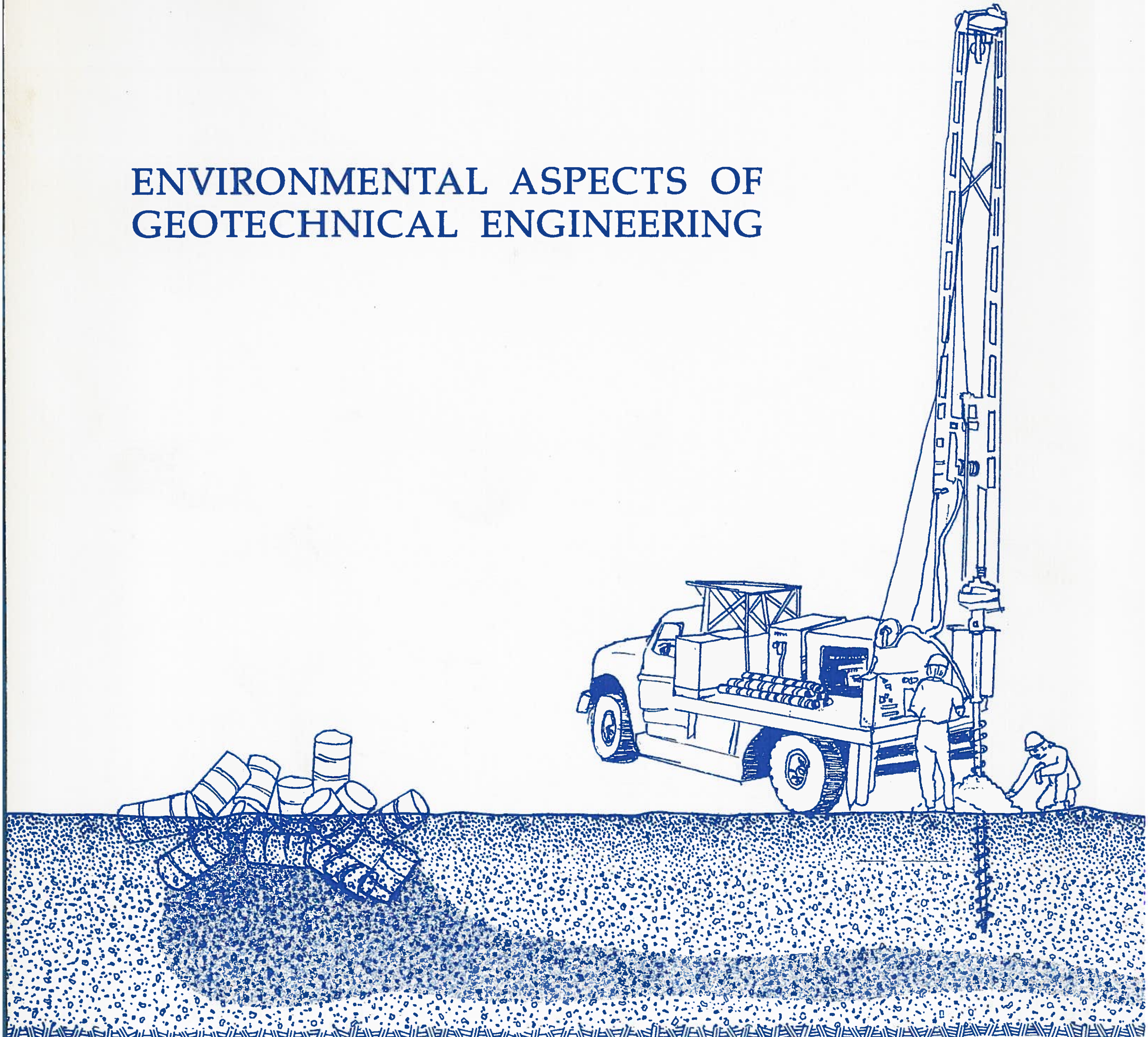
Ohio River Valley Soils Seminar XXI

ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING



PROCEEDINGS
Cincinnati, Ohio
October 26, 1990

PROCEEDINGS OF THE TWENTY FIRST OHIO RIVER VALLEY SOILS SEMINAR

ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING

October 26, 1990
Drawbridge Inn
Ft. Mitchell, Kentucky

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ORVSS - XXI

ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING

OCTOBER 26, 1990

PROGRAM

7:30 a.m.	Registration
8:30 a.m.	Welcome and Opening Remarks
8:45 a.m.	Keynote Speaker Address - Mr. Ronald D. Hill, Director of USEPA Waste Minimization, Destruction, and Disposal Research
9:15 a.m.	"Dynamics of Unsaturated Flow: An Examination of Environmental/Geotechnical Problems" - Dr. Andrew G. Heydinger, University of Toledo
9:45 a.m.	"Environmental Effects of Bottom Ash as a Geotechnical Material" - Dr. C.W. Lovell, Purdue University
10:15 a.m.	"The Permeability Test in Environmental Geotechnology" - Dr. Brian W. Randolph, University of Toledo
10:45 a.m.	Break
11:00 a.m.	"State of Stress and Hydraulic Fracturing Potential in Soil/Bentonite Cut-Off Walls" - Mr. R. Curtis Spence, The H.C. Nutting Company
11:30 a.m.	"Mechanisms, Impacts, and Modeling of Chemically-Induced Changes in Saturated Soil Hydraulic Conductivity" - Dr. Aaron A. Jennings, University of Toledo
12:00 p.m.	"Effects of Freeze/Thaw on the Hydraulic Conductivity of Compacted Soils" - Dr. John J. Bowders, West Virginia University
12:30 p.m.	Lunch
1:45 p.m.	"Environmental Drilling: The Critical Phase for Geoenvironmental Consultants" - Mr. Richard C. Wells, Trigon Engineering Consultants
2:15 p.m.	"Computer Aided Assessment of Contaminated Sites" - Mr. P.R. Cluxton, University of Cincinnati

2:45 p.m.	"Increased Permeability of Soils by Hydraulic Fracturing: A Field Test" - Mr. L.C. Murdoch, University of Cincinnati
3:15 p.m.	"Interpretation of Field Permeability Test Results on Full Scale Liner Systems" - Mr. John A. Mundell, ATEC Environmental Consultants
3:45 p.m.	Break
4:00 p.m.	"Geotechnical Considerations at the Lake Sandy Jo Superfund Site" - Mr. David J. Lane, CH2M Hill
4:30 p.m.	"Geotechnical and Environmental Considerations for Highway Construction in Mountainous Terrain with Acid-Producing Bedrock" Mr. Daniel J. Hurst, ERC/Edge
5:00 p.m.	Social Hour and Buffet

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PROGRAM SPEAKERS

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Dynamics of Unsaturated Flow:

An Examination of Environmental/Geotechnical Problems

Dr. Andrew G. Heydinger

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The University of Toledo
Toledo, OH 43606

ABSTRACT

For situations where it is possible that contaminants will move through soil, it is necessary to determine the movement of water through soils in order to know the likely direction and extent of contaminant migration. This presentation examines the fundamental physical phenomena that occur in unsaturated flow through porous media. This is accomplished by illustrating the development of the equations governing the flow of fluids in unsaturated porous media using the continuum approach. Simplifying assumptions are discussed wherever they are required. The governing equations are posed with concluding remarks concerning required constitutive relations and possible boundary conditions.

After presenting the physical laws governing flow, the presentation focuses on Environmental/Geotechnological problems that have been and will continue to be confronted by practitioners and researchers. This discussion utilizes three examples that require a full understanding of the underlying physical phenomena of unsaturated flow. The examples included are: flow through waste disposal facility liners; flow from an above-grade low-level radioactive waste disposal facility; and subsurface discharge to surface waters with and without underdrain dynamics. Each of the examples are depicted conceptually with a description of the flow domain. The impacts of the various factors that effect flow are discussed. The emphasis of this part of the discussion is on the possible flow of water and plausible environmental hazards that could result.

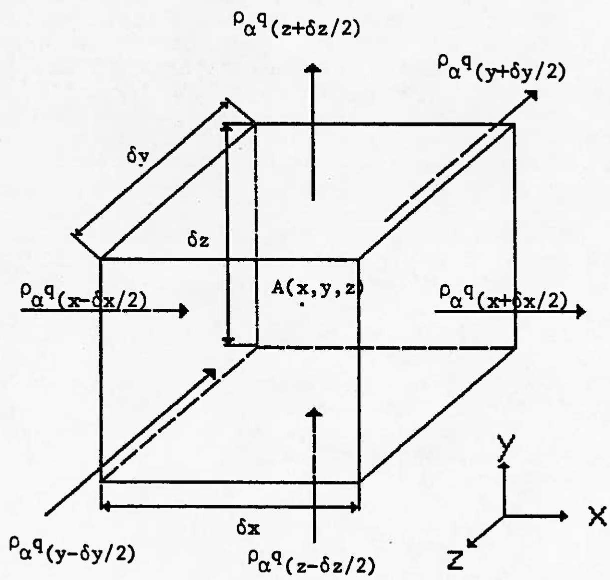
INTRODUCTION

Environmental problems have impacted geotechnical engineering only relatively recently. Sanitary engineers have dealt with the production of potable water and disposal of waste waters for centuries. Environmental contamination produced by modern industry spurred the advent of environmental engineering as a discipline in the 1960's. However, it has been more recent that engineers have dealt with problems posed by contaminants moving through soil which has required the expertise of geotechnical engineers to perform site characterization studies, to design and supervise construction of monitoring and recovery systems, facility liners and subsurface barriers, and to conduct analysis of the fate of subsurface contaminants.

The purpose of this presentation is to discuss the governing equations for unsaturated flow in soil and to illustrate how the various factors effecting flow might impact environmental hazards. The physical laws governing flow and simplifying assumptions are discussed, and the governing equations are posed. Since the intent is to provide a better understanding of the dynamics of flow in soil, this discussion is limited to the flow of water in soil and does not include contaminant transport. After discussing the equations, three examples where environmental hazards exist are illustrated. The examples are discussed in light of the underlying physics so as to examine factors which might effect the flow and the resulting environmental hazards.

DYNAMICS OF UNSATURATED FLOW

Equations governing unsaturated flow are derived by considering the mass conservation of the gas, liquid and solid phases in the soil. Fig. 1 depicts a control or representative volume in a porous media. The total quantity of flow for either the gaseous or liquid phases can be obtained by summing up the net flow, the difference between the flow into one face and out of the opposite face of a representative volume, in each of the three directions. Then, by the principle of mass conservation, the total flow of a particular phase must be equal to the



Net Quantity of flow in x-direction:

$$[\rho_\alpha q_{(x+\delta x/2)} - \rho_\alpha q_{(x-\delta x/2)}]\ \delta y\ \delta z$$

$$= \partial(\rho_\alpha q_x)/\partial x\ \ \delta x\ \delta y\ \delta z$$

Change of mass per unit time:

$$\partial(\rho_\alpha\ n\ S_\alpha)\ /\ \partial t\ \delta x\ \delta y\ \delta z$$

Fig. 1 - Control Volume for Flow Through Porous Media.

change of mass per unit time, as shown in Eq. (1).

$$\partial(\rho_\alpha q_x)/\partial x + \partial(\rho_\alpha q_y)/\partial y + \partial(\rho_\alpha q_z)/\partial z = \partial(\rho_\alpha\ n\ S_\alpha)/\partial t \qquad (1)$$

Where ρ_α is the mass per unit phase volume; n is the porosity; S_α is degree of saturation; q_α is the discharge or "Darcian" velocity; and the subscript α refers to the phase. Eq. (1) must be written for both the gaseous and liquid phases and the two equations must be solved simultaneously with Eq. (2), where subscripts a and w refer to the air and water phases.

$$S_a + S_w = 1 \qquad (2)$$

A requirement for the governing equations is that the fluids must be homogeneous. For the types of problems normally encountered with groundwater flow, the assumption of a homogeneous fluid is appropriate. However, for many cases of unsaturated flow, it is possible that there will be dissolved air in the water and water vapor in the air. This mass is accounted for by adding the mass densities of the dissolved air and the water vapor to the mass densities for each respective phase in Eq. (1). The mass density of the water vapor is obtained by assuming that the gas follows the "ideal gas" law. The mass density of the dissolved air is obtained by assuming that the gass dissolution follows Henry's Law.

To obtain a mathematical solution to flow problems, it is necessary to have an equation relating the discharge (equal to the discharge velocity times the cross-sectional area of flow) to the cause of flow, the hydraulic gradient. This law, Darcy's law, is based on experimental evidence that, for a range of gradients, the discharge q_i is proportional to the hydraulic gradient as shown in Eq. (3).

$$q_{ij} = k_{ij}(\gamma/\mu)\ \partial h/\partial x_j = K_{ij}\ \partial h/\partial x_j \qquad (3)$$

In the above equation, k_{ij} is the intrinsic permeability; γ and μ are the fluid's unit weight and coefficient of viscosity; h is the total or piezometric head equal to the sum of the pressure and elevation heads; and K_{ij} is the hydraulic conductivity or coefficient of permeability. The subscript indicates that for anisotropic media a second rank tensor must be used to represent permeability. Solutions are obtained by substitution of Eq. (3) into Eq. (1) which then results in the continuity equation.

Darcy's law was first evidenced to be true for saturated flow. For unsaturated flow where the gas could also flow, the gas and liquid components can be treated as immiscible fluids (1). Darcy's law is then applied to the gas and the liquid phases separately with each having their own permeability. For this case the gas is compressible. Alternatively, it may be assumed that there is a single-phase, compressible fluid. For compressible fluids it is necessary to define a potential in terms of the fluid compressibility which is normally expressed as a function of pressure but which also may be dependent on temperature.

Also for unsaturated flow, the permeability is highly dependent on the degree of saturation. Darcy's law is modified with the addition of a factor multiplying permeability, relative permeability, which is the ratio of the permeability of the fluid with saturation less than 100% to the permeability at 100% saturation. Thus relative permeability is a number between zero and one. Relationships for relative permeability as a function of degree of saturation are unique functions for each fluid and medium. Additionally, there is a hysteresis effect for fluids between cycles of wetting and drying. The influence that multi-component flow has on relative permeability is not well understood.

Mass conservation and Darcy's law exemplify the continuum approach. It is assumed that material properties and physical phenomena occurring within a control volume can be modeled for the entire volume rather than at discrete points. This macroscopic treatment of media does not account for microscopic or molecular effects directly. For example, viscous flow effects are "averaged out" over the media and accounted for using a parameter, hydraulic conductivity. Other effects which are highly dependent on temperature, such as molecular diffusion, are assumed to be negligible compared to convective mass transport for most flow problems. However these may not be negligible for contaminant transport problems. The continuum approach enables one to diregard variables such as tortuousity in order to solve problems which would otherwise be impossible to solve.

FLOW IN DEFORMING MEDIA

For many problems the assumption is made that the soil is nondeformable. This assumes that the soil solids are incompressible and that there is no particle movement. The soil porosity would then be constant and Eq. (1) can be simplified. However, there are situations where the soil is being loaded (e.g. consolidation) or where soil stresses are changing because of fluid flow (e.g. subsidence due to pumping) in which the soil is deforming.

For the case of deforming media, it is necessary to account for the particle mass movement. One approach that can be used in a solution is to uncouple the solid deformation from the flow effects by first solving for flow and the resulting changes in stress in the soil and then using an equation of state relating stress and volume to compute the state.

A second approach that is used is to couple flow and deformation by solving equations for the mass conservation of the fluids and the solids simultaneously with the force equilibrium equations. An equation similar to Eq. (1) can be written for the mass of soil solids. In using Darcy's law for the fluid flow, it may be necessary to account for the solids' velocity since Darcy's law is given in terms of the fluid velocity relative to the solid velocity (2). Relative velocity can be computed by taking the difference between the fluid and solid velocity. It can also be assumed that solid velocity is negligible.

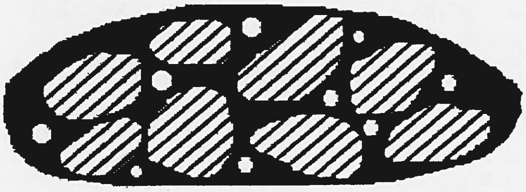
To have a complete set of equations sufficient to solve for all the unknown quantities (say fluid pressure, soil effective stress, state volume), it is necessary to use a constitutive model that expresses the relationship between the soil's effective stress and strain. Numerous models have been proposed employing different theories (e.g. elasticity or plasticity) each requiring different material parameter identifications. Compatibility equations are used to relate soil deformation and strain.

A major difficulty that is encountered when using either of the two approaches mentioned previously is to determine the effective soil stresses in partially saturated soils. Effective stress is defined as the difference between the total stress resulting from all mechanical forces acting on or in the soil and the fluid pressures acting within the soil. For partially saturated soils it is possible that the air and liquid pressures will differ. It is difficult to relate the fluid pressure in the soil and the capillary tension acting on the fluid to the soil effective stress (3,4).

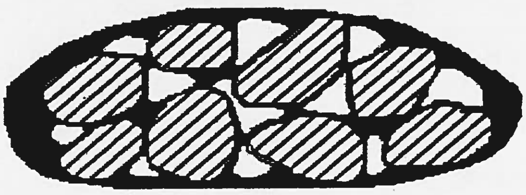
ASSUMPTIONS FOR PARTIALLY SATURATED SOILS

The flow in partially saturated soil is dependent on the degree of saturation of the soil (5,6). For soils in which the degree of saturation approaches 100% (e.g. soil compacted at optimum moisture content or above), there is negligible air flow (4). The air present in the soil is assumed to behave much like the soil particles (i.e. acting like occluded "particles" that are not flowing) (See Fig. 2 a). The pressure in the air is assumed to be equal to the liquid pressure. Since the air is not flowing, it is not necessary to include mass conservation of the air provided that fluid compressibility is accounted for. If these conditions exist, then the effective stress of the soil can be computed as if the soil were saturated.

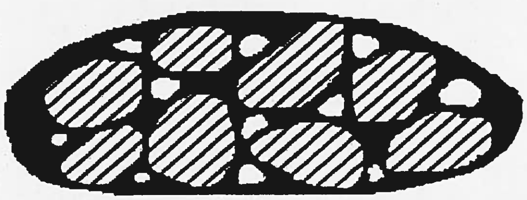
Another condition occurs when the degree of saturation is low (See fig 2 b). This situation occurs in soil zones that are in contact with the atmosphere. The air flow is highly dependent on atmospheric conditions. It can be assumed that the air pressure is equal to atmospheric pressure and so consequently liquid flow



a.) Soil With High Degree of Saturation.



b.) Soil With Low Degree of Saturation.



c.) Soil With Intermediate Degree of Saturation.

Fig. 2 - Degrees of Saturation.

is also dependent on ambient conditions. The liquid flow is vertical only and can be upward if the capillary forces are larger than gravitational forces. The direction of flow depends upon whether the soil is experiencing a wetting or drying cycle. The effective stress in the soil is highly dependent on the fluid pressure. However it is very difficult to determine how much of the fluid pressure is transferred to the soil.

Conditions intermediate between the two previous illustrations can exist (see Fig. 2 c). This occurs in soils in which both the liquid and air are nearly continuous in the soil. Atmospheric conditions do not impact the soil fluid pressure directly. The air can be flowing or it can be occluded. The liquid can have horizontal and vertical components of velocity. Again it is difficult to determine the effective stresses in the soil.

BOUNDARY CONDITIONS

Sufficient boundary conditions are required for engineering problems in order to obtain solutions for the flow domains. The first requirement is that all the initial conditions must be known. This necessitates knowledge of the piezometric head or velocity potential, location of any free surface or surface of seepage, and knowledge of the conditions of state (e.g. porosity, degree of saturation, effective stress).

Surface boundary conditions occur wherever the piezometric head along the boundary is known (usually constant) at a given time. If a flow domain is in contact with a free body of water or the piezometric surface does not change, then the piezometric head along the boundary is constant. The solution must be formulated so that the piezometric head or the velocity potential (equal to the product of the coefficient of permeability and the piezometric head plus a constant) is satisfied along the boundary.

The flux or flow normal to a boundary can be assumed for some problems. For this case the discharge velocity would be known so the gradient, derivative of the piezometric head with respect to the direction normal to the boundary, would be known. The flux can either be given as a function or may be constant. For an impervious boundary the flux is equal to zero.

A special case of the previous condition exists if there is a thin layer of nearly impervious material along a boundary. For this situation the flux is controlled by the piezometric heads along either side of the layer, the thickness of the layer and the hydraulic conductivity of the layer. Using these factors, the gradient and the flux through the layer can be computed.

Transient conditions can exist for the above examples. That is the piezometric head or the flux can change with time. It is necessary to be able to express the change as a function of time. The boundary conditions can then be given in terms of a function that is dependent on time so that a solution can be obtained in time.

For other problems the phreatic surface, defined as the location where the pore water pressure is equal to zero, can vary. This is a different condition from the transient conditions just described in that the location of the phreatic surface is unknown and is dependent on the solution. Since it is necessary to solve for an unknown (say the flow) in terms of unknown piezometric head, then an iterative scheme must be developed that ensures the solution for both unknowns. For this case it is necessary to calculate the velocity and the flux of water flowing with the phreatic surface.

A flow domain can also be bounded by a surface of seepage. A surface of seepage exists wherever flow is exiting from the medium. The pressure is equal to atmosphere pressure (zero). Since the pressure is equal to zero, the piezometric head is equal to the elevation head. The direction of flow across the boundary is not perpendicular to the boundary since the piezometric head in not constant along the boundary. Transient conditions can also exist in which the location of the intersection of the phreatic surface and the surface of seepage varies.

For some problems it might be necessary to consider the state of water in a capillary fringe. A capillary fringe exists above the phreatic surface wherever the soil is saturated due to capillary rise of water. The capillary fringe is bounded by the phreatic surface, the top surface where the pressure is less than atmospheric (negative), and possibly by a surface directly exposed to the atmosphere. For steady conditions the boundaries of the capillary fringe remain constant so the flux is equal to zero. For transient conditions the boundaries may be changing and there could be some flux. The flux can be expressed in terms of Darcy's law.

Interface conditions also occur along the interfaces between two layers with different hydraulic conductivities. Solutions for layered systems can be obtained by assuming that the piezometric head along the boundary is equal on both sides of the boundary or that the flow is continuous across the boundary. Because of the continuity condition, it can be shown for two-dimensional flow that the streamlines (lines of the direction of flow) are refracted as flow passes through the boundaries.

FLOW THROUGH LINER SYSTEMS

Liner systems are designed and constructed as an integral part of waste containment facilities. They typically consist of at least one layer of low permeability (compacted clay) and one layer of impervious (geosynthetic) material which are intended to act as a barrier to contaminants. Liners also include pervious layers with collection conduits that are used to monitor or collect fluid. (See Fig. 3.) If a liner is designed and constructed properly, the liner will effectively eliminate the release of materials to the subsurface. However, as in the case of most geotechnical design, it can not simply be assumed that a system will function as intended.

Initially the compacted clay in a liner system is partially saturated with no flux or driving gradients at the boundaries. The soil should be compacted at or above the optimum moisture content and kept from drying at all times to prevent the development of desiccation fissures (7). The soil will then be near

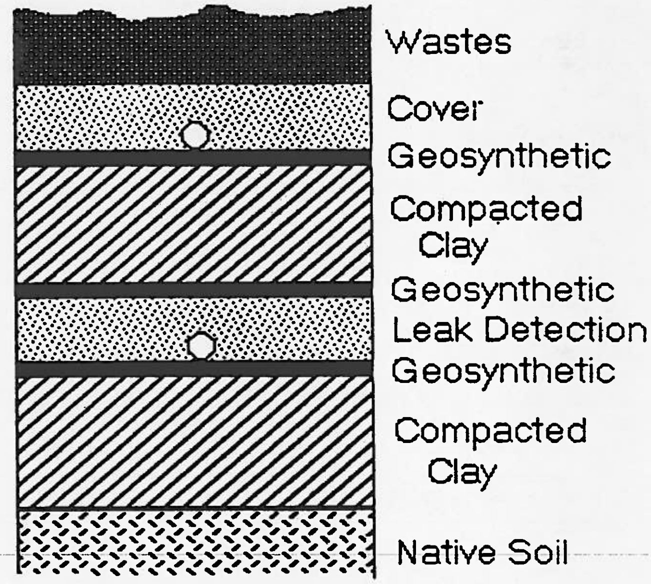


Fig. 3 - Vertical Section of a Redundant Liner System.

100% saturation and should be nearly isotropic with respect to permeability. The geosynthetic layer placed above the compacted soil protects the compacted clay from environmental effects (wetting and drying) and ensures that liquids that collect above the liner do not immediately result in the inflow of liquids into the clay. Unless the clay is vented, air is trapped in the clay.

A number of boundary conditions can occur after the facility becomes operational. The upper or lower boundary may be subject to constant or varying hydraulic gradients. There can be a constant or varying flux if there is a leaky flexible membrane liner above the soil layer. The weight of the disposal materials imposes a loading on the liner. The loading will result in the development of hydraulic gradients which then cause fluid compression or flow with resulting changes in soil porosity, degree of saturation and permeability (i.e. soil consolidation).

In order to solve for the flow through liners, it is necessary to model the physical phenomena that occur. For situations where flow is induced by a gradient or flux acting at a boundary, the flow might consist of gaseous or liquid phases. As flow continues, the soil will become more saturated (degree of saturation approaching 100%) as the air flows from the soil or is dissolved by the liquid. Air may then reach a point where it is occluded. The soil might be assumed to be of constant volume (nondeforming) so the soil porosity would not change. Flow can be modeled by satisfying mass conservation of the liquid and gaseous phases and by using a constitutive model relating relative permeability to the soil's phase state (usually using volumetric moisture content).

If it is anticipated that large loads will be applied, then the soil will be deforming (decrease in porosity) and it is necessary to consider the mass conservation of the soil solids. Depending on how a solution is formulated, it is necessary to have a constitutive model relating effective stress and strain or a model that expresses volume relationships in terms of the state of effective stress. If a stress-strain model is used, then it is necessary to determine the extent to which the fluid pressure is applied to the soil. For the case of a compacted clay it may be reasonable to assume that the air and liquid pressures are equal and that the fluid pressure is fully applied to the soil.

For the case of a deforming soil, it is necessary to consider the soil's relative permeability and its intrinsic permeability. It may be necessary to express relative permeability as a function of both porosity and degree of saturation instead of volumetric moisture content which is equal to the product of the two. According to conventional testing procedures, relative permeability is obtained from tests on nondeforming soil (constant volume) where the degree of saturation is varied (6). The intrinsic permeability is highly dependent on soil porosity so the use of an expression relating intrinsic permeability and porosity would be a valid approach.

There are several questions that arise when flow is detected in a monitoring system. One must first consider the potential sources of flow. Is the liquid coming from expulsion of water from the compacted liner, from the disposal area or from regional ground water? The site conditions must also be considered. Was there a conspicuous design or construction problem? Are there large breaches in the impervious liner? Is there a chemical-liner compatibility problem? The fate of contaminants and the possible health risks caused by the contamination are a major concern. For this type of problem, the flow domain may include the liner system (including any collector system) and the subsurface soil.

LOW-LEVEL RADIOACTIVE WASTE FACILITY

Low-level radioactive waste facilities are presently being designed in several states because of recent congressional legislation (Public Law 99-240). Each facility will receive wastes from member states that have formed compacts. The facilities will replace three below-grade facilities that presently receive most of the low-level radioactive wastes in the United States. Because of past problems with disposal of radioactive wastes (8) and because of great public concern over the disposal of radioactive wastes, it has been decided to use above-grade, vaulted

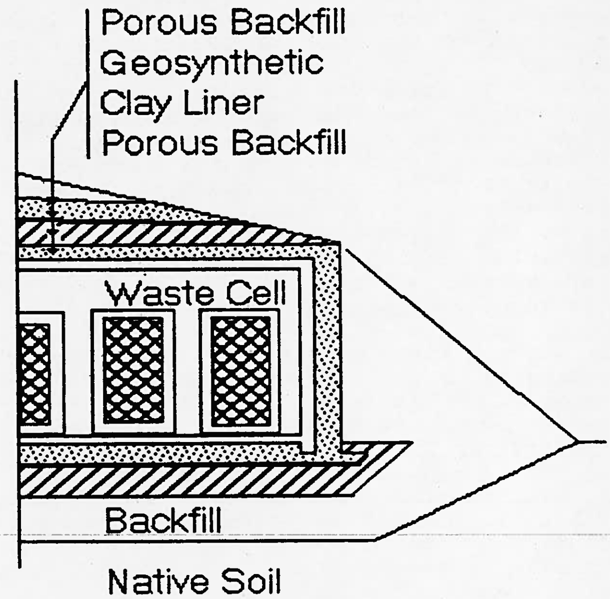


Fig. 4 - Section Through a Low-Level Radioactive Waste Cell.

disposal facilities.

Presently no facility design has been finalized and the governmental agencies that will be responsible for reviewing the designs have not necessarily adopted their final regulations. Therefore it is necessary to speculate the details of an actual design. Fig. 4 illustrates a cross-section of a vaulted structure based on a preliminary plans released in May, 1990 for a facility to be constructed in Illinois. The design will include redundant monitoring systems within the individual vaults and an access chamber below the vaults. The designs will probably also include a liner system that will be placed in the soil cover around the vaults. All radioactive materials must contain no liquids and will be placed in approved containers before being transported to the disposal sites.

There are several federally mandated regulations that severely restrict the site conditions for a facility. Facilities must be isolated from all surface and ground water. Thus sites with ground water tables near the ground surface will probably be eliminated. There must be a layer of low permeability soil (10^{-6} cm/sec) that is at least six meters thick below the site. The travel time for ground water migrating from the vaults to the boundaries of the buffer zone surrounding the vaults (approximately 3000 feet) must exceed 500 years. There can be no ground water discharging to surface waters within the buffer zone. The surface conditions must be such that there is no surface ponding of water.

In spite of the above-mentioned stipulations, a plausible source of release of radioactive wastes could come from combined subsurface and surface water transport. Fig. 5 shows a section of a facility where radioactive contaminants are released. In order for the release to occur, there would have to be a failure of a waste container. The wastes would have to be in contact with liquids that are migrating from the containment structure through an inadvertent mode of access. The radioactive material would then migrate though the soil following the same general path as moisture or gas that is moving though the soil until it reaches the ground surface. The contaminant could then be carried away by surface runoff following the next heavy rainfall.

In order for this particular episode to occur, there would have to be a period of seasonal drying. If it is assumed that the location where the contaminants exit from the vault is located above the point where they intersect the ground surface, then there would have to be capillary forces or other forces due to atmospheric conditions acting on the moisture in order for it to migrate to the ground surface. The domain through which contaminants are migrating includes zones of partially saturated soil (below the structure and above the capillary fringe) and zones of saturated soil (below the ground water table and in the capillary fringe).

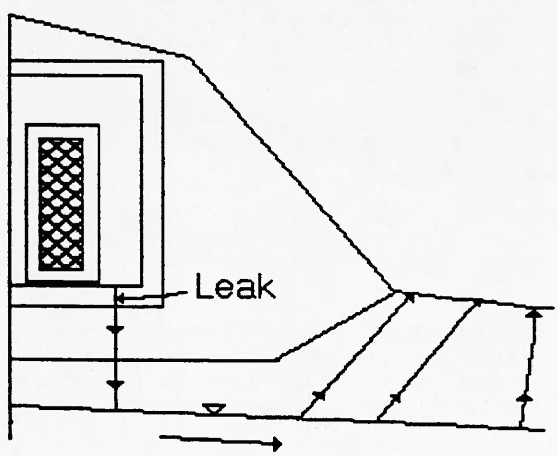
A solution can be obtained for moisture migration assuming convective transport in nondeforming soil. Convective transport would be the driving mechanism for the moisture as it moves downward due to gravitational forces and upward towards the ground surface due to capillary forces. A solution is obtained using the continuity equation and empirical relationships for unsaturated permeability as a function of degree of saturation. The solution would be highly dependent on the ambient atmospheric conditions, rain water infiltration, location of the ground water table and transpiration uptake. It is also possible where the hydraulic gradient is very low that flow would not be governed by Darcy's law in which case flow may be due to molecular diffusion.

SUBSURFACE DISCHARGE TO SURFACE WATERS

Problems associated with ground water discharge to surface waters are primarily encountered in agriculture where surface water quality can be endangered by fertilizers and pesticides. Clean water legislation passed in recent years has mandated practices that reduce the discharge of agricultural byproducts to surface waters. Solutions for groundwater discharge are consequently employed in order to assess the effects of different practices (such as application rates) under possible conditions (9).

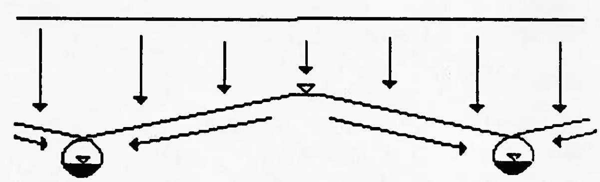
The greatest likelihood for surface water contamination occurs when the ground water table is located close to the ground surface. Fig. 6 depicts flow to surface waters with and without underdrain systems. The figure indicates that the direction of flow is primarily vertical above the water table and is horizontal below the water table.

A solution can be obtained for the convective transport of the moisture. One approach is to include in the continuity equation a term that accounts for evapo-transpiration. The resulting equation is referred to as the Richard's equation for saturated-unsaturated flow. This assumes that the soil is nondeforming which is not true particularly near the ground surface. It is necessary to model the flow of water with time beginning with known initial conditions and including episodes of rainfall and agricultural interactions.

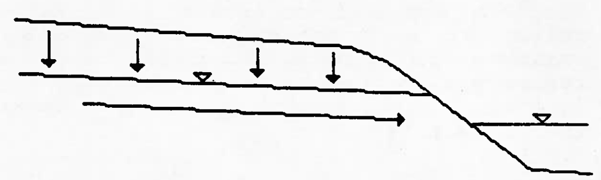


Groundwater Flow

Fig. 5 - Transport of Radioactive Waste to the Ground Surface



a.) Flow to Underdrain System



b.) Flow to Surface of Seepage.

Fig. 6 - Groundwater Discharging to Surface Water.

Solutions for moisture migration are highly dependent on the boundary conditions. The location of the ground water table changes with time. The location and capacity of underdrains significantly impact ground water movement. Regional ground water flow effects domains without underdrains. The uptake of water by plants can be estimated for various stages of plant growth. The amount of water lost to evaporation is highly dependent on atmospheric conditions. The amount of infiltration is highly dependent on ground surface conditions which depends on agricultural use.

The use of exact solutions to model ground water discharge has been limited due to the large amount of information that is required for the solution and due to the large amount of computer time required. It is necessary to input specific information on climatic and agricultural conditions that occur over time. Soil properties including intrinsic permeability and relative permeability must be available for homogeneous, isotropic soils. If it is not assumed that soil permeability is homogeneous or isotropic, then it is necessary to estimate the spatial variation of permeability (nonhomogeneous soils) or the horizontal and vertical permeabilities (anisotropic soils). A solution that models the ground water discharge for a single growing season would require a large amount of CPU time (9).

Approximate solutions that model ground water flow in the unsaturated zone are more widely used. The solutions make use of the water balance approach which accounts for the volumes of water entering and exiting the unsaturated zone by using approximate methods to calculate the various components of flow (e.g. drainage, evapotranspiration, infiltration). These solutions have been used for agricultural purposes and for a computer solution, Hydraulic Evaluation of Landfill Liners (HELP) that models the flow through landfill liners.

SUMMARY

It is very difficult to develop mathematical solutions that solve the governing equations while satisfying all possible initial and boundary conditions. The required soil parameters are also very difficult to quantify. Therefore, the use of exact solutions is usually reserved for research applications while practitioners resort to more approximate solutions to estimate flow. It is emphasized here that, regardless of the method of analysis, it is important to understand all the factors that effect flow.

It may be apparent from the previous discussion that the final result of an environmental assessment would be the determination of concentrations or quantity of contaminants released to the environment. However, it is also necessary to determine the health risks posed by releases. Thus a determination might be made that, although contaminant release will occur, the hazard to living organisms is not sufficient to warrant remediation. The objective of engineers should be to eliminate or reduce environmental hazards to the maximum degree that is technically and economically feasible. Thus it is necessary to analyze situations for possible environmental hazards as discussed in this paper.

BIBLIOGRAPHY

1. Bear, J., Dynamics of Fluids in Porous Media, 1'st ed., Amer. Elsevier Publ. Co., Inc., New York, 1972, 764 p.

2. Biot, M.A., "Theory of Elasticity and Consolidation for a Porous Anisotropic Solid," J. of Appl. Phys., 26(2), Feb, 1954, 182-185.

3. Aitchison, G.D., Bishop, A.W., Jennings, J.E. and Skempton, A.W., "Pore Pressure and Suction in Soils," Conference, Instit. of Civil Eng., London, 1960.

4. Chang, C.S. and Duncan J.M., "Consolidation Analysis for Partly Saturated Clay by Using an Elastic-Plastic Effective Stress-Strain Model", Inter. J. for Num. and Anal. Meth. in Geomechanics, 7, 1983, 39-55.

5. Sparks, A.D.W., "Various Aspects of Soil Moisture", Proc. of 3rd Reg. Conf. for Africa on S. Mech. and Found. Eng., Salisbury, S. Rhodesia, June, 1963, 211-214.

6. Bear, J., "Flow in the Unsaturated Zone", Hydraulics of Groundwater, 1'st Ed., McGraw-Hill Book Co., New York, 1979, 190-224.

7. Daniel, D.E., "Predicting Hydraulic Conductivity of Clay Liners", J. of Geot. Eng., 110(2), Feb, 1984, 285-300.

8. Daniel, D.E., "Shallow Land Burial of Low-Level Radioactive Waste", J. of Geot. Eng., 109(1), Jan, 1983, 40-55.

9. Skaggs, R.W., "Model Development, Selection and Use", Proceedings, Third Inter. Workshop on Land Drainage, Ohio State University, 29-39, Dec, 1987.

ENVIRONMENTAL EFFECTS OF BOTTOM ASH AS A GEOTECHNICAL MATERIAL

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ABSTRACT: Although recent investigations indicate that bottom ashes have engineering properties similar to granular soils and can be extensively used as a geotechnical material, the environmental acceptability of their utilization remains uncertain. The evidence presented in this paper reduces this uncertainty. Laboratory studies performed on selected Indiana bottom ashes included the evaluation of five possible environmental impacts. The findings show that untreated bottom ashes are of a nonhazardous nature, have minimal effects on the groundwater quality, are of low radioactivity, and have low erodibility. However, they may be potentially corrosive to metals. In general, bottom ashes are very promising as embankment, subgrade, subbase, and even base materials, in accordance with their favorable mechanical behavior and minimal environmental effects. However, in some cases, they should not be placed in the near vicinity of any metal structure to avoid possible corrosion failure of such a structure and metal ion pollution.

