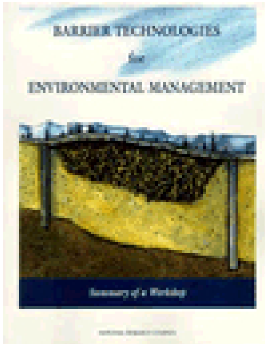

Barrier Technologies for Environmental Management: Summary of a Workshop

Committee on Remediation of Buried and Tank Wastes, National Research Council

ISBN: 0-309-56159-0, 188 pages, 8.5 x 11, (1997)

BARRIER TECHNOLOGIES for ENVIRONMENTAL MANAGEMENT

Summary of a Workshop

Committee on Remediation of Buried and Tank Wastes
Board on Radioactive Waste Management
Commission Geosciences, Environment, and Resources
National Research Council

NATIONAL ACADEMY PRESS
Washington, D.C. 1997

NOTICE: The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competencies and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by the Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

The work was sponsored by the U.S. Department of Energy, Contract No. DE-FC0194EW54069/R. All opinions, findings, conclusions, and recommendations expressed herein are those of the authors and do not necessarily reflect the views of the Department of Energy.

Library of Congress Catalog Card Number 96-72353
International Standard Book Number 0-309-05685-3
Additional copies of this report are available from: National Academy Press 2101 Constitution Ave., NW Box 285 Washington, DC 20055 800-624-6242 202-334-3313 (in the Washington Metropolitan Area) http://www.nap.edu

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Printed in the United States of America

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DENISE BIERLEY, Roy F. Weston, Inc., Albuquerque, New Mexico
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[*] Chair as of May 9, 1996.
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Acknowledgments

Many people assisted in the design and conduct of the Workshop on Barriers for Long-Term Isolation. The Committee on Remediation of Buried and Tank Wastes thanks John Lehr, Office of Environmental Restoration of the U.S. Department of Energy, for his support and participation as a general chair of the workshop. Other general chairs for the workshop were committee members James Clarke of ECKENFELDER INC. and Paul Witherspoon of the University of California at Berkeley. The committee also thanks Julie D'Ambrosia of EnviroTech Associates, Inc., for her assistance with the planning and conduct of the workshop, and for providing notes on the workshop to the committee. In addition, the committee thanks Glendon Gee of Pacific Northwest National Laboratories, David Daniel of the University of Texas (now at the University of Illinois), Robert Mutch of ECKENFELDER INC., and Paul Witherspoon, all of whom served as session chairs at the workshop.

Committee members James Clarke and Paul Witherspoon, along with committee staff officer Robert Andrews and DOE contractor Julie D'Ambrosia, formed a steering group to develop the concept and structure of the workshop. Susan Mockler, research associate for the Board on Radioactive Waste Management, assisted with preparation and editing of the report and the articles prepared by the presenters. Dennis DuPree and Patricia Jones, senior project assistants for the board, assisted in workshop logistics and registration and in preparation of this report. Although this report is the product of the committee, we acknowledge initiatives of the steering group to organize and conduct the workshop and to help prepare an early draft of the report.

The committee also acknowledges the contribution of the speakers at this workshop for providing their papers for inclusion in this report.

Thomas Leschine, Chair

Committee on Remediation of Buried and Tank Wastes

Contents

Executive Summary

Remediation of radioactive and mixed waste located in the U.S. Department of Energy (DOE) nuclear weapons complex will require increased use of physical barriers to prevent the spreading of contaminants during interim periods of cleanup and the migration of contaminants left behind upon completion of the cleanup.

To raise the level of awareness of available technologies and to provide information on the current knowledge of barrier performance through technology development and actual installation, the Committee on Remediation of Buried and Tank Wastes and representatives of the DOE Office of Environmental Restoration organized a 1-day workshop on engineered barriers. Participants in this workshop included government researchers and contractors, as well as barrier designers and builders from private industries.

This summary report is a synthesis of the oral discussions at the workshop. It does not express opinions of the committee. The committee issued a report recently, entitled *The Potential Role of Containment-in-Place in an Integrated Approach to the Hanford Reservation Site Environmental Remediation* (National Research Council, 1996), on the potential use of barriers at a DOE site.

Not all waste problems can be solved by excavating and treating the wastes. Proper use of effective barrier technologies can provide both interim containment while more permanent remedial technologies are being developed, and longer-term isolation of radioactive and hazardous contaminants remaining after remediation. Consequently, barriers such as surface caps and subsurface vertical and horizontal barriers will be needed as important components of remediation strategies.

Several themes emerged during the discussions at the workshop:

- The importance of employing proper installation techniques and quality control measures, especially during construction, including using contractors with demonstrated experience and skill.
- The need for knowledge concerning effective lifetimes of selected barrier materials and resultant barrier systems.
- The importance of periodic inspection, maintenance, and monitoring of containment barriers.
- The current dearth of barrier performance monitoring data.
- The advantages of using barriers in combination with pump-and-treat approaches.
- The importance of compiling data on both successful and unsuccessful barrier installations.

Although these issues were not explored fully during the workshop, they will serve as good starting points for future discussion on containment technology. Appendix D to this summary report contains papers prepared by the workshop presenters. These papers will serve as a supplement to other recent compilations of work on barrier technology.

Introduction

The U.S. Department of Energy (DOE) is the federal agency responsible for the remediation of this country's nuclear weapons complex, a large network of industrial facilities for the research, production, and testing of nuclear weapons. This enormous undertaking currently is estimated to cost several hundreds of billion dollars over the next 75 years (U.S. Department of Energy, 1996).

Not all radioactive waste problems can be solved by excavating and treating wastes. Properly engineered containment systems can provide both interim isolation of contaminants, while remedial technologies are being developed, and longer-term isolation of those contaminants that will remain at DOE sites after remediation. Consequently, engineered containment structures (collectively referred to as "barriers" in this report) such as surface caps and subsurface vertical and horizontal barriers will be needed as important components of remediation strategies.

The Committee on Remediation of Buried and Tank Wastes (hereafter, the "committee") was appointed by the National Research Council and has the general task of addressing critical generic and site-specific issues relevant to remediation of the environmental contamination from buried and tank-contained defense radioactive and mixed waste. Among the issues under study by the committee is the application of new and evolving remediation technologies and strategies. During its studies, the committee found that effective containment-in-place approaches are needed at the Hanford Site in Richland, Washington, and across the DOE complex (National Research Council, 1996). The committee also found that DOE was performing significant research, including prototype evaluations at facilities such as the Hanford Site in Washington and the Idaho National Engineering Laboratory, Idaho Falls. In addition, DOE has constructed and maintained surface barriers under the Uranium Mill Tailing Remedial Action (UMTRA) Project. Other entities have used barrier technologies successfully for many years to isolate waste materials and contaminated ground water and soil.

Consequently, the committee and representatives of the DOE Office of Environmental Restoration agreed that a workshop on containment barriers would be useful. Participants in such a workshop would include government researchers and contractors, as well as barrier designers and builders from the private sector. DOE and the committee cosponsored the workshop on August 13, 1995, in conjunction with the DOE ER'95 (Environmental Restoration 1995) Conference in Denver, Colorado.

The workshop program was designed to cover a wide range of barrier approaches using various materials, both natural and man-made, and different installation techniques. Information was presented on surface and subsurface barrier technologies being evaluated or used within the DOE complex and by other entities. Two overview presentations were followed by sessions on surface and subsurface barriers. Each session was completed by discussion with a panel of the presenters. The next day, the session chairs presented a summary to the attendees of ER'95. The program for the workshop is included as Appendix B, and workshop participants are listed in Appendix C.

The purpose of this summary is to report on some of the oral discussions. The papers presented (included in Appendix D) provide more detailed information on barrier technology development and implementation. Significant information on barrier technology has been published in reports of two recent, DOE-cosponsored meetings (Gee and Wing, 1994; Rumer and Mitchell, 1996) and a DuPont Company workshop (Rumer and Ryan, 1995).

The text that follows is a synthesis of the oral discussions at the workshop. It does not represent the opinions of the committee.

Workshop Overview

The use of surface barrier research within the DOE nuclear weapons complex and the installation and use of vertical subsurface barriers at sites primarily outside of the DOE complex were the focus of two introductory presentations at the workshop (see Appendix B for program of the workshop). It was noted that about 70 million cubic meters of radioactively contaminated soil within the DOE complex require remediation. Regardless of the remedial methods pursued for individual sites, engineered containment barriers will be needed to ensure short-term (tens to hundreds or years) to long-term (hundreds to thousands of years) isolation of residual materials. Representatives from DOE mentioned that estimates of costs for development, construction, and maintenance of barriers at DOE sites are on the order of tens of billions of dollars.

Following the introductory speakers, there were six presentations on surface barriers and five on subsurface barriers, both horizontal and vertical. In addition, the findings of a study of ground water cleanup alternatives by the National Research Council (1994) were summarized. The presentations addressed such topics as types of barrier construction materials, biointrusion, application of freezing to achieve temporary subsurface confinement, and barrier installation techniques. The subject of barriers and regulatory compliance was not presented formally, but it was raised several times during the workshop.

To be effective, surface barriers must control infiltration of precipitation and surface runoff, erosion, and biointrusion, with minimal maintenance. However, it was noted that it is impractical to eliminate entirely the potential for degradation of surface barriers over long time periods. It was suggested that surface barrier sites should be visited at least once a year for maintenance and monitoring to ensure long-term performance as designed.

Workshop participants discussed the importance of being able to demonstrate that a surface barrier will remain effective for periods of 200 to 1,000 years of isolation. They also expressed concern that surface barriers may require a large volume of construction materials. If a thick surface cover is to be constructed from natural materials, sufficient amounts of the materials with proper characteristics may not be readily available at an affordable cost to the sites in question.

Technologies for constructing surface barriers under a variety of site-specific conditions, including climate, are still being developed and demonstrated. Although there have been instances of failures of surface barriers, there are locations where barriers have been effective generally in their application to a range of waste site remediation conditions. Some of the failures are associated with applying a technology to a site where the design parameters are inconsistent with site conditions; others may be the result of a lack of well-defined performance standards having good quality assurance and quality control metrics, or of poor construction practices. Undoubtedly, surface barriers will continue to play an important role in the future, but even the best design can fail if the barrier was installed and maintained incorrectly.

Subsurface barriers are likely to be effective as a temporary measure to prevent migration of contaminants of concern while more effective removal or neutralization technologies are developed and demonstrated. In addition, some subsurface barriers appear to offer the potential for long-term containment of contaminants. Subsurface barriers have been used in the private sector for nearly 30 years, and vertical cutoff walls have been constructed to depths of several hundreds of feet. The installation of subsurface horizontal barriers beneath large structures or contaminated areas (such as under a tank farm) is likely to challenge current installation technology.

Work is still- needed in the design of surface and subsurface barriers that would lead to more effective construction and testing, as well as minimizing costs without jeopardizing protection to the public and the environment. Some workshop participants suggested that research and development efforts in barriers need to be continued by DOE in areas such as (1) collection and use of both laboratory and field data to advance the development and application of mathematical models and to bring about greater confidence in model predictions regarding barrier system performance, and (2) techniques for monitoring migration of contaminants contained by barriers and for detecting defects in barriers.

Participants discussed the challenge of pursuing innovative containment technology within the DOE waste complex, addressing both regulator and stakeholder skepticism associated with unproved approaches, plus the need for selecting experienced contractors within the DOE procurement system. A participant noted that the industry is not sufficiently mature to enable companies to take legal responsibility for emplacement of barriers requiring long-term integrity. It was suggested that DOE and regulators might consider the approach taken in Europe, where the contractor accepts liability related to substandard performance of the containment system for a period of 10 years. Over this period, it is anticipated that the technology may improve such that further modifications to the system, if necessary, may act to ensure satisfactory performance for an extended time.

It was noted that data on the effective performance lifetime as a function of climate, hydrology, and geology should be compiled for selected barriers constructed of both natural and synthetic materials. Convincing scientific and engineering evidence that barriers retain their effectiveness over sufficiently long time periods is needed. A representative from the U.S. Nuclear Regulatory Commission reported that the agency is examining how much credit for isolation, as defined for regulatory purposes, can be given to various engineered barrier systems. Of concern to regulators and the public is the lack of available supporting technical bases and scientific proof of isolation.

The greatest chance of success for barrier deployment will result from use of proper installation techniques by contractors with demonstrated experience and skill, along with quality control and quality assurance measures. Successful installers may be able to provide some useful information to researchers, and vice versa, so that the technical engineering concept may be married to the construction process. Participants encouraged the collection and publication of case studies of valuable information on the performance of barrier systems that could be acquired by instrumenting existing barriers.

The summary of the National Research Council (1994) report on ground water cleanup noted the difficulty of cleaning up contaminated aquifers using pump-and-treat methods (pumping contaminated ground water to the surface for treatment). This presentation prompted a discussion of the causes of this difficulty, including inadequate technology, misapplication of existing technology, and lack of sufficient knowledge regarding the behavior of contaminants in the subsurface environment. The use of barriers to isolate materials in-place might be a reasonable alternative to pump-and-treat systems, or it might enhance the effectiveness of pump-and-treat technology for waste site cleanup.

One cannot install a barrier and leave it unmonitored after only a few years. It is very difficult to develop engineering plans for unexpected, uncertain, or unpredictable long-term events such as climatic changes because such events may affect local weather patterns and the attendant physical, chemical, and biological factors acting on a barrier. Participants noted that development and refinement of non-invasive monitoring techniques, such as shallow exploration geophysics, may be useful in ensuring that barriers are functioning as designed, as well as for detection of defects in the barriers. More sensitive and robust instruments may be needed to monitor subtle changes that may be forerunners of contaminant migration in the case of long-term isolation; such instruments would require periodic calibration. Tracers that follow migrating contaminants may increase the effectiveness of monitoring instruments. Participants suggested that some case studies should explain the absence of data collection and performance monitoring data for barriers (factors may include cost, absence of short- and long-term in-place monitoring instruments and methods, ambiguity of regulatory requirements, and level of interest in performance evaluation).

There was agreement by the workshop participants that mathematical simulation modeling is also important to predict the performance of barrier design and installation. However, participants noted that extended monitoring can be used to verify models of barrier performance.

Several presenters noted the importance of both surface and subsurface barriers to prevent vapor transport of contaminants of concern. Vapor transport and infiltration of precipitation into and through a barrier may cause migration of contaminants to the air above the barrier or to the ground water, respectively. It was suggested that, where appropriate, barriers in which such conditions may exist should provide for controlled vapor venting.

Several participants noted a need for improved communication among operators at the various DOE sites that may need to use barriers to meet their site-specific remediation objectives and regulatory statutes and agreements. For example, barriers constructed under the UMTRA Project appear to be functioning successfully, but information on these barriers needs to be communicated effectively to other DOE sites where barriers are needed. However, significant potential exists for selecting the wrong barrier for a specific site. A design that works in one situation may not work at all in another. This observation supports the importance of documenting and publicizing both successes and failures of barrier development and operation case studies.

Themes Identified at the Workshop

Several themes emerged during the panel discussions.

- The importance of employing proper installation techniques and quality control measures, especially during construction, including using contractors with demonstrated experience and skill.
- The need for knowledge concerning effective lifetimes for selected barrier materials and resultant barrier systems.
- The importance of periodic inspection, maintenance, and monitoring, both short- and long-term, of containment barriers.
- The current dearth of barrier performance monitoring data.
- The advantages of using barriers in combination with pump-and-treat approaches.

- The importance of compiling data on both successful and unsuccessful barrier installations.

Although these issues were not fully explored during the workshop, they will serve as good starting points for future discussion on containment technology. The papers prepared by the workshop presenters, included in Appendix D, should provide a useful supplement to other compilations of work on barrier technology from recent meetings and workshops.

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APPENDIX A

Biographical Sketches Of Committee Members

THOMAS M. LESCHINE, *Chair,* is associate professor in the School of Marine Affairs at the University of Washington, Seattle. He is a former Fellow in Marine Policy and a Policy Associate at the Woods Hole Oceanographic Institution, Woods Hole, Massachusetts. He is the Chair of the National Research Council's Committee on Remediation of Buried and Tank Wastes and also serves on the National Research Council Committee, on Risk Assessment and Management of Marine Systems. His major research interest is in the area of environmental decision making as it relates to marine environmental protection and the use of scientific and technical information in environmental decision making. He is particularly interested in the use of mathematical modeling and systems analysis in environmental management. Dr. Leschine received his Ph.D. in mathematics from the University of Pittsburgh.

DENISE BIERLEY is a project director for Roy F. Weston, Inc. in Albuquerque, New Mexico. Her specialties are broad environmental issues and program management. Prior to joining Weston, she dealt with various environmental, regulatory, and water resource issues for federal and state agencies. Ms. Bierley holds B.S. degrees in biology and geology from Wright State University, Dayton, Ohio.

ROBERT J. BUDNITZ has been President of Future Resources Associates, Inc. in Berkeley, California, since 1981. Before that, he was at the U.S. Nuclear Regulatory Commission (19781980) and was a member of the technical staff and held several management positions at the Lawrence Berkeley National Laboratory of the University of California (1967-1978). He received his B.A. degree from Yale University and his Ph.D. in physics from Harvard University. His professional interests are in environmental impacts, hazards, and safety analysis, particularly of the nuclear fuel cycle. He has served on numerous investigative and advisory panels of scientific societies, government agencies, and the National Research Council.

THOMAS A. BURKE is associate professor of health policy and management at The Johns Hopkins University School of Hygiene and Epidemiology in Baltimore, Maryland. His work includes the evaluation of population exposure to the environmental pollutants, assessment of environmental risks, and the application of the epidemiology and health risk assessment to public policy. Prior to his appointment at Johns Hopkins, he was deputy commissioner of health for the State of New Jersey. He is a member of the Council of the Society of Risk Analysis and has served on Office of Technology Assessment advisory panels on Risk Assessment of Chemical Carcinogens and Managing Nuclear Materials from Warheads. He received a B.S. from Saint

Peter's College, an M.P.H. from the University of Texas, and a Ph.D. in epidemiology from the University of Pennsylvania.

ROBERT J. CATLIN is a licensed medical physicist and certified health physicist. He retired in 1995 as executive director, clinical and laboratory safety, at the University of Texas Health Sciences Center, Houston, where he also served as executive director of the Positron Diagnostic and Research Center and taught radiological science at the School of Public Health. Previously, he served as scientific adviser for the Electric Power Research Institute and had careers in federal service and industry. Mr. Catlin is a member of Sigma Xi, the American Academy of Health Physics, and other professional societies. He has participated as a consultant to the former Soviet Union and to the U.S. Department of Energy on radiological matters for incidents at Chernobyl and at Chelyabinsk. He has served on numerous industry and government advisory committees, including those of the National Council on Radiation Protection and Measurements and the National Research Council's Board on Radioactive Waste Management. Mr. Catlin received his A.B. degree in biology from Princeton University and an M.S. equivalent in health physics at Oak Ridge National Laboratory.

GREGORY R. CHOPPIN is the R.O. Lawton Distinguished Professor of Chemistry at Florida State University, Tallahassee. Dr. Choppin's research includes nuclear chemistry, physical chemistry of actinides and lanthanides, environmental behavior of actinides, chemistry of the f-Elements, separation science of the f-Elements, and concentrated electrolyte solutions. During a postdoctoral period at the Lawrence Radiation Laboratory, University of California, Berkeley, he participated in the discovery of mendelevium, element 101. His research activities have been recognized by the American Chemical Society's Award in Nuclear Chemistry and Southern Chemist Award, the Manufacturing Chemists award in Chemical Education, and a Presidential Citation Award of the American Nuclear Society. He has served on numerous National Research Council committees and currently, is a member of the Board on Chemical Sciences and Technology. He received his B.S. in chemistry from Loyola University, New Orleans; his Ph.D. in chemistry from the University of Texas, Austin; an honorary degree from Chalmers University, Goteborg, Sweden; and an honorary D.Sc. from Loyola University.

JAMES H. CLARKE is Chairman, President, and CEO of ECKENFELDER INC., Nashville, Tennessee, an environmental science and engineering firm specializing in industrial waste management. He has over 25 years of experience in environmental chemistry and chemical risk assessment. His primary areas of interest include the fate and transport of chemicals in the environment, the design of environmental data acquisition programs for evaluation of the risks associated with chemical releases, and innovative and emerging technologies for hazardous waste site remediation. He is an Adjunct Professor with the Department of Civil and Environmental Engineering of Vanderbilt University and serves on the faculty of several continuing education programs, including those of the American Institute of Chemical Engineers, the Center for Professional Advancement, and several universities. Dr. Clarke received a B.A. in chemistry from Rockford College, Rockford, Illinois, and a Ph.D. in theoretical physical chemistry from The Johns Hopkins University, Baltimore, Maryland.

THOMAS A. COTTON is vice president of JK Research Associates, Inc., Arlington, Virginia, where he is a principal in activities related to radioactive-waste-management policy and strategic planning. Before joining JK Research Associates, he dealt with energy policy and radioactive-waste-management issues as an analyst and project director during nearly 11 years with the Congressional Office of Technology Assessment. His expertise is in public policy analysis, nuclear waste management, and strategic planning. He received a B.S. in electrical engineering from Stanford University, an M.S. in philosophy, politics, and economics from Oxford University, and a Ph.D. in engineering-economic systems from Stanford University.

ALLEN G. CROFF is associate director of the Chemical Technology Division at Oak Ridge National Laboratory (ORNL). His areas of focus include initiation and technical management of research and development involving waste management, nuclear fuel cycles, transportation, conservation, and renewable energy. Since joining ORNL in 1974, he has been involved in numerous technical studies that have focused on waste management and nuclear fuel cycles, including supervising and participating in the updating, maintenance, and implementation of the ORIGEN-2 computer code; developing a risk-based, generally applicable radioactive waste classification system; multidisciplinary assessment of actinide partitioning and transmutation; and leading and participating in multidisciplinary national and international technical committees. He has a B.S. in chemical engineering from Michigan State University, a degree in nuclear engineering from the Massachusetts Institute of Technology, and an M.B.A. from the University of Tennessee.

RODNEY C. EWING is a Regents Professor in the Department of Earth and Planetary Sciences at the University of New Mexico, Albuquerque, where he has been a member of the faculty for 23 years. His professional interests are in mineralogy and materials science. He has conducted research in Sweden, Germany, Australia, and Japan, as well as the United States. Dr. Ewing is a fellow of the Geological Society of America and the Mineralogical Society of America. Presently, he is the vice president and president-elect of the International Union of Materials Research Societies. He has served on several National Research Council committees. Dr. Ewing received M.S. and Ph.D. degrees in geology from Stanford University.

DONALD R. GIBSON, JR., is Department Manager of the Systems Analysis Department and Acting Lab Manager at TRW's Ballistic Missiles Division in its survivability and engineering laboratory. Prior to these positions, he was a design physicist and senior project engineer. Dr. Gibson holds M.S. and Ph.D. degrees in nuclear engineering from the University of Illinois.

JAMES H. JOHNSON, JR., is professor of civil engineering and Dean of the School of Engineering at Howard University in Washington, D.C. Dr. Johnson's research interests have focused mainly on the reuse of wastewater treatment sludges and the treatment of hazardous substances. His recent research has included the refinement of composting technology for the treatment of contaminated soils, chemical oxidation and cometabolic transformation of explosive contaminated wastes, biodegradation of fuel-contaminated ground water, the evaluation of environmental policy issues in relation to minorities, and development of environmental curricula. Currently, he serves as Assistant Director of the Great Lakes and Mid-Atlantic Hazardous

Substance Research Center, member of the Environmental Engineering Committee of the U.S. EPA's Science and Advisory Board, and the National Research Council's Board on Radioactive Waste Management. Dr. Johnson received his B.S. from Howard University, M.S. from University of Illinois, and Ph.D. from the University of Delaware. He is a registered professional engineer and a diplomate of the American Academy of Environmental Engineers.

W. HUGH O'RIORDAN is an attorney with Givens, Pursley, & Huntley in Boise, Idaho. He received a B.A. and J.D. from the University of Arizona, Tucson, and an L.L.M. from George Washington University, Washington, D.C., in environmental law. Since entering private practice in 1980, he has specialized in environmental, natural resources, energy and administrative law on state and federal levels. He has represented corporate and individual clients in matters involving environmental statutes.

GLENN PAULSON is president, Paulson and Cooper, Inc., an environmental and energy consulting company in Jackson Hole, Wyoming. Formerly, he was a research professor with the Pritzker Department of Environmental Engineering, Illinois Institute of Technology. He received a B.A. in chemistry from Northwestern University, and a Ph.D. in environmental sciences and ecology from the Rockefeller University, New York. Dr. Paulson served as a member of the National Research Council's Board on Radioactive Waste Management from 1989 to 1996 and has served on several other National Research Council committees dealing with hazardous and radioactive waste.

BENJAMIN ROSS is president of Disposal Safety Incorporated (DSI), a firm in Washington, D.C., specializing in analysis of contamination by hazardous radioactive and chemical waste. Dr. Ross was a senior research scientist at GeoTrans, Inc., and a risk analyst with the Analytic Sciences Corporation prior to working at DSI. Dr. Ross received his A.B. in physics from Harvard University and his Ph.D. in physics from the Massachusetts Institute of Technology. He is a certified ground water professional with the Association of Ground Water Scientists and Engineers.

PAUL A. WITHERSPOON is professor emeritus of Geological Engineering at the University of California, Berkeley, where he was a member of the Department of Materials Science and Mineral Engineering from 1957 to 1989. During the same period, he was associate director and head, Earth Sciences Division, Lawrence Berkeley National Laboratory (1977-1982). He has been president of Witherspoon, Inc., in Berkeley, California, since 1988. He received his B.S. from the University of Pittsburgh and Ph.D. from the University of Illinois. His professional interests include the flow of fluids in fractured and porous rocks, underground storage of natural gas, and underground disposal of liquids and radioactive waste. He is a fellow of the American Geophysical Union, American Association for the Advancement of Science, and Geological Society of America. He is also a member of the National Academy of Engineering.

RAYMOND G. WYMER is currently an independent consultant based in Oak Ridge, Tennessee, and is retired director of the Chemical Technology Division at Oak Ridge National Laboratory, where he worked for over 37 years. His professional interests embrace all aspects of the nuclear fuel cycle. Prior to his work at Oak Ridge, he served as associate professor at the Georgia Institute of Technology and as chief nuclear chemist for Industrial Reactor Labs. Dr. Wymer is currently active on several National Research Council committees including the Committee on Environmental Management Technology and its Subcommittee on Tanks and the Committee on Electrometallurgical Technology. He is a fellow of the American Nuclear Society and a member of Sigma Xi and the American Institute of Chemical Engineers. He received his Ph.D. from Vanderbilt University.

