL INVESTIGATION

M. Zoghi[1], P.J. Wolfe[2], B.H. Richard[3], G.F. Mitchell[4], T. Nogami[5]

ABSTRACT

The ground penetrating radar and seismic sounding system are used for a broad range of applications in mining, geophysical, geological and hydrogeological, archaeological, civil infrastructure system (CIS), and environmental engineering investigations. The recent advances in instrumentation and data acquisition have made these techniques cost effective in solving a wide variety of related subsurface and infrastructure problems. An overview of this technology along with various applications in reference to existing as well as proposed projects will be presented herein.

INTRODUCTION

In recent years, significant advances have been made in the field of geophysical exploration; in terms of both instrumentation and scope of application (Ohtomo 1993). Its conventional use, being limited to prospecting of natural resources such as petroleum, coal and ore minerals, has been extended to include a wide variety of applications in the field of civil infrastructure system (CIS) and environmental engineering as well (Ohya 1978). The high resolution and real-time data profile afforded by the new models along with the sophisticated computer software are some of the added advantages of modern geophysical systems (GS, Inc. 1995).

In the area of civil infrastructure system (CIS) which encompasses geotechnical, structural, construction, and transportation engineering, geophysical techniques have been employed for non destructive, rapid, subsurface investigations; non destructive evaluation (NDE) of roads, bridges, and buildings (Yokoya 1990). Specific applications in the geotechnical area include: geological strata profiling, bedrock mapping, rock fracture mapping, crevasse detection, borehole profiling, river and lake bottom profiling, permafrost mapping, sinkhole prediction, water table detection and mapping (Ulriksen 1983). Typical structural and construction engineering projects comprise: void detection, concrete thickness verification, reinforcing bar location and evaluation, post tension tendon locating, embedded conduit locating, pipe leak detection (gas, water, and oil-filled electrical), buried pipe and cable mapping. Transportation engineering types of projects include: pavement base/subgrade thickness verification, voids under pavement, moisture in asphalt and base course, runway integrity testing, railroad bed profiling, and tie

---

[1] Assoc. Prof. of Civil Engrg., Univ. of Dayton, Member of Ohio Infrastructure Institute (OII) Exec. Comm.
[2] Prof. of Physics and Geological Sciences, Wright State Univ., Dayton, OH.
[3] Prof. of Geological Sciences, Wright State Univ., Dayton, OH
[4] Prof. & Chair, Civil Engrg. Dept. & Dir. of Center for Geotech. & Envir. Research (CGER), Ohio Univ.
[5] Assoc. Prof. of Civil Engrg., Univ. of Cincinnati.

evaluation. The environmental engineering applications consist of hazardous waste mapping, landfill boundaries, trench boundaries, buried drums, underground storage tank detection, and contamination intrusion mapping (FGS 1992).

Recently, a team of researchers in Ohio has acquired state-of-the-art instrumentation for conducting multidisciplinary research, utilizing geophysical exploration techniques, as part of the Ohio Infrastructure Institute (OII) collaborative programs. The expertise of the researchers include geotechnical, structural, environmental, geological, and geophysical. An overview of the geophysical equipment acquired by the OII researchers along with their applications in reference to existing as well as potential projects will be presented in this paper.

MODERN GEOPHYSICAL METHODS AND THEIR APPLICATIONS

Following are some of the modern geophysical techniques which are commonly used to conduct the wide spectrum of investigations listed above (Ohtomo 1993).

Seismic Surveys

The principle of seismic exploration technique is based on the pulse arrival times within the soil medium. Generally, a sound pulse is introduced into soil by a small explosive charge or by means of a sledge hammer, while detecting the artificially generated seismic waves utilizing a special vibration detector called a geophone (Spangler and Handy 1982). From the plots of pulse arrival times versus distance, the subsurface velocity can be obtained. The seismic velocity is then used to interpret the hardness of the material and hence subsurface characteristics.

Three types of seismic surveys are routinely used for site characterization (Ohtomo 1993).

1. Surface-to Surface Seismic Survey

This group of techniques includes P-wave and S-wave refraction through soft soil deposits, SH-wave refraction through soft soil deposits, and high-resolution P-wave reflections in soft soil and bedrock deposits.

2. Surface-to-Borehole Seismic Survey

This group of techniques includes zero-offset vertical seismic profiling (VSP) and offset VSP. For these methods, seismic receivers are lowered into boreholes and seismic sources are energized on or just below the ground surface.

3. Borehole-to-Borehole Seismic Survey

This group uses crosshole approaches with seismic sources and receivers both in boreholes. Seismic tomography can reconstruct the velocity distribution of a target area utilizing first arrival travel times.

Resistivity Surveys

Resistivity, defined as "electrical resistance per unit length of a unit cross-sectional area," measures the resistance of soil to flow of an electrical current (Spangler & Handy 1982). In other words, it indicates the water content and the concentration of dissolved ions in a soil. Accordingly, the soil characteristics can be identified. For instance, saturated clays possess low resistivity, whereas dry soils and bedrock have high resistivity. The resistivity of saturated granular materials falls somewhat in between these two extremes. There are two types of resistivity methods: the Vertical Electrical Sounding (VES) and Horizontal Electrical Profiling (HEP). The VES is typically used to outline the vertical resistivity of a horizontally layered subsurface, whereas, the HEP is adopted to detect anomaly due to fault or other discontinuities.

A common application of the resistivity method, using either one of the above techniques, is to conduct an area survey to delineate the limits of a particular deposit by depicting a resistivity contour map (Spangler & Handy 1982). For instance, it can be utilized far more efficiently in lieu of conventional drilling to verify the extent of a buried firm layer designated for a building foundation, or outline of a buried acquifer, gravel deposit, or caverns.

Ground Penetrating Radar (GPR)

The ground penetrating radar (GPR), also referred to as electromagnetic subsurface profiling, electromagnetic pulse radar, pulsed microwave, pulsed radio frequency, and ground probing radar, utilizes electromagnetic waves (EW) for site characterization (EPA 1993). Very High Frequency (VHF) waves are generated into soil by means of an antenna in about a 90° cone close to the ground surface, whereby the reflections from boundary planes between two media are recorded by a receiving antenna. The reflections relate to the propagation velocities and dielectric constants, and hence detecting discontinuities, e.g., identifying depth to the water table or bedrock, and extent of cavities present underground (Dominic, et al 1995). A high resolution continuous profile is obtained by dragging the antenna along the ground surface. The depth of penetration, however, ranges between 1 to 10 meters.

SEISMIC SOUNDING AND GPR SYSTEMS ACQUIRED BY OII

The Ohio Infrastructure Institute (OII), a collaboration of the Engineering Colleges in Ohio, was established by the Engineering Deans' Council in 1992 to pool the resources and expertise in engineering and management to address the State's infrastructure needs. Recently, the OII delegates procured, among others, state-of-the-art seismic sounding and ground penetrating radar systems to conduct research involving the condition assessment of infrastructure and their surrounding soil. A brief description of these instrumentation along with their capabilities follows.

Seismic Sounding (SS) System

As described earlier, seismic survey can be utilized to investigate subsurface characteristics of an extensive area expeditiously. Furthermore, at higher frequency ranges it can be employed for condition assessment of concrete structures and pavement systems. Ordinarily, a seismic energy source, a receiver, control and recording unit make up the components of a system. The impulsive energy source is used to generate seismic waves, whereas geophones or hydrophones are employed to pick up the signal. The specific components procured by the OII researchers are listed below.

A seismic recording system possessing 36-channels with built-in roll along capability to roll 24 channels. Operation will be controlled by a built-in 486 computer system. Seismic recording cables, including adaptors. Number of hydrophones and geophones for receiving the seismic signals. An elastic wave generator and an air-gun system for generating seismic waves.

Ground Penetrating Radar (GPR)

The GPR system acquired by the OII has several unique features in addition to the basic unit described above. The noteworthy trait of the designated OII's GPR is the fact that it will be coupled with a Cone Penetrometer Test CPT) and a Soil Moisture Resistivity (SMR) probe which will enhance the site characterization.

The Cone Penetrometer Technology (CPT) was procured by the Center for Geotechnical and Environmental Research (CGER) at Ohio University in 1993 which consists of a 25 ton truck containing the CPT and additional peripherals; computer hardware and software. Recently, the OII via the Ohio Board of Regents (OBOR) teamed with the CGER-OU to enhance the CPT by acquiring a GPR and a SMR probe. The CPT system portion of the GPR consists of a set of borehole antennas, one transmitting and one receiving, controlled by the GPR Radar Control Set. The radar signal propagates through the soil between the boreholes from the transmitting antenna to the receiving antenna. This radar signal is modified by the soil condition. Geotechnical analysis of the radar data via tomographic software provides such information as soil moisture variations, soil stratigraphy, soil type, water table level, and possible hazardous waste contamination between the boreholes. Additional uses of the system include determining the thickness of light non-aqueous phase liquid floating on top of the water table and location of voids beneath highways, buildings or bridge foundations.

The highway portion of the GPR system consists of a set of very high resolution antennas, one transmitting and one receiving, controlled by the GPR Radar Control Set. The radar pulse reflects off of the pavement/base layer and base/subbase layer, indicating the layer thickness. The highway unit is used for the non-destructive testing of highway pavement systems, such as measurement of pavement thickness. This measurement is made continuously as the unit is moved down the highway. Thickness readings are made every few inches of forward travel. Highway maintenance problems can also be detected, such as voids under pavement and excess moisture in the base material. Excess moisture can lead to asphalt "stripping" and potholes.

The Soil Moisture Resistivity (SMR) system is used for continuously measuring soil moisture and electrical resistivity of the soil around the hole as the CPT probe is pushed into the ground. This information by itself is very important for geotechnical analysis of soils; however, the CPT/SMR data coupled with the CPT/GPR data provides a much better "representation" of subsurface soil conditions in a large volume of soil, greatly reducing the number of required boreholes.

MULTIDISCIPLINARY RESEARCH

As indicated earlier, the researchers in Ohio with the following expertise; i.e., geology and geophysics, environmental, geotechnical and structural engineering, will be collaborating on a number of cross-discipline research projects. The following are among the immediate and potential collaboration research involving utilization of reference SS and GPR systems.

U.S. 23 Ohio SHARP Test Road Project. This is an on-going federally funded pavement related project, located on U.S. 23 approximately 25 miles north of Columbus in Delaware County. Currently, researchers from six different universities in Ohio are involved with various aspects of pavement drainage, subgrade, and instrumentation. The GPR will be integrated as part of this research endeavor once the construction is finalized.

Investigation on Configuration of Bedrock and Depth of Water Table. Wright State University faculty and students have conducted a series of geophysical investigations for the assessment of possible disruption of ground water flow by the construction of U.S. 68 near the Ceder Bog for the past several years (Wolfe, et al 1991). A combination of gravity and seismic refraction and reflection surveys, and magnetic surveys have been utilized to gain information on the bedrock configuration and the valley-fill stratigraphy (Wolfe, et al 1990). The reference SS and GPR systems will be employed in the future to complement the existing data.

NDT & E of Drilled Shafts and Abutments. The proposed project is a combination of substructure evaluation and surrounding subsurface investigation for a three-span prestressed concrete bridge to be constructed over the Stillwater river in Miami County, Ohio. The reference geophysical equipment will be used, among others, to verify the integrity of foundation by identifying (Hearne, et al 1981): (1) cave-in along the caisson-soil interface, (2) squeeze or necking of caisson shaft, (3) segregation and voids in concrete, and (4) soil inclusion.

CONCLUSIONS

An overview of geophysical techniques was presented in this paper. The series of Seismic Sounding (SS) and Ground Penetrating Radar (GPR) systems procured by the Ohio Infrastructure Institute (OII) researchers were outlined along with their applications involving several different on-going, as well as proposed projects.

Compared with SS system, the GPR system possesses a more enhanced resolution. It has, however, a much more limited depth of penetration. Therefore, the SS system will be primarily used to cover an extensive area under consideration in order to identify the critical and susceptible locations requiring detailed investigation. Subsequently, the GPR system will be utilized for a more localized and detailed exploration. These two sets of instrumentation are considered to be complementary to each other and will provide invaluable information on the condition assessment and site characterization.

REFERENCES

Dominic, D.F., Egan, K., Carney, C., Wolfe, P.J., Boardman, M.R. (1995)
Delineation of Shallow Stratigraphy Using Ground Penetrating Radar. Journal of Applied Geophysics, Vol. 33, pp. 167-175.

Environmental Protection Agency (EPA, September 1993)
Use of Airborne, Surface, and Borehole Geophysical Techniques at Contaminated Sites. A Reference Guide, EPA/625/R-92/007, Office of Research and Development, Washington DC, pp. 6-1/1-10.

Finnish Geotechnical Society - FGS (1992)
Ground Penetrating Radar, Geophysical Research Methods. The Finnish Building Center Ltd., 64 p.

Geophysical Survey Systems, Inc. - GS (1995)
Subsurface Interface Rdara. Geophysical Survey System - Technical Note, North Salem, NH, USA.

Hearne, Jr., T.M., Stokoe, II, K.H., Reese, L.C. (1981)
Drilled-Shaft Integrity by Wave Propagation Method. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT10, pp. 1327-1345.

Ohtomo, H. (1993)
New Geophysical Survey Methods and Their Applications. OYO Geoforum, Singapore, OYO Corporation, Head Office, Tokyo, Japan, 21p.

Ohya, S., Takeuchi, T., Imai, T., Ito, K. (1978)
Geophysical Investigation for Civil Engineering Purposes in Japan. OYO Technical Note, OYO Corporation, Tokyo, Japan, 34p.

Spangler, M.G., Handy, R.L. (1982)
Soil Engineering. Harper & Row Publishers, New York, 819p.

Ulriksen, C.P. (1983)
Application of Impulse Radar to Civil Engineering. Lund University of Technology, English Translation, 175p.

Yokoya, N., Kanemori, T. (1990)
Application of Seismic Refraction Survey to Investigation for Tunnel Construction. Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems, Sponsored by the Society of Engineering & Mineral Exploration Geophysics, Golden, CO, pp. 105-221.

Wolfe, P.J., Richard, B.H. (1991)
Integrating Geophysical Techniques to Study a Small Confined Aquifer. Exploration Geophysics, Vol. 22, pp. 457-460.

Wolfe, P.J., Richard, B.H. (1990)
Geophysical Studies of Ceder Bog. Geotechnical and Environmental Geophysics, Vol II: Environmental and Groundwater, Soc. of Expl. Geophysics.

INVESTIGATION AND INSTRUMENTATION OF DEEP LACUSTRINE CLAY DEPOSITS IN THE CUYAHOGA RIVER VALLEY

Paul H. Anderson, P.E.
Senior Project Manager
BBC&M Engineering, Inc.

Stuart P. Ravary, E.I.T.
Staff Engineer
BBC&M Engineering, Inc.

Abstract

Deep deposits of lacustrine clay interbedded with glacial tills are responsible for the behavior of slide prone slopes affecting structures located within the Cuyahoga River Valley near Cleveland, Ohio. The west abutment of the Interstate 90 bridge crossing the river valley has been sliding for 25 years, impacting the friction piled piers which support the arched truss steel deck section spanning the river. This paper details the extensive field investigation and instrumentation program, laboratory testing and analysis performed to determine the lateral extent and depth of the sliding mass of soil to be resisted by the proposed bridge remediation. A preliminary parametric stability study was performed to plan an investigation program ultimately comprised of 10 slope inclinometers and 12 pneumatic piezometers installed in boreholes, drilled to bedrock to depths of up to 235 feet. Approximately 2600 lineal feet of drilling including hollow stem auger and tricone mud rotary techniques were employed to sample soil and install the instrumentation through surficial granular alluvial deposits, glacial till and deposits of lacustrine clays up to 150 foot thick. Existing land use presented challenges in installing and securing the instrumentation. Ninety undisturbed samples were obtained and consolidated drained triaxial tests and direct shear tests were performed, many requiring high pressure cells for the large confining pressures used to simulate overburden conditions. A strength envelope for residual effective strength parameters for the lacustrine clay deposit was developed. STABL 5M stability analyses were performed to predict location of failure planes and aid calculation of lateral soil forces on pier foundations. A year of slope deformation and piezometric level data, concluding in the summer of 1995, was collected bimonthly for the purpose of corroborating the theoretical slope failure surface prior to initiation of costly remediation measures.

INTRODUCTION

Deep deposits of lacustrine clay interbedded with glacial tills are responsible for the behavior of slide prone slopes affecting structures located within the Cuyahoga River Valley near Cleveland, Ohio. BBC&M Engineering of Dublin, Ohio as the geotechnical subconsultant, in contract with Richland Engineering Limited of Mansfield, Ohio and the Ohio Department of Transportation (ODOT), is investigating slope instability associated with the Interstate-90 Bridge over the Cuyahoga River Valley approximately one mile from the shore of Lake Erie. The I-90 bridge, completed in 1959, is approximately 5,100 long and consists of a series of continuous beam and girder type approach spans with main spans of arched steel deck truss construction. In 1988 the traffic rate on the bridge was 146,640 vehicles per day; the bridge is a vital link in the east - west transportation corridor across northern Ohio and the City of Cleveland. The location of the bridge is shown on the Vicinity Map as Figure 1.

Geologic literature indicates that the preglacial valley in the project area is filled with lacustrine clay deposits overlain with granular alluvial terraces. Previous monitoring by others has indicated that the west end of the bridge is moving, presumably as a result of slope instability, towards the river. These observations have shown very slow pier movement (2.3 inches over 20 or more years). In addition to pier movements, a sheet pile wall along the bank of the Cuyahoga River and down slope from Pier No. 1 has exhibited rotation. Initial stability analyses indicated that possible failures may be individual slope rotations at the west side piers and abutments or a deep-seated slide of unknown lateral extent.

BBC&M's involvement on this project was to determine if: soil foundation movement was still occurring and whether the movement was shallow or deep-seated; the slope was only marginally unstable or if the rate of movement was likely to accelerate; and to make recommendations for stabilization if needed. Our approach was to compare measured field data from slope inclinometers with theoretical analyses using extensive site-specific data.

The investigation was divided into the three phases. The first phase, preliminary analyses, included collecting readily available data so that a thorough subsurface investigation and monitoring program could be formulated for the west abutment of the bridge. Phase two was the performance of the field work consisting of drilling, soil testing, instrument installation, and stability analyses and phase three included the instrumentation monitoring.

Several unique technical challenges were faced by the project team. These included: a very small reported rate of slide movement which is difficult to detect in short periods of time, the presence of existing movement implying the necessity of using residual shear strengths; high subsurface pore pressures reportedly encountered during drilling of earlier investigations; the need to locate and test thin horizontal stratified deposits known to exist in lacustrine clays; the depth to bedrock in excess of 200 feet; the need to perform an investigation in the middle of an on-going aggregate stockpiling operation for an asphalt hot mix plant; and the need to protect the delicate instruments from heavy truck traffic and future construction for a period of several years. These challenges were met through careful planning and coordinating of drillers, instrument vendors, land owners, contractors, laboratory technicians and engineers.

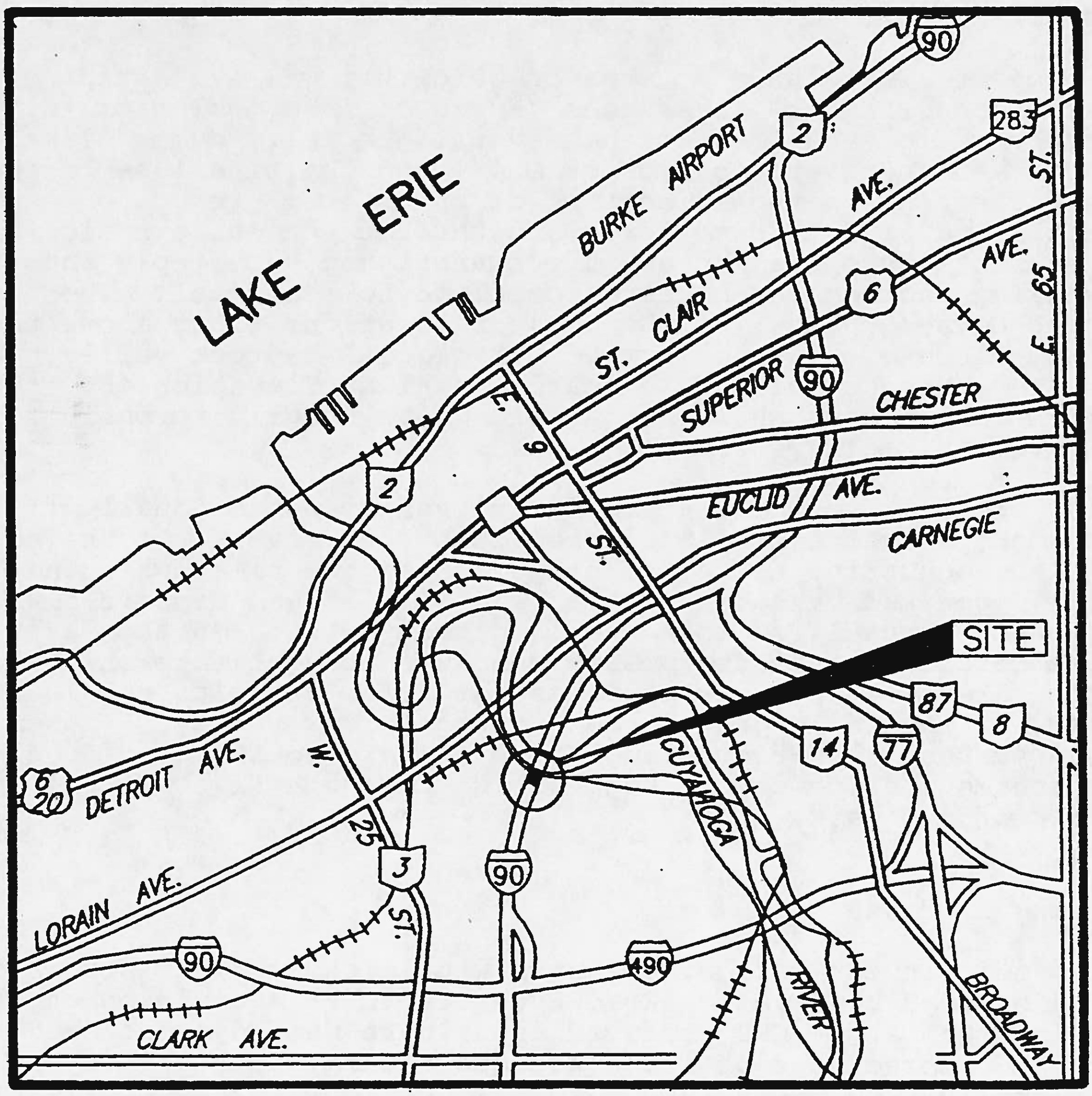


Figure 1
Vicinity Map

PRELIMINARY ANALYSES

The scope of Phase 1 included the review of available geotechnical data and based upon that information, the performance of preliminary slope stability analyses and the identification of probable failure planes. The slope stability analyses assisted in the selection of boring locations, sample depths and types, and instrumentation needed to proceed with the field portion of the program for Phase 2. We anticipated installing both slope inclinometers and pneumatic piezometers and obtaining undisturbed samples for shear strength testing at critical locations.

General Geologic Conditions

The project lies within an area of Ohio that was subjected to glaciation during the Pleistocene Epoch of late geological time. Near the end of the Pleistocene several glacially-dammed lakes occupied the northern portion of Ohio, and the site lies in the Lake Plains Physiographic section of the state. Prior to glaciation a river flowed northward through the eastern side of the present-day Cleveland and the channel had cut deeply into the underlying bedrock to an elevation close to sea level. The present Cuyahoga River at the project locations flows above the western slope of this now buried, preglacial bedrock valley. A the site the rock surface is nearly level at Elevation 450. The bedrock consists of shale which belongs to Chagrin formation of the Devonian rock system.

During the Pleistocene the preglacial valley was gradually filled with sand, gravel and silt carried in by the river from the south and with lacustrine silts and clays during the times when the area was covered by lakes. Some glacial till was deposited when tongues of glacial ice advanced into the valley. In the later stages of the Pleistocene mostly sand was brought into the project area by the river and these granular deposits remain as terraces along the edges of the Cuyahoga River Valley. Present ground surface elevations range from 580 along the toe of the west abutment slope at the edge of the Cuyahoga River to 674 at the top of the bank.

Existing Land Use

During this investigation, the area within the project boundaries was being used by Cuyahoga Road Products (CRP) a division of The Osterland Company. CRP produced asphalt at the adjacent batch plant and stored some of its aggregate within the right-of-way. Additionally, the CRP access road was just down slope of Pier 1. During the paving season the sand storage pile, just up slope of

Pier 1, was continuously resupplied with large dump trucks. A coarse aggregate storage pile was resupplied approximately once per month by a lake freighter.

Three sets of piers support the west side of the I-90 bridge. The plan and profile layout of these piers are shown on Figures 2 and 3.

Previous Investigations

Four borings were obtained by ODOT in the area of the West End Pier and Pier No. 1 as part of the original soils investigation in 1954. These borings show standard penetration numbers and the distinction between surficial granular materials and underlying clays, but provide limited detail. In 1990, nine borings were performed by ODOT and slope inclinometers were installed in these bore holes. In 1992 one additional boring was obtained by ODOT. These borings provide more detailed descriptions, classifications, grain size analysis and Atterburg Limits. All of these borings are clustered directly under the bridge and do not extend to bedrock. The borings indicate granular materials consisting of gravelly sands and sandy silts overlying cohesive deposits of gray silty clay.

Deformation information in the form of expansion joint measurements, roller bearing tilt measurements and pier movements, available from other studies performed for ODOT, indicate that Pier No. 1 has moved east between 2 and 3 inches and that the West End Pier has moved east up to 1 inch. At the time of our investigation, some conflicting information existed concerning the exact magnitude of movement, depending upon the three types of measurements examined, but all data substantiates that both the West End Pier and Pier No. 1 had moved east and that Pier No. 1 has moved substantially more than the West End Pier.

Preliminary Slope Stability Analysis

The purpose of the preliminary slope stability analyses is to determine the probable location(s) of the failure plane to aid in the layout of the field investigation program. Analyses were performed which assumed the slope is near a factor of safety of one since movement had already been mobilized (Duncan and Stark, 1992). Analyses provide probable slide plane locations for the slope at incipient failure. The information assists in the location of slope inclinometer, pneumatic piezometer, and shelby tube samples in the areas of possible failure planes.

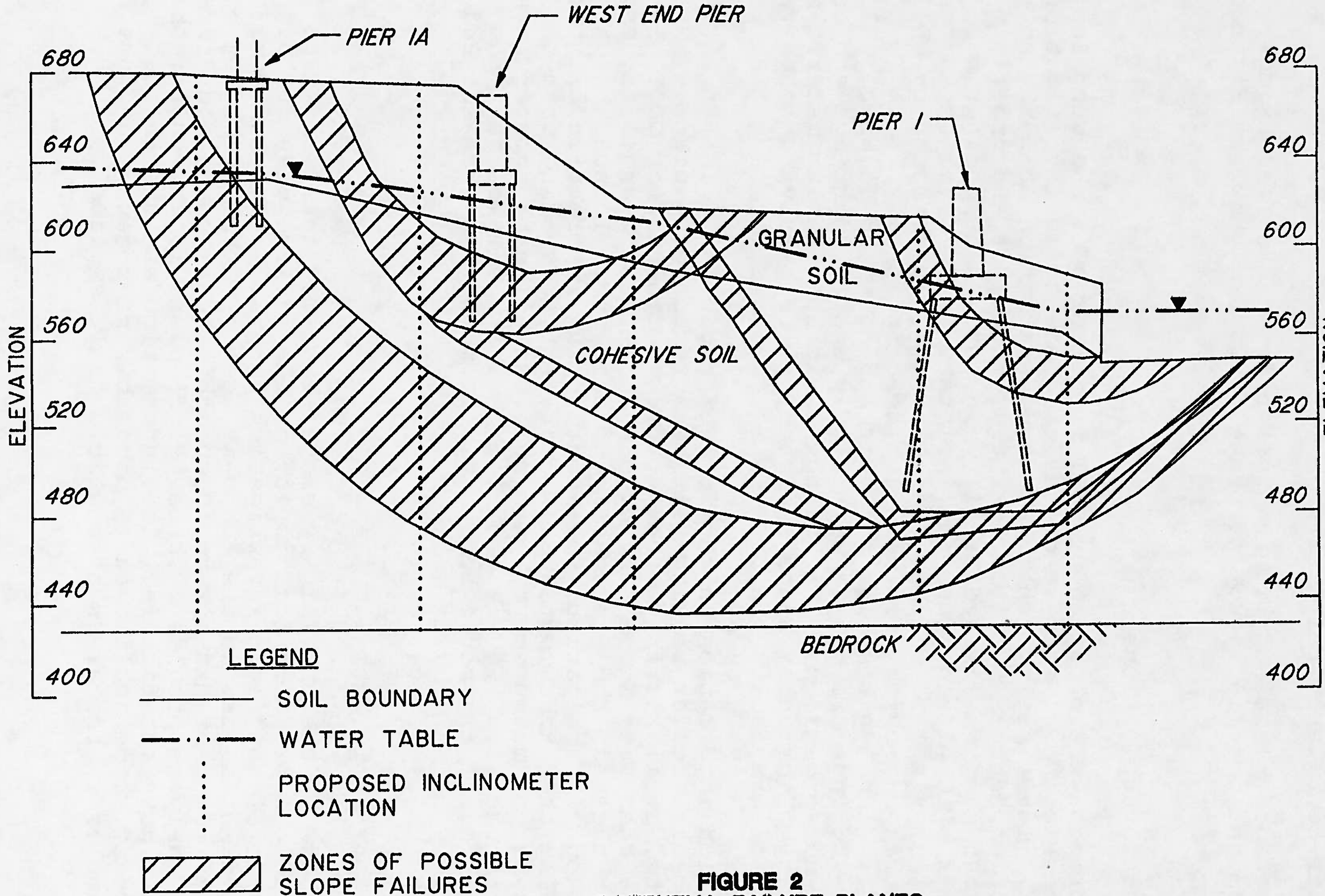


FIGURE 2
POTENTIAL FAILURE PLANES
BASED UPON PHASE 1 STABILITY ANALYSIS

Stability analyses were performed using the PC STABL5M computer program developed by Purdue University. Both circular failures and block failures were modeled using Modified Bishop analysis which assumes a circular failure surface and the Janbu sliding block method. The stratigraphy encountered in borings performed by ODOT was used to model the general site conditions. This simple profile consisted of granular soil overlying a cohesive soil extending to bedrock. Bedrock was reported to exist at Elevation 428 near Pier No. 1. The shear strength parameters for the soils were chosen, by successive trials, which yield factors of safety near one, for both drained and undrained analyses.

Three different slide scenarios (locations) were modeled representing:

1. Deep seated slide extending from the river side of the bulkhead to the landward side of the West End Pier.

2. Shallow slide extending from the river side of the bulkhead to the landward side of Pier No. 1.

3. Shallow slide extending from the toe of the slope just down slope of the West End Pier to the top of the slope located up slope of the West End Pier.

Figure 2 shows the various probable failure zones with factors of safety near one. All analyses assumed an aggregate pile exerting an unit load up slope of Pier No. 1. The preliminary analysis presented in Figure 2 represents a range of soil parameters and slide geometries which allowed the field investigation program to be designed.

DRILLING AND TESTING

Boring Layout

The possibility of deep seated movement occurring over a large areal extent required that the boring program extend from the bulkhead along the west side of the Cuyahoga River to the landward side of Pier 1A. The geometry of the slide planes made borings and monitoring of the east side of the Cuyahoga River unnecessary. Since the lateral extent of the slide was unknown, some boring information was obtained beyond the immediate limits of the bridge in both the up and down river directions.

Figure 3 is a Plan of Borings showing general topographic information, pier locations and boring locations. Table 1 shows details of the 10 slope inclinometers and 12 piezometers that were installed in the 15 boreholes.

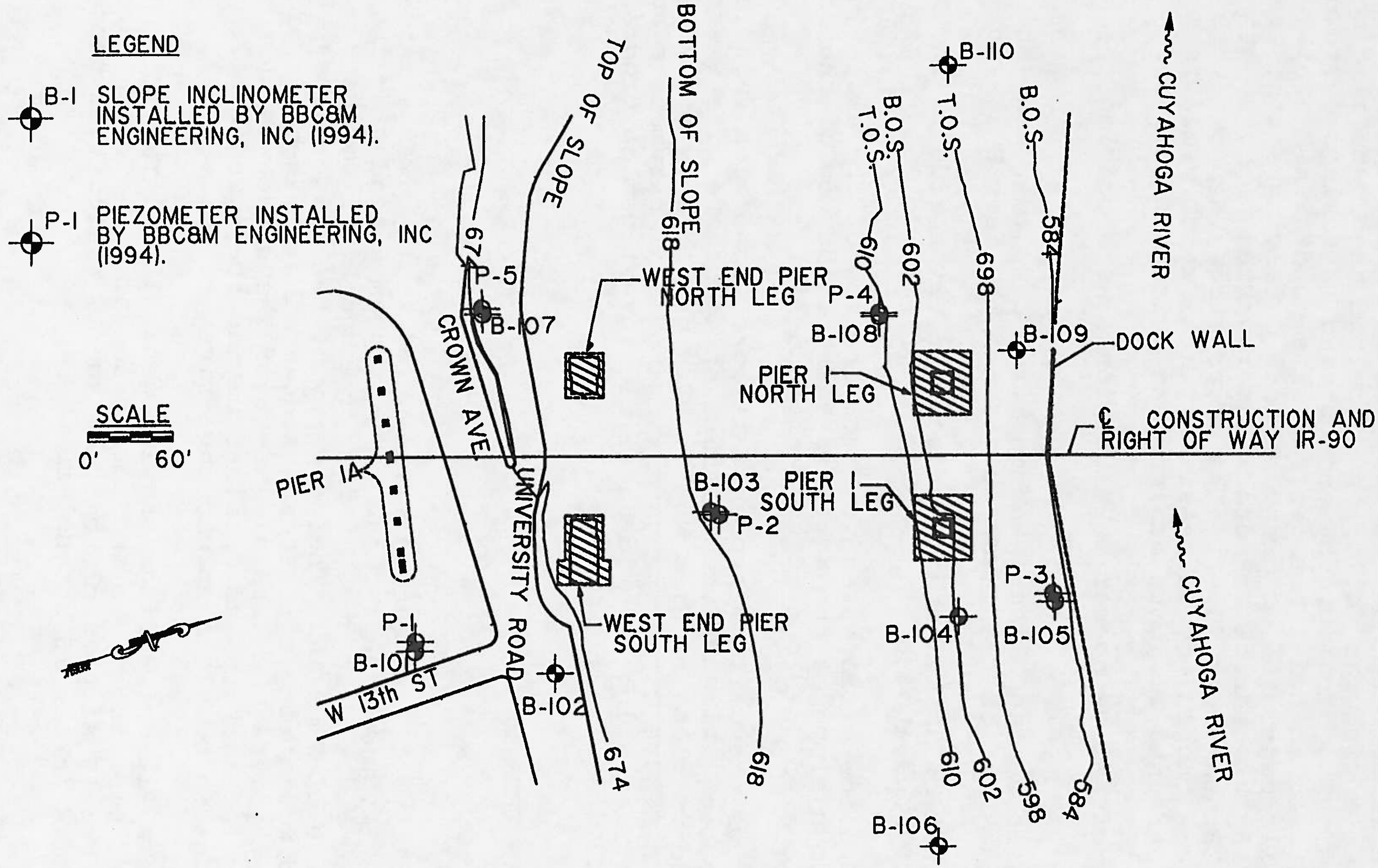


FIGURE 3

PLAN OF BORINGS AND EXISTING TOPOGRAPHY

Boring Number	Date		Ground Surface Elev.	Length of Rock Core	End Depth	Instrument Installed
	Begin.	End				
B-101	8/23	8/25	676.6	5.6	230.1	Slope Inc.
B-102	9/9	9/14	675.7	5.0	232.0	Slope Inc.
B-103	8/15	8/17	616.2	5.0	175.0	Slope Inc.
B-104	9/13	9/15	601.8	5.0	165.5	Slope Inc.
B-105	8/17	8/19	585.4	5.0	150.0	Slope Inc.
B-106	9/8	9/12	612.3	5.0	174.2	Slope Inc.
B-107	9/19	9/22	662.7	5.0	215.0	Slope Inc.
B-108	8/8	8/10	610.4	5.0	170.2	Slope Inc.
B-109	8/25	8/29	593.3	4.5	163.2	Slope Inc.
B-110	8/31	9/8	596.7	5.0	155.0	Slope Inc.
P-1	9/1	9/2	676.6	-	158.5	Piezometer
P-2	8/18	8/19	616.3	-	168.1	Piezometer
P-3	8/22	8/24	585.6	-	140.0	Piezometer
P-4	8/11	8/12	610.1	-	149.0	Piezometer
P-5	9/26	9/27	662.3	-	175.0	Piezometer

Table 1
General Boring Information

The drilling program began in mid-summer of 1994. The borings were performed by Ohio Test-Bor, Inc. of Parma, Ohio using two drill rigs. A Senior Project Geologist or a Staff Engineer was assigned to each drill rig to provide overall supervision of drilling, log and preserve samples and install instrumentation.

The borings were performed with a truck-mounted drill rig and advanced through the top 30 to 60 feet of granular soil using a 3¼-inch I.D. hollow-stem auger. At regular intervals, soil samples were obtained by lowering a 2-inch O.D. split-barrel sampler to the desired sampling depth, and then driving the sampler into the soil at depth with blows from a 140-pound hammer freely falling 30 inches.

At a depth where consistent cohesive soil was encountered the hollow-stem augers were removed and replaced with flush-coupled 4-inch I.D. casing. This casing remained in place to prevent the granular soil from caving into the hole while the boring was being advanced. The borings were then advanced through the soil using a 3⅞-inch rotary bit that used recirculated water as drilling fluid. At desired depths the soil samples were obtained by removing the rotary bit and lowering the 2-inch O.D. split-barrel sampler to depth and obtaining a sample. Numerous undisturbed soil samples were obtained by hydraulically pressing a 3-inch O.D. thin-walled Shelby tube into the soil at the desired depth. These samples were subjected to shear strength testing performed in our soils laboratory in Dublin, Ohio. In Borings B-101 through B-110, 5.0 feet of bedrock were cored, with an NXM double-tube core barrel and a diamond bit, using recirculated water as a circulating/cooling fluid. Bedrock cores were preserved for further testing in the laboratory.

Slope Inclinometers

Borings B-101 through B-110 were fitted with 2-¾ inch I.D. slope inclinometer casing (SINCO Part No. 51101100) for the entire depth of the boring. A quick connect grout valve (SINCO Part No. 51106200) was joined to the bottom of the first piece of casing and following ten foot pieces were joined together as the casing was placed in the boring. The ten foot sections were coupled with a standard flush coupling (SINCO Part No. 51101200) that was glued, riveted with four rivets per casing piece and duct taped at each coupling joint. The bore hole was filled with drilling fluid (recirculated water) therefore the water tight casing was buoyant. A grout tremie pipe, which consisted of ten foot sections of ¾ inch I.D. galvanized pipe was placed inside the casing to provide the additional weight necessary to submerge the casing. Once the entire length of the slope inclinometer casing was joined and placed in the borehole, one set of inner groves of the casing were aligned in a plane parallel to the centerline of the right-of-way. With the casing held in place by the tremie pipes, an 1:4 cement/bentonite grout mixture was pumped through the tremie pipe and grout valve, and filled the annular space in the boring. The tremie was removed and the inclinometer casing was flushed with clear water to remove any grout debris.

Piezometers

In Borings P-1 through P-5 pneumatic pore pressures transducers were installed so pore water pressure could be monitored at strategic levels in the soil stratum. Each transducer (SINCO Part No. 51417800) was set within 5 feet of No.5 sand that was sealed at each end with 2 to 3 feet bentonite hole plug or pellets. In nested applications the space between the transducers was filled with either bentonite hole plug or an 1:4

cement/bentonite grout mixture. Figure 5 is a schematic subsurface section that shows the location and depths at which the pneumatic pore pressure transducers were installed.

The drilling portion of the field work took 7 weeks using 2 rigs to drill 2621 feet of boreholes and install 1830 feet of slope inclinometers and 12 piezometers. All instruments were tested immediately upon installation and were found to function properly. The instruments were protected with a locking steel protective casing, and where appropriate, concrete filled steel pipe bumper posts.

Laboratory Testing

In the laboratory, split-barrel soil samples were visually identified and on selected representative samples natural moisture content, Atterburg limits, sieve and short hydrometer tests were performed. Undisturbed soil samples were subjected to strength and compressibility tests. Seventeen 3-point triaxial shear strength tests were performed on the selected samples. Fourteen series were conducted on samples obtained from the lacustrine clay strata.