INTRODUCTION

Recently the diminishing supply of natural high-quality mineral aggregates has directed attention toward substitute materials. In the Midwest a huge quantity of coal ash is produced from utility power plants and at an increasing rate. Disposal of this ash is costly and may cause environmental problems, especially in urban areas. If greater ash utilization were to occur, not only could a solid waste disposal problem be solved, but also an economic alternative construction material could be provided. Thus a saving could be realized, on a local or regional basis, for both the electrical utilities and construction agencies. However, before such benefits are achieved, chemical and physical properties as well as mechanical behavior of coal ashes must be thoroughly examined, to insure that they are suitable alternates for natural mineral aggregates.

Coal ash includes two components: bottom ash and fly ash. Bottom ash is the slag which builds up on the heat-absorbing surfaces of the furnace, and which subsequently falls through the furnace bottom to the ash hopper below. It is a relatively coarse material. In contrast, fly ash is the fine-grained dusty material that is recovered and collected from furnace flue gases by ash precipitators.

In the past, considerable information has been accumulated on the properties of fly ash, and it has seen extensive use in soil stabilization and as cement replacement for concrete. However, the information for bottom ash is much more limited. Recent study of Indiana bottom ashes has shown them to have engineering properties similar to typical granular soils [1]. Accordingly, they are suitable for use as a geotechnical material in a variety of applications such as highway embankments, subgrades, subbases, and bases.

Environmental acceptability of bottom ash must also be established prior to routine use. With a growing awareness of environmental protection, only those uses of bottom ash which can be shown to have minimal environmental impacts will be successful. This study evaluated five primary environmental impacts associated with bottom ash utilization. They are: the status of ash as a hazardous material; the effects of ash on groundwater quality; ash corrosiveness; ash radioactivity; and as erosion potential.

FORMATION AND NATURE OF BOTTOM ASH

Coal ash is the incombustible mineral matter which accounts for 10-20% by weight of the coal consumed in power plants. Ash characteristics are affected by the type of boiler and coal combustion method, as well as by the type of coal burned. Depending upon the boiler type, the bottom ash under the furnace is categorized as "dry bottom ash" or "wet bottom ash".

If the ash is in a solid state at the furnace bottom, it is termed dry bottom ash. The ash particles are usually gray-colored, irregularly shaped, and highly porous. Wet bottom ash is more often called boiler slag. The word "wet" refers to the molten state of the ash which leaves the furnace as a liquid. The molten ash is quenched in the water-filled hopper to form boiler slag. Boiler slag is a hard, black, and angular material with a smooth surface texture.

Dry bottom ash is a relatively well-graded material, with less than 10% fines. The fines are non-plastic and silt-like particles. In contrast, wet bottom ash has a quite uniform gradation and is essentially free of fines. Grain size distributions affect the permeability as well as the erodibility of bottom ash layers.

The chemical composition of bottom ash affects its leaching properties, although it is not evident that a proportional relationship exists between the constituents of ash and ash leachate. The principal constituents of bottom ash are silica (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3). There are smaller quantities of calcium oxide (CaO), magnesium oxide (MgO), potassium oxide (K_2O), sodium oxide (Na_2O), and sulfur trioxide (SO_3), as well as minute traces of other elements.

Other engineering properties of bottom ash are reported by Sears [2] and Huang [3].

ASH DISPOSAL

Basically, ash handling and disposal is accomplished either by a wet or a dry method. Dry disposal implies transport and deposition of dry or moistened ash. This may involve temporary storage of ash in silos, subsequent hauling by trucks, and compacting at a waste fill. Most power stations in urban areas are handling their ash by the dry method, due to land limitation.

The wet method of disposal uses water to flush ash through pipelines to settling ponds or lagoons. This method is more commonly used and is more economical. Ash ponds also minimize dust problems and are simple to operate. Generally, crushing of bottom ash from the hopper is required for both dry and wet disposal methods to facilitate handling.

On a national scale, ash disposal costs ranged from $5 to $10 per ton, and the total cost of ash disposal to the electric utility industry in 1980 was estimated between $375 and $740 million [4]. This estimated cost did not include the cost for studying and possibly mitigating potential environmental effects of the ash disposal.

PRODUCTION AND UTILIZATION

The production of ash in the United States has steadily increased along with the increase in the coal-fired generating capacity. Based on the data from American Coal Ash Association (ACAA) [5], the annual ash production in the U. S. has increased from 39.2 million tons in 1970 to 66.8 million tons in 1986.

Of the 17.5 million tons of bottom ash produced in 1986, 13.4 and 4.1 million tons were dry bottom ash and wet bottom ash, respectively. Only 26.7 percent of the dry bottom ash was used, whereas 51 percent of the wet bottom ash was used [5]. The utilization trend of coal ash in 1980s, also reported by ACAA [5], shows that the use of fly ash has improved significantly, while the percentage of bottom ash uses has remained unchanged.

ENVIRONMENTAL EFFECTS OF ASH UTILIZATION

Considerable concern has been expressed about the environmental acceptability of coal ash as a construction material. In 1976, a national concern about solid waste disposal was expressed by Congress in the Resource Conservation and Recovery Act (RCRA) [6]. In amendments to the RCRA in May 1980, Congress specifically exempted conventional coal combustion waste (including fly ash, bottom ash, and flue gas desulfurization sludge) from regulation as a hazardous waste until further studies were conducted [7]. This temporary exemption holds until the U. S. Environmental Protection Agency (EPA) completes studies to establish actual environmental impacts of waste management of coal combustion wastes, and issues a final decision as to the hazardous or non-hazardous nature of these materials. The evidence of the study on Indiana bottom ashes provides information which not only reinforces the logic for the exemption of bottom ash, but also shows its potential for a number of productive uses. Before providing this information in this paper, the possible environmental effects associated with ash utilization are discussed.

An obvious environmental concern in either the disposal or utilization of coal ash is dust spreading, and the unaesthetic appearance of ash deposits. Dust can be mitigated by rapid covering of exposed ash surfaces, and by appropriate handling during construction, e.g., sprinkling. Because typical bottom ash has less than 10% fines, dust spreading is not expected to be a significant problem.

The principal environmental concern about the use of coal ash is the possible leaching of toxic substances or other potentially harmful constituents from the ash and the associated degradation of ground water quality. Coal ash, like the coals from which it is produced, does contain trace amounts of certain elements which, if released in sufficient concentration, may be harmful to the environment and subsequently to human health. These contaminants may be leached out by and carried along with percolating precipitation into ground water and surface waters.

The degree of environmental impact can be primarily determined by the amount of these elements leached from an ash deposit. Even small amounts of heavy metals released to the environment may constitute a hazard to animal and plant life. A high concentration of salts may adversely affect the quality of ground water, although it does not cause any danger to human health. These salts are mainly calcium and sulfate, but also chloride, sodium, potassium, and magnesium.

The complicated electrochemical characteristics of coal ash and its interactions with the surroundings may cause corrosion of any metal structure adjacent to or embedded in the ash deposit. Serious corrosion caused by a ash-water matrix may result in failure of the metal structure or the structure reinforced by metal. As far as environmental effects are concerned, the metal ions released from the corroded metal surface may cause a groundwater pollution problem if the concentration is high.

Environmental risks due to radioactivity are present only in deposits of certain peat ashes. Radioactivity can be limited by covering the deposit with a low radioactivity material. Prior to 1978, solid wastes were classified as radioactive if the radium-226 activity was above 5 picoCurie/g (pCi/g). Currently, no regulatory criteria exist for solid waste radioactivity. A study of 69 eastern and western fly ashes [8] showed that the mean values of radium-226 activity for ash produced from eastern coal and for that from western coal were 3.7 and 2.6 pCi/g, respectively. No data have been reported on the radioactivity of bottom ash.

When exposed to the atmosphere without appropriate protection, the particles of an ash deposit may be seriously eroded causing sediment pollution of nearby drainage systems. High turbidity in streams may affect the aqueous ecology, and degrade the quality of potential water supplies. To prevent erosion, soil covering and plant growth on ash fills and slopes are essential. Another benefit from vegetation is to efficiently reduce leachate production by increased evaporation and transpiration. Such removal by vegetation is estimated to involve more than half of the yearly precipitation, and thus limits leachate production significantly.

In summary, the environmental effects which need to be considered in either use or waste disposal of coal ash are:

1. the release of salts to groundwater and surface water;
2. the release of heavy metals to groundwater and surface water;
3. the ash corrosiveness to metal structures;
4. the radioactive emission from the deposit;
5. the erosion potential of the deposit;
6. the spreading of dust from the deposit or from construction; and and
7. the aesthetical influence on the surrounding landscape.

A schematic of the possible effects of ash deposits (except corrosion and erosion) on human health and environment is shown in Figure 1. Because effects 6 and 7 can be effectively solved by soil covering and vegetation, this study focuses on the first five environmental effects in the list.

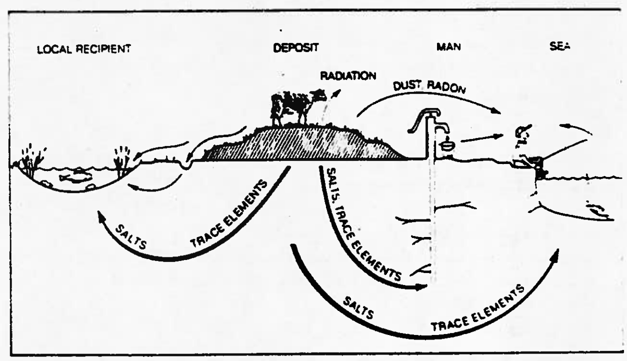

Figure 1. The possible effects of an ash deposit on human health

EXPERIMENTAL PROGRAM

SELECTION OF ASH SAMPLES

Four candidate bottom ashes were selected for study from 3 power stations in Indiana, with consideration to boiler type, geographic distribution, and ash disposal techniques. They are Perry ash, Gibson ash, and the ashes produced from Unit 14 and Unit 17 of the Schahfer station. Of the samples selected for testing, Schahfer 14 ash was the only wet bottom ash, the others being dry bottom ashes. During sampling, Perry K ash was collected directly from the silos since the dry disposal method was used at this station; while the other 3 ashes were wet-disposed and were collected as grab specimens from ash deposits at the end of sluice pipe. The approximate locations of the selected sources of bottom ash are starred in Figure 2.

Figure 2. Approximate locations of bottom ash sources in Indiana

The chemical composition and a variety of engineering properties of these selected ashes have been examined and reported by Huang [3]. The leachate properties were assessed by performing the two leaching tests described below. Ash corrosiveness was estimated based on measurements of four electrochemical characteristics of bottom ash. The radioactivity and the erodibility of bottom ash were estimated by radium-226 specific activity and soil erodibility factor, respectively.

HAZARDOUS NATURE

Bottom ash leachates generated from the Extraction Procedure (EP) toxicity tests [9] were analyzed for heavy metals to define their potentially hazardous nature. The EP toxicity test was designed by the EPA to simulate the leaching of a solid waste occurring in a sanitary landfill. In this test, a representative sample of a solid waste is extracted with deionized water maintained at a pH of 5 using acetic acid. The maximum contaminant levels (MCL) specified for characterizing hazardous solid wastes are such that they are one hundred times the National Primary Drinking Water Standards (NPDWS). Table 1 summarizes the analyzed results of bottom ash leachates. The concentrations of heavy metals for bottom ash extracts are far below the MCL specified by the EPA, in most cases one to three orders of magnitude lower. Therefore, bottom ashes are characterized by the EP toxicity test as nonhazardous. Moreover, it seems that bottom ash extracts would also satisfy the NPDWS.

EFFECTS ON GROUNDWATER QUALITY

The salt content of bottom ash leachate was tested by the leaching method test specified in the Indiana Administrative Code 329 IAC 2-9-3 [10]. The Indiana leaching method test is conducted as specified for the EP toxicity test, except with no addition of acetic acid. The concentrations of sixteen salts and pH were measured for each bottom ash leachate. Table 2 summaries the test results and the maximum concentrations specified for the most restricted waste site (type IV) in the code, along with the Secondary Drinking Water Standards. Again, the salt concentrations and pH of the bottom ash extracts meet all the requirements. As can be observed from Table 1 and 2, the wet bottom ash had the lowest concentrations of heavy metals and salts, due to its glassy and non-porous texture which retards the diffusion of the contained soluble substances into adjacent ground and surface water.

CORROSIVENESS

Bottom ash might be considered for use as backfills for reinforced earth or other metal structures. Corrosion of these metal structures caused by potential interactions between the ash mass and its surroundings may lead to structural damage and failure, as well as metal ion pollution of the groundwater. Like the leaching mechanism, such underground corrosion is a complicated process. A burial method with direct corrosion measurement of the buried metal(s) is considered to be the most reliable means to determine the corrosiveness of these underground media. Methods using electrochemical techniques for measuring corrosiveness hold promise for the future.

Based on literature review performed by Ke [11], the four electrochemical characteristics most related to corrosiveness of a medium are: minimum resistivity, pH, soluble chloride, and soluble sulfate, excluding the site factors. A medium with a lower minimum resistivity and pH, and higher contents of soluble chloride and sulfate is more corrosive. These parameters were used in predicting the corrosiveness of bottom ash samples, and were determined mainly following the California test methods [12,13,14]. In determining minimum resistivity and pH, the crushing method for preparing specimens suggested by Ke [11] was adopted, in order to obtain more representative measurements. To provide a larger data base for evaluation, some tests were conducted on additional 7 ashes sampled earlier by Huang [3]. The approximate locations of these ash sources are also shown in Figure 2 (unstarred

Table 1. Results of the EP toxicity test on candidate bottom ashes

Contaminant	Concentrations (mg/L)					
	Schahfer Unit 17	Gibson	Schahfer Unit 14	Perry	EPA allowable	Drinking water[a]
Mercury	0.0002	0.0001	<0.0001	0.0002	<0.2	<0.002
Silver	0.001	<0.001	<0.001	<0.001	<5.0	<0.05
Cadmium	0.0008	0.025	0.0007	0.0004	<1.0	<0.01
Chromium	0.0009	0.0005	0.0012	0.0009	<5.0	<0.05
Arsenic	0.020	0.010	0.005	0.008	<5.0	<0.05
Selenium	0.005	0.005	0.003	0.004	<1.0	<0.01
Barium	0.098	0.103	0.136	0.108	<100.0	<1.0
Lead	0.007	0.002	<0.001	0.005	<5.0	<0.05

[a] Primary Drinking Water Standard

Table 2. Results of the Indiana leaching method test on candidate bottom ashes

Contaminant	Concentrations (mg/L)				Indiana Maximum Allowable	Secondary drinking water standard
	Schahfer unit 17	Gibson	Schahfer unit 14	Perry		
Barium	0.098	0.103	0.136	0.108	1	1
Boron	0.21	0.19	0.02	0.47	2	-
Copper	<0.1	<0.1	<0.1	0.1	250	1
Chlorides	<1	<1	<1	1	250	250
Cyanide, total	<0.005	<0.005	<0.005	<0.005	0.2	-
Fluoride	<0.1	<0.1	<0.1	<0.1	1.4	1.4-2.4
Iron	0.1	0.4	0.1	0.1	1.5	0.3
Sodium	0.8	1.0	<0.5	1.5	250	-
Sulfate	31	55	19	26	250	250
Sulfide	<0.1	<0.1	<0.1	<0.1	1	-
Zinc	0.1	0.3	<0.1	<0.1	2.5	5
Total Dissolved Solids	90	140	10	145	500	500
Calcium	19	24	2	30	-	-
Magnesium	0.7	2.0	0.2	0.1	-	-
Potasium	1.0	0.7	0.1	2.0	-	-
	Standard units					
pH	8.9	8.4	7.8	7.7	6-9	6.5-8.5

locations). Table 3 summarizes the test results of the total 11 ashes examined. Based on the magnitudes of four parameters, the wet bottom ash is predicted to be less corrosive than the dry bottom ashes.

Based on the past corrosion experience with soils, the bottom ash may be expected to be non-corrosive [11] when the following values obtain: minimum 'minimum resistivity' of 1500 Ohm-cm; minimum pH of 5.5; maximum soluble chloride content of 200 ppm; and maximum soluble sulfate content of 1000 ppm. Of the 11 Indiana ash samples tested, 7 ashes (about 64%) were classified as corrosive. This high percentage provides sufficient warning that bottom ashes are potentially corrosive and need to be carefully investigated prior to use in the near vicinity of metal structures.

A noticeable difference in pH value was observed between Tables 2 and 3. This can be attributed to the different testing procedures and the different portions of ash particles used for testing. To avoid confusion in interpretation, a unified testing procedure needs to be developed.

Table 3. Corrosiveness of Indiana bottom ashes

Ash Source	ρ_{min}[a] (ohm-cm)	pH	Cl^- (ppm)	SO_4^{2-} (ppm)
Perry	980	4.8	15.5	589
Gibson	2201	7.6	7.3	1127
Schahfer 14[b]	>6663	9.6	0.4	50
Schahfer 17	3082	8.6	6.1	383
Gallegher	335	9.1	-[c]	-
Mitchell	1771	8.0	-	-
Wabash	1051	5.7	-	-
Richmond	247	8.2	-	-
Stout	4249	6.6	-	-
Culley	486	8.5	-	-
Brown	213	3.2	-	-

[a] minimum resistivity.

[b] wet bottom ash.

[c] not determined.

Table 4. Radioactivities of candidate bottom ashes

Ash Source	Specific Activity (pCi/g)
Perry	4.74
Gibson	3.96
Schahfer 14	4.03
Schahfer 17	3.02
Lafayette Soil[a]	1.02
Global average of soils[b]	0.8
Average of fly ash from western coal[c]	2.6
Average of fly ash from eastern coal[c]	3.7

[a] A clay soil obtained from Purdue University, West Lafayette, Indiana.

[b] From National Council for Radiation Protection and Measurements [15].

[c] From Elect. Power Research Inst. [8].

RADIOACTIVITY

Bottom ash samples were sealed in plastic canisters for 30 days to allow the short-lived radium daughters to reach equilibrium with radium. After the 30-day storage, detection of radium was accomplished by counting gamma rays emitted by its short-lived daughters in a secular equilibrium with radium. By calibrating the net count of a bottom ash sample with that of a standard material (NBS ID No. INJNS 8991) having an activity of 203 pCi and then dividing the sample weight, the specific activity for bottom ash was obtained.

The results of the radioactivity tests on candidate bottom ashes and a natural soil are tabulated in Table 4, along with the typical values for soils [15] and fly ashes [8]. As can be seen from this table, bottom ashes have slightly higher radium radioactivity than natural soils and fly ashes. However, all bottom ashes examined satisfy the 5.0 pCi/g criteria set before 1978.

ERODIBILITY

The most widely-used method to estimate the splash and rill erosion rate is the Universal Soil Loss Equation (USLE) [16], which involves a soil erodibility factor (K) and other external factors such as rainfall, slope length, slope steepness, covering, and support practice. If all external factors are set to be unity, the soil erodibility factor directly represents the erosion potential.

Wischmeier et al., 1972, [17] proposed a soil erodibility nomograph, as shown in Figure 3. The assigned erodibility factor ranges from 0 to 0.7 unit, a higher value

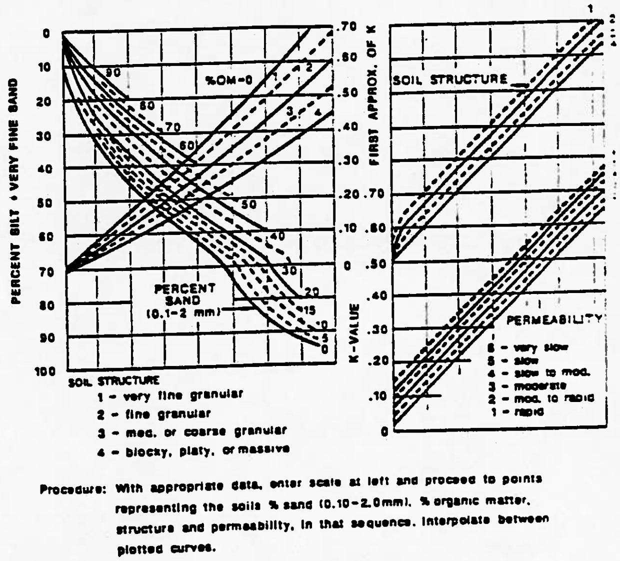

Figure 3. Soil erodibility nomograph (source: Wischmeier et al, 1972, [17])

implying more erodible. Following this nomograph, the obtained K values for candidate bottom ashes are presented in Table 5. All erodibility factors of bottom ashes studied are less than 0.1 unit. Compared with the typical values for benchmark soils [17], it is concluded that bottom ashes have low erosion potential.

SUMMARY AND CONCLUSIONS

In addition to evaluating the engineering suitability of bottom ash as a geotechnical material, the environmental acceptability of bottom ash for such a use

Table 5. Erodibilities of candidate bottom ashes

Ash Source	Perry Ash	Gibson Ash	Schahfer 14 Ash	Schahfer 17 Ash
% Sil+Very Fine Sand (0.002-0.1mm)	1	3	0	2
% Sand (0.1-0.2mm)	9	52	92	37
% Ignition Loss (Residual Carbon)	6.7	0.7	0.2	0.5
% Organic Matter[a]	0	0	0	0
First Approximation of Erodibility	0.005	0.01	0.0	0.005
Ash Structure	4	3	2	3
Permeability (in/hr)	20	7	143	48
Permeability Index	1	1	1	1
Erodibility Factor[b] (K)	0.075	0.10	0.02	0.045

[a] Set to be zero, on the conservative side (Residual carbon in bottom ash being different from organic matter for natural soils).

[b] For benchmark soils [17], K ranges between 0.1 and 0.5, in units of ton*acre*hour / (hundreds of acre*foot-tonf *inch).

should be considered. The environmental effects associated with ash utilization were discussed and evaluated. Based on the laboratory studies performed on selected Indiana bottom ashes, it can be concluded that untreated bottom ashes have a nonhazardous nature, minimal effects on the groundwater quality, low radioactivity, and low erodibility. However, they may be potentially corrosive.

With minimal adverse environmental effects and other favorable engineering properties documented in literature, the utilization of bottom ash will become more desirable in the future. From a technical standpoint, if the conclusions from the present work hold true for other power plants, bottom ash shows very good promise for use as embankment, subgrade, subbase, and base materials. However, those having high corrosion potential should not be placed in the neighborhood of any metal structure, to prevent either failure of such a structure or potential metal ion pollution.

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REFERENCES

[1] Huang, W. H., and Lovell, C. W., "Bottom Ash as Embankment Material," Geotechnics of Waste Fills - Theory and Practice, ASTM STP 1070, Arvid Landva and G. David Knowles, editors, American Society for Testing and Materials, Philadelphia, 1990, 14 pp.

[2] Seals, R. K., Moulton, L. K., and Ruth, B. E., "Bottom Ash: An Engineering Material," Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 98, No. SM4, 1972, pp. 311-325.

[3] Huang, W. H.,"The Use of Bottom Ash in Highway Embankments, Subgrades, and Subbases," JHRP-90-04, Purdue University, 1990, 269 pp.

[4] Valley Forge Laboratories, "Development of Federal Procurement Guidelines for Use of Power Plant Ash in Transportation Products," Devon, Pennsylvania, 1983.

[5] American Coal Ash Association, Inc., Ash Production and Utilization, Washington, D. C., 1986.

[6] U. S. Congress, Resource Conservation and Recovery Act, Public Law 94-580, Section 3001 (b) (3), Washington, D. C., October 21, 1976.

[7] U.S. Congress, Public Law 94-580, Amendments to the Resource Conservation and Recovery Act, Section 6921, Washington, D. C., 1980.

[8] Electric Power Research Institute, "Physical-Chemical Characteristics of Utility Solid Wastes," Report No. EPRI EA-3236, September, 1983.

[9] Environmental Protection Agency, "Test Methods for Evaluating Solid Waste, Physical/Chemical Methods," 2nd ed., Report SW-846, Washington, D. C., 1985.

[10] Indiana Register, Volume 12, No. 5, February 1, 1989.

[11] Ke, T. C., "The Physical Durability and Electrical Resistivity of Indiana Bottom Ash," JHRP-90-06, Purdue University, 1990, 335 pp.

[12] California Department of Transportation, "Method of Testing Soils and Waters for Sulfate Content," California Test 417, 1986, 2 pp.

[13] California Department of Transportation, "Method of Testing Soils and Waters for Chloride Content," California Test 422, 1978, 1 pp.

[14] California Department of Transportation, "Method for Estimating the Time to Corrosion of Reinforced Concrete Substructures", California Test 532, 1978, 7 pp.

[15] National Council for Radiation Protection and Measurements, "Environmental Radiation Measurement," NCRP No. 78, Bethesda, Maryland, 1976.

[16] Wischmeier, W. H. and Smith, D. D., "Predicting Rainfall Erosion Losses: A Guide to Conservation Planning," Agricultural Handbook No. 537, U. S. Department of Agriculture, Washington, D. C., 1976, 158 pp.

[17] Wischmeier, W. H., Johnson, C. B., and Cross, B. U., "A Soil Erodibility Nomograph for Farmland and Construction Sites," Journal of Soil Water Conservation, Vol. 26, 1972, pp. 189-193.

THE PERMEABILITY TEST IN ENVIRONMENTAL GEOTECHNOLOGY

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ABSTRACT

Increasing need for land disposal of wastes and the cleanup of existing landfills has involved many geotechnical engineers in environmental projects. The permeability test is now routinely performed to obtain the coefficient of permeability for use in subsurface flow analyses, waste facility siting and quality assurance of landfill construction. It is important that high quality, representative laboratory tests are performed. A thorough understanding of all the significant variables is required. This paper presents considerations for both saturated and unsaturated permeability testing of soils. The special requirements for environmental applications are discussed.

Laboratory permeability test procedures and equipment need to match the field conditions as closely as possible. Gradients should be representative of field conditions. Confining stresses and direction of flow should approximate conditions in the ground. Test permeants should be similar to those encountered in situ. Testing equipment and procedures should be modified for the requirements of hazardous permeants. The unsaturated permeability test should be performed if unsaturated flow will be significant.

INTRODUCTION

Environmental concerns pose a myriad of interesting problems for geotechnical engineers. Conventional geotechnical practice has been pressed to adapt to the requirements of environmental projects dictated by clients and regulators. Special thought must be given when testing soils for these purposes. Permeability tests are now routinely performed for waste disposal facility siting, remediation and subsurface modeling. This test must be carefully designed to match site conditions if the results are to be meaningful. Likewise, equipment and procedures should be carefully selected to reduce errors in the reported permeabilities.

Recent papers detail various permeameter designs and test procedures. Saturation techniques, sample size and driving gradients have been discussed. Recent evidence indicates the need for the unsaturated permeability function, as well as saturated permeability values, so that realistic subsurface flow modeling can be performed. These aspects are complicated when permeants other than water are used. Geotechnical engineers must be aware of the effects of selecting a specific testing program in order to obtain results that are compatible with the "environmental" application.

The goal of this paper is to outline the options for permeability testing of soils of environmental significance. Considerations of equipment type, sample condition, testing conditions, different permeants and presentation of results are discussed. This includes selecting the correct device to match the field application, effects of gradients and confining pressures, testing for unsaturated permeability, concerns about anisotropy and sensitivity of the results to test conditions.

GENERAL CONSIDERATIONS

Geotechnical engineers commonly apply Darcy's law to quantify the flow of water through a saturated soil:

$$q = -kiA \qquad (1)$$

where q is the volume of flow in a given time, i is the dimensionless hydraulic gradient (change in head per unit distance) and A is the gross cross sectional area of flow. The combined effect of a given permeant and soil is reflected by k. Geotechnical engineers refer to the test for k as the permeability test. In many fields k is called the hydraulic conductivity, however, civil engineers have traditionally termed k the coefficient of permeability. This paper will continue that nomenclature and use terms consistent with the glossary by Johnson (1). The coefficient will simply be termed permeability when the meaning is clear from the context. The purpose of this paper is to outline considerations for performing the tests to determine k.

SATURATED VS. UNSATURATED PERMEABILITY

Traditionally, geotechnical engineers have investigated problems of saturated flow through soils. Permeabilities of saturated soils are used in analyses of drainage, dewatering and flow under and through dams, among others. However, many environmental projects involve significant flows in unsaturated soils. A single value for the saturated permeability of a soil may be of limited use in geotechnical engineering for environmental concerns.

Many waste disposal sites are located in arid regions to minimize contamination of groundwater. Movement of soil moisture at these sites tends to occur in partially saturated soils. This flow includes infiltration and evapotranspiration at the surface, as well as all moisture movement above the phreatic surface of an actual or perched water table. Analyzing flow in these regions requires a permeability function rather than a single value of the coefficient of permeability. Testing to determine this function is sensitive to many of the same factors that effect saturated permeability tests.

Water pressures in the unsaturated soil are less than atmospheric pressure. These suctions are a function of the degree of saturation or volumetric water content of the soil. Permeability is also a function of the volumetric water content and therefore, a function of soil suction. Unsaturated permeabilities are lower than saturated permeabilities and decrease exponentially as the suction increases. Although studies are limited, Darcy's law appears valid for certain soils above 25% degree of saturation (2). Therefore, for unsaturated flow, k in Eqn. 1 becomes a function of the suction head or water content.

TESTING DEVICES

Conventional laboratory tests for hydraulic conductivity are generally conducted on cylindrical specimens of compacted or undisturbed soil. Three types of permeability cells are in wide use for saturated testing. Adaptations of these are also used for unsaturated testing.

The compaction mold provides a rigid wall, fixed volume cell. Permeant is introduced through the closed top and a porous base is used to collect outflow. A spring loaded platen can be incorporated in the top to provide confining stress and prevent swelling. Back pressuring is possible to aid in saturation. This type of cell is often used for compacted fine grained soils and also remolded coarse materials when their structure has been destroyed while the sample was taken. Compaction mold cells have the disadvantage of a fixed volume. Swelling samples are subjected to unpredictable stresses. Shrinking samples suffer loss of confining stresses, permitting shrinkage cracks to open as well as flow between the sample and the mold wall (3). An adaptation of the compaction cell by Anderson, et al (4) uses an annular ring to segregate the permeant that passes through the center from that which passes through the edges of the sample, thereby indicating when sidewall leakage is significant enough for further action.

A consolidation cell permeameter houses a cylindrical sample in a rigid wall yet permits volume changes that are stress controlled. Permeant flow is typically upward, and closed cell devices have been developed that permit back pressuring (5). One dimensional loading permits the sample to change volume at a constant stress while volume changes are measured. The vertical stress tends to reduce sidewall leakage by squeezing the sample to maintain wall contact. The possibility for sidewall leakage still exists, however. Also, the relatively thin specimens limit use for coarse soils and are less representative of large scale soil structure. An ASTM standard test method that would cover both fixed wall devices is currently under consideration.

The flexible wall permeameter resembles a triaxial shear cell and may be used in this form for triaxial consolidation, hydraulic conductivity and shear testing. This device is useful for untrimmed undisturbed specimens. Driving pressure, back pressure and confining pressure can all be varied to simulate realistic stress states and gradients. Volumetric changes can be monitored. Back pressures can be increased to ensure saturation. This device has the disadvantage of nonuniform effective confining stresses over the length of the sample. This can be significant if large hydraulic gradients are required (3). The flexible wall device is also more complicated and more expensive than the rigid walled devices. ASTM is currently considering a standard test method for this device.

All three types of permeameter cells can be adapted for testing unsaturated soils. The essential requirements for unsaturated testing are semipermeable boundaries at the inflow and outflow ends of the specimen which permit water to flow but preclude air from passing. These are typically porous ceramic plates and are rated based on the pressure required to pass air through them. Special piezometers, called tensiometers, can be built into the walls of the permeameter to determine the suction heads at various locations along the specimen. The tensiometers are also faced with porous ceramic. A sealed cell may be required if pore air pressure will be used to adjust the soil suction during the test.

BOUNDARY CONDITIONS

The permeability test takes one of two forms. The constant head test is typically performed on coarse grained soils with high permeabilities. The hydraulic gradient is constant and a steady state flow is established. The volume of permeant, Q, to pass through the sample in time t is used to calculate the coefficient of permeability from Darcy's law:

$$k = Q/iAt \qquad (2)$$

The steady state method to determine the unsaturated permeability function is a series of constant head tests performed at different degrees of saturation.

The falling head test is used for low permeability samples where only small quantities of flow occur. The hydraulic gradient decreases as the driving head is permitted to fall from h1 to h2 in a graduated standpipe of cross sectional area a. For a sample of length L, the coefficient of permeability is calculated as:

$$k = (aL/At) \operatorname{Ln}(h1/h2) \qquad (3)$$

Both tests can be run with elevated driving and back pressures to adjust the hydraulic gradient. At high pressures the falling head test of low permeability soil approaches the conditions of a constant head test since the change in fluid levels represents only a small portion of the total potential across the sample.

Determining the change of fluid level in a standpipe may require excessive testing times or very high hydraulic gradients for materials that have very low permeabilities. The standpipe can be replaced with a diaphragm transducer that has been calibrated for volume change vs. pressure reading (6). This could be the same transducer used to measure pore pressures when testing for complete saturation. Transducers can be selected to provide different ranges of pressures and volumes of displacement.

Darcy's law implies that the coefficient of permeability is independent of hydraulic gradient. However, studies have noted varying values of permeability at high gradients. Hubbert (7) has demonstrated that Darcy's equation should be valid whenever the inertial forces are small with respect to the viscous forces in the permeant. Restated, Darcy's law is valid when velocity head is negligible with respect to the elevation and pressure heads. Velocity head is negligible for fine grained soils at most naturally occurring gradients. However, hydraulic gradients may have to be limited to values less than one for coarse materials since flow velocities can become large, and the resulting turbulent flows invalidate the Darcy relation. Klute (8) suggests that the steady state unsaturated test be performed at a hydraulic gradient of unity.

Tests on fine grained soils at low gradients may also deviate from Darcy behavior. Swartzendruber (9) suggests several reasons for this, including flow induced structural changes and nonnegligible electro-osmotic potentials. Conventional wisdom suggests the use of the anticipated in situ gradients for the test, if possible.

Elevated back pressures can be used to control the hydraulic gradient. High pore pressures are also used to dissolve occluded air into solution, thereby producing saturation. If effective stresses are held constant then no structural changes are induced. Bishop and Henkel (10) calculated that a back pressure in excess of 100 psi is required to saturate a sample with an initial degree of saturation of 85%. The driving pressure must be even higher to induce flow. These pressures may be impractical for many testing laboratories. Additionally, gases dissolved in the permeant passing through the sample will come out of solution as the pressure drops. These occluded gas bubbles reduce the measured permeability. Therefore, deaired permeant should be isolated from any gas used to pressurize the system and to create driving heads.

The flexible wall permeameter cell has the disadvantage of creating nonuniform effective stresses in the sample. The inflow end of the sample is subjected to an effective stress equal to the confining stress minus the driving pressure. However, the outflow end has an additional effective stress equal to the total pressure difference across the sample. This higher effective stress at the outlet causes uneven consolidation to occur and reduces permeability. Low gradients reduce the effect of nonuniform effective stresses.

Wu, et al (11) reported horizontal permeabilities 15 times the vertical permeabilities in Toledo varved clays. Care should be taken to test the soil in the same direction that flow will occur in situ. Tests at different sample orientations should be considered for undisturbed samples with obvious varves, stratification or fissuring.

PERMEANTS

Environmental problems frequently expose soils to permeants other than water. Permeabilities of soils to other permeants may be several orders of magnitude different from that to water. Even the choice of water has a significant effect on the measured permeability for clay soils. Fireman (12) demonstrated a 25% decrease in permeability when tap water was used in place of natural pore water for a test on Hesperia sandy loam. He also found that the permeability with distilled water was 50% lower. Similar results were reported by Dunn and Mitchell (13). The results were attributed to an increase in cation attraction. Investigators have used deaired tap water (14) or a 0.005 N to 0.01 N calcium sulfate solution as a permeant to minimize the dispersion of clay particles (8,15,16). The two draft ASTM standards for permeability testing call for 0.005 N calcium sulfate.

Permeants should be deaired before use in the permeability test. Care must be taken to isolate the air/permeant interface for tests run at pressures above atmospheric pressure. Air will dissolve into the permeant and come out of solution when the pressure is reduced within the sample as it approaches the magnitude of the back pressure (8). Occluded gas bubbles cause desaturation and decrease the measured permeability. Dissolved air can come out of solution behind the porous plates of the unsaturated permeameter and cause loss of suction in the system. This area should be flushed with deaired permeant periodically (17).

Many authors have studied the temporal change of permeability of a soil subjected to contaminated permeants. Madsen and Mitchell (18) have compiled a comprehensive summary of investigations with organic chemicals. Permeabilities have been reported to increase, decrease or both depending on the combination of soil, permeant and permeameter. This has been attributed to soil/permeant interaction, chemical change in the permeant and shortcomings of the equipment. Ideally, the test permeant should be the same as the in situ permeant to minimize errors from this source. Many tests show rapid increases in permeability. Permeant supply reservoirs should be generously sized to avoid depletion of the system and desaturation of the specimen.

Inhibiting biological growth with phenol, thymol, propylene oxide or mercuric chloride has been suggested (8,19). This practice is inappropriate for most environmental work, since in situ permeabilities would also be affected by biological clogging(20).

Resistant materials should be used for all wetted parts of the permeability test apparatus when aggressive permeants are used. Teflon, Viton or other resistant polymers should be used for tubing and seals. Latex membranes used in the flexible wall test cell can be isolated from the permeant by first wrapping the specimen in Teflon tape (16,21,22). A mercury jacket can be used to reduce diffusion of air through the membrane during an unsaturated test (23). Stainless steel or resistant plating can be used for wetted parts of the permeameter to reduce corrosion and interaction with the permeant. Particular care should be taken to prevent organic solvents from contacting acrylic plastics and some types of elastomers to prevent crazing, softening or dissolution. This is particularly dangerous in pressurized systems. Some polymers suffer temporal changes in dimension when exposed to certain chemicals, including water. This is a concern when testing fine grained soils where small changes in volume measurements have a significant effect on observed permeability.

Temperatures should be maintained as constant as possible during the test to avoid changes in the density, volume and viscosity of the permeant. This will also prevent altering the dimensions of the volume measuring devices. Some authors have applied corrections to the measured permeabilities for the effects of temperature (20,23). While the effects of temperature on the viscosity and density are not typically large, the complex effects of temperature on reactions between the soil and permeant are still not fully understood. Therefore, the permeability test should be performed as near to temperatures in situ as is practical.

Hazardous chemicals should be handled with care to avoid subjecting personnel and equipment to harmful effects. Permeated samples should be considered as hazardous materials and separated from other samples leaving the lab for disposal. Standard chemistry laboratory practice for venting fumes should be followed. A closed permeameter with no free surface is suggested. Additionally, a vented chamber should be used with pressurized systems to contain sudden leaks (21). Aggressive permeants can soften or craze some permeameter materials and seals, leading to sudden system failure which invalidates the test and exposes personnel to hazardous materials. Manufacturers' pressure limits for permeability cells may be greatly reduced due to the effects of some permeants. The manufacturer should be contacted to provide information on chemical compatibility of their equipment with the anticipated permeant.