APPENDIX B

Workshop on Barriers for Long-Term Isolation: Program Outline

WELCOME/INTRODUCTION

J. H. Clarke, ECKENFELDER INC., P. A. Witherspoon, University of California at Berkeley, J. Lehr, U.S. Department of Energy

Overview of Barrier Research, Design, and Implementation Within the DOE Complex G. Gee, Pacific Northwest National Laboratory

Overview of Barriers Research, Design, and Implementation Within the Private Sector R. D. Mutch, Jr., ECKENFELDER INC.

SURFACE BARRIERS I CHAIR—G. GEE, PACIFIC NORTHWEST NATIONAL LABORATORY

Ecology, Design, and Long-Term Performance of Surface Barriers: Applications at a Uranium Mill Tailings Site

*W. J. Waugh, Roy F. Weston, Inc.

Soil, Plant and Structural Considerations for Surface Barriers in Arid Environments: Application of Results from Studies in the Mojave Desert near Beatty, Nevada

*B. J. Andraski and D. E. Prudic, U.S. Geological Survey

Natural Physical and Biological Processes Compromise the Long-Term Performance of Compacted Soil Caps

*E. D. Smith, R. J. Luxmore, and G. W. Suter, Oak Ridge National Laboratory

SURFACE BARRIERS II CHAIR—D. DANIEL, UNIVERSITY OF TEXAS

Geomembranes in Surface Barriers

*R. K. Frobel, Ronald K. Frobel & Assoc.

*Presenter at the workshop.

SURFACE BARRIERS II (CONT.)

Earthen Materials in Surface Barriers

*C. H. Benson, University of Wisconsin

Comparison of Clay and Asphaltic Materials for Use as Low-Permeability Layers in Engineered Covers at the Rocky Flats Environmental Technology Site

*M. J. Glade, Parsons Engineering Science, Inc.

SUBSURFACE BARRIERS I CHAIR—R. D. MUTCH, JR., ECKENFELDER INC.

Synopsis of the National Academy of Sciences Evaluation of Ground Water Cleanup Alternatives

*L. Preslo, Earth Tech

Construction of Deep Barrier Walls for Waste Containment

*M. Mauro, Rodio Inc.

Self-Hardening Slurries and Stable Grouts Cement-Bentonite to IMPERMIX®

*G. R. Tallard, EnviroTrench Co.

SUBSURFACE BARRIERS II CHAIR—P. A. WITHERSPOON, UNIVERSITY OF CALIFORNIA AT BERKELEY

A Field Test of Permeation Grouting in Heterogeneous Soils Using a New Generation of Barrier Liquids

G. Moridis, P. Persoff, *J. Apps, L. Myer, P. Yen, & K. Pruess, Lawrence Berkeley National Laboratory

Sealable Joint Steel Sheet Piling for Ground Water Pollution Control

*D. Smyth & J. A. Cherry, University of Waterloo, Canada

Artificially Frozen Ground as a Subsurface Barrier Technology

*S. A. Grant, U.S. Army Corps of Engineers

*Presenter at the workshop.

APPENDIX C

Workshop on Barriers for Long-Term Isolation: Participants

Brian Andraski, U. S. Geological Survey
Robert Andrews, National Research Council
John Apps, Lawrence Berkeley National Laboratory
Donald Baker, Aquarius Engineering
Steve Balone, U.S. Department of Energy, Office of Environmental Restoration
Craig Benson, University of Wisconsin
Robert Campbell, Rocky Mountain Remediation Services
James Clarke, ECKENFELDER INC.
Ann Clarke, ECKENFELDER INC.
Bud Cook, U.S. Department of Energy
Julie D'Ambrosia, EnviroTech Associates, Inc.
Sandra Dalvit Dunn, Science & Engineering Assoc.
David Daniel, University of Texas (now at the University of Illinois)
Dennis DuPree, National Research Council
Brian Dwyer, Sandia National Laboratories
Ronald Frobel, R.K. Frobel & Assoc.
Glendon Gee, Pacific Northwest National Laboratory
Michael Glade, Parsons Engineering Science, Inc.
Jack Glavan, EA Engineering Science and Technology
Steven Grant, U.S. Army Corps of Engineers
Rozanne Huntley, Lockheed Idaho Technical Co.
Patricia Jones, National Research Council
Paul Kalb, Brookhaven National Laboratory
Thomas Kiess, National Research Council
John Lehr, U.S. Department of Energy, Office of Environmental Restoration
John Lommler, Jacobs Engineering Group Inc.
Mario Mauro, Rodio, Inc.
Mark Miller, Roy F. Weston, Inc.
Robert Mutch, ECKENFELDER INC.
Philip Nixon
Tom Nicholson, U.S. Nuclear Regulatory Commission
Thomas Ontko, Ohio Environmental Protection Agency
Steve Parikh, Bechtel Hanford Inc.

Paul Pettit, Fernald Environmental Restoration Management Project
Lynn Preslo, Earth Tech
Robert Romine, Batelle, Pacific Northwest National Laboratory
Ellen Smith, Oak Ridge National Laboratory
David Smythe, University of Waterloo
Leon Stepp, Ground water Tech Inc.
Tom Szymoniak, Jacobs Engineering Group
Pieter Tackenberg, Oyo Corporation USA
Gilbert Tallard, EnviroTrench Co.
Herb Ward, Rice University
John Ward, consulting environmental engineer
William Waugh, Roy F. Weston, Inc.
Nancy Weatherup
Madeline Williams
Martha Windsor, Missouri Department of Natural Resources
Paul Witherspoon, University of California, Berkeley
Paul Zielinski, U.S. Department of Energy, Office of Environmental Restoration

APPENDIX D

Workshop on Barriers for Long-Term Isolation: Papers Presented

This appendix contains a collection of individually authored background papers that were presented at the August 14-15, 1995, *Barriers for Long-Term Isolation Workshop*, jointly sponsored by the Department of Energy and the Committee on Remediation of Buried and Tank Wastes of the National Research Council*. These papers have not been reviewed or approved by the National Research Council.

CONTENTS

(Papers appear in order that they were presented.)

* The original papers submitted by the authors were edited by the committee's staff and prepared in a common format

DEVELOPMENT AND TESTING OF PERMANENT ISOLATION SURFACE BARRIERS AT THE HANFORD SITE

Glendon W. Gee, Pacific Northwest National Laboratory, Richland, Washington; N. Richard Wing, Richland; and Anderson L. Ward, Pacific Northwest National Laboratory, Richland

ABSTRACT

Engineered barriers are being developed to isolate wastes disposed of near the earth's surface at the U.S. Department of Energy's (DOE) Hanford Site, near Richland, Washington. The surface barriers use engineered layers of natural materials to create an integrated structure with redundant protective features. For example, one current design incorporates a capillary barrier as well as a low-permeability asphalt component. The natural construction materials (e.g., fine-soil, sand, gravel, riprap, asphalt) have been selected to optimize barrier performance and longevity. The objective of current designs is to use natural materials to develop a maintenance-free surface barrier that isolates wastes for a minimum of 1,000 years by limiting water drainage to near-zero amounts; reducing the likelihood of plant, animal, and human intrusion; controlling the exhalation of noxious gases; and minimizing erosion-related problems.

A multiyear barrier development program was started at the Hanford Site in 1985 to develop, test, and evaluate the effectiveness of various barrier designs. A team of engineers and scientists have directed the barrier development effort. ICF Kaiser Hanford Company (KH) has provided design support for barrier-related projects, and Westinghouse Hanford Company (WHC), Bechtel Hanford Incorporated (BHI), and the Pacific Northwest National Laboratory (PNNL) have provided engineering and scientific support to the development effort. A prototype barrier, incorporating all essential elements of a long-term surface barrier, was constructed at the Hanford Site in 1994 and is currently being monitored.

This paper provides an overview of the barrier development work being conducted at the Hanford Site and the functional performance of the permanent isolation surface barrier. The paper focuses on the control of water movement into and through the barrier and discusses how various aspects of the barrier have been purposely designed to minimize water intrusion into underlying buried wastes. Field tests conducted on individual components of the barrier and more recently on the completed prototype barrier show that the combination of a capillary barrier (designed to store water and subsequently enhance near-surface water loss via evapotranspiration) and an asphalt sublayer (to shed water from side slope drainage) can be effective in keeping water from draining into underlying wastes. Extreme precipitation events, including 1,000-year storms, are accommodated by use of the multiple-layer design. Eight years of testing of individual components and 2 years of testing of a full-scale prototype surface barrier are providing engineering parameters needed for design of extensive cover systems planned for the Hanford Site.

INTRODUCTION

The In-Place Remediation Alternative

Permanent isolation surface barriers have been proposed for use at the U.S. Department of Energy's (DOE) Hanford Site, near Richland, Washington, to isolate and dispose of certain types of waste in-place. The exhumation and treatment of wastes may not always be the preferred alternative in the remediation of a waste site. In-place disposal alternatives, in certain circumstances, may be the most desirable alternative to use in the protection of human health and the environment. The implementation of an in-place disposal alternative probably will require some type of protective covering that will provide long-term isolation of the wastes from the accessible environment. (Even if the wastes are exhumed and treated, a long-term barrier may still be needed to dispose of the wastes adequately.) Currently, no "proven" long-term barrier is available. However, the Hanford Site Permanent Isolation Surface Barrier Development Program (BDP), which is described below, was organized to develop the technology needed to provide a long-term surface barrier capability for the Hanford Site and elsewhere.

The Hanford Site Permanent Isolation Surface Barrier Development Program

The Hanford Site Permanent Isolation Surface Barrier Development Program (BDP) was organized in 1985 to develop, test, and evaluate the effectiveness of various barrier designs. The BDP was supported by DOE and consisted of a team of engineers and scientists from Westinghouse Hanford Company (WHC) and the Pacific Northwest National Laboratory (PNNL), which directed the barrier development effort. ICF Kaiser Hanford Company (KH) provided design support for numerous barrier-related projects.

Fifteen groups of tasks were identified by the barrier development team to resolve the technical concerns and complete the development and design of protective barriers (Wing 1993). These major barrier development task groups are water infiltration control, biointrusion control, erosion/deposition control, physical stability testing, human interference control, barrier construction materials procurement, prototype barrier designs and testing, model applications and validation, natural analog studies, long-term climate change effects, interface with regulatory agencies, Resource Conservation and Recovery Act of 1976 (RCRA) equivalency, technology integration and transfer, project management, and final design. Figure 1 illustrates how the information and data generated within each of the task groups are input into the final design(s) of the barrier.

The information and insights gained from the development tasks previously mentioned have enabled the barrier program to progress to the point where design, construction, and testing of a full-scale prototype barrier has been possible. The full-scale prototype barrier is providing engineers and scientists with insights and experience on barrier design, construction, and performance that have not been possible with the individual tests and experiments conducted to date in the program. Construction of the prototype was completed in August of 1994, and testing and monitoring was initiated at that time. The testing and monitoring of the prototype barrier is planned to last for a minimum of 3 years. A comparison of the Hanford barrier design and testing program with other DOE sponsored surface barrier designs has been reported recently (Daniel et al., 1996). One of the major differences between the Hanford barrier development activities and those at other sites has been the emphasis at Hanford on design and testing of a surface barrier that has a high probability of lasting for 1,000 years or more. For example, tests for the prototype barrier have been designed to evaluate barrier surface and side slope response to 1,000-year storm events. These and other tests will be described in the following sections.

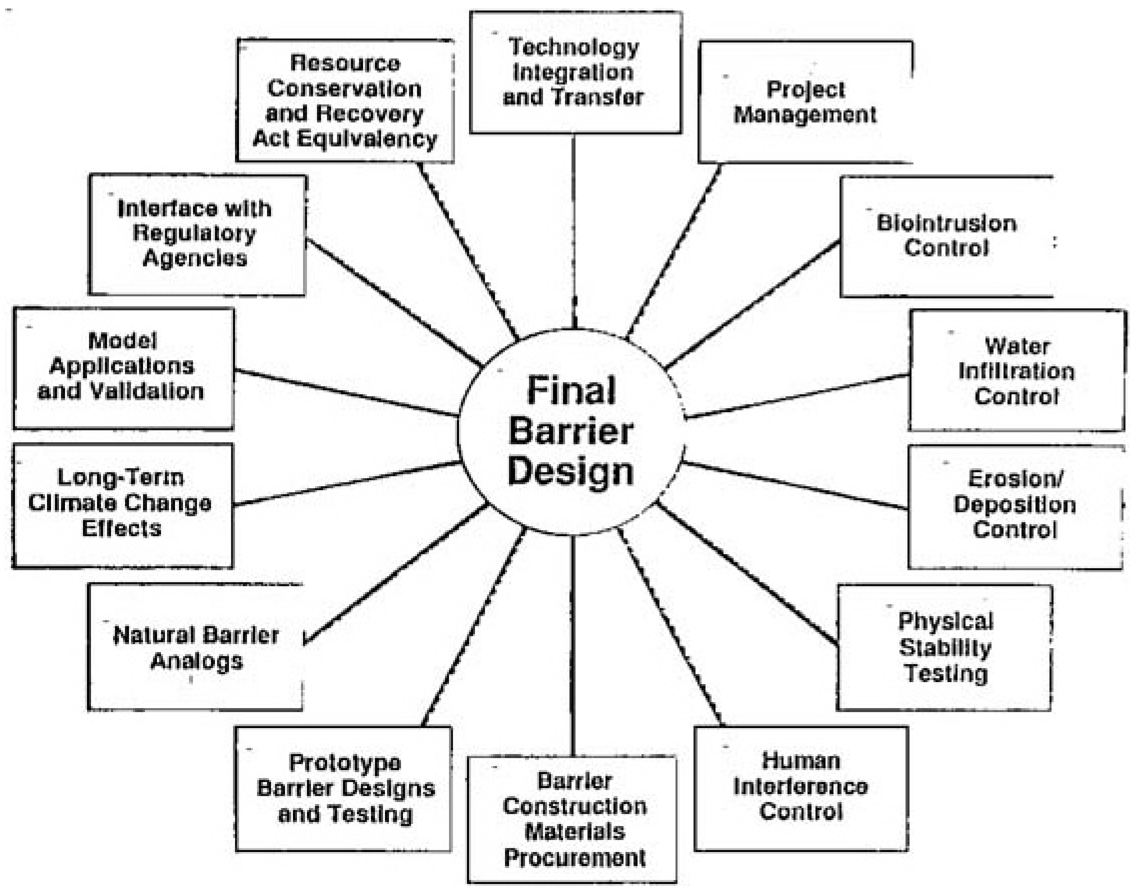

FIGURE 1
Barrier Development Tasks

FUNCTIONAL REQUIREMENTS FOR THE BARRIER

Much of the waste that would be disposed of by using in-place isolation techniques is located in subsurface structures, such as solid wasteburial grounds, tanks, vaults, and cribs. Unless protected in some way, the wastes could be transported to the accessible environment via the following pathways (Figure 2).

- Water infiltration. The infiltration and percolation of water through the waste zone, resulting in the leaching and subsequent transport of mobile radionuclides and other contaminants to the water table.
- Biointrusion. The penetration of deep-rooting plants and burrowing animals into the waste zone below. The deep-rooting plants could draw radionuclides and other contaminants into their root systems and subsequently translocate the contaminants to the above-grade portion

of the plant. The contaminants in the above-grade portion of the plant could then be dispersed by animals that eat the plants or by wind. Animals burrowing directly into the waste zone could contact contaminants and subsequently bring them to the earth's surface as part of the soil castings. Erodible loose soil cast to the surface by burrowing animals could contribute to accelerated erosion of the fine-soil surface layer. Also, the presence of animal burrows may provide preferential pathways for infiltrating water to gain access to the waste zone.

- Wind and water erosion. The removal of the surface soils at a waste site as a result of erosive forces. Erosion-related problems could provide a direct pathway for contaminant transport if the erosive forces are strong enough to remove the surface soils and expose the buried wastes to the accessible environment. A more probable scenario is for wind and water erosion to reduce the thickness of soils overlying a waste zone so another transport pathway (i.e., water infiltration) becomes a more serious concern.
- Human interference. The inadvertent or intentional intrusion of humans into the waste sites (assuming institutional control is lost) and subsequent dispersion of contaminants. The barrier will not be required to be designed to deter the intentional human intruder.

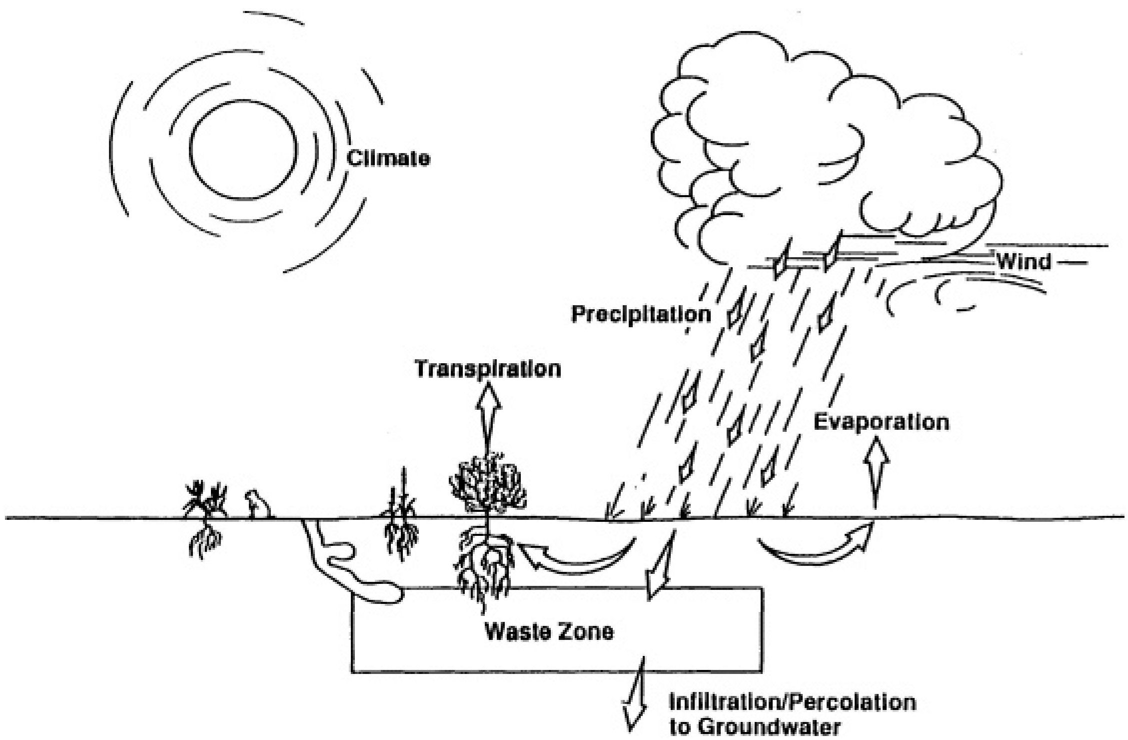

FIGURE 2
Potential Problems of the Current Waste Management Situation.

- Gaseous release. The diffusion of noxious gases from the waste zone to the accessible environment.

Permanent isolation surface barriers have been proposed to protect wastes, disposed of in-place, from the transport pathways identified. Surface markers, used to inform future generations of the nature and hazards of the buried wastes, are being considered for placement around the periphery of the waste sites. In addition, throughout the protective barrier, subsurface markers could be placed to warn any inadvertent human intruders of the dangers of the wastes below.

The protective barrier design consists of a fine-soil layer overlying other layers of coarser materials such as sands, gravels, and basalt riprap (Figure 3). Each of these layers serves a distinct purpose. The fine-soil layer acts as a medium in which moisture is stored until the processes of evaporation and transpiration recycle any excess water back to the atmosphere. The fine-soil layer also provides the medium for establishing plants that are necessary for transpiration to take place. The coarser materials placed directly below the fine-soil layer create a capillary break that inhibits the downward percolation of water through the barrier. The placement of fine soils directly over coarser materials also creates a favorable environment that encourages plants and animals to limit their natural biological activities to the upper, fine-soil

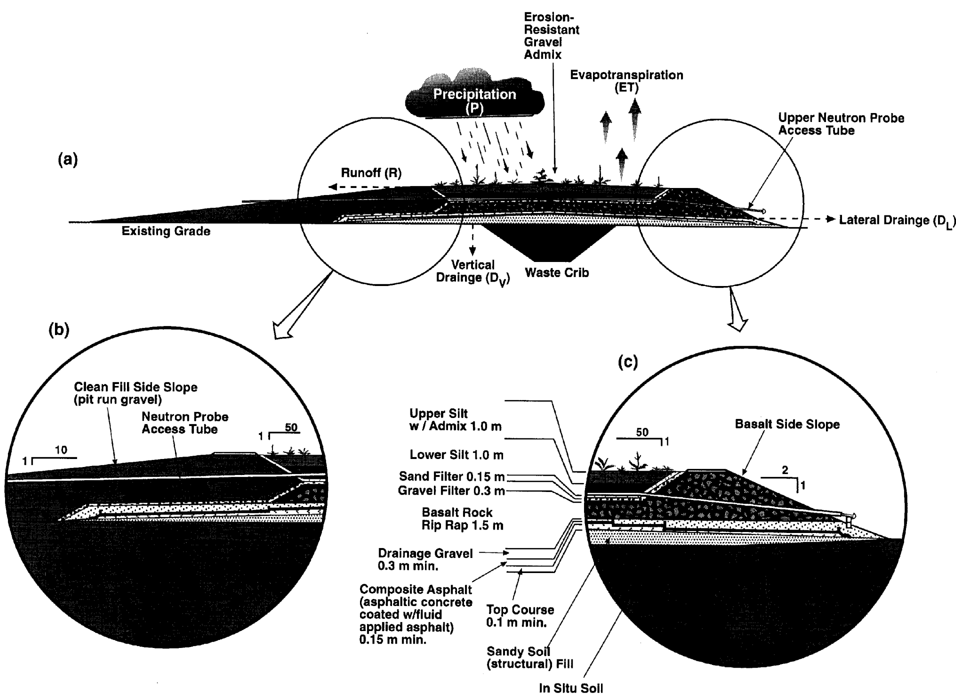

FIGURE 3
Barrier Cross Section.

portion of the barrier, thereby reducing biointrusion into the lower layers. The coarser materials also help to deter inadvertent human intruders from digging deeper into the barrier profile. Low-permeability layers, placed in the barrier profile below the capillary break, also are used in the protective barriers. The purpose of the low-permeability layers is (1) to divert away from the waste zone any percolating water that gets through the capillary break and (2) to limit the upward movement of noxious gases from the waste zone. The coarse materials located above the low-permeability layers also serve as a drainage medium to channel any percolating water to the edges of the barrier.

Because of the need for the barrier to perform for at least 1,000 years without maintenance, natural construction materials (e.g., fine-soil, sand, gravel, cobble, crushed basalt riprap, asphalt) have been selected to optimize barrier performance and longevity. Most of these natural construction materials are available in large quantities on the Hanford Site and are known to have existed in-place for thousands of years or longer (e.g., basalt). In contrast to the natural construction materials, the ability of synthetic construction materials to survive and function properly for 1,000 years is not known. Because of this uncertainty, synthetic construction materials cannot be relied upon to perform satisfactorily (or even exist) over centuries or millennia, so were not given any credit in the design.

Because of the desire for the barrier to remain maintenance free, an understanding of how natural processes affect barrier performance enables a design to be developed that meets performance objectives passively. This paper discusses the natural processes acting on the permanent isolation barrier, as well as the engineered features of the barrier that have been designed to protect buried wastes from the natural processes. Specifically, this paper focuses on how various barrier components are used to protect buried wastes from water has been incorporated into the design of the barrier.

WATER INFILTRATION AND PERCOLATION CONTROL

The control of water infiltration and percolation through the barrier depends on the amount of water available. The amount of water available depends on the climate. Because of the long time frame during which permanent isolation surface barriers must function (1,000+ years), the climatic conditions acting on the barrier may change.

Current Climatic Conditions

Since 1912, the amount of precipitation collected at the Hanford Meteorological Station (HMS) has averaged 160 mm (6.30 in.) annually (Stone et al., 1983). Most of this precipitation (67 percent) is received in the winter months (October through March), while only 13 percent is received July through September. About 38 percent of all precipitation is in the form of snow during the months of December through February. Total annual snowfall averages 335 mm (13.2 in.). Based on extreme-value analysis of Hanford Site climatological records from 1947 through 1969, the 60-min, 100-yr storm would result in 20.6 mm (0.81 in.) of precipitation, and the 24 hour, 1,000-yr storm would result in 68.1 mm (2.68 in.). No records have been kept for time periods less than 60 min. However, the rain gauge chart for June 12, 1969, shows that 14.0 mm (0.55 in.) of precipitation was collected during a 20-min period. In addition, an afternoon thunderstorm on June 29, 1991, dumped 11.2 mm (0.44 in.) of rain at the HMS in only 10 min.

The average monthly temperature at the HMS is 11.7 °C (53.0 °F). However, January monthly temperatures average -1.5 °C (29.3 °F), and July monthly temperatures average 24.7 °C (76.4 °F). Temperatures reach 32.2 °C (90 °F) or above an average of 55 days/yr, while minimum temperatures of 21.1 °C (70 °F) or above occur only an average of 8 days/yr.

The prevailing wind direction at the Hanford Site is either WNW or NW in every month of the year. The strongest winds are from the SSW, SW, and WSW. June, the month of highest average speed, has fewer instances of hourly averages exceeding 13.9 m/s (31 mph) than December, which has the lowest average speed. When extreme value analysis of peak gusts is performed on data from 1945 through 1980 (collected at an elevation of 15.2 m [50 ft] at the HMS), the 100-year return period for a peak wind gust is estimated to be 38 m/s (85 mph). The maximum gust recorded in the data set was measured in January 1972 at 35.8 m/s (80 mph). The 1,000-year peak gust is estimated to be 44 m/s (99 mph).

Projected Climatic Conditions

Projections of the long-term variability in the Hanford Site's climate have been developed so that barrier performance over its projected design life (1,000+ yr) could be predicted (Petersen, Chatters, and Waugh, 1993). One of many activities that has been performed as part of the climate-change task is the extraction of a pollen record from the lake-bottom sediments of Carp Lake, located near Goldendale, Washington, southwest of the Hanford Site (Wing et al., 1995). This pollen record, dating back 75,000 years or more, enables scientists to determine the types of vegetation that once grew in the vicinity of the lake. With an understanding of the vegetation species that once grew, scientists then are able to predict the climatic conditions that had to exist to support the growth of the types of vegetation determined from the pollen record.

Referring to the climatic conditions of the Columbia Basin inferred from the Carp Lake pollen record, Petersen et al. (1993) states the following:

> Throughout the record, mean annual precipitation ranged from 25 to 50% below modern levels...to 28% above...At no time did precipitation levels reach three times that of present-day. Three times modern precipitation has been taken as an upward bounding condition of precipitation to be used in barrier performance assessment...