The lacustrine clay is an "A-6a" or "A-6b" material according to the Highway Research Board (HRB) classification used by ODOT. The Plastic Indices generally range from 10 to 20 percent and Liquid Limits from 25 to 42 percent with typical values in the low 30's. Percent fines (passing the #200 sieve) range from 90 to 100 percent with clay contents from 30 to 40 percent. Shear tests were either triaxial or direct shear tests performed on consolidated samples with pore pressure measurements during strain controlled shearing (CD w/pp). The test were typically performed at a confining stress of ½, 1 and 2 times the existing effective overburden pressures. The samples were sheared to a strain of at least 20 percent. The resultant strength envelopes were developed for a residual strength condition, in general at 20 percent strain. Evidence that the bridge piers have sustained significant movement over many years suggests the use of drained residual shear strength parameters in the stability analysis. Effective residual shear strength parameters of $c' = 0$ and ϕ' ranging from 19° to 23° were used in the stability analyses. No relationship between depth of sample and friction angle was noted. The range of consolidated drained failure envelopes are plotted on Figure 4.

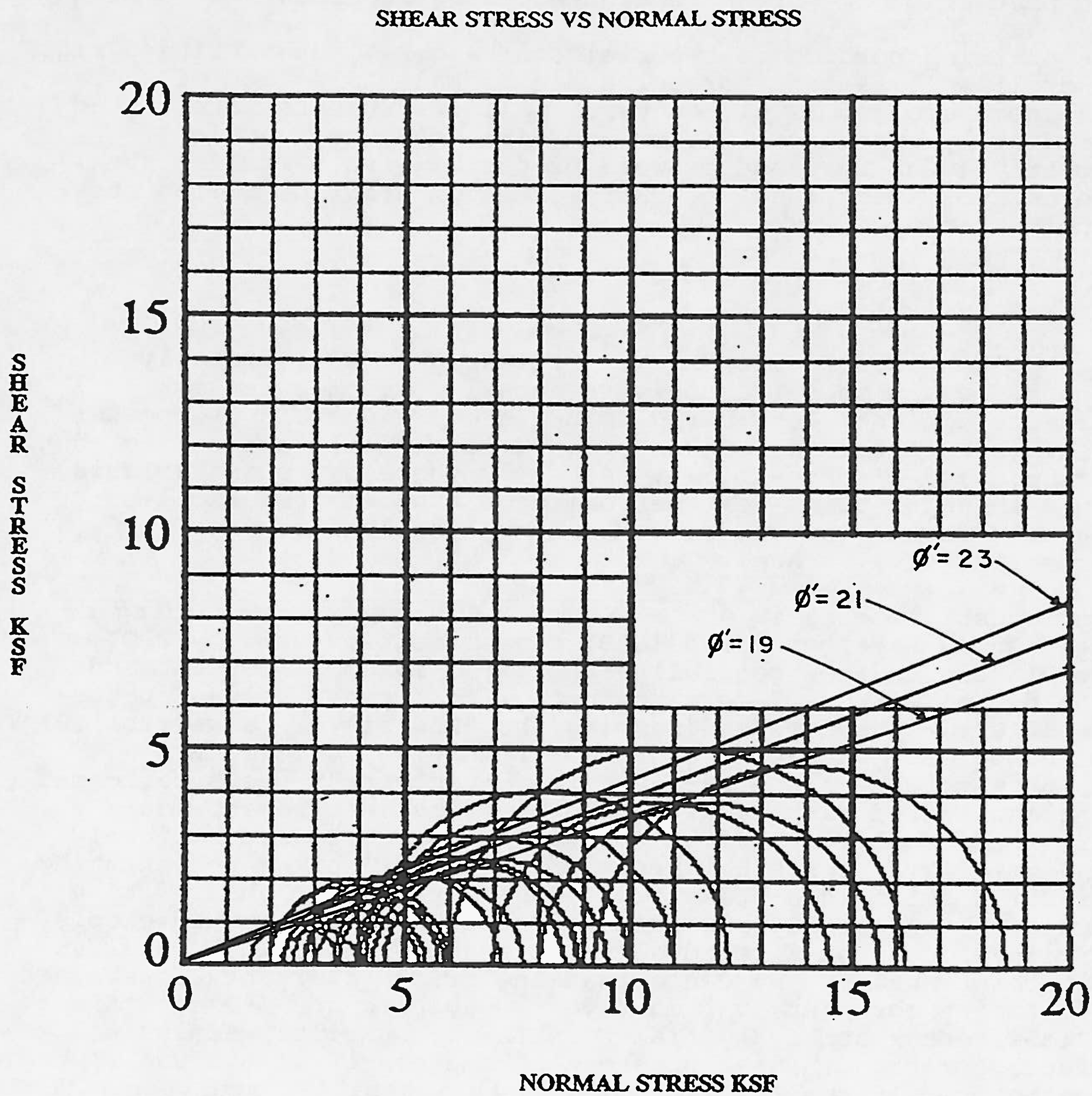


Figure 4
Summary of Triaxial Compression Tests
Lacustrine Clay - Residual Strengths

General Subsurface Conditions

In general the soils encountered during the boring program verify the geologic literature. For the most part, the borings encountered miscellaneous fill over 30 to 60 feet of sand and gravel. Below the granular material all borings revealed the lacustrine clay deposits consisting of 130 to 155 feet of very-stiff to hard gray clay with fine horizontal seams of silt. All borings revealed a 5 to 10-foot thick layer of dense silt lying within the gray laminated clay at about 20 to 30 feet above the bedrock. Bedrock consisting of soft to medium-hard shale was encountered in Borings B-101 through B-110 at about Elevation 450. The schematic subsurface section in Figure 5 depicts, generally the stratigraphy encountered at the centerline and the locations of the instrumentation.

STABILITY ANALYSIS AND DATA COLLECTION

Introduction

The data on the site specific stratigraphy and soil strengths from the field drilling program, combined with the empirical field measurements of pore water pressures and lateral slope movements provide two independent but complimentary approaches to defining the location of the probable failure plane. In addition, the stability analysis would provide a theoretical factor of safety against sliding; while the detection of movement by inclinometers would indicate a factor of safety either greater than or less than one.

Stability Analysis

The stability analysis performed earlier could now be updated with the results of our laboratory testing program. The soil profile model is shown in Figure 5.

Parametric studies were performed to model the range of input variables and loading conditions. These parameters included; lacustrine clay strength values, slope section, aggregated pile loading and search limits.

Table 2 is brief summary of the **factor of safety** against sliding at the centerline of the bridge. Acceptable factors of safety for long term slope stability are typically 1.3 to 1.5. As shown in Table 2, none of the calculated factors of safety are inadequate for long term stability. The factors of safety for the lower slope (Pier 1) indicate either movement or incipient movement is probable.

Slope	Centerline Factors of Safety		
	Existing Grade w/ agg piles	Existing Grade w/o agg piles	Proposed Regrading
Entire	1.17	1.20	1.17
West End Pier	1.05	1.05	1.27
Pier No. 1	0.85	0.85	0.90

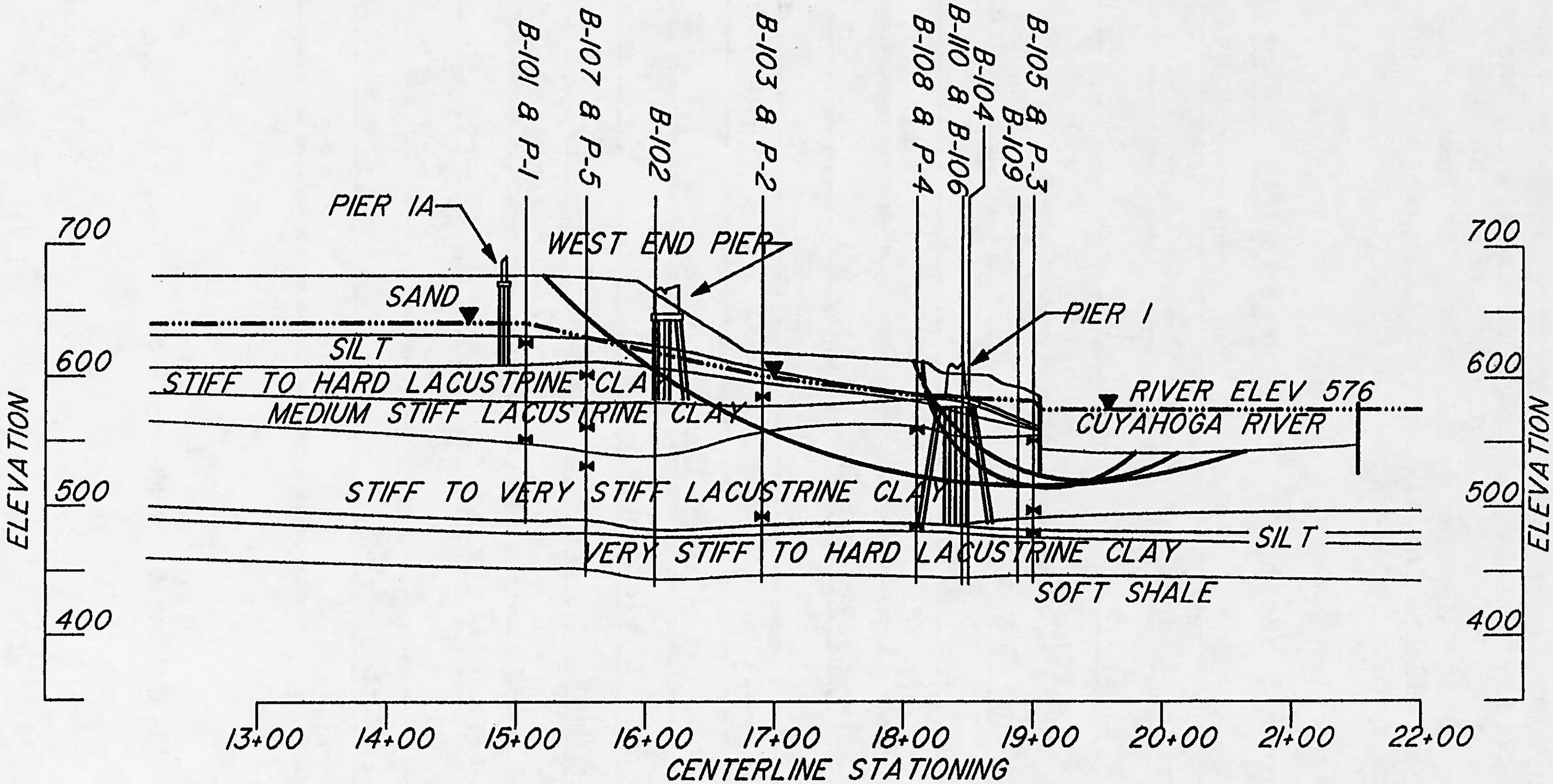
Table 2
Summary of Factors of Safety

Instrumentation Data

The ten inclinometers and twelve piezometers were monitored every two months over a one year period, beginning October 1994 and ending August 1995, for a total of six readings. Each site visit required about two days of field work and one day of data reduction.

Strict quality assurance procedures were utilized to provide quality control for the field data. These methods were necessary due to the slow rate of expected movements and the length of the monitoring period. Additionally, the project site was 100 miles from our office and it would have been impractical and costly to return to collect new readings if the data was not considered accurate after reduction.

The slope inclinometer probe was calibrated by the manufacturer immediately prior to the performance of the initial readings. All site visits were performed by the same Staff Engineer using the same monitoring equipment and measurements methods to reduce the risk of operator and procedure error. Additionally, the instrument measurements were performed twice during each site visit to provide a quality assurance of the data and the operator. Checks on data quality were performed by down loading the readings from the electronic data collector to a lap top PC and reducing the data before leaving the site.



NOTE: ⧓ DEPICTS PIEZOMETER INSTALLATIONS

FIGURE 5
SCHEMATIC SUBSURFACE SECTION CENTERLINE PROFILE

Piezometers

Borings P-1 through P-5 have a total of 12 pneumatic pore water pressures transducers installed at various levels in the soil strata. The bi-monthly readings indicated that the main groundwater table exists approximately 15 to 50 feet below the ground surface and follows the topographic gradient of the slope. Table 3 shows some typical results.

Transducer and depth	Water Surface Elevation December, 1994
P-1 @ Ground Surface Elevation 676.6	
A @ 50.5(ft)	641
B @ 124.3(ft)	624
P-2 @ Ground Surface Elevation 616.3	
A @ 30(ft)	607
B @ 123.5(ft)	590
P-3 @ Ground Surface Elevation 585.6	
A @ 38(ft)	579
B @ 88(ft)	585
C @ 106(ft)	589
P-4 @ Ground Surface Elevation 610.1	
A @ 50(ft)	586
B @ 125(ft)	589
P-5 @ Ground Surface Elevation 662.3	
A @ 60(ft)	630
B @ 100(ft)	618
C @ 170(ft)	591

Table 3
Typical Results
Piezometer Measurements

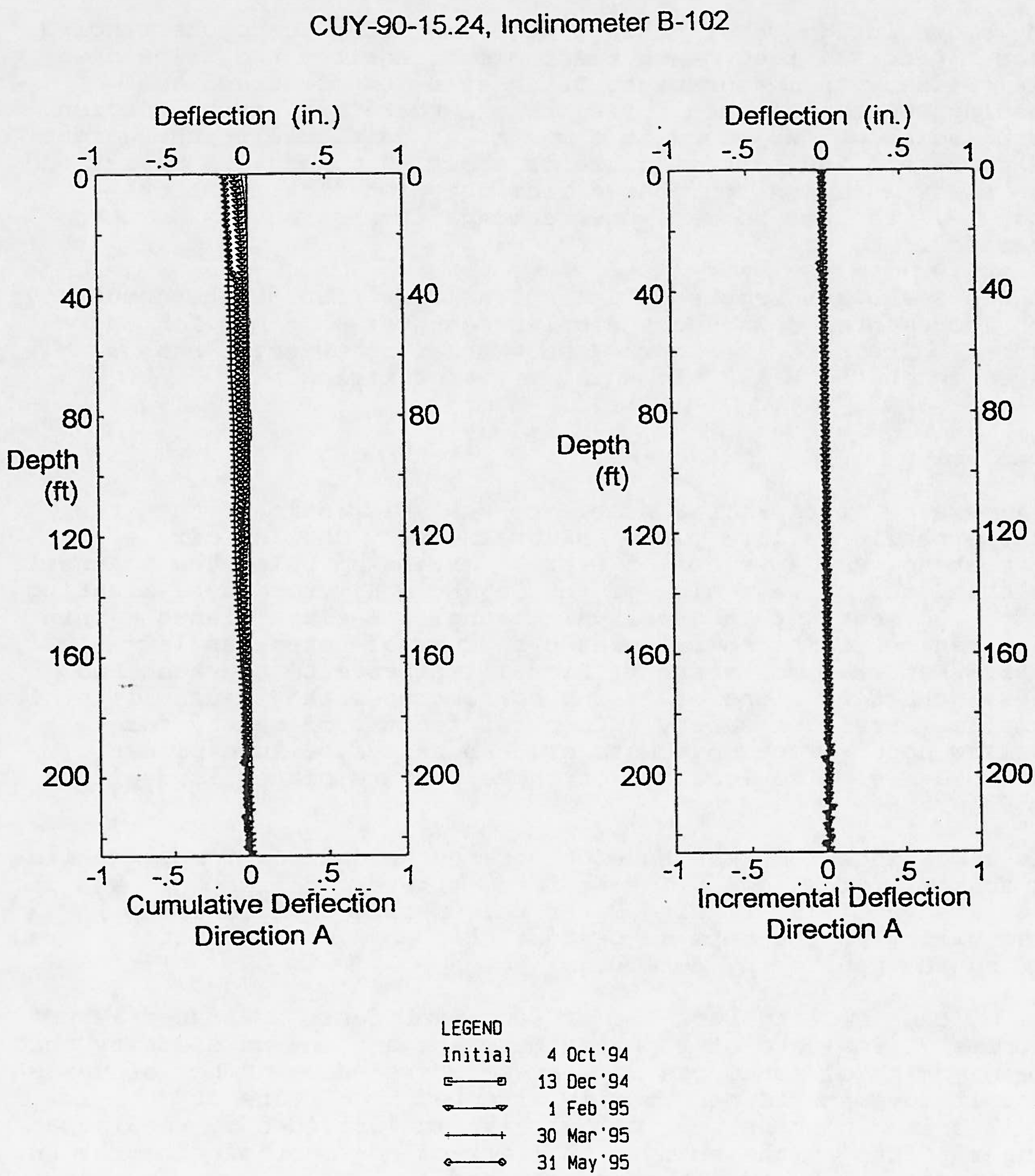


Figure 6
Typical Inclnometer Plot

Slope Inclinometers

Ten slope inclinometer casings were installed to depths ranging from 150 to 232 feet, each reading set required two trips down the casing with measurements taken every two feet and two readings were performed for each hole therefore, each location required about two hours to complete. Additionally, the weight of probe and 200 feet of cable is about 50 pounds. A pulley and cam cleat assembly, purchased from The Slope Inclinometer Company, was used to make the readings less strenuous on the operator.

Figure 6 shows a sample of an inclinometer plot of the cumulative and incremental down slope displacement verses depth for one casing. The plot in Figure 7 shows that no movement has been revealed in the B-202 slope inclinometer casing.

Conclusion

The results of the slope stability analysis indicate that the most probable failure plane is a relatively shallow circle initiating just up slope of Pier 1, extending below the sheetpile bulkhead on the west side of the Cuyahoga River, and terminating 70 to 100 feet within the river channel. Failure planes within this region exhibited calculated factors of safety as low as 0.85. Reported movements of Pier 1, relative to the West End Pier located up slope of Pier 1 corroborates the plausibility of this low factor of safety. Computed factors of safety for shallow upper slope movements or deep seated failure planes exceeded 1.0. The location of these failure planes is depicted on Figure 5.

The slope inclinometers have detected no significant movement in 12 months of readings. The number of these inclinometers and the consistency of the readings over this period of time lends credibility to the data suggesting that movement has not occurred during the monitoring period.

One of the challenges of the project previously indicated is the extremely slow rate of reported movement and the possibility that the majority of movements may have occurred some number of years ago. If movement is temporarily arrested by a state of equilibrium in the slope, or by restraint provided by the locked finger joints in the bridge, the lack of movement detected by the inclinometers should not be unexpected.

ODOT is planning to remove the aggregate piles and regrade the slope which will provide modest improvements in stability. Additional monitoring may continue to provide information on lateral deflection of the slope. If stabilization of the slope or piers is undertaken, the investigation and analysis also provides a determination of lateral loads for designing the stabilization system for the slope or new pier foundations. That part of the investigation is beyond the scope of this paper.

This investigation has provided a significant body of information on the lacustrine clays which are prevalent in the Cuyahoga River Valley in Cleveland. The information presented is a useful starting point for investigating similar slide related problems known to exist in the area.

REFERENCES

1. Soller, D.R. (1986). "Preliminary Map Showing the Thickness of Glacial Deposits in Ohio" U.S.G.S.

2. White, G.W. (1982). "Glacial Geology of Northeastern Ohio", Ohio Department of Natural Resources.

3. Ford, J. (1987). "Glacial and Surficial Geology of Cuyahoga County, Ohio", Ohio Department of Natural Resources.

4. Peck, R.B. (1954). "Foundation Conditions in the Cuyahoga River Valley", Proceedings, Volume 80, No. 513, American Society of Civil Engineers, October.

5. Duncan, M.J. and Stark, T.D. (1992). "Soil Strengths from Back Analysis of Slope Failures", Geotechnical Special Publication No. 31, American Society of Civil Engineers.

LABORATORY AND FIELD MEASUREMENTS OF COAL REFUSE PROPERTIES

Mohamed M. Nofal, PE
Ogden Environmental and Energy Services, Oak Ridge, TN 37830

R. Michael Holbrook, PE
Ogden Environmental and Energy Services, Oak Ridge, TN 37830

Abstract: Field and laboratory measurements of consolidation and shear wave velocity characteristics are presented for fine-grained coal refuse materials. Comparisons are made between the coefficient of consolidation, c_v, as measured from laboratory consolidation data and c_v estimated from long-term field monitoring measurements using pneumatic piezometers. Further, a relationship is proposed between Standard Penetration Test N-values and in-situ measurements of shear wave velocity. The purpose for presenting these relationships is to begin the development of a database to evaluate time rates of consolidation and shear wave velocity characteristics of coal refuse deposits from routine laboratory and field tests (i.e., laboratory consolidation testing and field Standard Penetration Testing).

INTRODUCTION

It is estimated that the coal industry in the United States produces more than 250 million tons of coal refuse (rejects) every year (Roman, 1995). These rejects are commonly produced in two forms:

1. coarse coal refuse which consists primarily of shale fragments; and

2. fine coal refuse, which consists primarily of fine-grained sand, silt, clay, and coal particles.

Due to the large quantities of coal refuse produced, waste disposal facilities built to accommodate these materials are often constructed relatively quickly. In some cases, embankments are built using the coarse refuse to create an impoundment for the disposal of the fine refuse. In steep terrain, these waste disposal impoundments are often built by the upstream method whereby fine refuse from a previous stage serves as a foundation material for subsequent stages of construction. In other cases, the coarse refuse and fine refuse are combined in a waste disposal fill. The engineering properties of the fine refuse are therefore important in the design of waste impoundments built by the upstream method and in combined refuse disposal fills.

Designers of these types of coal waste disposal facilities require data on the shear strength of the fine refuse for both static and dynamic loading conditions. For static conditions, time rate of consolidation characteristics of the fine and combined refuse are required to estimate excess pore water pressures that will exist during construction. Stability analyses can then be performed for effective stress conditions by subtracting the estimated excess pore pressures from the total overburden pressures for various stages of construction. Typically, the coefficient of consolidation, c_v, is used to estimate the time rate of consolidation characteristics and therefore excess pore pressures that exist for a given fine refuse deposit.

For earthquake loading conditions, the shear wave velocity of the fine refuse is an important characteristic in evaluating the liquefaction potential of a given fine refuse material. Generally, the higher the shear wave velocity of a given material, the more resistant that material will be to liquefaction during an earthquake. Moreover, for a given deposit, fine refuse with complete pore pressure dissipation will be more resistant to liquefaction than a deposit where excess pore pressures are still present.

In cases where unacceptable performance is predicted during design, mitigative measures can be incorporated to increase the rate of consolidation for a given fine refuse deposit. One effective mitigative measure involves the use of wick drains (Thacker, 1983). The wicks are used as horizontal drains within the fine refuse deposit to enhance pore water pressure dissipation and accelerate the consolidation of the fine refuse. However, there is a lack of published data on the coefficient of consolidation and shear wave velocity of fine refuse and the development of corresponding empirical relationships involving these parameters.

Presented herein are the results of tests performed on fine refuse and a mixture of fine refuse and coarse refuse to estimate the engineering characteristics delineated by the coefficient of consolidation and shear wave velocity. In the case of the coefficient of consolidation, comparisons are made between values measured in the laboratory and values calculated from field monitoring of construction pore pressures. In the case of shear wave velocity, a correlation is presented between field measured values and Standard Penetration Test data.

PHYSICAL PROPERTIES

General

Field and laboratory testing was performed on samples of fine refuse from three sites and a mixture of fine refuse and coarse refuse from one site. Results of the testing are presented in the following sections.

Index Properties

The fine-grained coal refuse samples have Unified Soil Classification system (USCS) classifications of SM, ML, and CL. Samples of mixed fine and coarse refuse ranged from GM to ML. Natural moisture contents ranged from about 10 to 30 percent. The plasticity index (PI) ranged from 5 to 15 percent and the specific gravities of these materials ranged between 1.6 and 2.0.

Shear Strength

The fine refuse materials have an effective angle of internal friction which ranges between 28 and 32 degrees. The effective cohesion is essentially zero.

CONSOLIDATION CHARACTERISTICS

The coefficient of consolidation, c_v, was calculated for the consolidation phase of triaxial compression test for the various fine refuse materials. The coefficient of consolidation calculated from laboratory data ranged from 2.0×10^{-3} to 4.4×10^{-3} in^2/sec. As would be expected, the higher value of c_v, was measured for the sample obtained from the site where the fine refuse was combined with coarse refuse during disposal.

Pneumatic piezometers were installed and monitored during the construction of a combined coal refuse embankment. The piezometers were used to measure the pore pressure development during construction, and compare the actual field results with laboratory data obtained from this site. As the height of the fill increased, the piezometers showed an increase in pore pressure. Upon completion of fill placement in the area above the piezometer, the pore pressures started to dissipate as shown in Figure 1. The value of c_v estimated from the field data with an estimated drainage distance of 80 feet is 4.3×10^{-2} in^2/sec.

SHEAR WAVE VELOCITY

Shear wave velocity tests were performed at several coal refuse disposal facilities. The cross borehole method was used where three holes were drilled at each site, one of which was sampled using the Standard Penetration Test. A PVC pipe was installed and grouted in each hole. A vertical survey was performed using an inclinometer to find the exact horizontal distance between the holes at each elevation. The shear wave velocities were determined at the same elevations where Standard Penetration Test (SPT) N-values were obtained. The shear wave velocities were correlated with the corrected blow counts as a function of depth. The results are summarized in Figures 2, 3 and 4.

DISCUSSION

The difference between the field estimated value and the value estimated from laboratory testing results could be caused by a combination of factors. The field test includes the effect of horizontal drainage, which accelerates the time rate of consolidation, while the laboratory test accounts only for the vertical drainage component. Further, the presence of larger particles within the material's matrix in the field causes the material to be more permeable, while the specimens tested in the laboratory contained more fines which reduces its permeability. It can be observed that the graph presents a full scale consolidation test estimated via measurements of pore pressure dissipation.

The shear wave velocities were plotted against the blow counts for the sites tested as shown in Figures 2, 3, and 4. A curve fitting algorithm was used to develop the following formula from the field testing:

$$\mathbf{S = 543.9 + 38.11(N_{60})}$$

Where: S = the Shear wave Velocity in feet per second,
and N_{60} = The blow counts corrected for depth.

The blow counts should be carefully evaluated to determine if their values are inflated by the presence of large particles, or deflated by unequalized water levels inside and outside the augers during sampling.

CONCLUSIONS

Conclusions obtained from the results of the testing program are presented as follows:

1. We believe that a relationship can be developed to estimate the shear wave velocity from the results of routine Standard Penetration Tests.

2. Engineering judgement is required when analyzing the SPT data to detect inflated or reduced N values.

3. At the site where both laboratory and field testing results were obtained, the laboratory estimated value of c_v provided conservative estimate of the time required for pore pressure dissipation.

4. Direct field measurements provide the most accurate representation of actual conditions. However, when time and cost are considered, routine testing results can be used to provide preliminary design and modeling parameters with an acceptable degree of confidence. Moreover, the correlations presented herein can be used to reduce the extent of direct field measurements where justified at a specific site.

REFERENCES

Holtz, R.D. and Kovacs, W.D. (1981)
An introduction to geotechnical engineering. Prentice-Hall, Inc., Englewood Cliffs, NJ.

Roman, G (1995)
Keystone Coal Industry Manual, Coal Magazine, Chicago, Ill.

Thacker, B.K. (1983)
Rapid construction of a combined coal refuse embankment. Proceedings of ORVSS XIV, pp 4-1 4-16.

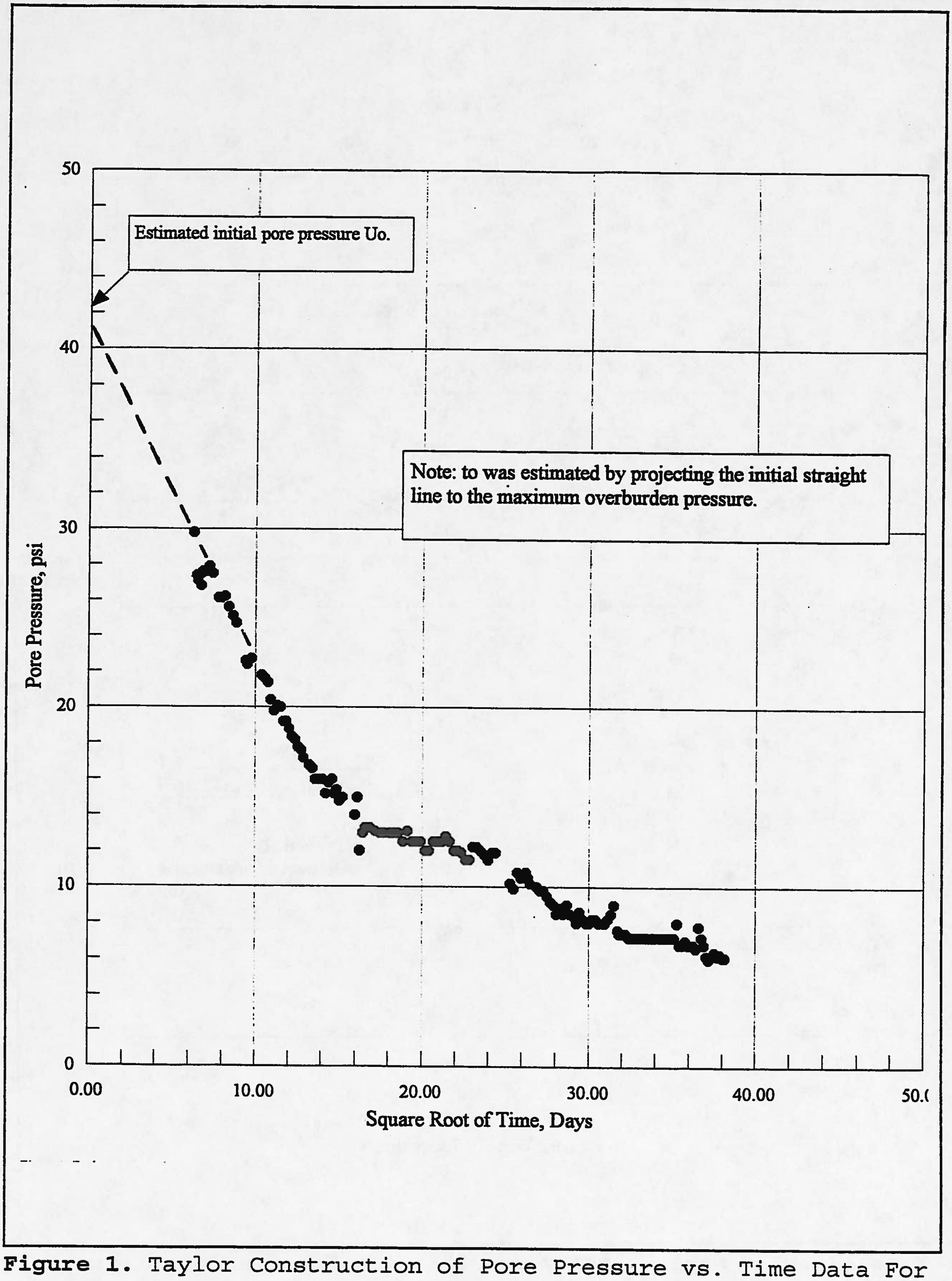


Figure 1. Taylor Construction of Pore Pressure vs. Time Data For Site D. (Combined Coarse and Fine Refuse Deposit).

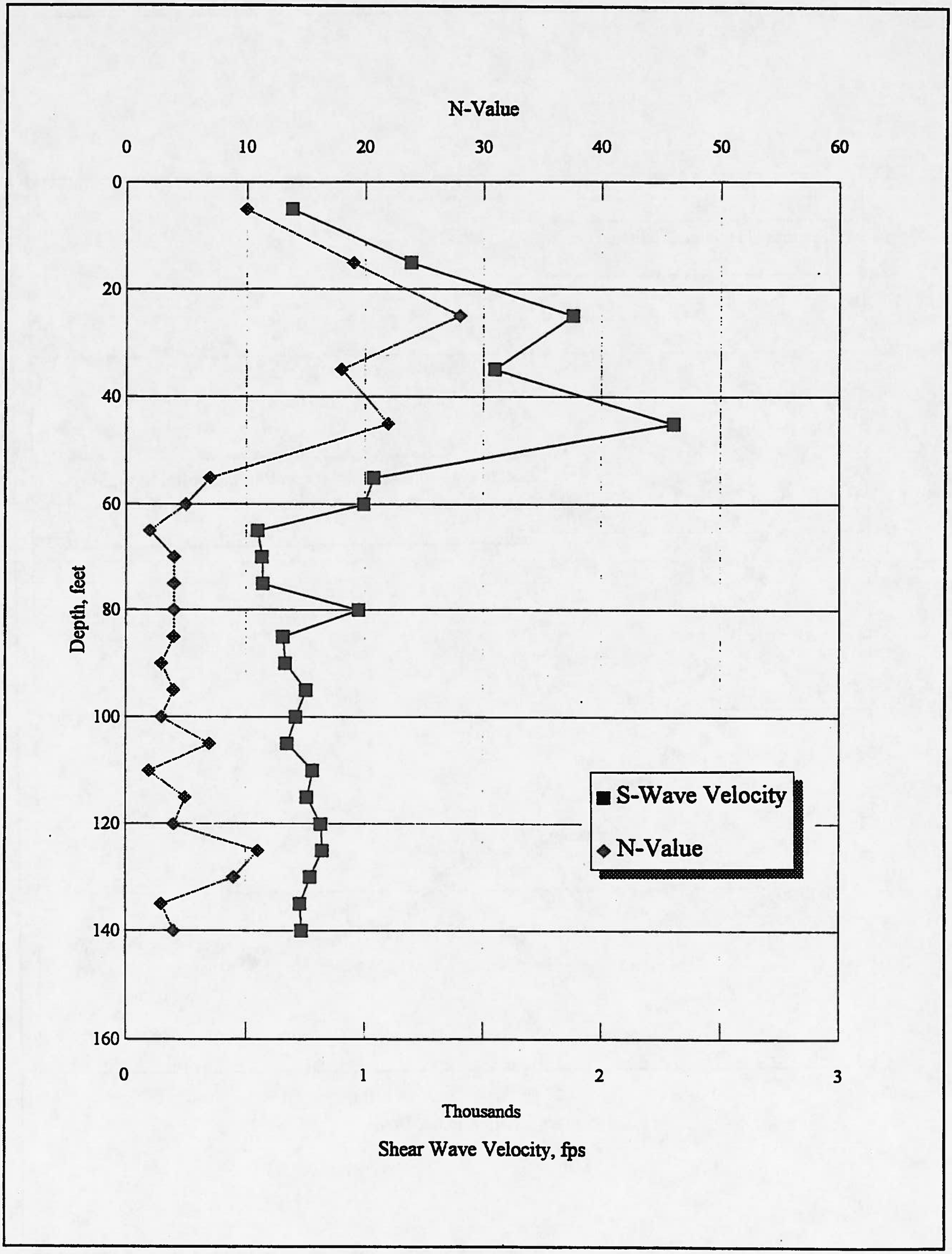


Figure 2. Shear Wave Velocity and Standard Penetration Test N-Values for Site A.

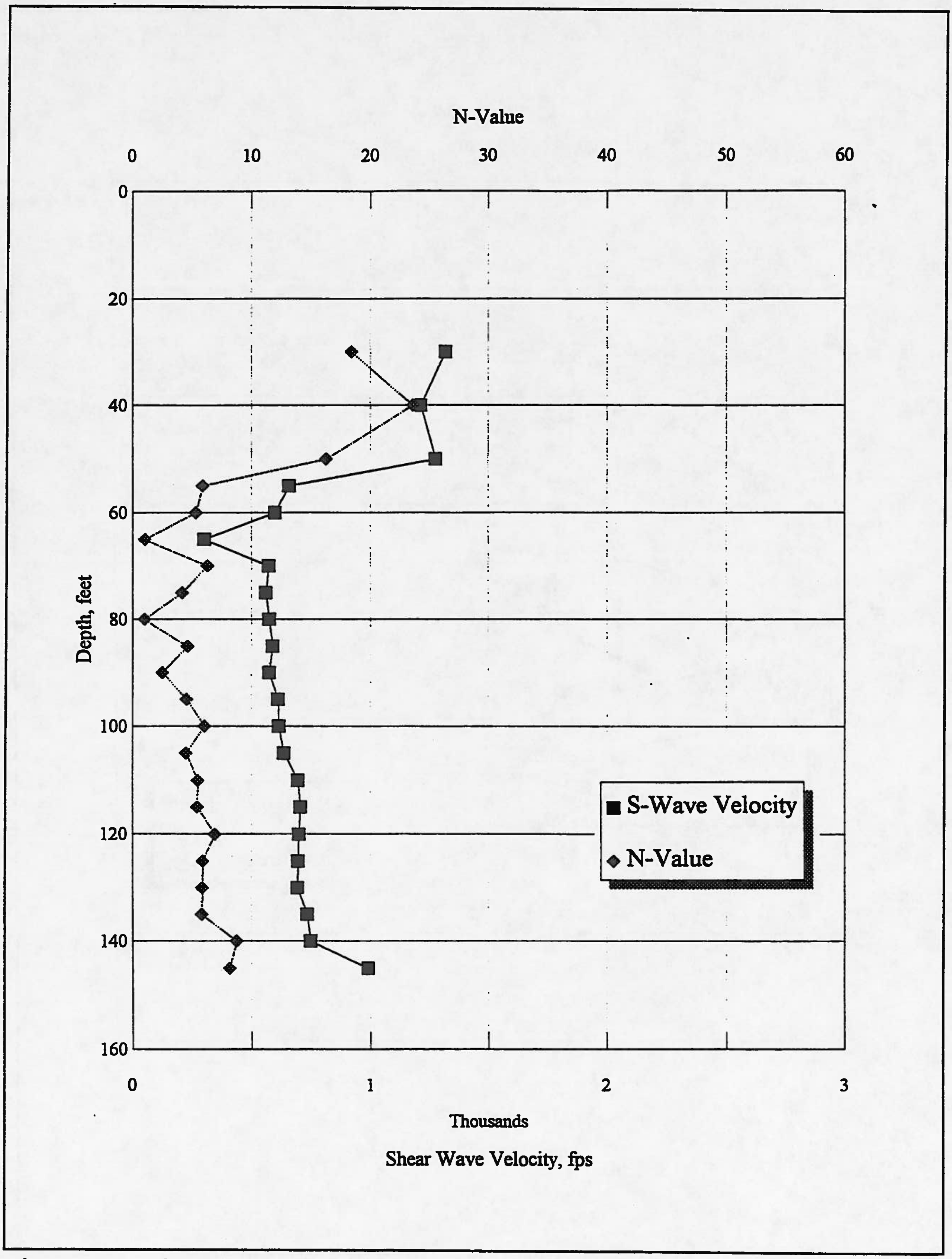


Figure 3. Shear Wave Velocity and Standard Penetration Test N-Values for site B.