CONCLUSIONS

Special considerations are required when performing the permeability test for environmental uses. Test conditions should match the field conditions as nearly as possible to obtain meaningful results. This requires that test procedures duplicate the confining pressures, gradient, direction of flow, temperature and permeant of in situ flows. This may not be possible or practical to achieve. However, this paper and the cited references can provide an indication of the magnitude of errors created for each deviation from field conditions.

The choice of permeameter is dictated by the type of sample, permeant and boundary conditions needed. The compaction mold permeameter is principally for testing recompacted soil specimens. Large soil particles are acceptable due to the relatively large sample size. This device provides only low level stress conditions since confining pressures can not be controlled. Sidewall leakage can be significant if the sample shrinks when subjected to certain permeants. The annular ring device by Anderson, et al (4) reduces this error.

The consolidation cell permeameter may provide the most representative test conditions for undisturbed or recompacted fine grained soils. The uniaxial loading and permeation duplicate the surcharge loading of a landfill on a compacted liner. High vertical stresses can be imposed. The lack of compliance in the fixed sidewalls precludes the closing of cracks that would exist in situ while sealing sidewall leakage. Volume changes are measurable and a consolidation test can be run in conjunction with the permeability test. Thinner samples can be tested more rapidly and at lower gradients. Closed cells are available to permit back pressure saturation and have been adapted for unsaturated testing (8).

The flexible wall permeameter is useful for irregular or untrimmable samples. Virtually any stress state can be duplicated in this device and sidewall leakage is eliminated. However, macroscopic cracks may be closed and consolidation will occur to different degrees along the length of the specimen. Triaxial consolidation and shear tests can be performed after permeation is complete if a triaxial shear cell is used as the permeameter. Membranes in the flexible wall device must be protected from aggressive permeants to prevent leakage. Sidewall tensiometers are difficult to mount, but successful unsaturated tests have been performed without them (23).

While laboratory permeability tests can only approximate in situ conditions, a carefully designed permeameter and testing procedure can provide reasonable coefficients of permeability for use in analysis. Care in the selection of the test is essential.

REFERENCES

1. Johnson, A.I., "Glossary," Permeability and Groundwater Contaminant Transport, ASTM STP 746, T.F. Zimmie and C.O. Riggs, Eds., American Society for Testing and Materials, Philadelphia, 1981, pp. 3-17.

2. Hamilton, J.M., Daniel, D.E. and Olson, R.E., "Measurement of Hydraulic Conductivity of Partially Saturated Soils," Permeability and Groundwater Contaminant Transport, ASTM STP 746, T.F. Zimmie and C.O. Riggs, Eds., American Society for Testing and Materials, Philadelphia, 1981, pp. 182-196.

3. Daniel, D.E., Anderson, D.C. and Boynton, S.S., "Fixed-Wall Versus Flexible-Wall Permeameters," Hydraulic Barriers in Soil and Rock, ASTM STP 874, A.I. Johnson, R.K. Frobel, N.J. Cavalli and C.B. Pettersson, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp. 107-126.

4. Anderson, D.C., Crawley, W. and Zabcik, D., "Effects of Various Liquids on Clay Soil: Bentonite Slurry Mixtures," Hydraulic Barriers in Soil and Rock, ASTM STP 874, A.I. Johnson, R.K. Frobel, N.J. Cavalli and C.B. Pettersson, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp. 93-103.

5. Silva, A.J., Hetherman, J.R. and Calnan, D.I., "Low-Gradient Permeability Testing of Fine-Grained Marine Sediments," Permeability and Groundwater Contaminant Transport, ASTM STP 746, T.F. Zimmie and C.O. Riggs, Eds., American Society for Testing and Materials, Philadelphia, 1981, pp. 121-136.

6. Remy, J.P. "The Measurement of Small Permeabilities in the Laboratory," Geotechnique, V. 23, 1973, pp. 454-458.

7. Hubbert, M.K., "Darcy's Law and the Field Equations of Flow of Underground Fluids," Bulletin of the Association of International Hydrologic Science, V. 5, 1957, pp.24-59.

8. Klute, A. and Dirksen, C., "Hydraulic Conductivity and Diffusivity: Laboratory Methods," Methods of Soil Analysis, Part 1, A. Klute, Ed., Soil Science Society of America, Madison, WI, 1986, pp. 687-770.

9. Swartzendruber, D., "The Flow of Water in Unsaturated Soils," Flow Through Porous Media, R.J.M. de Wiest, Ed., Academic

Press, New York, 1969, pp. 215-292.

10. Bishop, A.W. and Henkel, D.J., The Measurement of Soil Properties in the Triaxial Test, Edward Arnold Ltd., London, 1962, 220 p.

11. Wu, T.H., Chang, N.Y. and Ali, E.M., "Consolidation and Strength Properties of a Clay," Journal of the Geotechnical Engineering Division, ASCE, V. 104, N. GT7, 1978, pp. 889-905.

12. Fireman, M., Soil Science, V. 58, 1944, pp. 337-355.

13. Dunn, R. and Mitchell, J.K., "Fluid Conductivity Testing of Fine Grained Soils," Journal of Geotechnical Engineering, ASCE, V. 110, N. GT11, 1984, pp. 1648-1665.

14. Lentz, R.W., Horst, W.D. and Uppot, J.O., "The Permeability of Clay to Acidic and Caustic Permeants," Hydraulic Barriers in Soil and Rock, ASTM STP 874, A.I. Johnson, R.K. Frobel, N.J. Cavalli and C.B. Pettersson, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp. 127-139.

15. Brown, K.W., Thomas, J.C. and Green, J.W., "Permeability of Compacted Soils to Solvents Mixtures and Petroleum Products," Proceedings of the Tenth Annual Research Symposium on Land Disposal of Hazardous Waste, Report EPA-600/9-84-007, U.S. Environmental Protection Agency, Cincinnati, OH, 1984, pp. 124-137.

16. Foreman, D.E. and Daniel, D.E., "Effects of Hydraulic Gradient and Method of Testing on the Hydraulic Conductivity of Compacted Clay to Water, Methanol and Heptane," Proceedings of the Tenth Annual Research Symposium on Land Disposal of Hazardous Waste, Report EPA-600/9-84-007, U.S. Environmental Protection Agency, Cincinnati, OH, 1984, pp. 138-144.

17. Fredlund, D.G., "A Diffused Air Volume Indicator for Unsaturated Soils," Canadian Geotechnical Journal, V. 12, 1975, pp. 533-539.

18. Madsen, F.T. and Mitchell, J.K., "Chemical Effects on Clay Hydraulic Conductivity and their Determination," Open File Report, Environmental Institute for Waste Management Studies, University of Alabama, Tuscaloosa, 1987.

19. Poulovassilis, A., "The Changeability of the Hydraulic Conductivity of Saturated Soil Samples," Soil Science, V. 113, 1972, pp. 81-87.

20. Olson, R.E. and Daniel, D.E., "Measurement of the Hydraulic Conductivity of Fine-Grained Soils," Permeability and Groundwater Contaminant Transport, ASTM STP 746, T.F. Zimmie and C.O. Riggs, Eds., American Society for Testing and Materials, Philadelphia, 1981, pp. 18-64.

21. Bodocsi, A., Minkarah, I. and Randolph, B.W., "Reactivity of Various Grouts to Hazardous Wastes and Leachates," Proceedings of the Tenth Annual Research Symposium on Land Disposal of Hazardous Waste, Report EPA-600/9-84-007, U.S. Environmental Protection Agency, Cincinnati, OH, 1984, pp. 43-51.

22. Randolph, B.W., "A Test for the Effects of Hazardous Waste Leachates on the Permeability of Chemically Grouted Soils," M.S. thesis, University of Cincinnati, Cincinnati, OH, 1983.

23. Huang, C.S., The Permeability and Volume Change Behavior of an Unsaturated Clay-Shale, M.S. thesis, The Ohio State University, Columbus, Ohio, 1986.

STATE OF STRESS AND HYDRAULIC FRACTURING POTENTIAL IN SOIL-BENTONITE CUTOFF WALLS

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ABSTRACT

Hydraulic cutoff walls, constructed of low permeability mixes of natural soil, bentonite clay and water, have been used as seepage barriers since the early 1940's. Since then their use has expanded to include applications as hydraulic barriers to control movement of contaminated groundwater.

Overburden stress in a soil-bentonite wall, generated by the weight of the materials making up the wall, can be carried down through the wall to the underlying strata or be transmitted to the adjacent soils through wall friction. Due to the high depth to width ratio of a typical wall, the magnitude of vertical stress reduction due to side wall friction can be considerable.

The magnitude of stress at any level in a cutoff wall is of interest as it affects various wall properties including:

a. the permeability of the wall, since the effective overburden stress controls the consolidation of the wall which, in turn, determines its permeability;
b. the resistance to hydraulic fracturing caused by fluctuating hydraulic conditions in the adjacent soils;
c. the ability of the wall to close or "heal" slurry filled voids created during construction;
d. the ability of the wall to "heal" shrinkage cracks caused by aggressive chemical leachates.

A study of the state of stress distribution in soil-bentonite cutoff walls was conducted by the authors in a U.S. EPA funded program at the Center Hill Facility of the University of Cincinnati. Several tests were conducted on two different laboratory models. The results revealed that the consolidation of a wall under its own weight will mobilize certain values of side friction between the wall and adjacent soil. Furthermore, a mathematical model of the state of stress in soil-bentonite cutoff walls was derived and compared with the test results. This model was made applicable for modeling conditions in real-site cutoff walls.

This paper presents the results of the laboratory testing and the detailed development of a mathematical model. It also shows how the model can be used to assist a designer in choosing appropriate wall materials and geometric configuration to optimize the properties listed above. Furthermore, the model can help the geotechnical engineer in selecting representative confining stresses when testing for the design permeability values of cutoff wall materials.

INTRODUCTION

Soil-bentonite cutoff walls are constructed of low permeability mixes of natural soil, bentonite clay and water. Bentonite is primarily composed of the clay mineral montmorillonite which has a very high shrink-swell capacity and, when used in cutoff wall mixes, it will lower the hydraulic conductivity of the mixes and increase their plasticity. Soil-bentonite cutoff walls can be utilized as hydraulic barriers in many applications including preventing the migration of contaminated groundwater.

The backfill material typically used in hydraulic cutoffs is a soil-bentonite (s/b) mixture usually containing the material excavated from the trench combined with 1% to 4% bentonite. This mixture is designed to maximize the use of the trench spoils and other local material and provide the required hydraulic cutoff. Therefore, soil-bentonite mixes will vary from location to location. The result of soil-bentonite cutoff wall construction is a relatively economical wall of low permeability.

The performance of a soil-bentonite cutoff wall is controlled by several factors. One factor is the level of effective vertical stresses present throughout the wall. The greater the level of stress on an element in the wall, the greater that element will be consolidated. Consolidation reduces the pore space or porosity of the soil-bentonite and lowers its permeability. The effective vertical stress on an element of a cutoff wall is the effective weight of the overburden minus the sum of side friction above the element. Therefore, the magnitude of stress in the wall changes with depth.

The behavior of a soil-bentonite cutoff wall is somewhat analogous to a pile. The load on a pile can be carried by tip bearing, side friction or a combination of both. The same is true of the stress in a soil-bentonite cutoff wall. The only difference is that soil-bentonite consolidates causing vertical displacements until consolidation is complete. These displacements mobilize the shear strength at the soil-bentonite and insitu soil interface. This appears to give wall friction priority over end bearing with respect to the transmission of stress. Also, due to the high depth to width ratio of a typical wall, the total amount of wall friction can be high.

Evaluating the amount of vertical stress that is lost due to wall friction in soil-bentonite cutoff walls provides valuable information regarding cutoff wall behavior. If stress losses due to wall friction are significant, the walls ability to: consolidate to achieve required permeability; resist hydraulic fracturing due to fluctuating hydraulic conditions in adjacent soils; close or "heal" voids during construction; and close potential microfractures formed due to aggressive leachates is significantly reduced. Understanding this behavior may improve slurry wall performance through the development of design criteria to minimize wall friction and transmit the majority of the overburden stresses down through the wall.

The main purpose of this research was to investigate the state of stress in soil-bentonite cutoff walls in an attempt to provide criteria with which future walls can be evaluated. Other important goals were to determine the sensitivity of wall friction to variations in shear strength and overburden. Two approaches were taken in studying the frictional losses in cutoff walls. They included:

- Developing a mathematical model which, together with basic shear strength parameters, is used to evaluate the level of stress at any point in a cutoff wall.
- Constructing laboratory scale wall models in which the stresses could be measured.

MATHEMATICAL MODEL

The following discussion develops a mathematical model for evaluating the state of stress on an element of a soil-bentonite cutoff wall. Figure 1 is a free body diagram showing the forces on a wall element.

<u>State of Stress in Cutoff Walls:</u>

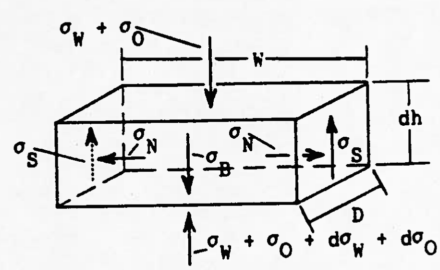

Figure 1. Forces on an Element of Soil-Bentonite Cutoff Wall

Nomenclature of terms used in Figure 1:

- w = Width of cutoff wall
- D = Length of wall element
- dh = Differential height of the wall
- σ_B = Stress from weight of soil element
- σ_N = Normal stress at edge of cutoff (lateral earth pressure)
- σ_S = Shear stress along wall-insitu soil
- σ_W = Stress on element of wall from wall weight above

$d\sigma_W$ = Change in stress over a change in depth (dh) from weight of element minus wall friction losses generated by this weight

σ_O = Stress at top of element of soil-bentonite wall due to applied overburden load

σ_{O_t} = Stress at the top of a soil-bentonite wall due to an applied surcharge load

$d\sigma_O$ = Change in stress over change in depth (dh) from wall friction losses generated by overburden load

σ_V = $\sigma_W + \sigma_O$ (total stress from both the weight of soil-bentonite and applied surcharge)

<u>From free body diagram :</u>

Σ Vertical forces = 0

Upward forces are assumed potitive.

$$-(\sigma_W + \sigma_O)D\,w - \sigma_B D\,w + 2\sigma_S\,D\,dh + (\sigma_W + \sigma_O + d\sigma_W + d\sigma_O)D\,w = 0$$

<u>Where:</u>

$\sigma_B = \gamma\ dh$ (Eq. 1)

γ = Total unit weight of s/b backfill

$\sigma_S = c + (\sigma_W + \sigma_O)K\ \mu$ (Eq. 2)

c = Cohesive strength of wall-insitu soil

K = Earth pressure coefficient ((1 - sin Φ) for at rest condition)

μ = Tan Φ

Φ = Angle of internal friction between wall and insitu soil

<u>Simplifying:</u>

$$-\sigma_B\ D\ w + 2\sigma_S\ D\ dh + (d\sigma_W + d\sigma_O)D\ w = 0$$

$$\frac{d\sigma_W + d\sigma_O}{dh} + \left(\frac{2}{w}\sigma_S\right) = \frac{\sigma_B}{dh}$$

<u>Substituting Eqs. 1 & 2 :</u>

$$\frac{d\sigma_W + d\sigma_O}{dh} + \frac{2}{w}\ (c + (\sigma_W + \sigma_O)K\ \mu) = \gamma$$

$$\frac{d\sigma_W}{dh} + \frac{d\sigma_O}{dh} + \left(\frac{2K\mu}{w}\sigma_W\right) + \left(\frac{2K\mu}{w}\sigma_O\right) = \gamma - \frac{2c}{w}$$

$$k_1 = \frac{2K\mu}{w} \quad \text{and} \quad k_2 = \gamma - \frac{2c}{w}$$

$$\frac{d\sigma_W}{dh} + \frac{d\sigma_O}{dh} + (k_1\ \sigma_W) + (k_1\ \sigma_O) = k_2$$

<u>Solve for σ_W and σ_O separately :</u>

$$\frac{d\sigma_W}{dh} + (k_1\ \sigma_W) = k_2 \quad \text{and} \quad \frac{d\sigma_O}{dh} + (k_1\ \sigma_O) = 0$$

<u>Solve homogeneous part for σ_W :</u>

$$\frac{d\sigma_{W_h}}{dh} = -k_1\ \sigma_{W_h} \quad \text{Rearranging} \rightarrow \frac{d\sigma_{W_h}}{\sigma} = -k_1\ dh$$

<u>Integrate :</u>

$$\ln \sigma_{W_h} = -k_1\ h + C \quad \text{Rearranging} \rightarrow \sigma_W = C_1\ e^{(-k_1 h)}$$

<u>Obtaining a particular solution satisfying boundary conditions $\sigma_W = 0$ at $h = 0$:</u>

$$\frac{d\sigma_W}{dh} + k_1\ \sigma_W = k_2 \quad \text{yields} \quad \sigma_W = \frac{k_2}{k_1}(1 - e^{-k_1 h})$$

$$\boxed{\sigma_W = \frac{\gamma\ w - 2c}{2K\mu}\left(1 - e^{\frac{-2K\mu h}{w}}\right) \quad \text{(Eq. 3)}}$$

<u>Solve for overburden loads (σ_O) :</u>

$$\frac{d\sigma_O}{dh} + k_1\ \sigma_O = 0 \quad \text{Rearranging} \rightarrow \frac{d\sigma_O}{\sigma_O} = -k_1\ dh$$

<u>Integrating :</u>

$$\ln \sigma_O = -k_1 h + C \quad \rightarrow \quad \sigma_O = e^{-k_1 h + C}$$

<u>Boundary condition : when h = 0, $\sigma_V = \sigma_{O_t}$:</u>

$$\boxed{\sigma_O = \sigma_{O_t}\left(e^{\frac{-2K\mu h}{w}}\right) \quad \text{(Eq. 4)}}$$

<u>Adding equations #3 and #4 to acquire complete solution of the state of stress in s/b cutoff walls:</u>

$$\sigma_V = \sigma_W + \sigma_O$$

$$\boxed{\sigma_V = \frac{\gamma\ w - 2c}{2K\mu}\left(1 - e^{\frac{-2K\mu h}{w}}\right) + \sigma_{O_t}\left(e^{\frac{-2K\mu h}{w}}\right)}$$

The vertical stress at any depth in a cutoff wall, without an applied additional surcharge load, is zero if: $\gamma w-2c=0$. This is because the cohesive strength between the wall and adjacent soil is capable of transferring all of the overburden stress into the adjacent soil. This prevents any of the overburden from being transmitted down through the wall and indicates that if a void or fracture was present anywhere in the wall, the soil weight could not act to close it. This lack of vertical stress transmitted through a cutoff wall makes the wall susceptible to leakage through shrinkage cracks and other voids, as the wall has no ability to close or "heal" its defects.

The long term effect of cohesional friction losses were not studied in this investigation. Spangler (1973) indicates in his article on "Loads on Underground Conduits" that cohesion acts to reduce the load on buried conduit initially but may diminish through time. This could potentially be true in soil-bentonite cutoff walls. This reduction in cohesion would act to enhance cutoff wall performance.

The frictional component of shear strength is also important but is a function of depth and cannot be simplified as easily as the cohesive strength portion of the equation. Further evaluation of this equation utilizing parameters obtained in the pilot scale study is presented below and in Figure Nos. 2 through 4.

Figure 2 indicates the state of total stress at various depths in cutoff walls of typical widths using the mathematical model presented previously and average shear strength values from Figure 5. Figure Nos. 3 and 4 are tables indicating the state of stress with variations in cohesion "C" and Fiction Angle. The shear strength values in Figure 5 were obtained from several triaxial and direct shear tests performed on the soil-bentonite mix utilized in the laboratory study. This mix contained in dry weight percentages approximately: 3.8% bentonite; 5.0% clay; 4.1% silt; and 87.1% sand. The mix had a moisture content of approximately 23.5%. This mix was originally designed to provide a permeability of approximately 1X10-7 cm/sec.

The cohesive strength of the soil-bentonite mix used in this study was found to be between 0.5 and 1.0 psi and this material had a wet unit weight of 118 lbs./cu. ft. This indicates the minimum width of a soil-bentonite cutoff wall must be greater than approximately 15 inches and 30 inches, depending on "c", before any stress is passed down through the wall. This is because the cohesion between the wall and adjacent soil will transfer all of the stress into the adjacent soil. This assumes the shear strength of the adjacent soil is greater with respect to cohesion and friction than the soil-bentonite. This is likely true due to the wet consistency and low shear strength of the mix.

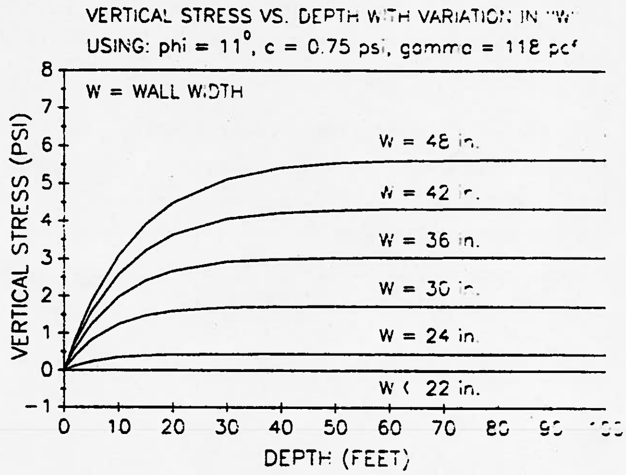

Figure 2: Vertical Stress Versus Depth for a Cutoff Wall Varying Wall Width

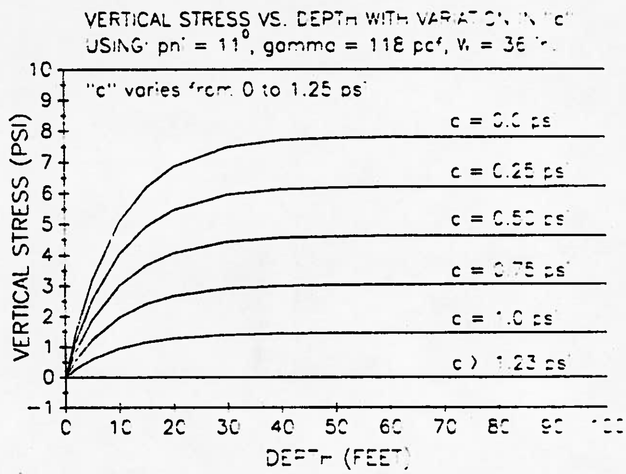

Figure 3: Vertical Stress Versus Depth for a Cutoff Wall Varying Cohesion "C"

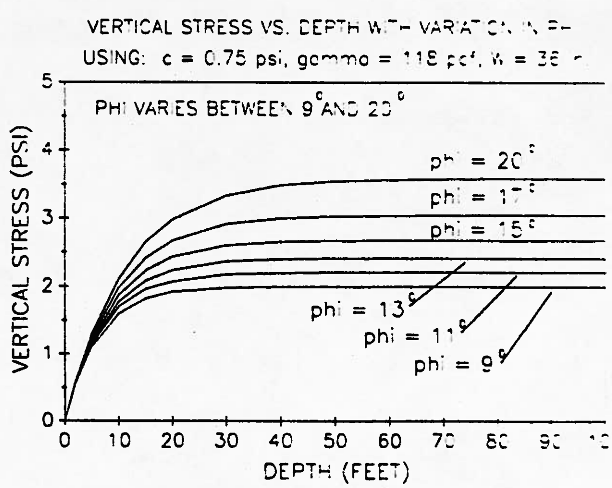

Figure 4: Vertical Stress Versus Depth for a Cutoff Wall Varying Friction Angle "O"

INSERT "STATE OF STRESS..." P. 4

by SPENCE

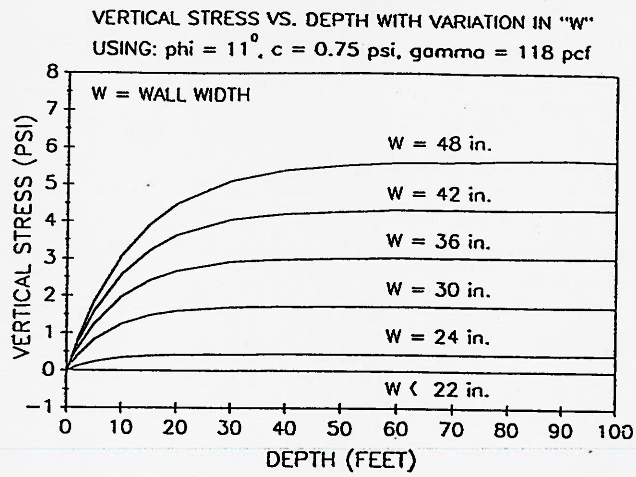

Figure 2: Vertical Stress Versus Depth for a Cutoff Wall Varying Wall Width

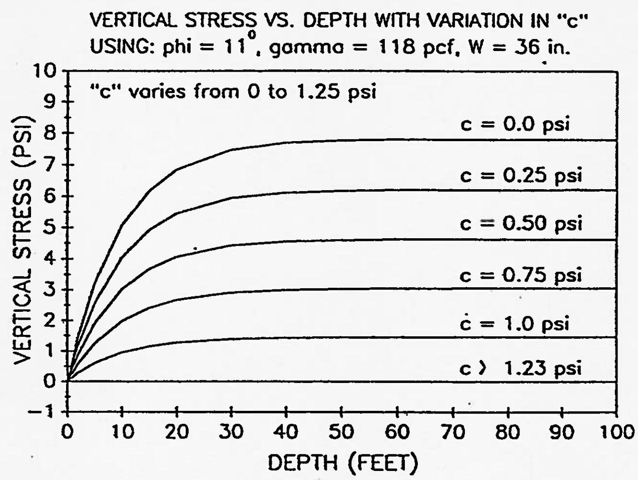

Figure 3: Vertical Stress Versus Depth for a Cutoff Wall Varying Cohesion "C"

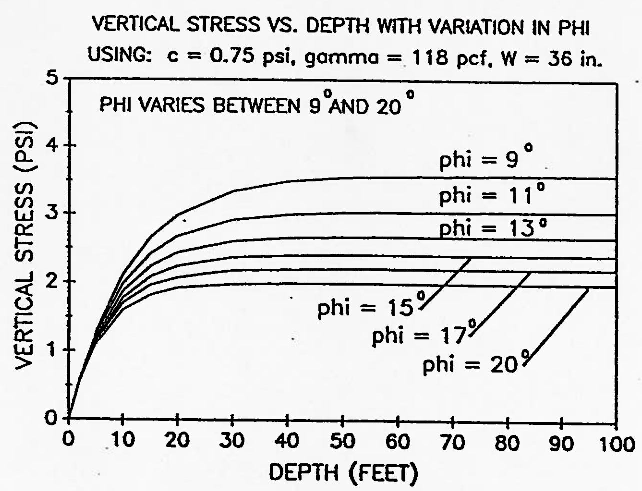

Figure 4: Vertical Stress Versus Depth for a Cutoff Wall Varying Friction Angle "ϕ"

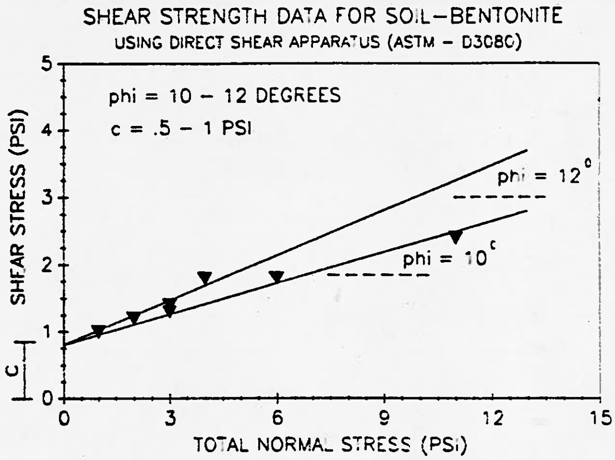

Figure 5: Shear Strength Data For Soil Bentonite Material

The results of this analysis indicates the vertical stress in a wall increases with depth until it reaches a maximum value. This maximum value is a function of the unit weight of the soil-bentonite, the shear strength of the soil-bentonite and insitu soil, and the width of the wall. Increasing the wall width will cause a sizable increase in the magnitude of vertical stress in the wall.

Example 1

The s/b material characterized in Figure 5 is to be used in the wall shown. Using Figure 2 , find the minimum width of the wall (w) to avoid hydrofracture.

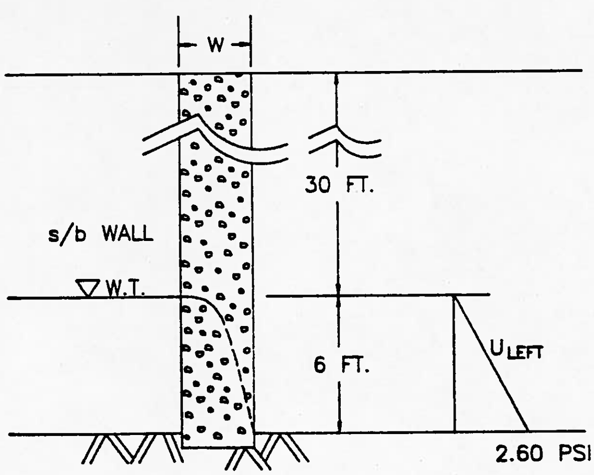

FIGURE 6 (NO SCALE)

Solution: Use at least a 36 in. wide wall, as the total stress at the bottom of a 36 in. wide wall will be at least 2.9 PSI which is greater than the pore pressure of 2.6 PSI.

Example 2

If designing a s/b mix for the wall shown in Example 1, what confining stress should be utilized in permeability tests using a 36 in. wide wall?

Solution: Use confining pressure equal to stress at top of highest anticipated water level. Therefore, the stress at 30 ft. equals 2.9 PSI so 2.9 PSI should be utilized to design for minimum stress situation.

Example 3

The s/b wall shown in Example 1 is to be designed with a S/B backfill whose characteristics are given in Figure 5, and a permeability vs. effective vertical stress relationship of:

$$k = 10^{-7} - (\sigma_V' / 20)(10^{-7} - 10^{-9}) \text{ cm/sec}$$

What permeability would you use in the design?

Solution:

σ_V' = 2.9 psi - ((6 ft. x 62.4 pcf)/144)
= 0.3 pcf; therefore, k = 10^{-7} cm/sec

LABORATORY STUDY

The evaluation of the wall friction in soil-bentonite cutoff walls was performed in several ways. This included using a cutoff wall model containing a bottom total pressure membrane and an apparatus called a friction column.

Data was collected from a liquid filled membrane placed under a soil-bentonite cutoff wall model constructed in a circular tank. A schematic of this tank is presented below in Figure 7. Data was obtained utilizing a pressure transducer connected to a liquid filled membrane located at the bottom of the model wall. This membrane was designed to evaluate the level of overburden stress at the tank for use in a permeability study. These initial results initiated the study outlined in the paper. The data obtained was analyzed by using the mathematical model presented earlier in this paper. Table 1 shows the results of the tests under different overburden loads.

The results indicate substantial friction losses existed in the cutoff wall model. These losses were less than the magnitude indicated by the equation. This was likely due to several reasons including: the model was constructed using plastic forms between the wall and adjacent soil which were removed after all soil-bentonite and adjacent soil had been placed; minimum vertical deflections of the wall reducing mobilization of friction as the wall was placed with a low slump to prevent excessive consolidation which could damage the model; the model was deflected upward versus the adjacent soils

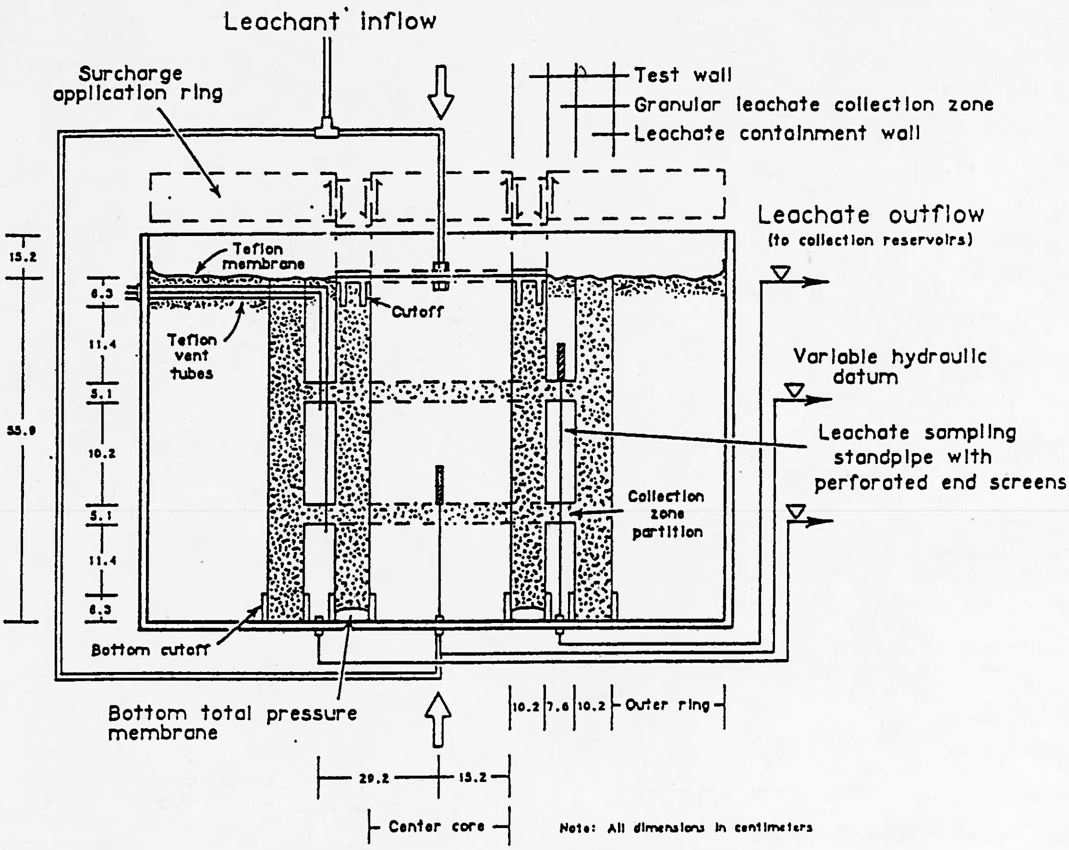

Figure 7: Schematic of Model Cutoff Wall

Total Applied Stress (Wall Weight+ Surcharge) (PSI)	Average Bottom Pressure Reading (PSI)	Theoretical Bottom Pressure (Math. Model c=0) (PSI)	Total Frictional Loss ** (LBS)	Average Wall Friction Loss (PSI)
2.5	1.10	0.81	281.4	0.13
3.5	3.50	0.98	0.0	0.00 *
3.5	2.20	0.98	221.7	0.10
8.5	4.50	1.86	804.2	0.36
12.5	5.30	2.58	1447.6	0.65
17.5	6.75	3.46	2161.4	0.98

** See Figure 7 for dimensions of the Model Cutoff Wall.

* Wall Friction removed by increasing the volume of the bottom membrane causing vertical displacement of the wall relative to the soil adjactent to the wall.

Table 1: Wall Friction Data from Model Cutoff Wall

early in the test as noted in Table 1 to see if friction could be removed and this may have prevented remobilization of friction. Therefore, a more simplified and representative model was devised. This model was also designed so the height of the model could be varied.

This model was called a friction column and a schematic of this model with both zero displacement and vertical displacement membranes are shown in Figures 8 and 9. The friction column was made of a six inch diameter PVC tube with two flat pieces of PVC cut to fit on the top and bottom. They were connected with threaded rods. The bottom contained a three inch diameter membrane filled with water. This membrane was connected with plastic tubing to a pressure transducer.

A three inch diameter column of soil-bentonite was constructed inside the PVC tube using a metal slip form. A medium sand was placed around the

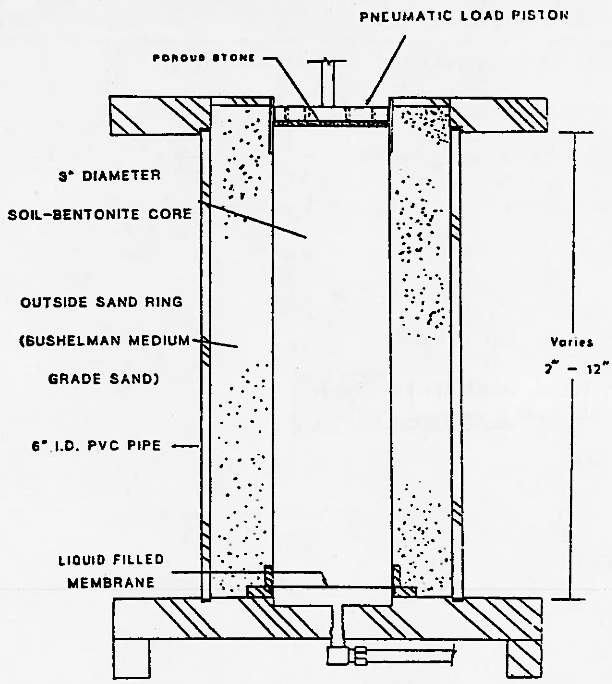

Figure 8: Schematic of Friction Column with Zero Displacement Membrane

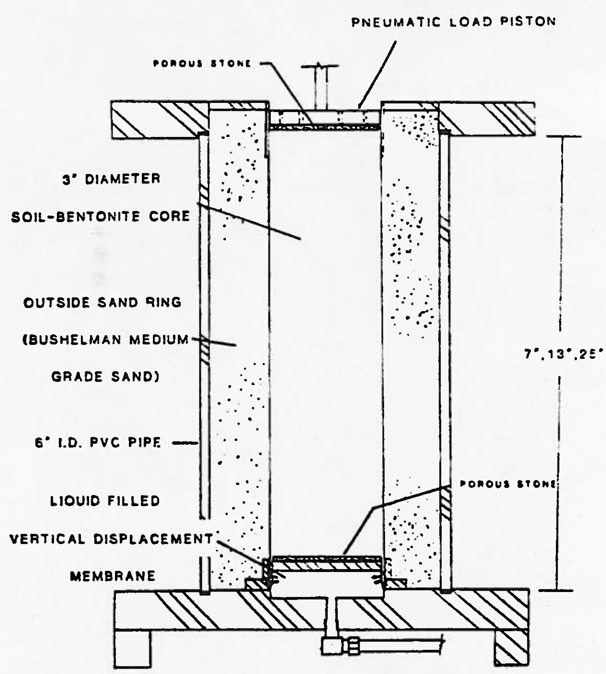

Figure 9: Schematic of Friction Column with Vertical Displacement Membrane

soil-bentonite column to provide lateral support and act as the in-situ soil. Test were run by first stirring the soil-bentonite with a rod which transforms the soil-bentonite into a heavy liquid, somewhat analogous to soil-bentonite flowing in a trench. Consequently, all its weight was transmitted to the bottom membrane. Shortly after the stirring action was stopped, the bottom membrane pressure reading began to drop. This was due to an increase in shear strength from the reduction in excess pore pressures, thixotropic strength gain, and consolidation which mobilized the shear strength.