The three times average annual precipitation (3X) projection has been used since November 1990 as the upper bound when applying supplemental precipitation to field test plots.

Designing a Barrier for Drainage and Percolation Control

Based on the climatological conditions and projections discussed previously, three methods are described for controlling the infiltration and percolation of water through a protective barrier: (1) engineering the barrier surface to maximize runoff, while at the same time minimizing erosion, (2) incorporating a capillary break (or capillary barrier) within the integrated barrier system, and (3) incorporating a low-permeability, umbrella-like layer below the capillary break to shed any infiltrating/percolating water away from the waste zone.

Runoff

The amount of water available for infiltration and percolation is a function of the amount of precipitation that falls on the barrier surface, minus the amount of water that runs off the barrier surface and away from the structure. The surface of the protective barrier has been designed with a slight slope or crown to maximize the runoff of water from the barrier surface while minimizing the erosion of the fine-soil layer. Tests have been conducted to aid in the design of this feature (Gilmore and Walters, 1993). The current barrier design uses a 2 percent sloped surface.

Capillary Barrier

The protective barrier is designed and constructed with a fine-soil layer overlying a layer of coarser materials (e.g., sands and/or gravels). The differences in textures between the barrier materials at this interface provide a capillary barrier for percolating water (Figure 3).

In an unsaturated system, the capillary pressures are much less than atmospheric pressure. For significant quantities of water to flow into and through the coarser sublayers, the water pressure must be raised almost to atmospheric pressure. The overlying fine-textured soils must become nearly saturated for the water pressure to approach atmospheric pressure and allow water to flow into the sublayers. This resistance to drainage increases the storage capacity of the overlying fine-textured soil. Keeping the water in the fine-textured layer provides time for the processes of evaporation and transpiration to remove it.

The critical component of the capillary barrier is the fine-soil layer. The fine-soil layer must be able to retain infiltrating precipitation until the processes of evaporation and transpiration can recycle the water back to the atmosphere. The removal of water from a barrier's fine-soil layer is increased significantly by the presence of vegetation. After the construction of a barrier, desired stands of vegetation on the barrier surface will be engineered and cultivated. However, during a barrier's design life, the engineered vegetative cover may be disturbed at times by range fires, drought, disease, or some other phenomenon. Because of the design objective to create a maintenance-free barrier, revegetating the barrier surface with the desired plant species may not always be possible. In these circumstances, a climax community of vegetation may not reestablish itself on the barrier surface for a long time (Waugh et al., 1994; Link et al., *1995).* Although the presence of vegetation on the barrier surface is ideal, the results of lysimeter tests, presented in the following paragraphs, provide interesting evidence that the capillary barrier concept performs effectively, even in the absence of vegetation.

The capillary barrier concept has been tested for several years at the Field Lysimeter Test Facility (FLTF) (Figure 4). Results from these tests indicate that the capillary barrier functions as designed. During the first 3 years of testing, twice the annual average precipitation (320 mm, or 2X) was added to lysimeters simulating a wetter climate. During the next 2 years, three times the annual average precipitation (480 mm, or 3X) was added to the same lysimeters. During this entire 5-yr testing period, water losses from evaporation and transpiration exceeded water gains by precipitation and irrigation-even for the lysimeters receiving treatments representative of wetter climatic conditions. These results were observed for both vegetated and unvegetated lysimeters. Although the vegetated lysimeters were most effective at removing soil water, even the soil water stored in the unvegetated lysimeters decreased during the 5-year test period. No drainage was collected from any of these lysimeters.

The capillary barrier concept does have its limits, however. During the commencement of the sixth year of testing, drainage was observed (during the unusually wet winter of 19921993, when record snowfalls occurred) from several unvegetated lysimeters receiving supplemental precipitation (Campbell et al., 1990). The routine supplemental irrigation treatments, when combined with the unusually large amount of precipitation received during that winter, caused more than 3X (>520 mm) precipitation to be added to these lysimeters. The net result was that the storage capacity of the fine-soil reservoir was exceeded and the unvegetated lysimeters began draining. The lysimeters with vegetation did not drain, even though they received the same amount of moisture (520 mm).

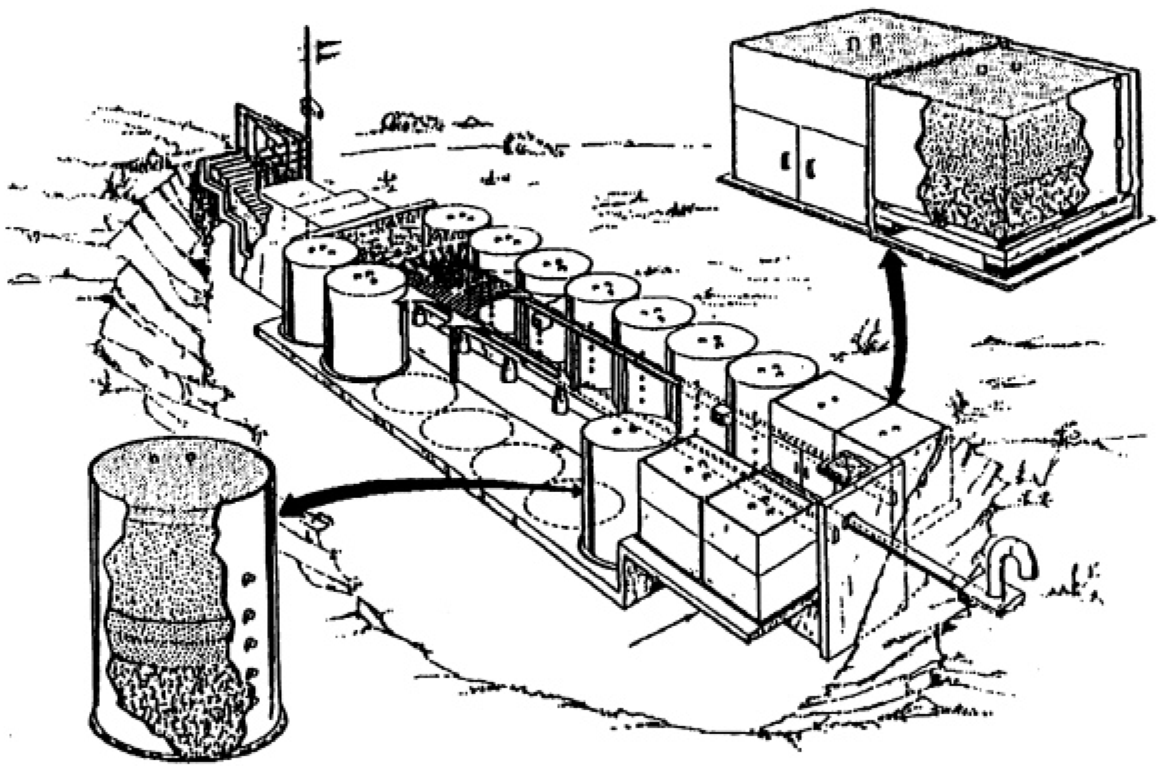

FIGURE 4
The Field Lysimeter Test Facility: Schematic View.

Because of earlier tests conducted on two of the lysimeters at the FLTF, some understanding existed of the limits of the capillary barrier's performance. In two of the drainage lysimeters at the FLTF, enough water was added to force water to break through the capillary barrier. As expected, virtually no water passes through the capillary barrier until the soil approaches saturation and pore pressure approaches zero. Once breached, the capillary barriers in the lysimeters drained only slowly until they reached a stable water content, resulting in a storage of over 500 mm, which was almost twice as high as that normally held (-250 mm) by the silt loam soil against gravity (Gee et al., 1993a).

Observations at the FLTF indicate that both vegetated and unvegetated barrier systems are able to store and evapotranspire at least three times the annual average precipitation -simulating the upper bound of projected climate changes at the Hanford Site during the next 1,000 years. Vegetated barrier systems are able to accommodate even greater amounts of precipitation because of the water extraction capabilities of plants, thereby providing increased storage capacity.

Low-Permeability Layers

The basic premise of the capillary barrier concept is that most, if not all, of the meteoric water that infiltrates the barrier surface can be returned to the atmosphere by surface evaporation and plant transpiration. However, for periods of unusually heavy, intense, and/or prolonged precipitation, the water-holding capacity of the fine soils may be exceeded, allowing water to break through the capillary barrier before it can be recycled back to the atmosphere. Unless checked in some way, the water would be free to migrate through the barrier and into the waste zone. Also, coarse-textured, sparsely vegetated side slopes will allow significant water infiltration. As a means of restricting the percolating water from the waste zone, a low-permeability component is placed strategically within the barrier profile below the capillary barrier to divert percolating water away from the buried waste. This diversion barrier is constructed of a material(s) with low permeability. The BDP is using asphaltic concrete and polymer-modified asphalt to create the low-permeability layer.

Several types of asphalt have been used in tests conducted by the BDP. Based on recommendations supported by laboratory test results, lysimeter studies at the Small-Tube Lysimeter Facility (STLF) have used two asphalt formulations: (1) hot rubberized asphalt and (2) an admixture of cationic asphalt emulsion and concrete sand containing 24 wt% residual thick asphalt. These asphalt formulations have been effective in limiting percolation (Freeman and Gee, 1989). A third type of asphalt formulation is being used in the prototype barrier. This formulation consists of a composite layer of asphaltic concrete (with ¬8% asphalt and very low voids) overlain by polymer-modified asphalt (5.1-mm thick). There are two major advantages to this third asphalt formulation. The first advantage is its high mechanical strength. The second advantage is that composite layers have been shown to provide much lower permeabilities than one layer alone (Daniel and Trautwein, 1991). Tests of the permeability of the asphaltic components indicate that the hydraulic conductivity of the combined components will be less that $1x10(-1^{1})$ cm/s (Petersen, Link, and Gee, 1995).

The low-permeability layers, in concert with (1) the engineered surface that maximizes runoff and (2) the capillary barrier, which blocks the downward movement of percolating water, is expected to perform in such a way that near-zero drainage rates through the barrier can be achieved.

BIOINTRUSION EFFECTS ON WATER INFILTRATION CONTROL

The presence of animal burrows (for both small and large-mammals) on the surface of the barrier has been a concern for scientists and engineers on the BDP. The presence of animal burrows could provide preferential pathways or conduits through which infiltrating water could bypass the fine-soil layer of the protective barrier and subsequently migrate deeper into the barrier profile or possibly into the waste zone below. Tests have been conducted to assess the impact of burrowing animals on the infiltration and percolation of water through protective barriers (Cadwell, Eberhardt, and Simmons, 1989; Landeen et al., 1990; Landeen, 1990, 1991, 1994). The results of these tests have provided interesting results.

From the results of lysimeter tests performed at the Animal Intrusion Lysimeter Facility (Figure 5), the presence of small-mammal burrows did not appear to have a significant influence on the deep percolation of water through the barrier (Cadwell et al., 1989; Landeen et al., 1990; Landeen, 1990, 1991, 1994). During the summer months, more water was lost from plots with animal burrows than from plots where no animal burrows were present. During the winter months, both the plots with animal burrows and the control plots gained water. In addition, water did not infiltrate below –1 m (36 in.), even though burrow depths always exceed –1.2 m (48 in.). The lack of significant water infiltration at depth and the overall water loss in the lysimeter plots occurred despite the following worst-case conditions:

- no vegetative cover (no water loss through transpiration);
- no water runoff (all incipient precipitation is contained);
- the burrow densities in the lysimeters are greater than the burrow densities found in "natural" settings;
- extreme rainfall events are applied frequently (three 100-year storm events in 3 months); and
- animals burrow deeper in the lysimeters than in "natural" settings.

The overall water loss from soils with small-mammal burrows appears to be enhanced by a combination of soil turnover and subsequent drying, ventilation effects from open burrows, and high ambient temperatures.

Similar water loss results have been observed for experiments conducted on existing large-mammal burrows found in a natural setting on the Arid Land Ecology Reserve at the Hanford Site. The large-mammal burrows studied were excavated by coyotes and badgers in search of prey. The soils into which the burrows were excavated consist of a silt loam similar to the sediments that will be used to construct surface barriers.

Large-mammals do appear to cause increased deep penetration of water in the fine-soil layer, but much of this water was later removed by a dense stand of vegetation (primarily mustards) that grew vigorously in the vicinity of the burrow. The density of the vegetation near the badger burrow was significantly greater than in adjacent undisturbed soils away from the burrows. The soil under the burrows was actually drier in midsummer than the adjacent soils away from the burrows.

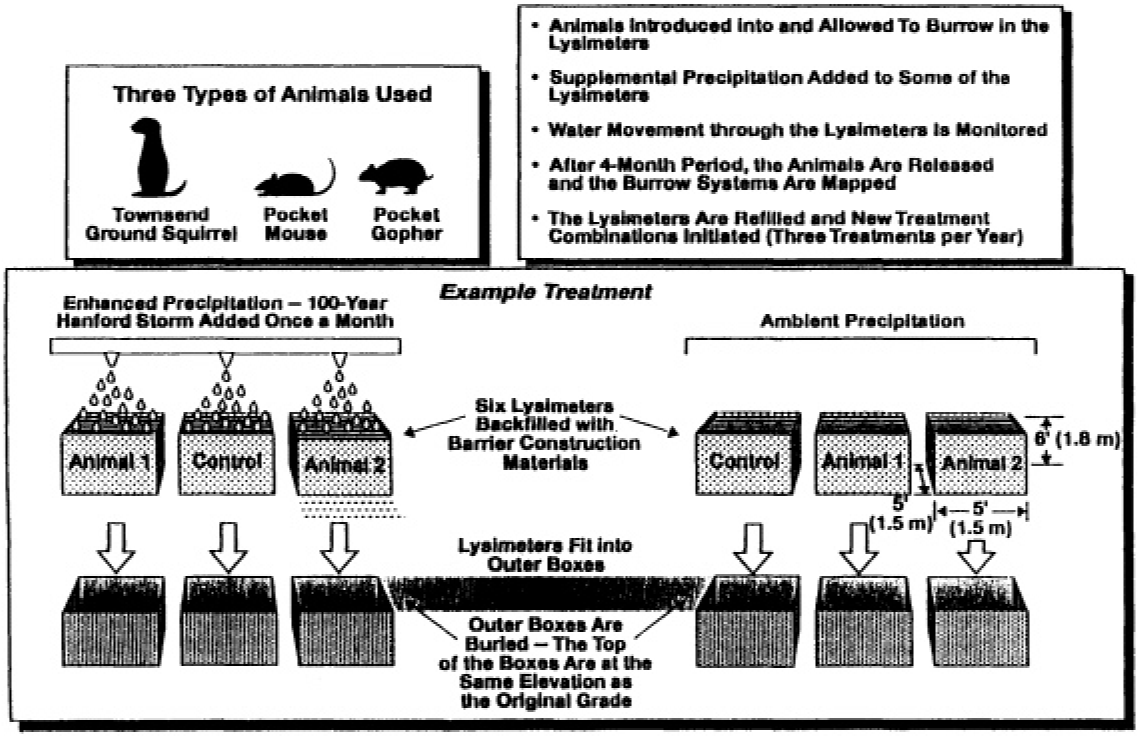

FIGURE 5
Animal Intrusion Lysimeter Facility: Experimental Design.

Other observations were made with the large-mammal burrows. Link et al. (1995) reported that characterization of existing marked badger burrows indicated that abandoned burrows are only temporary surface features that soon fill with soil and organic debris. Many of the badger burrows also connect with small-mammal burrows. The smallmammals appear to be instrumental in filling the larger burrows by casting soil into the openings. More importantly, the smaller burrows provide an opportunity for runoff that enters large burrows to drain.

The current barrier design does not include features to reduce the hazards of deep water penetration through large-mammal burrows because there has been no demonstrated need, based on work conducted to date. In addition, the presence of the low-permeability asphalt layers lower in the barrier profile will act like an umbrella to shed any percolating water away from the waste.

WIND AND WATER EROSION CONTROL

Protective barriers are being designed to minimize the effects of wind and water erosion of the surface cover, side slopes, and toe of a protective barrier. Understanding the effects that erosive forces (and the techniques being considered to stabilize the surface soils from the erosive forces) have on barrier performance with regard to water infiltration is important. For example, the erosive forces acting on the barrier could be strong enough to reduce the thickness of the fine-soil reservoir such that the moisture retention capability of the barrier is reduced. With reduced capacity for moisture retention, the infiltration of water through the barrier may become a greater concern. Also, the types of erosion control techniques used to stabilize the surface soils from erosive forces also may affect water infiltration through the barrier adversely. The following paragraphs describe the results of wind-and-water erosion studies with regard to water infiltration concerns.

Throughout the majority of its design life, vegetation will be growing on the surface of the protective barrier (Waugh et al., 1994). The presence of vegetation on the barrier surface will reduce the amount of fine-soil lost from the barrier by wind and water erosion significantly. However, to protect the barrier surface during periods when the vegetative cover is disturbed by range fires, drought, disease, or some other phenomenon, surface gravels will be admixed into the surface of the protective barrier.

Studies conducted in the PNNL Aerosol Wind Tunnel Research Facility have shown that field wind erosion stresses and surface conditions can be replicated in the wind tunnel. These studies have provided significant input for the design of protective barriers (Ligotke and Klopfer, 1990; Ligotke, 1993). For example, wind-tunnel tests have demonstrated that admixtures and layers of gravels (with partial sizes of 3- to 7-mm in diameter) provided superior surface protection. The best gravel admixtures reduced surface deflation rates by greater than 96% (compared to unprotected soil). Also, rounded river rock and gravel-sized, angular crushed rock provided equal surface protection, expanding the possibilities of finding adequate source materials for the least expense.

In addition to the wind-erosion studies, other studies have been conducted to optimize the design of the barrier surface to resist water erosion (Gilmore and Walters, 1993). Gilmore and Walters (1993) have stated the following:

> ...[the] most dominant factor in reducing runoff and sediment yield was the presence of vegetation cover...Another factor that has significance is the amount of antecedent moisture in the soil. For very dry conditions representative of the Hanford summer climate, runoff is greatly reduced...The dry soil conditions coupled with the presence of vegetation can reduce surface runoff to a minimal amount, less than 1% of the applied rainfall, with very little sediment yield. Gravel admix with the natural vegetation cover and dry soil conditions reduced the sediment yield to the lowest observed levels for these tests. An established vegetation cover with gravel admix could possibly reduce sediment yield by 10-100 times for equivalent storms.

From the studies cited previously, the presence of gravel admix has been demonstrated to be effective in reducing the deflation of fine soils from the barrier surface by wind and water erosion. The amount of gravel used to stabilize the surface of the protective barrier is a critical design consideration from a water infiltration perspective. If too much gravel is mixed into or spread onto the fine-soil surface, plant transpiration and surface evaporation could be reduced significantly, which would increase the potential for water drainage through the barrier. Conversely, if too little gravel is used, the ability of the gravel admix to reduce wind and water erosion may be limited severely.

At the Small-Tube Lysimeter Facility (STLF), the water storage and evapotranspiration in a permanent isolation barrier were determined to be affected significantly by the types of materials used on the barrier surface (Sackschewsky et al., 1995). The lysimeters at the STLF were backfilled with materials to test how various erosion-control surface treatments affect soil moisture balance (Figure 6). Data collected at the STLF show that when gravel is spread onto a fine-soil surface instead of being tilled or pug milled into it, plant transpiration and surface evaporation are reduced significantly, which increases the amount of water available for drainage through the barrier. Similar results were observed for lysimeters with a layer of dune sand overlying fine-textured soils. Drainage has occurred only in irrigated graveland sand-covered lysimeters. Because of these results, the use of admix gravels rather than gravel mulches is recommended to avoid water infiltration.

ASSESSING WATER INFILTRATION THROUGH THE PROTOTYPE BARRIER

The prototype surface barrier constructed at the Hanford Site in 1994 is shown in cross section and plan view in Figure 3. In addition to testing the performance of a capillary barrier design (fine-soil over coarse), the prototype is being used to test two different side slope designs: (1) a relatively flat apron (10:1, horizontal:vertical) of clean-fill materials (commonly called a clean-fill dike) and (2) a relatively steep (2:1) embankment of fractured basalt riprap (Gee et al., 1993b). From November 1994 through October 1996, soil (capillary barrier) plots on the northern half of the prototype barrier were subjected to a 3X irrigation regime. This treatment included application of sufficient irrigation water on March 24, 1995, and March 25, 1996, to mimic a 1000-year storm event (70 mm of water) and periodic applications to achieve a test design of 480 mm/yr for the entire water year (November 1-October 31).

Shrub and grass cover was established on the prototype surface successfully. Shrubs were planted at a density of approximately two plants per square meter in November 1994. Sagebrush *(Artemsia tridentata)* and rabbitbrush *(Chrysothamnus nauseosus)* were planted in a ratio of 4:1, sagebrush to rabbitbrush. Survival rate of the transplanted shrubs has been remarkably high; 97 percent for sagebrush and 57 percent for rabbitbrush (Gee et al., 1996). A heavy invasion of tumbleweed *(Salsola kali)* occurred in 1995 but was virtually absent in 1996. Grass cover, consisting of twelve varieties of annuals and perennials (including cheatgrass, several bluegrasses, and bunch grasses), dominated the surfaces, particularly those that were irrigated. Approximately 75 percent of the surface was covered by vegetation; a cover value typical of shrub-steppe plant communities. In all respects, the vegetated cover appeared to be healthy and normal. There was a surface response to irrigation, with nearly twice as much grass cover on the irrigated surfaces compared to the non-irrigated surfaces (Gee et al., 1996).

Prototype water-storage data are shown in Figure 7. All irrigation and natural precipitation plus all available stored soil water was removed via evapotranspiration (i.e., combined evaporation from plant and soil surfaces) during the first year of surface barrier operation. Water was removed from the entire soil profile so that by late summer (September) of

1995, water contents in both irrigated and non-irrigated plots had reached a relatively uniform lower limit of about 5 volume-percent throughout the soil profile. Correspondingly, water storage was reduced to levels near 100 mm (i.e., lower limit of plant-available water), for both the irrigated and non-irrigated soil surfaces. This is about one-fifth the amount of water required for drainage. Based on these observations and considering the irrigation treatment to represent the extreme in wet climate, the soil cover would not be expected to drain, even under the wettest Hanford climate conditions.

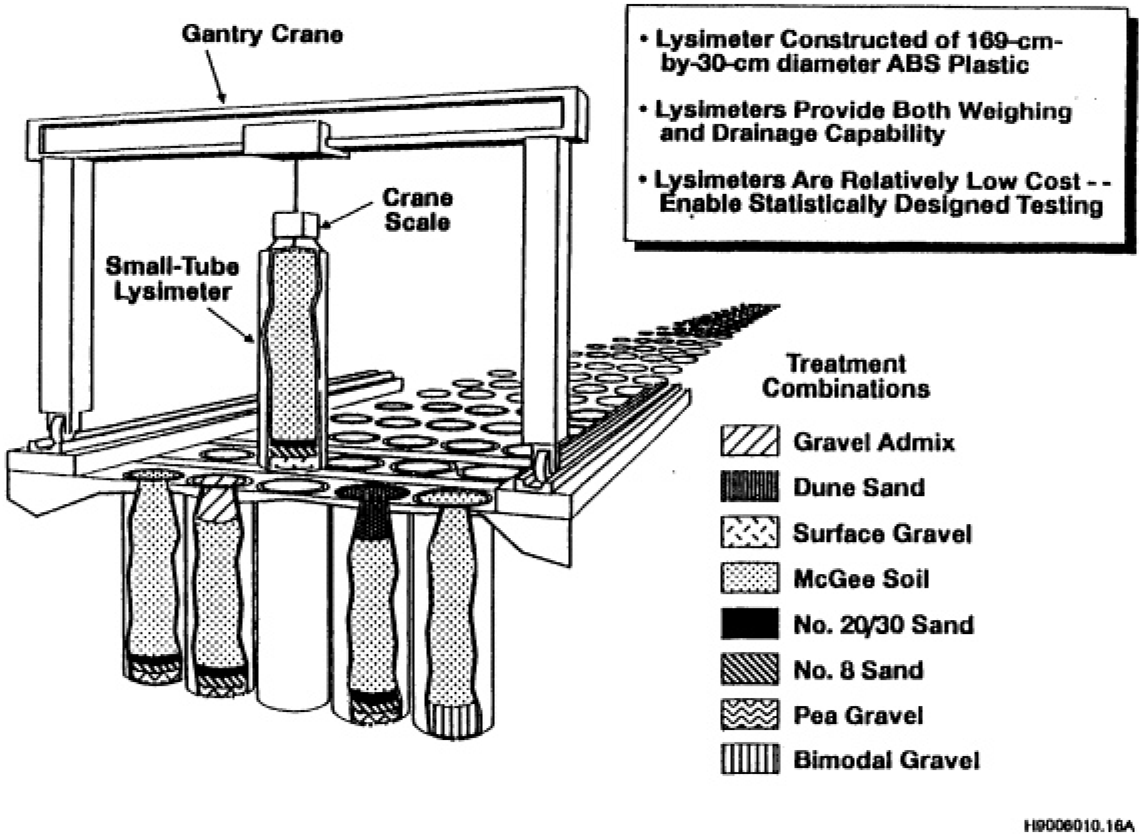

FIGURE 6
Small-Tube Lysimeter Facility: Experimental Design.

Figure 7 shows that in 1996, water from the second 1000-year storm was also removed from the soil profile by evapotranspiration, thus demonstrating the continued positive benefits of having vegetation on the barrier surface. Evapotranspiration for the irrigated surfaces was nearly double that for the non-irrigated (ambient) surfaces (Figure 8), suggesting that vegetation is capable of adjusting to rate limiting processes, such as water applications. It is apparent that the capacity of vegetation for water consumption has not been exceeded even at the 3X precipitation rates, even after the second year of testing. This further supports the hypothesis that the combination of vegetation and soil storage capacity is more than sufficient to remove all applied water under the imposed test conditions.