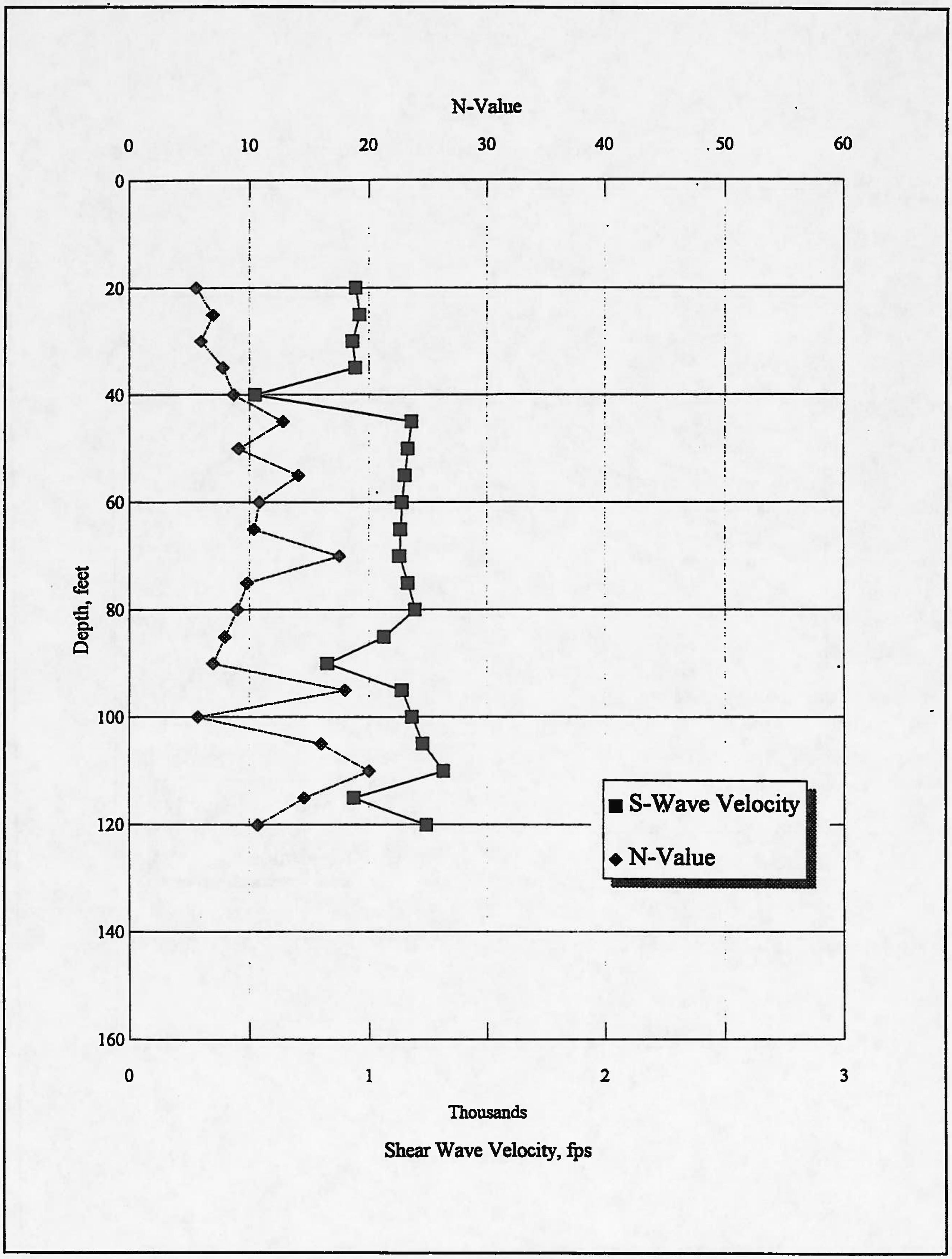


Figure 4. Shear Wave Velocity and Standard Penetration Test N-Values for Site C.

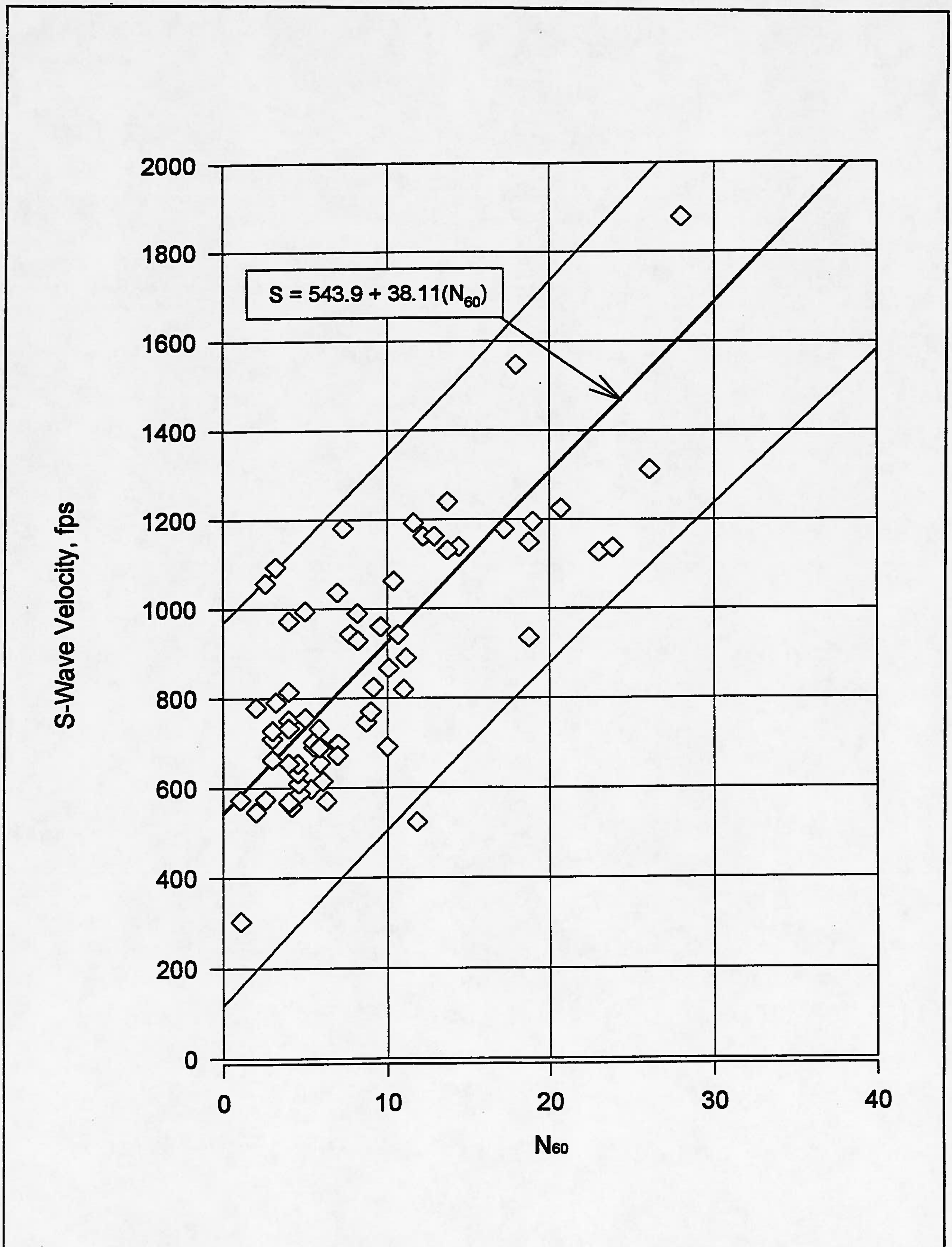


Figure 5. Correlation Limits with 95% Confidence level for Shear Wave Velocity vs. Corrected Blow Counts (N_{60})

DESIGN AND IMPLEMENTATION OF A MULTIPURPOSE GROUNDWATER MONITORING SYSTEM AT SELLAFIELD, U.K.

Chris D. Eldred - Director of Geotechnical Engineering
Law International
Sir Alexander Gibb & Partners Limited
Earley House, London Road
Reading, Berks, RG6 1BL, UK

John A. Scarrow - Managing Director
Soil Mechanics Limited
Askern Road, Carcroft
Doncaster, S. Yorks, DN6 8DG, UK

INTRODUCTION

Groundwater monitoring is a fundamental aspect of all civil engineering, environmental and development projects. The Sellafield geological investigations, currently being carried out by UK Nirex Limited with Sir Alexander Gibb & Partners Limited as management contractor, are one of the largest and most comprehensive site characterisation programmes currently being carried out in the world. A major part of the investigations have been the drilling and testing of a number of 123 mm and 156 mm diameter cored boreholes extending to depths of up to 1947 m. To date a total of 26 boreholes have been drilled by KSW Deep Exploration Group, (a Joint Venture comprising Kenting Drilling Services Limited, Soil Mechanics Limited, both of the UK, and Bohrgesellschaft Rhein Ruhr MbH of Germany), from some 19 sites and two boreholes drilled by others in 1908 and 1962 have been refurbished.

These investigations have identified a fundamental requirement for high quality multilevel groundwater monitoring data. Furthermore, where groundwater conditions are potentially complex it is vital that the maximum groundwater data is obtained from each borehole within the practical and financial constraints of the project. At Sellafield this objective has been met by utilising the Westbay MP38 and MP55 multilevel groundwater monitoring systems.

To date, twenty UPVC multipackered installations have been installed in 100 mm, 123 mm and 157 mm diameter vertical, inclined (up to 30°) and deviated (up to 36°) boreholes to depths up to 1200 metres with up to 30 separate monitoring zones within each installation. All of the installations have been equipped with strings of retrievable transducers to facilitate continuous recording of fluid pressures in each zone. In addition to the basic pressure monitoring function, the installations have been designed to allow access to individual monitoring zones for hydrogeological testing, purging and high quality groundwater sampling. The long term objective of the installations is to form a monitoring network to determine baseline environmental pressure gradients and natural changes to those pressure gradients. The majority of the installations will also make up an array of observation wells to monitor the hydraulic response of the fractured rock mass to pumping tests carried out in adjacent wells to depths of up to 1400 metres and to those induced by construction works. There are currently plans to utilise the system to carry out a tracer test which will involve injecting different tracers at various levels within the installations.

SELLAFIELD OVERVIEW

The Sellafield Geological Investigations being carried out by UK Nirex Ltd (Nirex) with Sir Alexander Gibb & Partners Ltd (GIBB) as management contractor have been ongoing for some six years. The investigations are designed to provide geological and hydrogeological data such that it can be determined whether the Sellafield site in Cumbria, England is suitable for the disposal of intermediate and certain low level radioactive waste.

The Sellafield District is an area of approximately 30 km by 20 km incorporating much of the coastal strip of West Cumbria, and including a small part of the Lake District National Park and a significant area offshore. Within this district, Nirex have defined a potential site area extending 6.5 km north-south by 8 km east-west as the area in which detailed site investigation would be carried out to provide geotechnical and hydrological data for the proposed repository, see Figure 1. Following results obtained from the regional Boreholes Nirex identified the Potential Repository Zone (PRZ) as an area for more detailed investigation.

Each borehole was partially or fully cored to depth using either 123 or 156 mm heavy duty wireline coring string. During hole construction and on completion a comprehensive programme of geophysical logging and hydrogeological testing has been carried out; which in some instances has been ongoing for over four years following completion of the drilling of the borehole.

Geology

Nirex is investigating the rocks of the Borrowdale Volcanic Group (BVG) as a potential host for the waste repository. The BVG underlies the whole of the Sellafield area outcropping to the east of the site and dipping steadily to the west where they are encountered at 1650 m depth at the coast. The rocks erupted in the Lake District in the Late Ordovician Period and in the site area are generally calc- alkaline in composition, consisting dominantly of pyroclastic deposits (Ignimbrites), with subordinate and intrusive igneous rocks (Basalts and Basaltic Andesites) and epiblastic volcanic sedimentary rocks.

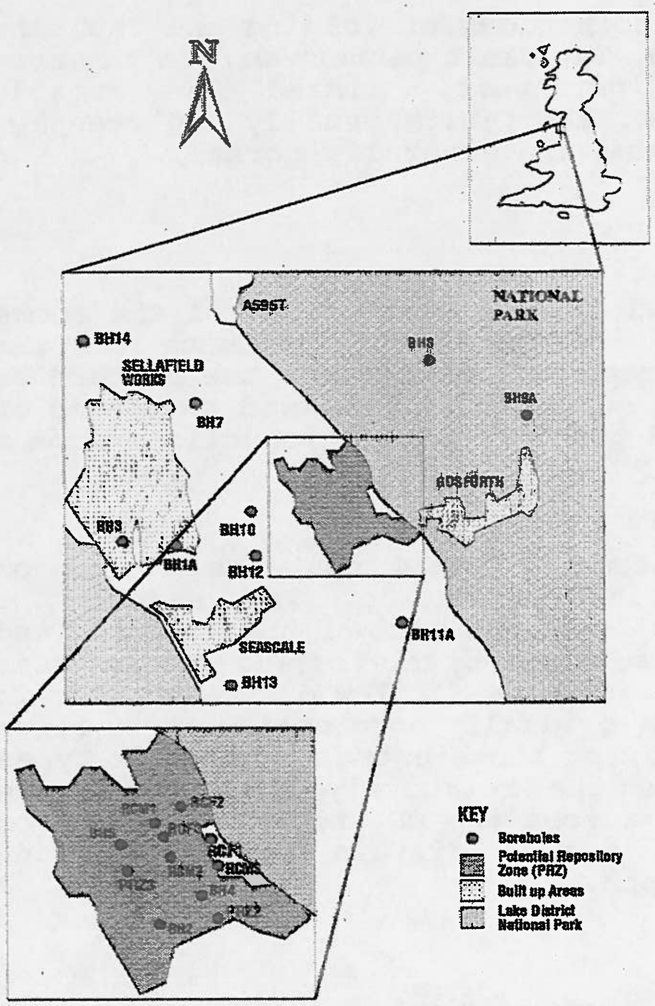


Figure 1 - Location of Site

The BVG is overlain by sandstones and mudstones of the Sherwood Sandstone Group at the base of which is a breccia known as the Brockram. To the west and north of the site the St Bees Evaporite and Carboniferous Limestone are also present. Thin glacial superficial material (clays, sands and gravels) overlie virtually the whole area.

Two major units within the Sherwood Sandstone Group (SSG) are the Calder Sandstone (CS) and the St Bees Sandstone (SBS), both of early Triassic age. The Calder Sandstone is a dominantly cross-bedded, reddish brown, fine to coarse grained sandstone and represents an aeolian environment, dominated by dune and interdune facies. The rocks of the St Bess Sandstone comprise quartz rich sandstones and secondary siltstones and claystones believed to have been deposited in an alluvial plain and braided river environment. In the site area the SBS is generally over 150 m thick but maybe up to 1000 m thick regionally.

Underlying the SSG in the site area the Brockram comprises mostly breccias containing almost exclusively clasts of extrusive igneous rocks derived from the BVG. These generally massive rocks are of Permian age and represent alluvial fan deposits flanking the steep mountains of the Lake District. Most of the breccia is poorly sorted and has a reddish brown, silty fine grained sandstone to granule and small pebble matrix which is weakly to moderately calcareous.

A schematic geological WSW-ENE cross section through the Sellafield area is presented as Figure. 2.

The rocks have been subjected to periods of folding and faulting during their geological history. The fault pattern within the area trends between north and northwest, linked by broadly east-north-east trending faults. The faults usually dip steeply, ranging from vertical to 60°, and are generally normal.

Hydrogeology

The definition of the geological units and structure of the rocks forms a framework which has an important influence on the hydrogeology of the site. The hydrogeology defines the pattern of groundwater flow beneath the site, which is assessed to be one of the mechanisms in the potential transport of radionuclides from a repository back to the surface.

The Sellafield District groundwater system contains waters of three salinities, fresh, saline and brine. In general the groundwater recharges in the hills to the east of the District and flows westwards to discharge into streams, rivers and the sea near the coast. Where the BVG crops out, initial flow of fresh water is downwards and westwards becoming gradually more saline as it flows from outcrop through the BVG. Mixing zones between the water types vary considerably ranging from the relatively sharp interface between fresh and saline water across the CS and SBS junction or near the base of the SBS to a more diffuse transition zone interface within the BVG at depth.

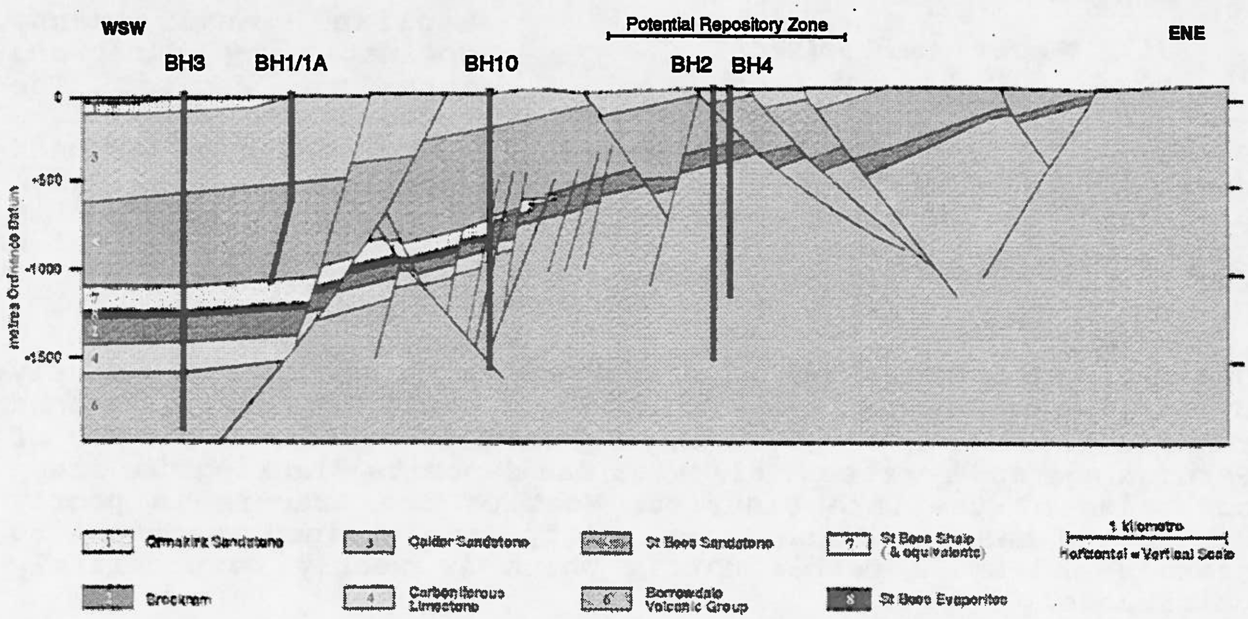


Figure 2 - Schematic Geological WSW - ENE Cross Section

The groundwater at depth within the BVG is saline, but is not related to present day seawater. Within the PRZ the water has a salinity level similar to that of sea water, whilst further west the salinity levels are up to six times that of sea water. Saline water is more dense than fresh water and so, in order to assess the possible driving force for a vertical component of flow, the measured groundwater pressures must be converted to 'environmental' pressures which take account of the influence of varying density.

The conductivity values within the BVG are typically low with half the values measured over 50 m lengths in the boreholes being less that 1×10^{-10} ms^{-1}. Conductivity values in the CS, SBS are higher than the BVG with median values of 1×10^{-7} ms^{-1} measured over 50 m lengths of borehole, whilst those measured within the Brockram are variable ranging from 10^{-7} to 10^{-10} ms^{-1}.

The available information indicates that the flow of water through the BVG is likely to be controlled by the presence of fractures in the rock. A similar pattern is noted for the lower part of the sandstone formations, whereas in the upper more conductive part of the sandstone sequence, the laboratory and field values are very similar, indicating that the rock is behaving as a porous medium and the groundwater flow is not primarily through fractures.

OBJECTIVES OF MONITORING SYSTEM

From the foregoing overview it can be appreciated that accurate measurements of 'environmental' pressure is fundamental to assessing the suitability of the site. On completion of the drilling and borehole testing programme, it is therefore a key objective of Nirex's site characterisation programme, to install a series of groundwater monitoring systems which will provide:-

1. baseline pressures within the rockmass, which can be converted to environmental pressure, given knowledge of the fluid column.

2. the ability to identify and characterise natural changes in groundwater pressure.

3. the ability to assess the impact of interborehole hydrogeological testing and identify hydraulic interference paths within the rockmass.

4. a network of monitoring boreholes near to the proposed location of the proposed underground laboratory, the Rock Characterisation Facility (RCF) at some 735 m depth in order to determine baseline groundwater conditions prior to construction of the RCF.

5. subsequently to monitor the impact of shaft sinking on the hydrogeology.

Subsequently the possibility of using the installed monitoring system to carry out controlled tracer injection testing has been identified. As can be appreciated from the above the monitoring systems are required to provide data to meet a variety of objectives and must operate, in the longterm, in circumstances of hostile saline conditions, proximal to construction contractors activities which will include blasting operations, and withstand large differential pressures due to dewatering during construction.

In order to meet the programme objectives alternative commercially available and bespoke monitoring systems were evaluated by Nirex. Based on technical and cost-benefit assessments the Westbay MP System offered by Soil Mechanics Limited was selected as the most appropriate system which most fully met the objectives, although it was acknowledged that in terms of these objectives the installations would be required to be installed to depths outside previous experience.

WESTBAY SYSTEM OVERVIEW

System Design

The Westbay MP System is a multi-level groundwater monitoring system utilising a closed access casing comprised of modular casing sections, packers and valved port couplings. The valved port couplings are used to provide a hydraulic connection through the casing for access by proprietary pressure monitoring and groundwater sampling tools. The modular nature of the system permits as many monitoring zones as desired to be established in a borehole, commensurate with strength constraints, with complete flexibility in the configuration of the system during the design and installation stages.

A diagrammatical representation of a Westbay system is presented in Figure 3.

Theoretically the number of monitoring zones is only dictated by the minimum length of the individual system components. However, based on depth and strength criteria and cost-benefit considerations, the maximum number of zones achieved at Sellafield is 30 across the 800 m exposed section of a 1200 m deep borehole. The casing components can be manufactured either from unchlorinated polyvinyl chloride (UPVC) or chlorinated polyvinyl chloride (CPVC) or in special circumstances of great depth and/or

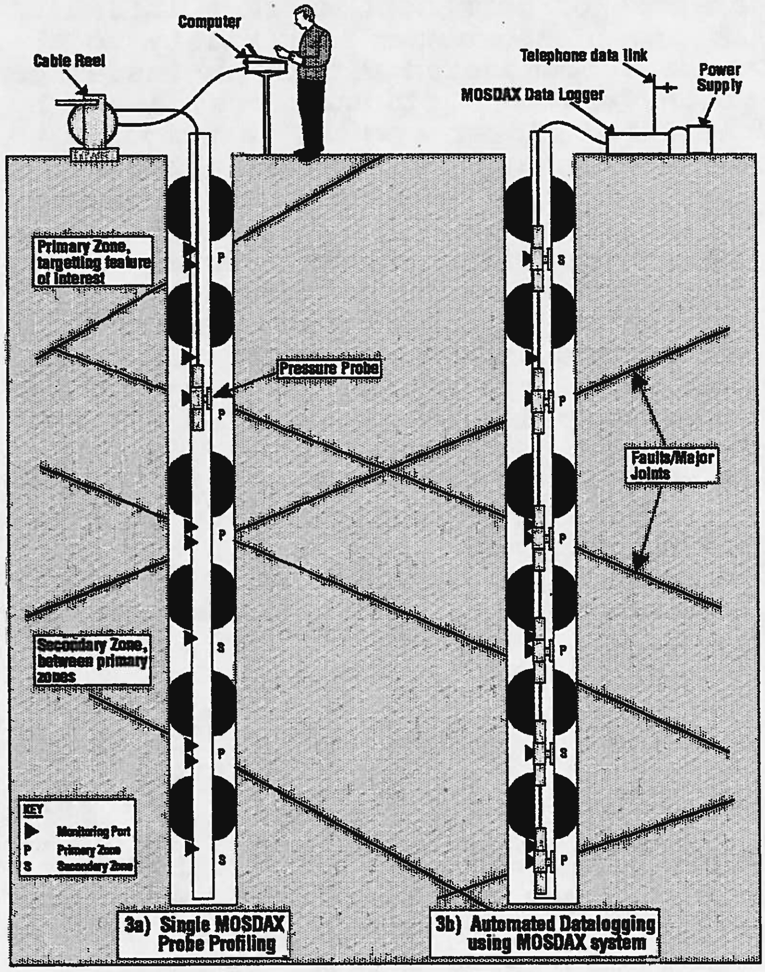


Figure 3 - The Westbay MP System

aggressive groundwater conditions, stainless steel. There are two sizes of plastic casing systems MP55, which is nominally 80 mm OD 55 mm ID, and MP38, which is nominally 48 mm OD and 38 mm ID. For the Sellafield environment consideration was given to both the plastic and steel system.

Based upon cost-benefit and technical considerations, a decision was made to proceed with predominantly the MP55 sizes of plastic systems which to date has been installed successfully to depths of 1200 m in a vertical borehole, in deviated boreholes (up to 36° from vertical) and to lesser depths in inclined boreholes (30° from vertical).

Two principle methods of monitoring the environmental pressure within the Westbay MP system are single probe profiling and automatic multi-probe monitoring. Groundwater pressure measurements can be made at each location where a measurement port coupling has been installed either by manually locating a single Mosdax probe (Figure 3A) at each required monitoring port in turn, or by installation of multiple probe Mosdax string linked to a datalogger on surface (Figure 3B). Single probe monitoring is used initially at Sellafield prior to installing a multiple probe string in every borehole. As a consequence of the probes design temperature measurements are taken along with pressure measurements.

System advantages

The modular nature of the casing system permits complete flexibility in the final string design and allows relatively low cost redundancy within the system. At Sellafield duplicate measurement port couplings were installed in zones of particular interest with an extremely modest cost increase to the final price of system components. The absence of permanent surface inflation lines leading to each packer provides greater flexibility to the numbers of packers that can be accommodated within an installed system and also eliminates zone pressure fluctuations caused by compliance of some surface inflated packer systems in response to changing surface barometric pressure and temperature changes.

A further key advantage to the system is that the modular design of the Mosdax string allows for easy retrieval and reconfiguration of individual strings to meet revised project requirements. The ease of retrieval of individual Mosdax strings also allows the opportunity for each Mosdax probe transducer to be function checked on surface against a known pressure standard as and when required. This was a fundamental requirement at Sellafield to enable the stringent quality standards for the project to be met. Further quality checking of individual Mosdax probe transducers can also be carried out at any time by simply deactivating the probe from its allocated measurement port and measuring the internal fluid column of fresh water within the Westbay casing string.

A particular and possibly unique cost saving aspect of the system is that it can be installed by floatation from ground level, where the groundwater is at or near surface, without the use of a work over rig, to depths of approximately 700 m. Below this depth, or where groundwater is relatively low, casing strings are installed generally using a small winch or crane. At Sellafield, where the groundwater is near surface, loads of up to 900 kgs have been recorded whilst handling casing strings up to 1200 m long.

Mosdax Probe Transducer Accuracy and Precision

The transducers installed in the Mosdax probes incorporate silicon strain gauge transducers with an accuracy of ±0.1% Full Scale CNLRH (Combined NonLinearity, Repeatability and Hysteresis). The resolution of the gauges is determined by the Analogue/Digital conversion that takes place in the Mosdax module. The resulting resolution is typically 1/30,000 (i.e. Full Scale/30,000). For the 0 to 2000 guages used as Sellafield this equates to an accuracy of ± 14 kPa and a resolution of ± 0.5 kPa.

SELLAFIELD MONITORING SYSTEM DESIGN

Prior to installation, zones of potential interest are targeted on the basis of geology, hydrogeological and geophysical testing carried out in the boreholes during and following completion of all drilling works. These zones are then reviewed with the required testing strategy for that borehole to derive a proposed monitoring system. Calliper logs are run in the hole prior to completing the exact design of the system to assist in the selection of the best available packer seat locations.

These zones can be divided into primary zones which are designed to monitor specific features and the intervening secondary zones within which there are no specific hydrogeological targets.

If only fluid pressure measurements and sampling are required, a measurement port coupling is installed in each monitoring zone. If sampling, fluid withdrawal or fluid injection is anticipated, both a pumping port and a measurement port coupling are installed in each monitoring zone. Duplicate measurement port couplings are installed in primary zones in order to provide redundancy in the system.

In order to standardise as much as possible and allow flexibility in any future restringing of Mosdax probes the majority of the Sellafield transducers have a 0 - 2000 psi (0 - 13800 kPa) range. These transducers were chosen because they could cover the depth range of a majority of the Sellafield boreholes and there was considered to be little advantage in reducing the resolution below 0.07 psi (0.5 kPa).

The Sellafield monitoring network currently comprises 22 instrumented boreholes within which there are some 270 zones capable of providing pressure measurements.

Of these zones some 160 are being monitored utilizing dedicated Mosdax probes providing data at 2 minute intervals in the RCF area and 30 minute intervals in the regional boreholes.

In order to centralise the gathering of data either mains electricity supplies or solar panels and associated batteries are provided to power the system at each borehole location. In addition, in order to facilitate remote communication with the Mosdax dataloggers, modems linked to either national telephone system land lines or cellular telephone communications are connected to each datalogger. This data management system, which is currently being completed, will allow the monitoring systems to be automatically accessed and downloaded from a single computer remote from the site. The communication system has to date been proven some 300 miles from the site.

This data capture system will automatically feed information through to an Oracle database mounted on a Sun workstation. The Oracle database is accessed by a data visualisation system which can present the data in a range of optional formats.

As detailed above the modular design of the monitoring system enables the system to be tailored to meet specific objectives. For long term pressure monitoring at Sellafield, which will continue for at least ten years, in general, only the primary zones will be monitored. However, in order to meet specific requirements, such as monitoring the effects of pump tests in specific boreholes, the monitoring strings may be reconfigured, by the addition of probes, to monitor both primary and secondary zones. Such reconfiguration was carried out for a crosshole test where 29 zones were instrumented in Borehole No.4 during a pump test carried out in Borehole No. 2 some 125 m away. Subsequently this string was reconfigured to monitor 9 zones only and the remaining probes installed in three other boreholes.

DATA ASSESSMENT AND PROCESSING

When the data from the first borehole to be instrumented, Borehole No. 14, was analysed it was evident from the presence of cyclic fluctuations in the data that monitoring zone interval pressures were being influenced by a variety of natural phenomena. A typical set of records from Borehole No. 14 are shown in Figure 4.

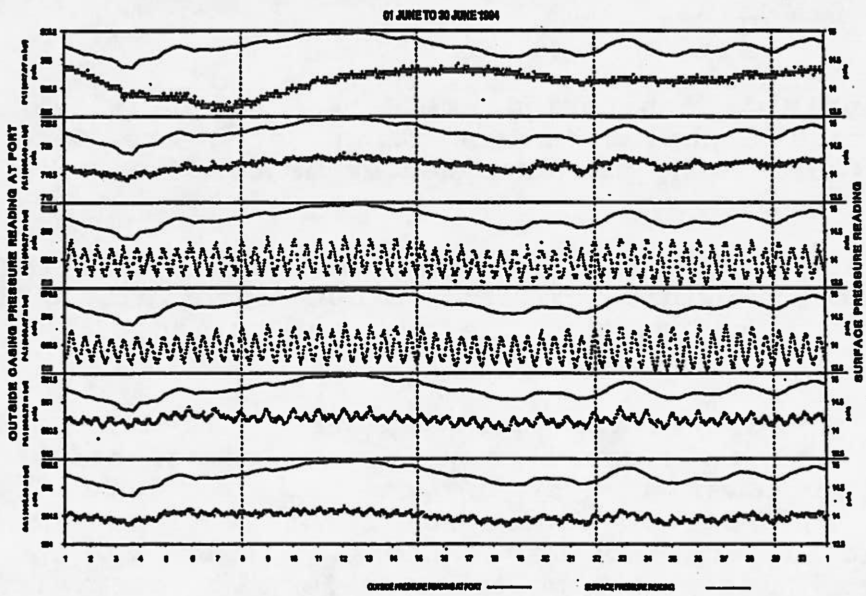


Figure 4 - Data Record from Borehole No. 14

In order to clearly establish natural small scale fluctuations in groundwater pressure it is therefore necessary to understand these other phenomena. In addition it was recognised that characterisation of these small scale fluctuations may assist in the overall hydrogeological characterisation of the site.

Many physical phenomena may cause changes in the measured zone interval pressure in deep boreholes. Some phenomena relate to processes that create pressure disturbances that may be used to derive hydrogeological properties; others are more clearly classified as instrument noise. The main phenomena which are considered to be acting are outlined below.

Aquifer Response to Barometric (Atmospheric) Pressure

It has been frequently observed that water levels in boreholes respond to barometric pressure. It is assumed that this is due to the variation in load on rocks overlying such aquifers and the strains induced by these loads. Authors have calculated atmospheric efficiencies of aquifers and have related these to the bulk elastic properties of the aquifer (Narasimhan et al 1984).

Ocean Loading

In coastal areas water levels in aquifers may fluctuate in response to the ocean/sea tides. This can either be due to hydraulic connection of the aquifer to the tidal regime or to strain induced by the varying loads placed on overlying rocks.

Earth Tides

Periodic aquifer dilations and associated water level fluctuations are known to result from lunar and solar tidal forces. Various authors have used spectral analysis methods to estimate aquifer properties from the responses observed in boreholes to these tides. Models for both homogeneous and fractured aquifers have been considered.

From a broad assessment of the Sellafield data set the maximum magnitude of the impact of these phenomenon are as follows:

* Instrumental Noise $\pm$ 0.5 kPa ($\pm$ 0.07 psi)
* Barometric Effects $\pm$ 1.5 kPa ($\pm$ 0.21 psi)
* Tidal Effects $\pm$ 3.0 kPa ($\pm$ 0.44 psi)

The scale of these phenomenon is relatively small when one is considering a large scale hydrogeological system over an extended period of time. However, in the Sellafield environment an understanding of these phenomena is important as such an understanding will:

* Allow removal of these effects to ensure identification of short term small scale transient responses to pumping from adjacent boreholes.

* Analysis of such phenomena may provide important information on the bulk hydrogeological properties of the rock mass.

In order to remove these phenomena the data is processed on a Sun workstation by removing the barometric effects, passing the data into the frequency domain and applying a Fast Fourier Transform

(FFT) followed by an appropriate filter (low pass, high pass, band pass or notch) and returning the data to the time domain for hydrogeological analysis.

To facilitate the removal of barometric effects from pressure data measured in a monitoring zone interval, it is necessary to calculate the Atmospheric Efficiency Factor (AEF) of the zone interval in question. The AEF of a zone is defined as the change in zone interval pressure caused by a 1 kPa change in barometric pressure following normalisation of the respective atmospheric data set to a mean atmospheric pressure of 100 kPa. In order to provide the best estimate of AEF, barometric pressure records are provided at each borehole site as an integral part of the Mosdax system.

For analysis of the Borehole 2 - 4 crosshole data, discussed below, the AEFs for all of the zones in Borehole No 4 were analysed and found to lie between 0.3 and 0.65. In the process of normalising the atmospheric data set to 100 kPa, with an average variation in atmospheric pressure of ± 2.5 kPa, the maximum correction to the data has been between 0.6 and 1.3 kPa.

The procedure used to filter the Borehole data is summarised on Figure 5. In order to pass data through a FFT it is necessary to have data points at regular time intervals. In practice this has not always been possible and it is necessary for data collected with an uneven time interval, with null points or data gaps, to be interpolated onto a regular time grid such that it becomes suitable for passing through a FFT.

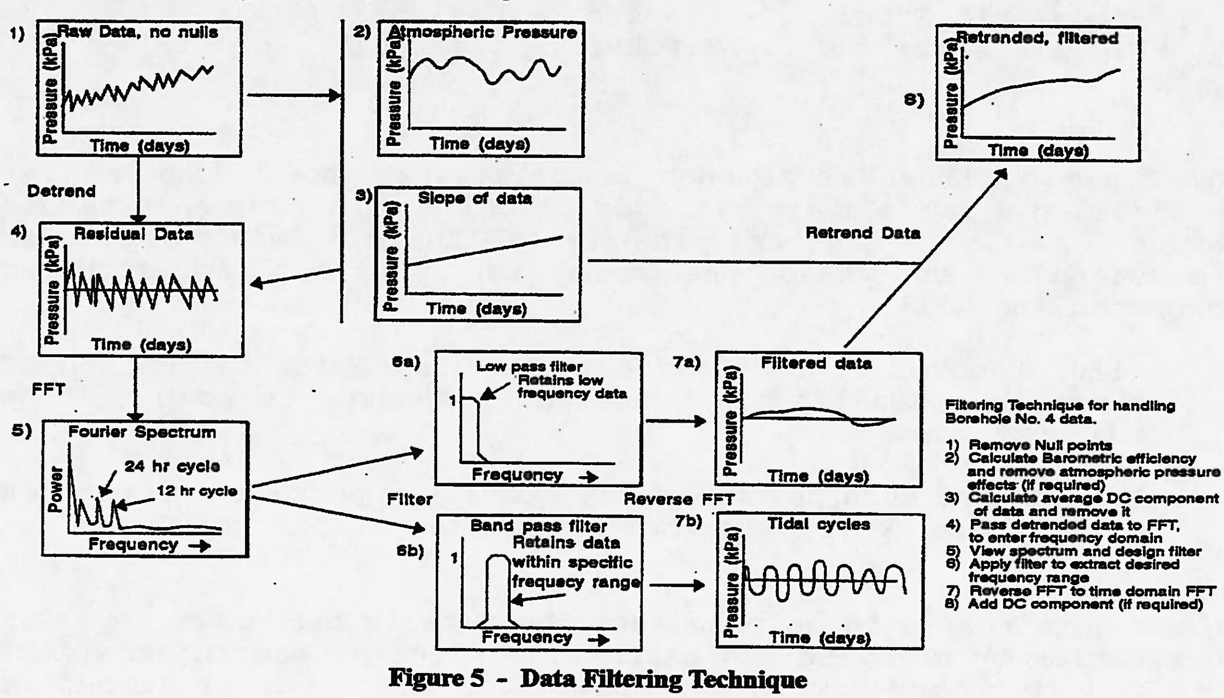


Figure 5 - Data Filtering Technique

The adopted procedure is to:

1) Identify null points and eliminate them from data array.

2) Calculate AEF and remove atmospheric pressure effects, if required.

3) Calculate average of first and last ten points in array, generate straight line and detrend data.

4) Pass data to FFT to transform it to the frequency domain.

5) Generate filter with required frequency cutoffs.

6) Apply filter.

7) Apply reverse FFT to go back into the time domain.

8) Re-trend the data by adding line from (3) above, if required

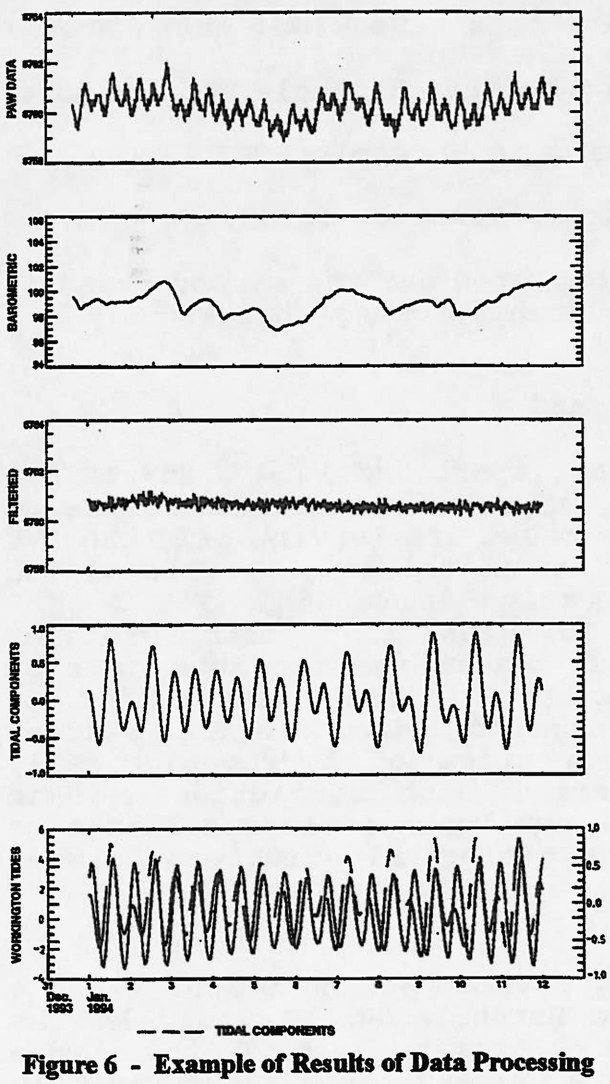


Figure 6 - Example of Results of Data Processing

For frequency domain filtering to be effective, the response being examined should have a characteristic frequency significantly different to the frequency of the background events being filtered out. Four general types of filters can be used in the frequency domain, low pass, high pass, band pass and notch filters.

It is possible, by the use of notch filters, to extract data of specific frequencies. Notch filters were used to extract the 12 and 24 hour tidal components from the Borehole No. 14A data. These tidal components have been displayed in Figure 5 alongside the barometric signal and calculated sea tide data at Workington which is situated just North of the Sellafield site. It can be seen that the tidal component of the data set is in phase with the sea tide.

In Figure 6 the process is illustrated with the raw data displayed in the top graph and the barometric pressure in the second, the derived filtered data in the third and extracted tidal information in the fourth. This is superimposed on the calculated sea tide data at Workington which is situated just north of the Sellafield site in the lower graph.

SELLAFIELD MONITORING NETWORK

The long term purpose of the monitoring network is to determine the natural environmental pressure gradients and natural changes to those pressure gradients. However, over the next few years the monitoring network within the area of the PRZ will play a key role in the site characterisation process providing data on the response of the hydrogeological system to imposed changes. The dataderived from such testing will be used to support the hydrogeological models of the site.

Three phases of active testing have been scheduled, these are:

* Crosshole interference testing between Borehole Nos. 2 and 4.
* Groundwater abstraction testing in Borehole RCF3.
* Construction of the Rock Characterisation Facility.

To date the first phase has been completed and the second phase is nearing completion, the data is currently being analysed.