In an alternate test, liquid was pushed upward into the bottom membrane which displaced the core of soil-bentonite upward relative to the sand. This removed the wall friction that was previously mobilized. This was registered as an increase in the bottom stress reading. The core of soil-bentonite was further displaced causing a reversal in the direction of shear between the soil-bentonite and the adjacent sand. This is noted as bottom stress readings higher than the stress applied by the weight of the soil-bentonite. Due to the fact the model was sheared faster than pore pressures could dissipate, the results indicate the effective adhesion mobilized.

When the equation for the state of stress in cutoff walls is rederived for a cylindrical wall (analogous to the friction column) the solution is identical with the radius of the column replacing the width of the wall.

Presented in Table 2 is a summary of the friction column tests using a zero displacement membrane. The tests were duplicated several times to assure their accuracy. The results were analyzed using the equation previously derived.

The tests were run prior to the derivation of the state of stress equation. Because of this it was not known at that time that the diameter of the friction column was not large enough to obtain positive data. Therefore, all of the tests run with the zero displacement membrane (fixed membrane at the bottom of the column) rendered a bottom stress of approximately zero. The results do demonstrate that the wall friction capacity is mobilized before the stress is transmitted to the bottom of the wall as was assumed in the derivation of the state of stress equation.

Due to the zero bottom stress reading, additional tests were run with a vertical displacement membrane. This membrane allowed the column of soil-bentonite to be displaced upward relative to the adjacent sand. The wall friction was therefore added to the weight of the soil-bentonite. The tests were run using a positive displacements pump that filled the membrane at a rate of 0.045 in./min. vertical displacement. Pressures were read by a pressure transducer connected to the bottom membrane.

The results of these tests reveal the potential for levels of wall friction higher than the stress generated by the weight of the soil-bentonite. Table 3 summarizes these results which does provide additional verification that wall friction can be significant in magnitude and is mobilized by differential movement

Friction Column Height (IN.)	Calculated Stress from S/B Weight (PSI)	Surcharge Stress (PSI)	Bottom Stress (PSI)	Average Wall Friction (PSI)
7.0	0.48	0.0	1.80	0.14
7.0	0.48	2.0	6.00	0.38
13.0	0.89	0.0	4.50	0.21
13.0	0.89	2.0	7.50	0.27
25.0	1.71	0.0	23.00	0.64

Table 3: Friction Column Data with Vertical Displacement Membrane

Friction Column Height (IN.)	Stress from S/B Weight (PSI)	Overburden Stress (PSI)	Bottom Stress (Experimental) (PSI)	Bottom Stress (Calculated) (PSI)	Actual Cohesion phi = 11 (PSI)
2.0	0.15	0.0	0.02	0.00	0.046
4.0	0.29	0.0	0.02	0.00	0.046
6.0	0.41	0.0	0.07	0.00	0.036
8.0	0.56	0.0	0.02	0.00	0.047
11.0	0.74	0.0	0.03	0.00	0.048
12.0	0.78	0.0	0.00	0.00	0.051
12.0	0.78	5.0	0.00	0.00	0.120
12.0	0.78	10.0	0.00	0.00	0.190
12.0	0.78	20.0	0.00	0.00	0.330

Table 2: Friction Column Data with Zero Displacement Membrane

between the soil-bentonite and adjacent material.

CONCLUSIONS

It can be noted from the mathematical model and laboratory tests that wall friction plays a significant role in the behavior of soil-bentonite cutoff walls. The mathematical model reveals the minimum level of stress or maximum potential frictional losses in an element of a soil-bentonite cutoff wall. The laboratory tests revealed significant losses associated with wall friction. The results of the laboratory tests did not however establish whether the full magnitude of wall friction is mobilized in soil-bentonite cutoff walls. Further testing utilizing larger models and potentially real-site walls is warranted to further evaluate this phenomena.

This study has produced a mathematical model which can be utilized to evaluate the minimum level of stress in a soil-bentonite cutoff wall. This model is useful in the design of a wall as it provides state of stress criteria with which the wall materials can be tested and evaluated for design. The model also can be utilized in the evaluation of wall width to aid in the design of a stable wall which will: resist hydraulic fracturing from fluctuating hydraulic conditions; be able to close voids created during construction; resist shrinkage cracks associated with aggressive chemical leachates; and consolidate to a level suitable to achieve acceptable level of permeability. Therefore, soil-bentonite cutoff wall performance can be enhanced through improved and more realistic design evaluation.

REFERENCES

Anderson, D., Brown, K., and Green, J., "Effect of Organic Fluids on the Permeability of Clay Soil Liners," Land Disposal of Hazardous Waste, Proceedings of the Eighth Annual Research Symposium; (EPA-600/9-82-002), January, 1982.

McCandless, R.M. and A. Bodocsi, Investigation of Slurry Cutoff Wall Design and Construction Methods for Containing Hazardous Wastes; Final Report, contract no. 68-03-3210, work assignment No. 0-7, U.S. EPA, HWERL, LPCD, Cincinnati, OH, August 1987

McCandless, R.M. and A. Bodocsi, Evaluation of Slurry Cutoff Wall/Permeable Barrier System with Organics; INTERIM REPORT, contract No. 68-03-3379, work assignment no. 0-1, U.S. EPA, HWERL, LPCD, Cincinnati, OH, September 1987.

Spangler, M. and Handy R., Soil Engineering, Harper and Row Publishers, New York, Third Edition, 1973, pp. 658-695.

Spooner, P., Slurry Trench Construction for Pollution Migration Control, Noyers Publications, Park Ridge N.J., 1985.

MECHANISMS, IMPACTS AND MODELING OF CHEMICALLY-INDUCED CHANGES IN SATURATED SOIL HYDRAULIC CONDUCTIVITY

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ABSTRACT Chemical interactions can alter the saturated hydraulic conductivity of soils. Numerous studies have reported changes of as much as 5 orders of magnitude when soils are subjected to concentrated organics or strong inorganic acids and bases. These chemically-induced impacts result from a large number of soil/permeant interaction phenomena. An overview of the mechanisms of soil/permeant interaction is presented. Experimental results are used to illustrate the potential magnitude of their impacts. Discrete Interface Penetration (DIP) models are also discussed. DIP models use simple phenomenological descriptions for making predictions of the impacts of hydraulic conductivity modifications. Examples are presented to show how well "calibrated" DIP models capture observed soil behavior.

INTRODUCTION

It is well known that a wide variety of chemical interactions can alter the saturated permeability of soils. A great deal of the recent literature on this subject has focused on the potential for soil permeability modifications at hazardous waste disposal sites. Numerous studies have reported permeability changes by as much as 5 orders of magnitude when soils are subjected to concentrated organic permeants such as acetic acid, acetone, aniline, benzene, cyclohexane, carbon tetrachloride, dioxane, ethanol, ethylene glycol, glycerol, heptane, isopropanol, methanol, nitrobenzene, phenol, trichloroethylene and xylene [1-8]. Similar results have been reported for inorganics such as hydrochloric/ hydrofluoric acid, sulfuric acid, sodium hydroxide, and ferric chloride [9-12]. Many of these studies have been quite controversial. Researchers have reported that impacts do not persist below full strength organics or very concentrated solutions [1,3,4,7]. Therefore, it is quite commonly held that laboratory measurements of soil/solute domain impacts may overestimate the realistic field impacts. There is also evidence that some laboratory results have been effected by permeameter design. Typically, flexible-wall permeameters do not yield the dramatic responses of fixed-wall permeameters [1,6,7]. This has lead some workers to conclude that many of the more dramatic soil/solute interaction data may have been corrupted by "wall effects". Nevertheless, the fact that soil/solute interactions can and commonly do alter the hydraulic properties of soils has been firmly established. What we must now do is expand our understanding of the conditions under which these interactions are likely to be significant, and our ability to analyze the consequences of such interactions.

The goal of this paper is to present an overview of what is known about the magnitude and mechanisms of chemically-induced soil permeability interactions. This will be accomplished using a variety of the experimental results. The paper will also discuss the modeling strategies that are evolving to quantify the impacts of dynamic permeability interactions. A class of models known as Discrete Interface Penetration (DIP) models has recently been proposed. DIP models use simple phenomenological descriptions for making useful "field" predictions of the impacts of permeability modifications. The development and calibration of DIP models will be illustrated. Experience has proven that these can be very successful in capturing the characteristic behavior of chemical interaction systems.

MECHANISMS OF SOIL/SOLUTE INTERACTION

Chemically-induced permeability impacts actually result from a broad class of phenomena that are of significance in many areas in addition to hazardous waste management. The impacts associated with changes in permeant ionic strength have been studied by soil scientists for many years. These impacts may be of considerable importance in irrigation and saltwater intrusion problems. The effects of "aquifer sensitivity" to permeant composition, and "aquifer stimulation" by acid leaching have also been studied by chemical and petroleum engineers. Both can exert significant field-scale impacts. In this discussion, the broad spectrum of potential soil/solute interaction mechanisms will be divided into the following six categories.

FLOCCULATION

Although Flocculation is more commonly thought of as a dispersed suspension phenomenon, it has been observed to yield significant impacts in colloidal soil systems (particle size < 2μm). Flocculation (and its counterpart deflocculation discussed in the next section) result from a change in the ionic strength of the pore water composition with which the soil is in equilibrium. Both mechanisms result from changes induced in the structure of the electric double layer that develops in the neighborhood of the soil/solution interface.

Soil surfaces in aqueous systems typically exhibit electrical charge characteristics. This charge can develop in many ways and be either positive or negative, although most soil particles exhibit a net negative charge. Because the total system (solid and liquid phase) must maintain charge neutrality, the net particle surface charge must be counterbalanced by an opposing charge that develops in the solution phase. This is often referred to as the "Diffuse Double Layer". The layer is "diffuse" because the solution ions of opposing charge are attracted to the charged soil surfaces yielding a concentration gradient in the solution phase. This steep gradient forces a counter-diffusion of ions back into the bulk solution and thus results in an "opposing" charge that is spread out over a "diffuse layer" for some relatively small distance out into the bulk solution [13].

The size and strength of the diffuse double layers that develop around charged solids are of significance when one considers the process of particle agglomeration. If particles move sufficiently close to one another, attractive forces such as Van der Waal's forces and electrostatic attractions will cause them to agglomerate. However, particles in aqueous suspension will resist agglomeration as their diffuse double layers interact resulting in a repulsive force. Therefore, the size and strength of the diffuse double layer is a significant factor in determining the solid structure of a soil. There are other factors such as the Born repulsive forces and hydration forces that come into play, but for the purpose of this discussion, they will be assumed to be negligable.

Consider the potential impact of altering the ionic strength of a solution with which a soil system has reached equilibrium. Assume first that one increased the ionic strength of the solution. As the concentration of ions in solution increases, the concentration gradient driving diffusion of the ions opposing the charge of the solid phase is decreased. This lower driving gradient decreases the size of the diffuse double layer, and thereby allows soil particles to approach one another more closely than before. This is illustrated in Fig. 1. Obviously, not all soil particles will be mobilized in response to this reduced constraint on motion, but if there is movement it will probably be characterized by the migration of smaller, more mobile solids toward the larger, more immobile solids. The net result should be a decrease in the size of the small pores separating these particles, and a increase in the size of the larger pores separating the largest solids. The cumulative result should be an increase in the soil's hydraulic conductivity.

Figure 2 illustrates data on the transient hydraulic conductivity response of a clay following an increase in permeant ionic strength [11]. The data are for a solution of ferric chloride and nickel nitrate displacing a low strength calcium sulfate solution. The test was conducted in a fixed-wall permeameter using a hydraulic gradient of 65 to 190. The soil was the Faceville clay of Claredon County, South Carolina which is primarily kaolinite. For the moment, consider only the actual experimental data

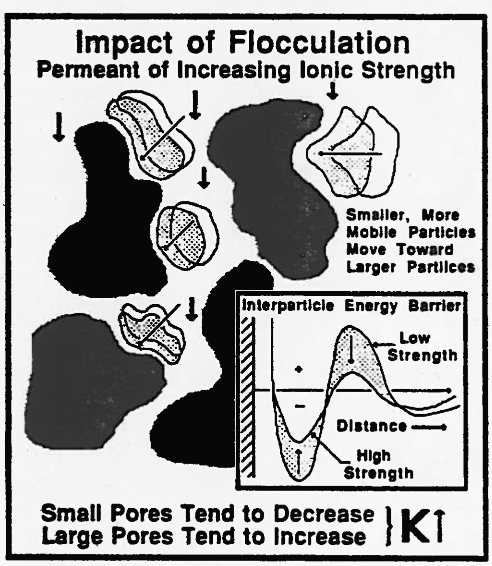


Fig. 1 - Soil/Permeant Flocculation Interactions

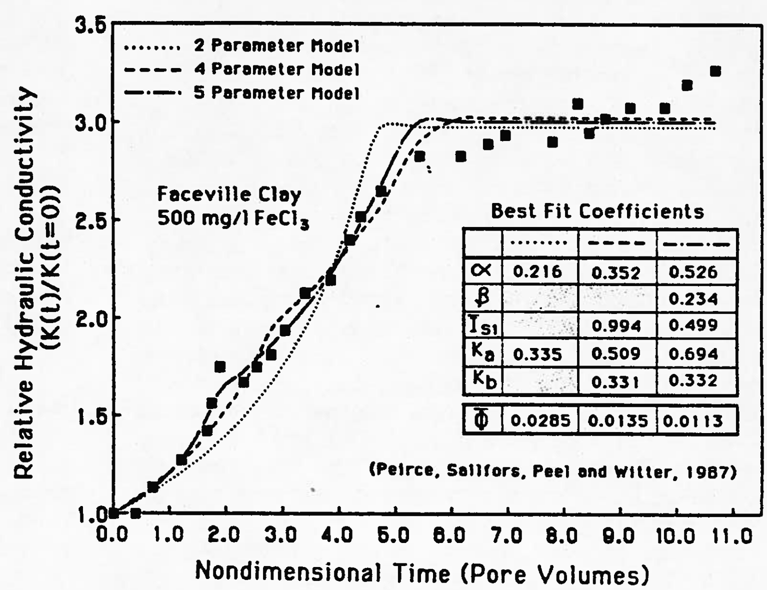


Fig. 2 - Hydraulic Conductivity Impact of Flocculation [11]

indicated by the solid squares. The lines and coefficients also indicated on this figure will be discussed in the subsequent section on response modeling. Note that the data for the relative hydraulic conductivity change (K(t)/K(t=0)) have been plotted versus a nondimensional time measured in pore volumes. One pore volume corresponds to the time required for the flow to displace the permeameter cell's pore volume exactly once.

Note that the data of Fig. 2 indicate a gradual increase in hydraulic conductivity. The experiment was apparently terminated before the ultimate equilibrium condition was reached. Also, the distinct change in the impact rate (at approximately 5.5 pore volumes) indicates that more than one mechanism was probably responsible for the impact. These data are characteristic of what one should expect from a system where flocculation is gradually shifting the pore size distribution toward larger pores.

DEFLOCCULATION

Deflocculation (also referred to as dispersion) is the reverse of flocculation. If the soil system is in equilibrium with a solution of relatively high ionic strength, the diffuse double layers will be "compressed" by the presence of so many ions in solution. If this solution is then displaced with one of lower ionic strength the diffuse double layers will expand, and will increase the probability of repulsive forces causing contiguous particles to migrate away from one another. The processes of deflocculation are illustrated in Fig. 3.

Deflocculation can have one of at least three impacts on the soil structure. If the net separation between soil particles is increased, but the "deflocculated" solids do not become entrained in the flow field, one should expect a modest decrease in hydraulic conductivity. The reason is that, although there will be no net change in the volume of the pore space, there will be a modest shift in the pore size distribution toward the smaller pore sizes as the smaller, more mobile particles move away from larger solids. If the "deflocculated" solids become entrained in the flow field, the results are normally more dramatic. If the pore structure allows the mobilized particles to be carried away, the total pore volume will increase, and will favor the formation of large pores. Under these conditions one should expect a dramatic increase in hydraulic conductivity. If the pore space is not sufficient to allow the passage of the mobilized solids, they will become lodged in the constrictions of pore channels and plug the pore space. If this occurs there will be no net increase in the pore volume, and the efficiency of the existing pore structure will be greatly reduced. Under these conditions on should expect a dramatic decrease in hydraulic conductivity.

Figure 4 presents data characteristic of the impact of deflocculation [14]. The data are for kaolinitic and montmorillonitic soils originally in equilibrium with a 1N $NaCl\text{-}CaCl_2$ solution and permeated with low ionic strength water. The data were measured in a fixed-wall, permeameter. Note that the data exhibit a rapid decline in hydraulic conductivity of nearly an order of magnitude. These results are characteristic of deflocculation followed by mobilization and subsequent entrainment. Several other workers have reported similar, or more dramatic results [15-19]. There has been some debate about whether the impacts are the result of deflocculation or swelling. Although both can occur and can have nearly identical impacts, sufficient direct measurements have now been made to confirm that particle migration is often the cause of the domain impact in "non-swelling"soils.

SHRINKING/SWELLING

A solid phase volume change (either shrinking or swelling) can also exert a considerable impact on a soil's hydraulic conductivity. When soils swell, the volume of the available pore space is reduced and the hydraulic conductivity generally declines. When soils shrink the volume of the pore space increases, and hydraulic conductivity generally increase (see Fig. 5). Shrinking/

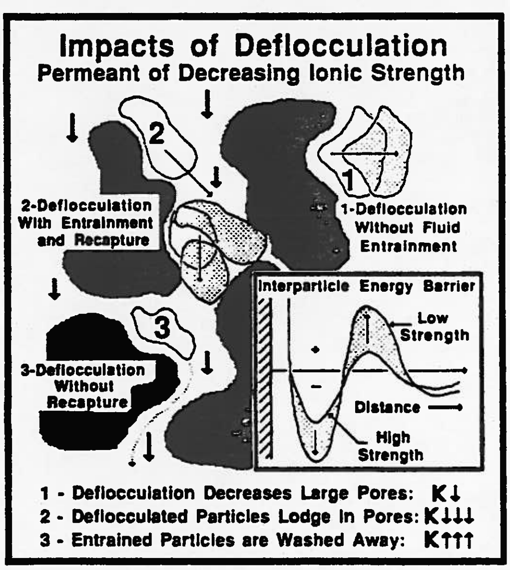


Fig. 3 - Soil/Permeant Deflocculation Interactions

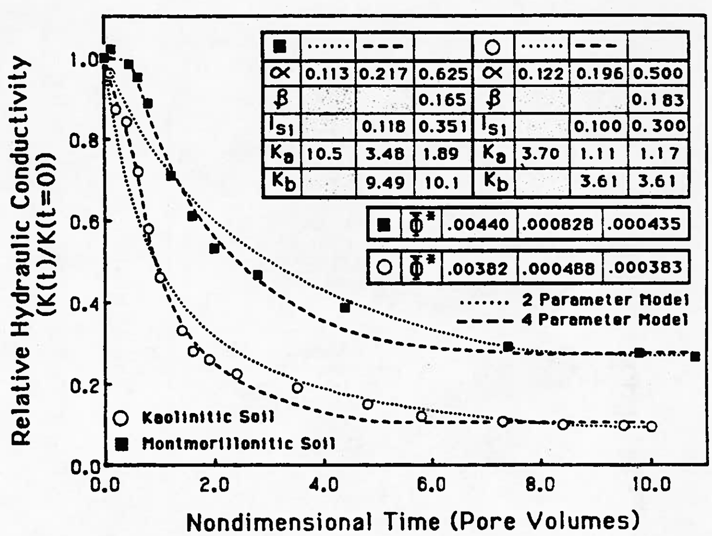


Fig. 4 - Hydraulic Conductivity Impact of Deflocculation [14]

Swelling interactions are most often associated with organic permeants in contact with clays. Although there is still some debate about the fundamental mechanisms involved, it is generally held that shrinking/swelling reactions of clays are due primarily to the hydration of their layer surfaces. Double-layer particle interactions are generally believed to play only minor roles [20].

Clay particles in water swell due to the uptake of water mass. The amount of swelling depends on the clay mineralogy. Montmorillonite can uptake (in interlayer and intercrystalline spaces) up to 10 g of water per gram of "dry" mass. Clay particles in pure organic solvents generally uptake far less solvent mass. Therefore, when a water-saturated clay is exposed to a organic solution, the displacement of water associated with the solid phase can lead to a significant decrease in the total solid phase volume. This decreased degree of swelling will be perceived of as shrinkage, and often leads to dramatic increases in hydraulic conductivity. The degree to which this shrinkage will occur depends on physical properties of both the clay and organic. Generally organic compounds that ionize in water have the greatest impact.

Figure 6 presents data for kaolinite saturated with 0.01N $CaSO_4$ and then permeated with full strength acetone [1]. The test was conducted in a fixed-wall permeameter using a hydraulic gradient of 100. Note that the shrinkage caused by the displacement of water from the solid phase resulted in a rapid and dramatic hydraulic conductivity increase of nearly two orders of magnitude. Similar results have been reported in several studies [2, 5, 19]. Results such as these were (are?) of great concern to the solid and hazardous waste disposal industry. The fear is that liquid chemical wastes or the leachates from solid wastes could attack clay soils used as hydraulic barriers, and substantially increase their hydraulic conductivity. An increase in a liner's hydraulic conductivity of one or two orders of magnitude would certainly degrade its role as a barrier to contamination migration. Fortunately, regulations now preclude most direct contact between bulk liquid wastes and soils, and research has indicated that dilute organic solutions are not likely to have the dramatic impacts of full strength organics. It should be noted that flexible-wall permeameter experiments on this same soil/permeant system did not show such dramatic results [1]. The differences were attributed to wall leakage in the fixed-wall permeameter resulting from soil shrinkage.

PRECIPITATION

Precipitations can also alter the hydraulic conductivity of soils by one of at least two distinct mechanisms (see Fig. 7). Generally these interactions consume pore volume in the domain and thus decrease the soils hydraulic conductivity. If the precipitant is formed in a reaction between a mobile (solution phase) species and a chemical component of the solid phase, the resulting precipitation will probably evolve as an addition to an immobile soil surface. The result will be a gradual consumption of the pore space and a modest decrease in hydraulic conductivity. The formation of metal oxide coatings on soil grains is an example of this type of interaction. Often the reaction is self governing and stops or greatly slows once the initial surface coating has formed. However, if the precipitation reaction is between previously soluble solution components, the impact can be much more dramatic. When a solution that previously satisfied all solubility constraints at one pH is swept into a domain with a significantly different pH, the once soluble species can begin to precipitate. If solid phase species do not enter into these reactions, it is likely that the precipitants will form as suspended colloidal solids. When these are small enough to be transported through the pore structure there will be little impact. However, as the particulates grow in size (due to continued precipitation and colloid flocculation induced by the domain tortuosity) they often become lodged in the pore space. Under these conditions the precipitants both consume pore volume, and greatly alter the efficiency of the pore structure.

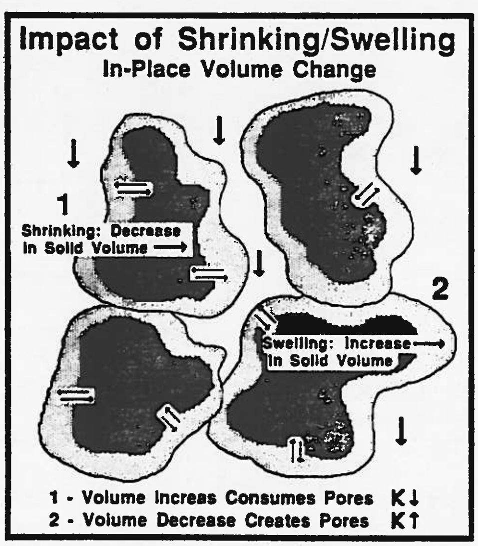


Fig. 5 - Shrinking/Swelling Soil Interactions

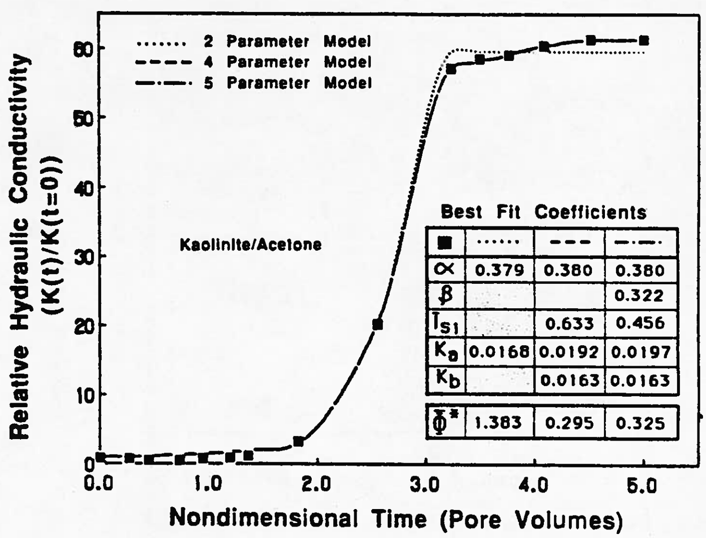


Fig. 6 - Hydraulic Conductivity Impact of Shrinking [1]

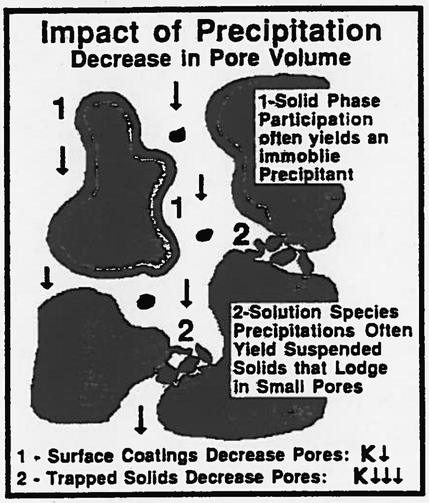


Fig. 7 - Soil/Permeant Precipitation Interactions

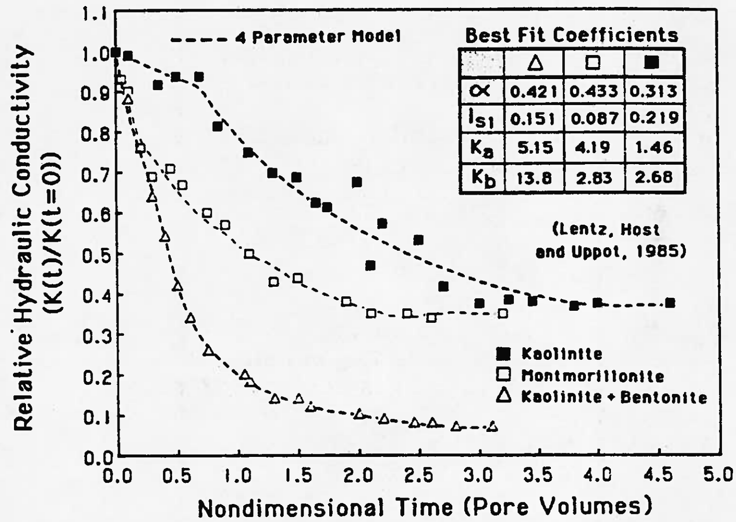


Fig. 8 - Hydraulic Conductivity Impact of Precipitation [10]

Figure 8 presents data characteristic of the impacts of precipitation reactions [10]. These data were measured during the permeation of kaolinite, magnesium montmorollionite, and a kaolinite/bentonite mixture with a high pH sodium hydroxide solution. The data were measured in a flexible-wall triaxial permeameter operated under a confining pressure of 34.5 kPa (5 psi) and a potential gradient of 400 to 500. The soils were tested with permeants at pHs of 9, 11, and 13. Little impact was observed below a pH of 13. The impacts at pH 13 were attributed to the precipitation of previously soluble calcium carbonate and magnesium hydroxide that formed and then plugged the soil's pore space.

DISSOLUTION

Dissolution reactions are the reverse of precipitation reactions. Precipitations involve the formation of new solid phase mass either by direct precipitation onto immobile surfaces or by the trapping of colloidal precipitants in the pore space. Dissolution involves the transfer of immobile (solid phase) mass to the liquid phase in response to an unsatisfied solubility constraint. Since dissolutions should increase the pore volume, they are expected to increase hydraulic conductivity. This is not always the case (see Fig. 9). If the species entering solution remain in solution, then one should expect the pore space to increase. This generally yields a relatively modest increase in hydraulic conductivity. The increase is often modest because the rates of dissolutions are effected by the solution species concentrations and will often be greatest in the zones of highest solution velocity. These are the locations where the products of the dissolution are most rapidly swept away from the reaction site thus supplying the highest possible concentration driving gradient. However, since the zones of highest solution velocity generally correspond to the largest domain pores, the impacts of the increased pore volume are less significant than might otherwise be expected.

It is interesting to note that dissolutions do not always yield the expected result. If, for example, the solution is quite aggressive, a great deal of the solid phase may attempt to dissolve. However, there is no guarantee that all of the products will remain in solution. If, for example, a carbonate mineral dissolves aggressively, it is quite possible for the result to exceed the Henry's law gas solubility for carbon dioxide (see Fig. 9). When this occurs, a gas phase can form and plug the pore space thus decreasing the hydraulic conductivity. Although this would probably be a temporary condition, "temporary" can be a very long time.

Figure 10 presents data illustrating the impact of a dissolution that was not constrained by gas formation [12]. These data were measured in a fixed-wall permeameter at a hydraulic gradient of 10. The soil was taken from the calcareous Aquia Greensand formation of Maryland and was permeated with a 0.005 M sulfuric acid solution. The impact was rather gradual, but ultimately the hydraulic conductivity was substantially increased. Dissolved solids testing of the permeameter effluent confirmed that previously immobile mass was being transported out of the cell. Figure 11 presents data for this same soil permeated with a more concentrated (0.025 M) sulfuric acid solution. Logic would suggest that the dissolution sould be more rapid, and the hydraulic conductivity should increase to an even higher value due to a higher degree of reaction completion. Contrary to this expectation, the hydraulic conductivity actually declined. The reason is that the increased permeant strength was sufficiently aggressive to dissolve more calcium carbonate than could remain in solution, and CO_2 gas formed within the domain.

ADDITIONAL INTERACTION MECHANISMS

The five classes of interaction mechanisms discussed above are certainly not exhaustive. There are several possible chemical interactions (ion exchanges, sorptions, surface complexations, chelations etc.) that can alter the surface properties of the solid phase and thus alter the particle-to-particle structure of the domain. Under field conditions many permeants also carry suspended solids

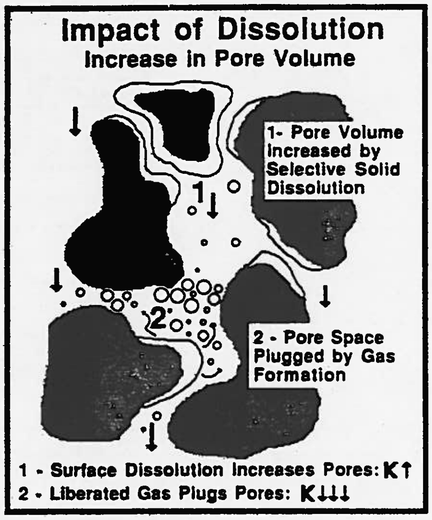


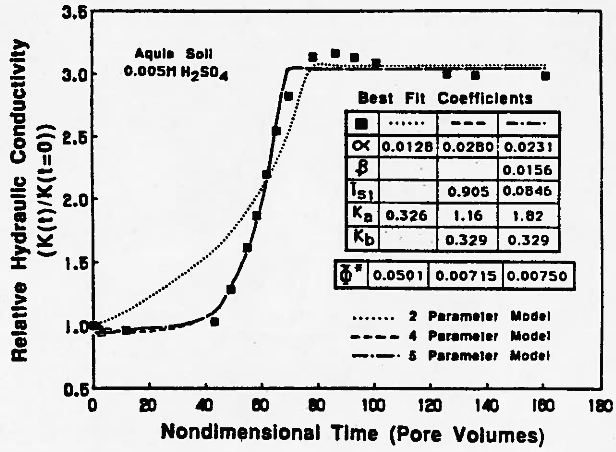
Fig. 9 - Soil/Permeant Dissolution Interactions

(often of very fine colloidal matter) that can become trapped in the soil by mechanisms such as filtration. The physical state of the soil domain can also alter the soil's hydraulic conductivity. As the waste disposal cell is filled above a soil liner system, the added load induces additional stress in the soil and can produce a consolidation that decreases the hydraulic conductivity. Finally the potential impacts of biological activity cannot be overlooked. The introduction of a new substrate source such as a groundwater pollution plume may accelerate the growth of native soil organisms. Contaminated permeants may also seed the domain with microbes that are acclimated to the new substrate. In either case accelerated biological growth can consume (plug) the pore space and lead to substantial declines in hydraulic conductivity. Such conditions are probably temporary since as the hydraulic conductivity declines, the flux of substrate to the biological population also declines and the increased biomass cannot be sustained. However, as in the problem with gas phase evolution, "temporary" conditions can exist for a surprisingly long time.

DISCRETE INTERFACE PENETRATION MODELS

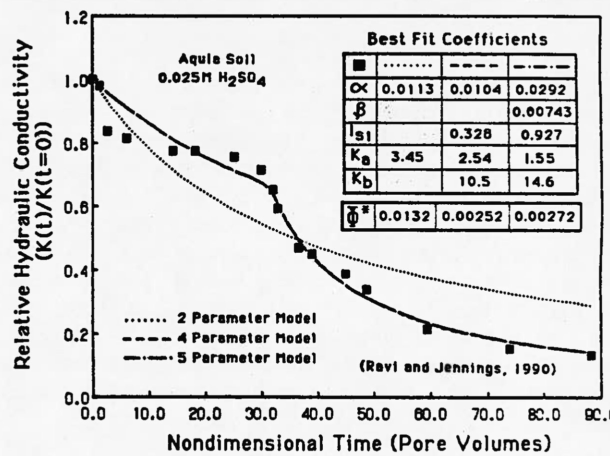
It has long been known that soil/solute interactions such as those discussed in the previous section can alter the hydraulic conductivity of soil. Numerous authors have published data on the change in hydraulic conductivity under a wide variety of conditions. Methods for actually quantifying the domain impact are more difficult to come by. Modeling attempts have been made [22,23] but these tend to be very specialized and difficult to generalize. The Discrete Interface Penetration (DIP) strategy for formulating domain impact models is an exception to this "rule". The DIP modeling strategy yields relatively simple but very general analytical solutions for the domain impacts that will result from dynamic soil/permeant interactions.

The fundamental concept of a DIP model is illustrated in Fig. 12. Basically, it is assumed that the domain may



■	·······	– – –	–·–·–
α	0.0128	0.0280	0.0231
β			0.0156
T_{S1}		0.905	0.0846
K_a	0.326	1.16	1.82
K_b		0.329	0.329
Φ^*	0.0501	0.00715	0.00750

Fig.10 - Hydraulic Conductivity Impact of Dissolution Resulting From a 0.005 M H_2SO_4 Permeant [12]



■	·······	– – – –	–·–·–
α	0.0113	0.0104	0.0292
β			0.00743
T_{S1}		0.328	0.927
K_a	3.45	2.54	1.55
K_b		10.5	14.6
Φ^*	0.0132	0.00252	0.00272

Fig.11- Hydraulic Conductivity Impact of Dissolution Resulting From a 0.025 M H_2SO_4 Permeant [12]

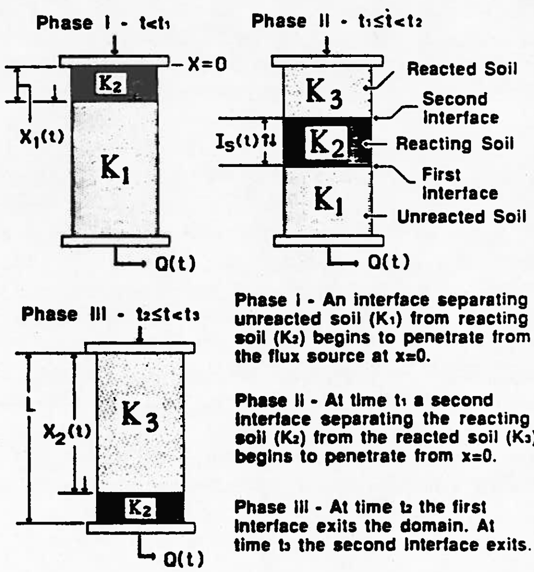


Fig. 12 - The DIP Model Concept of Traveling Interfaces Separating Zones of Altered Material Properties [26]

be divided into distinct zones separated by traveling interfaces. The distinct zones define the areas where soil/solute interactions are occurring. Distinct physical properties may be specified for each zone. The zones are separated by discrete interfaces that are propagated at velocities dictated by the rate of the soil/permeant interaction. Several DIP models have been formulated. Otte and Jennings [24] describe a single interface model based on a "unreacted" zone of unaltered soil and a "reacted" zone of altered soil properties. Mundell and Jennings [25] extended this concept to include a "reacting" zone of fixed length where the soil's properties could take on values other than their equilibrium values. Most recently, Jennings and Ravi [26] extended the three zone DIP model to allow the "reacting" zone to vary in size as the interactions develop. They also present nondimensional solutions that are independent of boundary conditions. These may be applied to a wide variety of permeameter designs (constant head, constant flux, falling head) to evaluate the required coefficients.

Jennings and Ravi [26] present a detailed discussion of how DIP models are formulated and solved. The analytical details of this procedure will be omitted here. Let it suffice to say that once the basic domain discretization has been postulated, the mathematical problem may be solved analytically to yield a prediction of the transient Darcy velocity q(t), the transient potential field that will evolve during the interaction $\phi(x,t)$, and the domain's bulk hydraulic conductivity response K(t). The solutions are often nonlinear, and time must be discretized into analytically distinct phases, but the solutions have proven to be quite successful in representing the responses observed in laboratory studies. Bulk hydraulic conductivity solutions for three DIP modes are summarized in Table 1.