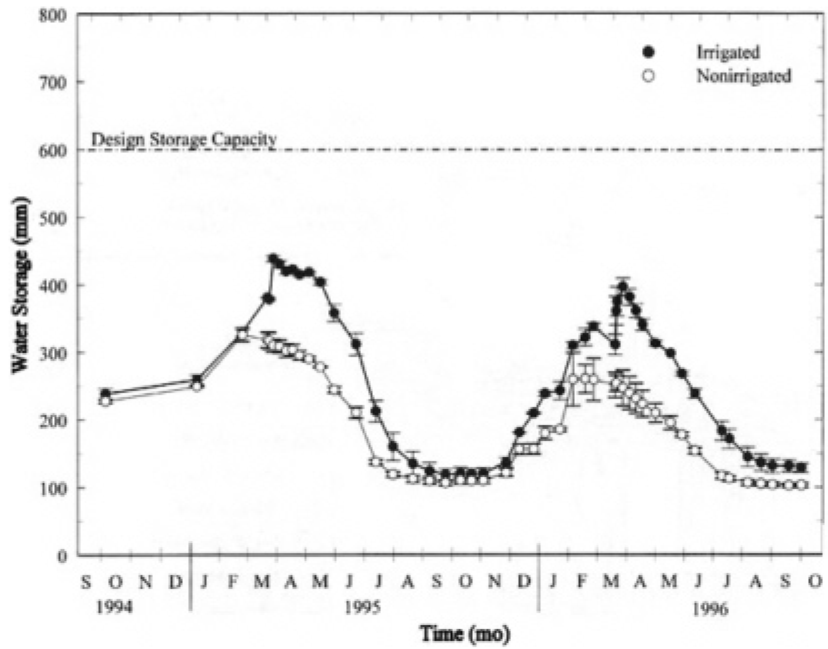

FIGURE 7
Temporal Variation in Soil Water Storage at the Prototype Barrier Since September 1994.

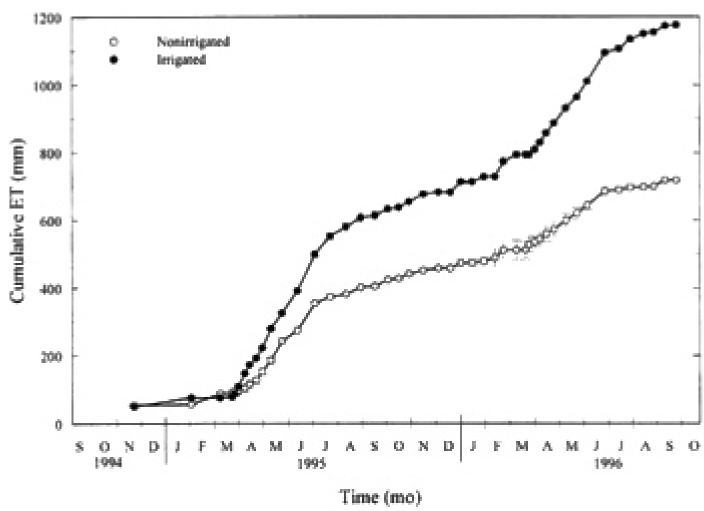

FIGURE 8
Cumulative Evapotransipiration at the Prototype Barrier Since September 1994.

Drainage did not occur from the soil-covered part of the prototype barrier, even under the extreme conditions of 3X precipitation. These observations from the prototype agree with the extensive lysimeter testing of capillary barriers (Campbell et al., 1990; Gee et al., 1993a) and suggest that water storage capacity of the soil is well in excess of the 3X (480 mm) precipitation. In contrast, the side slope plots all drained (Figure 9). Side slope drainage was expected since the surfaces are coarse and bare, with no vegetation growing on the rock riprap and only a sparse (less than 10 percent) cover growing on the clean-fill gravel. Surprisingly, the elevated drainage from the clean-fill side slope was greater than that from the basalt riprap side slopes. We speculate that the lower drainage on the riprap side slopes may be in part due to advective drying similar to that described by Stomont, Anxemy, and Tansey, (1994) and Rose and Guo (1995). Additional testing will better document the effect of advective drying on the side slopes.

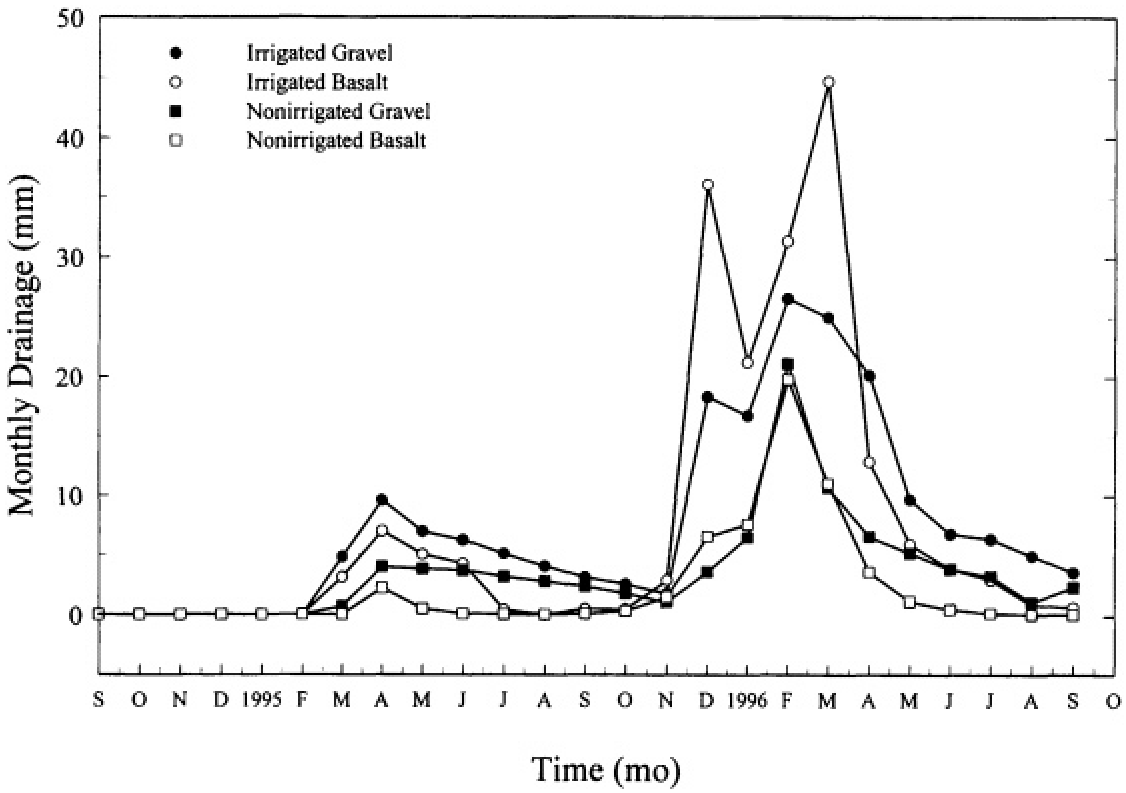

FIGURE 9
Monthly Drainage From Side slope at the Prototype Barrier Since September 1994.

The rapid establishment of vegetation on the soil surface was thought to be responsible for at least three positive benefits to surface barrier performance. First, the vegetation was dominant in the water removal process from the soil surfaces. Second, the surface was stabilized against water erosion and runoff. Runoff from the 1000-year storm in *1995* was 1.8 mm (about 2 percent of the 70 mm applied). There was no runoff in 1996. The improvement was attributed to vegetative growth and plant establishment. Root growth caused a lowering in soil bulk density, resulting in an increase in hydraulic conductivity and an increase in water infiltration capacity of the soil surface. Finally, there has been a positive benefit in controlling wind erosion. After plant establishment in November 1994, there have been no measurable losses of soil from the surface of the prototype by wind erosion. This is attributed to the vegetation and lack of surface disturbance during the past 2 years.

A minimum of 3 years of testing is planned for the prototype barrier. Because only a finite amount of time exists to test a barrier that is intended to function for a minimum of 1,000 years, the testing program has been designed to "stress" the prototype so that barrier performance can be determined within a reasonable time frame. Continued monitoring of prototype barrier performance for extended periods is desirable because the succession of vegetation types, the full development of root profiles, and the natural colonization of the barrier surface by burrowing animals will occur over a longer time period. Long-term monitoring of the prototype barrier would be a valuable asset for hydrologic model validation studies and in the assessment of the long-term performance of cover systems at the Hanford Site.

CONCLUSIONS

The study of surface barriers at the Hanford Site has evolved into an integrated demonstration of key features of barriers designed to minimize water intrusion, erosion, and biointrusion. The results of field tests, experiments, and lysimeter studies are providing a defensible foundation on which barrier designs can be based. Test results show that for the Hanford Site's arid climate, a well-designed capillary barrier limits drainage to near-zero amounts. A subsurface asphalt layer provides additional redundancy. The data collected under extreme events (excess precipitation) are building confidence that the barrier has the ability to meet its performance objectives for the 1,000-year design life. A prototype barrier, constructed in 1994 is providing data for evaluating both cover and side slope performance data needed for final design of surface barriers at the Hanford Site. Data from the prototype confirm earlier observations with lysimeters and field-plots and show that all available water can be removed from the soil surfaces by evapotranspiration, even under elevated precipitation conditions. Side slopes, in contrast, drain because they are barren. The side slope drainage is less than predicted because of wind action and possibly advective heating. Asphalt sublayers can be successful in extending the area of surface protection and can divert drainage water away from underlying wastes.

ACKNOWLEDGMENTS

This work is supported by the U.S. Department of Energy under contract DE-AC0687RL10930.

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NEW TECHNOLOGIES FOR SUBSURFACE BARRIER WALL CONSTRUCTION

Robert D. Mutch, Jr., P.Hg., P.E., Robert E. Ash, IV, P.E., Nashville, Tennessee, and Jeffrey R. Caputi, P.E., CHMM, ECKENFELDER INC., Mahwah, New Jersey

INTRODUCTION

During much of the 1980s, barrier walls of any type were regarded in some quarters as crude and antiquated. It was predicted correspondingly that remediation would be dominated by emerging treatment technologies such as bioremediation, air sparging, and surfactant flushing. Notwithstanding the considerable successes of these emerging technologies, particularly bioremediation, the fact remains that a significant percentage of Superfund, RCRA-corrective action and other waste disposal sites present hydrogeologic, chemical, and waste matrix complexities that far exceed the capabilities of current treatment-based remedial technologies. Consequently, containment-based technologies such as subsurface barrier walls and caps are being recognized once again as irreplaceable components of practical remediation programs at many complex sites.

Until quite recently, most barrier walls were constructed using traditional technologies such as soil-bentonite slurry trench, conventional sheet piles, vibrating beam technology, and in the case of shallow cutoff walls, compacted clay. Today, the remediation engineer considering a subsurface barrier wall-based cleanup is confronted with a baffling array of new technologies and permutations of these technologies. Table 1 presents a partial listing of available barrier wall technologies.

TABLE 1 Subsurface Barrier Wall Technologies

Compacted Clay	Vibrating Beam
Soil-Bentonite Slurry Trench	Ground Freezing
Self-Hardening Slurries	Waterloo Barrier® Sheet
Plastic Concrete Slurry	Pilings
Trench	Permeation Grouting
Deep Soil-Mixing	Geomembrane Technologies
Jet Grouting	

Each of the technologies listed in Table 1 also many permutations. For instance, there are many varieties of self-hardening slurries that can be tailored to specific site conditions and design objectives. There are also a wide variety of permeation grouts and several different geomembrane technologies, as well as a variety of different materials, that can be used in vibrating beam barrier walls.

Subsurface barrier walls have been constructed of compacted clay, soil-bentonite slurry trench and vibrating beam techniques for many years. These technologies are well-understood and well documented and are thus considered conventional cutoff wall technologies. Each technique has inherent advantages and disadvantages, and the cost of each is typically tied to site-specific factors. (For further information on these conventional technologies, see, e.g., Mutch and Ash (1994) and Cavalli et al. (1992).)

ADVANCEMENTS IN BARRIER WALL TECHNOLOGIES

In the remainder of this paper, we overview the different emerging barrier wall technologies, addressing their advantages, disadvantages, limitations, documented track records, and costs.

Deep Soil-Mixing Barrier Walls

Development of deep soil-mixing (DSM) barrier walls can be traced back to the early 1960s when the Intrusion Prepakt Co. patented a process for "mixed-in-place piles" (Jaspers, 1989). In the early 1970s, Japanese geotechnical companies developed several different types of soil-mixing methodologies and, to date, have conducted thousands of deep soil-mixing projects. Deep soil-mixing involves a crane-supported set of leads that guide a series of two to four hydraulically driven mixing paddles and augers 30 to 36 inches in diameter. As the auger guides make their way through the earth, they break the soil loose and lift it to the mixing paddles, which blend the soil with a slurry that is injected through the augers. The slurry can consist of lime, bentonite, cement, or proprietary mixtures designed to solidify or stabilize the soil. A continuous barrier wall is created by sequential penetration of the augers overlapping with previous auger-treated zones. DSM can be used to create cutoff walls more than 100 feet deep.

DSM offers several advantages over conventional cutoff wall methods. First, the soil does not have to be fully excavated, minimizing soil disposal costs if soils are contaminated. Since the wall is constructed in small sections, there is considerably less danger of collapse in soft soils. The technique is also capable of construction within confined areas and requires less staging and above-ground mixing areas than slurry trench techniques. A disadvantage of the technique lies in the fact that in-situ soils are used in the slurry soil admixture. If the soils are unsuitable or if waste materials are encountered, then additional costs and construction difficulties can result. The cost of deep soil-mixing usually falls in the range of $6 to $12 per vertical square foot.

Jet Grouting Cutoff Walls

Jet Grouting evolved in Japan during the early 1970s from a water cutting technology originally used in American coal mines (Guaterri, 1988). Jet grouting is a general term describing construction techniques where ultra-high-pressure fluids are injected into the soil at about 800 to 1,000 feet per second. The high-speed fluid is used to cut, replace, and mix the native soil with a cementing material, typically a cement-based grout. There are three general forms of jet grouting that involve injection of a single fluid (grout), two fluids (grout/air), or three fluids (grout/air/water). Jet grouting proceeds first by drilling a vertical guide hole down to the required depth. Actual jet grouting then follows, proceeding typically from the bottom to the top of the borehole. Panels or columns can be formed by controlling the rotation of the drill rods while lifting the jet grouting device. Columns are formed when the drill rods are rotated during lifting. Panels can be created by lifting the drill rods without rotation. Subsurface cutoff walls can be created by jet grouting adjoining columns of soil sequentially. Although jet grouting has been used extensively in Japan, Italy, Germany, and South America, it has received only limited attention here in the United States, but it is expected that the technique will be used more frequently here as acceptance of the procedure grows.

An advantage of jet grouting over other cutoff wall techniques is the fact that it can be used to stabilize a wide range of soils, ranging from gravel to heavy clays. Another advantage is that large-diameter columns or panels can be created, starting from relatively small-diameter boreholes. Therefore, cutoff walls can be constructed beneath buildings with limited disruption of the structure itself. Jet grouting also has been conducted to depths in excess of 200 feet. All three forms of jet grouting have some portions of their process covered by U.S. patents. The cost of a nominal three-foot-wide barrier wall constructed by jet grouting generally lies in the range of *$15* to $30 per vertical square foot.

Waterloo Sealable Sheet Piles

The University of Waterloo has developed a sealable sheet pile wall that reportedly is capable of achieving bulk hydraulic conductivities of less than 10^{-8} centimeters per second. This product, which has patents or patents pending in several countries, is termed the Waterloo BarrierTM. The technology involves specially fabricated sheet piles with a sealable cavity incorporated into the pile interlock. Figure 1 depicts the dimensions at the medium wall Waterloo BarrierTM constructed of 0.295-inch-thick steel. A heavier gauge, 0.375-inch-thick steel, Waterloo BarrierTM is scheduled to go into production during November of this year. The sealable cavity of the Waterloo BarrierTM can be sealed with clay-based, cementitious, polymer, or mechanical sealants. A footplate at the toe of the sealable cavity prevents most of the soil from entering the cavity during driving. After driving, the sealable cavities are water-jetted to remove loose soil in preparation for injection of sealant. Waterloo Barriers can be installed to depths of 70 feet and deeper if necessary by splicing piles together. Costs of the Waterloo Barriers are on the order of $15 to $30 per vertical square foot (R. Jowett, personal communication, 1995).

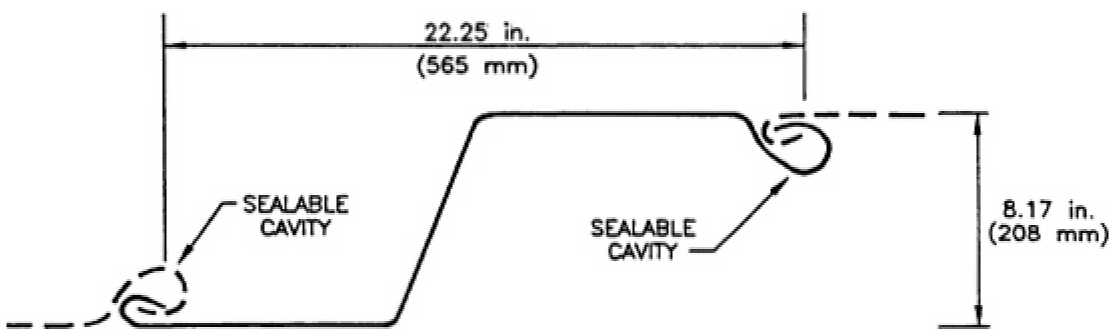

FIGURE 1
Medium wall Waterloo Barrier.™

Self-Hardening Slurries

Slurry trench cutoff walls can be constructed using self-hardening slurries, designed to set up in-place, producing low-strength barrier walls. This type of cutoff wall is constructed in panels. A continuous wall is formed by re-excavating the end of the adjacent panel after it has set up. Alternatively, an end-stop pipe can be placed between panels. The pipe is removed prior to full setting of the slurry, allowing the self-hardening slurry in the active panel to flow up against and form a good seal with the hardening slurry in the previously completed panel.

The most commonly used self-hardening slurry consists of Portland cement and bentonitic clay (cement-bentonite). Bentonite is blended with water to produce a hydrated slurry, typically consisting of 6 percent bentonite by weight. Cement is added just prior to pumping the slurry to the trench. The cement content is typically in the range of 10 to 20 percent by weight. Upon setting, the mix resembles a stiff clay with ultimate strength in the range of 5 to 50 psi. While cement-bentonite cutoff-wall technology has been around for many years, its use in site remediation has been limited by the inability to achieve sufficiently low-permeability. The permeability is typically in the range of 10^{-5} to 10^{-6} centimeters per second, whereas 10^{-7} centimeters per second commonly is specified in site-remediation applications. The relatively high permeability of cement-bentonite slurries is due, in part, to the adverse effect of the Portland cement on the swelling properties of the bentonitic clay.

Alternative self-hardening slurry mixes are now available that consistently can achieve permeabilities below 1×10^{-7} centimeters per second. Ground, granulated blast furnace slag can be blended with Portland cement to produce slag cement. When slag cement with a slag to Portland cement ratio of 3:1 to 4:1 is combined with bentonite slurry, the permeability of the mixture is generally in the range of 10^{-7} to 10^{-8} centimeters per second (Jefferis, 1985; Chipp, 1990). The use of slag cement also enhances chemical resistance and ultimate strength. A proprietary mix marketed by Liquid Earth Support, Inc., called Impermix®, consists of slag cement (containing no Portland cement) and attapulgite. Attapulgite is a clay mineral with a different crystalline structure than bentonite. The combination of slag cement and attapulgite produces a mix with extremely low-permeability, as well as greater resistance to chemical attack and higher ultimate strength. Permeabilities of less than 10^{-9} have been attained (Tallard, 1992).

An advantage of self-hardening slurries in comparison to conventional soil-bentonite slurry trench cutoff walls is that there is no separate backfilling operation. The slurry can be prepared in a remote area and pumped to the trench. This allows for construction in limited access areas and also minimizes the time workers must spend in the exclusion zone in the case of highly contaminated sites. In addition, there is little or no slurry displaced from the trench that could require special treatment or handling. Additionally, the panel methods of construction is advantageous when working in unstable soils or near structures, since the length of open trench can be minimized. Panel lengths typically range from 10 to 30 feet. This method also allows the barrier wall to be constructed in discontinuous sections, where necessary for coordination with other site activities. The cost of self-hardening slurry walls is typically in the range of $10 to $20 per vertical square foot for a nominal two-foot wide barrier and depths less than 100 feet.

Permeation Grouting

Permeation grouting has been used extensively in the United States and abroad in the mining and geotechnical engineering fields. The most common type of grout is a mixture of Portland cement and water. Other types of grout, which have been used under certain conditions, include cement-sand-water, cement-rock flour, cement-hydrated lime, cement-calcium chloride, cement-diatomaceous earth, lumnite cement, cement-clay, or cement-bentonite (Krynine and Judd, 1957). Asphaltic emulsions and other bituminous compounds have also been used for grouting (Krynine and Judd, 1957). More recent advancements include the use of microfine cement, mineral wax, sodium silicates, and colloidal silica gel.

The amenability of various soils to grouting is in large measure a function of the soil's permeability, as indicated in Table 2 (Karol, 1990). Soils with permeabilities less than 1×10^{-6} centimeters per second essentially are ungroutable, while soils with permeabilities greater than 10^{-1} centimeters per second require suspension grouts or chemical grouts containing filler materials. Grouting is also more difficult in heterogeneous soil, as the grout tends preferentially to follow pathways of least resistance through the soil.

TABLE 2 Approximate Relationship Between Soil Permeability and Groutability (Karol, 1990)

Permeability (cm/sec)	Groutability (Ability of Soil to Receive Grout)
10^{-6}	Ungroutable
10^{-5} to 10^{-6}	Groutable with difficulty by grouts with viscosity <5 cP and ungroutable with grouts having a viscosity >5 cP
10^{-3} to 10^{-5}	Groutable with low-viscosity grouts, but difficult with grouts with a viscosity greater than 10 cP
10^{-1} to 10^{-3}	Groutable with all commonly used chemical grouts
10^{-1}	Requires suspension grouts or chemical grouts containing a filler material

The Department of Energy, through its Sandia National Laboratories, has been conducting a study of two grouting materials, a montan wax emulsion and a glyoxal-modified sodium silicate material. Montan wax is a fossilized plant wax with properties similar to that of natural plant waxes, such as those found in canauba palms. It is a hard, high-melting point material comprised of waxes, resins, asphaltene-like materials with C-24 to C-32 carbon chain esters of long-chained acids and alcohols (Voss et al., 1995). Laboratory testing of soils permeated with a montan wax emulsion showed a significant reduction in soil permeability. The initial permeability of the soil tested varied from 6.5×10^{-4} to 3.6×10^{-2} centimeters per second. After permeation grouting by the montan wax emulsion, the soil permeability was reduced to between 3.7×10^{-8} and 1.6×10^{-4} centimeters per second.

The glyoxal-modified sodium silicate material originally was developed by a French company, Société Française Hoechst. The glyoxal-modified sodium silicate material consists of four proprietary components, the composition of which can be adjusted to modify set times. Laboratory testing of glyoxal-modified sodium silicate-grouted soil revealed final permeabilities of 6.2×10^{-6} to 5.1×10^{-5} centimeters per second. The initial soil permeability was the same as that cited above for the montan wax permeation study. It also should be noted that they were unable to grout soil with a permeability less than 5×10^{-4} centimeters per second with either the montan wax emulsion or the modified glyoxal sodium silicate material.

DuPont has also developed a permeation grouting material based on a colloidal silica gel. This technology was adapted from a technology developed by Conoco for oil field applications and is termed Ludox®. The gel times of the Ludox® material can be controlled to vary from a few hours to a few thousand hours. Its density and viscosity are similar to that of water. In laboratory experiments, it is reported that three orders-of-magnitude decrease in permeability was achieved following injection grouting by the Ludox® technique. Soils with an initial permeability of 7.7×10^{-5} to 8.6×10^{-4} centimeters per second were reduced in permeability to between 3.5×10^{-8} and 5.4×10^{-7} centimeters per second. However, in a larger sandbox-sized study, the permeability of the Ludox®-grouted soil was found to be 4×10^{-6} centimeters per second. Nonetheless, this represented a four orders-of-magnitude improvement over the permeability of the ungrouted sand (Noll et al., 1993).

Ground Freezing

Artificial ground freezing has been used in geotechnical construction for more than 100 years. The first application of ground freezing for construction purposes reportedly took place in Germany in 1883 (Braun and Nash, 1985). The maximum frozen depth achieved has been 3,000 feet (Braun and Nash, 1985). Ground freezing is accomplished by circulating a coolant through a network of closely spaced vertical or inclined pipes. The coolant can be calcium chloride brine, liquid nitrogen, or ethylene glycol. Due to relatively high maintenance costs, ground freezing generally is considered as a temporary containment measure.

The Department of Energy has undertaken a pilot-scale study of ground freezing at its Oak Ridge, Tennessee facility (Peters, 1994). The test site is approximately 60 feet by 60 feet and 28 feet deep. It consists of a double ring of inclined and vertical freeze pipes to form a V-shaped bathtub ring within which is a 750-gallon steel tank. An inner, single ring of heat pipes is used to control inward growth of the freeze zone. A variety of tests are planned to evaluate the integrity of the frozen ground barrier (Peters, 1994). It is reported that the cost of maintaining a frozen ground barrier for approximately 70 days is comparable to the cost of constructing a conventional soil-bentonite slurry trench (Iskander, 1987).

Geomembrane Cutoff Walls

Geomembranes may be used alone or in combination with other technologies to create low-permeability cutoff walls. A method developed by Nick Cavalli of Hayward-Baker in the early 1980s consisted of placing a geomembrane into a previously excavated slurry trench. Vertical panels of high-density polyethylene are welded to HDPE pipe. Each connection consists of panels each with a large-diameter pipe on one edge and a smaller-diameter pipe on the other. The larger-diameter pipe is slotted vertically, and the smaller-diameter pipe and membrane of the adjacent panel are inserted into the slotted, larger-diameter pipe. The interstitial space is then grouted. A leak detection zone may be created with this technology by placing a geonet within an envelope constructed of two geomembranes (Cavalli, 1992).

Gundle Lining Systems, Inc. (now GSE) also has developed a process for placement of high-density polyethylene (HDPE) as a cutoff wall. This process consists of driving vertical panels of HDPE into the soil with a steel-driving apparatus. Alternatively, the panels can be lowered into a previously excavated slurry trench. Gundle uses a jointing system that consists of an interlocking joint similar to steel sheet piling. Each half of the jointing system is welded to the vertical panels prior to installation. Each successive panel is then driven into the soil and through the interlock of the previously placed panel. A hydrophilic gasket, which expands to several times its own volume in water, is placed within the joint to create a water tight seal. A typical cross section of the Gundle jointing system is shown in Figure 2 (Steve Blume, personal communication, 1996; Blume, 1995).