Crosshole Testing Borehole Nos. 2 and 4

Borehole Nos. 2 and 4 are some 125 m apart and 1600 m and 1250 m deep respectively. During drilling of both boreholes the Sherwood Sandstone sequence was permanently cased off leaving only the BVG exposed to testing. Since completion of drilling and associated geophysical and hydrogeological testing in Borehole No. 2 on 9 April 1991 and Borehole No. 4 on 19 August 1991, both boreholes have been well characterised by an extensive programme of post completion testing. This testing has included assessment of stratigraphy and fracture orientation and hydrogeological studies to identify the location and properties of hydrogeologically transmissive fractures. Furthermore a high resolution seismic tomography survey has been carried out between the two boreholes in order to provide details of the geological structure between the two boreholes.

For the proposed crosshole testing between Borehole Nos. 2 and 4 it was decided in late 1993 that Borehole No. 2 would be the source hole and Borehole No. 4 the monitoring hole. To this end a 30 zone Westbay system, comprising 9 primary zones (approximately

20 m in length) and 21 secondary zones (18 to 55 m in length), was installed in Borehole No. 4. Here primary monitoring zones were targeted on identified hydrogeological conductive features, and zones adjacent to structural features that intercept both boreholes.

In Borehole No. 2 conventional oilfield surface inflate packers were deployed to 1600 m on standard 2.875 inch EUE tubing string. The string comprised twelve packers, which isolated the six known hydrogeologically conductive features. Within each of the six target zones a hydraulically operated sliding sleeve was installed, which once opened allowed the extraction of groundwater from the target zone. In total there were 15 downhole quartz pressure sensors, these being positioned within each target zone, each intervening zone between adjacent packers with no sliding sleeve, the borehole annulus and positions within the tubing string.

In order to provide access to the main tubing string for geophysical logging tools and specialist sampling tools the Moyno pump used was mounted in a secondary string sidemounted off the main test string at 260 m. The sidemounted production string incorporated two pump stators so as to allow use of two different sizes of Moyno pump such that flow rates from 0.2 to 200 litres per minute could be attained at drawdowns of 150 m.

Each test comprised a period of pumping from the target zone followed by shutting in of the system to allow full recovery. Due to the low permeability of the formation, constant rate extraction tests were technically difficult with a lower cutoff of 0.2 litres / minute. The majority of the tests were therefore carried out as constant head extraction tests with imposed drawdowns of up to 200 m.

The configuration of the testing allowed the drawdown to be applied and held for approximately 10 days whilst the responses in the adjacent monitored zones in Borehole No. 2 and all those in Borehole No. 4 were monitored. In this manner the rate and magnitude of the propagated signal could be measured. A diagrammatical layout of a section of a typical test is shown in Figure 7. The left hand column shows the zone response to an imposed 1800 kPa drawdown applied at a discrete depth interval at 591.3 m in Borehole No. 2. The right hand column shows typical depth-discrete response data, of the order of 2 to 20 kPa, from the Westbay system in Borehole No. 4 following post-processing to correct for atmospheric effects and other cyclic phenomena, as discussed above. Between the two columns has been superimposed a part of the seismic tomogram.

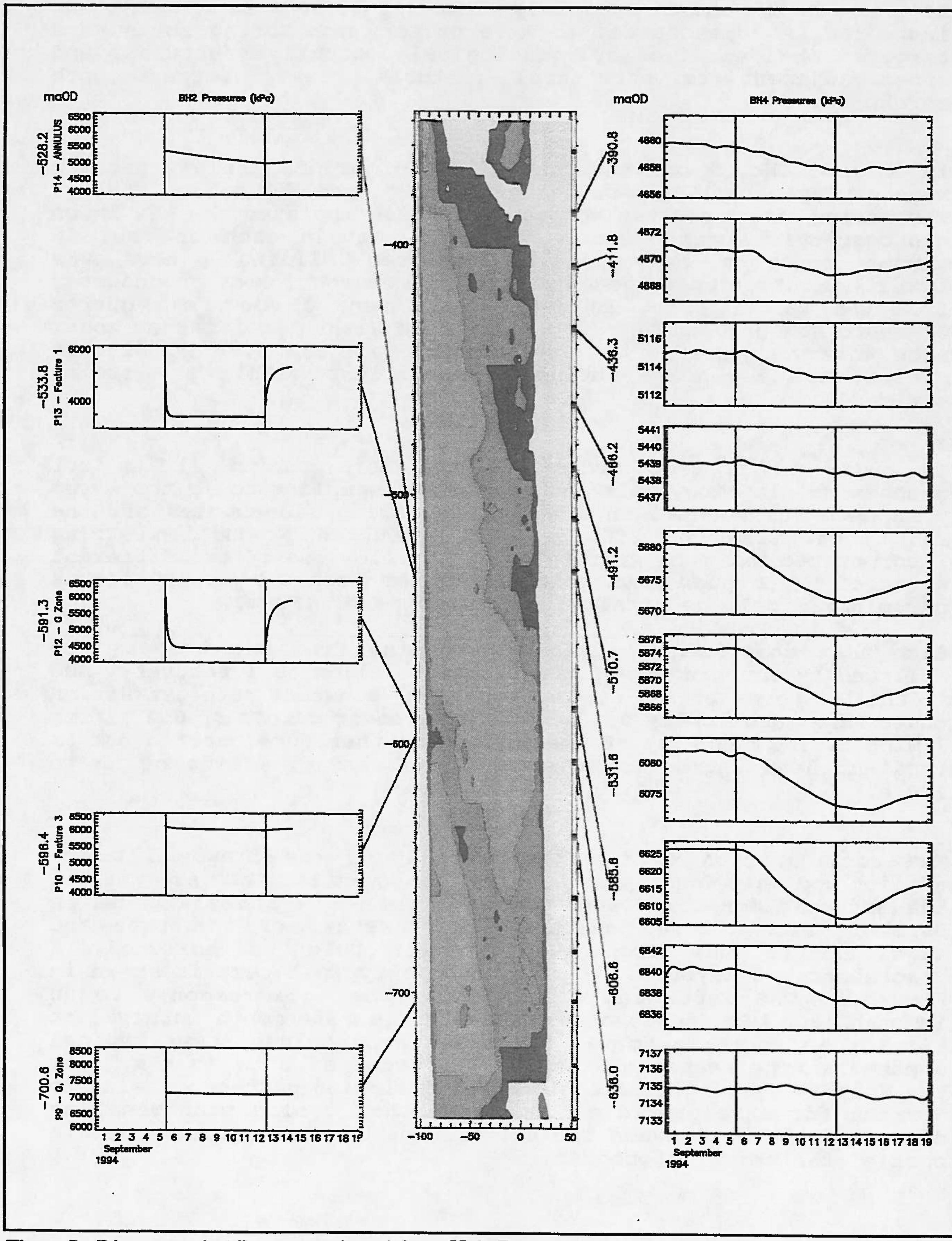


Figure 7 - Diagrammatical Representation of Cross Hole Responses between Borehole No. 2 and No. 4

RCF3 Groundwater Abstraction Testing

Following on from the crosshole testing programme a large scale pump test was proposed and at the time of writing is nearing completion within the RCF area. The principles developed for the crosshole testing program were enhanced and are being applied on a larger scale. The 12 packer string previously installed in Borehole No. 2 has been withdrawn and reinstalled to a depth of 990 m in Borehole No. RCF3, which has been drilled down the centreline of one of the proposed RCF shafts. Following removal of the pump test string from Borehole No. 2 a Westbay system was installed.

Within all of the boreholes adjacent to Borehole No. RCF3, including Borehole No. 2, monitoring systems have been installed with Mosdax probes installed in the primary zones, with the exception of Borehole Nos. RCM 1 and 2 which are approximately 35 m from Borehole No. RCF3, where both the primary and secondary zones are monitored. Within this rock mass there are in excess of one hundred individually monitored zones which are providing data at two minute intervals throughout the seven month test period.

A diagrammatical representation of the pumped well Borehole No. RCF3 and the adjacent monitoring holes is presented in Figure 8.

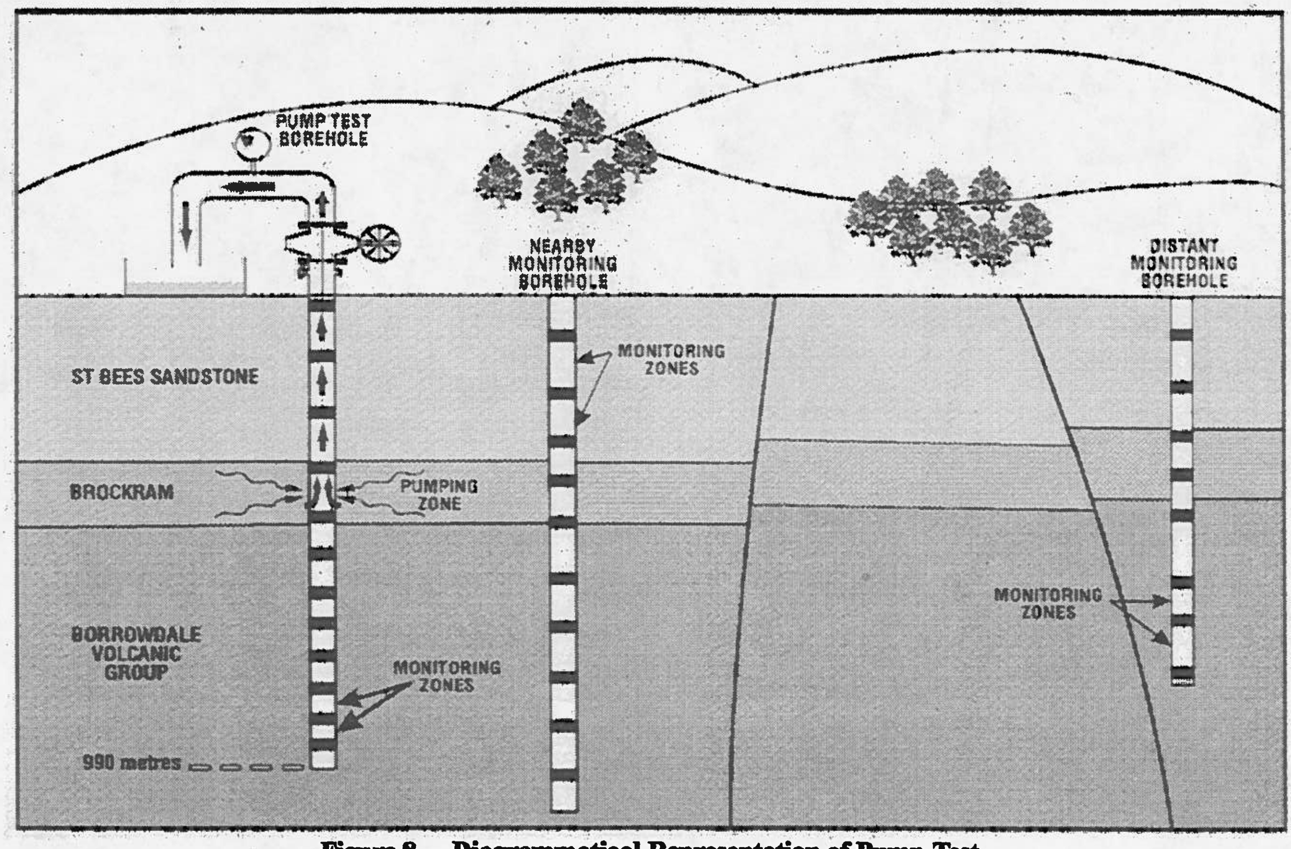


Figure 8 - Diagrammatical Representation of Pump Test

Following completion of the installation of the pump test string, in Borehole No. RCF3 and prior to the commencement of the main pump test, the complete pump test equipment was "tuned" by opening of individual sliding sleeves in turn and carrying out "mini extraction tests" under test conditions envisaged for the main testing. During this period the automatic pump control software was radically enhanced.

To date with this system it has been possible to achieve constant rate extraction tests with flow rates as low as 0.2 litres/minute maintained within ± 1% and constant head extraction test maintained within ± 2 kPa of the target drawdown. It is considered that one of the main reasons that these testing conditions have been achieved is as a result of obtaining individual test zone ground responses to enable optimum pump test equipment settings prior to commencement of the main pump test.

A three dimensional representation of the monitoring network around the Borehole No. RCF3 pump test is presented in Figure 9.

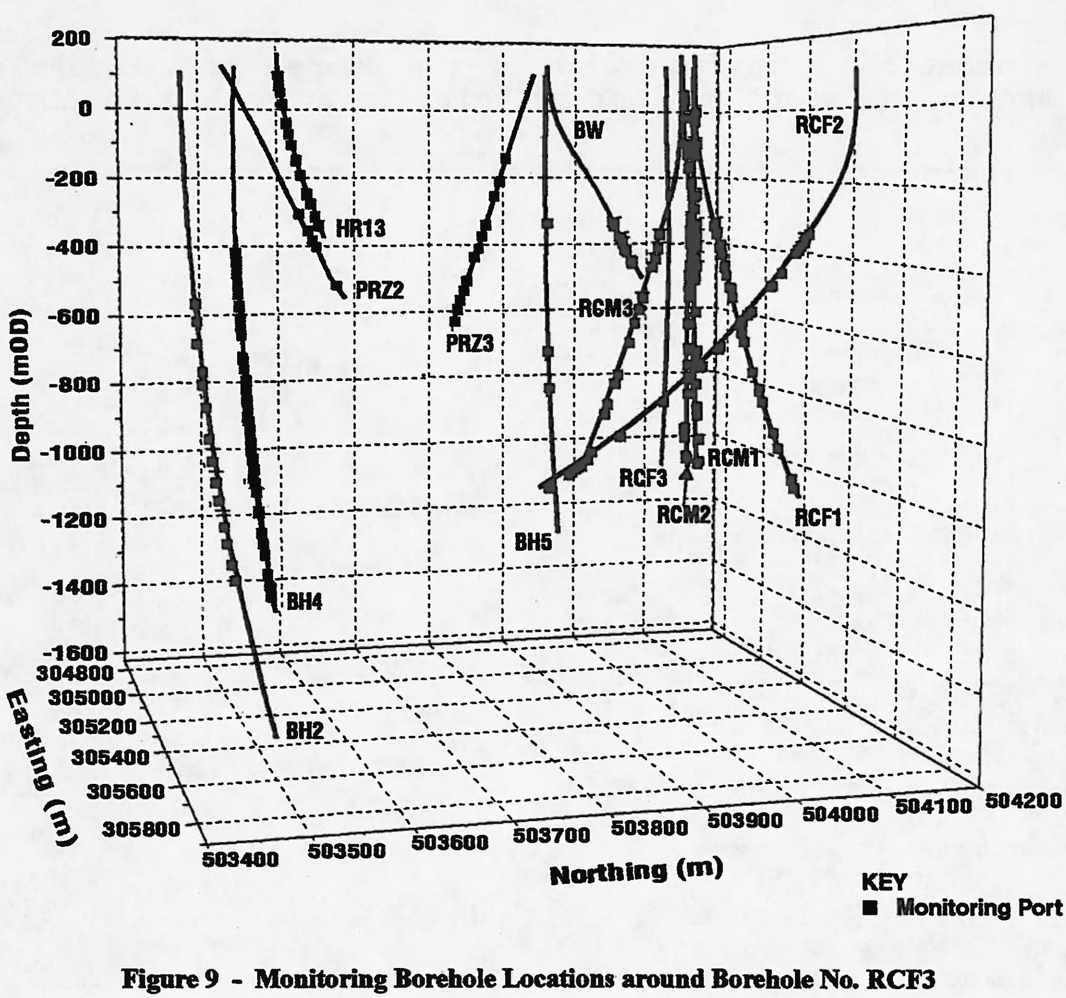


Figure 9 - Monitoring Borehole Locations around Borehole No. RCF3

FURTHER TESTING

Proposals are currently being considered to utilise the system to carry out multiple injection tracer tests. The programme will in principle be similar to the RCF3 pump test with extraction of groundwater from a selected zone in Borehole RCF3 and simultaneous injection of multiple conservative and non conservative tracers in selected zones within the Westbay systems installed in Boreholes RCM1 and RCM2.

CONCLUSIONS

The Nirex Sellafield Geological Investigations are one of the largest and most comprehensive site characterisation programmes currently being carried out in the world. The adoption of the described monitoring system has provided a very flexible and cost beneficial approach to groundwater monitoring and testing. The quality of the data that is being acquired is of the highest order and has enabled the identification of pressure changes of down to 1 to 2 kPa (100 to 200 mm H_2O) at depths of some 1000 m in response to natural and imposed drawdowns in adjacent boreholes. The monitoring network is currently being integrated with other time series data derived from weather stations, river gauging and National River Authority boreholes. When complete the system will be accessible from a single computer workstation, if required located offsite, that will be capable of integrating and processing the data, not only as simple time series graphs, but also as 2D and 3D visualisation of pressure changes.

REFERENCES

CHEIN Y M, BRYCE R W, STRAIT S R, YEATMAN R A (1986)
Elimination of frequency noise from groundwater measurements. In Disposal of High Level Nuclear Waste. Burkholder (ed).

CLARKE W E (1967)
Computing the Barometric Efficiency of a Well, Proc. ASCE, J. Hyd, Div., pp 92-98.

MARINE I W (1975)
Water Level fluctuations due to earth tides in a well pumping from slightly fractured crystalline rock. Water Resources Research Vol. 11, No. 1 February 1975.

MELCHOIR P (1978)
The tides of planet earth.

NARASIMHAN T N, KANEHIRO BY, WITHERSPOON P A (1984)
Interpretation of earth tide response of three deep confined aquifers.

UK Nirex Limited Reports 524 and S/94/006.

ACKNOWLEDGEMENTS

The authors would like to thank UK Nirex Limited for giving their permission to publish this paper and the Directors of Sir Alexander Gibb & Partners Limited and Soil Mechanics Limited and colleagues in KSW for their support.

SITE CHARACTERIZATION METHODS FOR THE DESIGN OF A GROUNDWATER EXTRACTION SYSTEM IN A BEDROCK AQUIFER

D. Joseph Wessley, P.E.
Montgomery Watson

Richard H. Weber, P.E.
Montgomery Watson

Daniel A. Otzelberger, P.E.
Waste Management of Wisconsin, Inc.

ABSTRACT

Several volatile organic compounds (VOCs) have been detected in bedrock monitoring wells since 1984 at a solid waste landfill. Continued detection of VOCs in the bedrock aquifer led to construction of a remediation system to address the contaminated aquifer. This paper discusses the site background, geology, hydrogeology, and landfill characteristics; the methods and results of the landfill monitoring and investigations performed; and the groundwater extraction system which was designed and constructed to control further off-site migration of contaminants in the bedrock aquifer.

BACKGROUND

A groundwater extraction system has been installed in a bedrock aquifer at a closed landfill in southeastern Wisconsin. The 83 acre landfill operated from 1978 to 1989 and accepted municipal, industrial, and commercial waste. The site also accepted hazardous and non-hazardous liquid and sludge wastes up until October 1982. Portions of the site began receiving waste sometime around 1959. The total volume of waste disposed at the site is estimated to be approximately 12 to 14 million cubic yards.

GEOLOGY

The landfill is directly underlain by glacially derived unconsolidated deposits. These deposits consist primarily of fine-grained clays and silts, with intermittent sand seams. Silurian Age Niagara dolomite bedrock underlies the glacial deposits. The dolomite is a light gray to white, slightly weathered, moderately fractured, massive dolomite or dolomitic limestone, and is several hundred feet thick. Due to the shallow nature of the sand seams, they are not contiguous with the dolomite bedrock, and are often separated from the bedrock by up to 100 feet of clayey till. A dolomite bedrock high which trends northeast to southwest appears to exist in the north central portion of the site. The bedrock surface slopes downward to the northwest and southeast.

HYDROGEOLOGY

The upper water table is contained in the unconsolidated glacial deposits in most areas. The water table surface is generally quite shallow and typically reflects land surface topography. Horizontal groundwater flow is characterized by an outward groundwater flow direction from the landfill in all directions. The water table mound and outward flow directions are influenced by the accumulation of leachate within the landfill.

The Niagara dolomite aquifer is under semiconfined conditions except in areas where the overlying glacial deposits are relatively permeable. Most private wells in the area draw water from this formation. Groundwater flow in the dolomite is primarily along horizontal bedding planes which originated from fractures and solution cavities within the dolomite.

Groundwater flow directions in the bedrock aquifer mimic the surface of the Niagara dolomite with a piezometric divide paralleling the previously described bedrock high beneath the landfill. Groundwater flow is generally southeasterly away from the divide. Groundwater flow is also to the northwest of the divide.

LANDFILL CHARACTERISTICS

Based on site operating records and subsurface investigation data, portions of the landfill directly overlie the natural glacial deposits. Base grades are estimated to be less than 10 feet to more than 100 feet above the bedrock surface. A liner exists beneath a portion of the landfill and consists of a compacted clay layer between 4 and 5 feet thick. Leachate collection systems were also installed beneath portions of the landfill. The final cover was constructed in phases and consists of a minimum of 4 feet of compacted clay and 6 inches of topsoil.

Since 1982, several remedial measures were undertaken to mitigate water quality impacts to the shallow sand seams, enhance leachate removal and containment within the site, and lessen the outward flow of leachate from the site. The remedial measures have consisted of perimeter clay cut-off walls, leachate collection systems, groundwater collection systems, and final cover upgrades.

BEDROCK AQUIFER GROUNDWATER MONITORING ACTIVITIES

Hydrogeologic investigations conducted from 1981 through 1983 indicated that bedrock water quality may have been impacted at the site. During 1982, low levels of several VOCs were detected in some private wells, generally to the east of the site. A bedrock aquifer investigation program was developed, and in 1984 a study was initiated to explore water quality conditions in the dolomite bedrock aquifer. The results of the bedrock investigation discovered that low levels of VOCs (40 to 250 ug/L) were detected in several bedrock and private wells east of the site. Additional groundwater monitoring performed in 1986 and 1987 concluded that several bedrock wells east of the site have been affected by VOCs at relatively low concentrations which may be attributable to the landfill.

Subsequent monitoring of bedrock wells indicated a continued presence of VOCs in these wells. Some VOCs detected were above state groundwater quality standards. This led to concerns raised by the state regulatory agency with regard to the potential source of groundwater contamination and apparent migration of VOCs to the area east of the landfill. Based on these concerns, additional investigations were performed in 1990 and 1991 to further assess geologic, groundwater flow, and groundwater quality conditions at the site.

BEDROCK AQUIFER INVESTIGATION

The tasks performed in the additional investigations in 1990 and 1991 were as follows. A 180.2 foot bedrock boring was drilled to provide a vertical profile of VOC concentrations in the bedrock aquifer and determine the vertical hydraulic gradient at this location. Three 45 degree angle borings were installed to downline depths of 156 and 188 feet to determine fracture occurrence and direction in bedrock, to determine vertical distribution of VOCs in the bedrock aquifer, and to provide additional piezometric head monitoring points during the subsequent aquifer pumping test. The angle borings were instrumented with multi-level groundwater monitoring devices which allowed piezometric head level measurements and groundwater sample acquisition from discrete intervals in each borehole.

A 120 foot deep (approximately 90 feet into bedrock) groundwater pumping well was installed and a 72 hour bedrock aquifer pumping test was conducted at 40 gpm to determine bedrock aquifer hydraulic properties. From the pumping test results, a groundwater model was developed using MODFLOW to simulate the effect of alternative extraction system scenarios which would effectively control the further migration of VOCs within the dolomite bedrock away from the east side of the landfill.

The conclusions from the 1990 and 1991 investigations were as follows:

- Regional and local fracture orientations within the dolomite bedrock consist of a primary fracture set orientated southeast-northwest, with two distinct secondary fracture sets oriented southwest-northeast.

- Site-specific data indicates the presence of a southwest-northeast trending fault beneath the landfill located northwest of the pumping well. The steep hydraulic gradient in the vicinity of the suspected fault and the lack of response during the pumping test at wells opposite the fault indicate the fault acts as a hydraulic barrier.

- Groundwater sampling while drilling a bedrock aquifer monitoring well indicated VOC contamination to a depth of 90.2 feet (approximately 60 feet into bedrock). The absence of VOCs below this depth indicates that contamination is limited to the uppermost part of the bedrock aquifer. VOC concentrations were highest (1,182 ug/L) in the uppermost interval (31.2 to 50.2 feet) sampled where the bedrock is fractured and more permeable. Total VOC concentrations in the second (50.2 to 70.2 feet) and third (70.2 to 90.2 feet) sampled intervals decreased significantly (297 ug/L in second interval, 308 ug/L in third interval).

- Groundwater concentrations detected in the bedrock aquifer exceed state groundwater quality standards for several VOCs.

- Packer pressure testing of the bedrock in the angle borings showed bedrock hydraulic conductivities ranging from less than 3.3×10^{-7} to 4.1×10^{-4} cm/sec. The weathered upper portion of the bedrock generally had the highest permeability within this range.

- A pumping rate of 40 gpm effectively reversed the southeastward groundwater gradient at the pumping well for a distance of approximately 450 feet. Drawdown at the end of the pumping test was almost 22 feet at the pumping well. Because of the low permeability hydraulic barrier created by the suspected fault, no significant response was observed at monitoring wells across the fault and the drawdown cone was elongated parallel to the fault.

- Calculations from pumping test data showed that the hydraulic conductivity for the bedrock aquifer ranged from 2.1×10^{-4} to 2.4×10^{-3} cm/sec.

- Analyses of four groundwater samples collected from the pumping well during the pumping test indicated that total VOC concentrations ranged from 93.3 to 160 ug/L. VOC concentrations at the well generally increased during the pumping test.

- Extraction system simulations on a computer model using pumping test data showed that a four well extraction system with a total discharge capacity of 80 gpm could effectively control contaminant migration from the landfill. Specifically, the results of the simulations indicated that a total of three pumping wells southeast of the fault with a combined pumping rate of approximately 50 gpm and one extraction well northwest of the fault with a pumping rate of 30 gpm would create an effective barrier to flow and contaminant migration. Corresponding drawdown at each extraction well was estimated to be between 10 and 20 feet.

BEDROCK AQUIFER EXTRACTION SYSTEM DESIGN

A design plan was prepared and finalized in August 1992 which outlined the components of a groundwater extraction and discharge system for the bedrock aquifer. The primary intent of the groundwater extraction system was to control further off-site migration of contaminants in the bedrock aquifer. The design plan provided a description of the remedial design including extraction well installation, system components, and system performance monitoring.

The August 1992 design plan recommended constructing the bedrock aquifer extraction wells in two phases. The first phase recommended installing three extraction wells on the site. The design plan also recommended installing the extraction wells in 20 foot increments into bedrock. During installation of the extraction wells, groundwater samples would be collected and analyzed for VOCs from each 20 foot increment. Sampling and drilling of extraction wells would continue in 20 foot increments until encountering a zone with no VOCs detected.

After the first phase of the extraction system is constructed and in operation for a short period of time, the design plan recommended that the performance of the system be evaluated. At the conclusion of this evaluation, one additional well would be added if bedrock aquifer drawdown is not adequate to control migration of contaminated groundwater. In this way, the location of the fourth extraction well could be optimized.

No on-site treatment for the extracted groundwater was recommended because anticipated bedrock groundwater quality was below POTW pretreatment limits. Consequently, discharge of the contaminated bedrock groundwater to the sanitary sewer was recommended.

BEDROCK AQUIFER GROUNDWATER EXTRACTION SYSTEM CONSTRUCTION

Construction of the bedrock aquifer groundwater extraction system began in January 1993 with the installation of two extraction wells to depths of 127 and 213 feet below the ground surface (approximately 84 and 104 feet into bedrock). A third extraction well was created by modifying the pumping test well.

An electric submersible pump was installed between 60 and 75 feet into bedrock at each of the three extraction wells. Each extraction well has a well head structure which encloses the stick-up portion of the well casing out of which the pump discharge pipe exits. Also within each well head is a strainer, check valve, gate valve, flow meter, and sampling port. The pump discharge pipe from each well enters back into the ground inside the well head and connects to a gravity discharge pipe. The gravity discharge pipe from two extraction wells connects to a lift station manhole. A forcemain conveys water from the lift station manhole to a metering manhole. The gravity discharge pipe from the third well connects directly to the metering manhole. From the metering manhole, the extracted groundwater is conveyed to the sanitary sewer system.

The extraction pumps, discharge piping, well heads, and manholes were installed in May and June of 1993. Permission to discharge to the POTW was obtained in December 1993, and start-up of the extraction system occurred in January 1994. Start-up activities were performed from January to May 1994, and included monitoring water levels and pumping rates. At the end of start-up, the extraction pumps were pumping at about 40 gpm. Drawdown at the extraction wells varied between 22 and 48 feet.

A fourth extraction well will be installed if the extraction system performance evaluation currently being conducted indicates this well is required to further control migration of contaminated groundwater in the area. As part of the interim evaluation, pumping flow rates, drawdown, and water quality in site monitoring wells are being monitored and compared to pre-start-up conditions to evaluate system performance. Based on the monitoring data collected to date, a fourth extraction well is anticipated to provide effective control of VOC migration from the landfill.

REFERENCES

WARZYN ENGINEERING INC. (1985)
Bedrock Aquifer Investigation, Omega Hills Landfill and Development Project, April 1985, Project No. C10990.

WARZYN ENGINEERING INC. (1988)
Bedrock Aquifer Investigation Supplemental Report, Omega Hills Landfill, June 1988, Report 13335.

WARZYN INC. (1991)
East Side Bedrock Aquifer Investigation, Omega Hills Landfill, August 1991, Report 15220 (Volumes 1 and 2).

WARZYN INC. (1992)
Construction Observation Report - Groundwater Monitoring Well Installation, Omega Hills Landfill, August 1992, Project No. 10003401.

WARZYN INC. (1992)
Design Plan, East Side Bedrock Aquifer Groundwater Extraction System, Omega Hills Landfill, August 1992, Project No. 10004201.

WARZYN INC. (1993)
Project Manual, Groundwater Extraction System East Side Bedrock Aquifer (Volumes I and II), March 1993, Project No. 10004201.

WARZYN INC. (1994)
Construction Documentation and Start-Up Report, Phase I Groundwater Extraction System East Side Bedrock Aquifer, August 1994, Project No. 10004201.

USE OF EXISTING GEOTECHNICAL DATA TO SUPPLEMENT SITE INVESTIGATIONS

William J. Pfalzer
Kentucky Transportation Cabinet
Division of Materials
Geotechnical Branch

INTRODUCTION

The Kentucky Transportation Cabinet, through its Geotechnical Branch, and the Kentucky Transportation Center of the College of Engineering, University of Kentucky (formerly the in-house Division of Research of the Transportation Cabinet), have been involved for many years in the development of a geotechnical database. From the beginning, the database was conceived to be a tool which would allow the routine use of existing data to facilitate the development of current projects. The initial efforts in developing and implementing the database were made in the mid-to-late 1970's. The capabilities of computers, at that time, were very limited relative to current capabilities, and information could be shared by various potential users only with great difficulty. Primarily for that reason, the data collected at that time were never widely used. In the last two years, however, database development efforts have been revived, and it seems quite certain that this system will be more widely used. The amazing increase in the power and capabilities of computers is allowing the development of a more comprehensive database with a simplified input. Of even greater importance, however, is that vastly improved methods of sharing information (by computer disks, tapes, Internet, etcetera) will allow a wide variety of potential users to access the database. The geotechnical databank will include essentially all drilling and lab testing results. At this time, however, the database is still very much in developmental phases. This paper, therefore, deals far less with the past use of existing data than with the capabilities we anticipate having in the near future. It is believed that the development of this and similar databases will eventually have the capability of making a significant impact upon the way in which the geotechnical engineer performs site investigations. In many cases, it will allow him to develop a more detailed and accurate model of site characteristics. Alternatively, it offers the opportunity to provide an equivalent degree of information at a lower cost.

PROBLEM STATEMENT

The cost of conducting a site investigation represents a substantial investment by the owner. Typically, these costs represent a controlling factor in project development to the extent that the geotechnical engineer would prefer to have more information on subsurface conditions than he is able to develop within budget restraints. Obviously, the use of any available pre-existing data would be of value provided the data were accurate and sufficiently accessible. I believe that the first of these two factors, accuracy of the data, is not a significant problem. If data developed in the last 20 years were not sufficiently reliable, it would imply either that the methods used by geotechnical engineers are changing so rapidly that drilling and/or lab testing results obtained only 15 or 20 years ago are already out-of-date, or that they were never reliable in the first place. Obviously, neither of these scenarios is true. Drilling techniques have changed very little over the last 20 or 30 years. The same is true for most laboratory test procedures; while the methods by which we apply the test results in our analyses methods may have changed considerably, in some cases, the actual testing methods have not changed. In terms of accuracy, while some errors undoubtedly must be present in the old data, the frequency of errors is probably little different from that of current practice.

It is believed, therefore, that essentially the entire reason why existing data are not currently widely reused relates to accessibility. Typically, these data have been stored in handwritten or typed paper files, often in an out-of-the-way storage facility where they are difficult to locate. Often due to problems of record keeping, the individuals working on a current project, unless they have personal knowledge of other jobs in the area, may be unaware whether previously developed data exists. Understandably, under these conditions, performing any extensive search for pre-existing data is generally considered to be more trouble than it is worth.

It is recognized that in many cases, existing data may not be close enough to the project site to be of any real value. Within the Transportation Cabinet, we have many projects involving entirely new routes, or significant realignments which cut across the pastures and cornfields far from any earlier development. Having worked for over ten years in the Geotechnical Branch's Structure Foundations Section, however, I have seen the large number of bridge replacement projects where the bridge is located immediately upstream or downstream of the existing structure. In many other projects, part-width construction is performed in stages, allowing traffic to be maintained while the new bridge is constructed in the exact location of the existing structure; the new bridge is simply widened to carry additional lanes and offer a higher capacity rating. As another example, when the Transportation Cabinet was widening the Watterson Expressway in Louisville, there were areas where the desired drilling could not be performed because the proposed borings fell in the driving lanes of the Interstate Highway. Due to considerations relating to the inconvenience that lane closures would cause to the public, and to the hazards to which this type of work would expose the drillers and flagmen, we decided that it was not practical to drill these locations. Ironically, the locations where we wanted but were unable to perform drilling were probably very near the sites drilled previously when the roadway was initially designed and built in the 1940's. Due to the age of this work, we were unable to utilize (or even locate) these old records. Many similar cases, however, will undoubtedly present themselves in the future, as bridges along existing routes are replaced, or as aging roadways are reconstructed, at least partially along their current alignments. It is on the basis of such considerations that we believe that development of the database, over the longterm, will provide a sufficiently high benefits-to-costs ratio to justify the expense of developing the database.

DETAILS OF THE DATABASE

The Basic Record, and Key Identification Fields

Each record in the database corresponds to an individual sample for borings with multiple samples. In the case of soundings, or borings from which only a single sample was obtained, a single record will correspond uniquely to an individual boring, but for borings with more than one sample, a series of records would be required to fully characterize the boring. The following fields are all used in conjunction to uniquely identify a record:

- State
- County
- Site
- Hole
- Sample

For data collected from Transportation Cabinet records, a site corresponds to an entire route within a given county. This could represent several different projects, such as a series of individual bridge replacements along US 60, for example. Hole numbers are assigned sequentially, uniquely, and with no numbers skipped within a given site. They do not necessarily, therefore, correspond to the hole numbers used for them in their original projects since this could involve having several hole 1's, or perhaps not having a hole five when their is a hole six. To make it easier to correlate the database records with the original project files, we have used a "driller's hole number" field which gives the original hole number designation.

Sample numbers within a boring are assigned the same way - sequentially from the surface down, with no numbers skipped. Samples may be either soil or rock, and one way to determine this (the way we have the computer do it in order to count the number of soil samples and number of rock samples) is to compare the depth of the sample with the depth to the rock surface.

Providing Accurate Boring Locations

Much of the collected data was obtained from the files of the Transportation Cabinet. Previously, these data had been located only by station and offset. In terms of accessibility, this is not a very workable system. For users outside the Department who lack access to the roadway alignment (curve data), determination of the boring locations by station and offset is impossible. Even for many users within the Department, such as the author, who seldom or never work with the curve data, such a system would be a nightmare. Therefore, while locations by station and offset have been retained, they have been supplemented by other systems. Even for the data collected in the 1970's (approximately 5000 samples from 3000 borings) a particular emphasis was placed upon ensuring that all data were accurately located by latitude and longitude. Currently, either latitude/longitude or State Plane Coordinates can be input, and the other set of values is then easily calculated.

The accuracy of boring locations has always been pretty good, and it is improving. A "location accuracy" field indicates how precisely the location is known. For the data collected in the 1970's, the latitude and longitude of almost all borings was determined by plotting them on 1"=2000' geologic quadrangles. For such data we had assumed, and to a certain extent have since confirmed, that we could attain an accuracy of ±30 meters (100 feet). For the data collected recently, approximately ⅔'s of the data had to be plotted in the same manner, and have the same accuracy. The other ⅓ (those with State Plane Coordinates) are assumed to have an accuracy of ±3 meters (10 feet).

Elevations and Depths

Related to the fields which provide locations in two dimensions (that is the locations of individual borings on a map) is a series of fields which provide locations in the third dimension. These fields give the ground surface elevation, the depth of individual samples, depth to bedrock, depth to water, etcetera. Fields relating to elevations and depths are used as follows. A "datum type" field is used in conjunction with an "elevation correction" field. Datum type is coded "S" for a sea level datum determined by surveying off of a benchmark. In such cases, the reported surface elevation is the true elevation above sea level. Datum type is coded "A" for an assumed datum. Typically when an assumed datum is used, the listed elevation differs greatly from the true sea level elevation so that it will be obvious that an assumed elevation was used. In such cases, we have determined the correction required to yield an approximate sea level elevation. In the output the approximate sea level elevation would be shown, but it will be flagged as having been derived from an assumed datum and, therefore, as being an approximate value. In a few cases, we have determined that the project design engineers have worked off of an approximate sea level datum based upon a crossroads elevation listed on the topographic maps (and, therefore, not upon surveying off of a benchmark). Data from such sites have been coded as a datum type "C". For these values, no correction is applied and the elevations are flagged as "approximate sea level elevations".

To simplify the input and hold the required number of database fields to a minimum, the "depth to bedrock" and "refusal" fields have been used as follows. The value in the "depth to bedrock" field is the depth to bedrock if bedrock was encountered (whether or not a core was obtained), and it is the total depth of the boring if the hole did not reach bedrock. The "refusal" field (a logical field) is coded "T" (true) if bedrock were encountered, and it, therefore, indicates that the value in the "depth to bedrock" field is a true depth to bedrock. It is coded "F" (false) if the boring did not reach bedrock, which indicates that the value in "depth to bedrock" is actually the total depth of the boring, and that it only indicates that top of rock is at some greater depth.

The "RDZ" (Rock Disintegration Zone) field is used as follows. For borings in which a core was obtained, the thickness of the RDZ can be determined, and (if it has been) the depth (below the ground surface) to the base of RDZ is indicated. If there is a sharp transition from soil to unweathered rock, this depth will be equal to the depth to bedrock. If there is an interval of weathered rock between the soil and the unweathered rock, the value in the "depth to bedrock" field represents the top of the weathered rock zone, and the value in the "RDZ" field represents the bottom of the weathered rock zone. The RDZ value, therefore, will be greater than the depth to bedrock by an amount equal to the thickness of the weathered rock interval.

The "depth to water" field is coded so that water depth below the ground surface is listed as a positive number. For ponded water, therefore, the depth of water above the ground surface is listed as a negative number. Ponded water does not affect the ground surface; that is the listed surface elevation would be the sediment surface, not the (higher) water surface.

Identifying Sample Types

A "sampling method" field indicates whether a sample were obtained, whether the sample were soil or rock, and the sampling method. In some cases it may indicate why certain intervals could not be sampled (due to the presence of boulders, for example). Following is a list of the codes which can be used in the "sampling method" field:

- samples could not be obtained due to the presence of boulders
- samples could not be obtained due to the presence of deep sands below the watertable
- auger (jar or bag) sample
- split barrel sampler (standard penetration test - SPT) with blowcounts
- thin walled tube sample (Shelby tubes)
- sounding (no samples or core in this boring)
- rock cores
- augered through soft rock
- cored, but logged as boulders in overburden
- cored, but logged as overburden
- void (open solution channel)
- clay-filled void
- open-face log

Defining Sample Intervals

In borings where thin-walled tube samples or Standard Penetration Test samples have been obtained through the overburden, sample intervals (typically 1.5 or 2 feet in length) will probably be separated by unsampled intervals. These unsampled intervals are not numbered. In some borings, it may not have been possible to obtain samples at certain depths (or even throughout the depth of the boring) due to the presence of boulders. In these cases, even though no actual sample was obtained for testing, the interval may be identified as a sample interval in order to indicate the presence of boulders, and these "samples" may be of any length. Also, where bag samples have been tested to determine grain size distribution, Atterburg Limits, etcetera, "sample intervals" are usually identified by the driller and/or the field geologist or engineer based on visual classifications made on-site. Such samples may be of any length, and typically they are not separated by unsampled intervals.