Table 1 - DIP Model Solutions for K(t)/K(t=0)

Penetration Phase	Bulk Hydraulic Conductivity $\overline{K}(t)$	Interface Location Functions
	2-Parameter Model (Otte and Jennings, 1984)	
$0 \le t \le t_1$	$[1+(K_a-1)\overline{X}_1(t)]^{-1}$	$\overline{X}_1(t) = \alpha\theta$
	4-Parameter Model (Mundell and Jennings, 1987)	
$0 \le t \le t_1$	$[1+(K_a-1)\overline{X}_1(t)]^{-1}$	$\overline{X}_1(t) = \alpha\theta$
$t_1 < t \le t_2$	$[1+(K_a-1)\overline{I}_{S1}+(K_b-1)\overline{X}_2(t)]^{-1}$	$\overline{X}_2(t) = \alpha\theta - \overline{I}_{S1}$
$t_2 < t \le t_3$	$[K_a+(K_a-K_b)\overline{X}_2(t)]^{-1}$	$\overline{X}_2(t) = \alpha\theta - \overline{I}_{S1}$
	5-Parameter Model (Jennings and Ravi, 1990)	
$0 \le t \le t_1$	$[1+(K_a-1)\overline{X}_1(t)]^{-1}$	$\overline{X}_1(t) = \alpha\theta$
$t_1 < t \le t_2$	$[1+(K_a-1)\overline{I}_S(t)+(K_b-1)\overline{X}_2(t)]^{-1}$	$\overline{X}_2(t) = \beta(\theta-(\overline{I}_{S1}/\alpha))$ $\overline{I}_S(t) = (\alpha-\beta)\theta+(\beta\overline{I}_{S1}/\alpha)$
$t_2 < t \le t_3$	$[K_a+(K_a-K_b)\overline{X}_2(t)]^{-1}$	$\overline{X}_2(t) = \beta(\theta-(\overline{I}_{S1}/\alpha))$

$K_a = K_1/K_2$; $K_b = K_1/K_3$; $\overline{X}_1(t) = X_1(t)/L$; $\overline{X}_2(t) = X_2(t)/L$; $\overline{I}_S(t) = \overline{X}_1(t) - \overline{X}_2(t)$
α, β = interface penetration rate constants ; θ = time in pore volumes

APPLICATIONS OF DIP MODELING

DIP models have proven to be quite successful in capturing the dynamic responses observed in typical permeameter studies. As evidence of this, the "best fit" DIP model representations for a 2-parameter [24], 4-parameter [25] and 5-parameter formulation [26] have been included on Figures 2,4,6,8,10, and 11. All of these were achieved by a non-linear least squares optimization based on the Levenberg- Marquardt algorithm [12]. Additional examples of DIP model fits to existing permeameter data may also be found in that reference.

One should note that, in general, the DIP model results presented on Figures 2,4,6,8,10, and 11 are quite successful in capturing the observed soil behavior over a wide range of time and hydraulic conductivity responses and for a wide variety of interaction mechanisms. Not all three models are equally successful at capturing the observed behavior, but nearly any of the fits could be used as credible approximations of the observed behavior. In general, the 4-parameter model provides the best results. The 2-parameter model is often too inflexible to capture subtle behavior. The 5-parameter model always fits slightly better than the 4-parameter model when the absolute residual is considered, but when the residual (ϕ) is normalized by both the number of data observations (m) and the number of parameters (n) [i.e. $\phi^* = \phi/(m-n)$] the 4-parameter model often yields a superior fit. Applying the principle that, all else being equal, one should opt for the simplest solution, the 4-parameter DIP model should often be selected.

The examples fits presented here were selected to illustrate the impacts of several interaction mechanisms rather than to illustrate DIP model fits. Numerous other data sets have been analyzed with similar results. This is not to imply that DIP models always "fit" observed behavior, or that it is "easy" to fit DIP models to data. It is also true that DIP model parameters can suffer from non-uniqueness. This is generally the case for data with a substantial experimental randomness. For these conditions Monte Carlo techniques may be used to quantify the degree of parameter uncertainty.

SUMMARY AND CONCLUSIONS

There are numerous soil/permeant interaction mechanisms that are capable of altering the hydraulic conductivity of soil. These can occur whenever soil is exposed to a permeant for which it has not had ample opportunity to equilibrate, or whenever a disturbed soil is exposed to any permeant. The magnitude and direction of the hydraulic conductivity changes are difficult to predict. Normally experimental data are required.

Discrete Interface Penetration (DIP) models are a class of analytical tools designed to help predict the hydraulic behavior of soils exposed to aggressive permeants. DIP models may be calibrated from conventional laboratory data measured in nearly any type of permeameter. Experience has demonstrated that DIP models are usually very good at representing systematic soil response data. As laboratory data become more random, it becomes much more difficult to achieve meaningful DIP model fits. It is also possible for DIP model fits to suffer from non-uniqueness. However, even for cases where the parameter values are non-unique, the models generally still provide a reasonable good representation of the observed data.

DIP models may also be used to extend laboratory scale permeameter data to field-scale [27]. This can be done because DIP models provide a nondimensional analysis that may be re-dimensionalized for any desired scale. Engineers should used this DIP model feature with caution. Data gleaned from as small soil sample will probably not be characteristic of a whole soil formation. Furthermore, changes in scale can alter the fundamental mechanisms at work, and the multi-dimensional response of field situations can alter the way domain impacts are exerted. Nevertheless, DIP models offer a means of anticipating the long term responses of soil systems in extremely challenging situations such as hazardous waste disposal facilities. The wise engineer will also provide monitoring devices and a sampling program to gauge the "validity" of these prediction.

ACKNOWLEDGEMENT

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REFERENCES

1. Acar, Y.B., Hamidon, A., Field, S.D. and Scott, L., "The Effect of Organic Fluids on Hydraulic Conductivity of Kaolinite", Hydraulic Barriers in Soil and Rock, ed. A.I. Johnson et al., ASTM STP 874, 171-187, 1985.

2. Anderson, D.C., Brown, K.W. and Thomas, J.C., "Conductivity of Compacted Clay Soils to Water and Organic Liquids", Waste Management and Research, 3(4), 339-349, 1985

3. Bowders, J.J. and Daniel, D.E., "Hydraulic Conductivity of Compacted Clay to Dilute Organic Chemicals", Journal of Geotechnical Engineering, 113(12), 1433-1448, 1987.

4. Evans, J.C. and Kugelman, I.J., "Organic Fluids Effects on the Strength, Deformation and Permeability of Soil-Bentonite Slurry Walls", Proceedings of the 7th Mid-Atlantic Industrial Waste Conference, Technomic Publishing Co. Inc., 275-291, 1985.

5. Fernandez, F. and Quigley, R.M., "Hydraulic Conductivity of Natural Clays Permeated with Simple Liquid Hydrocarbons", Canadian Geotechnical Journal, 22(2), 205-214, 1985.

6. Foreman, D.E. and Daniel, D.E., "Permeation of Compacted Clay with Organic Chemicals", Journal of Geotechnical Engineering, 112(7), 669-681,1986.

7. Mitchell, J.K. and Madsen, F.T., "Chemical Effects on Clay Hydraulic Conductivity", in Geotechnical Practice for Waste Disposal '87, Proceedings of the ASCE-GT Specialty Conference, ed. R.D. Woods, Ann Arbor, Michigan, June 15-17, 87-115, 1987.

8. Schramm, M., Warrick, A.W. and Fuller, W.H., "Permeability of Soils to Four Organic Liquids and Water", Hazardous Waste and Hazardous Materials, 3(1), 21-27, 1986.

9. Fogler, H.S. and McCune, C.C.," On the Extension of the Model of Matric Acid Stimulation to Different Sandstones", AIChE Journal, 22, 799-805, 1976.

10. Lentz , R.W., Horst, W.D. and Uppot, J.O.,"The Permeability of Clay to Acidic and Caustic Permeants", Hydraulic Barriers in Soil and Rock, ASTM STP 874, 127-139, 1985.

11. Pierce, J.J., Sallifors, G., Peel, T.A. and Witter, K.A., "Effects of Selected Inorganic Leachates on Clay Permeability", Journal of Geotechnical Engineering, 113(4), 915-919, 1987

12. Ravi, V. and Jennings, A.A., "Penetration Model Parameter Estimation for Dynamic Permeability Measurements", Soil Science Society of America Journal, 54(1), 13-19, 1990.

13. Weber, W.J., Physicochemical Processes for Water Quality Control, Wiley-Interscience, N.Y, 1972.

14. Frenkel, H. Goertzen, J.O. and Rhodes, J.D. , "Effects of Clay Type and Content, Exchangeab;e Sodium Percentage, and Electrolyte Concentration on Clay Dispersion and Soil Hydraulic Conductivity", Soil Sci. Soc. Am. J., 42, 32-39, 1978.

15. Goldenberg, L.C., Magaritz, M., Amiel, A.J. and Mandel, S., "Changes in Hydraulic Conductivity of Laboratory Sand-Clay Mixtures Caused by a Seawater-Freshwater Interface", Journal of Hydrology, 40, 329-336, 1984.

16. Goldenberg, L.C., Magaritz, M. and Mandel, S., "Experimental Investigation on Irreversible Changes of Hydraulic Conductivity on the Seawater-Freshwater Interface in Coastal Aquifers", Water Resources Research, 19(1),77-85, 1983.

17. Mehnert, E. and Jennings, A.A., "The Effect of Salinity-Dependent Hydraulic Conductivity on Saltwater Intrusion Episodes", Journal of Hydrology, 80(3/4), 283-297, 1985.

18. Shainberg, I, Rhoades, J.D. and Prather, R.J.,"Effect of Low Electrolyte Concentrations on Clay Dispersion and Hydraulic Conductivity of a Sodic Soil", Soil Science Society of America Journal, 45(2), 273-277, 1981.

19. Yaron, B. and Thomas, G.W., "Soil Hydraulic Conductivity as Affected by Sodic Water", Water Resourses Research, 4(3), 545-552,1968.

20. Chen, S., Low, P.F. , Cushman, J.H. and Roth, C.R.,"Organic Compound Effects on Swelling and Flocculation of Upton Montmorillonite, Soil Sci. Soc. Am. J., 51, 1444,1450, 1987.

21. Green W.J., Lee, G.F. and Jones, R.A. and Palit, T., "Interaction of Clay Soils with Water and Organic Solvents: Implications for the Disposal of Hazardous Waste", Environmental Science and Technology, 17(5), 278-282, 1983.

22. Fogler, H.S. Lund, K. and McCune, C.C., "Predicting the Flow and Reaction of HCl/HF Acid Mixtures in Porous Sandstone Cores" Society of Pertoleum Engineers Journal, 248-260, 1976

23. Guin, J.A., Schechter, R.S. and Silberberg, I.H., "Chemically Induced Cjanges in Porous Media", Industrial and Engineering Chemistry Fundamentals, 10, 50-54, 1971

24. Otte, F.E. and Jennings, A.A.,"The Physical Consequences of Acid Reactions with Soil", in Proceedings of the 5th Indiana Water Resources Symposium, (ed.), J.D. Martin, Bloomington IN. June 20-22, 176-185, 1984.

25. Mundell, J.A. and Jennings, A.A., "Penetration Models for Dynamic Permeability Analysis", Journal of Environmental Engineering, 113(4), 881-899, 1987.

26. Jennings, A.A. and Ravi, V., "Analytical Extensions of the Discrete Interface Penetration Model for Dynamic Permeability Analysis", Soil Science Society of America Journal, 54(1), 7-12, 1990.

27. Ravi, V. and Jennings, A.A. , "Application of Discrete Interface Penetration Models to the Evaluation of Hydraulic Barriers", in Proceedings of the Twelth Annual Madison Waste Conference, Madison, WI Sept 20-21, 1989, 377-394

EFFECTS OF FREEZE/THAW ON THE HYDRAULIC CONDUCTIVITY OF COMPACTED SOILS

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ABSTRACT

Construction seasons for landfills located in the Ohio River Valley region are often shortened due to regulations limiting the exposure of liner components, soils and synthetics, to conditions of freeze/thaw. Manufacturers of synthetics have been subjecting their products to low and high temperatures and are reporting the effects thereof on the properties of their materials. Similar testing for the soil/clay component of liners has not been well documented. An on-going research project at West Virginia University is directed at determining the effects of freeze/thaw cycles on the hydraulic conductivity of compacted soils.

Two soils, including one from an actual landfill liner and a processed kaolin, were compacted in the laboratory and their hydraulic conductivities measured. The specimens were then removed from the flexible-wall permeameters, placed in an environmental chamber and subjected to various numbers of freeze/thaw cycles. Temperatures ranged from 0°F to 70°F. Specimens were removed from the chamber and again the hydraulic conductivities were measured. However, this time various effective confining pressures and hydraulic gradients were used. The gradients and effective confining pressure were increased as the permeability test progressed. Specimens were again removed from the permeameters and subjected to additional freeze/thaw cycles. Then they were repermeated.

Findings indicate that the freeze/thaw cycles affected the hydraulic conductivity of the soils. At low confining pressures, the hydraulic conductivities of the specimens after freeze/thaw showed the greatest increase, sometimes as great as three orders of magnitude. At increased confining pressures the hydraulic conductivities returned to the values before freeze/thaw. It appears that any increase in porosity due to freezing can be altered by moderate increases in the effective confining stress, resulting in a decrease in the hydraulic conductivity.

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INTRODUCTION

Spurred by increasingly stringent regulations (WV DNR,1990) modern landfill waste disposal practice, hazardous or municipal, seeks to protect groundwater by isolating the groundwater from any liquids in the disposal cell. This is done by lining the water disposal cell with low-permeability barriers. The two main types of barriers are flexible membranes of organic polymers which protect due to their extremely low permeability and liners based on clay minerals which provide protection due to a combination of low permeability and a thick barrier As an added factor of safety more than one liner is usually used. Frequently there is one of each type. If however, one of the barriers is damaged between installation and use the liner system will not perform as designed and may not serve to protect the groundwater at all.

One proposed mechanism of liner damage is freezing and thawing of clay liners. The purpose of this investigation is to examine the effects of freezing and thawing on the permeability of compacted clay. By measuring the permeability of specimens, repeatedly freezing and thawing them, and remeasuring the permeability, the effects of the freezing and thawing, should be apparent.

The objectives of this research are to examine the behavior of clay liner materials as a result of being subjected to freeze-thaw conditions. It is also desirable to see if a testing regime can be devised which is a suitable model to provide insight into what is happening in real clay liners and yet is simple and inexpensive enough that it can be performed by those designing clay liners for temperate climates. With insight into the behavior of clay liners under freeze-thaw conditions in the real world and a testing regime simple, timely, and inexpensive enough that it would really be conducted, clay liner design could be based on sound engineering design, fitted to the conditions at hand and founded on adequate theoretical and empirical considerations rather than being based on regulatory fiat founded on fear, ignorance, and speculation.

Concern with the possibility of changes in the vertical permeability of clay liners is so recent that the literature directly bearing on the subject is quite limited. There is a large, tangentially relevant literature concerning frost heave mostly done by highway departments. Much of this has been published by the Transportation Research Board over the years. There is also a literature which looks at depth of freezing and changes in soil structure from an agricultural (soil science) point of view. Much of this can be found in the publications of the state agricultural experiment stations in the northern states. Until the literature on changes in permeability in liners subjected to freeze-thaw cycles (the geotechnical point of view) matures and defines the most relevant soil properties and conditions to the problem, it will be difficult to tie in this older tangentially related literature.

Some workers have proposed models of the effects of freezing on soil structure but a concensus does not seem to have been reached. Konrad (1989) ascribes the effects to void ratio with increased packing leading to a more acute apex angle between clay plates. The more acute angle means that there are smaller channels filled with free unfrozen water. Dash (1989) ascribes frost heave to thermomolecular pressure associated with surface melting. At some point an adequate theoretical model will likely be developed which will be useful in explaining changes in permeability due to freezing and thawing. Currently, however, clay barrier design and placement is driven by socio-political pressure.

The literature that is most immediately useful to the liner question is that pertaining to artificial ground freezing as a soil stabilization technique. Temporary artificial freezing of soils to allow construction under difficult conditions has been practiced for some years. Since the behavior of the soil after it has been thawed is of interest to these practioners, i.e., a building successfully constructed on artificially frozen soil which falls down when the soil thaws would be useless, there have been a number of investigations into the changes of soil properties due to freezing and thawing. Chamberlain and Gow (1979) found that freezing and thawing changed the permeability and structure of four soils. They attributed this to the formation of polygonal shrinkage cracks or the reduction of the volume of the fines in the voids. They found an indication that more change occurred in soils of higher plasticity but were not able to conclusively establish a definite relationship. Chamberlain (1981) found that increased vertical permeability could be accounted for by the vertical ice-filled cracks and horizontal ice lenses observed in the section.

More recently there have been several investigations where the changes in permeability of caps and liners have been an objective. Chamberlain (1989) has summarized the state of the art in relation to the effects of freeze/thaw cycles on the engineering properties of clays. Chamberlain, Iskander and Hunsiker (1990) found that freezing did increase the permeability of four soils being considered for cap and liner materials. They also give a list of seven areas where more research is needed to assess the suitability of clay liners in seasonally frozen areas. Zimmie ands LaPlante (1990) investigated the effects of freeze/thaw cycles on a clay soil. The examined the difference between onedimensionsal and three-dimensional freezing and concluded that for practical purposes, they were identical. They also noted that additional research should be performed using three dimensional freezing and different soil types.

MATERIALS

Two soils were used in these tests. One was a commercially processed kaolinite. The other soil was a sample of the material (a ground shale) which was being used for the clay liner in an active municipal solid waste landfill. The properties of these soils are listed in Table 1.

Table 1 - Soil Characteristics

Property	Kaolinite	Wetzel County
Natural Water Content (%)	Dry	Dry
Max. Dry Unit Wt. (pcf)*	86	127
Opt. Water Content (%)	31	11
Spec. Gravity	2.59	2.70
Percent Finer Than #200	90	50
Liquid/Plastic Limits	58/34	33/24
Source	Georgia Kaolin	Wetzel County WV

METHODS

The soils were moistened to 1% wet of optimum moisture content and mixed in a commercial mixer. They were then compacted manually in a Proctor mold using the standard Proctor energy (ASTM D-698 Method A). After compaction the samples were kept in self-sealing plastic freezer bags in a moist room (as they were any time there was a pause in testing). The samples were tested in flexible wall permeameters. They were initially saturated by flowing one pore volume of the permeating liquid through the sample with back pressure. This procedure has been found to be effective in saturating soil specimens (Dunn,1985). After the sample had been permeated with one pore volume of liquid, the permeation was continued until the value of the coefficient of permeability was stable.

The permeating liquid was distilled, deionzized water. After the samples were saturated and the coefficient of permeability was measured, they were placed in a freezing chamber. The kaolinite samples were tested in a cycle of 0°F to 50°F three times per day. The Wetzel County soil was subjected to a 0°F to 70° F cycle once per day regime for six days. After the samples were removed from the freezing chamber they were visually inspected for signs of changes caused by the freezing and thawing. The specimens were then remounted in the permeameters. The latex membrane was not removed at any time during the test. The samples were then tested for changes in the coefficient of permeability caused by the freezing and thawing. The initial readings were made at a low hydraulic gradient with a low confining stress. Confining stress and gradient were increased for later readings so that the effects on permeability of different gradients and confining stresses could be determined.

RESULTS

When the specimens were removed from the environmental chamber they were visually inspected for macroscopic damage from the freezing and thawing. The filter papers at the ends of the specimens were removed but the latex membranes were not.

All specimens showed some effects although not necessarily a large effect. The kaolinite specimens which seem to have frozen once and stayed frozen and the processed shale specimens from Wetzel County showed only small effects. The kaolinite specimens which under went the 0 F to 110 F cycles showed dramatic changes. Two of the three specimens had large cracks which significantly (3 orders of magnitude) increased the permeability (Fig. 1).

Some samples Were dyed to see if there were preferred flow paths within the compacted samples. The samples did not section well; when sawed with either a hacksaw or a very thin diamond impregnated copper wire the process smeared and polished the texture of the dried samples. They could be split by holding a knife across the specimen and hitting the back of the knife with a hammer. This split the specimen although not in a good cross-section. The dye was mixed to a deep red color but only enough dye was retained in the soil to color the white kaolinite pink. The specimen which was dyed and then split without freezing was a uniform pink throughout. The specimen which was dyed and then frozen and repermeated was a uniform pink except near the inflow water end where the specimen was white. Since the unfrozen dyed specimen had a white patch near the outflow end and since the tailwater in the frozen, dyed specimen was colored even when it was being repermeated with clear water it seems that the red dye was only lightly adsorbed on the kaolinite particles and is easily dislodged.

When the dyed, frozen specimen was repermeated the effluent water was opaque and salmon colored. When this effluent water was put in a sealed flask and left undisturbed fine kaolin particles settled out of suspension and the fluid was clear and light red. A macroscopic visual inspection of the specimen after it was taken out of the environmental chamber showed that it has been strongly affected by the freeze thaw cycles and that there were open longitudinal cracks. The measured permeability increased four orders of magnitude (10^{-8} to 10^{-4} cm/s) under low confining stress and a low hydraulic gradient. However there was a leak at the base of the cell so the numbers should be used with some caution. With increasing confining stress the value of k returned to the prefreezing levels even at a high hydraulic gradient.

The third and fourth kaolin specimens were dyed after being frozen. The third specimen like the second showed strongly the effects of the freezing and thawing with open longitudinal cracks like the second specimen. The fourth specimen was also strongly affected by the freezing and thawing but did not have the open longitudinal cracks of the second and third specimens.

The effluent water of the third specimen was deep red within a few minutes of the start of the test. The edges of the longitudinal cracks were dyed light red. As flow continued, the red edges on the cracks lightened in color even though more dye was being added at the headwater. The entire specimen eventually became a uniform light pink and the concentrated dye seemed to diffuse away from the edges of the longitudinal cracks. There was a three order of magnitude (10^{-8} to 10^{-5} cm/s) increase in k after the freeze-thaw cycles at low confining stress and gradient which disappeared with increased confining stress and higher gradient (Fig. 1).

The fourth specimen which was less damaged did not have a quick dye breakthrough like specimen three. This was probably due to the lack of open longitudinal cracks such as were noted on specimens two and three. There was a two order of magnitude (10^{-8} to10^{-6} cm/s) increase in k at low confining stress and hydraulic gradient which likewise disappeared at higher confining stress and hydraulic gradient (Fig. 2).

Four samples of a ground gray shale from the Wetzel County showed little effect of being subjected to a week of 0°F to 70°F one cycle per day of freezing and thawing (Fig. 3). In contrast to the heavy damage to the kaolinite samples only one of the Wetzel County Landfill specimens showed any longitudinal cracking. This crack was only about 3/4" deep and 1mm or less wide. This specimen was also the only one of these specimens which showed an increase in k. The increase was less than half an order of magnitude.

CONCLUSIONS

The results of the tests on compacted clay specimens show that permeability usually increases after the samples are subjected to freeze thaw cycles. This increase may be small, 1/2 order of magnitude or less (Fig. 3), or it may be large, up to three orders of magnitude (Fig. 1). This effect is decreased with increased confining stress (Fig. 2).

Although the testing method may somewhat mask the effects of freezing and thawing on clay liners. The results obtained tended to indicated some implications for the use of clay liners in areas where the temperature drops below freezing. One is that clay liners could be damaged by freezing and thawing and should be protected where possible. Another implication is that the damage is not irreversible or irrepairable. Careful attention to confining stresses during design may be sufficient. Another result is that materials vary in their response and that implies that case by case regulation based on testing specific materials may be preferable to fiat prohibition.

ACKNOWLEDGEMENTS

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REFERENCES

Chamberlain, E.J. (1981) "Overconsolidation Effects of Ground Freezing," Engrg. Geology, 18, 97-110.

Chamberlain, E.J. (1989) "Physical Changes in Clays Due to Frost Action and Their Effect on Engineering Structures," VTT Symp. Offprint, Technical Research Center of Finland, Espov.

Chamberlain, E.J. and Gow, A.J. (1979) "Effect of Freezing and Thawing on the Permeability and Structure of Soils," Engrg. Geology, 13, 73-92.

Chamberlain, E.J., Iskander, I. and Hunsiker, S.E. (1990) "Effect of Freeze-Thaw Cycles on the Permeability and Microstructure of Soils," US Army Cold Regions Res. and Engrg. Lab., Special Rept. 90-1.

Dash, J.G. (1989) "Thermomolecular Pressure in Surface Melting: Motivator for Frost Heave," Science, 248, 1591-1593.

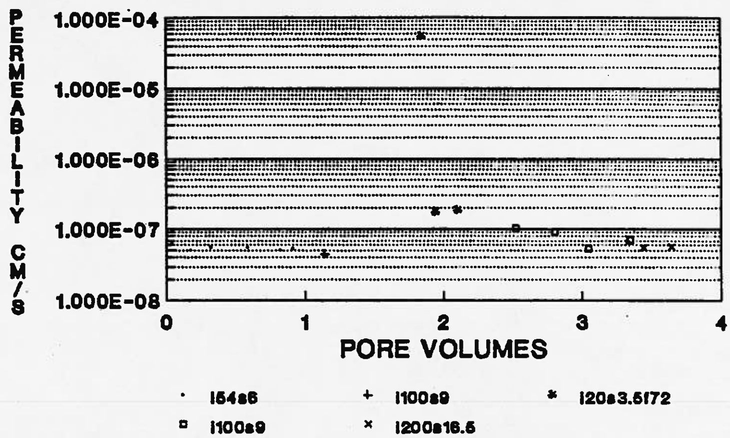
Dunn, R.J. (1985) "Laboratory Measurement of Fine-Grained Soil Fluid Conductivity," Engrg. Geology, 21, 215-223.

Konard, J.M. (1989) "Influence of Overconsolidation on the Freezing Characteristics of a Clayey Till," Canadian Geotechnical Journal, 26, 9-21.

West Virginia Dept of Natural Resources (1990) "Solid Waste Management Rules," Title 47, Series 38.

Zimmie, T.F. and LaPlante, L. (1990) " The Effect of Freeze/Thaw Cycles on the Permeability of a Fine - grained Soil," Proc. 22nd Mid-Atlantic Industrial Waste Conf., Drexel Univ., July 24-27, pp 580-593.

Figure 1- Effect of Freeze/Thaw on Permeability of Compacted Kaolin



(i- Hydr. Grad.;s- effective stress)

Figure 2- Freeze/Thaw Effect on Permeability of Compacted Kaolin

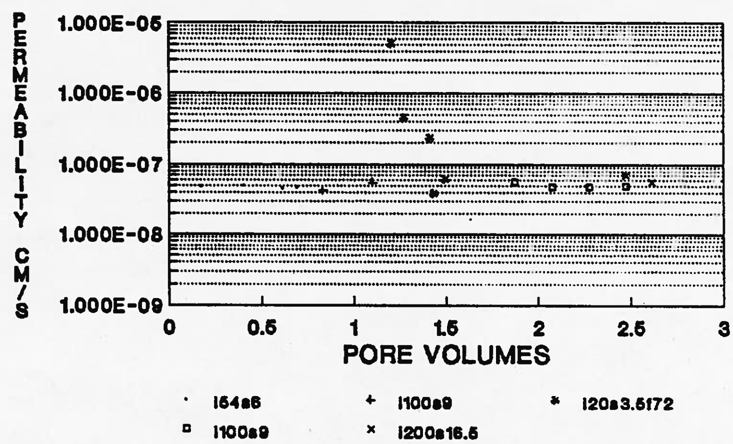
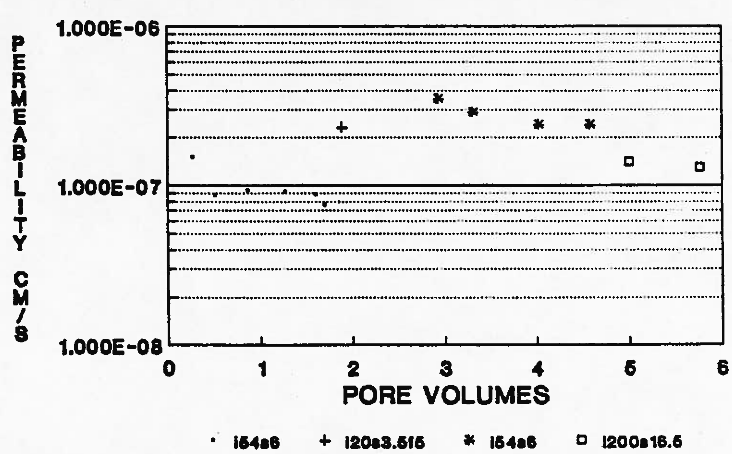


Figure 3- Effect of Freeze/Thaw on Permeability of Wetzel County Soil



(i- Hydr. Grad.;s- Effective Stress)

ENVIRONMENTAL DRILLING: THE CRITICAL PHASE FOR GEOENVIRONMENTAL CONSULTANTS

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ABSTRACT

The drilling portion of any environmental project is a very important phase. Without quality drilling services performed in a timely manner, the data being analyzed by the consultant can be erroneous, misleading and could possibly result in a professional liability exposure for the consultant. The drilling phase can be properly executed within a reasonable budget and time frame only if preplanning of the project scope and knowledge of the drilling requirements are considered.

Planning and scope preparation are important aspects to achieving a quality performance. Site geology, potential contaminates and site history are critical items to be initially considered. These items control the type of drilling such as soil test borings, rock coring and/or monitoring well installation and permits needed. They also control the site health and safety plan including medical monitoring requirements, protection levels of personnel, equipment and field control during the execution of the drilling phase. Consideration must also be given to decontamination of tools and equipment, waste containment and disposal. Also, the proper collection and disposal of the decontamination waste is an important consideration.

If monitoring wells are to be installed on site the material selection, screen location and suspect contaminates are all very important considerations. The well materials selected can impact the drill tool size and installation time on a project. The drilling methods must also be determined during the planning phase.

Safety of personnel and equipment during the execution of the drilling phase is the most critical item of the entire project. Contractual agreements covering the drilling phase should address limits of liability, indemnification from the spread of the contaminate from the drilling process, payment terms, and reporting confidentiality.

INTRODUCTION

I need a maximum not to exceed price for a confidential site somewhere within the Ohio River Valley for the installation of 25 monitoring wells. We need you to start as soon you can while you are filing for a permit. The client is in a hurry, so you probably need to plan on working 12 hours per day. We will, of course, execute our standard agreement whenever we get around to it which requires listing us as additional insured and the indemnification holding us and the client harmless in all cases.

This appears to be another successful geoenvironmental project getting started in a very loose fashion. Without proper planning and preparation to begin the drilling phase for a geoenvironmental project, there is little hope of the project being successful with regard to obtaining adequate information without causing financial difficulties to the drilling concern, jeopardizing the health and safety of the personnel and compromising the quality of the samples for analytical testing to be used by the geoenvironmental consultant for his analysis and conclusions.

Environmental drilling is even more exacting than the already high standards we place on geotechnical drilling. Drilling personnel and equipment used for years in the conventional geotechnical investigation phases are the most logical choice to perform this critical phase for the geoenvironmental projects. During the field investigative program, all aspects of the drilling, monitoring well installation and development must be performed to the best of everyone's abilities so that accurate information is obtained. Many times the decisions made by the geoenvironmental consultant are based on parts per million or parts per billion in the samples obtained after the drilling phase of the project. Any possibility of cross hole contamination or introduction of foreign contaminants to the samples can result in inaccurate information and possible liability for the geoenvironmental consultant.

Proper planning and communication with the people responsible and actually performing the drilling services prior to the start of a geoenvironmental project is ultimately important so that the probability of success during this phase produces reliable information for analysis. This planning and communication phase is important when using an outside subcontract environmental drilling company, but is also equally important when the drilling functions are performed by an internal drilling unit. The major objective of the drilling phase is to collect the needed information as accurately as possible while protecting personnel during the performance of field drilling phase and hoping that a reasonable profit is obtained from the drilling services. The environmental drilling function should be considered as a key ingredient in the planning and pricing for a geoenvironmental project since without success in the environmental drilling phase there is little hope for success in the overall project.

In the remaining areas of this manuscript, different phases of the environmental drilling services are discussed relative to planning and communications that must be performed to allow the project the potential for success. These areas including project background, scope of services and contractual requirements necessitate significant input prior to the start of a project.

PROJECT BACKGROUND

It is important that the drilling personnel know the site location which may control the certifications of the people or license of the company to perform the monitoring well installation. In addition, permits are usually required on a state by state basis and the information and time that it takes to acquire a permit may control the beginning date and criteria for installation of the monitoring wells.

The site surface conditions are equally important in establishing the proper means to execute the project with the adequate tools and equipment required for the project. A site reconnaissance can reveal whether clearing is required for drill rig access or whether the terrain requires an all-terrain vehicle (ATV). It is also important to know if the monitoring wells or services will be performed on the property owned by the client or if off-site well locations are planned requiring special access, permission, or possibly security. The site reconnaissance should also include information on available potable water sources for decontamination and drilling in sufficient quantities. On site utilities must also be located prior to initiating the drilling phase to prevent their damage.

Knowledge of the site geology is very important as it relates to the selection of equipment, tools and installation procedures. Other critical information, if available, is the probable depth to groundwater or the aquifer that may have been affected by the possible contaminants.

Information relative to the potential contaminants that may have impacted the soil or groundwater at the project site is ultimately important. Only with this information can proper planning with regard to the type of monitoring wells, material selection and the development of a specific health and safety plan be adequately performed.

SCOPE OF SERVICES

Groundwater monitoring well permit requirements vary from state to state, but are generally a requirement prior to the installation of groundwater monitoring wells. To obtain a permit, many times the number and location of the monitoring wells have to be established on a site plan and the procedures to be utilized during the installation determined for submittal to the State and/or Local regulatory bodies for their approval. The permits should be obtained in the name of the current property owner since liability issues of the monitoring wells could be a post-construction problem if obtained in the drilling or geoenvironmental consultants name. In addition, the required personnel certifications for installation of the monitoring wells are dictated by the individual states or governing bodies. Most often, the actual driller has to be certified or licensed to perform the installation of the groundwater monitoring wells. Therefore, depending on the state in which the site is located, the drilling personnel may have to be identified during the initial portion of the project. On the permit application, generally the size of the monitoring wells have to be identified and the state requirements often dictate the minimum size bore hole that must be drilled for proper installation of the monitoring wells.

The characteristics of the probable contaminant is very important when selecting the type of monitoring wells, materials and the installation procedures to be utilized during the drilling phase. The knowledge of whether the contaminants are "floaters" or "sinkers" definitely control the scope of services required including screen placement, the need for rock coring and/or rock monitoring wells, and the location of the monitoring wells on the site. The contaminants present can also dictate the type of safety equipment required, and the safety level under which the work would be executed and the work schedule of the drilling crew.

If the project scope is to include special sampling, it is important to know during the planning phase the type of sampling required. Many times undisturbed samples are obtained from either soft or hard materials to establish engineering properties such as permeability for modeling purposes. Also, typically standard penetration samples are obtained as the bore holes are advanced to better understand the subsurface stratigraphy and for possible analytical testing. On some geoenvironmental projects, geophysical testing in the bore holes are performed which can control the bore hole size and method of drilling. If the contaminants are "sinkers", then some projects require rock coring and possible packer testing to determine flow characteristics in the rock formations.

The materials to be utilized during the construction of the monitoring wells can significantly impact the drilling budget and installation time. Important considerations include the well diameter, material type, and wall thickness which control the availability and definitely the price. The anticipated screen lengths, opening sizes and the requirement for tail pieces are also factors that impact the drilling phase. The requirements of the filter materials, such as gradation and installation are needed. The materials to form the seal above the screen, placement thickness and set time are all factors that must be known during the planning phase. The grouting procedures and material requirements above the seal are items which can affect the set up time and quality of the finished monitoring well. Many times the monitoring wells are covered with either a flush mount manhole if installed within pavement or high traffic areas, with stick-up type covers generally provided in non-traffic areas. The need for bollards or protective barriers must be specified and included within the cost of the drilling phase. ID plaques and locks on the monitoring wells are also usually required.

After the majority of the information listed above is obtained, the selection of the drilling equipment can be performed. The drilling equipment is vitally important to the success of the job and is dictated by the method required for the installation of the monitoring wells. A reprint from the "Groundwater Age," February, 1989 edition is included as Appendix A which discusses the pros and cons of four different drilling procedures. Obviously, the equipment, drill tool size and pump capacity are all a function of the many variables listed above. Some things that must be considered are the accessibility to the monitoring well locations (by truck or ATV), or if monitoring wells are required in low clearance areas or within buildings. After the tools and equipment are chosen, then a more accurate volume of materials for well installation can be obtained, as well as the characteristics of the solid or liquid waste generated during the drilling phase and the number of crew members required.

The development of the completed monitoring wells is an important facet which is sometimes done by drilling personnel and sometimes by others. In any case, the acceptance criteria for the development of the monitoring wells must be established to include temperature, pH, turbidity, the number of well volumes to be extracted or other acceptance criteria determined. The development method such as pumping or bailing must also be known.

Many times during the drilling phase of a groundwater monitoring well project, the waste disposal requirements are overlooked. On many projects the auger cuttings and wash water add up to a fairly significant volume of contaminated material that must be handled, stored or disposed of properly. The auger cuttings generated from hollow stem augers sometimes must be dewatered and may require disposal on site, drummed in 55-gallon drums capable of shipping by others or be placed in roll-off boxes for later disposal. The wash water, or liquid obtained during the drilling process, monitoring well development, or decontamination of the tools may require special handling such as placement in drums or into a liquid tanker if they can not be discharged on the ground.

The scope of services should outline the decontamination requirements that would be performed during the drilling phase of the project. A major cost factor might be the availability of an adequate water supply for the decontamination process. In addition, a suitable decontamination location must be chosen to prevent difficulties with on-site operations or site neighbors. The requirements of a special decontamination pit and containment of the waste water are important time and cost factors.

The health and safety of the employees working during the drilling phase of the project should be a prime concern. The safety of the employees should never be compromised for a cost or installation time savings. The health and safety plan should be developed prior to starting the drilling phase and must be site and contamination characteristics specific. The health and safety plan should outline specifically the safety and monitoring equipment as well as the personnel training and monitoring that would be required during the execution of the drilling phase. This plan should specify if OSHA 40-hour health and safety training for all personnel is needed and if medical monitoring of the personnel such as pre and post medical physicals are a project requirement. The health and safety plan should also include requirements for heat sickness and work schedules that should be implemented to prevent this problem while working in safety equipment. The health and safety plan should also require a pre-construction safety meeting prior to the initiation of any field work.

CONTRACT REQUIREMENTS

The pricing for the drilling phase of geoenvironmental projects is normally done on hourly or footage prices. with unit prices for various contingencies. Very rarely are projects done on lump sum or maximum not to exceed basis since so many variables impact the ultimate cost. The lump sum, low price bid very rarely results in a successful project for the geoenvironmental consultant or the drilling entity. Without proper financial incentives, quality services may not be obtained during the drilling phase.

The contract requirement should include some idea of the anticipated work schedule and time frame for the execution of the services. Often the installation time for monitoring wells is hard to predict because of the many variables. The work schedule is a very significant factor in pricing projects and must be established during the planning and pricing phase.

The drilling entity must know the limits of insurance required or if a performance bond is to be obtained. Most often, bonds can be difficult to obtain within the needed time. The payment terms should be specifically spelled out and adhered to for the project. Many drilling entities have been abused in this area since the time for the analytical testing, data reduction and report preparation extend significantly after the completion of the drilling phase. The contract should include phased billing such as monthly invoicing, or invoicing upon the completion of the field services so that proper compensation in a timely manner is achieved.

As a drill rig owner, several other terms and conditions in the contract phase are important including indemnification from the owner or engineer for the spread of contaminates to other aquifers during the drilling process or from third party suits. Provisions for lost tools or equipment during the drilling phase, special security provisions for personnel and equipment, and the confidentiality requirements for the project must also be specifically addressed in the contract.

CONCLUSION

The drilling phase of a geoenvironmental project when properly planned and managed can be a successful phase of the project producing the quality information needed by the consultant in a safe manner. The conclusions and recommendations reached by the geoenvironmental consultant from the field data are only as good as the quality of the data obtained from the field. The drilling phase is the beginning of a successful project and is an important aspect of the services offered by the geoenvironmental consultant.