Gundle and Ground water Control, Inc. have teamed to develop an alternative method for installing six foot wide, 80-mil HDPE panels with the jointing system described above. Ground water Control utilizes a one-pass trencher to install perforated pipe and gravel drains. The trencher has been modified to include a boot or narrow trench box that allows installation of the HDPE panels within the boot as shown in Figure 3. The trailing edge of the boot is fitted with flexible seals that move along and pass the installed panel through the end of the boot (including the joints). This allows space for installation of a subsequent panel. The bottom key typically is

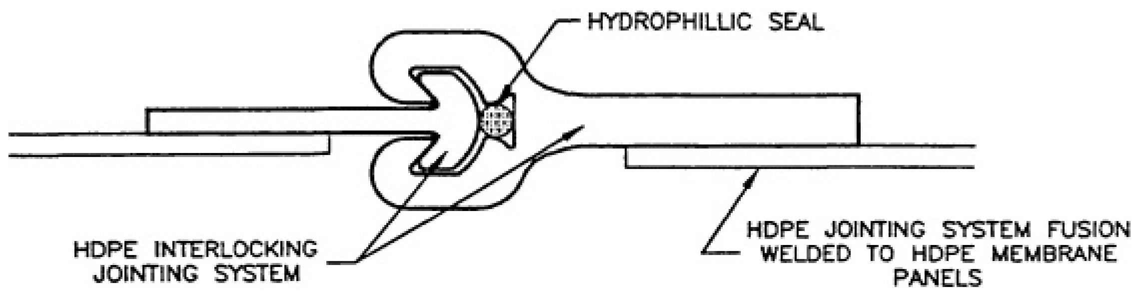

FIGURE 2
Geolock panel jointing system detail.

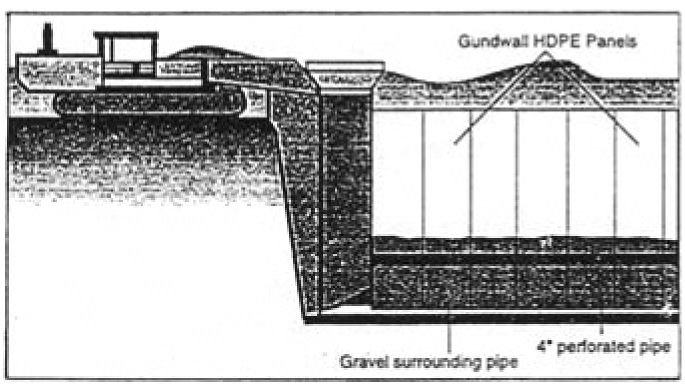

FIGURE 3
Ground water control/gundle trencher panel installation.

constructed of approximately one foot of bentonite pellets. A drain pipe and gravel also can be constructed within the same trench outside of the boot. This installation method can accommodate trench depths up to approximately 24 feet.

SLT Environmental, Inc. (now GSE) has also developed a process for using HDPE as a cutoff wall. This process also includes an interlocking joint system. The SLT panel jointing system is shown in Figure 4. The joint interlock is fusion welded to vertical panels of varying width prior to a placement. The panels are then lowered into a slurry trench or specialized steel trench boxes (for shallower trenches). During this process, the interlocking joint enters the interlock of the previously placed adjoining panel. Depending on the installation, the interlock is sealed by the slurry present in the trench, by grouting, or by using several hydrophilic sealant gaskets. Slurry Systems of Gary, Indiana, recently installed two cutoff walls consisting of a

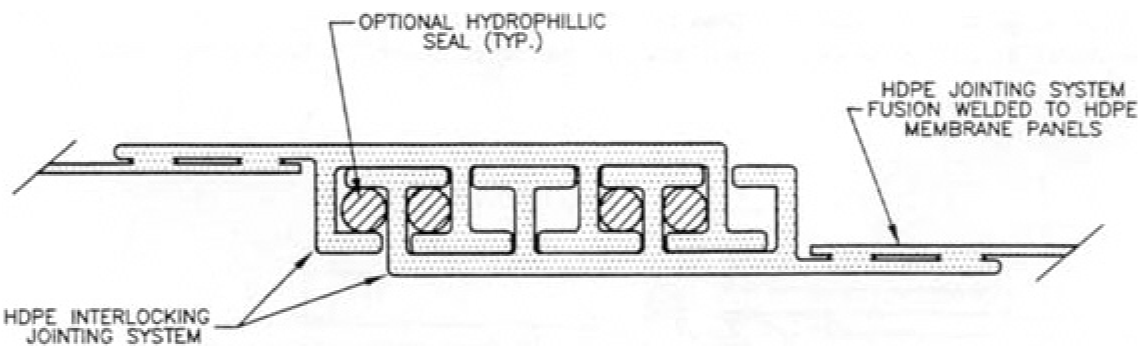

FIGURE 4
SLT panel jointing system detail.

combination of two technologies, including a vibrating beam wall and HDPE panels. These installations consisted of constructing a vibrating beam slurry wall using an attapulgite/cement slurry. SLT HDPE cutoff-wall panels with the SLT joint interlock were then vibrated into the approximately five-inch-wide cutoff wall using a steel-driving apparatus to create a composite cutoff wall.

Rodio S.p.a. of Italy has developed a composite cutoff wall involving placement of a HDPE within a self-hardening cement-bentonite slurry. The process begins with excavation of a vertical trench to the desired depth under a self-hardening slurry. HDPE sheets varying in width from 2 to 8 meters and mounted on a steel framework then are lowered into the self-hardening slurry. The steel framework is withdrawn after installation of the HDPE. Sealing of adjacent HDPE membranes is achieved by either overlapping, a variety of socket joints, expansion strip joints, or in-situ welding. This type of composite cutoff wall was constructed around an ash landfill near Florence, Italy, in 1992. The wall attained depths of up to 30 meters below ground surface (De Paoli, 1993).

Although dynamic and subject to a variety of site-specific factors, the costs of geomembrane barrier walls generally fall in the $10 to $30 per square foot range.

Deep Barrier Walls

The hydraulic effectiveness of barrier wall systems is heavily dependent upon the integrity and permeability of the aquitard into which the barrier wall is keyed (Mutch et al., 1981). Barrier walls that fail to penetrate deeply enough to key into an aquitard at all are usually only marginally effective in reducing ground water flow (Mutch et al., 1981). The growing importance of barrier walls, together with the fact that in many geologic environments, suitable aquitards may be at depths of 150 feet or more, has spawned considerable interest and advancement in the technologies to construct deep barrier walls (defined herein as walls greater than 150 feet in depth).

Conventional, soil-bentonite slurry trenches generally are not used for trenches in excess of depths of 150 feet due to stability considerations. Conventional slurry trenches are constructed in a continuous manner with backfill and excavation done in the same trench as shown in Figure 5. The backfill is placed in the trench from the bottom upward until the backfill reaches grade. When backfill reaches grade, it will have an angle of repose under the slurry that normally is between seven horizontal to one vertical, to ten horizontal to one vertical. This presents two major problems. In the case of a 150-foot-deep trench, the toe of the backfill is as much as 1,500 feet from the top of the backfill. Along with this, there is often an additional 100 feet of completed trench and approximately 200 feet of trench being excavated. Therefore, as much as 1,800 linear feet of slurry trench is open at any time. At depths of 150 feet or more, the stability of the trench often becomes marginal. Second, when the excavation ends, the slurry being displaced must be recovered and disposed. A trench of this size can necessitate disposal of six to seven million gallons of contaminated slurry.

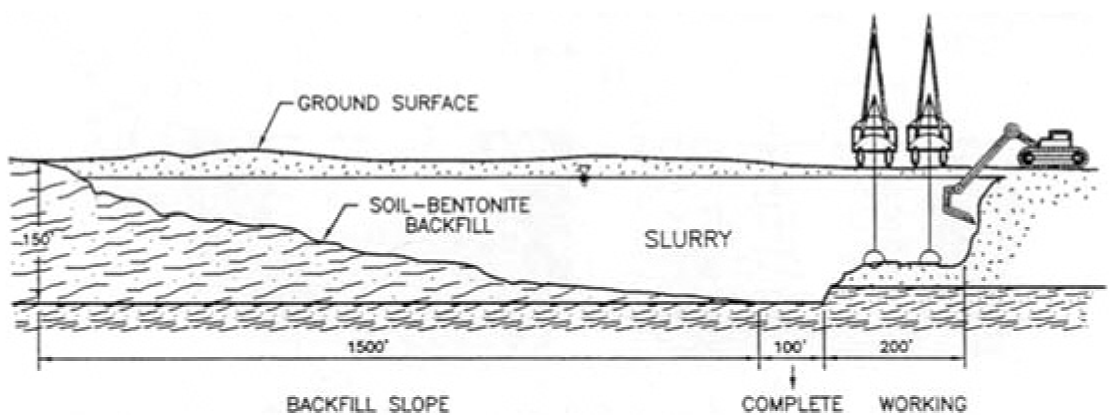

FIGURE 5
Construction of a deep soil-bentonite slurry trench.

Deep soil-mixing, vibrating beam, and jet grouting pose a different problem. Each of these methods involves a series of interconnected, short sections. Verticality, therefore, becomes critical. At depths of over 150 feet, it is difficult to ensure that gaps will not occur between adjacent panels.

Plastic concrete walls constructed in short panel sections of one to three clamshell bites have become the preferred type of barriers for deep walls. Alternately, a hydromill can be used. In this technique, a clamshell or hydromill is used to excavate a short length of trench to the desired depth under a bentonite slurry. Plastic concrete is then tremied into the trench, displacing the slurry that is reused in subsequent excavations. Panels are interconnected either forming a joint by placing an endpipe and extracting it after backfill has been placed, or by re-excavating several feet of the adjacent completed panels. Re-excavating portions of completed panels creates the problem of verticality, and therefore, continuity. The joint system, when constructed in a three-bite secondary panel between primary panels, is more forgiving with respect to verticality.

In the joint system, a series of primary panels is constructed at pre-set spacings with endpipes at both ends (Figure 6). Once the plastic concrete has set (normally 3 days), secondary panels are excavated in a three-step process. First, the clamshell or hydromill excavates a slot at the midpoint of the space between completed panels. Then on one side of this slot, the remaining soil between the slot and the adjacent completed panel is excavated, and the endpipe is removed. The same procedure is then repeated at the opposite side. The removal of the endpipe ensures that a good interconnection of panels is achieved. If a primary panel is out of vertical and the first bite is out of vertical, the excavating tool is guided by the space created when the endpipe is extracted and by the excavation of the first bite. Although the barrier may not be perfectly vertical, it should be continuous at all points.

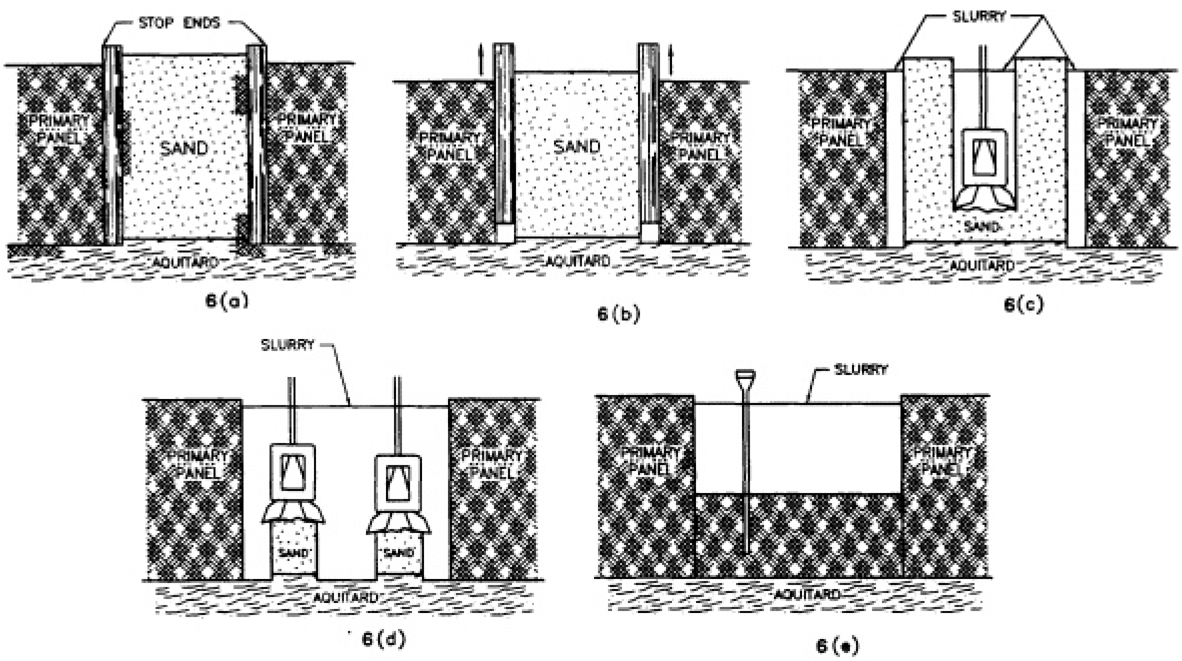

FIGURE 6
An alternating panel method of plastic concrete barrier wall construction. (a)Primary panels tremie concentrated with stop ends in-place. (b)Stop ends lifted. (c)Excavation of midsection of secondary panel. (d)Excavation of each end of secondary panel. (e)Tremie concreting secondary panel.

Projects such as the Wolf Creek Dam (281 feet deep), and the Manicougan Dam (430 feet deep) have been constructed using this methodology although, due to extreme depths, special equipment was employed. It is believed that subsurface barriers can be built by this technique to depths of at least 500 feet.

Vanel (1992) describes a related technique wherein a caisson beam of high-strength steel is used as an end stop. However, rather than being withdrawn prior to excavation of the adjacent panel, it is left in-place to serve as a guide to a specifically designed excavating tool that slides down the beam. The beam is pulled laterally away from the concreted primary panel once the secondary panel is fully excavated. The beam can also be fitted with an additional grooved caisson into which one or more plastic or rubber water stops can be inserted. The free half of the water stops becomes concreted into the primary panel. Lateral extraction of the beam uncovers the other half, which is then sealed in the concrete of the secondary panel.

CONCLUSIONS

The limitations of treatment technologies to remediate many waste disposal sites fully have led to increasing usage of and reliance upon subsurface barrier walls to control contaminant migration from such sites. This more common usage has been paralleled by considerable advancements in construction technologies. Barrier walls can now be constructed by many different techniques, each offering particular advantages, disadvantages, and limitations. Many of these techniques also can attain much greater depths than earlier conventional technologies.

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Robert D. Mutch, Jr., P.Hg., P.E., is an Executive Vice President and Corporate Director for Hydrogeology and Waste Management at ECKENFELDER INC. Mr. Mutch was formerly a Senior Vice President with Wehran Engineering in Middletown, New York. He is also an Adjunct Professor at Manhattan College in Riverdale, New York, where he teaches a graduate course in Applied Geohydrology. He has about 24 years experience in the fields of landfill design, hydrogeology and remedial engineering. His work has included the investigation and remedial design of hundreds of municipal and hazardous waste disposal sites, including dozens of Superfund sites. He has designed and, in most cases, supervised construction of 14 miles and 1,500,000 square feet of subsurface cutoff wall, 2 1/2 square miles of low-permeability landfill caps, 15 miles of retrofitted leachate collection systems, and numerous ground water extraction systems ranging in size from 50,000 gallons per day to 2,000,000 gallons per day. He has also provided consultation to the United Nations Environmental Program (UNEP) in regard to international landfill problems. He holds B.S. and M.S. degrees in Civil Engineering from New Jersey Institute of Technology. He is certified by the American Institute of Hydrology as a professional hydrogeologist (P.Hg.) and is a licensed professional engineer in several states.

Robert E. Ash, IV, P.E., is Assistant Division Director of the Waste Management Division for ECKENFELDER INC. in its Nashville, TN office. He has over 14 years of experience in the environmental consulting field, including site remediation and solid and hazardous waste management. Mr. Ash is a registered professional engineer with a B.S. degree in Civil Engineering from Rutgers College of Engineering. He is a member of the National Society of Professional Engineers and the American Society of Civil Engineers.

Jeffrey R. Caputi, P.E., CHMM is a Senior Manager in the New Jersey Waste Management Division at ECKENFELDER INC. where, for the past six years, he has worked extensively on hazardous site remediation projects. His work has primarily included feasibility studies and remedial designs for Superfund and RCRA sites, as well as sites regulated under various state programs such as the New Jersey Industrial Site Recovery Act and the Massachusetts Contingency Plan. Prior to joining ECKENFELDER INC. Mr. Caputi spent four years at Malcolm Pirnie, Inc. conducting remedial investigations, feasibility and treatability studies, remedial actions, and environmental audits. He has a Bachelor of Science degree in Environmental Engineering Technology and a Master of Science degree in Environmental Engineering from the New Jersey Institute of Technology. Mr. Caputi is a licensed professional engineer and a certified hazardous materials manager.

ECOLOGY, DESIGN, AND LONG-TERM PERFORMANCE OF SURFACE BARRIERS: APPLICATIONS AT A URANIUM MILL TAILINGS SITE

W. J. Waugh, Roy F. Weston, Inc., DOE Grand Junction Office, Grand Junction, Colorado; and G. N. Richardson, G.N. Richardson and Associates, Inc., Raleigh, North Carolina

ABSTRACT

Conventional engineering approaches for designing surface barriers for uranium mill tailings repositories fail to fully consider ecological processes that can have either beneficial or deleterious effects on long-term performance. The U.S. Department of Energy developed an alternative design for the semiarid Monticello, Utah, Superfund site that combines fundamental ecological principles with the required engineered barriers (e.g., geomembranes, compacted soil layers). The design relies on soil water retention enhanced by a capillary barrier and soil/plant evapotranspiration to seasonally return precipitation to the atmosphere. A compacted soil layer, which can fail because of desiccation cracking and biointrusion, is included only as a secondary infiltration barrier. The design relies on a combination of vegetation and a simulated desert pavement to limit soil loss without influencing the soil water balance. Rock riprap (coarse gravel used to prevent erosion), which can increase water infiltration and create habitat for deep-rooted plants, is used only on clean-filled side slopes. The design also controls radon releases, biointrusion, and protects critical layers from disturbance by frost. Preliminary analog studies of climate change, ecological change, and pedogenesis suggest that this design may improve with time. Field performance data and quantitative evaluations of analogs are needed before this alternative design, without the redundant engineered barriers, is used at other sites. Analog studies are needed to understand and evaluate possible long-term changes in the ecology of surface barriers that do not occur during short-term laboratory and field tests or that cannot be modeled numerically.

INTRODUCTION

The U.S. Department of Energy (DOE) is in the midst of cleaning up more than 20 million metric tons of low-level radioactive and sometimes chemically toxic tailings at abandoned uranium mills in the Four Corners region (Portillo, 1992). The accepted remedial action is to cover tailings and other contaminated materials either in-place or in landfill repositories. DOE faces the unprecedented legislative and engineering requirements that these tailings repositories persist for 200 to 1,000 years (USEPA, 1983). Engineered surface barriers or covers for tailings repositories typically consist of compacted soil layers, sand drains, and rock riprap intended to function as physical barriers to radon releases, water infiltration, and erosion (USDOE, 1989). This conventional engineering approach fails to fully consider the ecology of cover environments. After only a few years, biological disturbances threaten cover integrity at many sites (USDOE, 1992).

DOE developed an alternative cover design for the disposal of uranium mill tailings at the Monticello, Utah, millsite. This design is the product of unique combinations of regulatory and technical drivers. The Monticello repository design must satisfy both minimum technology guidance (MTG) for hazardous waste disposal facilities (USEPA, 1989) under Subtitle C of the Resource Conservation and Recovery Act of 1976 (RCRA), and design guidance for radon attenuation and 1,000-year longevity (USDOE, 1989) under the Uranium Mill Tailings Radiation Control Act of 1978 (UMTRCA). This required engineering guidance was refined by incorporating some fundamental ecological principles. Our goal was to design a cover that will improve rather than degrade over the long-term as inevitable natural processes act on the repository.

We summarize contaminant release mechanisms at uranium mill tailings repositories and then compare the design and intended functional performance of the Monticello cover with conventional RCRA and UMTRCA covers. Recommendations for design improvements, cost reductions, and assessment of long-term performance issues are also presented.

CONTAMINANT RELEASE MECHANISMS

Several concomitant release mechanisms acting on the cover potentially could cause environmental transport of tailings contaminants.

Water Infiltration

Rainwater and snow melt not lost by runoff and evaporation will enter the rock and soil layers overlying the tailings and become distributed in the these materials in response to various water potential gradients (Hillel, 1980). Depending on the properties and thicknesses of these layers, soil water could evaporate from the cover surface, be extracted by plants and returned to the atmosphere as transpiration, remain stored in the soil, pass into and remain stored in the tailings, or drain from the tailings and potentially mobilize and release contaminants.

Radon Release

Residual radioactive materials (radium-226) in uranium mill tailings emit radon-gas. Rates of radon escape into the atmosphere above the repository will depend on the physical, hydrological, and radiological properties of the tailings and overlying soil layers. The properties that most influence radon release are the soil moisture content of the cover, the radon diffusion coefficient for the cover, radium-226 concentrations in the tailings, and the emanating fraction for radon in the tailings (Smith, Nelson, and Baker, 1985).

Erosion

Removal of fine-grained material by sheet-flow erosion, rilling, gullying, and wind deflation could expose and disperse tailings under extreme conditions or, more likely, reduce the thickness of overlying layers leading to contaminant transport by other pathways (e.g., water infiltration). Soil loss by sheet-flow erosion involves the detachment of soil particles from the cover by raindrop splash and overland flow. If storm runoff is intense, flow may concentrate and cut rills and gullies deep into the cover (Walters and Skaggs, 1986). Wind transports soil particles by surface creep, saltation, and resuspension and may be particularly rapid leeward of topographic highs formed by mounded repositories (Ligotke, 1994).

Frost Penetration

As temperatures drop and soil layers within the cover freeze, water drawn toward the freezing front can cause desiccation cracking (Chamberlain and Gow, 1979), freeze/thaw cracking, and frost heaving (Miller, 1980), particularly in compacted soil layers. Desiccation and frost cracking may lead to increased permeability and gas diffusion in compacted soil layers within the frost zone (Kim and Daniel, 1992). Frost heaving may also cause distinct engineered soil layers to become mixed, thereby disrupting the integrity of critical layer interfaces (Bjornstad and Teel, 1993).

Plant Root Intrusion

Plants growing in the cover potentially could root into tailings, actively translocating and disseminating contaminants in above-ground tissues (Foxx, Tierney, and Williams, 1984; Morris and Fraley, 1989; Markose, Bhat, and Pillai, 1993). Roots may also alter tailings chemistry, potentially mobilizing contaminants (Cataldo et al., 1987). Macropores left by decomposing plant roots act as channels for water and gases to bypass compacted soil barriers effectively (Hillel, 1980; Passioura, 1991). Plant roots may concentrate in and extract water from buried clay layers, causing desiccation and cracking (Reynolds, 1990). This water extraction can occur even when overlying soils are nearly saturated (Hakonson, 1986), indicating that the rate of water extraction by plants may exceed the rehydration rate of the buried clay. Roots can also clog lateral drainage layers (USDOE, 1992), potentially increasing infiltration rates.

Animal Intrusion

Burrowing animals can mobilize contaminants by vertical displacement of tailings or by altering erosion, water balance, and radon release processes (Hakonson., Lane, and Springer, 1992). Vertical displacement results as animals excavate burrows and ingest or transport contamination on skin and fur (Hakonson, Martinez, and White, 1982). Once in the surface environment, contaminants may then be transferred through higher trophic levels and carried off site (Arthur and Markham, 1983). Loose soil cast to the surface by burrowing animals is vulnerable to wind and water erosion (Winsor and Whicker, 1980). Burrowing influences soil water balance and radon releases by decreasing runoff, increasing rates of water infiltration and gas diffusion, and increasing evaporation because of natural drafts (Landeen, 1994).

Cover Design and Performance

The Monticello cover (Figure 1) is structurally similar to the RCRA Subtitle C design for hazardous waste disposal facilities (USEPA, 1989). The seemingly subtle structural differences, however, represent salient conceptual and functional differences in performance. Table 1 compares components of the Monticello and RCRA designs.

Water Infiltration Control

Water Balance System

Water infiltration and leakage through the cover must not exceed the leakage rate of the repository liner (USEPA, 1989). The Monticello repository liner includes a geosynthetic clay layer with a design permeability of 1×10^{-9} cm/s. The Monticello cover design for controlling water infiltration is essentially an MTG RCRA design (sand drainage layer, geomembrane, and compacted soil layer) but with a thicker topsoil layer. The reliance of RCRA and UMTRCA designs on low-permeability compacted soil layers is well documented (Daniel, 1994; USDOE, 1989), and the failure of compacted soil layers to achieve performance objectives because of desiccation and shrinkage is also documented (Melchoir et al., 1994). The sand drainage layer, geomembrane, and compacted soil layer in the Monticello design serve as a backup for what we call a water balance system. The water balance system is the primary means for limiting infiltration over the long-term.

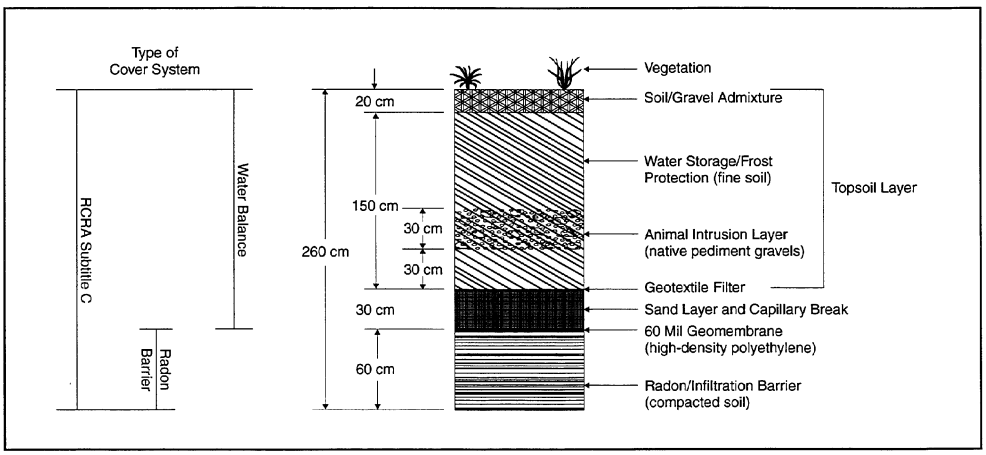

FIGURE 1
DOE Cover Design for the Monticello Repository.