Definition of bedrock sample intervals is accomplished by inputting the elevations of the top and bottom of each sample. The highest rock sample (if any were obtained) will be sample 1 if there are no soil samples, or will have a number one greater than the deepest soil sample. Rock samples are defined on the basis of lithologic change, and may be of any length. An entire core could be a single sample, if it has a constant lithology throughout, or it could be divided into several samples of contrasting lithologies. That is, a long core consisting of a single material (all shale, or all limestone with minor shale partings, etc.) would be a single sample, even if it were 10 meters (30 feet) or more in length. Alternatively, if a sandstone, several feet thick, were underlain by several feet of shale, which was in turn underlain by another sandstone, they would be defined as three sequential samples of different thicknesses. In contrast with the samples of overburden (soil) materials, the rock samples must abut one another, covering the entire interval from top of rock to bottom of core.

A rock sample can contain more than a single lithology. If, for example, the core were interbedded shale and limestone, and over the entire interval they occurred in approximately the same proportions, the core could be defined as a single sample consisting of interbedded shale and limestone. Obviously, the alternative, treating each thin shale bed and each limestone bed as separate samples, could give a hundred or more samples over a ten foot core run, and would not be at all practical. If the interbedded shale and limestone interval were predominantly shale near the top, and predominantly limestone near the bottom, they could be input as two samples. The system for defining samples, therefore, is somewhat subjective (as is geologic mapping, and the defining of mapping units in general) but it is quite flexible.

Input Coding for Soil Samples

For soil samples, in addition to the location fields previously discussed, the results of laboratory testing are presented; these include both classification tests and strength tests. Stored data for soil samples includes natural moisture content, in-situ dry density, grain size distribution, Atterburg limits, specific gravity, maximum density/optimum moisture, CBR, AASHTO and Unified Classifications, unconfined compression, triaxial tests, and consolidation test parameters (e_o, c_c, c_r, P_c, and c_v).

The format used for inputting strength test data employs the use of a "test type" field which is coded as follows:

- "QU" for unconfined compression
- "UU" for unconsolidated undrained triaxial
- "CU" for consolidated undrained triaxial
- "CD" for consolidated drained triaxial
- "VS" for vane shear

Related fields allow the input of moisture content, wet density, confining pressure (if applicable), friction angle (if applicable), and cohesion.

Input Coding for Rock Samples

The following information can be coded for the cored intervals of sample borings; sampling method, % recovery, RQD, SDI, Jar slake, and lithologic description. The lithologic description gives primary, and if applicable secondary and tertiary lithologies. Each lithology can be further described by a series of modifiers. While the rock description is entered in a coded form which is somewhat complicated, it is intended that logs will be produced which will give the user a written description of the rock, the same as that presented in geologists logs. It will not be necessary, therefore, for the end-product user to interpret the coded records. On the other hand, use of the coding will allow the files to be used more easily to pull up selected groups of data, such as all oolitic limestones, all crossbedded sandstones, or all oil stained lithologies, etcetera.

Allowable input lithologies are:

- shale
- fissile shale
- clay shale
- siltstone
- sandstone
- conglomerate
- limestone
- dolomite
- chert
- coal
- underclay

The actual coding for lithologies is as follows: The principal lithology is coded "1". This may be the only entry in the eleven lithology fields. If there is a secondary lithology, it is coded with a "2". A lithology, third greatest in abundance, if present, is coded with a "3".

Allowable input for descriptive modifiers are:

- calcareous
- dolomitic
- carbonaceous
- cherty
- oolitic
- fossiliferous
- crossbedded
- oil stained
- silty
- conglomeritic
- argillaceous (clayey)
- arenaceous (sandy)
- karstic
- vuggy

Coding for lithologic modifiers is as follows: The input forms contain two identical sets of modifiers. One set applies to the primary lithology, the second set to the secondary lithology. Each modifier coded "T" indicates that the marked attribute is present. Any attribute coded "I" indicates that it is increasingly characteristic (or abundant) with depth. Any attribute coded "D" indicates that it is decreasingly characteristic (abundant) with depth. A blank means that this characteristic is not applicable to the associated lithology.

The relationship between the primary and secondary lithologies can be described in terms of the secondary lithology (and possibly a tertiary lithology as well) forming interbeds, partings, or laminae, and the percent of the secondary lithology can be given. These terms are not mutually exclusive. An approximate definition of terms (as used here) is as follows: "interbeds" are strata of substantial thickness laid between, or alternating with, other strata of different composition; "partings" are generally thinner layers of rock occurring within some different rock type which makes up the greater portion of the interval; laminations are less than 10 mm thick, and are often paper-thin. The two digit numeric field, indicates the approximate percentage of the sample composed of the secondary lithology. This would most commonly be used when the secondary lithology occurs as interbeds, and makes up 10 or 20 to 50 percent of the unit. A percentage is never shown when the secondary lithology is less than 10%. When the percentage is left blank, this probably implies that the secondary lithology is not more than 20%. Necessarily, this percentage must be 50% or less; were it greater, it would, of course, be the primary lithology.

We can also provide an indication of each lithology's grainsize or crystal size. Allowable input terms for particle sizes are:

- very fine grained
- fine grained
- medium grained
- coarse grained
- lithographic/microcrystalline
- very fine crystalline
- fine crystalline
- medium crystalline
- coarse crystalline

All of the above are single-digit fields, which will accept as allowable input values, a "1", a "2" or a "3". "1" means that the primary lithology has the indicated size, "2" means that the secondary lithology has the indicated size, and "3" means that the size indicated refers to both the primary and secondary lithologies. As an example, if fine crystalline and medium crystalline are coded "3", coarse crystalline is coded "2", and all other particle-size fields are blank, then the primary lithology is fine-to-medium crystalline, and the secondary lithology is fine-to-coarsely crystalline. Finally, the input fields allow the color of both the primary and secondary units to be input without coding.

Geologic Correlations

All of Kentucky was mapped geologically, by the US and Kentucky Geological Surveys, in approximately the twenty year period from 1960 to 1980. This mapping is presented on 7.5 minute geologic quadrangles at a scale of 1:24,000 (1"=2000'). The availability of this mapping, plus the determination of precise locations of each boring, means that it is not particularly difficult to determine the uppermost consolidated bedrock material determined by the mapping to occur at each boring location. This is being done, and the rocktype is listed in the "bedrock" field. We have also adopted a system recommended by the Committee on Standard Stratigraphic Coding, of the American Association of Petroleum Geologists, which provides a numeric indication of the age of the rock. This is done by means of a three digit code in the "geoage" field. In this code, higher numbers represent older rocks. The first digit is always either a 1, 2 or 3. "1" represents the Cenozoic Era, "2" the Mesozoic Era, and "3" the Paleozoic Era. The second digit indicates the system. In the Paleozoic, for instance, if the second digit is a "3" it represents the Mississippian System; "4" is Devonian, "6" Ordovician, etcetera. The third digit is for the series. In the Mississippian, 2, 3, 8 and 9 are the only valid entries, and they indicate the Chester, Meramec, Osage and Kinderhook Series, respectively. This system alone, however, was not entirely adequate to our needs. In the four Mississippian series discussed above, for example, there are numerous formations, members, and combinations of these, which have been used as mapping units on the geologic quadrangles. Four mapping units in the Kinderhook Series (Sunbury, Bedford-Sunbury, Berea, and Bedford) would all have the geoage 339. In the Chester Series, there are forty different mapping units, all of which would have the same geoage (332). We have, therefore, provided a unique coding by combining a two digit sequence number with the geoage (Pfalzer, 1980). The detailed listing presented in 1980 has been modified slightly, and a revised listing will soon be published (Pfalzer, Hopkins and Tollner, in preparation).

Providing the bedrock as mapped at each boring location represents a good deal of information, and because the detailed mapping is available throughout the entire state, it should be very accurate. Still the information this provides deals entirely with the surficial geology, identifying only the uppermost bedrock material. Through the use of a "stratigraphic control" field, we are attempting to go beyond this, providing, wherever possible, the geologic relationships of each sample in the boring. At this point it becomes necessary to discuss the definitions of soil and bedrock, since geotechnical engineers and geologists use different definitions. For the engineer, the rock materials must be indurated, consolidated materials, in which some cementing of the constituent particles binds the entire mass together, making it hard. Such a material should possess at least some small tensile strength and considerable compressive strength. All unconsolidated materials are grouped as "soils" or "overburden". For the geologist, on the other hand, soil is an unconsolidated material that has enough organic material (nutrients) to grow crops. All of the materials underlying these soils are grouped as rock, and this includes some deep unconsolidated materials. [As engineers, we might point out that we have the public perception on our side, and might argue that John Q. Public better knows a "rock" when he sees one than do the geologists! But be that as it may.] Where the unconsolidated materials are relatively thin, they are ignored for purposes of mapping, and the uppermost hard bedrock unit is shown in these areas. In areas where unconsolidated materials are thicker, however, they are shown as the mapping unit. This covers all of far western Kentucky (the Jackson Purchase), where the hardrock units are present only at very great depths. Through the rest of the state, areas along riverbeds commonly have somewhat deep deposits of unconsolidated materials, and these are generally mapped as alluvium, terrace deposits, or locally as river-deposited gravels. Areas of large old lakebeds (remnants of the ice age) may also have relatively deep deposits of soft unconsolidated sediments, and they are shown in a "lacustrine deposits" mapping unit.

In our "depth to bedrock" field, we have consistently used the definition of the engineer for bedrock. In the input for "stratigraphic code", however, it must be recognized that the "bedrock mapping units" may refer to either consolidated or unconsolidated deposits. Where possible, I have attempted to indicate formational changes with depth (although at my skill level, and considering the complexities of the naturally occurring materials, of the mapping, and in the definition of units, this frequently cannot be done).

The allowable input values for stratigraphic control are:

- •A Soil above a bedrock mapping unit
- •B "Alluvium" mapping unit
- •C "Loess" mapping unit
- •D "Lacustrine Deposits" mapping unit
- •E "Terrace Deposits" mapping unit
- •F Glacial "Outwash" mapping unit
- •G The last soil sample in holes without rockcores code as "G" to ask for bedrock mapping unit. This will allow research of soil-bedrock (parent material) relationships
- •J Talus materials
- •K Boulders
- •L Unconsolidated Mississippi Embayment units
- •M Consolidated bedrock mapping unit
- •N Consolidated bedrock unit below the mapping unit
- •O possibly below the surficial geology mapping unit
- •P probably below the surficial geology mapping unit
- •R Rock outcrop/open face log
- •S Sounding
- •X artificial fill of soil and/or rock (such as embankments or levies)
- •Y artificial fill containing significant quantities of organic materials
- •Z artificial fill containing trash (landfills)

The use of these codes allows sampled materials to be identified as naturally occurring deposits (of either soil or rock) or as an artificial fill placed by the activities of man (codes "X", "Y" and "Z"). Codes "B" through "F" are for samples of unconsolidated materials which lie within (or below) the bedrock mapping unit. These codes identify the mapping unit and, consequently, the mapping unit does not need to be entered manually into the bedrock field. Code "L" means that the mapping unit is an unconsolidated material, one of several units occurring in the Mississippi Embayment (Jackson Purchase Region). The specific unit must be entered into the "bedrock" field. Code "A" is for thin "soil" materials occurring above the bedrock mapping units. The underlying bedrock is not identified for these samples, except in the deepest sample of the boring which is coded "G". Code "K" is for intervals containing boulders, but the bedrock type is not identified (we do not believe we could consistently do so with sufficient accuracy). Code "M" is for "rock" units falling within the mapping unit; the mapping unit must be specifically identified in the "bedrock" field. Code "N" is for "rock" samples from a unit definitely below the mapping unit. This rock unit, which is probably used as a mapping unit in some other parts of the map, but not at the boring location because there it is overlain by some higher (younger) lithology, is entered into the "bedrock" field. There will be many cases where we are uncertain of the boundaries between formations (or other mapping units). Codes O and P allow for this uncertainty, allowing us to indicate that we are possibly (code O) or probably (code P) below the mapping unit.

Obviously, we would prefer that providing this "stratigraphic control" would not be quite so complicated. On the other hand, this system does provide us with a great deal of flexibility. Logs retrieved from the database can be printed showing the stratigraphic relationships or, when preferred, these assumed relationships need not be shown. We have not yet developed retrieval capabilities beyond simple lists. However, we believe we can develop printouts representing drill logs, and including all of the input data. An example log is presented in figure 1. These logs could be produced in either English or SI units.

REFERENCES:

Pfalzer, W.J. (1980)
Input Guide: Kentucky Soils Data System, Department of Transportation, Bureau of Highways, Division of Research, Lexington, Ky., 42 pp.

Pfalzer, W.J., and Hensley, L.B. (1981)
Retrieval Guide: Kentucky Soils Data System, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, Ky., 46 pp.

Pfalzer, W.J., Hopkins, T.C., and Tollner, Neil (in preperation)
Geotechnical Engineering Data, Kentucky Transportation Center, College of Engineering, University of Kentucky, Lexington, Ky.

Breckinridge County, Kentucky
latitude : 37° 46′ 42.29″
longitude : 86° 28′ 21.25″
location accuracy : ± 100 feet

Hardinsburg quadrangle (# 1232)
bench mark datum
(elevations are in feet above sealevel)
depth to water 17.7′ = elevation 585.7

SOIL

elev	depth	AASHTO Unified	grainsize no 4	no 10	no 40	no 200	.002 mm	SPG	LL PL	Sample # stratigraphy
603.4										
	3-5	A-4(3) ML-CL		100	98.8	86.5	9.3	2.66	26 21	1 alluvium
	8-10	A-4(3) ML-CL		100	99.8	81.3	15.9	2.66	26 20	2 alluvium
	13-15	A-7-6(14) CL	85.3	83.9	81.0	64.9	33.9	2.71	44 19	3 alluvium
585.7	17.7									

ROCK

elev	depth	lithologic description	SDI	JS	core runs RQD	% Rec	Sample # stratigraphy
585.7	17.7	Shale, black, argillaceous	14	1	0	88	4 Big Clifty
580.7	22.7	Clay shale, gray, sandy					5 Big Clifty
579.8	23.6	Sandstone, gray, fine grained			64	96	6 Big Clifty
576	27						

strength tests

sample #	type tests	c	phi	M.C.	wet density	confining pressure
	none					

base of core runs

elevation	581.6	576
depth (feet)	21.8	27

location of SDI and jar slake tests

elevation	584.2
depth (feet)	19.2

Figure 1

SITE CHARACTERIZATION AIDED BY EVALUATION OF PUMPING TEST DATA ON ENVIRONMENTAL REMEDIATION PROJECTS

Barry K. Thacker, PE
Geo/Environmental Associates, Inc., Knoxville, Tennessee 37930

Abstract. Three examples are presented where analytical evaluation of pumping test data aided in the calibration of site conceptual models on environmental remediation projects. The sites are located in the steeply dipping, karst geologic setting of East Tennessee. Based on interpretation of the data, the sites behave as homogeneous confined aquifers with impermeable boundaries. The method for identifying apparent locations of the boundaries from drawdown-log time data is discussed as well as site reconnaissance used to confirm the presence of these boundaries.

INTRODUCTION

The first step in the regulatory driven process of environmental remediation is to gather adequate data to develop a site conceptual model. Remedial design and implementation do not begin until conditions at the site are understood well enough to "calibrate" the model. Because groundwater flow typically represents the most likely pathway for contaminant migration, understanding hydrogeologic conditions is a key aspect of the model. The purpose of the site conceptual model approach is to enable data to be assimilated in a useable manner so that predictions can be made before the next step in the process begins. The conceptual model is then updated as additional data are gathered during later phases of the process. The goal in predicting performance is to avoid unnecessary and costly actions. The problem with this approach is that no site can be fully characterized. Human tendency is to gather more data to qualify and quantify site conditions. As a result, the remedial design and implementation phases are often postponed indefinitely.

In response to the demand for characterizing environmental sites, many exotic technologies have been developed that employ lasers, radar, satellite imaging, and remote sensing. Some of these technologies are so revolutionary that the inventors cannot divulge the basis for their methods for proprietary reasons. These exotic methods can generate thousands of individual data points on site conditions, but they often provide little information on site behavior.

You can often tell more about the behavior of a site by observing its response under stress than by gathering and analyzing individual data points. A simple method for stressing a site is to perform a pumping test. Observing and analyzing behavior during the test will provide valuable data that will help in the calibration of the site conceptual model as described by the following examples.

GEOLOGIC SETTING

East Tennessee is blessed with karst formations that are often steeply dipping to near vertical with some beds being overturned. As part of the Valley and Ridge physiographic province, the strike of the bedrock is typically in a northeast-southwest orientation. Wells that are drilled 50 feet apart can encounter bedrock at a depth of 20 feet at one location and at a depth of 80 feet at the other location. Most of the thick residual soils associated with these karst formations have high clay content. However, due to the presence of relict weaknesses (joints and fractures) left from weathering of the parent bedrock and desiccation, the hydraulic conductivity of the residual clays can vary by several orders of magnitude over short distances.

Typically, groundwater flow is dependent on the fracture patterns in the bedrock, the degree of weathering at the soil/bedrock interface, and bedrock structure. In cases where the potentiometric surface is situated above the soil/bedrock contact and light non-aqueous phase liquids (LNAPLs) are present, the fracture pattern in the residual clay becomes important. In these cases, the potential for migration of the LNAPL is controlled by the relict weaknesses in the clay. Characterization and remediation of environmental sites in East Tennessee must deal with this diversity of conditions.

EVALUATION OF PUMPING TEST DATA

General

According to Freeze and Cherry (1979), "in contamination studies, the use of the pumping test approach is usually inappropriate. It is our opinion that the method is widely overused. Piezometer tests are simpler and cheaper, and they provide adequate data in many cases where pumping tests are not justified." Contrary to the assessment of Freeze and Cherry, at many sites in East Tennessee, piezometer (slug) tests merely provide more inconclusive data points that complicate the calibration of the site conceptual model instead of helping to characterize the site. Slug tests aid in the design of the pumping test and pumping tests aid in site characterization.

At first glance, it would appear that such a complicated hydrogeologic setting as described previously could not be modeled using analytical methods developed for homogeneous confined and unconfined aquifer systems. It might appear that computerized numerical models would be necessary for such sites. However, sites within the steeply dipping karst setting of East Tennessee are often heterogeneous to the point that even the numerical models can never achieve the required detail. Investing in the development of a computer model may not be justified. If boring, well, and piezometer test data indicate that the site can be characterized as "homogeneously heterogeneous", then the use of analytical models can often be justified.

The following examples illustrate an approach for site characterization using analytical interpretation of pumping test data. In each example, impermeable boundaries are identified by a change in the slope of drawdown-log time data as shown on Figure 1. If the change in slope is the result of impact from an impermeable boundary, then the slope of the second straight line segment should be equal to twice the slope of the first straight line segment.

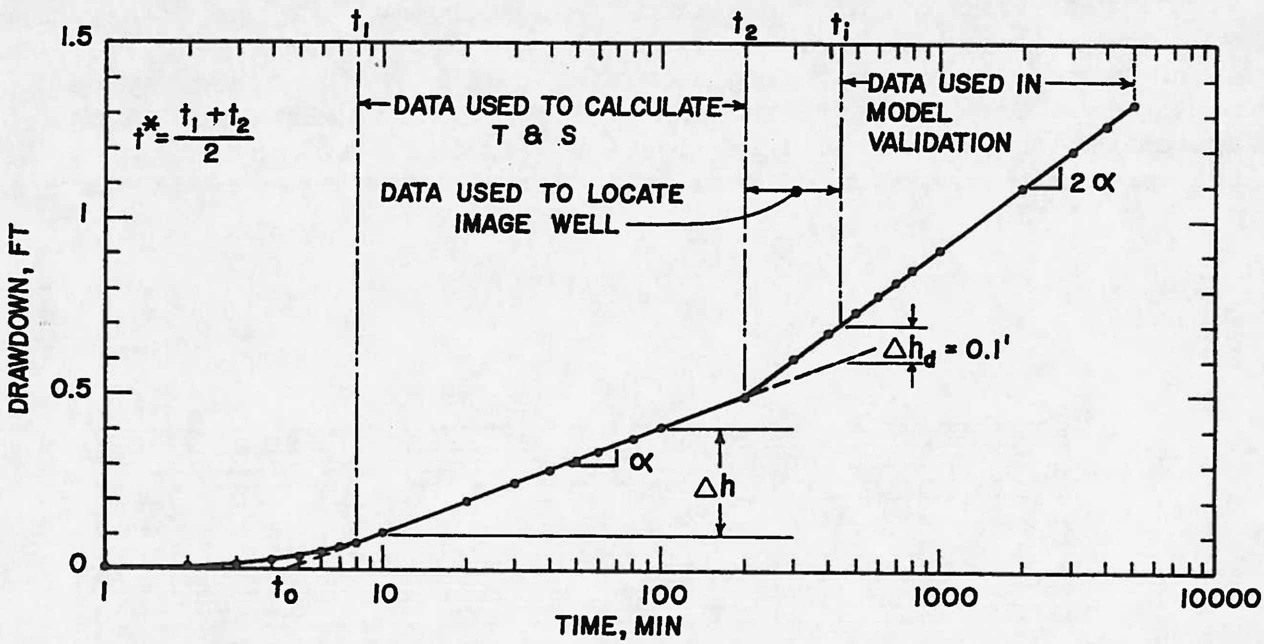


FIGURE 1. Terms and Definitions For Pumping Test Data Reduction at Bounded Sites

Aquifer characteristics for the confined systems can be calculated from the initial straight line portion of the relationship using the approximation of the Theis solution developed by Cooper and Jacob (1946) as follows:

$$T = \frac{2.3\ Q}{4\ \pi\ \Delta h} \qquad (1)$$

$$S = \frac{2.25\ T\ t_0}{r^2} \qquad (2)$$

where:
- T = coefficient of transmissibility, ft^2/min
- S = coefficient of storage (storativity)
- Q = pumping rate, ft^3/min
- r = distance from observation to pumping well, ft
- t_0 = time intercept at zero drawdown axis, min
- Δh = drawdown for one log cycle of time, ft

Errors associated with the Cooper and Jacob approximation can become significant when the distance from the pumping well to the observation well is large and the pumping time is small (Freeze and Cherry 1979). In such cases, curve matching according to the Theis method may be necessary. The percent error associated with the Cooper and Jacob approximation is presented in Figure 2. In the following examples, the percent error is calculated using $u = r^2S/4Tt^*$ where t^* is the time defined on Figure 1.

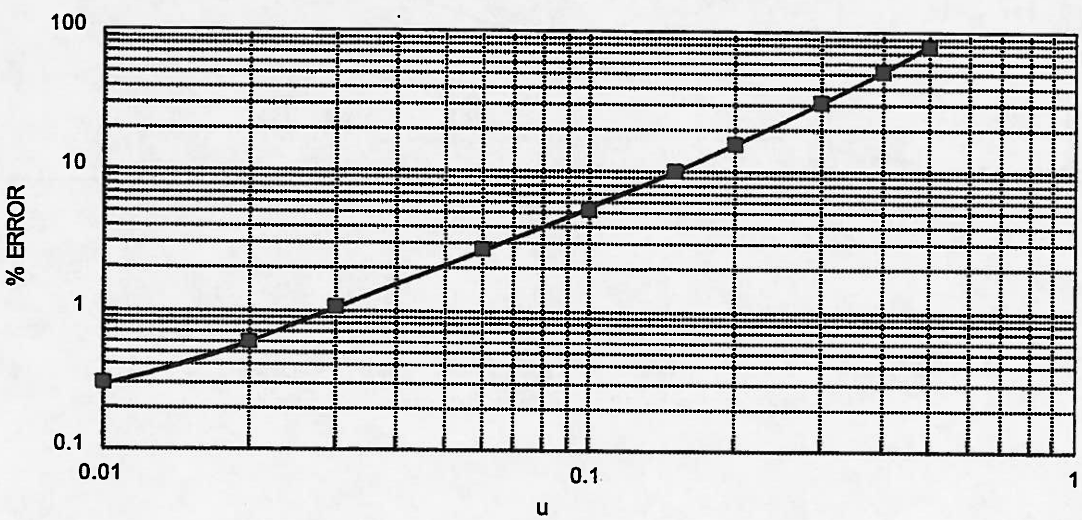


FIGURE 2. % Error for Cooper-Jacob Approximation

The location of the boundary can be delineated using image well theory. According to Walton (1962), the distance from an observation well to an image well can be determined using a time-departure relationship as shown on Figure 1. If data from three observation wells are available, then the location of the image well can be determined by triangulation from the respective observation wells. The image well is an analytical representation of a boundary that is located half-way between the image well and the pumping well and at a right angle to the line connecting the image well and pumping well.

In the following examples, a departure drawdown of 0.1 feet is used in the calculation of the distance from the observation well to the image well as follows:

$$r_i = (4T\ t_i\ u_i\ /\ S)^{1/2} \qquad (3)$$

where:
r_i = distance from observation well to image well, ft
t_i = time (min) where the departure equals 0.1 feet
u_i = value from Theis solution corresponding to the well function, $W(u_i)$
$W(u_i) = (\Delta h_d \times 4\ \pi\ T)\ /\ Q$
Δh_d = departure drawdown, 0.1 feet in this case

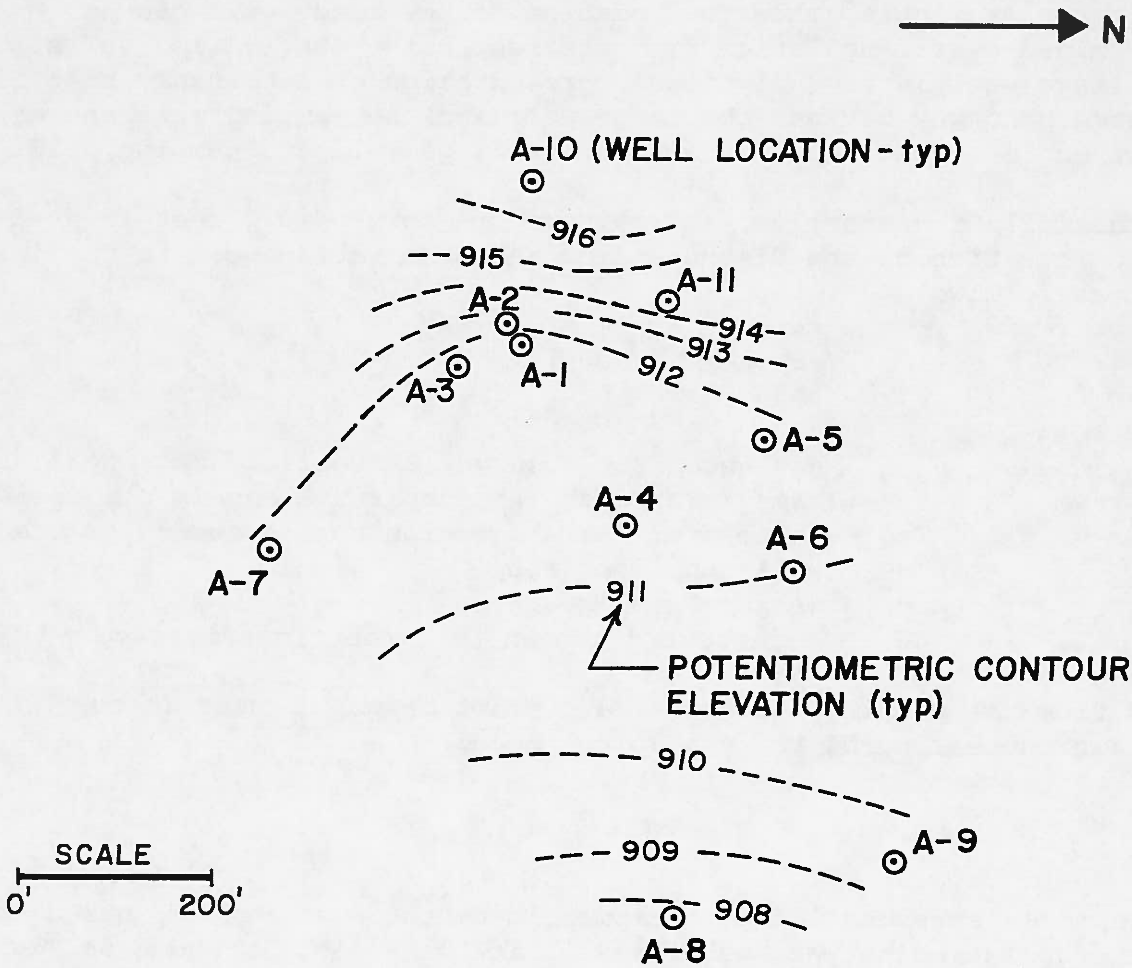
Data from the pumping test after a time of t_i can be used in the validation testing of the analytical model.

Site A

An initial assessment of potentiometric contours at Site A, based only on interpolation between water levels in the wells, is shown on Figure 3. The wells were generally drilled into the upper surface of the bedrock at depths ranging from 30 to 60 feet below ground surface. The potentiometric surface shown on Figure 3 ranges from 17 to 25 feet below ground surface. A pumping test was performed by withdrawing groundwater from well A-1. The response to pumping was measured continuously in wells A-2 through A-5 as shown on Figure 4. Aquifer characteristics calculated from the initial straight line segment of the drawdown-log time relationships are summarized in Table 1.

TABLE 1. Summary of Aquifer Characteristics for Site A

Characteristic	Value
Pumping Rate	0.668 ft^3/min
T	0.20 ft^2/min
S	0.0018
% Error	7%



NOTE: Potentiometric contours are interpolated from water levels measured in wells

FIGURE 3. Initial Interpolation of Potentiometric Contours on Plan View of Site A (Before Evaluation of Pumping Test Data)

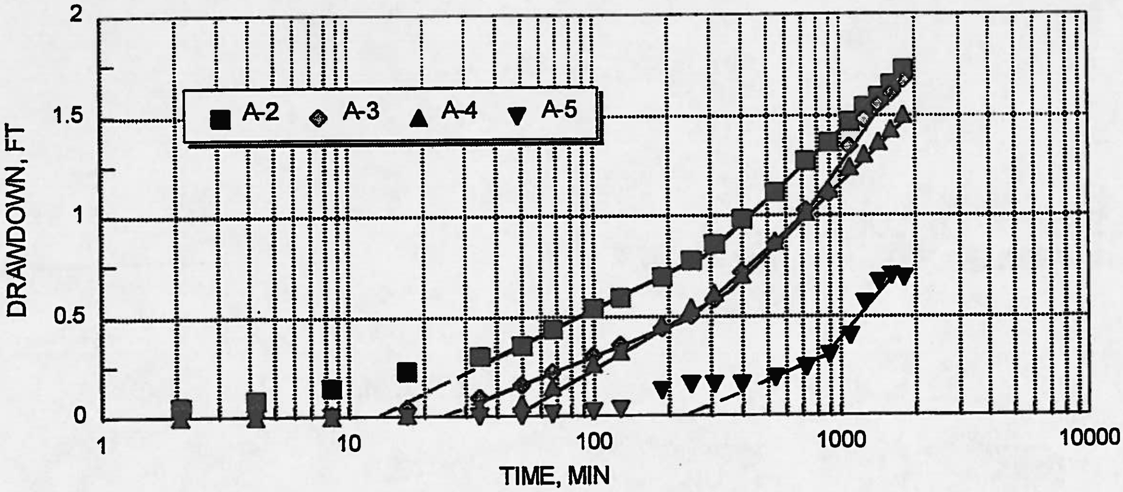


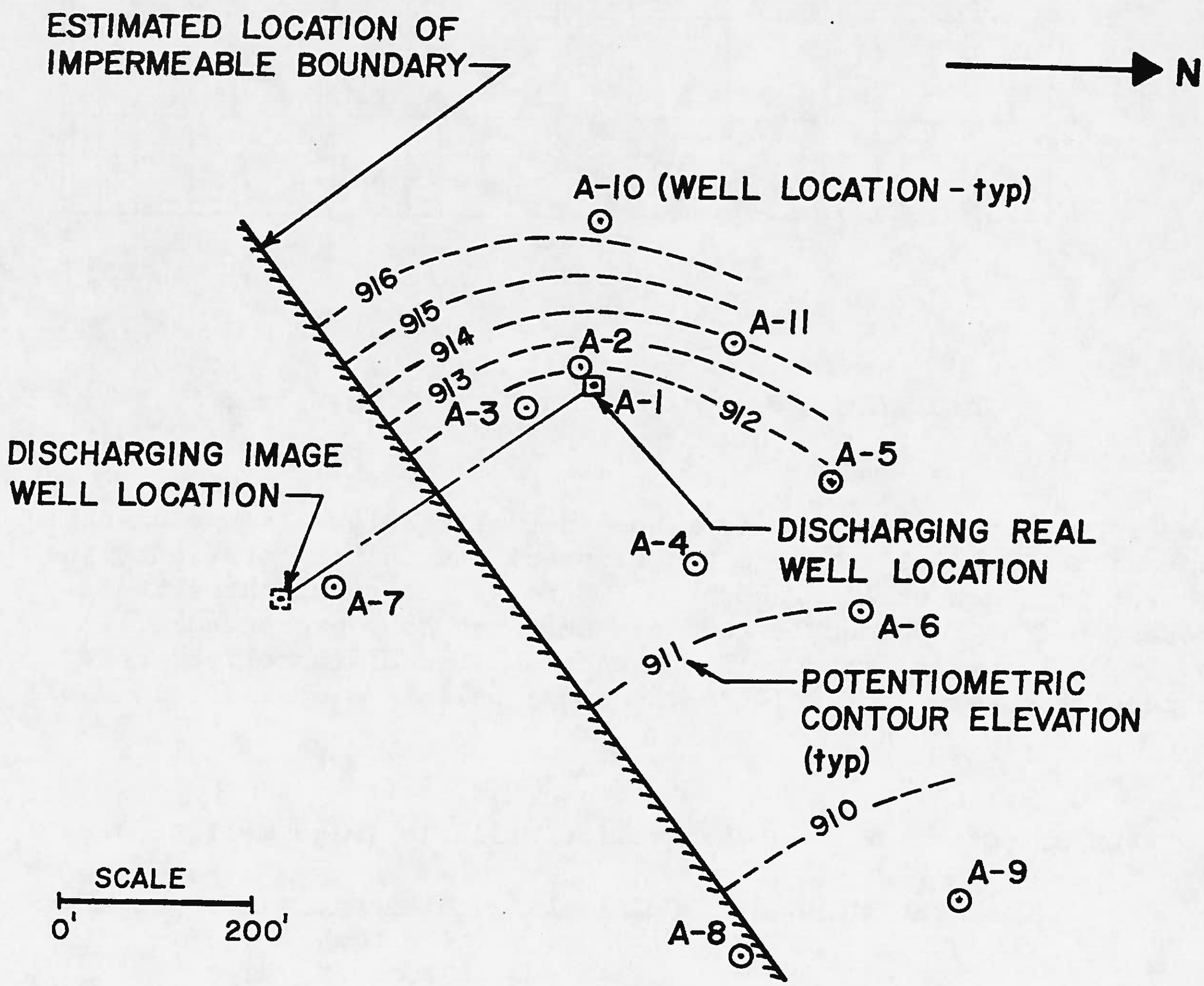
FIGURE 4. Pumping Test Data From Site A

The slope of the departure from the initial straight line segment is approximately equal to twice the slope of the initial straight line segment for each well as shown on Figure 4. This indicates the presence of an impermeable boundary that can be represented analytically by an image well. The calculated distance from the respective observation well to the image well is summarized in Table 2.

TABLE 2. Distance From Observation Wells To Image Well At Site A

Observation Well	Calculated Distance to Image Well
A-2	375 feet
A-3	349 feet
A-4	433 feet
A-5	586 feet

Based on the locations of the observation wells and the calculated distances to an image well, an apparent impermeable boundary was located as shown on Figure 5. Review of the published geologic quadrangle for the area of Site A shows that the location and orientation (i.e. in a northeast-southwest direction) of the apparent impermeable boundary coincides with a contact between two geologic formations. Evaluation of boring data from wells A-7 and A-8 shows a soil-bedrock contact that is about 20 feet deeper than the adjacent wells at the site which helps to confirm the location of the apparent boundary.



NOTE: Potentiometric contours are interpolated from water levels measured in wells and impermeable boundary location estimated from pumping test data.

FIGURE 5. Revised Interpolation of Potentiometric Contours on Plan View of Site A (After Evaluation of Pumping Test Data)

The model is validated using the aquifer characteristics and image well location as determined from data obtained early in the pumping test to predict the drawdown at the end of the pumping test. Figure 6 compares the predicted drawdown levels with the actual drawdown at the end of the test for the applicable wells that were monitored. Based on the information from the pumping test, a revised interpolation of the potentiometric surface is presented on Figure 5. Elevated benzene concentrations measured in well A-9, as compared to an absence of benzene in well A-8, helps to confirm the validity of the revised model.

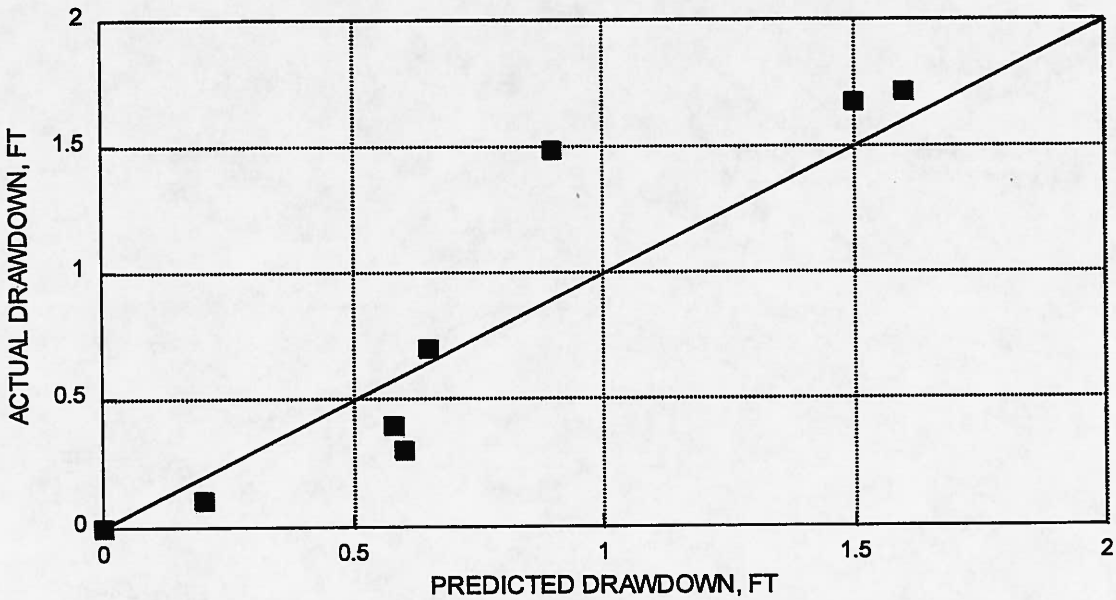
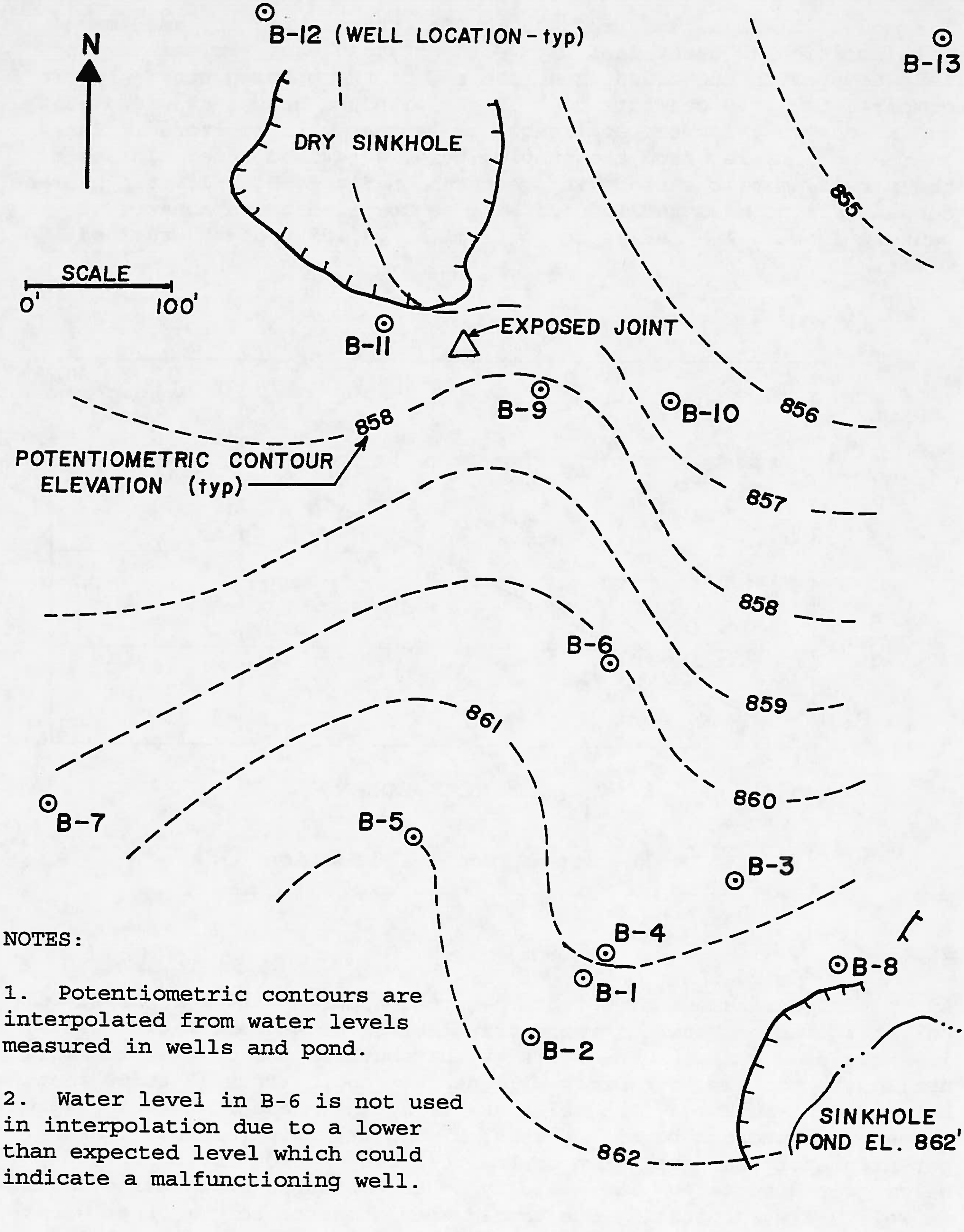


FIGURE 6. Site A Model Validation Data

Site B

An initial assessment of potentiometric contours at Site B, based only on interpolation between piezometric levels in the observation wells, is shown on Figure 7. The wells at the southern end of the site were drilled to the top of bedrock with depths ranging from 35 to 50 feet below ground surface. The wells at the northern end of the site were screened in bedrock because the holes were dry upon refusal. The potentiometric surface shown on Figure 7 ranges from 15 to 30 feet below ground surface. The validity of the measured piezometric level in well B-6 was initially questioned when compared to the other water levels. As a result, the piezometric level in B-6 was initially omitted from estimates of the potentiometric surface at the site.