DOWN THE HOLE

Drilling Methods for Monitoring Wells

HOLLOW-STEM AUGER

ADVANTAGES	DISADVANTAGES
• No drilling fluid is used, eliminating contamination by additives • Formation water can be sampled during drilling by using a screened auger or advancing a well point ahead of the augers • Formation samples taken by split-spoon or core-barrel methods are highly accurate • Natural gamma-ray logging can be done inside the augers • Hole caving can be overcome by setting the screen and casing before the augers are removed • Fast • Rigs are highly mobile and can reach most drilling sites • Usually less expensive than rotary or cable tool drilling	• Can be used only in unconsolidated materials • Limited to depths of 100 to 150 ft (30.5 to 45.7 m) • Possible problems in contolling heaving sands • May not be able to run a complete suite of geophysical logs

DIRECT ROTARY

ADVANTAGES	DISADVANTAGES
• Can be used in both unconsolidated and consolidated formations • Core samples can be collected • A complete suite of geophysical logs can be obtained in the open hole • Casing is not required during drilling • Many options for well construction • Fast • Smaller rigs can reach most drilling sites	• Drilling fluid is required and contaminants are circulated with the fluid • Drilling fluid mixes with the formation water and invades the formation and is sometimes difficult to remove • Bentonitic fluids may absorb metals and may interfere with other parameters • Organic fluids may interfere with bacterial analyses and/or organic-related parameters • During drilling, no information can be obtained on the location of the water table and only limited information on water-producing zones • Formulation samples may not be accurate

AIR ROTARY

ADVANTAGES	DISADVANTAGES
• No water-based drilling fluid used, eliminating contamination additive • Can be used in both unconsolidated and consolidated formations • Capable of drilling to any depth • Formation sampling is excellent in hard, dry formations • Formation water blown out of the hole makes it possible to determine when the first water-bearing zone is encountered • Field analysis of water blown from the hole can provide information regarding changes for some basic water-quality parameters such as chlorides • Fast	• Casing is required to keep the hole open when drilling in soft, caving formations below the water hole • When more than one water-bearing zone is encountered and hydrostatic pressures are different, flow between zones occurs during the time drilling is being completed and before the borehole can be cased and grouted properly • Relatively more expensive than other methods • May not be economical for small jobs

CABLE TOOL

ADVANTAGES	DISADVANTAGES
• Only small amounts of drilling fluid are required (generally water with no additives) • Can be used in both unconsolidated and consolidated formations; well suited for extremely permeable formations • Can drill to depths required for most monitoring wells • Highly representative formation samples can be obtained by an experienced driller • Changes in water level can be observed • Permeabilities for different zones can be determined by skilled drillers • A good seal between casing and formation is virtually assured if flush-jointed casing is used • Rigs can reach most drilling sites • Relatively inexpensive	• Minimum casing size is 4 in (102 mm) • Steel casing must be used • Cannot run a complete suite of geophysical logs • Usually a screen must be set before a water sample can be taken • Slow

Adapted from the Australian Drillers Guide, available from S.A. Smith Consulting Services, PO Box 88, Ada, OH 45810

FEBRUARY 1989 GROUND WATER AGE

APPENDIX A

COMPUTER-AIDED ASSESSMENT OF CONTAMINATED SITES

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ABSTRACT

A computer workstation dedicated to characterizing and assessing remedial actions at uncontrolled hazardous waste sites has been developed. The IBM-PC compatible system is composed of several off-the-shelf software and hardware modules, with software development limited to the creation of utility programs used to transfer data from one software module to another. The component modules include a Geographic Information System, a Database Management System, a Computer-Aided Design and Drafting System, a Contouring System, a Volume and Mass Calculation System, and a Groundwater Modeling System.

The computer system is intended to produce maps and cross-sections of the geology, hydrology, and distribution of contaminants from data attained at boreholes. It is capable of calculating volumes or masses of contaminated material, as well as modeling groundwater flow and contaminant transport.

These capabilities have been implemented and tested during case studies of contaminated sites. The case studies include several Superfund sites and emergency response sites throughout the United States.

INTRODUCTION

Data from hazardous waste sites is often collected by contractors and turned into tables buried in thick reports. The site managers are overloaded with voluminous laboratory analyses of contaminants, boring logs, monitoring well logs, and survey instrument readings. There is a need to organize and manage this data, and present it in easily understood graphic form.

Beginning in 1986, a project was initiated at the University of Cincinnati to address this problem. The project was centered on the concept that inexpensive, easy-to-use IBM-PC type computer equipment and readily available commercial and public domain software could be molded into a system useful for waste site characterization. The project was funded by the U.S. Environmental Protection Agency, Office of Research and Development, Risk Reduction Engineering Laboratory, in Cincinnati, Ohio. Work is based at the U.S. EPA Center Hill Solid and Hazardous Waste Research Facility, in Cincinnati. The Computer Assisted Site Evaluation (CASE) project has moved from the developmental stage to the implementation stage, and is now able to provide services to U.S. EPA regional offices, other government agencies, and contractors.

The system developed and evolved in response to the needs of actual waste sites. The specific software packages and hardware used have changed as new capabilities have become available. This paper describes the current status of the project, and presents examples of work done thus far using the CASE system.

SYSTEM DESCRIPTION

The system hardware is based on IBM-PC compatible 20 MHz 386 and 12 MHz 286 computers. These machines are equipped with large hard disks, with capacities of 100 megabytes or more. Core memory capacities range from two to four megabytes of RAM. Standard VGA color graphics monitors are used for display.

Two digitizing tablets are available for input. A 12" by 18" tablet is used for command templates, freehand drawing, and digitizing small maps. A larger 36" by 48" digitizer tablet is used for entering large maps.

Output devices include a 24" by 36" multi-pen drafting plotter, a laser printer, a wide dot matrix printer, and a screen camera.

System software is composed of several types of commercial and public domain packages linked together with file conversion utilities, as illustrated in Figure 1. The linkages are built in to some of the packages, such as the DXF conversion that permits the contouring package to transfer files to the Computer Aided Design and Drafting (CADD) package. In other cases, small utility programs were developed in the Turbo Pascal language to facilitate data transfer.

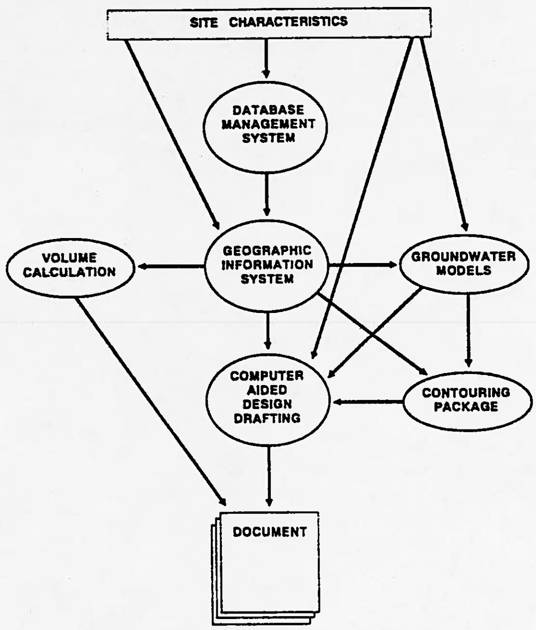

Figure 1. Information flow through the CASE system.

Site characteristics such as those resulting from soil and water analyses or water levels are entered into a database tailored for site characterization, created in the Borland REFLEX Database Management System (DBMS). This DBMS, though not as powerful as top-of-the-line packages, is quick and easy to use. Since it is interactive, rather than programmed and compiled, data files can be extracted from the database, graphs and tables can be created, and modifications can be made to the record structure by means of point-and-click operation.

The Geographic Information System (GIS), Spatial Information Systems pMAP, serves as the core of the CASE system. pMAP is a cell based GIS that is easy to use and well suited for small site studies. It provides an extensive command set that permits precise control of map manipulation. Repetitive operations are facilitated through use of macros. The GIS is used to create a three-dimensional model of the site hydrogeology. Cross-sections showing the geology and hydrology are extracted from this model and ported to the CADD for rendering. The GIS is also used to create contaminant maps, where a spectrum of color hatching indicates the level of contamination, and black stipple indicates areas where good data are not available.

Several contouring packages are available for use with the system. Most often we use Golden Software SURFER to make conventional contours, then port the file to the CADD for annotation and final rendering. Radian CPS/PC is used for more elaborate diagrams, and EPA GEO-EAS is used when kriging is required.

For calculation of volume or mass of contaminated material, a Turbo Pascal program was developed to work with the GIS to sum up the volume of an isopach. SURFER and CPS/PC are also capable of volume calculation.

Groundwater flow and contaminant transport can be modeled using any of a number of public domain and commercial packages, depending upon the site requirements. Althouth the CASE project lacks the resources to do a comprehensive modeling job, simple analytical models and preliminary numerical models are within our scope. For these, the U.S. Geological Survey MODFLOW numerical modeling system is most often used.

All of the graphical output from the CASE system goes to a CADD system for final production. Three-dimensional drawings are rendered in the Computervision Personal Designer CADD system. Generic Software Generic CAD Level Three is often used for two-dimensional drawings because of its simplicity. Both these systems are currently being superseded by Autodesk AutoCAD 386 version 10.

EXAMPLES OF OUTPUT

Figure 2 is an example of a contaminant concentration contour map. Information about the site is stored in data layers, similar to transparent overlays. The finished map uses a selection of layers. In this case the layers containing roads, rivers, sample locations, and log

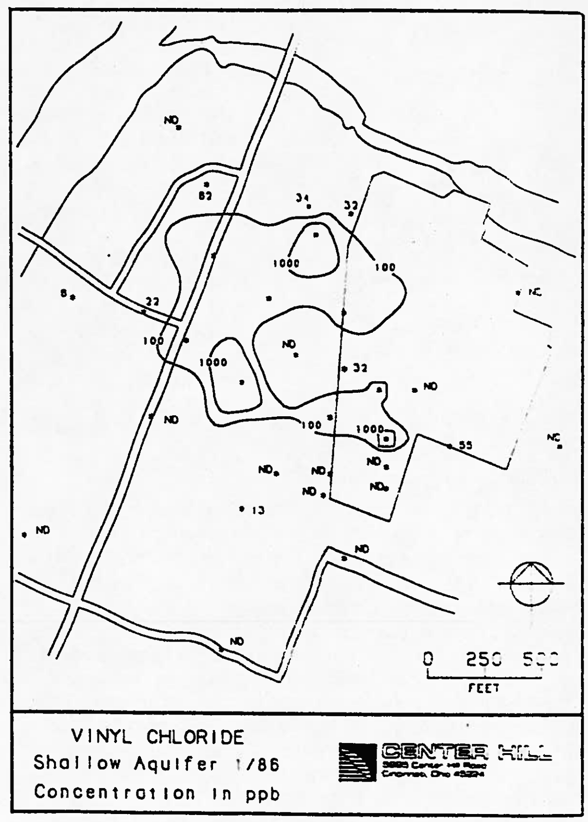

Figure 2. Contaminant concentration contour map.

scale contours of vinyl chloride concentration are utilized. Figure 3 uses some of the same layers as Figure 2, however this map shows the change in the plume of vinyl chloride from 1983 to 1986. Similar maps of tetrachloroethene were made for this site, to document the theory that a bioconversion process was generating vinyl chloride from tetrachloroethene.

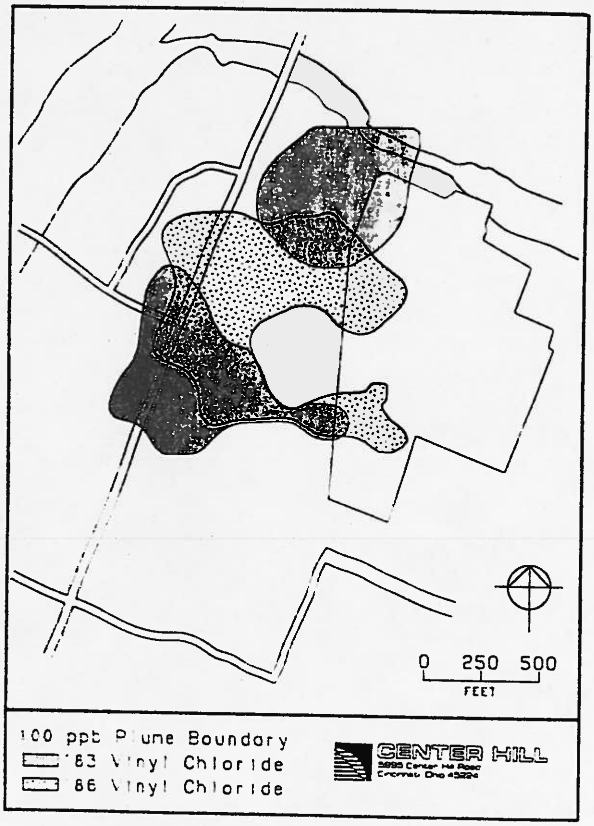

Figure 3. Plume evolution map.

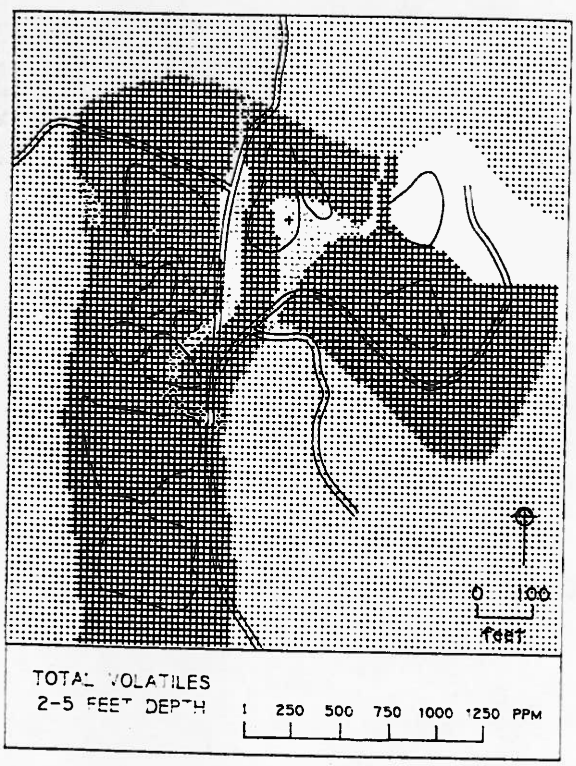

Figure 4. Color coded contaminant map.

Figure 4 is an example of a different type of contaminant concentration map, where a spectrum of five colors of cross-hatching indicate the level of contamination. Since the colors do not show in a black and white printing process, this type of map has limited application. An important feature of this map is the stipple pattern, which indicates an area that has been masked off because of lack of confidence in extrapolation to these areas.

Another type of color map is shown in Figure 5. A log scale histogram has been superimposed on an isometric view of a site map to indicate the concentration at each sample location. This type of map is especially suitable for sites with isolated lagoons, where it may be inappropriate to interpolate the contaminant concentration between lagoons.

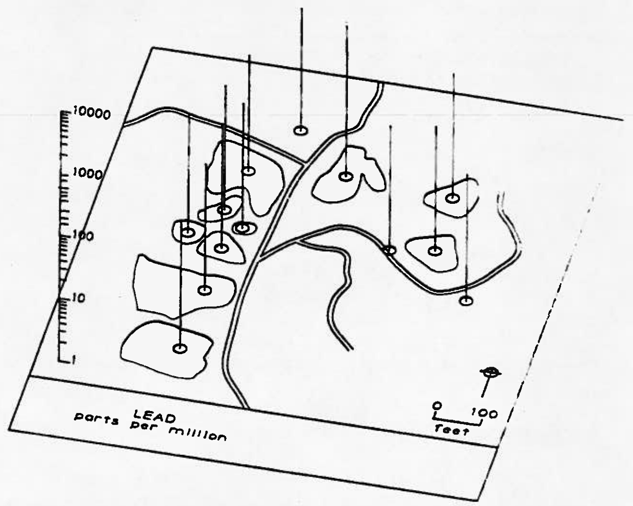

Figure 5. Color coded contaminant histogram keyed to site layout.

A hydrogeologic cross section is shown in Figure 6. Once a site model has been created in the GIS, cross sections can be taken along any traverse through the site, or can go from well to well as in this figure. These cross sections can be used to develop a conceptual model of the groundwater flow, as a prelude to numerical modeling. An earlier paper (Harrar, 1990) describes this process in detail.

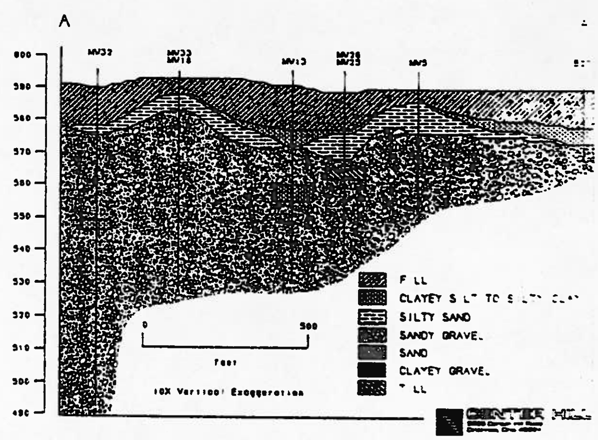

Figure 6. Cross-section of stratigraphy from well to well.

FUTURE PLANS

In addition to providing the services described above, in the future the CASE project plans to develop the capability to provide several new services.

A site information system for use in the field could be developed, where a user would point to a location on a site map, and the system would respond with a cross-section of the expected stratigraphy. This would aid in the placement of new boreholes.

Wellhead protection is becoming a concern of many local governments. A database could be designed to organize data collection, and then the CASE system could extract the necessary data to model flow and delineate wellhead protection zones.

Large complex industrial sites, which have many hazardous waste problem areas, are difficult to manage because of the volume of data from many sources. A comprehensive site mapping and database system could be set up using the ESRI Arc/INFO GIS, to organize all the site information.

SUMMARY

A set of computer hardware and software tools and a staff of experts have been assembled into a system dedicated to aid in the waste site characterization process. Since the system is based on IBM-PC compatible equipment, it is lower in cost and easier to use than typical mainframe and minicomputer applications. The system has been successfully applied to several waste sites, and is currently able to supply services on a regular basis.

REFERENCES

Harrar, W.G. et. al. Computer-Based Methods of Assessing Contaminated Sites: A Case History. Proceedings of the 15th Annual Research Symposium. U.S. Environmental Protection Agency, Cincinnati, Ohio. EPA/600/9-90/006. 1990. pp. 175-185.

INCREASED PERMEABILITY OF SOILS BY HYDRAULIC FRACTURING: A FIELD TEST

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ABSTRACT

Hydraulic fracturing was used to create permeable layers in low permeability soil formations. Flat-lying hydraulic fractures made with standard injection grouting equipment were filled with a coarse quartz sand mixture at depths of a few meters in overconsolidated silty clay glacial drift. Fractures created 1 to 1.5 m below ground surface reached maximum dimensions of 5 to 8 m and a maximum thickness of 20 mm. As many as four fractures, vertically spaced at 15 to 30 cm, were generated from the same borehole. The fractures increased the rate of inflow into unsaturated silty clay by factors between 3.2 and 9.1. This method may improve the performance of in-situ remediation techniques such as vapor extraction, pump and treat, soil flushing and bio-remediation.

INTRODUCTION

Recovering hazardous chemicals from contaminated ground is often difficult and sometimes impossible using established techniques, so earth scientists have turned to related fields for innovative ideas. The problem of recovering hydrocarbons from oil reservoirs, for example, is similar to the problem of recovering contaminants from aquifers. Petroleum engineers have developed a wide range of techniques to enhance the recovery of oil from reservoirs, and one of the most effective is hydraulic fracturing. This research was motivated by the possibility of using hydraulic fractures to improve the remediation of contaminated ground.

Hydraulic fracturing, as it is used in the petroleum industry, begins with the injection of fluid into a well until the pressure of the fluid exceeds a critical value and a fracture is nucleated. A granular material, usually sand, is pumped into the fracture as it grows away from the well. Transport of this *proppant* is facilitated by using a viscous fluid, usually a gel formed from guar gum and water, to carry the proppant grains into the fracture. After pumping, the proppant holds the fracture open while the viscous gel breaks down into a thin fluid. The thinned gel is then pumped out of the fracture, creating a permeable channelway suitable for either the delivery or recovery of liquid or vapor.

Techniques of hydraulic fracturing have been used for more than fifty years to increase the yields of oil wells, and their effectiveness at great depths in rock is without question. The feasibility of creating sand-filled hydraulic fractures at shallow depths in soil (conditions of contaminated sites), however, has received little attention compared with the work in the energy industry. Since 1987, the USEPA has funded a coordinated program of laboratory, theoretical and field investigations into the use of hydraulic fractures to improve remediation (Murdoch and others, 1990). The research described in the following paper includes the second field test of this USEPA-funded program.

Initial field testing of the program was conducted in June, 1988, at a site 10 km north of Cincinnati, Ohio, USA. The site was underlain by overconsolidated Pleistocene glacial drift, which consists of silty clay and local beds of sand and gravel. Hydraulic fractures were created by using oil-field equipment to inject fluid into cased borings between 2 and 4 m deep. The vicinity of each borehole was excavated and the features of fractures were mapped in detail (Murdoch, 1989). The fractures were flat-lying to gently-dipping and they were as much as 10 m in maximum dimension.

Field observations during the first test indicated that it was feasible to create hydraulic fractures, but they also highlighted three shortcomings: 1) sand proppant was sparse or absent in seven out of ten fractures, so those fractures closed completely and would have had negligible effect on flow in the subsurface; 2) the maximum dimensions of the fractures were limited by venting to the ground surface; 3) the test was conducted using sophisticated equipment that would be inaccessible to most environmental engineers.

The method of creating hydraulic fractures was modified to improve upon the shortcomings of the 1988 tests. The new method was tested in the field during June and July, 1989. Details of that method and the results of the 1989 field tests are described in the following pages.

METHOD OF HYDRAULIC FRACTURING

Methods of creating hydraulic fractures typically consist of three components:

1.) Injection Fluid: A viscous fluid capable of both suspending proppant during injection and of breaking down to be recovered after injection.

2.) Above ground: mixers, tanks, and pumps to prepare and inject proppant-laden fluid.

3.) Below ground: equipment to isolate an interval of the borehole and nucleate a fracture.

The injection fluid used during this project is a gel formed from commercially-available guar gum and water mixed at a concentration of roughly 3.6 gm/l. The viscosity of the basic gel is roughly 20 centipoise, but it increases markedly upon addition of a borate compound, termed a *crosslinker*, at a concentration of 0.24 gm/l. The crosslinked gel is a thixotropic fluid with an apparent viscosity of roughly 200 centipoise. An enzyme is added that breaks down the gel to roughly 10 centipoise between 12 and 18 hrs after injection. The enzyme is added concurrently with the crosslinker at a concentration of 0.12 gm/l. Coarse quartz sand (0.8 to 1.5 mm average grain size) is mixed with the crosslinked gel to complete the formulation of the injection fluid. Concentrations of sand ranged to as much as 0.52 (vol sand/vol gel) during the field tests.

The gel-crosslinker-breaker fluid used for this work is similar to fluids used during hydraulic fracturing operations in the oil-industry. The above- and below-ground components of the fracturing process used here, however, differ from those used in the oil industry.

The above ground system (Fig. 1) consists of a tank for storing gel, two paddle mixers, and a progressive cavity pump (Robbins and Meyers Moyno 2J6 CDQ pump). The gel is mixed and hydrated in the tank prior to fracturing. During fracturing, gel is pumped into one mixer, the crosslinker and enzyme are added, and sand is mixed with the crosslinked gel. The resulting sand-laden slurry is emptied into the intake of the pump and injected into a fracture forming in the subsurface. Injection rates ranged between 20 and 60 l/min during the tests.

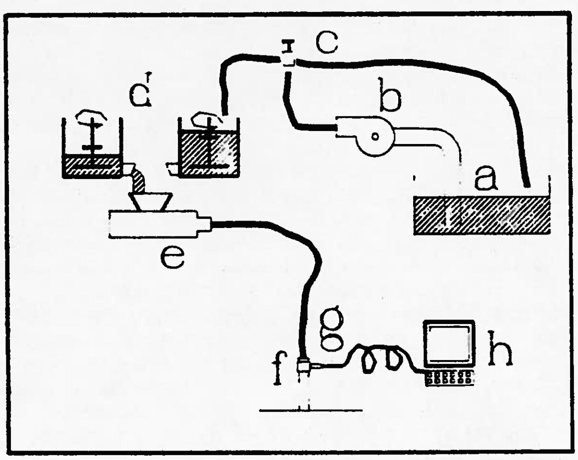

Figure 1. Schematic of above-ground equipment used to create hydraulic fractures during 1989 tests. a. Mixing tank; b. Circulation pump; c. Valve; d. Blender; e. Injection pump; f. Borehole/lance; g. Transducer; h. Data acquisition.

Below ground, a system of isolating an interval of a borehole was developed specifically for use in unlithified material. The system is based around a lance-like device (Fig. 2), composed of a casing and an inner rod, both of which are tipped at one end with hardened cutting surfaces that form a conical point. A drive head at the other end of the lance secures both the casing and the rod. Individual segments of the rod and casing are 1.5 m long and they are threaded together as required by borehole depth.

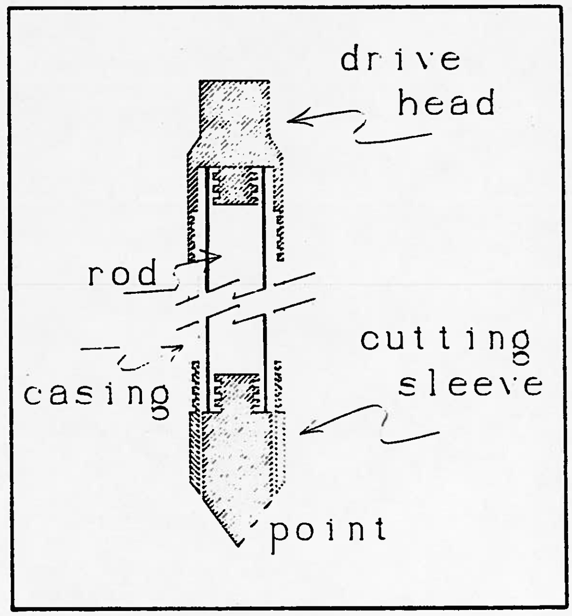

Figure 2. Lance-like device used during hydraulic fracturing.

During the 1989 tests, the lance was driven one to two dm below the bottom of a borehole (the boreholoe was either open or it contained a hollow stem auger). (Fig. 3). The rod and point were removed, leaving soil exposed at the bottom of the casing. Another device, composed of steel tubing with a narrow (0.025 cm diam.) orifice at one end, was inserted into the casing. Water pumped into the device (at 20 l/min and 17 MPa) formed a jet that cut laterally into the soil. The jetting device was rotated, producing a disk-shaped notch extending up to 40 cm away from the borehole (Fig. 3). A simple measuring apparatus, built from a steel tape extending the length of a tube and making a right angle bend at the end of the tube, was inserted into the casing to verify and measure the radius of the slot.

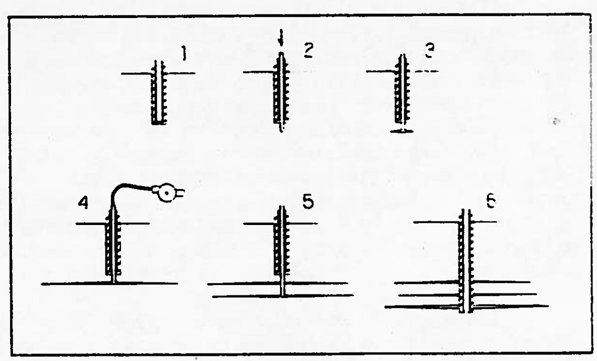

Figure 3. Hydraulic fracturing procedure. 1. Hollow-stem auger; 2. Drive lance; 3. Cut notch; 4. Inject slurry to create hydraulic fracture; 5. Advance lance; 6. Advance auger.

Hydraulic fractures were created by injecting the sand-laden slurry into the casing. Lateral pressure of the soil on the outer wall of the casing effectively sealed the casing and prevented leakage of the slurry. The fractures nucleated at the notch and grew away from the borehole.

During a typical field test, the onset of pumping was marked by a sharp increase in pressure of the injection fluid to between 0.1 and 0.4 MPa. The onset of fracture propagation, however, was marked by an abrupt decrease in pressure. During propagation, injection pressures either were roughly constant and on the order of 0.06 MPa, or they decreased slightly with time (e.g. Murdoch and others, 1990). Those pressures were for fractures 1.5 to 2.0 m deep, and pressures were slightly greater for other fractures 3 to 4 m deep.

After one fracture was created, the rod and point were inserted and the lance driven 7 to 30 cm lower, where another fracture was formed. This procedure was repeated as many a four times for each borehole.(Fig. 3).

FIELD TESTING

The method of hydraulic fracturing described above was field tested in June 1989 with the creation of 23 fractures at two sites in Cincinnati, Ohio. Nineteen of those fractures were created adjacent to the ELDA Landfill, within 100 m from the site of the 1988 tests (Murdoch, 1989). Tests at the ELDA site were located on a bench 10 to 15 m wide bounded to the southwest by a 5 m tall, steep ascent, and to the northeast by a 10 m deep descent. Fractures were created in unsaturated silty-clay at depths ranging from 0.9 to 1.9 m. The vicinity of each borehole was excavated and the fractures were mapped in detail.

RESULTS

Hydraulic fractures exposed by excavation at the ELDA site were remarkably similar in form. Three fractures created at borehole EL6 are a typical example (Fig. 4). The fractures are essentially horizontal and equant to slightly elongate in plan. They are highly asymmetric with respect to the borehole, however, with a preferred direction of propagation roughly parallel to the slope of the overlying ground surface. The fractures are stacked one on top of another at a spacing of 30 cm, which is maintained from borehole to leading edge (Fig. 5). The major plan axes of the fractures range from 5.5 to 8.5 m, and the maximum thickness of sand is 1.3 to 1.4 cm. The sand proppant is thickest near the centers of the fractures, and it thins as the edges are approached (Murdoch and others, 1990). Apparently, the distribution of sand is independent of the location of the borehole.

Inflow tests were conducted prior to excavation using a Guelph permeameter (Elrich and others, 1988), a device that yields the flow rate required to hold a constant water level in a borehole. Water levels were held at 1.0 m above the bottom of open boreholes during all tests. The average inflow rate into three boreholes in unfractured ground is 0.055 l/min. The rate of inflow into boreholes intersecting hydraulic fractures was initially 0.25 to 2.5 l/min, but decreased to between 0.175 and 0.5 l/min at steady state. We conclude that the steady-state rate of inflow increased by a factor between 3.2 and 9.1 as a result of the creation of the fractures.

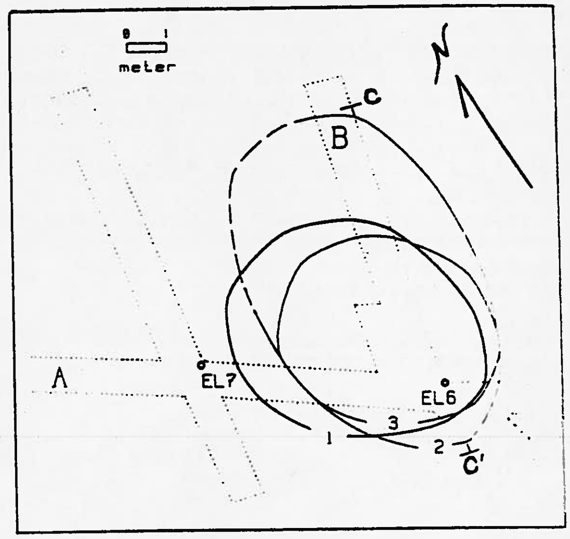

Figure 4. Map of three fractures created at borehole EL6. Dotted line is the wall of a trench. Dashed line where fracture 2 intersects a neighboring fracture (not shown) created from borehole EL7.

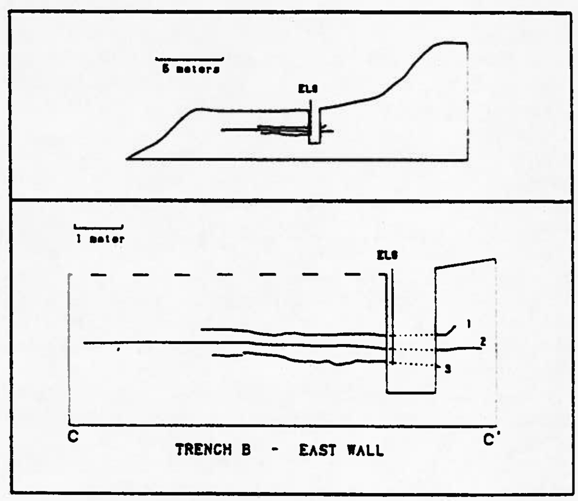

Figure 5. Cross-sections of three fractures created at borehole EL6, showing topographic profile (upper) and details of fracture traces (lower).

DISCUSSION

The principal shortcomings of the initial (1988) field tests were reduced using the method of fracturing described above. All 19 of the excavated fractures were filled with sand proppant, and they averaged 11.2 mm in maximum thickness. The fracturing method thus appears to be a consistent means of creating permeable layers in the subsurface, at least for the field conditions of this test.

Hydraulic fractures created during the 1988 tests climbed gently toward the ground surface where they vented, whereas the ones created during the 1989 tests were essentially horizontal and typically did not vent. There were at least three aspects of the 1989 fracturing procedure that could have inhibited the tendency of hydraulic fractures to vent:

1. Pumping rate was reduced -- from 75 to 420 l/min in 1988 to 20 l/min in 1989.

2. Density of injection fluid was increased by increasing the sand content -- 0.09 to 0.18 during 1988 to as much as 0.52 during 1989. The density increase reduces buoyancy effects.

3. Radius of the notch was increased. Vertical fractures nucleated at the wellbores during 1988, but the larger notch caused horizontal fractures to nucleate during the 1989 tests.

The mixers and pump, key components of the above-ground fracturing equipment, were rented from a geotechnical equipment company, who designed them to be used for injection grouting. Similar equipment should be widely available and accessible to most environmental engineers. Although propped fractures were consistently created with this equipment, it was far from ideal. The mixers and pump were underpowered for this application, and the rotor and stator suffered excessive wear and had to be replaced at the end of the tests. Mixing sand and gel using the continuous batch system was labor intensive and tedious. Slight modifications in the design of the pump and mixer should improve the efficiency of the field procedure.

The below-ground equipment performed adequately, facilitating the creation of multiple fractures from a single borehole. As many as four flat-lying fractures were stacked at spacings of 30 cm without intersecting their neighbors. When fractures were created at a spacing of 15 cm, the lower fracture would commonly climb and intersect the overlying fracture several m from the borehole. One spacing of 7 cm was attempted: the lower fracture intersected the upper one several dm from the borehole.

It has long been recognized that the orientation of a hydraulic fracture depends largely on the in situ state of stress, with the plane of the fracture normal to the direction of least principal compression. That direction is vertical in shallow bedrock and overconsolidated soil, which explains the horizontal orientations of fractures at the ELDA site. The orientation of hydraulic fractures at other sites, such as sites underlain by normally-consolidated soil or fill where the direction of least principal compression is expected to be vertical, may differ markedly from the orientation of fractures described here.

The results of this study indicate that it is feasible to create multiple, sand-filled hydraulic fractures in glacial sediments. The fractures increase the steady-state flow rate into a well by as much as nearly an order of magnitude. Similar results at a contaminated site could lead to important improvements during remediation.

This research is currently funded by the USEPA in their innovative technology demonstration program. We are actively seeking candidate sites where hydraulic fracturing can be used to augment traditional methods of remediation. Conditions that will favor the application of hydraulic fracturing techniques, in their current state of development, are as follows:

1. Contaminated material is either bedrock, till, or some other overconsolidated sediment. The state of stress in those materials should favor the creation of horizontal fractures, such as those described above.

2. In situ remedial methods (e.g. vapor extraction, pump and treat, bio-remediation, soil flushing, etc.) could be feasible if fluid flow in the subsurface could be increased.

ACKNOWLEDGEMENTS

We appreciate the support and encouragement of Herb Pahren, Mike Roulier, and Don Sanning of the USEPA. The field study was facilitated by John Stark and the management of ELDA landfill, who granted us permission to use their property. Contributions from Elizabeth Spencer, Nan Genther and our colleagues at the Center Hill Research Facility have been invaluable.

DISCLAIMER

Although the research described in this manuscript has been funded wholly or in part by the United States Environmental Protection Agency contract #68-03-3379-08 to the University of Cincinnati, it has not been subject to the Agency's review and therefore does not necessarily reflect the views of the Agency, and no official endorsement should be inferred.

REFERENCES

Elrich, D.E., W.D. Reynolds, and K.A. Tan. A new analysis of the constant head well permeameter technique. Proceedings of the Conference on Validation of Flow and Transport Models for the Unsaturated Zone, New Mexico, May 23-26, 1988.

Murdoch, L.C. A field test of hydraulic fracturing in glacial till. EPA/600/9-90/006, Proceedings of the USEPA 15th Annual Research Symposium, Cincinnati, Ohio, pp. 164-174, 1990.

Murdoch, L.C., G. Losonsky, P. Cluxton, B. Patterson, I. Klich, B. Braswell. The feasibility of hydraulically fracturing soil to improve remedial actions. Final Report USEPA contract #68-03-3379-08, 1990 (in review).

INTERPRETATION OF FIELD PERMEABILITY TEST RESULTS ON FULL SCALE LINER SYSTEMS

John A. Mundell[1] and Timothy A. Boos[2]

ABSTRACT

The evaluations of the as-compacted hydraulic conductivity of two full scale clay liner systems using the sealed double-ring infiltrometer (SDRI) are presented. Infiltration rate measurements, soil tensiometers, dye-tracing techniques, and measurements of post-test soil moisture contents were used to estimate the position of the wetting front to aid in calculating vertical hydraulic gradients and effective vertical saturated hydraulic conductivities of the liners at various times throughout the test. Laboratory permeability testing of remolded clay samples were used to establish the SDRI test liner compaction conditions to meet a specified minimum conductivity requirement. Undisturbed samples from the as-compacted liners were tested in the laboratory to provide a basis of comparison to field-determined conductivity values. The results showed that field-determined hydraulic conductivity values varied by less than one-half order of magnitude depending on the methods utilized and the assumptions made. Tensiometer measurements were shown to overpredict the position of the wetting front and the value of calculated hydraulic conductivity. Laboratory-determined hydraulic conductivities from the undisturbed liner samples slightly exceeded the range of SDRI testing results, providing a conservative estimate of the as-compacted conductivities of the liners.