TABLE 1 Comparison of RCRA Subtitle C (USEPA, 1989) and DOE MRAP Cover Designs

RCRA Subtitle	DOE Monticello Cover
Vegetation consists of locaay adapted perennial plants selected for erosion control	Vegetation consists of locally adapted perennial plants selected for erosion control and soil-water extraction
No gravel admixture layer	Soil/gravel admixture layer to enhance erosion control without adversely influencing plant water extraction • 20 cm thick • 40% by weight sand and gravel • 2- to 6-cm diameter gravel
Top slope between 3% and 5%	Tope slope between 3% and 5%
Topsoil layer • 60 cm thick • Fine-textured soil (e.g., loams)	Topsoil layer • 170 cm thick • Fine-textured soil (silt loam to sandy clay loam)
No animal intrusion barrier	30-cm-thick animal intrusion barrier consists of gravels and cobbles placed within the topsoil layer
Soil or geotextile filter as layer separator	Geotextile filter as layer separator
30-cm screened sand drainage layer • $K_{sat}{}^{a} = 1 \times 10^{-2}$ cm/s • 10-mm maximum particle size • Slope $\geq$ 3%	30-cm screened sand drainage layer • $K_{sat}{}^{a} = 1 \times 10^{-2}$ cm/s • 10-mm maximum paricle size • Slope $\geq$ 3%
Geomembrane $\geq$ 0.5 mm thick	60-mil high-density polyethylene geomembrane
Compacted, low-permeability soil layer • 60 cm thick • $K_{sat} \leq 1 \times 10^{-7}$ cm/s	Compacted, low-permeability soil layer • 60 cm thick • $K_{sat} \leq 1 \times 10^{-7}$ cm/s

[a]K_{sat} = saturated hyraulic conductivity.

At the semiarid Monticello site, ground water recharge is naturally limited where thick loess soils store precipitation until soil evaporation and plant transpiration seasonally return it to the atmosphere (Waugh and Link, 1992) The Monticello water balance design includes a sand capillary break that enhances this natural water conservation. In accordance with the "outflow law" of soil physics (Richards, 1950), the capillary barrier limits downward water movement and increases the water storage capacity of the topsoll layer because high tensions (suction) m the small pores of the topsoil impede movement of water into the larger pores of the underlying sand layer. Leakage into the sand occurs only if water accumulation at the topsoil/sand layer interface approaches saturation and tensions decrease sufficiently for water to enter the large pores of the sand layer (Hillel, 1980). The geotextile filter maintains the fine/coarse layer discontinuity until soil aggregation occurs by natural pedogenic processes (Bjornstad and Teel, 1993) Evapotranspiration can prevent excessive water accumulation above the textural break (Waugh et al, 1991; Anderson et al., 1993; Link, Waugh, and Downs, 1994). In short, the topsoil stores water while plants are dormant, then plants extract stored water during the growing season and return it to the atmosphere

Leakage from the water balance system is evaluated as the probability that water accumulation rates will exceed evapotranspiration and, eventually, the water storage capacity of the topsoil layer. Soil water storage capacity is the difference between the upper storage limit (before leakage occurs), sometimes referred to as the field capacity, and the lower storage limit (after removal of plant extractable water) (Ritchie, 1981). Field-plot and lysimeter tests conducted at other DOE sites (Waugh et al., 1991; Wing and Gee, 1993; Anderson et al., 1993) suggest that, with plants present to seasonally dry the Monticello cover, water accumulation likely will not exceed the topsoil storage capacity, even during higher than record precipitation years. Field and modeling studies are ongoing at Monticello to test this hypothesis. Preliminary results corroborate results of the previous studies. For the next generation of DOE cover designs, a water balance system without redundant geomembranes and compacted soil layers may be adequate to control water infiltration at arid and semiarid sites.

Revegetation

The calculated thickness of the Monticello topsoil not only provides an optimum water balance system but also creates a habitat more suitable for desirable vegetation. A thinner layer would encourage the establishment of a woodland plant community consisting of undesirable deep-rooted species. A diverse mixture of native plants on the cover will maximize water removal by evapotranspiration (Link et al., 1994) and remain more resilient to catastrophes and fluctuations in the environment (Begon et al., 1986).

Revegetation activities will attempt to emulate the structure, function, diversity, and dynamics of native plant communities in the area. The native sagebrush-grass vegetation at Monticello is a mosaic of many species that structurally and functionally changes in response to disturbances and environmental fluctuations (Tausch, Wigand, and Burkhardt, 1993). Similarly, biological diversity in the cover vegetation will be important to community stability and resilience, given variable and unpredictable changes in the environment resulting from pathogen and pest outbreaks, disturbances (overgrazing, fire, etc.), and climatic fluctuations. Local indigenous genotypes that have been selected over thousands of years are best adapted to climatic and biological perturbations. In contrast, exotic grass plantings, common on waste sites, are genetically and structurally monotonous (Harper, 1987) and, thus, more vulnerable to disturbance or eradication by single factors.

Radon Attenuation

The 60-cm compacted soil layer (radon/infiltration barrier in Figure 1) satisfies the requirement for a radon barrier that limits the average surface flux of radon-222 to less than 20 pCi m^{-2} s^{-1} (USEPA, 1983). The thickness was calculated with the standard method—the U.S. Nuclear Regulatory Commission (USNRC) model RADON (USNRC, 1989). This design approach is documented elsewhere (USDOE, 1989). As required for UMTRCA sites (USNRC, 1989), only the compacted soil layer (radon/infiltration barrier) of the cover was included in this calculation. All overlying layers were omitted. Further analysis suggests that the compacted soil layer may be unnecessary. RADON model results show a lower radon flux from a cover consisting of only a water balance system.

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Erosion Control

The primary erosion control issue is: Will vegetation alone adequately limit soil loss, or are gravel mulches, gravel admixtures, or rock riprap necessary to armor the soil when vegetation is sparse or less dependable? Vegetation and organic litter disperse raindrop energy, slow flow velocity, bind soil particles, filter sediment from runoff, increase infiltration, and reduce surface wind velocity (Wischmeier and Smith, 1978). Vegetation may be inadequate in the first years after construction. UMTRCA and alternative RCRA designs include cobble or rock riprap to control erosion in arid environments with sparse vegetation (USDOE, 1989; USEPA, 1989). However, these designs reduce evaporation (Groenevelt et al., 1989; Kemper, Nicks, and Corey, 1994), possibly increasing leakage through compacted soil layers and creating habitat for undesirable plants that root into the radon/infiltration barrier (USDOE, 1992).

Erosion control for the Monticello design consists of mixing gravel and sand in the top 20 cm of the topsoil (Figure 1) to mimic conditions leading to the formation of desert pavement. The method of Temple et al. (1987) was used to size the gravel (Table 1). The sand component was sized relative to the topsoil and gravel with Stephanson's (1979) method. Several erosion studies (Finley, Harvey, and Watson, 1985; Ligotke, 1994) and soil water balance studies (Waugh et al., 1994b; Sackschewsky et al., 1995) suggest that moderate amounts of gravel mixed into the cover topsoil will control both water and wind erosion with little effect on plant habitat or soil water balance. As wind and water pass over the surface, some winnowing of fines from the admixture is expected, leaving a vegetated erosion-resistant pavement. The sand "filter" and root cohesion of fines will impede continued soil loss beneath this pavement (Styczen and Morgan, 1995). The combination of vegetation and gravel pavement will control sheet-flow, minor rilling, and wind erosion by decreasing tractive sheer stresses. Rilling and gullying is controlled by maintaining top-slope gradients equal to surrounding terrain (which lack rills and intermittent gullies) and by limiting lengths of overland flow paths.

Frost Protection

The 170-cm composite topsoil layer (Figure 1) provides more than adequate depth to isolate the capillary break layer, drainage layer, geomembrane, and compacted soil layer (radon/infiltration barrier) from frost damage. The estimated maximum frost depth for a 200-year return interval in the topsoil layer is 115 cm. This value was extrapolated from soil physical properties for the loess soil and Monticello weather data by using the modified Berggren equation presented in DOE's *Technical Approach Document* (USDOE, 1989). UMTRCA rock riprap covers have essentially no frost protection for the radon infiltration barrier, and the 60 cm of frost protection offered by the RCRA cover is inadequate for Monticello.

Biointrusion Control

The Monticello cover includes barriers to biological intrusion by plant roots and burrowing vertebrates. By retaining soil water close to the surface, the combined topsoil and capillary barrier create a habitat for relatively shallow-rooted plant species and, thus, function as a de facto root intrusion barrier (Cline, Gano, and Rogers, 1980; Hakonson, 1986). Root growth generally is limited to regions within the soil where extractable water is available. The compacted soil layers in RCRA and UMTRCA covers may offer some protection. Agronomists have long observed that highly compacted soils cause stubby and gnarled root growth (Passioura, 1991) and can reduce rooting depths (Foxx et al., 1984). However, plants vary greatly in their ability to penetrate compacted soils (Materechera, Dexter, and Alston, 1991). At arid and semiarid sites, root densities can be higher in buried clay layers and cause seasonal desiccation (Hakonson, 1986; Reynolds, 1990).

The composite topsoil layer thickness is also the primary barrier to burrowing; it exceeds the maximum burrow depths of most vertebrates at Monticello. The 30-cm layer of native pediment gravel within the composite topsoil layer is an added deterrent. Loosely aggregated gravel and rock have been shown to deter burrowing mammals (Cline et al., 1980; Hakonson, 1986). This layer is above and protects the capillary break from bioturbation, a primary long-term threat to layer systems (Bjornstad and Teel, 1993). The native pediment gravels contain enough fines to prevent this layer from behaving like a secondary capillary barrier.

Longevity

The greatest uncertainties in designing the Monticello cover stem from the scientifically challenging need to extrapolate the results of short-term tests to the required 200- to 1,000-year performance period. Standard engineering approaches that are based on laboratory tests, short-term field demonstrations, and numerical predictions implicitly assume that initial conditions of material properties and of processes that drive contaminant transport will persist. In contrast, engineered covers must be viewed as evolving components of larger, dynamic ecosystems.

Natural analogs provide clues from past environments to possible long-term changes in engineered covers (Waugh et al., 1994a). Logical analogy is used to investigate natural and archaeological occurrences of materials, conditions, or processes that are similar to those known or predicted to occur in some part of the engineered cover system. As such, analogs can be thought of as uncontrolled, long-term experiments. Analogs may also have a role in communicating the results of the performance assessment to the public. Evidence from natural systems can help demonstrate that numerical predictions have real-world complements. Long-term performance issues at Monticello that can be assessed with the use of analogs include climate change, ecological change, and pedogenesis (soil development).

Climate Change

Climate greatly influences the release of hazardous materials from buried tailings at Monticello and the performance of the engineered cover designed to isolate tailings. With evidence of relatively rapid past climate change (Crowley and North, 1991) and model predictions of global climatic variation exceeding the historical record (Ramanathan, 1988), DOE recognizes a need to incorporate possible ranges of future climatic and ecological change in the repository design process (Petersen, Chatters, and Waugh, 1993). Paleoclimatic records may be useful not only as a window on the past, but also as analogs of possible local responses to future global change.

We reconstructed past climate change for Monticello by using available proxy data from tree rings, packrat middens, lake sediment pollen, and archaeological records (Waugh and Petersen, 1995). Interpretation of proxy paleoclimatic records was based on present-day relationships between plant distribution, precipitation, and temperature along a generalized elevational gradient for the region. For Monticello, this first approximation yielded mean annual temperature and precipitation ranges of 2 to 10° C, and 38 to 80 cm, respectively, corresponding to late glacial and Altithermal periods. These data are considered to be reasonable ranges of future climatic conditions that can be input to evaluations of water infiltration, radon-gas escape, erosion, frost penetration, and biointrusion.

Pedogenesis and Ecological Change

Pedogenic processes gradually will change the physical and hydraulic properties of earthen materials used to construct the Monticello cover (e.g., McFadden, Wells, and Jercinovich, 1987; Hillel, 1980). Plant and animal communities inhabiting the cover will also change in response to climate and disturbances. As the ecology of the cover changes, so also will performance factors such as water infiltration, evapotranspiration, water retention, soil loss, radon diffusion, and biointrusion.

Weighing lysimeters encasing 100-cm-deep soil monoliths were installed near the proposed Monticello repository site to measure the water balance of analog soils and vegetation (Waugh and Link, 1992). Monolithic lysimeters preserve, as well as possible, native soil profiles and vegetation. All precipitation received during the 1991 and 1992 bioclimatic years (November through October) was retained (no leakage occurred); close to normal precipitation was received for both years. Approximately 2.8 cm of leakage was measured during spring of 1993, indicating that soil water accumulation exceeded the storage capacity that year. The 1992-1993 winter (December-February) was one of the wettest on record (315 percent of normal); Monticello experienced the wettest February of this century. The increased storage capacity of a 170-cm soil layer over a capillary break would likely have retained all the excess soil water. These results suggest that with plants present to seasonally dry the topsoil layer of the cover, water accumulation likely will not exceed the topsoil storage capacity, except during years with higher than record precipitation.

SUMMARY

DOE plans to construct a lined landfill for disposal of tailings from an abandoned uranium mill at Monticello, Utah. The cover design, although similar in appearance, represents a departure from typical RCRA and UMTRCA designs. These designs are vulnerable to natural processes that will degrade the cover over the long-term. In contrast, the DOE design for the Monticello cover relies on natural processes to isolate tailings and to control the release of contaminants and is expected to improve over time.

The Monticello design should be considered as an alternative to RCRA Subtitle C and UMTRCA designs at other arid and semiarid sites:

- Compacted soil layers, as required for RCRA and UMTRCA designs to control water infiltration, are vulnerable to 5 damage by desiccation and biointrusion. In contrast, the

Monticello water balance cover relies on soil water retention, capillary barriers, and soil water extraction by plants.

- Rock riprap layers, as recommended for UMTRCA designs, control erosion but enhance water infiltration and biointrusion. The Monticello design includes a topsoil and gravel admixture. The admixture is designed to control erosion, much like a desert pavement, without adversely influencing desirable vegetation and the soil water balance.
- The Monticello design includes a geomembrane and a compacted soil layer as redundant infiltration barriers and to control radon release. These layers are also required to meet RCRA and UMTRCA design requirements. Results of small-scale field tests and numerical modeling suggest that the water balance cover will satisfy performance standards for water infiltration and radon releases without the engineered barriers.
- Field monitoring of water balance, erosion, and biointrusion are needed to evaluate the performance of the Monticello design under realistic conditions, before the design is used at other sites without the redundant engineered barriers. Similar measurements in natural analog environments may provide clues about long-term performance.

Engineered covers that are intended to last hundreds and thousands of years must be designed as evolving components of larger dynamic ecosystems. Four tenets accompany this principle: (1) cover components will not function and, thus, cannot be designed independently; (2) physical and ecological conditions will change over time; therefore, initial conditions cannot be extrapolated as tests of long-term performance; (3) designs should not rely on man-made materials of unknown durability; and (4) the design should not rely on physical barriers to natural processes but on the use of natural processes.

ACKNOWLEDGMENT

This work was performed under DOE Contract No. DE-AC13-95GJ87335 for the U.S. Department of Energy.

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SOIL, PLANT, AND STRUCTURAL CONSIDERATIONS FOR SURFACE BARRIERS IN ARID ENVIRONMENTS: APPLICATION OF RESULTS FROM STUDIES IN THE MOJAVE DESERT NEAR BEATTY, NEVADA

B.J. Andraski and David E. Prudic, U.S. Geological Survey, Carson City, Nevada

INTRODUCTION

The suitability of a waste-burial site depends on hydrologic processes that can affect the near-surface water balance. In addition, the loss of burial trench integrity by erosion and subsidence of trench covers may increase the likelihood of infiltration and percolation, thereby reducing the effectiveness of the site in isolating waste. Although the main components of the water balance may be defined, direct measurements can be difficult, and actual data for specific locations are seldom available. A prevalent assumption is that little or no precipitation will percolate to buried wastes at an arid site. Thick unsaturated zones, which are common to arid regions, are thought to slow water movement and minimize the risk of waste migration to the underlying water table. Thus, reliance is commonly placed on the natural system to isolate contaminants at waste-burial sites in the arid West.

Few data are available to test assumptions about the natural soil water flow systems at arid sites, and even less is known about how the natural processes are altered by construction of a waste-burial facility. The lack of data is the result of technical complexity of hydraulic characterization of the dry, stony soils, and insufficient field studies that account for the extreme temporal and spatial variations in precipitation, soils, and plants in arid regions. In 1976, the U.S. Geological Survey (USGS) began a long-term study at a waste site in the Mojave Desert. This paper summarizes the findings of ongoing investigations done under natural-site and waste-burial conditions, and discusses how this information may be applied to the design of surface barriers for waste sites in arid environments.

The waste-burial site is in one of the most arid parts of the United States and is about 40 km northeast of Death Valley, near Beatty, Nev. (Figure 1). Precipitation averaged 108 mm/yr during 1981-1992. The water table is 85-115 m below land surface (Fischer, 1992). Sediments are largely alluvial and fluvial deposits (Nichols, 1987). Vegetation is sparse; creosote bush is the dominant species. The waste facility has been used for burial of low-level radioactive waste (1962-1992) and hazardous chemical waste (1970 to present). Burial-trench construction includes excavation of native soil, emplacement of waste, and backfilling with previously stockpiled soil. Only the most recently closed hazardous waste trench (1991) incorporates a plastic liner in the cover. The surfaces of completed burial trenches and perimeter areas are kept free of vegetation.

FIGURE 1
Location of waste-burial site, Death Valley, and Mojave Desert of southwestern United States.

WATER MOVEMENT THROUGH DEEP UNSATURATED ZONE BENEATH UNDISTURBED, VEGETATED AREA

Field investigations to define the rates and directions of water movement through the deep unsaturated zone beneath an undisturbed, vegetated area began in the early 1980's and continue today. A vertical shaft allows personnel access for instrumentation of the upper 13 m of the unsaturated zone (Fischer, 1992), and additional test holes have been drilled (Prudic, 1994a). Thermocouple psychrometers are used to monitor water pressure and temperature, and a neutron probe is used to measure water content. Soil samples have been analyzed for chloride concentration in pore water (Prudic, 1994a), and water pressure of these samples has been determined using the water-activity meter described by Gee et al. (1992).

Chloride concentrations in pore water were used to estimate the period during which chloride has accumulated in the soil. The distribution of chloride in pore water within the upper 12 m of soils at the site is shown in Figure 2a. These data were determined from core samples collected from test holes (Prudic, 1994a) and from samples collected during excavation of two test trenches. Chloride concentrations at land surface range from 0.05 g/L at test hole UZB-1 to 2 g/L at the east trench. Concentrations are less than 0.5 g/L between the depths of 0.25 and 0.5 m and increase rapidly until the chloride peaks at 6-9 g/L between the depths of 1 and 3 m for test hole IB-1 and for the east and west trenches. Insufficient data are available from UZB-1 to determine a depth of peak concentration. In UZB-1, chloride concentrations decrease to about 0.05 g/L at a depth of about 12 m (Figure 2a) and remain less than 0.05 g/L to the last sampled depth of 85 m. Concentrations at depths greater than 12 m are less than the 0.08 g/L of dissolved chloride in ground water from a nearby well (Prudic, 1994a). The differences in the chloride distribution in the upper 5 m between the sites (Figure 2a) indicates that percolation is not distributed uniformly. Perhaps slight differences in topography or distribution of plants affect the depth of percolation and subsequent distribution of chloride in the soils.

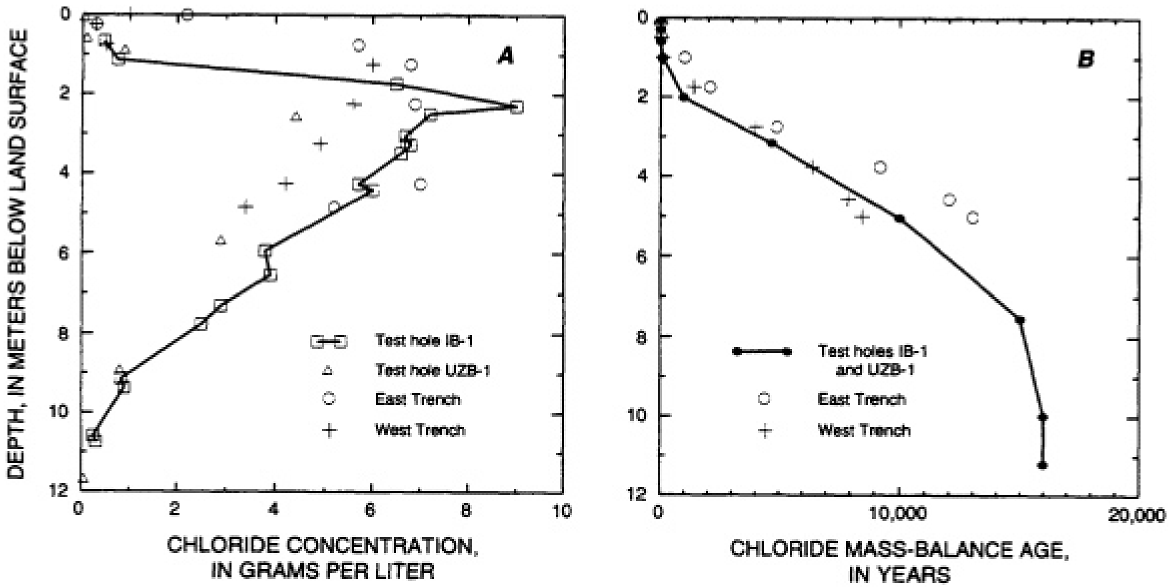

FIGURE 2
Chloride concentrations (a) and estimated chloride-mass-balance age (b) for pore water of soil at four locations at study site.

Estimated chloride-mass-balance ages of pore water were calculated by dividing the mass of chloride above a given depth by the atmospheric chloride-deposition rate at land surface (Phillips et al., 1988). Estimated ages are only approximate because long-term chloride-deposition is unknown. The ages in Figure 2b are based on a chloride-deposition rate of 1.6×10^{-5} g/cm^2/yr. This deposition rate assumes an average precipitation rate of 150 mm/yr and a chloride concentration of 1.1×10^{-6} g/cm^3 for precipitation and dry fallout. This rate is the greater of the two values used by Prudic (1994a) and is greater than the rate of 1.0×10^{-5} g/cm^2/yr reported by Phillips (1994) for the Nevada Test Site, about 40 km southeast of the study site. In the uppermost 0.5 m of soils, estimated ages of pore water are less than 50 years (modern). Below a depth of 1 m, estimated ages increase rapidly. At a depth of 5 m, ages range from about 9,000 years for the west trench to 13,000 years for the east trench. At a depth of 10 m, estimated age of pore water is 16,000 years. Owing to small chloride concentrations below a depth of 10 m, estimated ages increase only 2,000 years between the depths of 10 and about 85 m (Prudic, 1994a). Decreasing the estimated chloride-deposition rate to 8.2×10^{-6} g/cm^2/yr and recalculating results in pore water ages that are about twice those shown in Figure 2b. This deposition rate is based on a chloride concentration of 0.82×10^{-6} g/cm^3 measured at nearby Yucca Mountain (C.A. Peters, U.S. Geological Survey, written communication, 1992) and a precipitation rate of 10 cm/yr, which more closely approximates present-day conditions. Calculations based on either of the two deposition rates indicate that if the only source of chloride in the soils is from atmospheric deposition, then considerable time is needed to accumulate the quantity of measured chloride.

The low chloride concentrations below 10 m indicate either that the deeper soils were flushed with dilute water in the past or that chloride never accumulated in the soils. The estimated chloride age of 16,000 to 33,000 years at a depth of 10 m approximates the time when the climate in the area was wetter and cooler (Spaulding, 1985). Greater percolation and more frequent flooding of the Amargosa River during this period may have kept salts from accumulating in the soils. Since the end of the wetter period, the soils probably have been drying in response to the arid climate, and percolation of water during the past several thousand years has been limited to the upper 10 m, resulting in an accumulation of salts near the surface.

The lack of percolation below a depth of 10 m is consistent with observed upward water pressure and vapor-density gradients between the depths of about 12 and 48 m (Prudic, 1994b). Water pressures are less than -360 m (-3.5 MPa) between 3 and 12 m, then increase to -90 m at a depth of 48 m. Hydraulic heads calculated from water pressure (corrected for osmotic pressure) and elevation head are shown in Figure 3a. Water pressure data are based on psychrometer measurements made on September 16, 1993, and water-activity measurements on core samples collected during drilling of two test holes (UZB-1, November 1992; UZB-2, September 1993). The uncertainty in water pressures determined from water-activity measurements is about ±40 m and from psychrometers is about ±20 m. Considerable scatter appears in the hydraulic head of the upper few meters. The smaller hydraulic heads estimated from core samples near land surface may result from soil drying during drilling or sampling. Nevertheless, hydraulic heads in the upper 50 m are less than the hydraulic head at the water table, indicating a drying trend and upward liquid flow. In addition, vapor density decreases upward from 21.4 μg/cm at 48 m to 18.6 μg/cm^3 at 12 m in response to a temperature gradi*ent* of 0.06°C/m, indicating upward vapor flow (Prudic, 1994a).

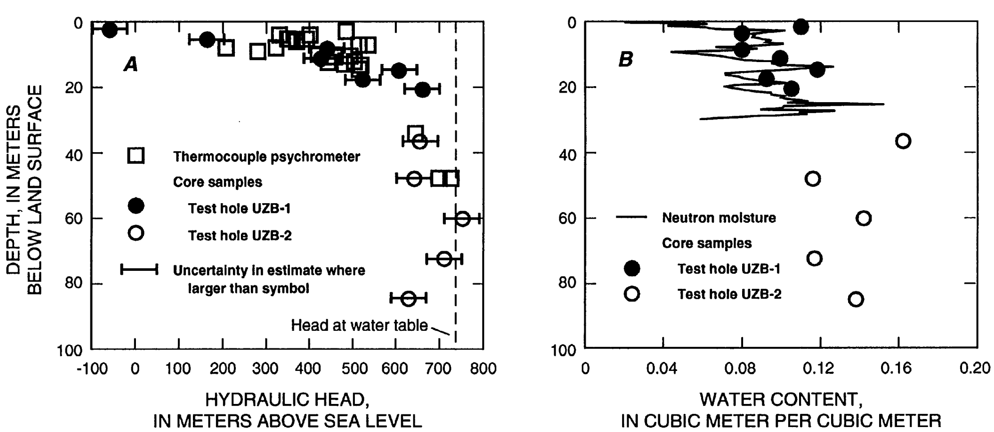

FIGURE 3
Changes in (a) hydraulic head and (b) water content with depth. Heads calculated from psychrometer data and from water pressure measurements of core samples; water content from core samples and from neutron-moisture probe measurements.

No psychrometers were installed below a depth of 50 m. Hydraulic head estimated from a core sample at 60 m is about equal to that at the water table and head from a sample at a depth of 85 m is less than that at the water table (Figure 3a). Whether the head at 60 m represents a past infiltration event is unknown. Chloride concentrations in the pore water at depths of 48- to 85-m range from 0.04 to 0.05 g/L, about twice as great as the concentrations of 0.02-0.03 g/L determined at depths of 15 m to about 37 m (Prudic, 1994a). Perhaps the greater concentrations at depth represent a previous near-surface accumulation of chloride that percolated downward under wetter climatic conditions.