NOTES:

1. Potentiometric contours are interpolated from water levels measured in wells and pond.

2. Water level in B-6 is not used in interpolation due to a lower than expected level which could indicate a malfunctioning well.

FIGURE 7. Initial Interpolation of Potentiometric Contours on Plan View of Site B (Before Evaluation of Pumping Test Data)

After this initial assessment was made, a pumping test was performed by withdrawing groundwater from well B-1. The response to pumping was measured continuously in wells B-2 through B-4 as shown on Figure 8.

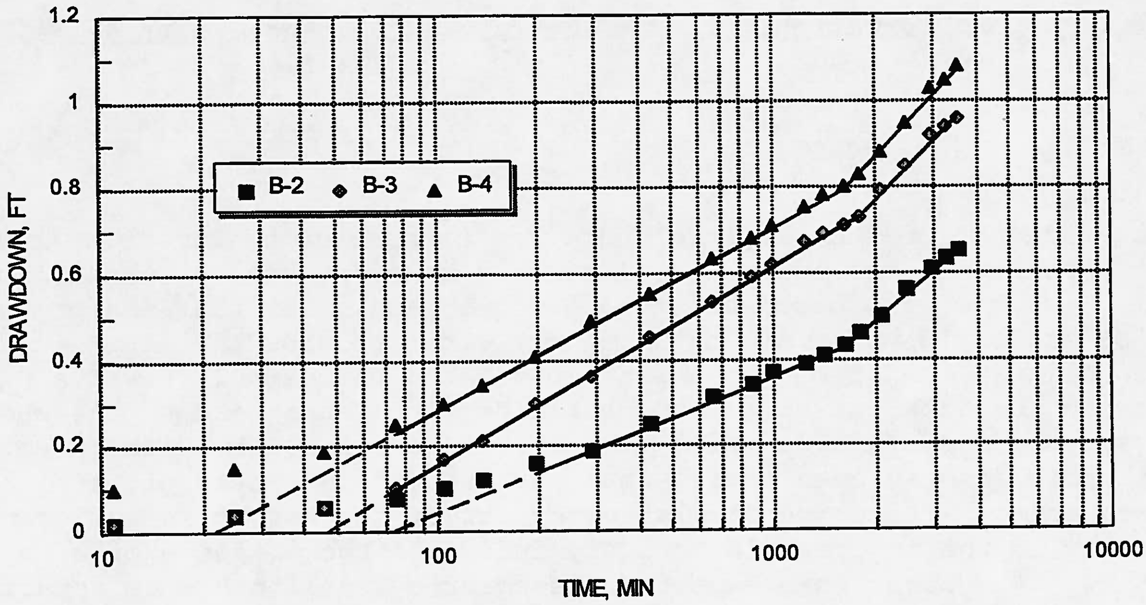


FIGURE 8. Pumping Test Data From Site B

Aquifer characteristics as calculated from the initial straight line segment of the drawdown-log time relationships are summarized in Table 3.

TABLE 3. Summary of Aquifer Characteristics for Site B

Characteristic	Value
Pumping Rate	0.174 ft^3/min
T	0.078 ft^2/min
S	0.0035
% Error	1%

The slope of the departure from the initial straight line segment is approximately equal to twice the slope of the initial straight line segment for each well as shown on Figure 8. This indicates the presence of an impermeable boundary that can be represented analytically by an image well. The calculated distance from the respective observation well to the image well is summarized in Table 4.

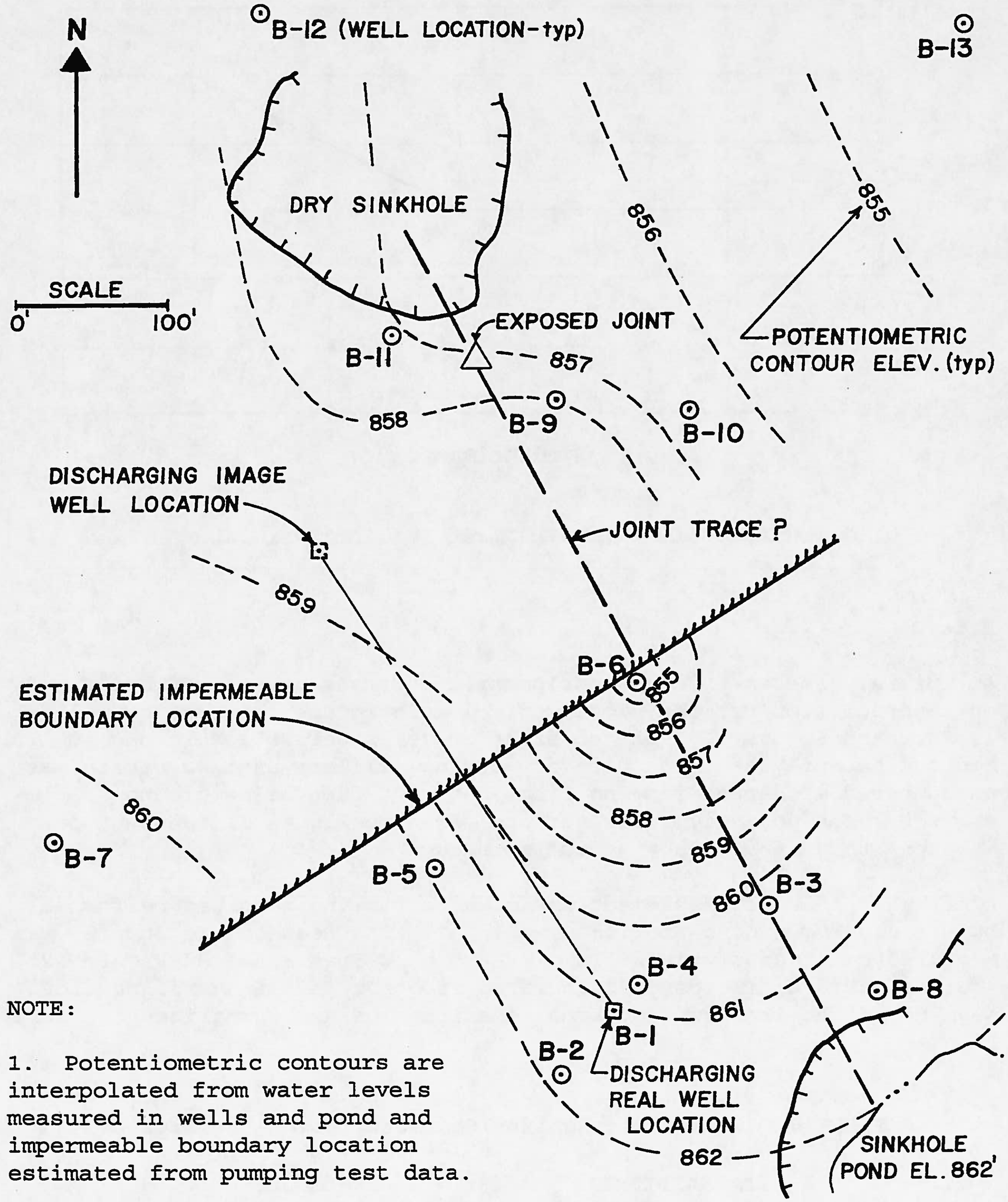
TABLE 4. Distance From Observation Wells to Image Well at Site B

Observation Well	Calculated Distance to Image Well
B-2	372 feet
B-3	365 feet
B-4	353 feet

The location of the image well and the corresponding location and orientation of the apparent impermeable boundary are shown on Figure 9. Review of the published geologic quadrangle for the area of the site and field measurements at nearby outcrops show that the location and orientation of the apparent impermeable boundary (i.e. in a northeast-southwest direction) coincide with the strike at the base of a steeply dipping syncline. Moreover, well B-6 aligns with three sinkholes and an open cavity that was exposed in a test pit excavation. The perpendicular alignment of the sinkholes and open cavity to the impermeable boundary indicates the presence of a major joint. The lower than expected piezometric level in B-6 is apparently the result of drainage into this joint.

The model is validated using the aquifer characteristics and image well location as determined from data obtained early in the pumping test to predict the drawdown at the end of the pumping test. Figure 10 compares the predicted drawdown levels with the actual drawdown at the end of the test for the wells that were monitored during the pumping test. Based on the information from the pumping test, a new interpolation of the potentiometric surface is presented on Figure 9.

An additional pumping test and a dye trace study performed for another project on the north side of the apparent impermeable boundary are indicative of unconfined aquifer conditions. The direction of groundwater flow in the unconfined system on the north side of the boundary is along strike (i.e. to the northeast). The irregular potentiometric surface in the area of B-9, B-10, and B-11 is probably caused by infiltration into the dry sinkhole. The presence of the impermeable boundary explains why the area to the south of the boundary behaves as a confined system with the potentiometric surface sloping in the direction of the joint trace and not along strike. Finally, the boundary in the direction of groundwater flow explains why the sinkhole at the southeast corner of the site holds water.



NOTE:

1. Potentiometric contours are interpolated from water levels measured in wells and pond and impermeable boundary location estimated from pumping test data.

FIGURE 9. Revised Interpolation of Potentiometric Contours on Plan View of Site B (After Evaluation of Pumping Test Data)

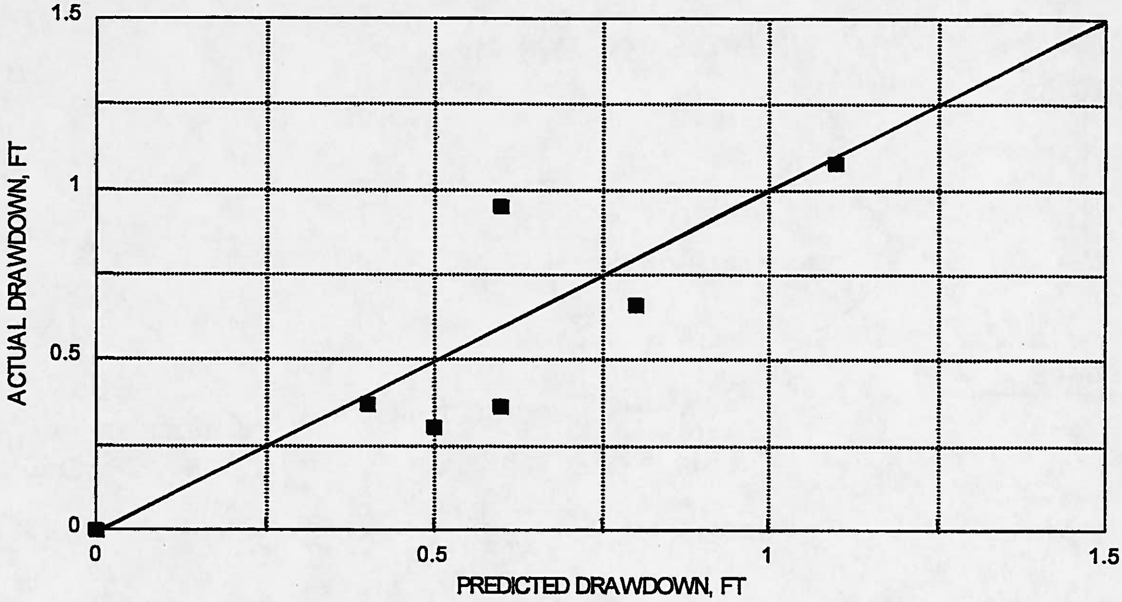


FIGURE 10. Site B Model Validation Data

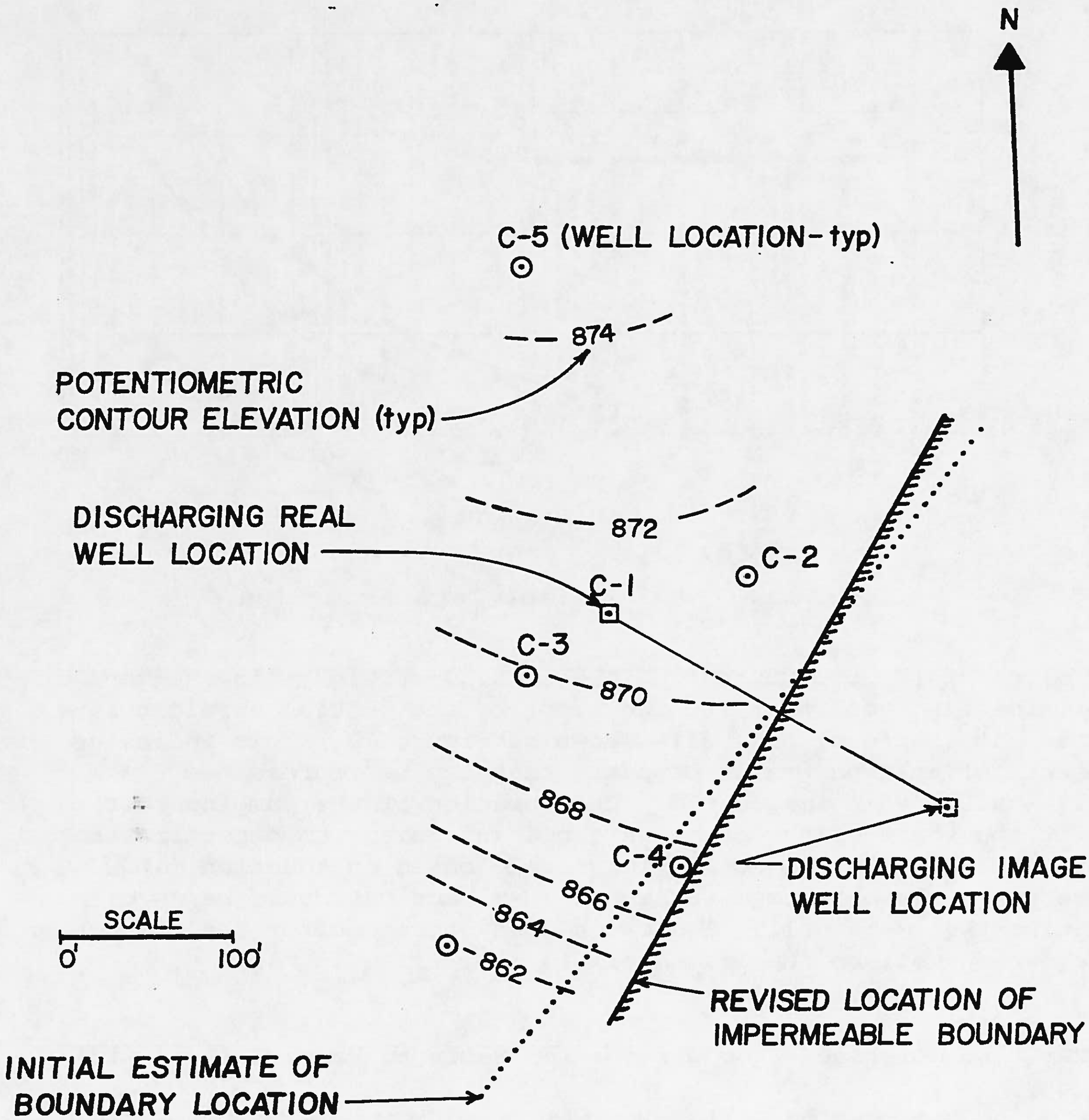
Site C

An initial assessment of potentiometric contours at Site C, based only on interpolation between piezometric levels in the observation wells, is shown on Figure 11. The observation wells were all screened in bedrock because the holes were dry during drilling until a cavity was encountered at depths ranging from 70 to 100 feet below ground surface. The potentiometric surface shown on Figure 11 ranges from about 55 to 65 feet below ground surface.

After this initial assessment was made, a pumping test was performed by withdrawing groundwater from well C-1. The response to pumping was measured continuously in wells C-2 through C-5 as shown on Figure 12. Aquifer characteristics as calculated from the initial straight line segment of the drawdown-log time relationships are summarized in Table 5.

TABLE 5. Summary of Aquifer Characteristics for Site C

Characteristic	Value
Pumping Rate	0.133 ft^3/min
T	0.051 ft^2/min
S	0.0030
% Error	8%



NOTE: Potentiometric contours are interpolated from water levels measured in wells.

FIGURE 11. Interpolation of Potentiometric Contours on Plan View of Site C

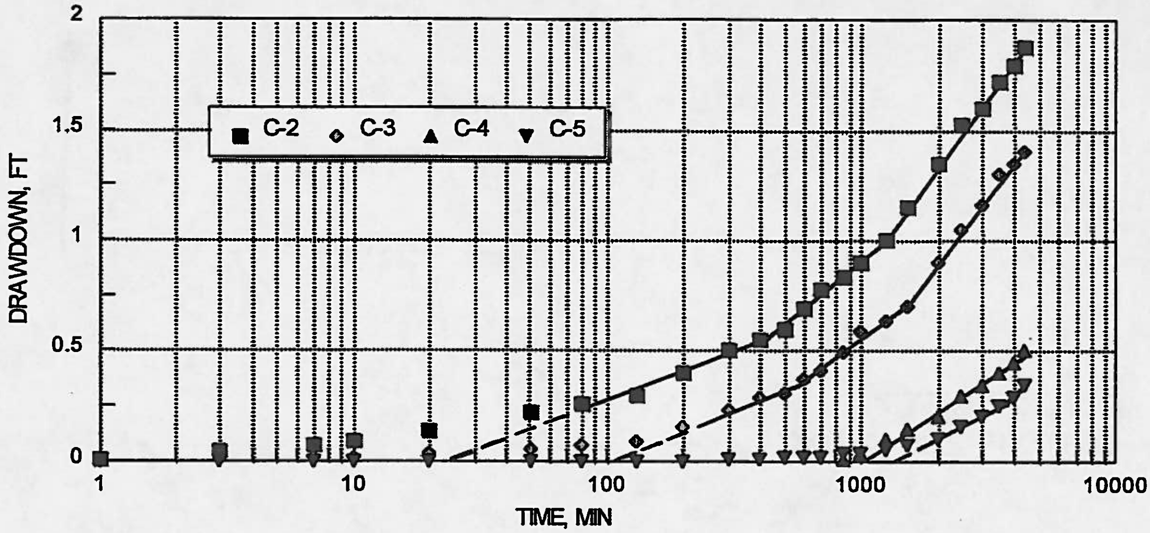


FIGURE 12. Pumping Test Data From Site C

The slope of the departure from the initial straight line segment is approximately equal to twice the slope of the initial straight line segment for three of the wells shown on Figure 12. This indicates the presence of an impermeable boundary that can be represented analytically by an image well. The duration of the pumping test and/or the shape of the curve were not sufficient to determine the distance from well C-4 to the image well based on equation (3). Therefore, only data from wells C-2, C-3, and C-5 could be used in locating the image well. The calculated distance from the respective observation well to the image well is summarized in Table 6.

TABLE 6. Distance From Observation Wells to Image Well at Site C

Observation Well	Calculated Distance to Image Well
C-2	175 feet
C-3	235 feet
C-5	390 feet

The location of the image well and the corresponding location and orientation of the apparent impermeable boundary were initially estimated as shown on Figure 11. Although the drawdown data from well C-4 could not be used to locate the image well, its straight line slope at approximately the slope of departure in the other observation wells indicates that it is located at an equal distance from the pumping well and the image well. Furthermore, drawdown measurements from well C-4 indicate that it is located on the same side of the boundary as the pumping well. The location of the image well and the orientation of the boundary were then adjusted accordingly as shown on Figure 11 (i.e. well C-4 and the pumping well are located on the same side of the boundary and the distance from well C-4 to the pumping well is the same as the distance from well C-4 to the revised image well).

Review of the published geologic quadrangle for the area of the site was inconclusive with regard to the cause of the apparent impermeable barrier. Contacts between the steeply dipping formations at the site were only estimated on the geologic quadrangle due to the relatively thick overburden, a lack of outcrops, and extensive development. Furthermore, no borings or wells were drilled on the southeast side of the apparent impermeable boundary for comparison and contrast. However, the apparent impermeable boundary is oriented in a northeast-southwest direction which coincides with the strike at exposed formations to the east and west of the site area.

The model is validated using the aquifer characteristics and image well location as determined from data obtained early in the pumping test to predict the drawdown at the end of the pumping test. Figure 13 compares the predicted drawdown levels with the actual drawdown at the end of the test for the wells that were monitored. Because none of the wells were installed on the opposite side of the boundary from the site, the initial interpolation of the potentiometric surface shown on Figure 11 appears to be correct.

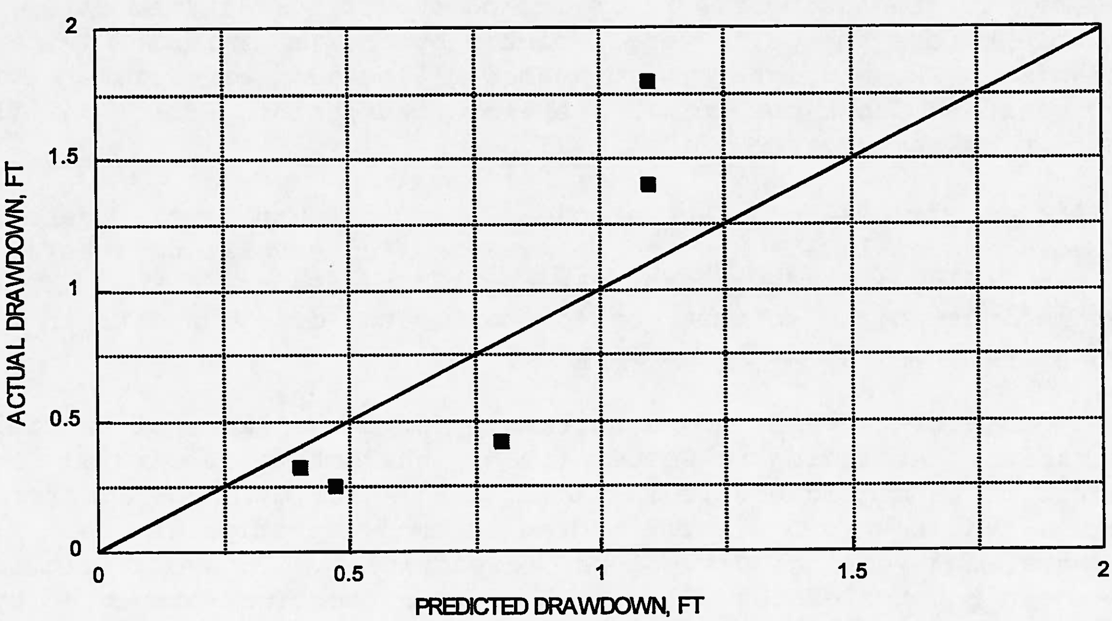


FIGURE 13. Site C Model Validation Data

OBSERVATIONS AND CONCLUSIONS

Observations and conclusions from the three site evaluations are presented as follows:

1. Based on a comparison of measured drawdown to predicted drawdown at the end of the three pumping tests, the mean error ranged from -0.04 feet to 0.06 feet, the absolute error ranged from 0.14 feet to 0.34 feet, and the root mean squared error ranged from 0.19 feet to 0.39 feet. Because no standard protocol is available for modeling validation (Anderson and Woessner 1992), the acceptance criteria is subjective and is based on sensitivity studies performed during the remedial design phase of these projects.

2. Response to pumping in some of the wells indicates the presence of additional impermeable boundaries (e.g. A-3, C-2, and C-3) and a possible constant head boundary or partial leaky aquifer condition (e.g. A-5). Inadequate data were generated during the pumping tests to delineate these features. It is possible that the actual boundaries that are defined are curved instead of being straight as modeled, thus explaining the apparent deviations from the modeled response. The deviations from the initial departure in these respective wells resulted in the greatest errors in the validation testing. However, the validation of the models was still considered to be within acceptable limits.

3. The errors associated with the use of the Cooper and Jacob method at Sites A and C are somewhat excessive. However, similar errors could be expected using the Theis method due to the limited data available before the wells were impacted by the impermeable boundaries. The straightforward method of locating boundaries with the Cooper and Jacob method was therefore the deciding factor in the selection of this method.

4. Natural groundwater fluctuations can impact pumping test data. Therefore, water levels need to be measured for several days before the test and several days after the test to assess the extent of natural fluctuations so that corrections to the drawdown data in response to pumping can be made if needed.

5. No corrections were judged necessary for the effects of partial penetration. According to Fetter (1988), the effect of partial penetration is negligible if $r>1.5b(K_h/K_v)^{1/2}$ where b is the aquifer thickness and K_h and K_v are the hydraulic conductivities in the horizontal and vertical directions, respectively. This criteria is met except for two of the wells. Based on corrections presented by Walton (1962), the effects of partial penetration for the two observation wells closest to the pumping wells are also negligible.

6. Current groundwater references often use the term "impermeable" boundary whereas past references use the term "barrier" boundary. As shown at Site B, the term "barrier" boundary is probably more accurate. As is the case throughout the groundwater field of literature, "impermeable" is a relative term. 95

48
13
35

7. The three examples presented illustrate the importance in obtaining drawdown measurements in all wells at a site even when little or no response is expected. The presence of boundaries can result in relatively large drawdown when none is expected and no drawdown when drawdown is expected. Due to cost constraints, continuous monitoring is not prudent in all wells. However, periodic monitoring of drawdown throughout the test in all wells can provide valuable data in evaluating site conditions. As a minimum, the data can be used in validation testing as shown in the previous examples.

82/13

8. The concept that these steeply dipping karst formations behave as bounded confined aquifers can be rationalized by examining the response of a confined sand aquifer if it could be turned on end and capped with a clay deposit. If the sand aquifer was recharged such that the new potentiometric surface was above the base of the upper clay layer, then the aquifer would be a confined aquifer with boundaries. In the case of the dipping karst formations, they were horizontally bedded at one time. The alternating formations of limestone and shale had varying hydraulic conductivities. In the geologic past, these formations were turned on end and the upper surfaces have weathered to create thick clay deposits. If a source of recharge is present such that the potentiometric surface is situated above the base of the thick clay layer, then these formations behave as confined aquifers with boundaries.

9. One common complaint associated with the use of some of the new exotic site characterization techniques is the element of "black box magic" involved. Analytical evaluation of pumping test data requires a great deal of interpretation which can be subject to the same criticism as the new exotic techniques. However, every groundwater textbook written in the past 40 years contains documentation on the interpretation of pumping test data. Reviewers and users of the interpretations, such as the ones presented herein, can perform their own evaluations using published methods without having to rely on a "black box".

10. Even though pumping tests provided valuable data at the sites presented, some conflicting data had to be resolved. For example, at Site A, little or no drawdown was measured in wells A-10 and A-11 even though drawdown would normally have been predicted. The analytical methods are based on the assumption that the potentiometric surface is flat. At wells A-10 and A-11, the potentiometric surface is comparatively steep, thus violating the assumptions. The lack of

drawdown in these wells during the pumping test could be due to the comparatively steep groundwater surface which could be caused by a localized recharge condition. Nevertheless, these wells are located beyond the area where drawdown is needed in the proposed remediation scheme and therefore the data from these wells are not used in the validation testing.

11. A detailed knowledge of local geologic conditions is essential to the proper interpretation of pumping test data. This point is vividly illustrated by the examples presented in this paper.

ACKNOWLEDGEMENTS

The author expresses his appreciation to Dr. Bob Ullrich, PE from the University of Louisville and Dr. James Smoot, PE from the University of Tennessee for teaching him the magic of pumping test data interpretation. Also, special thanks to Carol Goldinger Ford, PG, John Grabeel, PG, and Steve Blair, PG from Ogden Environmental and Energy Services Company for teaching the author that the earth is still round even when engineers try to simplify it to one or two dimensions.

REFERENCES

ANDERSON, M.P., and WOESSNER, W.W. (1992)
Applied groundwater modeling. Academic Press, San Diego, CA, 381 pp.

COOPER, H.H., and JACOB, C.E. (1946)
A generalized graphical method for evaluating formation constants and summarizing well field data. American Geophysical Union, Vol. 27, pp. 526-534.

FETTER, C.W. (1988)
Applied hydrogeology. Macmillan Publishing Company, New York, 592 pp.

FREEZE, R.A., and Cherry, J.A. (1979)
Groundwater. Prentice-Hall, Inc., Englewood Cliffs, NJ, 604 pp.

WALTON, W.C. (1962)
Selected analytical methods for well and aquifer evaluation. Illinois State Water Survey Bulletin 49, 81 pp.

GEOTECHNICAL CHARACTERIZATION OF THE WASTE PIT MATERIAL FOR THE FERNALD ENVIRONMENTAL MANAGEMENT PROJECT

Mark T. Bowers, PhE, PE
Associate Professor of Civil Engineering
Department of Civil & Environmental Engineering
University of Cincinnati

Dale A. Lutz
Former Graduate Research Assistant
Department of Civil & Environmental Engineering
University of Cincinnati

BACKGROUND

The United States Department of Energy developed the Feed Materials Production Center (FMPC) in 1952 to process high purity uranium metal. The plant is located in Fernald, Ohio, approximately twenty miles northwest of Cincinnati. This facility was operated until 1989 at which time the plant was placed on standby to investigate the environmental impacts from known and potential releases of hazardous and/or radioactive waste. The FMPC was placed on the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) National Priorities List in 1989. A consent agreement was signed soon thereafter by the Department of Energy and the United States Environmental Protection Agency implementing a site-wide remedial investigation/feasibility study. This study included evaluation of cleanup alternatives, public review and comment, and selection of preferred final remedial actions. At the same time, the FMPC was renamed the Fernald Environmental Management Project (FEMP). Under the consent agreement, the FEMP was divided into five CERCLA/RCRA units (CRUs), each responsible for the remediation of a specific area of the site (Stokes, 1995). The subject of this paper is CRU1 which consists of Waste Pits 1 through 6, the Clearwell, and the Burn Pit. Figure 1 is a depiction of the plan layout of the CRU1 study area, while Table 1 contains data on the construction of the pits, their storage volume, contents, and current status (ALTER, 1993).

REMEDIATION PLANS

A number of different methods have been proposed to treat/dispose of the waste materials from the FEMP waste pits including:

- *Vitrification
- *Solidification/Stabilization using cement
- *Encapsulation using polymer binders
- *Thermal processing
- *Drying of the material and compaction in landfill cells at a licensed disposal facility in another state

All of the methods require information concerning the character of the waste stream. This research was undertaken to provide information concerning these wastes, particularly the following:

*Visual/Manual Identification
*Moisture Content
*Grain Size Analysis
  -Sieve Analysis
  -Hydrometer Analysis
*Atterberg Limits (LL, PL, SL)
  -Liquidity Index
  -Activity Index
*pH
*Reaction to hydrochloric acid for carbonate presence
*Reaction to hydrogen peroxide for organic content
*Specific Gravity of Solids
*Standard Proctor
*Unconfined Compression Strength
*Shear Strength
  -Direct Shear
*Permeability

Data was gathered on 55 samples that were taken from drums in 1993. The drums contained samples taken from the waste pits in 1992. Samples from Waste Pits 1 through 4 and the Burn Pit came from monitoring wells that were installed in 1992. Clamshell and backhoe samples were taken to provide the samples for Waste Pits 5 and 6 and the Clearwell. It is recognized that such sampling techniques are not very conducive to providing representative samples. This study was meant to provide an base understanding of the wastes and their relation to terrestrial soils. This study made use of existing archived samples. Subsequent research has made use of sampling techniques such as Shelby tubes and piston samplers for undisturbed material, and split spoon samplers and Vibra-core samplers for disturbed material. ASTM Standards were followed wherever possible.

Preparation to Work with Low-Level Radioactive Waste

Each member of the research team from the Accelerated Life Testing and Environmental Research (ALTER) Facility at the University of Cincinnati was required to complete the following training before he/she could participate in the research:

*General Employee Training for the Fernald Site
*Site Worker (Access) Training
*Chemical Hygiene Training
*Radiation Worker II Training
*Quality Assurance Training
*Respirator Training
*Full Medical Evaluation

This training is required to be reviewed annually in order to maintain access to the site and its laboratories.

Personal Protective Equipment

Since the wastes do contain low-level radioactive material (and particularly Thorium), ALTER personnel were required to don personal protective equipment (PPE) whenever they were involved in any testing of the waste materials. Such PPE may have consisted of any of the following:

*Laboratory coats
*Whole body coveralls
*Rubber boots
*Gloves (usually two pairs of Latex gloves)
*Safety glasses with side shields
*Respirator (either full or half-face)
*Air monitor in the room
*Air monitor worn on the body
*Dosimeters (TLD and sometimes ring)
*Taped openings at wrists

In addition, all of the work completed on dried material was required to be completed within a fume hood equipped with a HEPA (high efficiency particulate air) filter. Both alpha and beta/gamma friskers were placed in the laboratories for monitoring use as well as personal contamination monitors at the main laboratory exits. Radiation Technicians were required to be at hand to render any assistance needed by the ALTER research staff.

GEOTECHNICAL CHARACTERIZATION

This work was completed under a contract entitled the Cooperative Remedy Screening Project (CRSP). The work began with visual/ manual identification of the basic behavior of the wastes. Representative samples of the 55 drums of waste were placed in separate one gallon cans. Each can was opened and the contents were placed in large porcelain bowls. Table 2 lists the results of this portion of the analysis. Once all the samples were identified visually as to their expected USCS classification, the samples were subjected to a battery of basic tests including natural moisture content, mechanical analysis, and Atterberg limits. Seventeen samples were chosen to represent the original 55 samples in further tests including specific gravity of solids, hydrometer, and pH tests. Finally, seven composite samples representing the full range of basic characteristics of the mixed waste were compiled. For these seven composites, optimum moisture, Standard Proctor maximum dry density, unconfined compression strength, consolidated drained shear strength parameters, and the coefficient of permeability were determined. It is noted that this parameters were determined under the assumption that the soils would be disposed as compacted landfill materials following rail transport to an acceptable disposal site.

Table 3 contains a listing of the geotechnical characteristics determined for the various samples. Figure 2 is a plasticity chart of Unified Soil Classifications with FERMCO CRU1 waste pit data superimposed. It is noted that the majority of the samples classify as silts which makes sense given the fact that the materials represent the waste stream from an ore processing facility. Table 4 contains a listing of the mechanical properties of the seven composite CRU1 waste pit sample groups.

Comparison to Expected Behavior of Terrestrial Soils

One may question how well the waste pit materials model terrestrial earth materials in their response to common laboratory tests. The following data presented in Table 5 substantiate the hypothesis that the composite Fernald waste pit materials do respond as terrestrial soils would. These "expected" values were developed using empirical rules that have been developed for terrestrial soils. A few examples will suffice to show that these soils do model naturally occurring soils. The expected optimum moisture contents are approximations derived from the following relationship (Hausmann, 1990):

$$w_{opt\ (standard\ Proctor)} = PL - 5 \text{ when } w_{optimum} = 10\%$$

$$= PL - 2 \text{ when } w_{optimum} = 30\%$$

The expected maximum dry unit weight produced using the compactive energy of the Standard Proctor may be approximated using the following equation (Hausmann, 1990):

$$\text{Max. Dry Unit Wt. (pcf)} = 6.37[20.48 - 0.13(LL) + 0.05(PI)]$$

where LL = the liquid limit
PI = the plasticity index, LL - PL

Figure 3 depicts the moisture-density relationship for one of the nonplastic silt samples. Figure 4 is a graphic depiction of the results of unconfined compression testing conducted on samples of highly plastic silt from the pits. The three samples were compacted to approximately 98% of the maximum dry density by ASTM D698 at near optimum moisture content. Figures 5 and 6 yield further information on the behavior of the highly plastic silt waste as revealed by direct shear testing. The consolidated drained friction angle, corrected to account for minor friction in the machine itself, was 33° with a cohesion of 1.7 psi when compacted at near optimum moisture content and at 98% of the standard Proctor maximum dry density. Figure 6 contains the stress-strain data for a series of three MH samples tested in direct shear. Finally, Figure 7 is a plot of a sample that classified as CL that was tested to determine its permeability coefficient when compacted at approximately 95% of standard Proctor maximum dry density and near optimum moisture content. The graphical depictions represented here all mimic terrestrial soil behavior.

CONCLUSIONS

The results of this geotechnical characterization in the screening project were found to be very useful. Further testing is proceeding at this writing on samples taken using Shelby tubes and split-spoon samplers for the DEEP (Dewatering, Excavation, Evaluation) Project. This new data will further expand the geotechnical database being used to design remedial action efforts and to narrow the disposal options at the Fernald site.

ACKNOWLEDGEMENT

These writers wish to thank the officials of the Fernald Environmental Restoration Management Corporation for their permission to present this information. We acknowledge the willing assistance and contributions of the personnel associated with the Accelerated Life Testing and Environmental Research Facility at the University of Cincinnati, particularly Linda Rieser, Director.

REFERENCES

ALTER (1993)
OU1 CRSP work plan; Accelerated Life Testing and Environmental Research (ALTER) Facility document #CRSP-FO-013, submitted to the Fernald Environmental Restoration Management Corporation, Fernald, Ohio, July.

Hausmann, M.R. (1990)
Engineering Principles of Ground Modification, McGraw-Hill Publishing Company, New York.

Lutz, D.A. (1994)
Geotechnical characterization of waste pit material for the Fernald Environmental Management Project. A thesis submitted to the Department of Civil and Environmental Engineering, University of Cincinnati, in partial fulfillment of the requirements for the degree Master of Science.

Stokes, J.H. (1995)
Long-term durability of cement solidified/stabilized Fernald pit waste. A thesis submitted to the Department of Civil and Environmental Engineering, University of Cincinnati, in partial fulfillment of the requirements for the degree Master of Science.