INTRODUCTION

As the demands required to limit the impact of stored solid and hazardous wastes on the environment have increased over the last decade, field permeability testing methods to determine the hydraulic conductivity of compacted clay liners has received considerable attention. Laboratory testing of small diameter, undisturbed samples on several liner systems has been shown in many instances to underestimate the hydraulic conductivity of the actual earthen liners in comparison to field testing performed on much larger areas [1-3]. This has been attributed to the ability of the field tests to measure a larger, more representative volume of material and include the effect that secondary features such as fissures, macropores, and slickensides have on increasing the effective hydraulic conductivity.

A number of in-situ testing methods have been used over the years to estimate the field hydraulic conductivity of natural soils. These have been broadly grouped into borehole, porous probe, infiltrometer, and underdrain tests, and the relative advantages and disadvantages of each has been previously discussed [4].

This paper describes the application of one of the infiltration methods - the use of the sealed double-ring infiltrometer (SDRI) - to assess the as-compacted hydraulic conductivity of two full-scale test fills. The impact of the variation in the interpretation of the measurements on the estimated conductivities is discussed. Comparisons are also made between the field and laboratory-determined conductivity values.

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TABLE 1
LABORATORY PHYSICAL PROPERTIES OF CLAYS

Soil Characteristic [a]	Test Fill 1 Gray Silty Clay: No. of Tests	Mean	Standard Deviation	Range	Test Fill 2 Reddish Brown Silty Clay: No. of Tests	Mean	Standard Deviation	Range
Natural Moisture Content (%)	42	20.8	5.4	10.7 - 34.3	3	18.2	0.4	17.9 - 18.7
Atterberg Limits	24				3			
Liquid Limit (%)		22.3	6.0	15.4 - 37.5		19.8	3.4	17.1 - 24.6
Plasticity Index (%)		40.6	8.8	30.0 - 59.4		40.0	7.9	33.0 - 51.3
Grain Size Distribution [b]	20				6			
Clay (%)		38.0	8.0	25.0 - 49.0		25.5	11.5	19.0 - 43.0
Silt (%)		39.2	8.5	15.0 - 53.0		44.0	6.5	33.0 - 51.0
Sand (%)		22.6	12.1	4.0 - 40.0		30.0	8.5	10.0 - 43.0
Moisture Density Relationship [c]	5				5			
Maximum Dry Density (pcf)		109.1	5.0	104.2 - 114.8		107.0	1.2	105.2 - 108.1
Optimum Moisture Content (%)		15.8	2.1	12.5 - 17.8		16.2	3.0	11.5 - 19.3

[a] ASTM Test Procedures for all Characteristic Determinations
[b] Defined as in ASTM D 422
[c] Standard Proctor ASTM D 698

SEALED DOUBLE-RING INFILTROMETER (SDRI)

The SDRI testing method evolved from the earlier standard describing double-ring infiltration testing (ASTM D 3385). Because of the equipment utilized, this standard method could not be reliably used to determine hydraulic conductivities of low permeability soils. First described by Daniel and Trautwein [5-7], the SDRI was developed to accurately measure very low infiltration rates indicative of clay liner systems so that hydraulic conductivities on the order of less than 10^{-7} cm/sec could be determined.

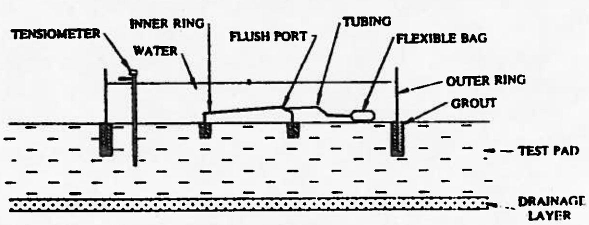

Figure 1. Schematic of the SDRI System

The SDRI system used in the two full scale tests was manufactured by the Trautwein Soil Testing Equipment Company [7] and consists of two rings: a 12 ft square aluminum outer ring and a 5 ft square sealed fiberglass inner ring (Figure 1). The rings are embedded into bentonite grout-filled trenches dug into the surface of the liner system, the outer ring to a depth of about 18 in, the inner ring to a depth of about 5 in centered within the outer ring.

When both rings are filled with water, the smaller thickness inner ring is submerged and separated from the outer ring water by its sealed cover. The outer ring water is left open to the atmosphere. The measurement of infiltration through the inner ring is made by the connection of a flexible bag filled with a known weight of water to a port on the inner ring. Weighing the loss of fluid from the bag at periodic time intervals rather than measuring a drop of elevation in the water level of the rings, as has been the practice in other ring infiltrometer methods, allows for improved resolution and accuracy of low flow measurements.

By sealing the inner ring and using a protective cover over both rings, the influences of evaporation and temperature fluctuations are also minimized. The use of the larger outer ring also provides improved confidence in the assumption of one-dimensional flow immediately beneath the inner ring.

A limited number of field tests [5,8-9] has yielded SDRI-measured hydraulic conductivity values about one order of magnitude greater than laboratory determined values. Unpublished data by Trautwein [4] on more than a dozen case histories show SDRI-measured conductivities to vary between one to ten times laboratory-measured values. Recent results on a test pad liner in Jamestown, California [10], however, showed good agreement between laboratory and SDRI-determined values.

LABORATORY PROGRAM

In order to develop stringent construction quality controls for the selection, preparation, and monitoring of appropriate soils at each site for the SDRI tests, laboratory testing programs were undertaken to fully characterize the soil properties. The test results for the soils selected as clay liner material at the two sites are presented in Table 1.

At the Test Fill 1 site, a northern Indiana glacial till clay was selected. The clay in its undisturbed natural state was described as a stiff to very stiff, mottled gray silty clay with a U.S.C.S. classification of CL. At the Test Fill 2 site, a southern Indiana residual clay was used as the liner material. The residual clay was described as a very stiff, reddish brown silty clay also with a U.S.C.S. classification of CL.

During borrow pit selection, several samples of the available clay soils were subjected to fixed-wall permeameter and triaxial permeability testing to evaluate the effectiveness of the soils as a landfill cover barrier layer. Samples were remolded and compacted in standard Proctor and Harvard miniature molds at densities varying from 90 to 110 percent of their maximum dry density (ASTM D 698) wet of the optimum moisture content. The relationships established between hydraulic conductivity and dry density were used to select the dry density and moisture ranges that would result in hydraulic conductivities that would meet the minimum U.S. EPA requirement of less than 1×10^{-7} cm/s for landfill cover barrier layers.

Construction specifications developed from these laboratory permeability tests required dry densities above 95 percent of the standard Proctor maximum and moisture conditions between 2 to 5 percent wet of the optimum moisture content for the Test Fill 1 site. Specifications for the Test Fill 2 site required the dry density of the soil to exceed 100 percent of the standard Proctor maximum, with moisture contents wet of optimum. If these conditions could be satisfied in the field, the testing results indicated that hydraulic conductivities of less than 5×10^{-8} cm/sec were possible at each test location.

TEST FILL CONSTRUCTION

The two test fill areas were prepared by removing all topsoil and vegetation and placing a one foot thick coarse sand layer at the base of the test clay liners to define the lower boundary condition for the SDRI test [5]. The clay soils in each case were placed in successive 6 to 8 in (15 to 20 cm) loose lifts and compacted with a sheepsfoot roller. A 79,000 lb Caterpillar 835 self-propelled compactor was used on Test Fill 1. At the Test Fill 2 location, a pull-behind type vibrating sheepsfoot with a rated dynamic force of approximately 15,000 lbs was used to compact the soil.

The completed Test Fill 1 surface area measured about 30 ft by 30 ft (9 m x 9 m) and the thickness approximately 30 in (0.8 m). The liner was constructed in five lifts. Test fill 2 was constructed to an area of about 30 ft by 45 ft (9 m x 14 m) and a thickness of approximately 39 in (1 m). Six lifts were necessary to achieve the final desired thickness. Field moisture and density tests were performed on each lift to monitor adherence to the compaction condition specifications. After completion of the compaction of the top lift, the clay was covered with black plastic to prevent desiccation.

The as-compacted condition of each test fill, as determined from field measurements, indicated that the moisture and density requirements were met or exceeded at all test locations. Test Fills 1 and 2 were compacted to mean dry densities of 98.2 and 101.7 percent of the standard Proctor maximum, respectively. Fixed-wall permeameter and triaxial permeability tests were performed on undisturbed Shelby-tube samples taken from the as-compacted test fills. Mean hydraulic conductivities of 5.2 x 10^{-8} and 5.7 x 10^{-8} cm/sec were determined for Test Fills 1 and 2, respectively.

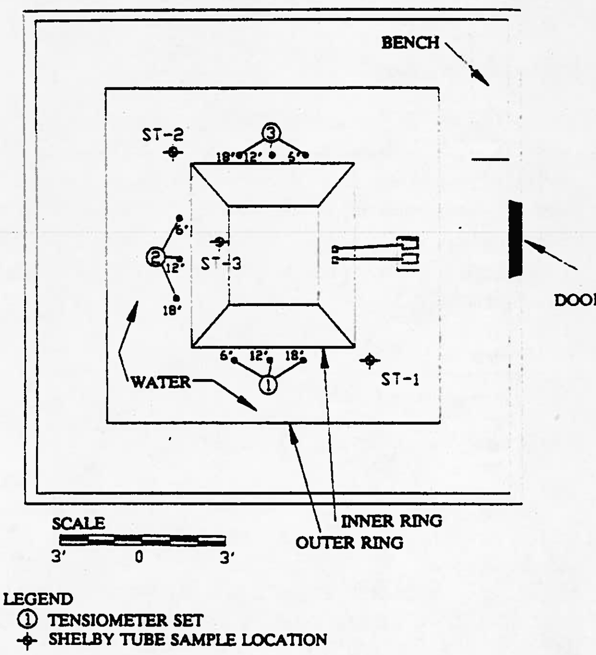

Figure 2. SDRI Set Up - Test Fill 1

SDRI INSTALLATION

The SDRI test apparatus set-up used at Test Fill 1 is shown in Figure 2. A similar arrangement was also utilized at Test Fill 2. A trenching machine was used to excavate a 4 in (10 cm) wide by 18 in (46 cm) deep trench for the outer ring placement. The outer ring was placed, leveled, and sealed with a Volclay grout. The 2 in (5 cm) wide by 5 in (13 cm) deep trench for the inner ring was hand excavated with a brick hammer, the ring positioned, leveled, and grouted into place. Complete details of the assembly and installation procedures of the SDRI may be obtained from the manufacturer [7].

As a means of providing an indication of the movement of the wetting front beneath the rings during the test, three sets of soil tensiometers were placed at depths of 6, 12, and 18 in (15, 30, 45 cm) below the surface of the liner at the locations shown on Figure 2. The tensiometers were installed by drilling a vertical pilot hole to about one-half the tensiometer depth. Once the tensiometer casing was lowered through the hole into place, another smaller diameter hole was advanced beyond the casing to the desired depth and the tensiometer lowered and sealed into place.

Protective wooden-framed housings were constructed for each SDRI installation. At Test Fill 1, the housing air temperature was controlled by a thermostat set to 80 ° F. For the Test Fill 2 location, two portable electric submersible water heaters controlled by a thermostat were used to maintain the actual water temperature at about 60 ° F. Constant ponded water levels in each SDRI were maintained by manual refilling when required.

INFILTRATION RATES

The infiltration rate of the inner ring at test duration time t, I_{ir_t}, is defined as the volume of infiltrating water per unit soil surface area per unit time, and is determined from the test measurements as the total change in the volume of water measured from the flexible bag, ΔV_{t_i}, during a specific time interval, Δt_i, per unit soil surface area of the inner ring, A_{ir}, or

$$I_{ir_t} = \frac{\Delta V_{t_i}}{\Delta t_i A_{ir}} \cdots\cdots (1)$$

where $\Delta V_{t_i} = V_{t_i} - V_{t_{i-1}}, \Delta t_i = t_i - t_{i-1}$.

Measured infiltration rates versus time for the inner ring for the two test fills are shown in Figure 3. During the 143 day and 161 day test durations for Test Fills 1 and 2, respectively, both tests showed initially higher infiltration rates in the 10^{-6} cm/sec range steadily decreasing at a higher rate within the first 10 to 20 days and then more slowly after that. Observed fluctuations in infiltration rates were most likely caused by changes in the viscosity and density of the water due to temperature fluctuations, barometric pressure variations, or small differences in the manner of weighing the flexible bag by the field technicians that occurred during the long duration of testing.

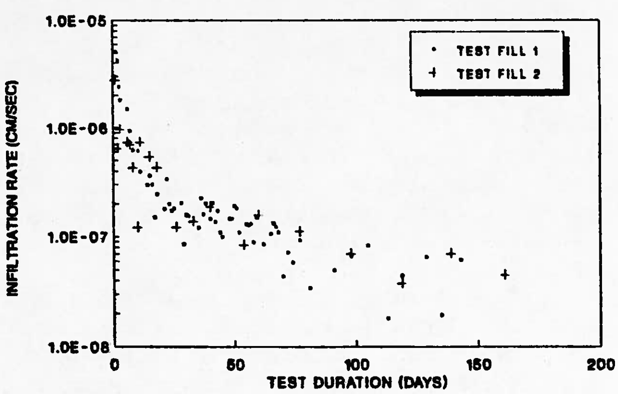

Figure 3. Infiltration Rate Versus Time

Cumulative infiltration through the inner ring, I_{irc_t}, was determined as the total change in the volume of water measured from the flexible bag at test duration time t per unit area of the inner ring soil surface area, or

$$I_{irc_t} = \frac{\sum_{i=1}^{m} \Delta V_{t_i}}{A_{ir}} \cdots\cdots (2)$$

where $m\Delta t_i = t$.

The cumulative infiltration was calculated to be 1.6 cm and 2.2 cm for Test Fills 1 and 2, respectively.

WETTING FRONT POSITION

Several methods were employed during the SDRI tests to estimate the position of the wetting front. A theoretical approach based on the sharp wetting-front Green-Ampt model [11] assumes that as infiltration occurs, the soil profile is uniformly wetted to a constant, saturated, final volumetric moisture content, θ_f, above the wetting front, and at the front a step-like change in moisture content back to the initial constant volumetric soil moisture content, θ_i, occurs. Using this approximation, the wetting front position, L_{f_t}, below the inner ring after an elapsed test duration time t, may be determined as

$$L_{f_t} = I_{irc_t}/\Delta\theta \cdots\cdots (3)$$

where $\Delta\theta = \theta_f - \theta_i$. If complete saturation of the soil is assumed above the wetting front [12], then the final volumetric moisture content can be taken to be equal to the porosity, n, of the compacted soil, or

$$\theta_f = \eta = 1-(\rho_b/\rho_s) \cdots\cdots (4)$$

where ρ_b is the bulk dry density of the soil and ρ_s is the mean soil solid density. The mean dry density and moisture content of the as-compacted fills determined from the field testing programs were utilized in the porosity value and wetting front calculations.

The estimated theoretical wetting front position during the SDRI test on Test Fill 1 based on these equations is shown in Figure 4. Wetting front locations at the end of each test were calculated by the Green-Ampt model to be 9.5 and 14.1 cm for Test Fills 1 and 2, respectively.

Measured soil suction head values from the three sets of tensiometers were also used to calculate the wetting front location. It was assumed that as the wetting front arrived at the depth of a tensiometer, the soil suction head value recorded by the tensiometer would be zero. Interpretation of the averaged readings from the various depths of the tensiometer sets at Test Fill 1 results in the wetting front prediction shown in Figure 4. Constant velocity movement of the front was assumed between the tensiometer depths. From the tensiometer readings, the wetting front positions at the end of the tests were estimated to be at 45.7 and 43.7 cm for Test Fills 1 and 2, respectively.

Rhodamine dye was initially added to the outer SDRI ring at the Test Fill 2 location to determine the

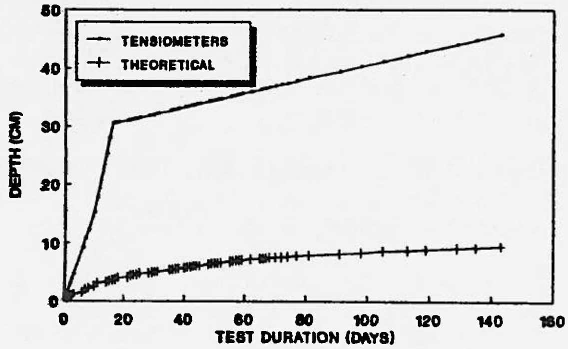

Figure 4. Wetting Front Position Versus Time for Test Fill 1

wetting front location at the end of the test. A sharp dye front was observed at a depth of about 16.0 cm at 161 days.

The results of soil moisture content measurements from undisturbed Shelby tube samples taken at the end of the tests were used to estimate the final position of the wetting front. Moisture variations indicate wetting front positions at 7.6 and 15.2 cm for Test Fills 1 and 2, respectively.

HYDRAULIC CONDUCTIVITY

The apparent saturated vertical hydraulic conductivities (k_s) of the two clay liners during the SDRI tests were calculated from the infiltration data and the estimates of the wetting front position using both the theoretical Green-Ampt model and the tensiometer readings previously discussed. Conductivities were calculated using the following equation:

$$k_{s_t} = \frac{I_{ir_t}}{i_t} = I_{ir_t}\left[1 + \frac{H + \psi_f}{L_{f_t}}\right]^{-1} \qquad (5)$$

where i_t is the hydraulic gradient at test duration time t, H is the depth of the ponded water on the surface of the liner, and ψ_f is the wetting-front suction head, taken as zero in the absence of direct measurements (this is a conservative assumption that results in smaller calculated hydraulic gradients and thus larger hydraulic conductivities).

The calculated hydraulic conductivities for Test Fills 1 and 2 are shown in Figure 5. Initially higher infiltration rates in the first 10 to 20 days resulted in conductivities about one order of magnitude higher in the early stages of the tests than calculated near the end of the tests. In each case, the use of the tensiometer readings resulted in estimated conductivity values greater than the theoretical predictions.

Test Fill 1 exhibited hydraulic conductivities near the end of the test in the range of 8 x 10^{-9} cm/sec using the Green-Ampt approximation, and 3 x 10^{-8} cm/sec based on the tensiometer data. Hydraulic conductivities for Test Fill 2 near the end of the test were calculated to be about 2 x 10^{-8} and 4 x 10^{-8} cm/sec using the Green-Ampt model and tensiometer data, respectively.

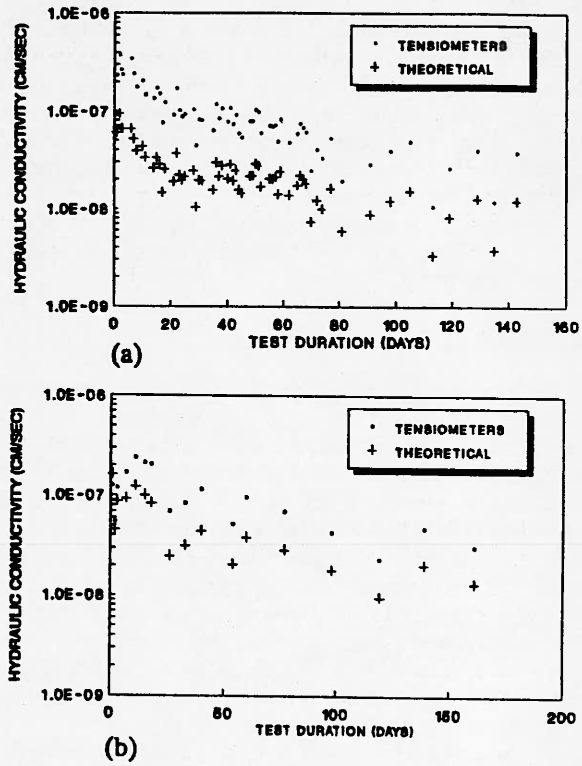

Figure 5. Hydraulic Conductivity Versus Time for (a) Test Fill 1 and (b) Test Fill 2

DISCUSSION OF RESULTS

Reductions in infiltration rates and thus hydraulic conductivity were observed during each SDRI test. Prior studies have also noted this phenomenon [5, 9-10] and have explained that it may result from higher initial soil suction [5] or swelling of the soils tested [10]. Inadequate preparation and protection of the surface of the liner prior to the initiation of the test could also result in surficial drying, lower initial moisture contents, and an apparent high conductivity. This behavior may also be produced by dry density variations within the upper lift (e.g. a lower dry density in the upper few inches) resulting from typical effective compactive effort differences within the liner system.

Despite the long duration of the two tests, the wetting front movement occurred only within the upper lift of the multi-lift liners. Similar flow behavior within the lower lifts may not be extrapolated from the test results.

A comparison of laboratory permeability test results on both remolded and as-compacted liner soils to the field-determined values indicates that the laboratory testing accurately predicted the observed field behavior of the liner systems. Hydraulic conductivities of less than 5 x 10^{-8} cm/sec were predicted and observed in the field. Test durations of greater than 10 to 20 days were necessary to reach this conclusion.

Good agreement in the predicted wetting front position was observed using the Green-Ampt model, post-test soil moisture content measurements, and the dye tracing observations. Although it has been suggested that the use of tensiometers may be desirable for measuring the wetting front position and hydraulic gradient above the front [6,7], this study suggests that the readings produced by carefully installed tensiometers consistently tend to overpredict the movement of the front position. New enhancements in tensiometer installation and sealing techniques are apparently required before reliable readings may be obtained.

CONCLUSIONS

The SDRI is currently the most reliable apparatus available to determine the as-compacted hydraulic conductivity of low permeability clays. Increased confidence in predictions may be gained by using multiple methods of tracking the wetting front as was done in the testing reported herein. It is apparent from the results of the two tests reported, however, that there are limits to its accuracy at low infiltration rates. Considerably more scatter in the data was observed at the lower infiltration rates observed near the end of the tests, most probably due to the greater influence of small variations in external factors such as temperature.

Although the SDRI has the advantage of testing larger areas that are presumably more horizontally representative of the actual liner than smaller laboratory samples, the wetting front during the test may only infiltrate the top few inches of the upper compacted liner lift even after long test time durations. As a result of this, the SDRI test does not examine the representative flow behavior of the entire liner system vertical cross section.

Finally, the results of this study indicate that contrary to past findings on some earlier liners, a comprehensive laboratory permeability testing program [13] on undisturbed liner samples taken from well-controlled, multiple-lift clay liners may yield as reliable on indicator of the as-compacted hydraulic conductivity as more cumbersome, more expensive, longer-term field tests. Additional well-documented case histories are required to further support this assertion.

REFERENCES

[1] Olson, R. E., and Daniel, D.E., "Measurement of the Hydraulic Conductivity of Fine-Grained Soils," ASTM STP 746, pp. 18-64, 1981.

[2] Daniel, D. E., "Predicting the Hydraulic Conductivity of Compacted Clay Liners," Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 2, pp. 285-300, February, 1984.

[3] Day, S. R., and Daniel, D. E., "Hydraulic Conductivity of Two Prototype Clay Liners," Journal of Geotechnical Engineering, Vol. 111, No. 8, pp. 957-970, August, 1985.

[4] Daniel, D. E., "In Situ Hydraulic Conductivity Tests for Compacted Clay," Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 9, pp. 1205-1226, September, 1989.

[5] Daniel, D. E., and Trautwein, S. J., "Field Permeability Test for Earthen Liners," Proc., In Situ '86, ASCE, Blacksburg, Virginia, pp. 146-160, June 23-25, 1986.

[6] Daniel, D. E., "Hydraulic Conductivity Tests for Clay Liners," Proc. Ninth Annual Symposium on Geotechnical and Geohydrological Aspects of Waste Management, Fort Collins, Colorado, February 1-6, 1987.

[7] Trautwein Soil Testing Equipment Company, Installation and Operating Instructions for the Sealed Double Ring Infiltrometer, Trautwein Soil Testing Equipment Company, Houston, Texas, 1987.

[8] Chen, H. W., and Yamamoto, L. O., "Permeability Tests for Hazardous Waste Management Unit Clay Liners," Geotechnical and Geohydrological Aspects of Waste Management, Proc. of the 9th Annual Symposium on Geo-Aspects of Waste Management, D.J.A., van Zyl et al., eds., Lewis Publishing, Inc., Chelsea, Michigan, pp. 229-243, 1987.

[9] Albrecht, K. A., and Cartwright, K., "Infiltration and Hydraulic Conductivity of a Compacted Earthen Liner," Groundwater, Vol. 27, No. 1, pp. 14-19, Jan.-Feb., 1989.

[10] Fernuik, N. and Haug, M., "Evaluation of In Situ Permeability Testing Methods," Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 2, pp. 297-311, February, 1990.

[11] Green, W. H., and Ampt, G. A., "Studies on Soil Physics Part I--The Flow of Air and Water Through Soils," Journal of Agricultural Science, Vol. 4, pp. 1-24, 1911.

[12] Goldman, L. J., Truesdale, R. S., Kingsbury, C. M., and Damle, A. S., "Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities," Draft Technical Resource Document, Office of Solid Waste and Emergency Response, U.S. EPA, Washington, D.C., EPA/530-5W-86-007, 1986.

[13] Mundell, J. A., and Bailey, B., "The Design and Testing of a Compacted Clay Barrier Layer to Limit Percolation Through Landfill Covers," Hydraulic Barriers in Soil and Rock, ASTM STP 874, A. I. Johnson, R. K. Frobe, J. J. Cavalli, and C. B. Pettersson, Eds., American Society for Testing and Materials, Philadelphia, pp. 246-262, 1985.

GEOTECHNICAL CONSIDERATIONS AT THE LAKE SANDY JO SUPERFUND SITE

David J. Lane[1]

ABSTRACT

The remedial action for the Lake Sandy Jo Superfund site near Gary, Indiana, included a soil cover for contaminated surface soils on the landfill, excavation of contaminated sediments from nearby ditches, and monitoring of site groundwater for possible contaminant migration. CH2M HILL was selected by the U.S. Environmental Protection Agency to design the remedial action and to provide technical assistance during construction, which was managed by the U.S. Army Corps of Engineers.

As one of the first Superfund sites to proceed through remedial design and construction, Lake Sandy Jo presented several challenges that could not be anticipated from work on similar, non-hazardous geotechnical/civil projects. Necessary considerations included ensuring the health and safety of workers, preventing the spread of contamination, and providing facilities readily operated and maintained by the state of Indiana. The issues faced and the solutions devised are discussed in this paper.

BACKGROUND

A former borrow pit and recreational lake in Gary, Indiana, Lake Sandy Jo was used as a landfill for construction and demolition debris during the 1970s. Reports of illegal hazardous waste dumping led the U.S. Environmental Protection Agency (EPA) to add the site to the Superfund list in 1982. CH2M HILL conducted a remedial investigation and feasibility study (RI/FS) to determine the extent of contamination and assess alternatives for remedial action; the subsequent record of decision recommended remediation because polyaromatic hydrocarbons (PAHs) in landfill soils and nearby ditch sediments and benzene in groundwater posed potential risks to human health in excess of the one-in-a-million criterion.

Because of the large volumes of material involved and the relatively low levels of contamination found, EPA selected a remedial action with an emphasis on preventing risk to the public and the environment by minimizing the potential for direct contact with contamination. CH2M HILL was selected to perform the remedial design, which included covering the landfill with clean soil, excavating ditch sediments and placing them on the landfill before covering, and monitoring groundwater for contaminant migration (use of onsite groundwater was restricted).

The design was reviewed by the U.S. Army Corps of Engineers (COE), who subsequently managed the construction while CH2M HILL provided technical assistance to EPA. Remediation was completed in the spring of 1990, and site operation and maintenance will be the responsibility of the Indiana Department of Environmental Management (IDEM). A plan view and section of the remediated landfill are shown in Figure 1.

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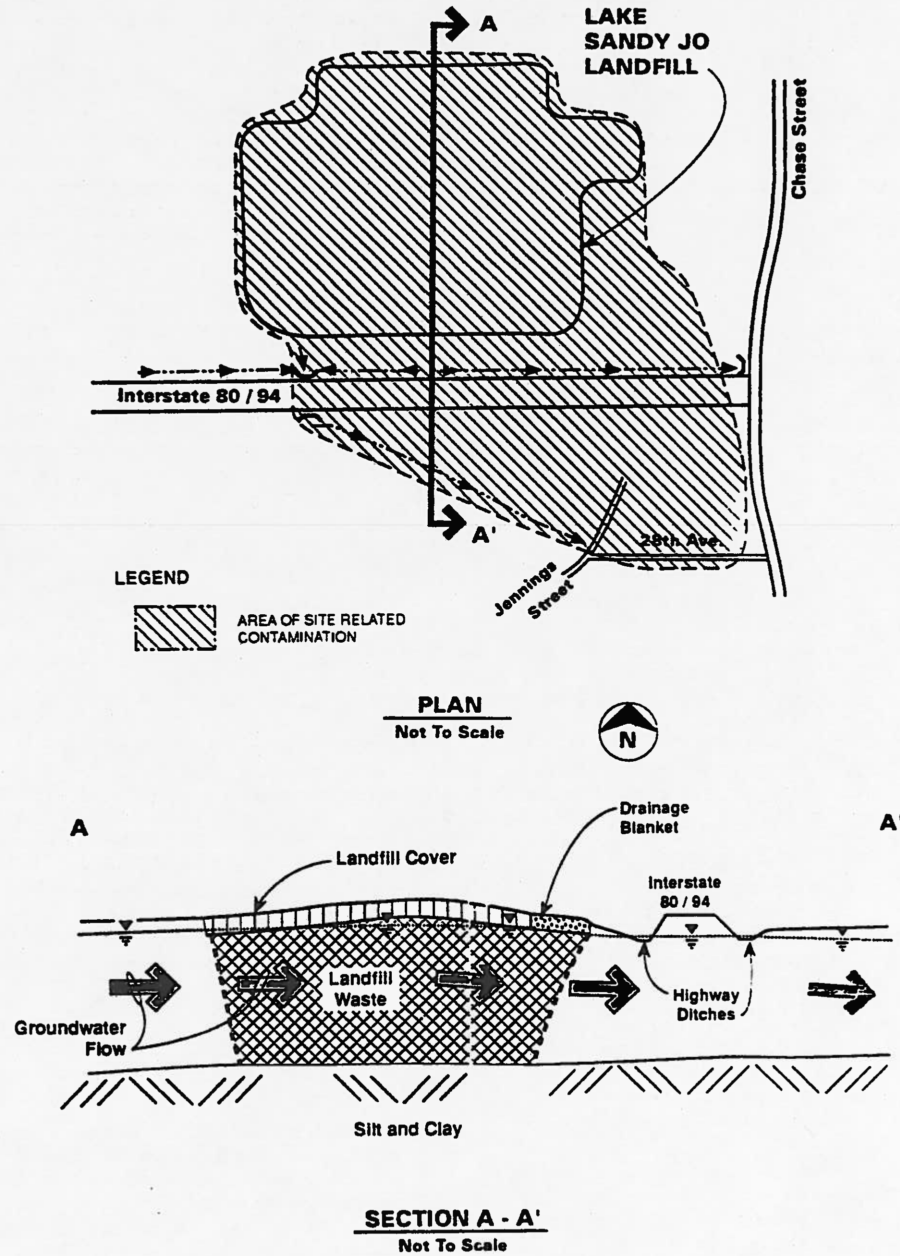

Figure 1. Plan view and illustrative section of remedial action at Lake Sandy Jo site.

Although the components of the remedial action were not particularly complex, their design and construction on a hazardous waste site involved many considerations atypical of a similar geotechnical/civil project. Lake Sandy Jo was also one of the first Superfund sites to achieve remediation, so little precedent was available for dealing with many of the situations that arose. This paper describes several of the aspects of the remedial design and construction that required special attention from the designer, contractor, and other parties involved. The views expressed in this paper are those of the author and not necessarily those of EPA.

HEALTH AND SAFETY

As would be expected, significant attention was given to ensuring that design and construction activities would not expose workers or the offsite environment to the potential contaminant pathways of ingestion, inhalation, or other direct contacts. This required consideration of many issues that are not typical factors in a design.

An early problem was finding surveyors who would certify that they would comply with the health and safety requirements of the Occupational Safety and Health Administration (OSHA) for working at a hazardous site. Of the three bids received, one would provide acceptable certification but proposed a price five times above the bid estimate, and another appeared unwilling to implement the program required by the regulations. The third bidder was acceptable both in certification and price but was still unable to provide a qualified site safety officer (SSO). As a result, a CH2M HILL SSO had to be assigned to the site throughout the 3 weeks of surveying.

Later, at the start of the construction phase, a similar problem occurred in selecting contractors. Although several national firms were able to meet the requirements for working on hazardous waste sites, public opinion in the underemployed, economically depressed area was strongly in favor of using a local firm. Unfortunately, none of those interested was health and safety qualified. The scarcity of qualified contractors also led to an exemption from the federal quota for small business involvement, which would have been very difficult to meet.

The specifications provided minimum health and safety requirements that, along with OSHA regulations, had to be addressed by the contractor in a Safety, Health, and Emergency Response Plan (SHERP). Topics included qualifications of personnel, work zones, training, medical surveillance, worker protection, monitoring, emergency response, and spill contingency. The SHERP and its program were to be implemented by a Certified Industrial Hygienist, and all site work was to be supervised by a full-time onsite SSO.

A major health and safety related requirement was for personnel protective equipment. The specifications required at a minimum Level D (work clothes) for such activities as monitor well installation, cover placement, and seeding; Modified Level D, which adds coveralls, gloves, and overboots, for staging area activities, clearing, and sediment excavation; and Level C (all the previous equipment plus an air purifying respirator) for grubbing and grading. Requirements could be more stringent depending on actual site conditions, so the SHERP had to explain the criteria the contractor would use to evaluate the risk level and determine if protection was adequate.

A primary method for evaluating worker exposure was through air monitoring, as migration of contaminated dust particles through the air during site activities was considered the most likely exposure route. The specifications required four perimeter stations set up to collect samples with air pumps and sampling tubes, filters, and appropriate media for analyses at an offsite laboratory. Also required were real-time measurements using photo or flame ionization detectors for volatile and semivolatile organic constituents and real-time aerosol monitors (RAMs) for total dust. Samples in the breathing zone were also taken for time-weighted average testing.

Baseline monitoring was required to establish background levels for a minimum of arsenic, cadmium, chromium, lead, and PAHs. The instruments were then to be operated continuously during intrusive activities. Testing only had to be performed on the time-weighted average samples once per week or when the RAMs recorded total dust above the acceptable level (based on toxicity and metal concentrations in site soils). An upwind and downwind sample also had to be tested each day to monitor potential offsite migration.

The contractor implemented air monitoring essentially as designed and reported no readings above background on any of the instruments.

SOIL COVER

STAGING AREA

The area within the site for contractor staging was very limited, but having to set up offsite had potential operational and cost disadvantages for the contractor. The specifications, therefore, provided three staging options that would be acceptable: (1) cover a portion of the site with clean fill to use for initial staging, then move to a remediated part of the site and clean up the first area; (2) use an offsite staging area initially, then move onsite when sufficient remediated area becomes available; or (3) use an offsite staging area for the entire job. The contractor selected the first option and implemented it without any difficulty.

CLEARING AND GRUBBING

Burning at a hazardous waste site was not considered publicly acceptable, although the risk of contaminant migration was low. The contractor was required to chip and spread clearing debris across the site before cover placement so that no area received more than 6 inches thickness. The intent was to prevent formation of a pile of material that would later decompose and cause surface settlement. Offsite burning at the contractor's convenience and risk was allowed, but the contractor chose to implement the chipping option.

COVER MATERIAL AND VEGETATION

The major design criterion for covering the landfill was to provide a minimum of 2 feet of material over all contaminated surface soils. The cover was not intended as a cap, so there were no infiltration or permeability criteria, and natural soil from commercial borrow pits offsite was tested and specified. To provide erosion control and an aesthetic appearance, an unusual choice was made for vegetating the cover--prairie grasses.

The prairie environment is native to the region and has recently gained public favor, although many of the natural systems have fallen victim to grain fields and pastures in the last 150 years. The perennial prairie grasses establish deep roots that help them withstand harsh conditions; again, because the landfill was not being capped, this was considered a benefit, not a detriment. A possible obstacle to successful propagation was the COE's decision not to allow burning for maintenance because of fire potential in the landfill; however, while fire is very helpful to prairie grass growth and weed control, maintenance through mowing was felt to be an adequate substitute that would also facilitate cover inspection.

Preparing a specification for prairie grass vegetation was challenging, as little local expertise could be found. Information was obtained from a nursery specializing in prairie vegetation, the Indiana Department of Highways (which had begun planting prairie grasses along highways), and the Indiana Department of Natural Resources Division of Nature Preserves. The result was a grassing type specification for prairie seeding, equipment, maintenance, and acceptance. A requirement was included that any deviation from the specification had to be evaluated by a prairie grass specialist.

The choice of this type of vegetation became an important issue during construction, when the contractor submitted a Value Engineering Cost Proposal (VECP) to substitute slag, which is abundant locally, for the specified soil fill. CH2M HILL's initial response to the verbal VECP was to note that slag was a waste material with potentially toxic properties that could impede or impair prairie grass growth, and designers concluded that evaluation of that potential would be costly and time consuming with no perceivable project benefits.

The contractor was not dissuaded and submitted a formal VECP that addressed some concerns but could not satisfactorily demonstrate that the slag would be compatible with the prairie grasses. In particular, the high pH and cementitious nature of the material were felt to be generally detrimental to plant growth, an opinion substantiated by several botanical specialists. While it was possible that some native grasses could survive with slag as a root medium, additional testing and redesign would be needed to ensure a workable prairie system. The potential for toxicity and long-term leaching would also require further investigation. Again, possible benefits of using slag were not outweighed by the potential risk of failure or the cost for continued evaluation, and the VECP was denied.

SUBDRAINAGE

A major reason for not capping the landfill was that prevention of vertical infiltration was not expected to have much effect on contaminant movement. This is because the groundwater table at the site is high and flow moves primarily in a horizontal direction through the landfill. The result is groundwater surfacing at the lower, southern end of the site, as evidenced by seeps and perennial wet areas and subsequent flow into the nearby ditch.

This natural seepage presented problems for construction of the soil cover and was also a potential pathway for contamination. A subdrain was designed using perforated pipes in gravel overlying filter cloth on natural soils to direct groundwater to two outfalls into the ditch. The system facilitates future collection and sampling of the near-surface groundwater.

SETTLEMENT

Because documented information on the contents of the landfill was scarce, calculations for predicting settlement after filling produced results ranging from zero to more than 20 feet, depending on the assumptions used. The initial response was to limit the amount of fill placed (and therefore the load on the landfill) by requiring a finished slope of only 0.5 percent, which was still expected to provide adequate drainage. During review, the COE expressed concern that settlement would lead to ponding and higher maintenance requirements for the cover, so the criterion for finished slope was increased to 1 percent.

After rough grading was performed during construction, the contractor's subgrade plan (before site fill) was compared to the existing topography and found to be significantly lower in several areas. The cause was attributed to greater densification and settlement during rough grading than had been expected. The result was an increase in the amount of required fill, which had been based on an estimate of the volume between the existing topography and the design final grade, minus 6 inches of topsoil.