Core samples collected from test holes UZB-1 and UZB-2 show generally greater water content below a depth of 35 m (Figure 3b). The water content of core samples in the upper 20 m generally corresponds with the water content determined from neutron-moisture measurements made in November 1992. Seasonal changes in water content at the undisturbed, vegetated site have been observed only within the uppermost meter of soils. This interval corresponds to a zone where chloride concentrations are generally low (Figure 2a). Within the zone of higher chloride concentrations, water content is not measurably changing, but water pressures, temperatures, and vapor densities change seasonally (Fischer, 1992).

EVALUATION OF PROCESSES UNDER WASTE-BURIAL CONDITIONS

The USGS test trench studies, which began in September 1987, combine field and laboratory experiments to define and evaluate quantitatively the interacting factors and processes that can affect waste-isolation. Three disturbed sites were established to simulate burial operations at the waste facility: two nonvegetated test trenches and one profile of undisturbed soil where vegetation was removed (Figure 4) (Andraski, 1990). Herbicide keeps the disturbed sites free of vegetation. The effects of disturbance on the water balance are evaluated in terms of observed differences between data collected at the undisturbed, vegetated site and data collected at the disturbed sites. Erosion of the trench covers is estimated by measuring the distance between the top of monitoring pins and the trench surface. Subsidence is determined by measuring the elevation of monitoring pins and plates with a rod and level. Meteorological data are collected by an automated weather station (Wood and Andraski, 1995).

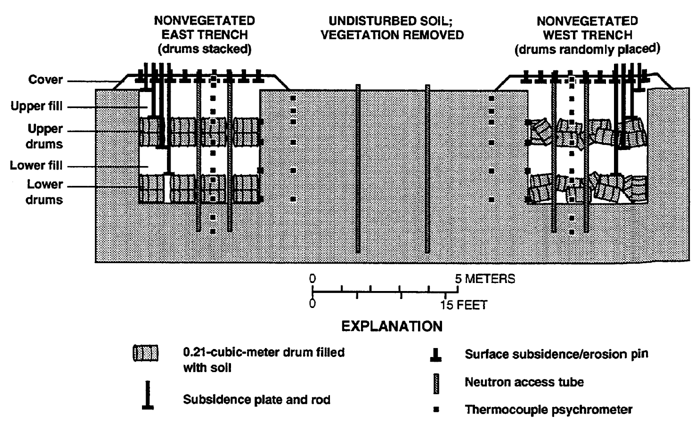

FIGURE 4
General design and instrumentation of three disturbed sites.

Precipitation and cumulative changes in water storage greatly varied during the first 5 years of concurrent monitoring at the four sites (Figures 5a,b). Continual monitoring of nonvegetated soil began in September 1988. Annual water-year precipitation (October through September) ranged from 14 mm (1988-1989) to 162 mm (1987-1988). Storage increases following precipitation were typically greatest for undisturbed soil. During spring and summer, rates of water depletion were greatest for vegetated soil. Even under conditions of extreme aridity (14 and 32 mm of precipitation in 1989 and 1990, respectively), storage values for the three disturbed sites remained greater than those measured initially. Storage values typically were greatest for nonvegetated soil.

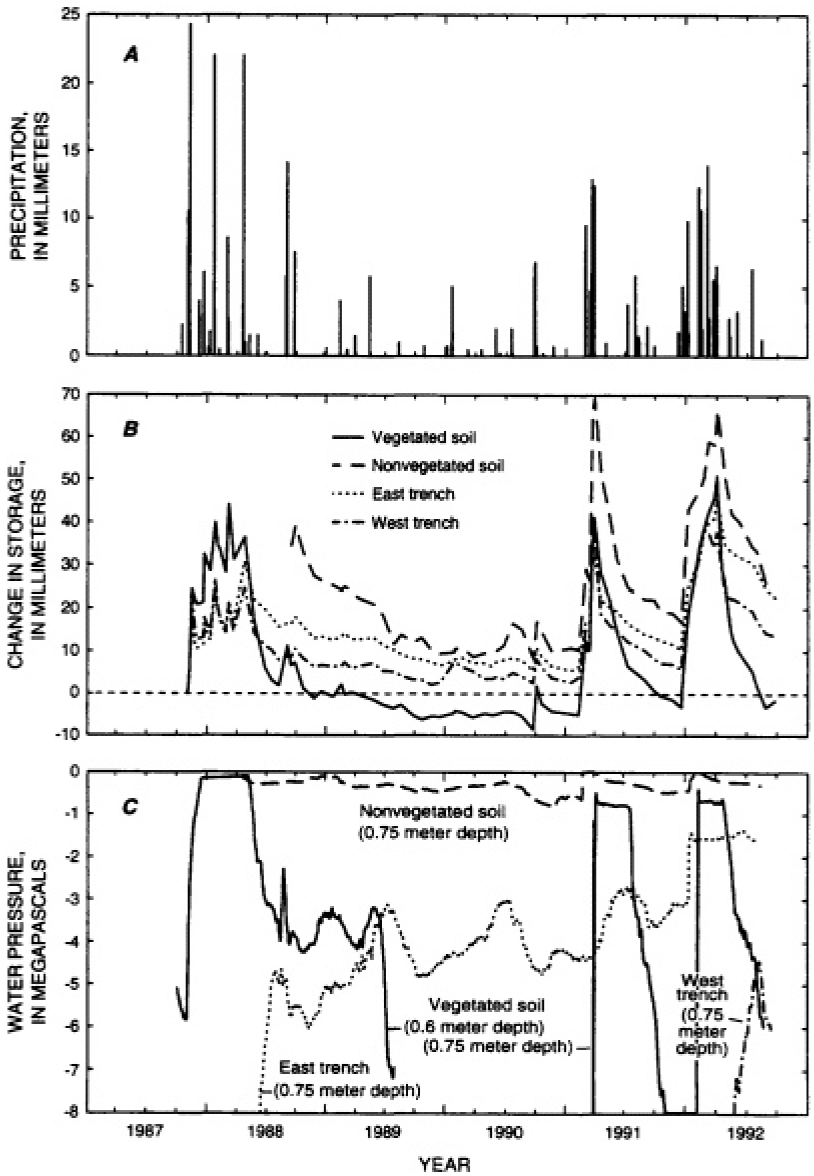

FIGURE 5
Daily precipitation (a) cumulative change in water storage, 0-to 1.25-m depth (b), and daily water pressures (c) measured at four sites.

Water pressure data illustrate some of the differences in the rates and depths of water accumulation and depletion among the four sites (Figure 5c). Concurrent monitoring at the four sites began in April 1988. Rapid percolation for the undisturbed soils, both vegetated and nonvegetated, resulted in high water pressures during the springs of 1988, 1991. and 1992. Pressures for the east trench show that the wetting front did not reach the *0.75-m* depth until June 1988, and pressures for the west trench show that the wetting front did not reach that depth until June 1992. More rapid and deeper percolation in undisturbed soil resulted in smaller evaporative losses. For the trenches, a greater quantity of rock fragments and greater initial water content for the cover of the east trench retarded evaporation and enhanced internal drainage [rock fragments (kg/kg): east = 0.45, west = 0.23; water content (m^3/m^3): east = 0.036, west = 0.021]. Water pressures for vegetated soil show substantial decreases due to water uptake by plants (Figure 5c). Water pressure for vegetated soil (0.6-m depth) decreased to values outside the psychrometer's calibration range between August 1989 and January 1991; this psychrometer was replaced by one at a 0.75-m depth in January 1991.

Although plants have a significant effect on the water balance, the potential for deep percolation also is influenced by soil properties (Gee et al., 1994). Hydraulic properties and their vertical variations in the upper 5 m of soil and trench fill at the site were measured over a water content range that is representative of arid conditions, but is seldom studied (Andraski, 1996). In contrast to the native soil profile, vertical (layer to layer) variability for trench fill was negligible. Hydraulic characteristics for the two uppermost soil layers (referred to hereafter as layer 1 and 2, respectively) and the trench fill are shown in Figure 6. Water-retention functions were calculated using the Rossi and Nimmo (1994) model. Unsaturated hydraulic conductivity (K_1) was calculated using the Mualem (1976) model, and isothermal-vapor conductivity (Kv) was calculated as described by Fayer, Rockhold, and Campbell, (1992). The -1.5-MPa pressure-plate data were omitted from the analysis because water-activity measurements showed that the actual pressures were significantly greater than the expected -1.5-MPa value. The data indicate that use of standard -1.5-MPa pressure-plate data, which commonly serve as the lower limit of retention measurements, can lead to significant errors in the description of hydraulic properties and prediction of water flow in dry soils.

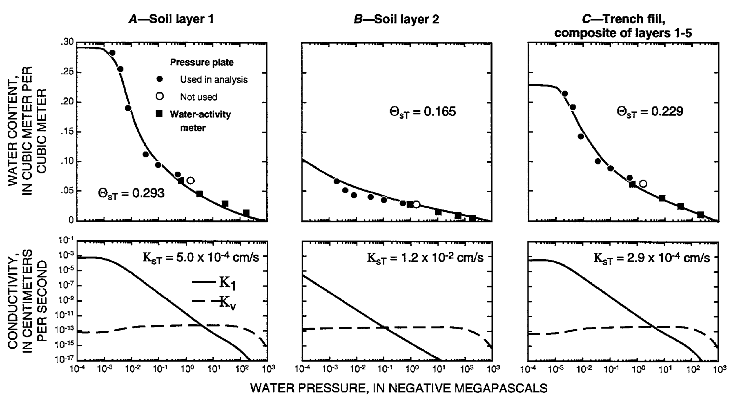

FIGURE 6
Hydraulic properties of soil layers l(a) and 2(b), and trench fill (c). ϴsT is saturated water content; KST, K_1, and Kv are saturated -, unsaturated-, and isothermal-vapor conductivity, respectively.

The textural difference between soil layer 1 (loamy sand) and layer 2 (gravelly sand) is reflected by their hydraulic properties (Figures 6a,b) and is an important factor in the water balance of the undisturbed soils. Soil layer 1 is about 0.75 m thick, and layer 2 is about 1 m thick. Except for water pressure values near-zero, K_1 values for soil layer 1 are greater than those for layer 2. Lower K_1 for layer 2 impedes movement out of layer 1. At saturation, layer 1 has the capacity to store 220 mm of water, or about twice the annual average precipitation (108 mm).

Backfilling with the dry fill (< -8 MPa) produced by trench construction, at least initially, will increase the importance of vapor flow in the fill (Figure 6c). As shown by data in Figure 5, however, depending on specific but commonly transient conditions, the relative importance of vapor and liquid flow may differ dramatically. Unlike the native soil profile, the fill provides no textural stratification to impede deep percolation of infiltrated water.

Changes in the structural integrity of trench covers through erosion or subsidence can reduce the waste-isolation potential of a burial site. No measurable soil loss was observed for the east trench, but soil loss for the west trench totaled about 9 mm during the first 5 years of monitoring (Figure 7a). Greater soil loss for the west trench may be attributed to fewer rock fragments in the near-surface. Most of the soil loss appeared to be due to deflation. During November and December 1987, two periods of high winds occurred during which hourly average wind speeds of 8-14 m/s persisted for 16 h or more. Nearly 55 percent of total soil loss for the west trench occurred during this time. The decreased rate of soil loss with time for the west trench may be due to increased surface armoring by rock fragments and also surface crusting, which occurs in response to wetting and drying cycles. Data for the east trench indicate a general trend of increased surface elevation with time. This trend may be due to deposition of eolian material (McFadden, Wells, and Jercinovich, 1987) or to the development of vesicular soil structure, which is induced by wetting and drying cycles (Miller, 1971).

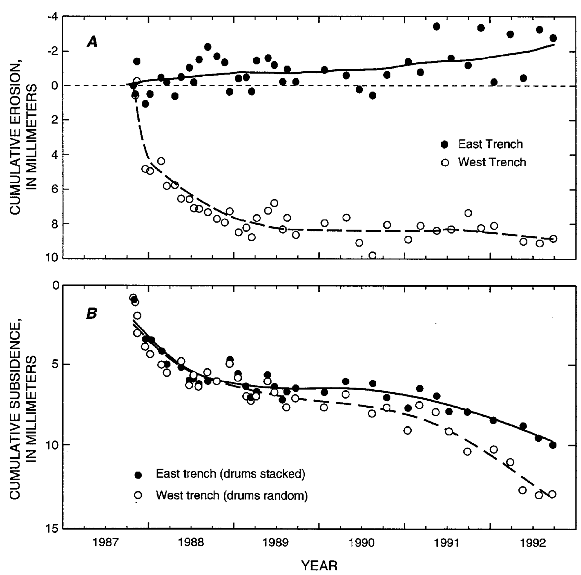

FIGURE 7
Cumulative erosion data (a) and subsidence data (b) measured since October 1987 and trend lines for east and west trenches.

General trends in subsidence were similar for the two test trenches (Figure 7b). During the first year following trench construction, differences between the east trench, where drums were stacked, and the west trench, where drums were randomly placed, were negligible. Subsidence measured during the first year probably represents the natural settling of the uncompacted fill material in response to precipitation and freeze-thaw cycles. During 1990-1992, effects of drum placement became evident, with greater subsidence for the trench where drums were randomly placed. Aside from drum placement effects, rates of subsidence appeared to be correlated with precipitation and concomitant increases in water storage and percolation within the trench cover and upper fill.

APPLICATION TO DESIGN OF BARRIERS FOR LONG-TERM ISOLATION

Investigations at the Mojave Desert site show that, even under extremely arid conditions, the interactive effects of climate, soils, and plants must be considered in the design of surface barriers for long-term waste isolation. The episodic precipitation patterns common to arid regions show the importance of multiple-year field studies. Comprehensive laboratory studies are needed for evaluating the factors and processes controlling waste isolation at arid sites. Ongoing investigations indicate that, under present climatic conditions, the natural soil-plant system effectively limits the potential for deep percolation. The stratified soil profile, in combination with native plants, provides for rapid infiltration, which reduces runoff; limited depth of percolation; high storage capacity for infiltrated water; and effective seasonal depletion of water accumulated in the root zone. Thus, the natural soil-plant system provides an excellent model for design of surface barriers intended to limit deep percolation and transport of soluble contaminants to ground water in an arid environment.

Construction of burial trenches and elimination of native vegetation markedly alter the natural water balance. In the absence of vegetation, infiltrated water accumulates and continues to percolate downward. Unlike the native soil profile, however, the homogeneous trench fill provides no stratification to impede deep percolation. Thus, changes to the natural-site environment may increase the potential for transport of buried waste. Preliminary evidence indicates that gas flow though the thick unsaturated zone and in the dry backfill potentially may serve as an important contaminant-release pathway at an arid site. The potential magnitude for contaminant transport by this process needs to be considered in the design of arid waste-burial and monitoring systems.

Greater rock fragment concentration in the near-surface of trench covers resulted in greater accumulation of infiltrated water and decreased erosion. Incorporation of this factor into barrier design may enhance vegetation establishment and control erosion. Effects of drum placement (stacked versus random) on trench cover subsidence were not observed until the third year of monitoring, when subsidence became greater for the trench where drums were placed randomly. Rates of subsidence appear to be correlated with precipitation and concomitant increases in water storage in the trench fill. Establishment of plants on trench covers may minimize cumulative subsidence by reducing water accumulation in trench fill, which, in turn, will reduce the physical load on waste buried below.

Continued long-term monitoring at the Mojave Desert site is critical to documenting how factors and processes controlling waste isolation may change with time. Data from the site provides a much needed, long-term benchmark against which short-term data from other arid sites can be compared. The data base and facilities at the site provide a foundation upon which to build collaborative efforts to further our understanding of hydrologic processes in arid environments. Results show that native plants are extremely important in minimizing deep percolation. Natural revegetation processes at arid sites may be extremely slow, however, and studies to develop strategies for establishment of native vegetation on trench covers are needed. To date, studies at the Mojave Desert site have focused on present-day climatic conditions. Additional study is needed to evaluate how the long-term waste isolation potential of the site might change under wetter climatic conditions.

ACKNOWLEDGMENTS

J.L. Wood, K.J. Hill, and M.L. Tumbusch for assistance in data collection; Drs. J.R. Nimmo and D.A. Stonestrom for helpful review comments; and R.L. Sweet, S.C. DeMeo, and G.A. Patterson for editing and illustrations.

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NATURAL PHYSICAL AND BIOLOGICAL PROCESSES COMPROMISE THE LONG-TERM PERFORMANCE OF COMPACTED SOIL CAPS

Ellen D. Smith, Robert J. Luxmoore, and Glenn W. Suter II, Environmental Sciences Division, Oak Ridge National Laboratory, Oak Ridge, Tenn.

Compacted soil barriers are components of essentially all caps placed on closed waste disposal sites. The intended functions of soil barriers in waste facility caps include restricting infiltration of water and release of gases and vapors, either independently or in combination with synthetic membrane barriers, and protecting other man-made or natural barrier components. Review of the performance of installed soil barriers and of natural processes affecting their performance (Suter, Luxmore, and Smith, 1993) indicates that compacted soil caps may function effectively for relatively short periods (years to decades), but natural physical and biological processes can be expected to cause them to fail in the long-term (decades to centuries). This paper addresses natural physical and biological processes that compromise the performance of compacted soil caps and suggests measures that may reduce the adverse consequences of these natural failure mechanisms.

PHYSICAL PROCESSES INFLUENCING BARRIER INTEGRITY

Physical phenomena that affect the integrity of compacted soil barriers in shallow land burial facilities and remediation sites are related to natural cycles of temperature and precipitation. Cycling occurs over three distinct time scales: diurnal (hourly changes), precipitation evapotranspiration (few days to months), and annual (seasonal). The diurnal cycle determines temperature gradients through the soil surface and influences root water uptake and gradients of soil matric potential. Wetting and drying cycles associated with precipitation and evapotranspiration directly change soil water content and matric potential. The annual climatic cycle provides the greatest range of temperature and moisture gradients. Long-term climatic cycles have much smaller amplitude and lower frequency and therefore are less important, although low-frequency extreme weather events can damage caps.

Effects of Wetting and Drying Cycles

A wet soil may shrink as it dries, forming cracks that penetrate from the surface to some depth in the soil. Soils vary in their propensity to form cracks, due largely to the type and quantity of clay minerals in the soil. Cracking may occur predominantly at the surface and may result in the formation of a crust. A mechanically strong crust will crack while maintaining its integrity, whereas in other soils a profusion of fine cracks may result in a friable surface. Crusts reduce the rate of soil drying by creating a resistance to liquid and vapor flux. Expanding-lattice clay minerals such as montmorillonite have a large propensity for shrinking and swelling with drying and wetting cycles, and soils with these clays have the greatest problems with cracking. Expanding-lattice clay minerals are present in soils through much of the continental United States (Lutton, Regan and Jones, 1979). When montmorillonite in the form of bentonite is added to soil barriers, the potential for problems with cracking is increased. Fixed lattice clays (e.g., kaolinite) may develop small cracks, so subsoils composed of such minerals usually form aggregates, with small cracks between aggregates. Repeated annual wetting and drying cycles lead to clay aggregation in the subsoil through annual shrink and swell changes. Cracks tend to remain as planes of weakness that become enhanced in subsequent shrink-swell cycles (Brady, 1974). Several clay minerals (e.g., illite and vermiculite) exhibit partial lattice expansion and are intermediate between montmorillonite and kaolinite in their propensity to shrink and swell.

If the water content of a soil barrier could be maintained in a narrow range near the optimum moisture content for compaction of the soil, problems with shrinkage and swelling might be avoided. A modeling study for landfills in a mesic eastern Tennessee environment predicted that a well vegetated soil cover of 0.6 m thickness over a clay cap could provide a stable environment for the cap. Very little variation in cap water content was found through several multi-year simulations, although barrier water contents were lower for years with lower rainfall than normal (Luxmoore and Tharp, 1992). An experimental study of compacted soil caps in a mesic environment in Germany suggests, however, that field performance would be less than predicted by the model. Melchior et al. (1994) found that, even with a 1-m thickness of topsoil and sand over a compacted 0.6-m soil cap with a low (17 percent) clay content, after five summer seasons, desiccation had caused sufficient soil shrinkage that infiltration through a flat cap increased from an initial value of 0.007 to 0.1 m/yr.

In an experimental study of water budgets of two landfill cover designs in the semiarid environment of Los Alamos, New Mexico, Nyhan, Hakonson, and Drennon (1990) found that the combination of a thick topsoil layer (0.7 m) and a coarse subsurface capillary barrier (0.5 m of gravel and 0.9 m of river cobble stones) over a compacted soil layer limited seepage through the soil cover, relative to a more conventional cover design including just 0.2 m of topsoil over compacted soil. Perennial grass vegetation was established on both covers. Deep seepage was monitored over three years including two years with higher precipitation than normal. The thicker barrier had greater evapotranspiration from the vegetative cover and much less infiltration through the soil cover.

Freeze-Thaw Effects

Freezing and subsequent thawing can destroy the integrity of a soil barrier by creating cracks and by reducing soil stability on sloping surfaces. The depth of freezing is highly dependent on the soil material, water content, and surface cover. Fine-textured soils with high water content are most susceptible to structural modification by freezing (Gillott, 1968). Frost heaving results to a small extent from the 9.1 percent volume increase as liquid water becomes ice. Most heaving is due to the upward movement of soil water into the frozen layer contributing to the formation of ice lenses (Miller, 1980). Clay-based caps are highly susceptible to frost heaving due to fine grain size and high water content.

Thawing of frozen soil can reduce its bulk density, and in the melting process, the side slopes of a landfill can be vulnerable to slumping due to the formation of a saturated layer with reduced cohesive strength above the frozen layer. Cycles of freezing and thawing also can lead to particle sorting with coarse materials rising above fine-textured particles, changing the hydraulic attributes of the soil. Frost damage to a landfill cap will increase percolation of water into the buried wastes.

The potential for frost damage to a soil cap can be reduced if the cap is protected by a vegetated topsoil layer that is thicker than the local depth of frost penetration. Frost penetration depths in much of the northern United States range from 0.6 to 1.8 m (Suter et al., 1993), which exceeds the recommended 0.6-m thickness of topsoil layers on hazardous waste facility caps (U.S. Environmental Protection Agency (1989).

Soil Erosion

Wind and water erosion can contribute to cap failure. Both can be largely prevented with vegetative cover. In semiarid sites in the western United States, maintenance of a vegetative cover may be difficult (Webb and Voorhees, 1984). In this situation, a rock riprap may be the most effective means for preventing wind and water erosion (Voorhees et al., 1983). However, this method of long-term erosion control may encourage burrowing animals and may increase infiltration of water to the waste by slowing runoff and decreasing evapotranspiration.

Subsidence

Continuing changes in wastes due to decomposition, consolidation, and settling can result in differential subsidence, which can produce depressions and cracking of the cap. The amount of subsidence depends on the characteristics and density of the waste. Poorly compacted municipal refuse can settle as much as 33 percent (Brunner and Keller, 1972). Most municipal landfill subsidence occurs during the first few years after a landfill has been closed. The hazardous and radioactive wastes typically found at remedial sites are less susceptible than municipal waste to subsidence due to decomposition, but poor initial compaction of waste can lead to cap subsidence unless wastes are stabilized prior to cap placement.

BIOLOGICAL PROCESSES INFLUENCING BARRIER INTEGRITY

Without continual human intervention, ecological succession will occur on waste sites resulting in the establishment of a series of biotic communities that will modify soil barriers. Two biological processes that are particularly significant with respect to soil caps are the intrusions of plant roots and burrowing animals.

Root Intrusion

Plant roots can have several effects on soil barriers and the underlying waste: (1) plants can decrease drainage through the barrier by taking up water; (2) roots may penetrate the soil cap; (3) decomposing roots leave channels for movement of water and vapors through the cap; (4) roots may dry clay layers, causing shrinking and cracking; (5) uprooted trees may leave depressions in the cap, thinning the cap and allowing water to collect; (6) roots may enter the wastes, take up constituent chemicals, and transport them above ground; and (7)roots may increase waste decomposition rates and release exudates that mobilize metals (Browning and Molz, 1978; Cataldo et al., 1987). Vegetation is planted on soil caps routinely to prevent erosion of the cap and because water uptake by plants can reduce the quantity of water available for leaching (Lutton et al., 1979). However, the other effects of roots reduce the long-term protection actually provided by vegetated soil caps.

The occurrence of plant root penetration into caps depends on the plants established by revegetation and subsequent ecological succession, the rooting depth of the plants, and the roots' ability to penetrate the cap material. Radioactive waste sites provide strong evidence for root penetration of soil caps because monitoring readily detects radionuclides in above-ground plant tissues. Suter et al. (1993) reviewed reports of operating experiences at several U.S. Department of Energy sites indicating that plant roots have taken up radionuclides from wastes buried 0.6 to 2.5 m. Lysimeter studies also demonstrate that grasses, herbs, and shrubs can penetrate soil caps and extend roots into wastes (Browning and Molz, 1978; Molz and Browning, 1977; Nyhan, 1989; Reynolds, 1990; Melchior et al., 1994). These results demonstrate that concern about increased mobilization of buried wastes by plant roots is not academic.

Ecological succession is the process by which the vegetation of a site changes through the establishment of plant species, competition among plants, herbivory, modification of the soil, and the actions of physical agents such as frost. It is possible to estimate the course of vegetation succession at a particular site, but because many factors affect succession, one cannot predict accurately what the vegetation of a site might be at any specific date after closure. For example, the succession from bare ground to shrub-steppe at Hanford Reservation, Washington, normally requires 30-40 years, but at some waste sites the ground was still bare after 35 years (Fitzner et al., 1979).

A major consequence of succession with respect to soil caps is that longer-lived and deeper-rooted species become established. In the eastern United States, this occurs by succession from grasses and weedy annual herbs through a brushy stage of woody perennial herbs, vines and pioneer trees, to forest trees. At Oak Ridge, Tennessee, succession from a planted grassland to a brushy stage would require *10-25* years, and succession to mature forest would occur in 65-150 years, absent intervention. The average depth of grass roots is less than 1 m, but tree roots average over 3 m, with maximum depth exceeding 60 m (Table 1). Once trees are established, they may grow up to 0.3 m/year in height with roughly equivalent rooting depths for about the first 10 years. Assuming that neither the waste nor the soil cap significantly inhibits plant growth, a soil cap in the eastern United States is in jeopardy within 25 years after abandonment, and after 100 years, wind thrown trees could begin tearing holes in the cap.

On arid and semiarid sites, there is not such a regular successional series. The changes in species composition are often slow, and pioneer species are often the permanent residents of the site. However, growth rates on arid sites tend to be low so that it may take as long to establish a "mature" desert shrub vegetation as a "mature" mesic forest. Shrubs are the major threat to soil caps in these environments (Webb and Voorhees, 1984), but perennial grasses that can extend roots to significant depths (Table 1) also pose a threat. Growth rates vary considerably, but soil caps in arid to semiarid environments could be breached by plant roots in 10-50 years if the sites are amenable to plant growth.