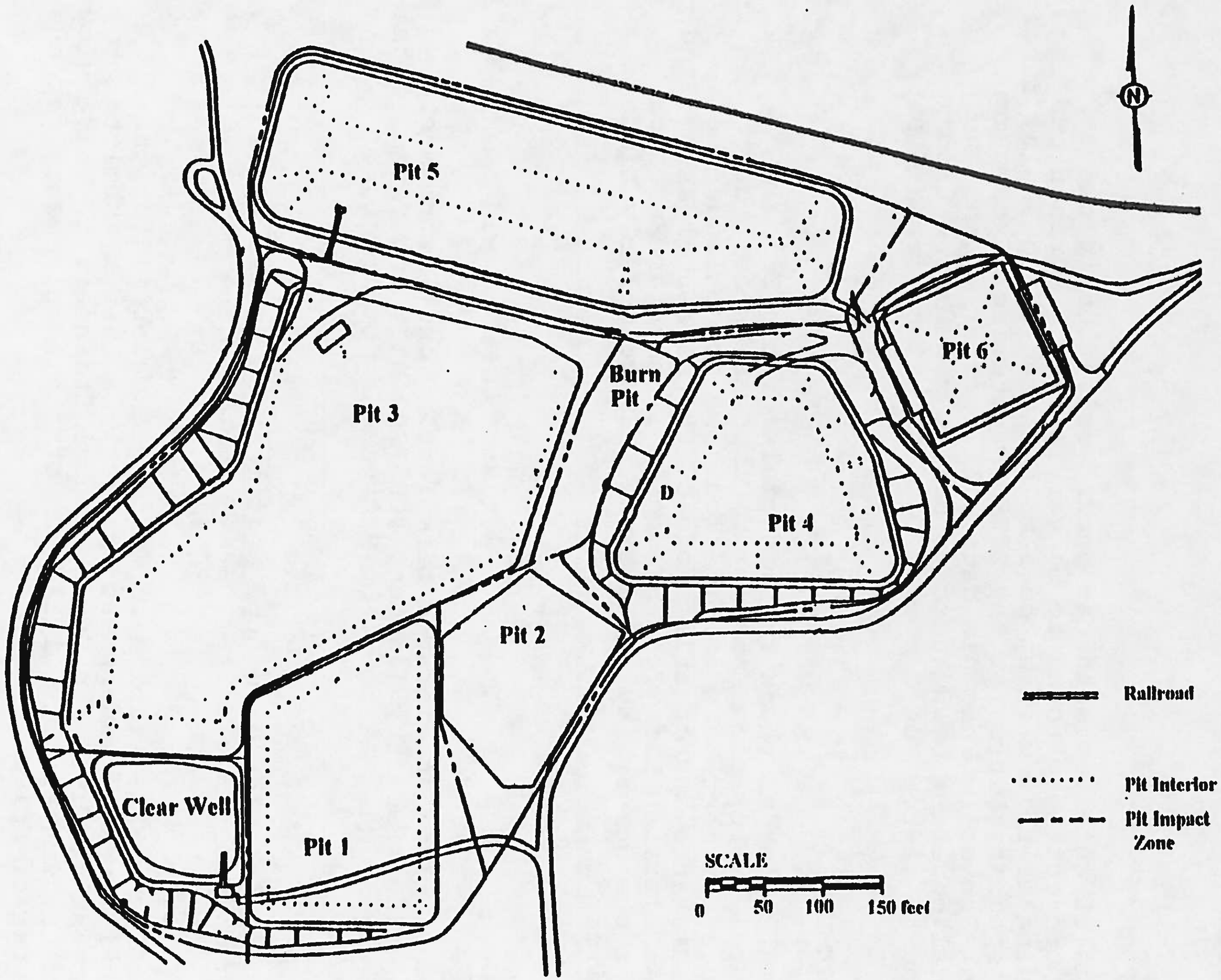


Figure 1. CRU1 Study Area, FEMP Waste Pits (ALTER, 1993).

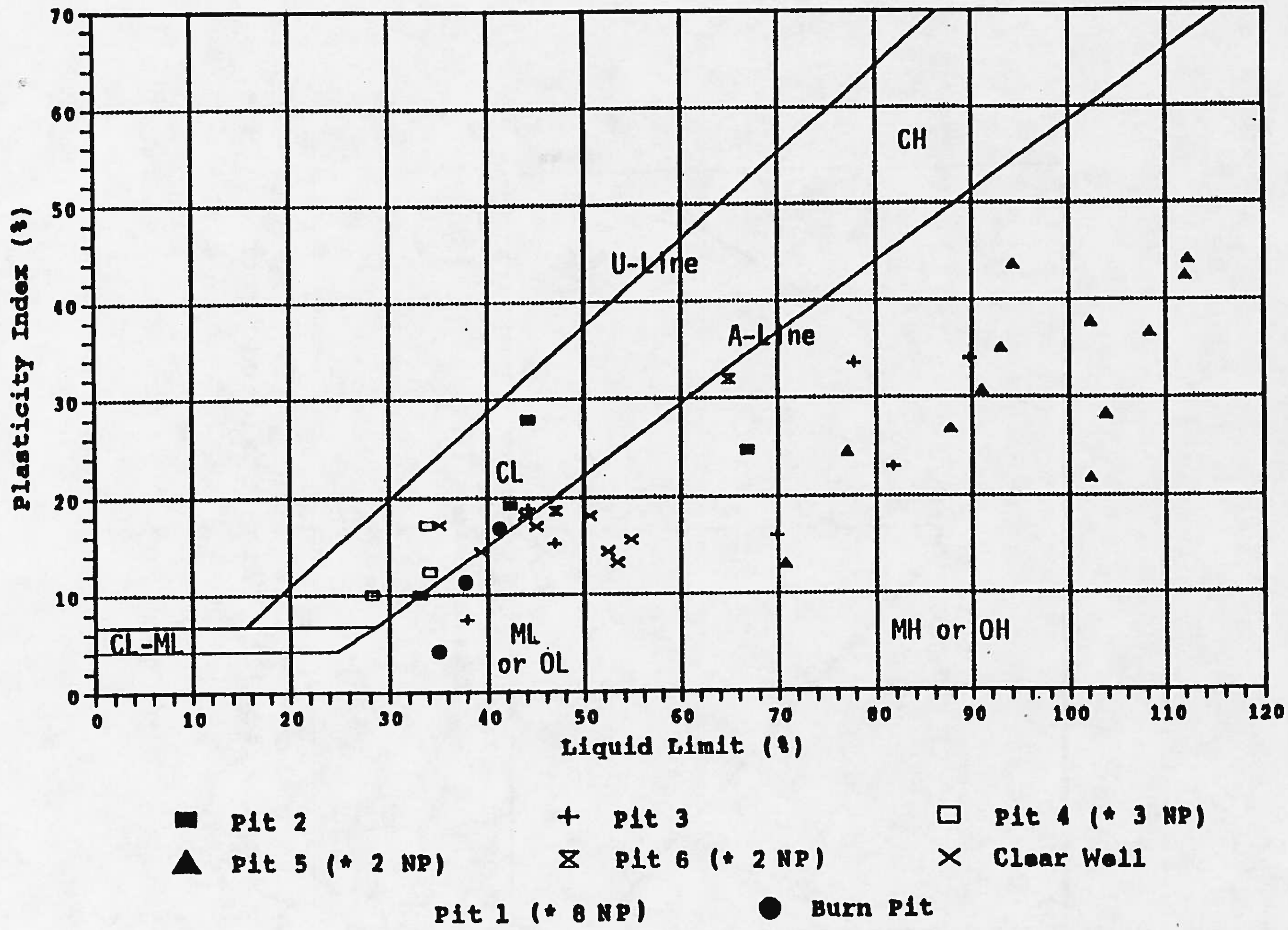


Figure 2. Plasticity Chart of USCS Classifications with FERMCO CRU1 Waste Pit Data Superimposed (Lutz, 1994).

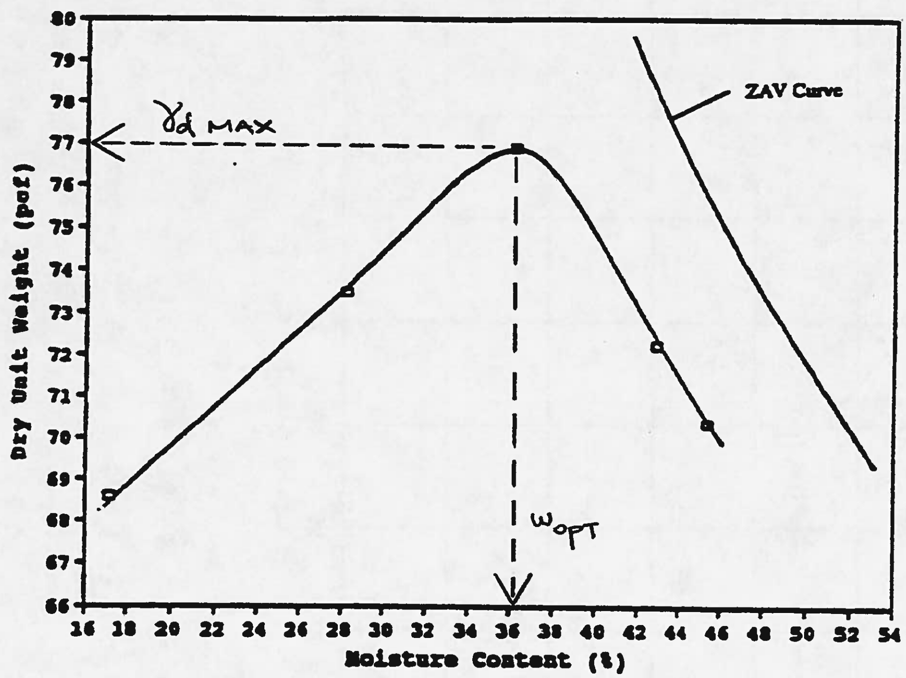
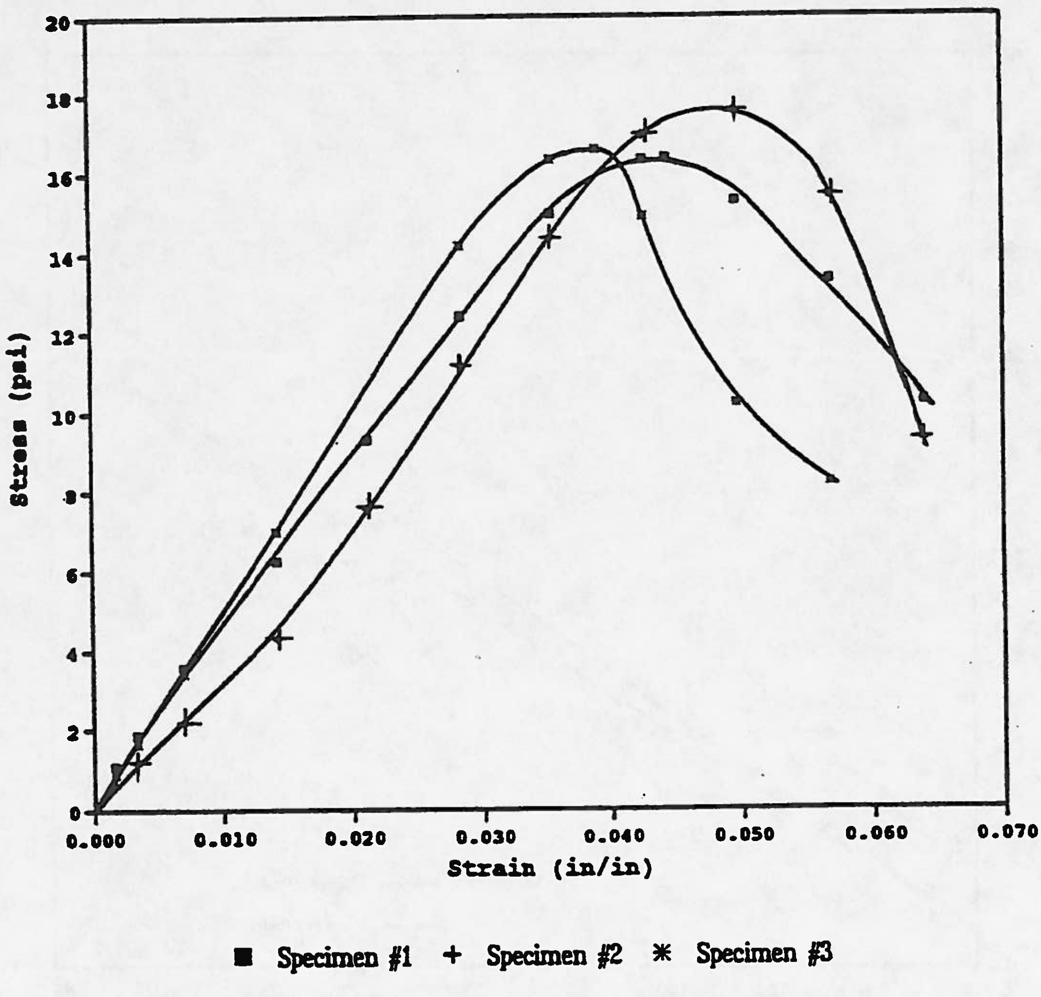


Figure 3. Moisture-density Relationship for Composite Sample of Nonplastic Silt Using Standard Proctor (Lutz, 1994).



(Specimen #1: q_u = 16.37 psi (2357 psf), 97% compaction, w = +1% of w_{opt})
(Specimen #2: q_u = 17.58 psi (2532 psf), 98% compaction, w = w_{opt})
(Specimen #3: q_u = 16.58 psi (2388 psf), 98% compaction, w = -1% of w_{opt})

Average q_u = 16.84 psi (2425 psf)

Figure 4. Unconfined Compressive Stress vs. Axial Strain for Composite Samples of Highly Plastic Silt (Lutz, 1994).

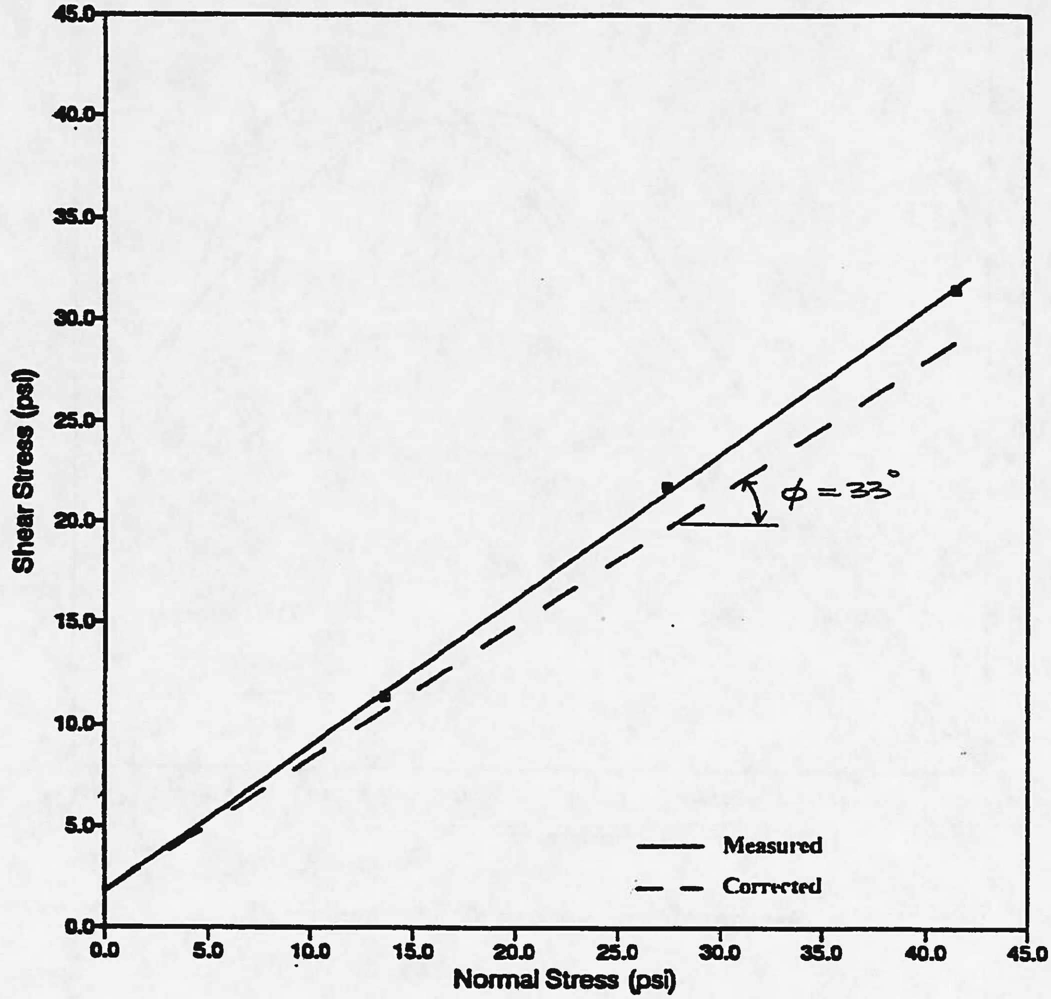


Figure 5. Consolidated Drained Direct Shear Test for Composite Sample of Highly Plastic Silt Compacted at 98% of Standard Proctor Maximum Dry Density and Near Optimum Moisture Content (Lutz, 1994).

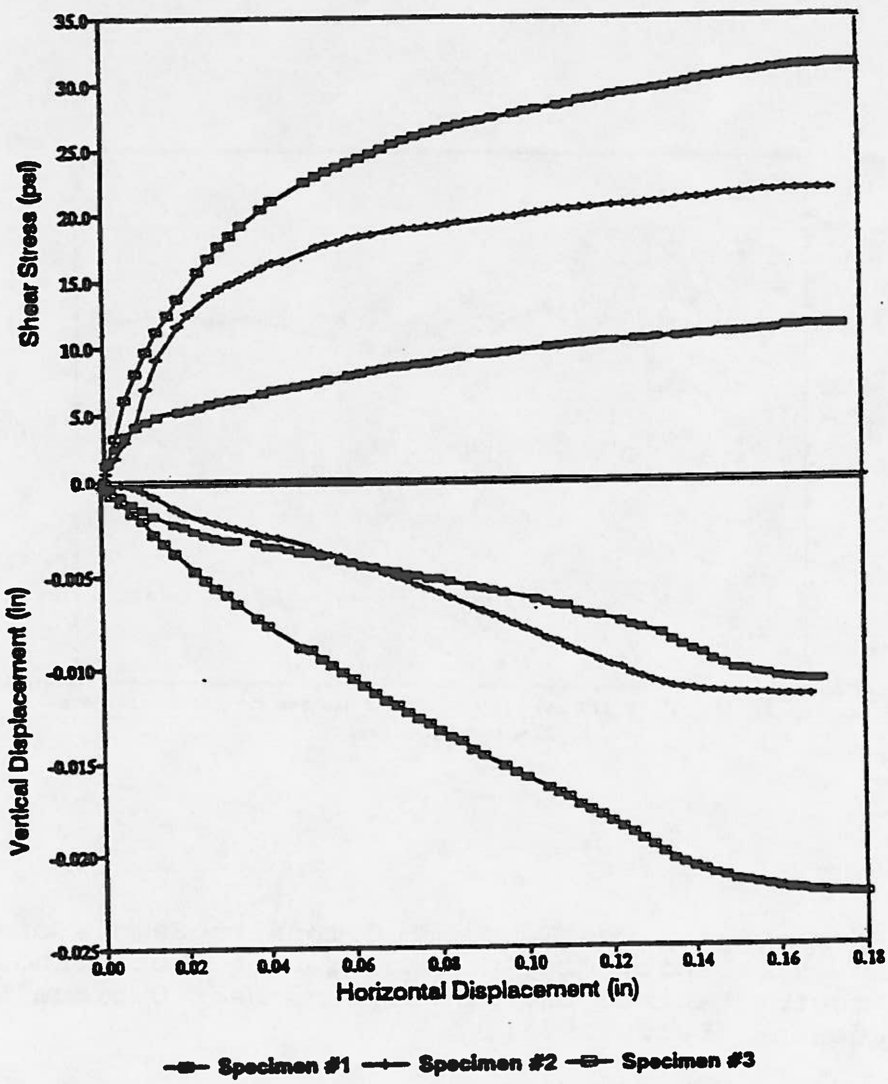


Figure 6. Direct Shear Stress and Strain Relationships for Composite Samples of Highly Plastic Silt (all Specimens at 98% of Standard Proctor Maximum Dry Density and Near Optimum Moisture Content; Strain Rate = 0.0002 in./min; σ_{v1} = 13.67 psi, σ_{v2} = 27.42 psi, σ_{v3} = 41.45 psi); (Lutz, 1994).

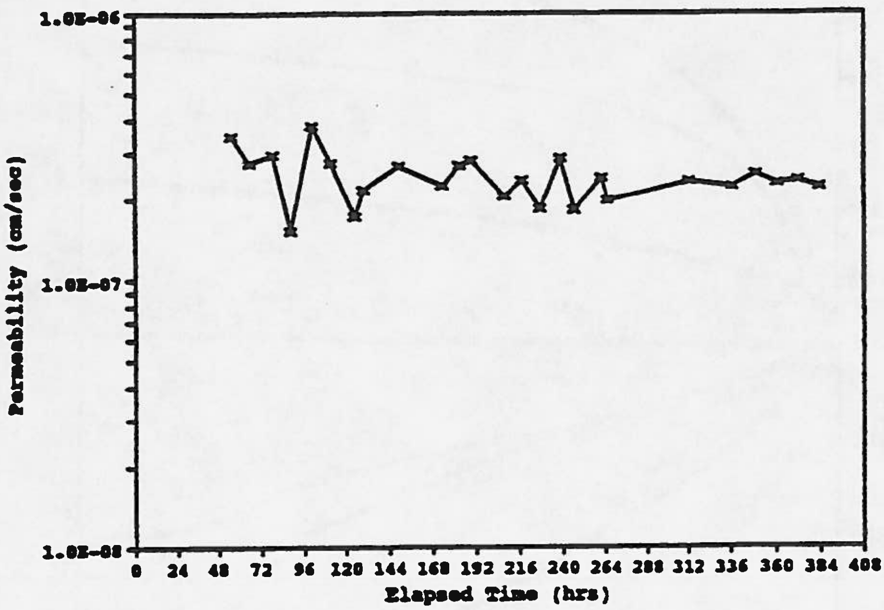


Figure 7. Permeability vs. Time for Composite Sample of Low Plasticity Clay Compacted at 94% of Standard Proctor Maximum Dry Density and Near Optimum Moisture Content (Lutz, 1994).

Table 1. Estimated Waste Storage Inventory in the CRU1 Waste Pits (ALTER, 1993).

Waste Pit No.	Estimated Waste Quantity (cu yd)	Depth (ft)	Contents	Estimated Radioactive Material (kg)	Construction	Operational Period	Current Status
1	33,676	17	Neutralized waste filter cakes, graphite, brick scrap, sump liquor and cakes, slag residues	Uranium-52,000	Excavated in clay layers and lined with clay	1952-59	Covered with clean soil
2	18,478	13	Dry low-level radioactive wastes; neutralized waste filter cakes, sump liquor and cakes, bricks, scrap, slag residues	Uranium-1,206,000 Thorium - 400	Lined with a compacted on-site clay	1957-64	Covered with clean soil
3	237,053	27	Lime neutralized raffinate, slag leach residues, filter cakes, flyash, & lime sludge	Uranium - 129,000 Thorium - 400	Excavated into clay layers and lined with clay along the pit walls	1959-77	Covered with clean soil

Table 1. Estimated Waste Storage Inventory in the CRU1 Waste Pits (ALTER, 1993), continued.

Waste Pit No.	Estimated Waste Quantity (cu yd)	Depth (ft)	Contents	Estimated Radioactive Material (kg)	Construction	Operational Period	Current Status
4	53,706	24	Process residues,trailer cakes, slurries, raffinates, graphite, asbestos, nonburnable trash, barium chloride	Uranium-3,000,000 Thorium - 61,800	Same as Pit No. 3	1960-86	Interim RCRA cap
5	98,841	30	Solids from neutralized raffinate, neutralized slag leach slurry, lime sludge, arsenic	Uranium - 50,309 Thorium - 17,000	Lined with 60 mil Royal-Seal DPDM elastomeric membrane	1968-87	Uncovered
6	11,556	24	Depleted slag, sump green salt, process residue, filter cake, U_3O_8	Uranium - 843,142	Same as Pit No. 5	1979-85	Uncovered
Burn Pit	9,074	20	Reactive chemicals, pyrophoric chemicals, oils, combustible wastes, scrap iron, wood, tin cans, ashes, and gravel	Unknown	Excavated in clay. Excavated clay used to line Pits 1 and 2	1957-86	Backfilled
Clear-well	5,008	27	Clear process effluents and surface runoff	Unknown	Lined with clay	1959-present	In use

Table 2. Visual/Manual Description of the CRU1 Waste Pit Samples (Lutz, 1994).

Samp. #	Pit #	Shine	Cohesion	Dilatancy	Dry Strength	Consistency	HCl Reaction	H_2O_2 Reaction	Odor	Color
1A	1			Quick	Low		None	None	Petroleum	Gray Black
2A	1	Dull	None	Quick	Low	Stiff / Moist				Gray Black
3A	1		Little	Quick	Low	Moist	Mod.	None		Gray Black
4A	1	Dull	Little	Quick	Low	Stiff / Moist	None	None	Petroleum	Gray Black
5A	1		Little	Quick	Low	Stiff / Moist			Petroleum	Gray Black
6A	1			Quick	Low	Stiff / Moist	None	None		Gray Black
7A	1	Dull	Little	Quick	Low	Stiff / Moist	None	None	Petroleum	Gray Black
8A	1			Quick	Low	Loose / Moist	None	None		Gray
9A	2			Slow	Mod.		Strong	None		Light Brown
10A	2		Mod.	None	High	Soft / Wet			Mothballs	Light Brown
11A	2	Dull	Mod.	Mod.	High	Soft / Wet				Med. Brown
12A	2	Mod.	Mod.	Slow	High	Loose / Moist	Mod.	Weak		Brown
13A	3				High		Weak	Weak	Ammonia	Brown Mot.
14A	3				Low	Soft / Wet	None	Weak		Brown Mot.
15A	3			Slow	Mod.	Loose / Moist				Brown Mot.
16A	3			Slow	Mod.					Red Brown
17A	3			Slow	Mod.		Mod.	Mod.		Red Brown
18A	3		Little	Slow	High	Soft / Wet	Mod.	None		Med. Gray
19A	3	Mod.	High	None	High	Stiff / Moist	Weak	Mod.		Brown Mot.
21A	4		Mod.	Slow	Mod.	Loose / Moist				Gray Black
22A	4	Dull	Little	Quick	Low	Stiff / Moist	None	None	Petroleum	Gray Black
23A	4			Quick	Low	Stiff / Moist			Petroleum	Gray Black
24A	4		Mod.	Slow		Loose / Moist	Strong	Mod.		Brown
25A	4		Mod.	None		Loose / Moist	None	None		Gray Black
26A	4	Dull	Little	Quick	Low	Stiff / Moist	None	None	Petroleum	Gray Black

Table 2. Visual/Manual Description of the CRU1 Waste Pit Samples (Lutz, 1994), continued.

Samp. #	Pit #	Shine	Cohesion	Dilatancy	Dry Strength	Consistency	HCl Reaction	H_2O_2 Reaction	Odor	Color
30A	5		Mod.	Mod.	Low	Soft / Wet	Mod.	None	Ammonia	Multicolor
31A	5		Mod.		High	Very Soft / Wet	Mod.	Weak	Ammonia	Salmon Mot.
32A	5			Slow	High	Very Soft / Wet	Weak	None	Ammonia	Salmon Mot.
33A	5			Mod.	High	Very Soft / Wet	Strong	None	Ammonia	Salmon Mot.
34A	5			None	High	Very Soft / Wet	Strong	None		Gray Mot.
35A	5		Mod.	None	Mod.	Soft / Wet			Ammonia	Brown Mot.
36A	5			Mod.	Mod.	Very Soft / Wet	Mod.	None	Ammonia	Gray Salmon
37A	5				High	Very Soft / Wet	Strong	Weak	Ammonia	Salmon Mot.
38A	5			Slow		Very Soft / Wet			Ammonia	Salmon
39A	5				Low	Very Soft / Wet	Strong	Weak		Salmon
40A	5				Low	Soft / Wet	Strong	None		Gray Mot.
41A	5		Mod.	Slow	Low	Soft / Moist			Ammonia	Brown Gray
57A	5		Mod.	Mod.	Low	Very Soft / Wet	Strong	Weak	Ammonia	Salmon Mot.
58A	5		None	None	High	Very Soft / Wet			Ammonia	Salmon Mot.
43A	6		Little	Quick	Low	Very Soft / Wet	Strong	None		Pea Green
45A	6	Dull	Little	Quick	Low	Soft / Wet	Strong	None		Gray Black
45C	6			Quick	Low	Soft / Wet	Strong	Weak		Pea Green
46A	6	Mod.	Mod.	Slow	Mod.	Soft / Wet	Strong	None		Gray Green
47A	6		Mod.	Slow	High		Strong	None	Septic	Gray
49A	CW	Dull	Mod.	Slow	High	Very Soft / Wet	Strong	Strong	Sulfur	Gray Brown
50A	CW		Mod.	None	High	Soft / Wet	Strong	Strong		Gray Brown
51A	CW		Mod.	None	High	Soft / Wet	Strong	Strong		Gray Brown
52A	CW		Mod.	Slow	Mod.	Soft / Wet	Mod.	Mod.		Gray Brown
53A	CW		Mod.	Slow	High	Mod. Stiff/ Wet	Strong	Strong		Gray Brown
54A	CW		Mod.	Slow	High	Soft / Wet	Strong	Strong		Light Brown
55A	CW	Mod.	Mod.	Slow	Mod.	Soft / Wet	Strong	Strong		Brown
56A	CW	Mod.	Mod.	Slow	Mod.	Stiff / Moist	Strong	Strong		Brown

Table 2. Visual/Manual Description of the CRU1 Waste Pit Samples (Lutz, 1994), continued.

Samp. #	Pit #	Shine	Cohesion	Dilatancy	Dry Strength	Consistency	HCl Reaction	H_2O_2 Reaction	Odor	Color
27A	BP		Mod.	Slow	High	Loose / Moist	Strong	Mod.		Brown
28A	BP		Little	Quick	High	Loose / Moist	Weak	Weak		Gray Black
29A	BP		Little	Quick	High	Loose / Moist	Strong	Mod.		Gray Black

Table 3. Basic Geotechnical Characterization Testing Results for the CRU1 Waste Pit Samples (Lutz, 1994).

Samp. #	Pit	Moisture Content %	% Gravel	% Sand	% Fines	% < 2μ	G_s	Liquid Limit %	Plastic Limit %	PI %	pH of Solids In H_2O	Liquidity Index	Activity Index	USCS
1A	1	20.3	0.0	14.2	85.8			--	--	--		--		NP
2A	1	24.7	0.0	13.2	86.8			--	--	--		--		NP
3A	1	39.1	2.9	26.8	70.3			--	--	--		--		NP
4A	1	18.0	0.0	9.0	91.0			--	--	--		--		NP
5A	1	20.7	0.4	10.8	88.8	0.0	3.02	--	--	--	7.27	--	--	NP
6A	1	21.1	0.0	9.0	91.0			--	--	--		--		NP
7A	1	20.7	1.1	9.8	89.1			--	--	--		--		NP
8A	1	22.7	3.9	37.6	58.5			--	--	--		--		NP
9A	2	35.4	1.0	55.1	43.9			33.1	23.2	9.9		1.23		SM
10A	2	38.3	7.6	26.1	66.3	22.2	2.72	42.3	23.1	19.2	7.67	0.79	0.86	CL
11A	2	70.5	5.5	27.8	66.7	11.7	2.67	66.8	42.2	24.6	7.53	1.15	2.10	MH
12A	2	16.7	4.2	22.3	73.5			44.1	16.2	27.9		0.01		CL
13A	3	145.4	3.2	33.7	63.1			89.7	55.8	33.9		2.64		MH
14A	3	152.7	3.5	23.8	72.7			81.8	58.9	22.9		4.10		MH
15A	3	42.2	6.5	25.8	67.7	3.1	2.69	46.9	31.7	15.1	7.74	0.70	4.87	ML
16A	3	96.8	3.3	27.0	69.7	15.7	2.72	77.8	44.3	33.5	7.76	1.57	2.13	MH
17A	3	142.0	1.0	15.3	83.7			69.9	53.9	16.0		5.51		MH
18A	3	41.9	2.5	40.9	56.6			37.9	30.5	7.4		1.54		ML
19A	3	36.1	6.0	19.9	74.1			44.1	25.2	18.9		0.58		CL

Table 3. Basic Geotechnical Characterization Testing Results for the CRU1 Waste Pit Samples (Lutz, 1994), continued.

Samp. #	Pit	Moisture Content %	% Gravel	% Sand	% Fines	% < 2μ	G_s	Liquid Limit %	Plastic Limit %	PI %	pH of Solids In H_2O	Liquidity Index	Activity Index	USCS
21A	4	18.0	6.4	29.5	64.1	6.6	3.25	28.4	18.5	9.9	7.79	-0.05	1.50	CL
22A	4	23.3	0.0	12.0	88.0			--	--	--		--		NP
23A	4	22.2	0.8	15.2	84.0	8.4	3.02	--	--	--	7.73	--	--	NP
24A	4	10.1	13.4	24.7	61.9			33.9	16.7	17.2		-0.38		CL
25A	4	25.4	1.2	34.7	64.1			34.1	21.8	12.3		0.29		CL
26A	4	29.3	1.2	18.0	80.8			--	--	--		--		NP
30A	5	173.0	0.0	5.0	95.0			70.7	57.7	13.0		8.87		MH
31A	5	497.5	0.0	5.2	94.8			94.2	50.6	43.6		10.25		MH
32A	5	483.0	0.0	3.4	96.6			102.2	80.5	21.7		18.55		MH
33A	5	507.3	0.0	1.3	98.7			108.4	71.8	36.6		11.90		MH
34A	5	451.7	0.0	1.9	98.1			102.2	64.6	37.6		10.30		MH
35A	5	161.1	0.0	11.9	88.1	14.5	2.70	90.9	60.2	30.7	8.53	3.29	2.12	MH
36A	5	342.4	0.0	8.1	91.9			103.8	75.6	28.2		9.46		MH
37A	5	620.0	0.0	0.2	99.8			93.0	58.0	35.0		16.06		MH
38A	5	495.7	0.0	2.7	97.3		2.74	112.3	68.1	44.2	8.72	9.67	--	MH
39A	5	448.5	0.0	5.9	94.1			112.0	69.6	42.4		8.94		MH
40A	5	114.5	0.0	5.5	94.5			--	--	--		--		NP
41A	5	109.3	0.0	6.0	94.0	8.9	2.71	--	--	--	8.34	--	--	NP
57A	5	242.3	0.0	14.6	85.4			87.7	60.9	26.8		6.77		MH
58A	5	295.7	0.0	10.4	89.6	10.2	2.70	77.1	52.7	24.4	8.22	9.96	2.37	MH

Table 3. Basic Geotechnical Characterization Testing Results for the CRU1 Waste Pit Samples (Lutz, 1994), continued.

Samp. #	Pit	Moisture Content %	% Gravel	% Sand	% Fines	% $< 2\mu$	G_s	Liquid Limit %	Plastic Limit %	PI %	pH of Solids In H_2O	Liquidity Index	Activity Index	USCS
43A	6	425.1	0.0	3.7	96.3			--	--	--		--		NP
45A	6	26.8	0.0	27.6	72.4	9.6	3.21	--	--	--	9.19	--	--	NP
45C	6	117.6	0.0	51.3	48.7			--	--	--		--		SM
46A	6	107.4	0.0	13.8	86.2	27.9	2.98	47.0	28.4	18.6	9.43	4.25	0.67	ML
47A	6	111.4	0.0	9.8	90.2	35.2	2.85	64.9	33.1	31.8	9.40	2.46	1.45	MH
49A	CW	53.4	3.3	31.1	65.6			43.9	25.6	18.3		1.52		CL
50A	CW	65.1	10.9	36.4	52.7			50.6	32.5	18.1		1.80		MH
51A	CW	61.9	0.0	51.5	48.5			45.0	28.0	17.0		1.99		SM
52A	CW	40.2	8.5	47.0	44.5			39.3	24.9	14.4		1.06		SM
53A	CW	35.4	0.0	39.4	60.6	22.3	2.69	35.1	18.0	17.1	8.35	1.02	0.77	CL
54A	CW	73.9	0.5	22.5	77.0			54.7	39.2	15.5		2.24		MH
55A	CW	71.7	1.8	23.7	74.5	23.8	2.78	52.4	38.0	14.4	8.24	2.34	0.61	MH
56A	CW	68.5	3.3	31.3	65.4			53.3	40.2	13.1		2.16		MH
27A	BP	25.5	2.6	26.2	71.2			41.3	24.7	16.6		0.05		CL
28A	BP	25.6	4.4	37.0	58.6			35.1	31.0	4.1		-1.32		ML
29A	BP	30.0	8.5	38.2	53.3	15.0	2.50	37.8	26.7	11.1	7.74	0.30	0.74	ML

Table 4. Mechanical Properties of the Seven Composite CRU1 Waste Pit Sample Types (Lutz, 1994).

Sample Group	Liquid Limit %	Plastic Limit %	Shrinkage Limit and Ratio		105 °C Liquid Limit %	Optimum Moisture %	Std. Proctor Max. Dry Density (lb/ft^3)	UCS*		Direct Shear*		Permeability* (cm/s)
			SL %	R				q_u (psf)	c_u (psf)	c_{cd} (psi)	ϕ_{cd} (° Δ)	
NP1	21.3	NP	20.9	1.77	21.7	13.4	116.6	N/A	N/A	0.0	40	4.3 E^{-06}
NP2	48.9	NP	53.4	1.05	43.7	36.2	76.9	N/A	N/A	6.3	31	N/A
MH1	89.3	56.9	67.5	0.91	58.4	56	65	2425	1212	1.7	33	1.2 E^{-07}
MH2	59.8	40.1	35.2	1.36	50.9	35.4	77.8	2936	1468	7.6	29	6.6 E^{-08}
ML	36.8	27.8	22.8	1.60	31.7	19.9	99.0	5744	2872	2.6	37	5.9 E^{-07}
CL	28.2	19.4	17.1	1.96	24.9	18.7	118.4	3014	1507	0.0	36	2.3 E^{-07}
SM	32.2	23.8	22.8	1.65	28.5	20.1	102.3	2962	1481	2.4	32	2.6 E^{-07}

* - @ approximately 95% standard Proctor maximum dry density

Table 5. Summary of Comparisons Between Expected Versus Observed Mechanical Behavior.

Sample Group	Liquid Limit	Plastic Limit	Optimum Moisture Content %	Expected Optimum Moisture Content %	Standard Proctor Maximum Dry Unit Weight (pcf)	Expected Maximum Dry Unit Weight (pcf)
NP1	21.3	NP	13.4	---	116.6	112.8
NP2	48.9	NP	36.2	---	76.9	90.0
MH1	89.3	56.9	56.0	55.0	65.0	66.8
MH2	59.8	40.1	35.4	38.0	77.8	87.2
ML	36.8	27.8	19.9	25.0	99.0	102.9
CL	28.2	19.4	18.7	16.5	118.4	109.9
SM	32.2	23.8	20.1	20.8	102.3	106.5

GEOTECHNICAL ENGINEERING IN ENVIRONMENTAL SITE CHARACTERIZATION AND RESTORATION PROJECTS

Michael J. Saffran, P.E.
U.S. Army Corps of Engineers - Louisville District

Chris R. Karem, P.E., P.G.
U.S. Army Corps of Engineers - Louisville District

INTRODUCTION

Contamination assessment and remediation projects require a multi-disciplined team approach; however, in many instances, the most important member of the team is the geotechnical engineer. Whether the subject is a leaking underground storage tank, a release from a hazardous waste storage area, an uncontrolled dump site, or the potential liability associated with owning or acquiring contaminated real estate, the most important issues often hinge on the interpretation of the subsurface conditions at and around the project. The object of this discussion is to demonstrate how important the geotechnical engineer is to efficiently characterizing and resolving contamination issues.

A general discussion of the most relevant geotechnical concepts related to the transport and fate of contaminants is followed by some basic procedures used to integrate the acquisition and evaluation of geotechnical and chemical data to resolve contamination issues on a wide variety of project types. Some of the most poignant example cases that have occurred on US Army Corps of Engineer projects within the Louisville District are then described. The cases selected for discussion contrast the effects on project resolution, schedule and budget between when poor and high quality geotechnical engineering input are maintained throughout the course of the project. The projects include a civil works flood damage reduction project; assessment and remediation at leaking UST sites and petroleum spills; site characterization for a base closure project; and remediation at a superfund landfill.