The site fill quantity overrun was reduced somewhat because the original design slopes exceeded the 1 percent criterion in several areas. The grading plan was revised to reduce slopes to no more than 1 percent wherever possible and still achieve the final grade.

<u>SEDIMENT REMOVAL</u>

A major issue for sediment removal was determining the depth of excavation required so that material with higher than background levels of PAHs would not be left in the ditches. Instead of requiring confirmatory testing at each location, a method specification was developed for excavation to the depth PAHs were detected during remedial design sampling plus an additional 6 inches for a margin of safety. Because the depth of sediment was difficult to determine in some locations, a further requirement called for excavation to continue to native soil if the minimum depth shown on the plan did not result in complete removal of the sediment. This requirement was acceptable because the black, silty, organic sediments were easily distinguishable from the brown, silty sand native to the area.

The sediment excavated from the ditches was to be hauled to the landfill site for disposal, so the specifications had requirements for the contractor to either decontaminate hauling equipment or protect it from contamination, to prevent exposure along the route. Conditions were fairly dry, so a backhoe was able to excavate and deposit the sediments directly into a sealed-bed truck, and the operators were careful to keep sediment from contacting the truck exterior. Contamination was prevented along the access roads, and the truck was kept on clean fill at the site.

The contaminated sediment was dumped at the edge of the clean fill and pushed by a bulldozer to its final location. Both the bulldozer and the backhoe were dedicated to their respective tasks and did not require decontamination until the work was finished. Because of the efforts made during loading and unloading, the exteriors of the hauling trucks did not need decontamination; the beds were decontaminated when removal ended.

GROUNDWATER MONITORING

The record of decision recommended monitoring groundwater at the site for signs of contaminant migration, as earlier sampling had reported small quantities of benzene from a perimeter well and elevated metals offsite. To better define groundwater flow patterns and gradients before designing the long-term monitoring program, piezometers were installed during the design phase and measurements collected for a year. The results were used to optimize plans for installing the additional monitor wells.

Special considerations for monitor well design included decontamination requirements for installation of wells and to minimize cross-contamination. Offsite wells were located in a residential neighborhood, so work zone, health and safety, and decontamination requirements were specified to mitigate against potential contamination of the aquifer.

SPECIFICATION PREPARATION

CH2M HILL worked with COE to provide specifications that would support their construction methods. Technical sections were produced either from COE guidance specifications or from CH2M HILL sections adapted to the COE format. COE provided the invitation to bid and other legal sections.

To assist the contractor in developing the SHERP and other plans in which contamination was an issue, some information from the RI/FS was included as appropriate in the specifications and the reports were cited as sources of additional information. Appendixes to the specifications included available physical data and summaries of analytical data from field investigations.

CONCLUSION

Many of the issues encountered during this project were unique to a Superfund site, but were able to be resolved through the combined skills of hazardous waste specialists and civil and geotechnical design engineers. This cooperative effort, which involved participants from EPA, COE, IDEM, and CH2M HILL, successfully overcame the many obstacles that must be faced in remedial work on a hazardous waste site.

GEOTECHNICAL AND ENVIRONMENTAL CONSIDERATIONS FOR HIGHWAY CONSTRUCTION IN MOUNTAINOUS TERRAIN WITH ACID-PRODUCING BEDROCK

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ABSTRACT

A proposed highway relocation project in the mountainous area of southeastern Tennessee presents a unique combination of environmental and geotechnical considerations. The proposed route, located in a National Forest containing environmentally sensitive trout streams, will encounter intensely folded and fractured Precambrian age bedrock which contains zones rich in sulfide minerals, primarily pyrite. The sulfide minerals produce sulfuric acid upon weathering, and previous highway projects within this geologic setting have caused significant environmental degradation where acidic run-off was produced from fills constructed of pyritic rock. In addition, the rock units present a significant risk of slope instability and the design must contend with cut slopes in excess of 100 feet in depth. In one instance where cuts of over 130 feet in depth would be required within pyritic bedrock, a twin bore tunnel approximately 500 feet in length is planned.

This paper briefly reviews several previous highway projects which required post-construction environmental remediation and describes current efforts to anticipate and mitigate environmental concerns for a proposed highway project. Laboratory analysis of the bedrock core and geophysical techniques were employed in an effort to locate and characterize pyritic deposits. The study was complicated by rugged mountainous terrain and concerns related to disturbance of the National Forest. The final design must address many complex, interrelated geotechnical and environmental issues. For example, the selection of horizontal and vertical roadway alignments must take into account the desire to minimize the excavation and exposure of pyritic bedrock while also addressing the need for balancing cut and fill volumes, maintaining stable slopes and safeguarding the environment and forest resources during construction.

Although detailed geotechnical and geoenvironmental data are yet to be developed by further study, preliminary data indicate that the impact of sulfide minerals can be mitigated by thoughtful route selection and geotechnical practices that involve encapsulating the pyritic rock within specially designed fills.

INTRODUCTION

As new highways are designed to replace outdated roads which cross the southern Appalachian Mountains, steep, rugged topography, geologically complex conditions and geotechnically difficult situations are to be expected. In addition, the bedrock often contains certain assemblages of minerals, including iron sulfide (also known as pyrite or "fools gold"). Acid drainage can occur when sulfide minerals react with water and oxygen to produce sulfuric acid. Previous highway construction

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in this setting has sometimes been associated with environmental degradation as a result of acidic runoff after sulfide minerals were exposed. However, there is also a pressing need for highway improvements within that setting. Therefore, a better understanding of how to plan the construction to avoid degradation of the environment is required. An economical method of assessing potential problems early in the decision making process is essential.

Within the mountainous terrain, large excavation and fill volumes are typically involved to maintain acceptable highway grades and the ability to forecast borrow availability is critical in economically constructing the highways. Therefore, in determining the preferred alignment for proposed highways, estimates of the type, quantity and handling requirements of borrow materials that might be generated by various alignments and grades must be considered. With the deep cuts required in such topography, the inclination of cut slopes, which might vary from near vertical to as flat as 3H:1V depending on soil and bedrock conditions, can tremendously influence the volumes of material that must be handled. When the alignment passes through environmentally sensitive areas and portions of the excavation can expose deleterious material that cannot be easily used as borrow, the handling of materials can create an environmental and economic impact to the project. If not recognized early in planning, it can lead to large cost overruns, delays in completing the work or even significant environmental damage.

There are various exploration techniques that can be employed to study the location and character of acid-producing bedrock. In the selection of new highway locations in mountainous terrain, the initial location studies might involve consideration of large tracts of land, as various alignments are analyzed to determine the optimum route. Obviously it would be difficult, costly and time consuming to conduct detailed subsurface exploration for each potential alignment. However, because the analysis of the best route must account for the likely subsurface conditions, it is a wise investment of time and resources to conduct generalized geotechnical and environmental assessments early in the planning process. The assessment can provide an early indication of major geotechnical or environmental difficulties that might be associated with particular areas of the corridor and help to determine if those obstacles can be avoided or their impact lessened by shifts in alignment or grade. If they cannot be avoided, the preliminary assessment can help determine their impact on the planned construction so as to help avoid major cost overruns or environmental degradation.

REVIEW OF PREVIOUS PROJECTS

Several roadway projects in this southern appalachian setting have encountered difficult subsurface conditions consisting of intensely folded and fractured bedrock with sulfide mineralization. The bedrock is known to pose a significant slope stability problem, such as along Interstate 40 through the Pigeon River Gorge, where reoccurring landslides have been a major maintenance expense. Other projects have been hampered by environmental problems in addition to slope instability.

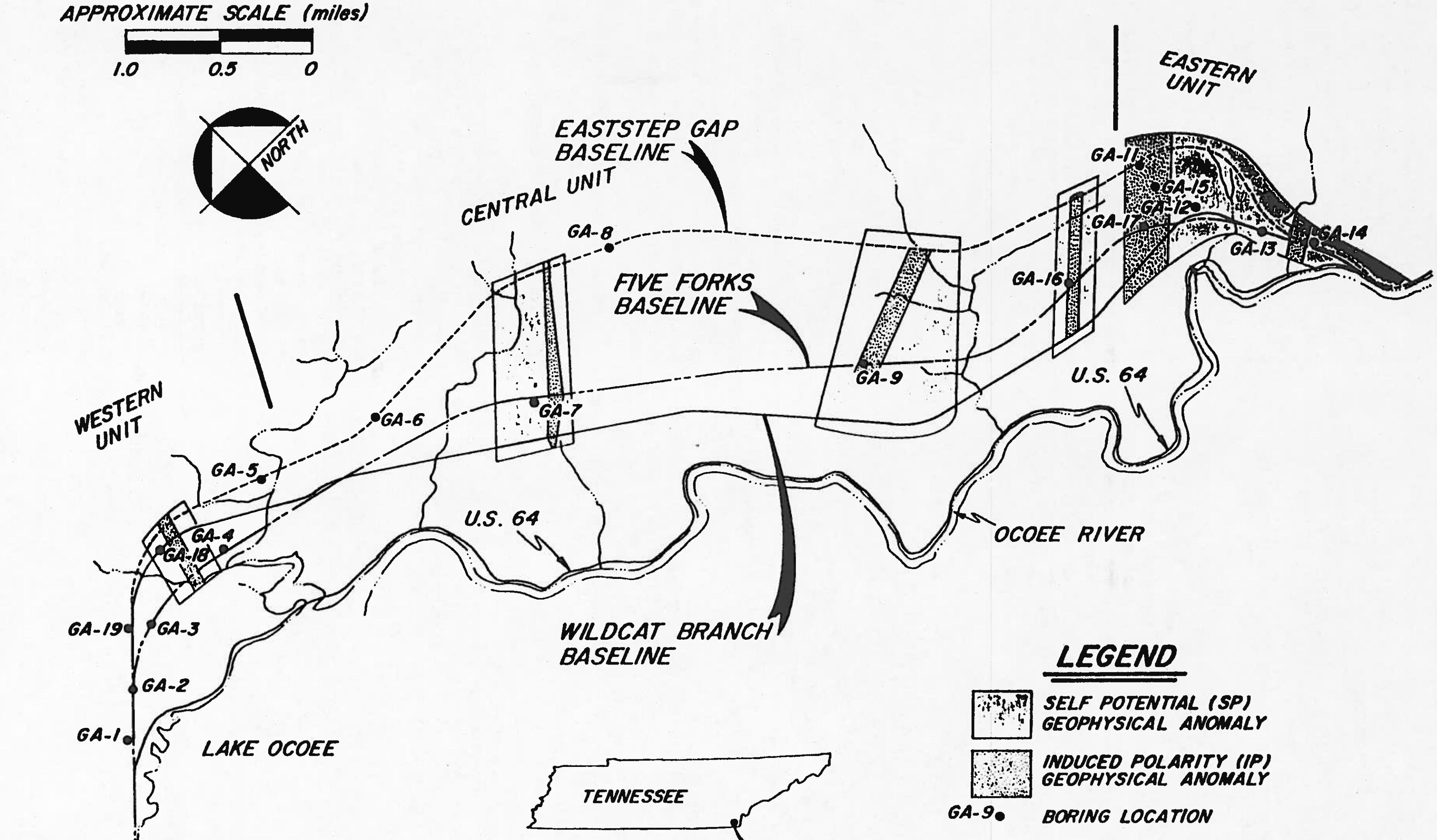
The new alignment for the road between Tellico Plains, Tennessee and Robinsville, North Carolina is located within the Cherokee and Nantahala National Forests and crosses numerous streams along its alignment. During construction in 1977, the U.S. Forest Service discovered that the formerly abundant fish life in several small streams had disappeared. The roadway work was halted while studies of the creeks were made. Follow-up analysis indicated that the affected streams had high concentrations of sulfate and extremely low pH, lower than found in most strip-mine runoffs. The investigation revealed that portions of the road alignment are underlain by bedrock containing sulfide minerals, predominantly pyrite. The study also revealed that approximately 300,000 cubic yards of sulfide rich material was placed in several fills located in the headwaters of the affected streams. After the problem was revealed, temporary treatment measures were applied. The first measure was the addition of agricultural lime to embankment slopes which were identified as producing acid drainage. Asphaltic pavement was installed onto the shoulders in order to help prevent surface runoff from percolating into the subgrade. In order to help neutralize the acid within the streams, a sodium hydroxide base was continuously fed into the streams through a series of tanks and valves. More permanent remediation efforts such as blanketing slopes with an layer of clay and additional lime treatments to the fill, along with drainage and erosion control measures, were started in July of 1978.

More recently, construction was halted for U.S. Highway 441 and for sections of the Foothills Parkway when acidic leachate was identified. These projects point to a need for an improved procedure of subsurface assessment for highway construction within that geologic setting.

PRELIMINARY STUDY FOR U.S. HIGHWAY 64

The Tennessee Department of Transportation (TDOT) has been considering relocating a portion of State Route 40 (also designated U.S. Highway 64) since before 1976. The subject section of U.S. Highway 64, which is about eight miles in length, is located in Polk County, Tennessee, and passes through a steep-walled, sinuous gorge cut by the Ocoee River. An eight mile long and roughly two mile wide corridor was to be investigated to locate a new alignment to bypass the Ocoee gorge, as illustrated in Figure 1. The new highway corridor is located in mountainous terrain and no detailed published geologic maps are known; however, the area has long been know to contain bedrock with pyrite mineralization.

TDOT began a program of subsurface exploration within the corridor but because of the difficult access, only six borings were completed before scheduling constraints led to the use of consulting firms to "fast-track" the project. The preliminary assessment was conducted by the Nashville, Tennessee office of ERCE as a service to the firm of Parsons, Brinkerhoff, Quade and Douglas. The overall goal of the study was to develop preliminary plans and cost estimates for relocating the highway out of the gorge. In order to help the designers select a new alignment, a preliminary geotechnical/geoenvironmental study was conducted to very generally investigate the subsurface conditions, regional geology and to preliminarily assess the engineering and geoenvironmental characteristics of the terrain.



PLAN OF STUDY AREA

FIGURE I

Site Description

The study area is in Polk County, in southeastern Tennessee, within the Blue Ridge physiographic province. In general the area consists of northeast trending ridges and intervening, narrow, steep-walled valleys with 200 to 600 feet of topographic relief between valley and ridge. Topographic relief across the study area is in excess of 1000 feet.

Several southward flowing streams cross the study area and some are classified as "major trout streams". All of the streams empty into, and the entire study area is within the watershed of, the Ocoee River. The Ocoee River is entrenched in a deep, sinuous gorge which cuts across the grain of the ridges and is used for white water rafting, which is a major economic factor in Polk County. The Ocoee River Gorge is narrow, with occasional rock bluffs in excess of 100 feet and with topographic relief of about 600 feet between the river and the ridge tops. The existing alignment for U. S. Highway 64 is located along the north side of the river as it winds through the gorge.

The study area is, for the most part, located within the Ocoee Ranger District of the Cherokee National Forest. There is a thick to nearly impenetrable undergrowth of laurel, rhododendron and vines covering many of the lower portions of the valley slopes and along stream banks. Forest Service roads and logging trails lead to portions of the study area; other more remote areas (particularly within the eastern portion of the site) are inaccessible by vehicle.

Geologic Setting

Bedrock units within the Blue Ridge province are generally considered to be PreCambrian in age and sedimentary in origin. Common rock units consist of thick interbedded sequences of quartzite, variably metamorphosed shale, phyllite, siltstone, sandstone and conglomerate with occasional limestone and dolomite units.

All of the bedrock within the study area has been assigned to the Ocoee Supergroup. In general, the Ocoee Supergroup consists of a thick sequence of fine to coarse-grained sedimentary rocks with changes from one rock type to another being gradational both vertically and laterally. The Ocoee Supergroup is thought to have an aggregate thickness of well over 30,000 feet, with few or no key beds to aid in mapping. The bedrock has undergone regional metamorphism, is intensely folded and fractured and is thought to contain major thrust faults. However, the lack of fossils or marker beds, the similarity of various subdivisions of the Ocoee Supergroup and the structural complexities of the area present major obstacles to geologic mapping and interpretation.

Exploration and Testing

Access to the sixteen square mile project area was severely restricted not only by the very steep and thickly overgrown topography, but also because the site is within a National Forest where disturbance to the land was prohibited by regulations. This meant that access to much of the site could be accomplished only by foot or by helicopter. A combination of visual reconnaissance, limited bedrock coring and geophysical exploration was selected as the best tools for accomplishing the project goals.

For the preliminary study, eighteen borings were drilled. Rather than being located to assess any particular alignment, the boring locations were based on considerations for drill rig access and to gather general subsurface data across the corridor concerning the quality of material that might be exposed by deeper excavations and to assess the presence of potentially acid producing bedrock. All of the borings were located in the higher elevation areas of the site because of the difficulty of accessing the narrow stream valleys. Three of the holes were accessed by airlifting a skid-mounted drill rig into the mountainous, wooded areas.

A total of approximately 2000 linear feet of core was obtained from the borings, averaging 110 linear feet of coring per location. The bedrock cores were logged by a geologist for lithology, weaknesses, rock quality designation (RQD) and the presence of visible mineralization. At 10 locations, ground water observation wells (standpipe piezometers) were installed in order to monitor ground water levels.

The bedrock core was subjected to a series of laboratory tests which included acid-base accounting to determine the potential acid production from all sulfides within the bedrock as well as geochemical analysis to determine the specific amount of pyrite. The acid-base accounting procedure has been used in evaluating the potential for acid mine drainage in the coal fields and has more recently been used as a means of evaluating the potential for acid drainage from highway fills. These tests were performed at approximately five foot intervals for all core recovered. Figure 2 is a graphic representative of the results of the acid-base accounting for Boring GA-17. Selected samples of core were tested for slake durability and unconfined and triaxial compressive strength.

In addition to the drilling program, the study included a search of pertinent geologic literature, a geologic reconnaissance of the corridor and a geophysical exploration program.

A significant effort was made to delineate areas underlain by conductive mineralization (assumed to be pyrite) through the use of geophysical techniques. A technique referred to as Induced Polarization (I. P.) has previously been used with some success in similar geologic settings to locate pyrite deposits. Specifically, a 1981 paper by Jones, Bell and Hanson describes the advantages and difficulties associated with the use of the I. P. technique to locate pyrite on the Tellico - Robinsville project. Because of time and access constraints and the need to find a more mobile technique, other survey methods were considered for the U.S. 64 relocation project. One procedure that was believed to be useful for this purpose was the Self-Potential (S. P.) technique. Accordingly, a test section was surveyed using both the Self-Potential (a previously untested technique for this application) and the Induced Polarization, which had previously been used for the other projects. The data from the test section indicated that the physically less rigorous Self-Potential technique would furnish the type of data required for an initial reconnaissance of the area. A Self-Potential survey totaling nearly thirty miles was then conducted along three base lines transecting the project area, as shown in Figure 1. Although the S. P. and I. P. surveys can both provide an indication regarding possible presence of conductive minerals, the Induced Polarization technique can be used to develop more detailed data concerning the depth and extent of mineralization. The S. P. data cannot determine the quantity of, nor the depth of, conductive mineralization. Therefore, selected areas which appeared to contain pyrite based on the reconnaissance (S. P.) survey were subsequently resurveyed using Induced Polarization in an effort to better assess the extent of the conductive deposits. Figure 1 indicates the location of the three S. P. surveys (the three baselines) and the location of both S.P. and I.P. anomalies along each of the baselines. The anomalies are areas where the data indicates a potential for pyrite mineralization. Borings performed within the areas of significant anomalies generally confirmed the presence of pyrite.

Subsurface Conditions

Western Unit The western end of the study area is characterized by deeply weathered sandstone and siltstone of the Sandsuck Formation. Several borings went to great depths (over one hundred feet) without encountering competent bedrock. Bedrock recovered from the borings in this unit (Borings 1, 2, 3, 4, 18 and 19) indicate extensive and deep weathering to form hard sandy soils and decomposed rock; little or no metamorphism is evident.

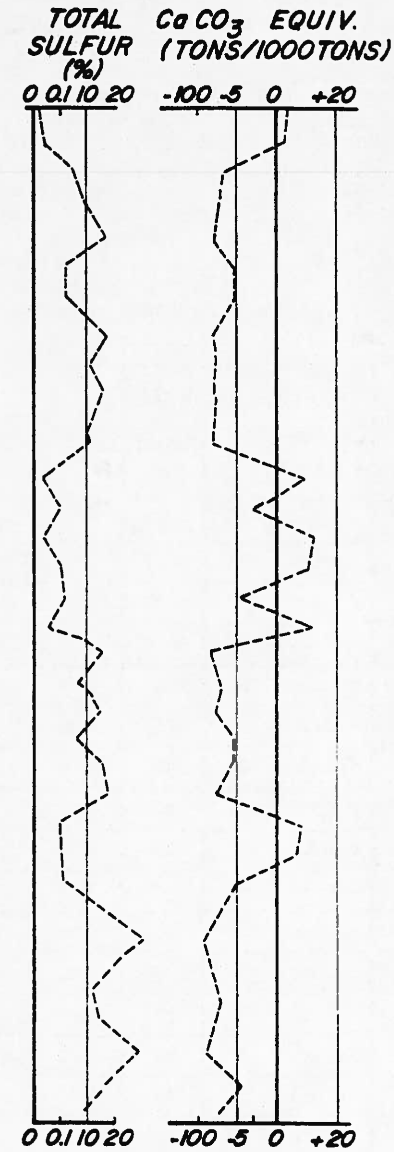
Within the western unit no evidence of pyrite mineralization was noted in the geochemical testing of the bedrock cores and only limited indications of pyrite were generated by the geophysical surveys.

Cental Unit The central portion of the site is generally underlain by fine-grained rocks, mostly slate, phyllite and siltstone with occasional quartz veins and fine-grained sandstone beds. This bedrock belongs to the slightly metamorphosed, undifferentiated formations of the Walden Creek Group. Weathering of the bedrock has resulted in a relatively shallow overburden of silty clay, ranging in thickness from five to twenty-five feet and generally less than 10 feet in thickness. Below the typical soil profile is an extensive and variable thickness of highly weathered rock with intervening soil layers. At the locations explored the highly weathered bedrock generally extends to depths of about fifty to seventy-five feet below the existing ground surface.

Evidence of pyrite mineralization was observed within the bedrock cores and by the geophysical surveys conducted within the central unit. Based on the bedrock cores obtained and analyzed, it appears that the amount of pyrite within the central unit is relatively minor. In addition, much of the bedrock below the zone of degradation contains significant calcium carbonate which would help negate potential acid production.

Eastern Unit The easternmost portion of the study area is generally underlain by bedrock of the Great Smoky Group. Bedrock obtained from the borings in this area include highly graphitic shales, siltstone, slate meta-graywacke and quartzite. The bedrock cores obtained from four borings contained zones with significant quantities of pyrite and minor amounts of pyrite were noted in some of the cores obtained from two other borings.

At the locations explored, weathering of the bedrock has produced a relatively shallow overburden of silty clay which ranges from two to ten feet in thickness. Extensive degradation of the bedrock was generally confined to the upper twenty to thirty feet of the cores obtained. Both the geochemical testing of the core samples, as well as the geophysical surveys, indicate that at least some of the bedrock within the eastern unit contains pyrite mineralization at levels which indicate a potential for significant acid production. In addition the bedrock was lacking significant calcium carbonate to help off-set acid production.



GRAPHIC RESULTS OF ACID-BASE ACCOUNTING CONDUCTED EVERY 5 FEET FOR CORE OBTAINED FROM BORING GA-17. SULFUR CONTENT ABOVE 1% OR $CaCO_3$ EQUIVALENT BELOW -5 INDICATE A POTENTIAL FOR ACIDIC LEACHATE.

FIGURE 2

Study Conclusions

At the locations explored, the subsurface conditions across the site can generally be categorized into three major units, which correspond to the three geomorphic divisions which are crossed from west to east across the site. It is apparent that the topographic differences are directly related to the differing nature of the bedrock. Slope stability problems can be anticipated for all of the units while pyritic bedrock is likely to be of concern only within the easternmost portion of the project.

The laboratory testing performed as part of this study involved tests of 420 core samples obtained from the borings within the study area. Of these, only 22 tests indicated a high potential for generating significant acidic runoff (calcium carbonate equivalent of -15 or less). Seventeen of the samples indicated some acid generation potential (calcium carbonate equivalent of -5 to -15 and pyrite content greater than .01) to the degree that caution should be exercised in the handling of those rocks. The remaining samples showed little or no potential for acid generation due either to the absence of pyrite or to the content of neutralizing carbonate minerals that occur in the same rocks with the pyrite. Figure 3 contains subsurface data and selected geochemical test data from each boring.

Boring #	Location	Soil Thick-ness	Depth to Sound Rock	Highest Sulfur %	Lowest[1] C_aCo_3 Value	Highest[2] C_aCo_3 Value
GA1	WESTERN	6'	>50'	0.02	0.14	1.49
GA2	WESTERN	50'	>50'	0.01	-0.31	1.49
GA3	WESTERN	35'	>110'	0.03	-0.031	15.46
GA4	WESTERN	15'	>50'	0.03	0.19	37.88
GA5	CENTRAL	5'	65'	0.74	1.49	41.91
GA6	CENTRAL	5'	75'	0.46	-0.31	219.38
GA7	CENTRAL	5'	45'	0.02	-0.31	44.42
GA8	CENTRAL	13'	70'	0.01	1.37	471.53
GA9	CENTRAL	25'	75'	0.14	-0.89	44.80
GA11	EASTERN	10'	30'	1.19	-27.50	2.43
GA12	EASTERN	10'	30'	0.32	-3.76	42.46
GA13	EASTERN	2'	35'	1.12	-30.70	1.62
GA14	EASTERN	7'	60'	0.44	-6.35	45.29
GA15	EASTERN	8'	20'	1.20	-23.84	39.95
GA16	CENTRAL	15'	75'	0.08	-1.25	40.33
GA17	EASTERN	10'	25'	3.15	-76.68	1.94
GA18	WESTERN	5'	55'	0.15	-1.04	42.78
GA19	WESTERN	10'	>120'	NA	NA	NA

1 - $CaCO_3$ equivalent values below -5 indicate a potential for acidic leachate.

2 - $CaCO_3$ equivalent values above +20 indicate a potential for buffering acids.

FIGURE 3 - SUMMARY OF BOREHOLE DATA

BENEFITS OF PRELIMINARY STUDY

The preliminary subsurface study proved to be extremely useful in planning for the project. The results of the study indicate that pyritic rock would be encountered along any potential highway alignment, but probably only within the extreme eastern portion of the project area. Based on the conclusion that pyritic rocks occurred in narrow bands that trend perpendicular to the potential roadway alignments, avoidance of pyrite was not a deciding factor in the selection of the final alignment, and geometric constraints took priority. Because of the high concentration of sulphide minerals within the eastern portion, a tunnel, rather than an open cut was selected as a beneficial option for transecting the highest eastern ridge. In addition, the study permitted planning of where encapsulated fills might be required and where sufficient volumes of cover material could be obtained. Also, the study confirmed that significant allowances would be required for the mitigation of slope instability.

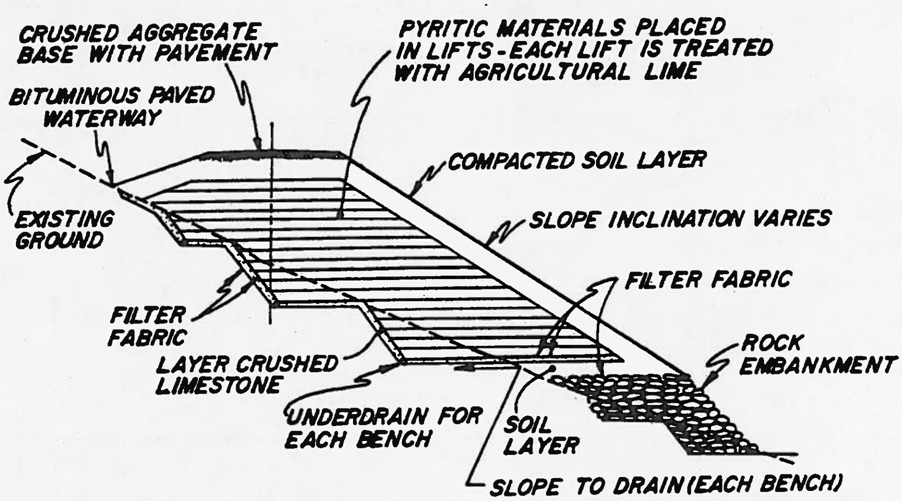
FUTURE STUDIES

As the project continues, more detailed subsurface exploration will be performed in order to develop specific design recommendations for the project. The preliminary exploration has been limited to only a few hole locations generally along the ridge crests where U.S. Forest Service roads provided some measure of access for drilling equipment. Because of the very complex geologic structure, oriented cores will be obtained in order to evaluate the impact of bedrock structure upon slope stability, foundation construction and tunneling. Eventually, detailed exploration at specific foundation locations for bridges and other structures will be performed so that appropriate foundation recommendations and design criteria can be developed.

Not only is there a need to develop additional geotechnical engineering data, the study of pyrite distribution will also continue on a parallel course utilizing the information developed from the subsurface exploration.

HANDLING OF ACID-PRODUCING MATERIALS

In response to the environmental problems experienced on some of the earlier projects where pyrite was encountered, methods have been devised to deal with the pyritic rocks in ways that have demonstrated success in precluding significant acid generation, and a draft version of an as yet unpublished document, "Guidelines for Handling Excavated Acid-Producing Materials" prepared for the FHWA by Dr. Don Byerly has been used in formulating some of the mitigation procedures noted below. Because water and oxygen are required to complete the conversion of sulphides to sulfuric acid, mitigation techniques seek to control the introduction of water or oxygen to the pyritic rocks. The primary method of treatment is encapsulation of the pyritic material within a fill embankment that is covered with compacted, low permeability soil. The encapsulation will also typically incorporate intervals of acid neutralizing carbonate rock at the base of the fill and possibly within regularly spaced layers within the pyrite bearing fill. A schematic cross-section of an encapsulation fill is shown in Figure 4. The encapsulation embankment can be designed as a structural fill to serve as part of the roadway



SCHEMATIC ENCAPSULATING EMBANKMENT DESIGN

FIGURE 4

embankment or it can be constructed as a special, engineered, disposal fill. In either case, it is necessary for the details of the encapsulation to be designed based on the character of the minerals to be encapsulated and the materials serving as the encapsulation (cover) medium. Other methods for treating the pyrite to mitigate acid generation could include chemical treatment with lime, encapsulation using manufactured geo-membranes, containment and treatment of runoff, or wholesale removal and disposal of all pyritic material off-site. Obviously, the practicality of some of these alternatives, such as wholesale removal and off-site disposal, will be highly dependent on the volume of pyritic material encountered. Chemical treatment of the pyritic rock, such as lime injection, could be effective but is probably not a viable long term solution and would most likely require routine reapplication and maintenance. The use of a manufactured geo-membrane is a viable alternative if sufficient volumes of low permeability soil are not available to cover and encapsulate the pyritic rocks.

In addition to pyritic rocks being incorporated into the fill construction, some pyrite will be exposed in cut sections. Methods for treating pyrite exposed by cuts have not been developed to the degree that a standard or well tested procedure is recognized. Possible treatments include covering of the cut faces with a layer of low permeability soil, but this requires laying back the side slopes to inclinations of about 2H:1V or flatter. Other possibilities include collecting and treating the runoff from cut slopes. Retention ponds could possibly be constructed at the bottoms of the rock cuts or in adjacent valleys. Combining the retention of runoff with chemical or biological treatment to reduce acidity should be reviewed in more detail as the design progresses. Although data regarding the potential impact of acid runoff from cut slopes is lacking, it is considered to be less of a potential environmental hazard than acid runoff from fills, primarily because of the smaller surface area of pyritic rock exposed in the cuts.

In conjunction with the mitigation measures planned for embankments and cuts, the installation of foundations into pyritic bedrock will require a plan for controlling the potential acid runoff during construction. Application of lime on a temporary basis during construction together with proper disposal of pyritic spoils should be sufficient. Measures to control siltation of the streams on-site will also be required during both roadway and foundation construction.

SUMMARY

Within geologic settings where acid production from pyritic bedrock is possible, the preliminary subsurface exploration should include environmental as well as geotechnical study. The first step of such a study is a review of all published geologic maps and literature concerning the area. Outcrops should be mapped and sampled whenever possible. A geologic exploration should be conducted using Self Potential (SP) for the initial reconnaissance and Induced Polarization (IP) can be used to more accurately delineate areas of pyritic rock. Those areas can then be explored in detail with conventional techniques, such as coring of bedrock. Review of the bedrock core for the presence of pyrite should be conducted and all rock core should be subjected to geochemical testing (acid-base accounting). Based on the results of the exploration and testing, techniques to mitigate environmental damage can be incorporated into the design process. During planning, there should be contingencies for handling unexpected pyrite and an on-site geologist should review day to day construction activities for any indication of pyritic material.

SELECTED REFERENCES

Byerly, Don W., March, 1989. Draft of unpublished FHWA Document, Guidelines for Handling Excavated Acid-Producing Materials.

Edick, T. O., 1978, A Report on Geologic and Water Quality Analysis of Acid Drainage Produced From Roadway Embankments and the Mitigating Measures for a Public Lands Highway - Cherokee National Forest, Tennessee. SASHTO Technical Session, Nashville, Tennessee, October, 1978.

Hadley, J. B., and Goldsmith, Rehard, 1963, Geology of the eastern Great Smoky Mountains, North Carolina & Tenn.: USGS Proff Paper 349-B, p. B1-B118.

Hadley, J. B., 1970 The Ocoee Series and its possible correlatives: in Fisher, G. W., Pettijohn, F. J., Redd, J. C., Jr., and Weaver, K. N., in Studies of Appalachian Geology, Central and Southern: New York, Intersci. Publishers, p. 247-259.

Hardeman, W. D., Miller, R. A. and Swingle G. D. and others, 1966, Geologic Map of Tennessee: Tennessee Division of Geol. Scale: 1:250,000

Hernon, R. M. 1968, Geology of the Ducktown, Isabella, and Persimmon Creek quadrangles, Tennessee and North Carolina: U. S. Geol. Survey, Open File Report, 71p.

Hernon, R. M., 1964 Geologic Maps & Sections of the Ducktown, Isabella and Persimmon Quadrangles: USGS, Open File Scale 1:24,000;

Hurst, V. J., and Schlee, John S., 1962, Ocoee meta-sediments--North Central Georgia--Southeast Tennessee: Guidebook No. 3, Geol. Soc. America, Southeastern Sec. Am. Mtg., 10p.

Jones, D.H., Bell, B.S., and Hansen S.H., 1981-Application of Induced Polarization in Highway Planning, Location and Design: 32nd Annual Highway Geology Symposium, University of Tenn, 30 p.

King, P. B. 1955, A geologic section across the southern Appalachians--an outline of the geology in the segment in Tenn., N. Carolina, and South Carolina: in Russell, R. J., Ed., Guides to Southeastern Geology, Geol. Soc. Amer., p. 332-373.

King, P. B., 1964, Geology of the Central Great Smoky Mountains, Tennessee & N. C. U. S. G. S. 65 Prof. paper 349-C, 148 p.

King, P. B, Hadley, J. G., Neuman, R. B., and Hamilton, Warren, 1958, Stratigraphy of the Ocoee Series, Great Smoky Mountains, Tennessee and North Carolina: Geol. Soc. America Bull., v. 69, p. 947-966.

Merschat, C. E. and Larson, L. T., 1972, Disseminated Sulphides in Lake Precambrian Ocoee rocks (Abs): Geol Soc. America Abstracts, v. 4, no. 2, p. 92-93.

Merschat, C. E., and Weiner, L. S., 1973, Provisional geologic map of the Ocoee Supergroup southwestern North Carolina and southeastern Tennessee (Abs.): Geol. Soc. America Abs. with Programs, v. 5, p. 420.

Nesbitt, B. E., and Kelly, William C., 1980, Metamorphic generation of sulfides, oxides and graphite in and around the orebodies at Ducktown, Tennessee: Econ. Glg., v. 75, p. 1010-1021.

Nesbitt, Bruce E., 1979, Regional metamorphism of the Ducktown, Tennessee, massive sulfides and adjoining portions of the Blue Ridge Province: Ph. D. Dissert., Univ. of Mich., 216p.

Rodgers, John, 1953, Geologic map of east Tennessee with explanatory text: Tennessee Div. Geology, Bull. 58, pt. 2, Nashville, Tennessee.

Runyon, Gary, A., 1983, The distribution and variation of Sulphide constituents of the Great Smoky Group, Ducktown area, Tennessee, with respect to metamorphic grade: unpub. Masters Thesis, Univ. of Tenn., 114p.

Skousen, J.G., Sencindiver, J.C., and Smith, R.M., 1987, A Review of Procedures for Surface Mining and Reclamation in Areas with Acid-Producing Materials; West Virginia University Energy and Water Research Center, 39 p.

Sutton, Thomas C., 1971, Relationship between metamorphism and geologic structure along the Great Smoky Fault System. Parksville quad, Polk & Bradley Co. Tenn.: Univ of Tenn., Unpub Doctoral dissertation, 148p.

Weiner, L. S., and Merschat, C. E., 1978, Summary of Geology between the Great Smoky Mountain fault at Parksville, Tennessee and basement rocks of the Blue Ridge at Glade Gap, North Carolina: Tennessee Div. Geol., Rept, Invs. 37, 86p.

APPENDIX

Past Proceedings of Ohio River Valley Soils Seminars

ORVSS	I:	BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, Kentucky
ORVSS	II:	EARTHWORK ENGINEERING, START TO FINISH, October 15, 1971, Louisville, Kentucky
ORVSS	III:	LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, Kentucky
ORVSS	IV:	GEOTECHNICS IN TRANSPORTATION ENGINEERING, October 5, 1973, Lexington, Kentucky
ORVSS	V:	ROCK ENGINEERING, October 18, 1974, Clarksville, Indiana
ORVSS	VI:	SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, Kentucky
ORVSS	VII:	SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN AND CONSTRUCTION, October 8, 1976, Lexington, Kentucky
ORVSS	VIII:	EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION, AND PERFORMANCE, October 14, 1977, Louisville, Kentucky
ORVSS	IX:	DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, Kentucky
ORVSS	X:	GEOTECHNICS OF MINING, October 5, 1979, Lexington, Kentucky
ORVSS	XI:	EARTH PRESSURES AND RETAINING STRUCTURES, October 10, 1980, Clarksville, Indiana
ORVSS	XII:	GROUNDWATER: MONITORING, EVALUATION, AND CONTROL, October 9, 1981, Fort Mitchell, Kentucky
ORVSS	XIII:	RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, Kentucky
ORVSS	XIV:	FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, Indiana

APPENDIX (CONT'D)

ORVSS	XV:	PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, Kentucky
ORVSS	XVI:	APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, Kentucky
ORVSS	XVII:	NATURAL SLOPE STABILITY AND INSTRUMENTATION, October 17, 1986, Clarksville, Indiana
ORVSS	XVIII:	LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Fort Mitchell, Kentucky
ORVSS	XIX:	CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES, October 21, 1988, Lexington, Kentucky
ORVSS	XX:	CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, Kentucky