We have learned one very important lesson from these experiences. That is that the resources typically expended to acquire and laboratory analyze soil and groundwater samples for toxic constituents are disproportionately high relative to the resources expended to evaluate and present the results on most site characterization projects. To relate this to the geotechnical engineer, the cost for one complete TCLP analysis on a soil sample is equivalent to the cost to the client of at least two days of engineering time.

Consequently, in the Louisville District, we are emphasizing practical decision-making based on sound geotechnical evaluation of the subsurface conditions. Putting more geotechnical engineering time into contamination assessment and remediation projects, reduces overall costs by significantly reducing the number of samples required for chemical analysis.

Having an experienced geotechnical engineer on-site during investigation and remedial activities facilitates rapid decision-making and adherence to project schedules and budgets.

GEOTECHNICAL ASPECTS OF CONTAMINATION ASSESSMENT PROJECTS

There are several basic geotechnical concepts that must be understood to design and manage environmental site characterization and remediation projects. A discussion of these concepts might seem like preaching to the choir, yet the most common mistakes in environmental site characterization projects come from inattention or lack of knowledge of these basic concepts. The limited discussion below focuses on preparatory work, soil grain size, drilling and soil sampling techniques and groundwater sampling. It is not intended to be a lesson in Geomechanics as much as it is to set the stage for some of the lessons we have learned on the projects that are subsequently described. The good practitioner depends on geotechnical engineering and hydrogeology text books in planning and conducting these types of projects to supplement the various regulatory guidance documents available. These texts are readily available.

Preparatory Phase

As with any subsurface investigation, the first step in the project development process for a contamination assessment or remediation project is to acquire and review relevant background data. A contamination assessment or remediation project typically requires geotechnical knowledge of soil stratigraphy, regional and local hydrogeology, and type and depth to rock. In a geotechnical subsurface investigation the process of obtaining this information typically entails three basic preliminary steps, none of which is generally documented in a report until the final subsurface investigation report is produced. The steps are: A discussion with the client about the history of the site and the project objectives; A review of available subsurface information in sources like, USGS Topographic and Geological Quadrangles, the SCS County Soil Survey, utility maps, previous boring logs and interviews with experienced geologist and geotechnical engineers familiar with the area; and A site reconnaissance to observe and document site features.

For most contamination assessment projects, the process is similar, but more effort is expended and the results are documented in a separate report. The title of this report may vary, i.e. Preliminary Assessment, Environmental Audit or Phase 1 Environmental Site Assessment (ESA), depending on the type of project. Parts of the extra effort for each step include: A more extensive review of literature and data bases of environmental records to identify potential sources and types of contaminants in the vicinity of the project; Interviews with persons knowledgeable about site history and operations, particularly waste disposal that may have occurred at and around the property; and A thorough visual inspection of the subject area by environmental professionals to observe surface conditions, looking especially for evidence of contamination such as soil staining or stressed vegetation. The focus of these inquiries is to establish whether there is a reasonable probability that contaminants exist on the property at concentrations and locations that could pose a significant risk to human health or the environment.

A geotechnical engineer should always participate on the inspection team and in the write-up because of the need to evaluate potential contaminant transport mechanisms on and below the ground surface. When properly done, this report alone is often sufficient to make a determination of whether there is a significant risk that contaminants are present on a piece of property. When these risks exist, but project objectives require that they be mitigated, the geotechnical engineer is best qualified to lead the project team through the next phase of the project. Whether the next step is further investigation or a remedial action, the success will depend on proper consideration of the geotechnical issues.

Soils

Soil type is of the most important factors controlling the transport and fate of contaminants through the subsurface. The basis for classifying soils is grain size, and the Unified Soil Classification System (USCS) is the most widely accepted standard of soil classification for engineering purposes. The four basic soil types are clay, silt, sand and gravel in order of increasing grain size. Silts and clays are termed fine grained, and sands and gravels are coarse grained. Fine grained soils transmit water at a lower rate than coarse grained soils; therefor, contamination from a leaking underground storage tank would tend to be more widespread if the surrounding soils are coarse grained than if they were fine grained.
The difference in size between grains of sand and gravel are detectable to the naked eye; where as, individual silt and clay particles are microscopic to submicroscopic. The submicroscopic size of clay particles plays a significant role in the way they behave. Clays are comprised of microscopic plate-like minerals. The thickness of the plate may vary from 10 nanometers (montmorillinite) to 2000 nm (kaolinite), while the lateral dimensions of most clay minerals are between 500 - 10000 nm. The clay particles carry a net negative charge on their surfaces, creating an attractive force for available or exchangeable cations. Larger negative charges and greater cation exchange capacities are associated with thinner particles, which have a larger specific surface (surface area per unit mass). The electro-chemical forces in the vicinity of clay particles result in a strong affinity of the clay particle for water, which is dipolar. When water is added to clay, a small zone around the clay particle is comprised of exchangeable cations and a small number of anions that make up what is referred to as the diffuse double layer. The cations are attracted to the surface of the clay, and they stackup around the individual plates. The concentration of cations diminishes with distance from the clay particle. The inner-most double layer of water around a clay particle is held very strongly, and this "adsorbed water" does not have the same viscosity of free water. While the thickness of the diffuse double layer is nearly the same for kaolinite as it is with montmorillinite, the change this has on the physical properties of the two materials is remarkable. The water that is bonded to individual clay particles gives clay soils their plastic properties. These plastic properties allow a geotechnical engineer to readily field identify a highly plastic clay from a clay of low plasticity or a silt. Quite simple, a clay can be rolled into a thread. A highly plastic clay can be rolled into a thread, then to a ball, then back to a thread numerous times. It behaves like a plastic over a wide range of moisture contents.

The significance of clay mineralogy and particle size to the business of site characterization, contamination assessment and remediation projects is that fine grained soils tend to immobilize or divert the flow of contaminants in several ways. One is that the flow path of liquids through a fine grained soil matrix is very tortuous. The physical size and space between grains prevents the passage of solids suspended in a liquid (filters), and the cation exchange capacity tends to electro-chemically bind some contaminants to the soil particles.

The importance of understanding the influence particle size has on soil moisture and groundwater cannot be overstated. Numerous publications provide ranges of properties for soils based on grain size and USCS classification. These properties such as moisture content, void ratio, porosity and permeability are used by the geotechnical engineer to model soil stratigraphy, and hydrogeology.

Groundwater and Groundwater Sampling

Soil and rock strata that are saturated and are large enough and porous enough to yield quantities of potable water sufficient for domestic use are generally classified as aquifers. The physical and biological filtering that occurs in aquifers purifies water by degrading organic materials and filtering suspended solids over long periods of time. Elevation and pressure gradients drive the flow of groundwater, and the constituents in groundwater from higher to lower energy levels. Temperature and electrical gradients can be influences on groundwater flow, as can chemical concentration gradients. The understanding and the tools to investigate and model these phenomenon are the forte of the geotechnical engineer. Groundwater flow is modeled by considering the combination of flow gradients with the relative layering and porosity of the soil and rock in the subsurface. Piezometers are the primary field measuring instruments of the geotechnical engineer from which to develop the model. They are used to determine the relative energy of water at various locations due to elevation differences.

The design and construction of a monitor well does not differ substantially from that of an open pipe piezometer. The same basic design criteria generally apply. The monitor well is a clean piezometer. A piezometer is designed to allow the inflow of water, but not soil particles. Therefor, the slot size of the screen and the make-up of the filter pack placed around the screen are selected based on the soil type of the subsurface zone being monitored. The depth, screened interval and length of sensing zone are all customized to site specific conditions. Bentonite is typically used to seal the sensing zone above the screened interval. In a piezometer, a soil/bentonite mixture is often used to backfill the annulus around the riser pipe; whereas a cement/bentonite mixture is usually used for this purpose in a monitor well. Good monitor well design requires detailed geotechnical knowledge of site specific conditions coupled with an understanding of the data needed to meet project objectives.

The goal in groundwater sampling for chemical analysis is to obtain a sample that is representative of the in-situ groundwater. Wells that have not been properly developed yield turbid, non-representative water samples because of the disturbances that occur during the drilling process. Evacuation of a minimum of three to five well volumes is typically specified to develop a monitor well, but the actual amount of water required for proper development can be much more. Also, water that has been contained within the stand pipe of the well for a significant time may become stagnant and chemically altered. Consequently, evacuation of three well volumes is usually specified to adequately purge a monitor well of nonrepresentative water. Three quick and simple field tests are usually specified to assure proper well development and purging. They are pH, specific conductance and temperature. Environmental sampling protocol typically requires that these parameters be stable over three consecutive readings prior to sample collection.

Soil Sampling and Field Screening

Environmental sampling protocol is relatively new, but it is similar to the basic geotechnical investigation and sampling techniques that have been used for many years. There are a variety of geotechnical techniques to explore and sample soils and rock. Selection of appropriate sampling techniques is a very important decision in the process of designing an environmental site investigation. The selection must be based on an integration of the available information with the project objectives and budget. Although variables such as material type, soil stratigraphy, depth to the water table and type and depth to rock effect selection of exploration techniques, the information is often unknown prior to initiating an investigation. Readily available text books provide descriptions of the commonly used geotechnical drilling and sampling techniques and their best applications.

The prevention of the introduction of contaminants to the sample is the primary focus of environmental sampling techniques. Because laboratory analyses to identify contaminants in soil and groundwater are capable of accurately determining concentrations down to the one part per billion level, great care must be taken to avoid the introduction of contaminants into the sample. This may happen several ways. Geotechnical drilling typically involves the use of petroleum based greases for thread compounds to facilitate connection and disconnection of drill rods and augers. Piezometer riser pipe and well screen sections are often glued together. These types of materials typically used in geotechnical drilling often contain chemicals that are regulated as hazardous or toxic substances, and they can contaminate samples submitted for quantitative chemical analysis. Contaminants can travel down the drill hole from an upper strata, or may be transferred from one boring location to another using standard geotechnical sampling procedures.

Consequently, environmental sampling protocol precludes the use of materials in drilling or well construction that could introduce chemical contaminants to the sample. The protocol requires thorough decontamination of sampling equipment prior to beginning each boring and each sample acquisition to prevent cross contamination between borings and samples. This can double the time and costs associated with obtaining soil and groundwater samples.

Additional health and safety precautions are called for when there is a reason to believe that toxic substances are present, and this increases costs. Formal continuing education and training is required for all site workers. Personal protective clothing is always required for environmental drilling to investigate for the presence of toxic chemicals. Respiratory protection is also warranted at times. Because many of the most dangerous toxic substances are volatile organic compounds, portable devices such as a Photo Ionization Detector (PID), Flame Ionization Detector (FID) or Explosivity Meter are typically used by the site safety officer to sample and analyze the air to determine when respiratory protection is warranted or danger of explosion is indicated during toxic substance investigation and remediation projects. The FID and PID typically detect total organic vapors down to the part per million level, and they can be used to provide valuable field screening data depicting the nature and extent of contamination. Many engineers managing contamination investigation and remediation projects utilize these devices to analyze soil samples by a technique called 'head space' analysis. The technique is simple. Soil from a discreet sample location is placed into an air tight container leaving a small amount of air space at the top of the container. After sufficient time, the probe of the organic vapor analyzer may be inserted into the head space of the container, and an air sample withdrawn and analyzed. The technique is often enhanced and quickened by subjecting the sample to a heat source to promote volatilization of organic chemicals if they are present. When variables such as the percent fullness of the sample container, duration between sampling and analysis and exposure to a heat source are kept constant and properly documented, this data can be used to help the engineer locate and evaluate contaminated areas.

The utility of this simple technique is that it provides an inexpensive, yet effective means of gathering data from which to base project decisions. For an investigation, two phases may be conducted. In the first phase, samples from geotechnical borings can be analyzed according to head space analysis techniques, and the head space data can be evaluated spacially in relation to the geotechnical data to evaluate contamination plumes. This can significantly reduce costs by reducing the number of environmental borings, monitor wells and samples for chemical laboratory analyses. For remedial actions at UST sites, this technique can be used to delineate the extent of excavation required to achieve a clean hole for closure sampling.

Chemical Data Acquisition

Liability issues associated with contaminated properties often create the need for legally defensible chemical data, which is very expensive, and very difficult to achieve. Investigations to acquire chemical laboratory analyses should never be performed without clearly established objectives and procedures. In far too many cases the results of chemical analysis raise more questions than they answer. This is usually because too much importance is attributed to small sets of chemical data that offer very little confidence from a statistical perspective. This occurs because the cost to analyze enough samples to achieve appropriate levels of confidence in the data is prohibitive.

Consider the heterogeneous nature of soils, and the range of soil types and their natural physical and chemical properties to the regulatory standards set at the part per million and part per billion levels. Does a concentration of one part per million of a chemical in sand pose the same risk to human health and the environment as the same concentration of that chemical in a clay? Do the electro-chemical forces present in a clay affect the results of a chemical analysis relative to the same analysis on a sand? These are subtle questions that few regulators are prepared to answer.

The sampling and analysis protocol developed by the USEPA are intended to help standardize procedures, but the issues raised above are but a few of the problems associated with the acquisition and interpretation of chemical data when USEPA protocol is adhered to. With that preface, the main additions to a geotechnical investigation to acquire chemical data are discussed below.

Investigation design is critical. The issues that must be addressed are: Clear definition of project objectives; Identification of chemicals of concern and their chemical and physical properties; Sample acquisition strategy; Chemical data quality objectives; Quality control and quality assurance objectives and procedures; and Health safety and accident prevention. These issues must all be raised and addressed in a formal work plan prior to initiating field work.

In the field, health and safety monitoring and equipment are selected to prevent exposure to the chemicals of concern, considering the methods being employed to acquire samples. The sample acquisition methods are developed to prevent the introduction of contaminants to the sample. Preservation, containers and holding times between sampling and analysis are all specified to prevent a change in the chemical constituents in a sample from the time of acquisition to analysis. Chain of custody documentation is required to track the samples through this interim. Approximately a 20% increase in the number of samples analyzed is typically required for appropriate QC/QA purposes.

The most common chemical analytes are contained in the USEPA's The Target Compound List (TCL) and Target Analyte List (TAL). The Toxic Characteristic Leaching Procedure (TCLP) includes 43 analytes from this list with the greatest potential to contaminate groundwater. The TCL includes (Volatile Organic Compounds (VOCs), SemiVolatile Organic Compounds (SVOCs), Pesticides and PCBs. The TAL generally includes metals. The TCLP was developed to simulate the acidic conditions that can occur during the life of a landfill.

One big problem with evaluation of contamination using chemical analytical testing is that often unaffected soils contain natural or background concentrations of suspect chemicals greater than levels considered to be a problem. This often results in phantom issues and wasted time and dollars. To determine if chemicals present in the subsurface are a result of a chemical release or are typical background levels can be very difficult.

Identification of soil type is imperative when determining background levels of contaminates. For the reasons described above, fine grained silts and clays generally contain higher background levels of the TCL and TAL analytes, especially metals, than do sands and gravels. Yet background levels of contaminants are often determined using only depth data without considering and identifying soil characteristics.

CASE EXAMPLES - LESSONS LEARNED

The excerpts below are from projects within the Louisville District of the Corps of Engineers related to the characterization and remediation of chemical contaminants. They provide illustrative examples of some of the most common mistakes and problems that arise on these types of projects, as well as a few success stories that demonstrate the value of good geotechnical engineering input.

FT. Knox Railroad - Oil Spill

An example of the over reliance on chemical analyses associated with petroleum contamination can be demonstrated by examining a proposal to evaluate waste oil contamination associated with a Rail Road track renovation at Ft. Knox, Kentucky. Waste oil was present on the track just outside a building where train engines were pulled from the main track for maintenance. The oil staining was evident on the track, and was apparently the result of years of draining the oil directly onto the ground. A proposal to evaluate this contamination was submitted by an Architect/Engineering (A/E) firm subcontracted by the General contractor performing the design of the overall renovation (a relatively much larger construction project). The proposal called for several soil borings implementing traditional splitspoon sampling techniques along with the added decontamination, and sampling, and health and safety protocols, as typically required in the environmental arena. Several samples were to be submitted to the laboratory for a wide range of chemical analyses. The proposal seemed a reasonable approach to evaluate the nature and extent of the contamination. However, all reasonableness seemed to go away with the price tag, $130,000. One hundred-thirty thousand dollars to evaluate an oil stain on the ground. There was no math error. Environmental sampling and Chemical laboratory analysis is very expensive, and the costs to perform the proposed scope of work were commensurate with those that would have been estimated by any A\E firm in the business.

The problem with the proposal was the approach. It was mentioned earlier that a lot of engineering time can be substituted for chemical laboratory analysis. Since petroleum contamination is usually very evident to the senses and can be often be detected with field screening equipment, the need for numerous expensive chemical analyses to determine the precise degree of contamination is usually not necessary. With an understanding of the geotechnical nature of the subsurface, a project like this can be accomplished much more effectively than the way proposed by the A/E.

Most local geotechnical engineers know the majority of the Ft. Knox is underlain by silty clays of medium to high plasticity, ideal material for a landfill cap. Combining this knowledge with a basic understanding of how in-situ contamination remediation techniques work, it was obvious that excavation of the contaminated soils was considered inevitable. Again, based on knowledge of the soils, one would not expect the contamination to spread very far vertically or horizontally through the low permeability clays because of the limited potential subsurface migration pathways. Combined with the surface topography in the area, which was relatively flat, we concluded that the contamination had probably not spread very far.

Our recommendation to Fort Knox to cleanup the contamination was to excavate the contaminated soils based on visual and organoleptic indications and field screening techniques. We proposed that the only samples submitted for laboratory analysis be those necessary to show the excavation side walls and bottom were clean, and those necessary for disposal of the contaminated soil in a nearby landfill. The client, however, wanted a very precise estimation of the cost of the cleanup. Therefor, an investigation approach was implemented. The A/E refused to corporate with the approach we recommended, so the assessment was performed in-house.

The contamination was evaluated using a shovel, a stainless steel handauger and an Organic Vapor Analyzer. The analytical testing performed was limited to that necessary to characterize waste oil contaminated material for landfill disposal. An obviously contaminated sample was obtained and submitted to the analytical laboratory. The vertical extent of contamination was evaluated by moving the RR ballast aside with the shovel and advancing the hand auger. The auger cuttings appeared highly contaminated to a depth of about 2.5 feet. By the time a depth of 5 feet was achieved, neither the OVA nor organoleptic indications revealed contamination. The horizontal extent of contamination was similarly evaluated. The assessment was accomplished by an engineer and an engineering student (big and strong enough to move the ballasts aside to facilitate the hand auger) on our cooperative internship program in one day. The total cost to the client for this assessment, including the report, was less than $3,000. The estimated cost to the construction project to excavate, remove and dispose of the petroleum contaminated soil, including verification sampling that all contamination had been removed is under $25,000.

Site Characterization at a LUST Site at the Keweenaw Research Center in the Upper Peninsula of Michigan

The one aspect of this project worth mentioning relates to acquiring groundwater samples and interpreting the results. In Michigan, all work done at a Leaking Underground Storage Tank (LUST) site must be performed by a contractor on the Michigan Qualified Consultant's List. One such contractor investigated the site of a former 4,000 gallon gasoline UST and two 750-gallon heating oil tanks. The tanks were taken out of service in 1983 by pumping out the product and backfilling them with sand. In 1992 the tanks were pulled from the ground along with 475 cubic yards of soil that were petroleum contaminated. One closure sample exceeded the Michigan clean closure criteria. Based on direction/suggestion by the Michigan Department of Natural Resources, a technique referred to as vertical aquifer profiling was used to assess the horizontal and vertical extent of contamination at the site.

In using this method, the contractor augered to a certain depth, pulled the hollow-stem augers back slightly, and inserted a prefabricated well screen and riser down the hollow stem augers to obtain a water sample. The A/E maintained very poor records regarding the volume of water that was removed for development, purging and sampling. Further, there was no documentation that pH, conductance or temperature were ever monitored during the sample acquisition process, nor was there any indication as to whether or how the TAL samples were filtered and preserved. A total of 41 samples was acquired by this method and submitted for laboratory analysis for the full TCL and TAL analytes. Believe it or not, the results indicated concentrations of several metals in excess of Safe Drinking Water Act standards.

A year later we still have unresolved issues at this site because of the blatant ignorance of basic geotechnical principles and sampling techniques that are necessary to conduct contamination assessment projects.

UST Remediation at Fort Sheridan, Ill.

The Louisville district was tasked with "closing" multiple Underground Storage Tank (UST) sites located within Ft. Sheridan, Illinois. This required removal of the USTs and remediation of affected soil and water at these sites. Experience indicates many UST sites have been contaminated to some degree. Therefor, it was anticipated that soil and water at most of the sites would be contaminated and require remediation. No sampling and laboratory analysis was performed prior to removal of the USTs. Geology at the base was obtained through review of existing information (including geotechnical boring logs formally drilled at the base for building construction) and interviews with geotechnical engineers experienced with the geologic conditions beneath the base. The available information indicated that all of Ft. Sheridan is underlain by over 200 feet of glacial till consisting primarily of tight clays with thin discontinuous sand and gravel seams which generally contain water in somewhat insignificant quantities with respect to groundwater control in excavations.

Because the site soils are predominately clay (CL and CH in accordance with the USCS), in-situ remediation techniques were not considered effective. Therefor, excavation of contaminated soil was considered inevitable for the most part. It was anticipated that water would be perched in many of the tankpits as most USTs are backfilled with granular material. In an otherwise clay lithology, a hole in the ground backfilled with sand would surely have collected water over the years especially in the larger tankpits. It was anticipated that the water could be pumped from the tankpits easily with little to no recharge. For these reasons it was considered most cost effective to remove contaminated soil and water at the sites at the time of the UST removal.

Petroleum contaminated soil and water can largely be identified through visual and organoleptic indications and inexpensive field screening techniques; therefor, soil was removed from the tankpit walls and bottom until field observations indicated the extent of the contamination had been excavated. Water was removed as necessary to remove the contaminated soil. When field observations indicated the extent of contaminated soil had been excavated, soil samples were taken from the tankpit sidewalls and bottom for laboratory analysis.

Because multiple UST sites were to be addressed simultaneously, the cumulative quantity of water was anticipated to be large. Water was treated on site due to the extremely high cost of transporting and treating petroleum contaminated water at an off-site Treatment Storage and Disposal (TSD) facility. Contaminated water from each individual tankpit was bulked in a large capacity holding tank then treated using a mobile oil/water separator, an air stripper and a carbon adsorption unit in series. Treated water was discharged as surface water through the NPDES permitting process. The water treatment plan and procedures were identified early in the project development because of the time required for the permitting process.

The UST sites generally contained contaminated soil in quantities ranging from 50 to 1500 cubic yards. The majority of the sites were remediated (dig and haul to a landfill) successfully at the time of the UST removals. The cumulative quantity of contaminated water removed and treated exceeded 200,000 gallons. Contaminated water was easily removed from the individual tankpits with little to no recharge. The on-site treatment of the contaminated water averaged about 15 cents per gallon where off-site treatment would have averaged well over 1 dollar per gallon.

Toxic Chemical Release at Scott Air Force Base

Phantom issues can be very costly to resolve as shown in this example regarding the closure of two RCRA hazardous waste storage pads located at Scott AFB in Illinois. A contractor removed and disposed of the containerized hazardous wastes (primarily F-listed spent solvents) and cleaned the concrete pads. He then preceded to take surface soil samples on each side of the pad to evaluate the potential contamination (per IEPA protocols). His laboratory analyses revealed the presence of Methylene Chloride in the samples. As a result, the contractor proposed a Phase II sampling and analysis program to evaluate the extent of contamination. The estimated cost of the investigation was approximately $200,000.

For perspective, the concrete pads both had a footprint of about 15 feet x 15 feet. It should also be noted that these were two of four pads cleaned and evaluated by the contractor. The other two pads revealed contaminants other than methylene chloride. The contractor submitted his findings along with his estimate for the phase II investigation to the Illinois Environmental Protection Agency (IEPA), per IEPA requirements. The contractors cost estimate was between $550,000 and $850,000. for the evaluation of all four pads, about $200,000 of which was specifically to address the methylene chloride contaminated sites. The IEPA accepted the contractors plan.

When asked to review the situation, we had to laugh. Anyone with experience in contamination assessment knows that methylene chloride is a very common laboratory introduced contaminant, often showing up in analyses for Volatile Organic Compounds, (VOCs). When confronted, the contractor explained he had considered the possibility of the contamination being a laboratory artifact; but was sure the contamination was present and that the concrete pads had, in fact, stored methylene chloride.

In short, the contractor was fired. The IEPA was submitted a new plan to investigate all four sites. The plan was designed to show that the reported presence of methylene chloride at the two pad locations was a false positive, as well address the contamination issues at the other two pads. The cost to execute the new plan was approximately $25,000. The IEPA not only accepted the new plan, they laughed out loud at the contractors original proposal, stating that "yes, you can do all that sampling and analysis if you want to, but it's not necessary". The methylene chloride did prove to be a laboratory artifact. The one thing that still concerns us is that the IEPA had no regard for the waste of taxpayer dollars evident in the contractors original plan, although they thought the waste was humorous in light of our counter proposal.

Pond Creek Flood Damage Reduction Project

Another example of phantom issues created by laboratory chemical analysis and ignorance of the basic geology at a site is illustrated by an A/Es' evaluation of the material to be excavated for a detention basin in the southend of Louisville, Ky. The A/E recommended a full blown chemical evaluation of the proposed detention basin site (analysis for the presence of the approximate 150 chemical contaminants that USEPA has determined to be the most problematic and has published as the Target Analyte and Target Compound Lists). The reason for such a thorough chemical evaluation was that the proposed site was located adjacent to the Northern Ditch (a major surface water drainage feature in south Louisville). Numerous industries exist along this ditch and its tributaries, and surface water runoff, and potentially waste from the industries, are discharged into the ditch and its tributaries. Samples obtained from the sediments in the ditch and tributaries revealed significant levels of a variety of contaminants at some of the sample locations. The contractor informed us he was going to compare the contaminants at the Melco Greer site to the sediments within the ditch. Also, the contractor recommended TCLP analysis for disposal purposes as it was desired to dispose of the excavated materials at the site at a nearby outerloop landfill. The contractor had not contacted the landfill, by the way, for their input on desired testing.

It should be noted that geotechnical issues were evaluated at the Melco Greer site prior to the proposal by the contractor. The geotechnical borings were performed, inhouse, and an experience environmental professional was present during the borings to evaluate potential contamination through organoleptic indications and field screening techniques. The site was underlain by 3 to 8 feet of fill consisting of material from past pond excavations on site. The fill consisted of the native silty clay soils and New Albany shale fragments (the rock underlying the native clays at the site). No indications of contamination were observed during geotechnical boring operations. Therefor, we approached outerloop landfill to determine if they would except the material as clean fill. Outerloop, after reviewing the background information (historical information and geotechnical borings results) indicated they would like potential petroleum contamination evaluated in the top two feet of the site because it was used for an equipment staging area at one time in the past.

Despite the protest of Louisville District Environmental Engineering Section personnel, the Corps' project study team approved the contractor's proposal. This was primarily due to one individual in a position of review authority at the Ohio River Division Level, who concluded that full blown chemical evaluation (including TCLP) should be Standard Operating Procedure (SOP) for all projects.

The results of the contractor's testing indicated no problems with the TCLP, TAL or TCL analysis; however, the contractor had also tested the material tested for TPH by SW-846 method 418.1. The laboratory analysis revealed considerable TPH levels but nothing else. For this reason, the contractor informed us of a contamination problem. The Division level review authority concluded that this had grave implications toward the future of the project.

Project management personnel did not like the Division conclusion, and they requested District Environmental Engineering Section review and comment on the latest submittal, with an interpretation of what the positive TPH data meant to the future viability of the detention basin site. After two phone calls, the cover pages to a dozen geologic reports were received by telefax documenting the natural oil bearing characteristics of the New Albany Shale. A meeting was held with regulators from the state at the project site to discuss the TPH issue. The state concurred with the District Environmental Engineering Section evaluation that the source of the positive TPH results were from residual petroleum within the naturally occurring New Albany Shale. Without this interpretation, development of the flood damage reduction project would have been much more expensive, and it may have resulted in an unacceptable benefit to cost ratio and abandonment of the project.

Fort Benjamin Harrison, Indiana - Background Sampling

The importance of soil properties (especially grain size) is critical to the evaluation of contaminates. The Corps is involved in several Base Closure and Realignment (BRAC) projects associated with the downsizing of the military. One is at Ft. Benjamin Harrison in Indianapolis, Indiana. With numerous potentially contaminated sites at the base, the A/E decided background contaminant levels should be determined for the entire base. In summary, the contractor evaluated his background data using statistical techniques based on soil type (as indicated by the published USDA Soil Survey) and depth. No consideration was given to assessing the physical soil properties of the actual samples acquired and analyzed.

The statistics obtained form the data gathered contained many anomalies. This caused much wasted effort when coming to an agreement with the regulatory agencies regarding appropriate background conditions. It was later determined through review of the boring logs that much of the soils that were considered to be of similar type (based on the soil survey and depth) were, in fact, much different. Native clays results were combined with sands and random fills containing coal fragments. No geotechnical parameters such as grain size or atterburg limits testing was accomplished. No soil classifications based on USCS designations were performed. These consultants won the contract to do this work based on their expertise in the arena, yet they failed to consider one of the most basic issues when attempting to quantify background conditions.

Polynuclear Aromatic Hydrocarbons, Duck Creek and Urban Areas

Among the most common chemical contaminant issues encountered in the chemical assessment of sites are a special class of compounds referred to as Polynuclear Aromatic Hydrocarbons (PNAs). Some of the PNA compounds are carcinogenic, and PNAs are used as criteria to evaluate petroleum cleanups in some states including Ky and Illinois. PNAs result from incomplete combustion of organic fuels. The less efficient the combustion process, the higher the PNA emission factor is likely to be. Many sources of PNAs are stationary, such as heat and power generation, refuse burning, industrial activity, such as coke ovens and coal refuse heaps. However, literature indicates that some PNAs can occur from forest fires and other natural sources. Transportation sources account for only about 1% of emitted PNAs on a national inventory basis, but transportation generated PNAs may approach 50 % of urban resident exposures. In other words PNAs exist throughout an urban environment.

The problem is that carcinogenic PNAs have been shown to be toxic at very low concentrations. A lot of military bases used coal for heating purposes, and the refuse was often used as fill throughout the base. This complicates the evaluation of background determinations when soil characteristics do not accompany background data. It also can create phantom issues with any type assessment performed in an urban environment. Are the PNAs associated with a release or are they ubiquitous in a particular area.

A good example of the problems associated with PNAs occurred on a flood control project for Duck Creek in Cincinnati, Ohio. We made the mistake of testing for the full TAL and TCL of an area the Phase I assessment indicated had potential metal contamination. The results of the analysis did not indicated metal contamination; however, the SVOC analysis revealed parts per million levels of certain carcinogenic PNAs (such as benzo-a-pyrene). The levels detected were above Kentucky and Illinois cleanup objectives for petroleum UST releases as well as most toxicity based cleanup objectives for these PNAs. The anomaly is that background samples were obtained in conjunction with the analysis that also revealed similar levels of PNAs. The state of Ohio EPA authorities were in agreement that the levels detected were background to the area and do not present a contamination problem. It should be noted that this is rare to have a state EPA authority make such a reasonable decision with all the conservative toxicity data available. However, our Division Level Corps personnel considered this to be a major issue and stated some or all the material could be hazardous waste even with the EPAs' assessment. Therefor, the Division implemented a $50,000 assessment, not including all the professional time wasted vapor-locking over the situation, to answer <u>all</u> questions and resolve the situation. The $50,000 assessment was generally the same as the previous assessment (cost $10,000) with the exceptions that composite sampling was not implemented and TCLP analysis was included. The results of the $50,000 assessment revealed the material was not characteristic hazardous waste (passed the TCLP criteria) and that PNAs were present in the soils throughout the project area, including the background area (as did the previous assessment).

BRAC Site Investigation at the Indiana Army Ammunition Plant

The Louisville District was tasked by the US Army Toxic and Hazardous Materials Agency

(USATHAMA) to conduct a Site Investigation at the Indiana Army Ammunition Plant (INAAP), near Charlestown, Indiana. The investigation was the follow-up to a Preliminary Assessment of an 860-Acre parcel to be excessed under the Base Closure and Realignment Act (BRAC). The Preliminary Assessment was of marginal quality, and it had never been submitted to the USEPA or the Indiana Department for Environmental Management (IDEM). Based on a review of the Preliminary Assessment, other pertinent information about the property conditions and a discussion of the expectations of USATHAMA, we developed a scope of work, schedule and cost estimate to conduct the investigation. The defined objective was to confirm the presence and locations of chemical contaminants that could pose a significant risk to human health and the environment if they existed. The alternate objective, if no significant contaminates were found, was to convince the Army, IDEM and the USEPA that if they were present, we would have found them.

The INAAP encompassed 10,649 acres along the Ohio River at the time of the investigation. The 860-acre parcel bounds Fourteen Mile Creek at its confluence with the Ohio River. Established in 1941, the INAAP produced large quantities of propellant explosive chemicals for World War II, the Korean War and the Viet Nam War. In addition to production, the plant had many labs, storage and test facilities. Fortunately, only two potential source areas for chemical contamination were located within the subject parcel. They were a rocket test facility and a burn area to dispose of off-spec products and other wastes. However, numerous sinkholes exist in the main production areas of the plant within the Fourteen Mile Creek watershed, and there were reports and evidence that they had been used for waste disposal in the distant past.

We developed a plan to investigate the site that integrated geotechnical, chemical and biological investigation procedures. We submitted the plan to USEPA for review, but we had great difficulty in getting them to provide formal review comments. At this point, a state Senator with an interest in the project began to push USEPA for a formal reply to our request for comment. About five months after the plan was submitted and some of the investigation activities had been initiated to try and stay on schedule, we received a very negative comment letter on the plan. Our evaluation of the comments was that the commenters lacked a fundamental understanding of the size of the area being studied, and the costs of what they were requesting. Convinced that we had the proper approach, we appealed for the reviewers to visit the site to reexamine the relevance of their comments. When USEPA declined this offer, we incorporated the relevant comments we had received, and rejected the ones we did not agree with. The primary point of disagreement revolved around the methods to investigate the potential for groundwater contamination at the site. USEPA requested us to evaluate groundwater quality throughout the site with hundreds of monitor wells.

Instead, we conducted an extensive survey and mapping exercise to locate, inspect and evaluate all of the sinkholes on INAAP property within the watershed. This effort was complimented by a survey during the early spring to locate, inspect and evaluate all of the springs that daylighted along the walls of the Fourteen Mile Creek Valley. Based on these surveys, sample locations were developed for an integrated program of biological and chemical evaluation of water and sediments at the potential source and receptor locations on and off of the subject parcel.

Although we had reasonable concurrence from some of USEPA's technical experts with our approach to incorporating biological, chemical and geologic data into an integrated assessment, USEPA never approved our Work Plan. We submitted the report to USEPA, and received a very critical review regarding the need to evaluate groundwater with monitor wells. We responded by requesting a meeting at the site where all issues could be properly raised, and hopefully resolved, within the context of the physical setting of the property. USEPA reluctantly agreed, and sent a team of five technical reviewers to meet with us at the site. After a thorough site reconnaissance with the report in hand, and our experts there to explain and answer questions, USEPA made a 180 degree shift. They formally approved the report several weeks later, and the property was donated to the state of Indiana for development of a park.

The total cost of the investigation was within the budget of $230,000, and the report was approved within the twenty month schedule originally proposed. The cost was a fraction of the cost that may have been incurred had we accepted the USEPA comments as written. Although the project could only be accomplished by a team of environmental professionals, (nobody can be an expert in every subject, except maybe Chris Karem), the contributions of the project geologist and geotechnical engineers were most influential in accomplishing the objectives within the estimated time and budget for the project.

Superfund Remediation at the Fort Wayne Reduction Site

The Fort Wayne Reduction Site was a municipal solid waste and industrial waste landfill along the Maumee River in Fort Wayne, Indiana. Supposedly, most of the 35-acre site received solid waste, but large volumes of industrial wastes were also disposed of there. There was also a pit at the top of the river bank approximately 50 feet in diameter where liquid waste were disposed directly to the ground. Numerous discolored zones of seepage were located along the river bank. The purpose of including a discussion of this project is to demonstrate all of the geotechnical engineering that went into the remedial design and construction at the site.

The remedial action at the municipal waste portion of the landfill was a soil cover in accordance with RCRA Subtitle C requirements. The geotechnical components of the cover included selecting and evaluating borrow sources based on USCS classifications, stratigraphy and moisture content relative to the compaction and permeability criteria.

The remedial action at the industrial waste portion of the landfill included a similar cover, but there were numerous other aspects of the selected remedial action that required significant geotechnical engineering evaluation, design and construction oversight. The record of decision called for an impermeable barrier extending up to 40 feet deep to be placed between the Maumee River and the waste, and a groundwater collection trench to the same depth be placed on the land side of the barrier. The total length of these features was approximately 1200 feet. Groundwater was to be pumped and treated at a plant to be brought on site. After the trench and barrier were in-place, excavation and removal of and estimated 5,000 buried 55-gallon drums was to be performed.

Design and construction of the trench and barrier were very challenging because of the proximity to the river. A platform was required from which to install these features, which created additional slope stability issues along the bank. Ultimately, a reinforced earth wall was installed over an approximate 400-foot length of the work platform. A vibrating beam slurry injection process was used to construct the impermeable barrier through the work platform. Lab tests indicated that site contaminants would permeate the 6-inch design thickness if a bentonite slurry was used. Instead, an attupulgite slurry was used. Keeping the collection trench open to the desired depth was another geotechnical design issue, and a revert type drilling fluid was necessary for this purpose.

The estimated 5,000 buried drums ended up being closer to 15,000 buried drums, the removal of which delayed the project significantly. The remedial actions at the site are complete, with the exception of the on-going pump and treat operations and the long-term groundwater monitoring. To complete the geotechnical design components of the project, several sampling and drilling operations were necessary during the remedial action process.

APPENDIX
PAST OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I *BUILDING FOUNDATION DESIGN AND CONSTRUCTION*, October 16, 1970, Lexington, KY

ORVSS II *EARTHWORK ENGINEERING, START TO FINISH*, October 15, 1971, Louisville, KY

ORVSS III *LATERAL EARTH PRESSURES*, October 27, 1972, Fort Mitchell, KY

ORVSS IV *GEOTECHNICS IN TRANSPORTATION ENGINEERING*, October 5, 1973, Lexington, KY

ORVSS V *ROCK ENGINEERING*, October 18, 1974, Clarksville, IN

ORVSS VI *SLOPE STABILITY AND LANDSLIDES*, October 17, 1975, Fort Mitchell, KY

ORVSS VII *SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION*, October 8, 1976, Lexington, KY

ORVSS VIII *EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION, AND PERFORMANCE*, October 14, 1977, Louisville, KY

ORVSS IX *DEEP FOUNDATIONS*, October 27, 1978. Fort Mitchell, KY

ORVSS X *GEOTECHNICS OF MINING*, October 5, 1979, Lexington, KY

ORVSS XI *EARTH PRESSURES AND RETAINING STRUCTURES*, October 10, 1980, Clarksville, IN

ORVSS XII *GROUNDWATER: MONITORING, EVALUATION, AND CONTROL*, October 9, 1981, Fort Mitchell, KY

ORVSS XIII *RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE*, October 8, 1982, Lexington, KY

ORVSS XIV *FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION*, October 14, 1983, Clarksville, IN

ORVSS XV *PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING*, November 2, 1984, Fort Mitchell, KY

ORVSS XVI *APPLIED SOIL DYNAMICS*, October 11, 1985, Lexington, KY

ORVSS XVII *NATURAL SLOPE STABILITY AND INSTRUMENTATION*, October 17, 1986, Clarksville, IN

ORVSS XVIII *LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION*, November 6, 1987, Fort Mitchell, KY

ORVSS XIX *CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES*, October 21, 1988, Lexington, KY

ORVSS XX *CONSTRUCTION IN AND ON ROCK*, October 27, 1989, Louisville, KY

ORVSS XXI *ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING*, October 26, 1990, Fort Mitchell, KY

ORVSS XXII *DESIGN AND CONSTRUCTION WITH SYNTHETICS*, October 18, 1991, Lexington, KY

ORVSS XXIII *IN-SITU SOIL MODIFICATION*, October 16, 1992, Louisville, KY

ORVSS XXIV *GEOTECHNICAL ASPECTS OF INFRASTRUCTURE RECONSTRUCTION*, October 15, 1993, Fort Mitchell, KY

ORVSS XXV *RECENT ADVANCES IN DEEP FOUNDATIONS*, October 21, 1994, Lexington, KY