The soil atmosphere above buried wastes may inhibit plant growth. Root inhibition by methane and other gases generated by decomposing municipal waste hinders, but does not necessarily prevent, revegetation of closed sanitary landfills (Gilman, Leone, and Flower, 1981). However, most hazardous wastes do not produce such gases or produce them for a relatively short period of time because they contain relatively little decomposable organic matter.

TABLE 1 Mean and maximum rooting depth of ten plant life forms (from Foxx et al., 1984)

Life Form	Number of Observations	Depth (m)	
		Mean	Maximum
Evergreen trees	40	3.4	61.0
Deciduous trees	107	3.3	30.0
Shrubs	87	3.5	17.0
Subshrubs	36	1.4	6.4
Vines	4	1.7	2.8
Perennial forbs	370	1.7	39.0
Biennial forbs	9	1.1	1.5
Annual forbs	81	0.8	3.0
Perennial grasses	305	1.4	8.2
Annual grasses	50	0.5	1.1

Once deep-rooting vegetation is established on the site, the only remaining condition that must be met before the cap is breached is that the material of the cap be penetrable by roots. Clay provides some inhibition of root growth. Foxx, Tierney, and Williams, (1984) found that plants rooted less deeply on adobe clays, but half of 82 specimens growing in clay soils rooted to depths greater than 0.6 m and 7 percent rooted to below 4.5 m. Reynolds (1990) found that three species of grass and one shrub planted in lysimeters had all penetrated 0.6-m clay barriers after three years. The clay barrier was less effective on average than equal thicknesses of scoria or of gravel and cobble.

Root penetration of soil caps is affected by the inherent strength of the soil, which is enhanced by compaction. Veihmeyer and Hendrickson (1948) found the maximum bulk density permitting root penetration could vary from 1.46 Mg/m^3 in clays to 1.75 Mg/m^3 in sands. The compacted density of clay materials in soil barriers can be in the vicinity of 1.75 Mg/m^3 (Goldman et al., 1988), sufficient to provide a significant barrier to root growth. Roots do not enter pores with diameters smaller than that of the root, and typically roots become thickened in dense soils (Russell, 1977). Lateral roots with smaller-diameters than the main root may be able to enter some small pores in a compacted layer. Numerous studies have shown a linear decline in root density with increase in mechanical impedance.

At high water content soil penetration resistance is low, but a soil with high bulk density and high water content may be unsuitable for root growth due to poor aeration. Oxygen diffusion to roots may limit root respiration in dense clay layers with high water content due to low gas filled porosity. There is evidence that roots become less effective in penetrating a dense soil layer as soil aeration is diminished (Russell, 1977).

Zones of poor compaction may permit roots to penetrate a compacted soil barrier. It is very difficult to ensure complete compaction of barriers with field-scale compaction equipment (Suter et al., 1993). Most compacted soil barriers have some form of preferential flow paths, and roots tend to proliferate in these zones once intersected. With time, old root channels, worm holes, and cracks between soil aggregates will increase (Edwards, Fehrenbacher, and Vavra., 1964; Ehlers et al., 1983; Wang, Hesketh, and Woolley, 1986). Root exploration eventually will lead to penetration of a compacted soil barrier, initially through zones where compaction was ineffective.

The fate of tombs and ceremonial mounds, which are built of soil and often contain clay layers, may be analogous to that of soil caps at waste sites. In the eastern United States, the Mississippian culture built thousands of mounds. The soil of the mounds is compacted and often consists of clay or clay loess, apparently in a deliberate attempt to reduce erosion (Lindsey et al., 1982). However, those mounds that have not been kept cleared now support trees and shrubs. Presence of trees has encouraged slumping and other erosional processes, but grasses seem to help protect the structures if trees are suppressed (Lindsey et al., 1982). Animal Intrusion

Animals can have several effects on soil caps and the underlying waste. (1) They may burrow through the caps resulting in direct channels for movement of water, vapors, roots, and other animals. (2) Even when they do not penetrate to the waste, burrows may increase the porosity of the soil, thereby increasing rates of infiltration and movement of water and vapors. (3) Animals may become externally contaminated or consume the waste, thereby spreading the waste in their feces, urine, and flesh and increasing decomposition of the waste. (4) They may carry the waste directly to the surface during excavation. (5) By working the soil and by transporting seeds, they may hasten establishment of deep-rooted plants on the cap (Cadwell, Eberhardt, and Simmons., 1989). (6) They cast soil on the surface thereby increasing erosion of the cap (Day, 1931; Winsor and Whicker, 1980). (7) Animals construct their burrows for natural ventilation, which may dry the soil thereby decreasing water intrusion (Vogel, Ellington, and Kilgore, 1973; Cadwell et al., 1989; Landeen and Kemp, 1991).

Animal intrusions into buried wastes have been documented to occur at waste sites. At Grand Junction, Colorado, prairie dogs *(Cynomys ludovicianus)* have burrowed through soil caps and brought uranium mill tailings to the surface (McKenzie et al., 1982). At Hanford, a large-mammal believed to be a coyote *(Canis latrans)* or badger *(Taxidea taxus)* burrowed into a waste trench, exposing radiocontaminated salt that was consumed and distributed by other wildlife (O'Farrell and Gilbert, 1975). At Los Alamos National Laboratory, pocket gophers *(Thomomy*s *bottae)* burrowed through a 0.25-m soil cover and brought the biobarrier, crushed tuff, to the surface (Hakonson, Martinez, and White, 1982). At the Idaho National Engineering Laboratory, several species of rodents and their surface castings have been contaminated by burrowing into trenches of radioactive waste even in areas with 1.2-m caps (Arthur and Markham, 1983).

The extent to which burrowing damages soil barriers depends on the depth of burrows, and in particular, whether the burrows extend through the soil cover of a compacted layer or even further through the cap. Depths of burrows for vertebrate animals present at major Department of Energy sites are shown in Table 2 (Suter et al., 1993).

The presence of animals on a waste site depends on the habitat quality of the site. Habitat quality depends in turn on the management of the site and, in the absence of vegetation management, on ecological succession. Many waste sites that are not too arid are maintained as mowed lawns. Frequent mowing not only minimizes the rooting depth of the vegetation but also makes the site unattractive to burrowing vertebrates. In the eastern United States, the vertebrate that is most likely to burrow through a cap, the woodchuck *(Marmota monax),* occurs primarily in the earliest successional stages, 10-25 years after site abandonment. The abundance of burrowing small rodents is also highest during that period. At arid and semiarid sites, faunal succession is unimportant; all burrowing animals that are likely to occur at a site are likely to be present as soon as vegetation is established. Although rock armoring would seemingly protect caps from burrowing animals, large rocks attract burrowing mammals, apparently because they provide dry sites for den chambers and discourage den excavation by large predators.

TABLE 2 Maximum reported depth of burrows created by some animals occurring at or in the vicinity of DOE waste sites (from Suter et al., 1993)

Species or Taxon	Depth(m)
Earthworms *(Lumbricidae)*	2
Harvester ants *(Pogonomyrmex spp.)*	3
Mole *(Scalopus aquaticus)*	0.6
Shorttailed shrew *(Blarina brevicaudata)*	0.5
Pine vole *(Microtus pinetorum)*	0.3
Montane vole *(Microtus montanus)*	0.5
Pocket mouse *(Perognathusparvus)*	1.4
Pocket gopher *(Thomomys talpoides)*	>1
Deer mouse *(Peromyscus maniculatus)*	0.5
Kangaroo rat *(Dipodomys ordii)*	0.7
Ground squirrel *(Spermophilis townsendii)*	1.4
Chipmunk *(Tamias striatus)*	1
Woodchuck *(Marmota monax)*	1.5
Box turtle *(Terrapene carolina)*	0.1

Earthworms and other soil invertebrates, which simply require a vegetation cover to provide plant detritus for food, are not discouraged by mowing. In humid areas, earthworms probably cause more significant damage to soil barriers than vertebrates because they are abundant and may burrow deeply (Table 2). In addition, the extensive burrowing of earthworms provides channels for root penetration of clay layers; the roots provide more food for worms resulting in more and deeper channels.

CONCLUSIONS AND RECOMMENDATIONS

Clearly, natural physical and biological phenomena may cause failure of earthen barriers. Site closure or in-situ remediation designs that depend solely on compacted soil caps or composite caps therefore cannot be depended upon for long-term isolation of waste, but should be used only in combination with effective stabilization of the underlying waste materials. Additionally, the following design features and maintenance measures should be considered to delay barrier failure or reduce its potential adverse consequences:

(1) Infiltration barriers should be covered by a soil layer sufficiently thick to extend below the frost line, to accommodate the typical rooting depths of native plants expected to invade the site, and to extend below the probable depth of animal burrows (i.e., at least 3 m in most areas).

(2) Composite caps (i.e., including both compacted clay soils and synthetic membranes) may be superior to those constructed with either material alone, as they provide some defense against the problems that can occur when sole reliance is placed on either earthen or synthetic barriers. If composite caps are to be used effectively, additional research may be needed to understand the interaction of the two types of barriers. For example, Vielhaber et al. (1994) reported formation of desiccation cracks in a compacted soil layer below an intact synthetic membrane barrier, probably due to natural thermal cycles and the exothermic decomposition of underlying wastes.

(3) Post-closure land uses should not only avoid disturbance to the final cover, but should also discourage biotic disturbances. For example, land uses that involve regular lawn maintenance or maintenance of a paved area over a waste site are likely to limit root penetration and discourage most burrowing animals.

(4) Monitoring and maintenance of inactive hazardous waste-burial sites should be continued indefinitely (not just for the 30-year period often mandated by regulation).

ACKNOWLEDGMENT

Based on work performed at Oak Ridge National Laboratory, managed for the U.S. Department of Energy under contract DE-AC05-96OR22464 with Lockheed Martin Energy Research Corporation, Environmental Sciences Division publication number 4607.

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GEOMEMBRANES IN SURFACE BARRIERS

Ronald K. Frobel, R.K. Frobel & Associates, Lakewood, Colorado

Geomembranes are very low-permeability synthetic liners used to control fluid or gas migration within soil, rock, earth, or any other geotechnical material, as an integral part of a man-made product, structure, or system. As a synthetic component used within the ground, they are technically a geosynthetic, the prefix "geo" indicating usage on or in the earth. The other primary geosynthetics are geotextiles, geonets, geogrids, geocomposites, and geocomposite clay liners.

Geomembranes have been used in a variety of containment applications since the early 1940's. Within the past 25 years, the growth of flexible membrane liners (FML's) in the form of prefabricated plastic or elastomeric sheet materials has increased rapidly in the construction industry and is now an accepted civil engineering material along with other geosynthetics. In fact, there are over 30 individual applications that have been developed for geomembranes.

The original use for geomembranes was for the distribution, storage, and containment of potable/agricultural water supplies. It still remains as an important element of this market, except now it has been broadened to contain a wide variety of liquids. These liquids span the entire spectrum of liquid wastes (acids to bases), potable water, water for recreation, power, flood control, etc. Closely related to geomembranes containing liquids in the static state are geomembranes used for canal liners and other hydraulic conveyance systems.

The rise of geomembranes for cover systems is a growing application area. The major concentration is to cover solid waste sites, liquid waste ponds, and potable water reservoirs, but numerous other applications such as odor containment, potential sabotage reduction, methane gas collection, and temporary waste covers are evident. The largest growing segment of the geomembrane industry has to do with cover systems for solid waste containment. Currently required for hazardous waste containment, there is a growing tendency, fueled by environmental regulations, to use geomembranes to contain all types of solid waste. These include municipal, industrial, mining, and low-level radioactive wastes.

Geomembranes have become the design choice as part of a cover system due to a variety of factors. These factors include such design considerations as imperviousness, chemical resistance, inertness to surrounding soils, ease and variety of seaming, mechanical strength and elongation, ease of application, and economics. Environmental concerns and subsequent regulations have also compelled engineers to include synthetic geomembrane systems in their waste containment designs, mining applications, retention ponds, and municipal landfills.

Although the current U.S. geomembrane market is volatile, all information collected thus far clearly points to a growth rate through 1995 of between 20 and 25 percent, with 20 percent being a conservative estimate. At the lower-bound 20 percent growth rate, the total estimated installed quantity should approach 114 million m2/year by 1996. This growth can be attributed t*o* six factors (Koerner, 1994):

1. existing companies stepping up their marketing services,
2. new companies with highly visible marketing,
3. new high-tech materials and seaming methods,

4. the emergence of new geocomposites requiring geomembranes,
5. new government regulatory standards in waste disposal, and
6. education through conferences, publications, advertising, etc.

Although there are many polymeric types of geomembranes, fully 90 percent of the market has been comprised of four geomembrane systems of varying sheet configurations, i.e., smooth, textured, reinforced, composite, etc. Table 1 lists these geomembranes and their approximate formulations.

TABLE 1 Types of Commonly Used Geomembranes and Their Approximate Formulations by Percent

Geomembrane Type	Resin %	Plasticizer %	Filler %	Carbon Black or Pigment %	Additives %
HDPE[a]	95-98	0	0	2-3	0.25-1.0
VLDPE[b]	94-96	0	0	2-3	1-4
PVC[c]	45-60	25-35	0-10	2-5	2-5
CSPE[d]	40-60	0	30-45	5-40	5-15

[a] HDPE (Medium- to High-Density Polyethylene in the density range of 0.92 to 0.96 gm/cm³)
[b] VLDPE (Very Low-Density Polyethylene).
[c] PVC (Polyvinyl Chloride - Soft)
[d] CSPE (Chlorosofonated Polyethylene or Hypalon)

It is evident that the semi-crystalline and very low-crystallinity thermoplastics such as HDPE, VLDPE, CSPE, and the newer Flexible Polypropylenes, are taking an increasing part of the waste closure market and will continue to do so. From a low percentage of use in 1981 to over 25 percent in 1988 and over 50 percent in 1995, semi-crystalline/low-crystallinity thermoplastic materials are outperforming competitive products for the following reasons:

- relatively low cost
- wide extruded sheets
- wide variety of thicknesses
- wide variety of surface textures
- excellent chemical resistance
- reliable field welding (thermoplastic)
- good mechanical properties
- less field seams
- durability

GEOMEMBRANE MANUFACTURE

Most geomembranes are made in a manufacturing plant using one of the following highly technical and controlled manufacturing processes: extrusion, spread coating, and calendaring.

The extrusion process is a method whereby a polymer of the polyolefin family (such as polyethylene or polypropylene) is extruded by blown vertical or slot die horizontal extrusion methods into a nonreinforced sheet. Immediately after extrusion, when the sheet is still warm, it can be bonded to a fabric, through light calendaring forming a geocomposite. During extrusion, the sheet material can also be textured for high surface friction using a variety of dies or processes.

The spread coating-process usually consists of coating a fabric (woven, nonwoven, knit) by spreading a polymer or asphalt compound on it. These geomembranes are therefore reinforced. Non-reinforced geomembranes can be made by spreading or spraying a polymer on a sheet of paper that is removed and discarded at the end of the manufacturing process.

Calendaring is the most frequently used manufacturing process. Calendared non-reinforced geomembranes usually consist of a single sheet of compound made by passing a heated polymeric compound through a series of heated rollers (calendar). Calendared reinforced geomembranes are produced by simultaneously running sheets of compound and scrims through heated rollers. A three-ply calendared reinforced geomembrane is made of the following layers: compound/scrim/compound.

Geomembranes manufactured by the above processes are produced in rolls approximately *1.5-10* m in width. Geomembranes that are produced in wide rolls, typically 5-10 m, are commonly transported to the field site where they are seamed together. Geomembranes that are produced in narrow, lighter rolls (PVC and CSPE) are first transported to a fabrication factory where they are seamed into large panels. Panels can be fabricated to any designed shape and are limited only by handling weight and dimension, commonly less than 1860 m^2. Panels are packaged and transported to the construction site where they are seamed together. Small sites can often be lined with a single panel, thereby eliminating the need for field seaming. Figure 1 schematically illustrates the geomembrane industry as it presently exists from polymer manufacture through installation.

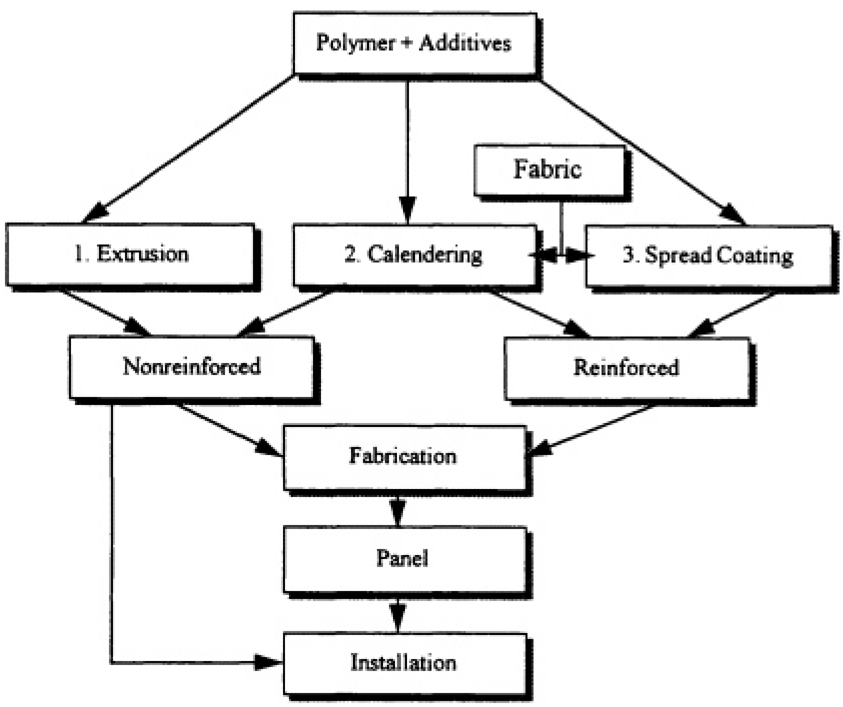

FIGURE 1 Schematic of the Geomembrane Industry (Haxo, 1986).

GEOMEMBRANE SEAMING

The most critical element of an installed geomembrane system is the seam. The mechanism of seaming polymeric geomembrane sheets together is to melt or otherwise bond the overlapped edges of a geomembrane roll or panel in a controlled manner. The bonding of two polymeric geomembranes results from an input of energy that originated from either thermal or chemical processes. There are generally four categories of seaming methods: extrusion welding, thermal fusion (melt bonding), chemical fusion, and adhesive bonding. Figure 2 illustrates the four categories.

Thermal fusion or melt bonding is the most common seaming method for the thermoplastic geomembranes. In particular, the dual hot-wedge method and hot-air method are predominant. The dual hot-wedge method provides an air channel between weld tracks that can be nondestructively tested by air pressurization. Extrusion fillet welds are used primarily for detail and patch welding.

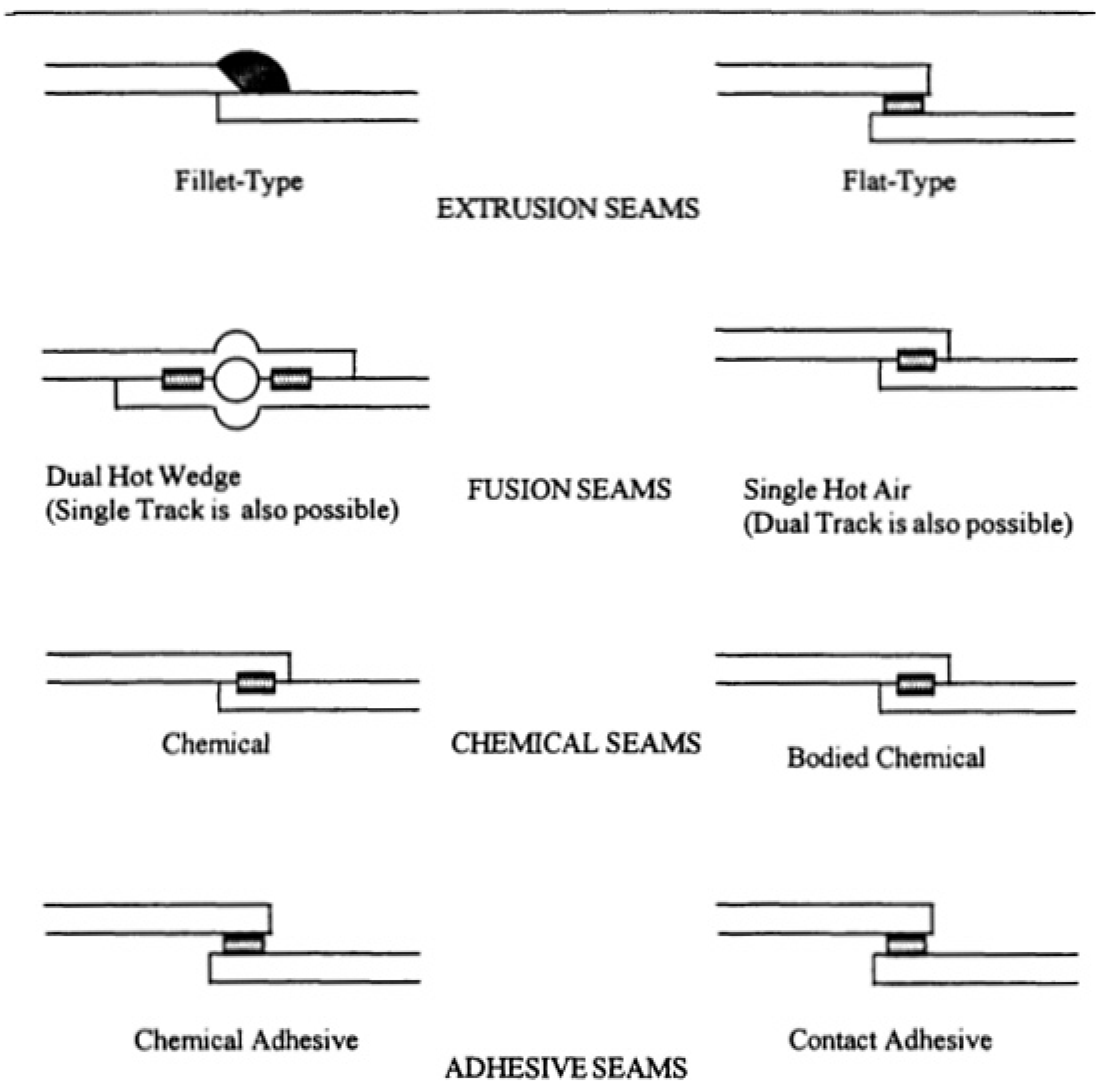

FIGURE 2 Various Geomembrane Seaming Methods (USEPA, 1993).

SUBTITLE C COVER DESIGN INCORPORATING GEOMEMBRANES

The objectives of a surface barrier system is to limit the infiltration of water into the contained waste to minimize leachate generation that potentially could escape to ground water in close proximity to the bottom of the waste cell. Minimizing leachate in a closed cell requires that liquids be kept out and that leachate that does exist be collected and removed. The surface barrier system must be designed at the time the site is selected and the design of the containment system is chosen. The site location, availability of low-permeability clay soil, and good topsoil, the use of geosynthetics, and the use of the site after the post-closure care period are typical considerations in design. The ultimate goals of the barrier system are to minimize maintenance and to protect human health and the environment.

The Resource Conservation and Recovery Act (RCRA) Subtitle C regulations form the basic requirements for cover systems being designed and constructed today. In general, after the waste cell is closed, the U.S. Environmental Protection Agency (EPA) recommends as a minimum that the final cover system consist of the following major elements from bottom to top (USEPA, 1991):

1. Low Hydraulic Conductivity Geomembrane/Soil Layer. A 60-cm layer of compacted natural or amended soil with a hydraulic conductivity of 1×10^{-7} cm/sec in intimate contact with a minimum 0.5-mm thick geomembrane.
2. Drainage Layer. A minimum 30-cm soil layer having a minimum hydraulic conductivity of 1×10^{-2} cm/sec, or a layer of geosynthetic material having the same characteristics.
3. Top, Vegetation/Soil Layer. A top layer with vegetation (or an armored top surface) and a minimum of 60 cm of soil graded at a slope between 3 and 5 percent.

The basic generic design incorporates natural soil, geomembranes, drainage, and vegetative layers. It is obvious that the geomembrane/natural soil layer is an integral part of the barrier concept. Figure 3 illustrates the EPA recommended minimum cover system with optional geosynthetics layers as well as barrier intrusion layers.

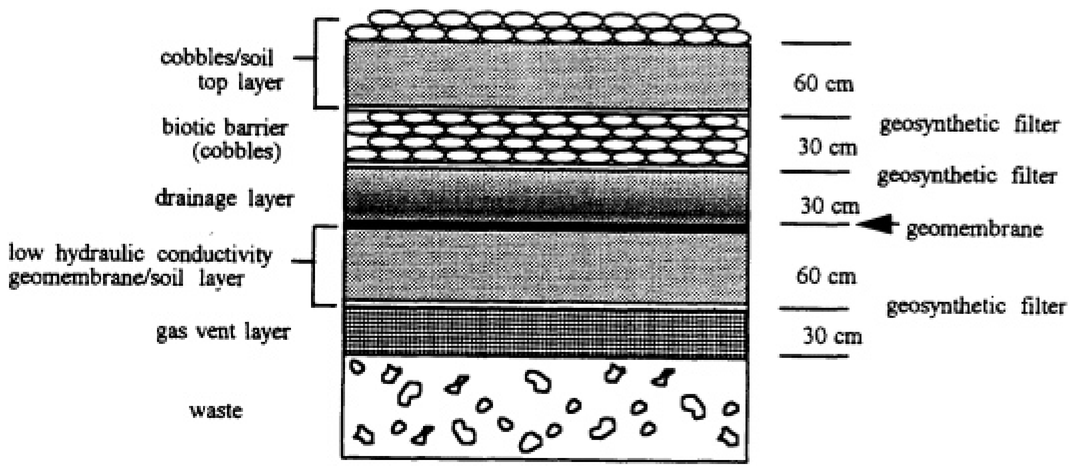

FIGURE 3 EPA Recommended Cover System With Optional Layers (USEPA, 1991).

Because the design of the final cover must consider the site, climate, waste characteristics, and other site-specific conditions, the minimum recommendations may be altered providing the alternative design is equivalent to the U.S. EPA-recommended design or will meet the intent of the regulations (USEPA, 1991). In extremely arid regions, for example, a gravel top surface will compensate for reduced vegetation. Where burrowing animals might damage the geomembrane/low-hydraulic conductivity soil layer, a biotic barrier layer of large-sized aggregate will be needed above it. Where the type of waste may create gases, soil or geosynthetic gas, vent layers and escape vents would need to be included. The recommended Subtitle C cover design has a relatively short design life of usually no more than 30 years, during which time the barrier can be monitored. Many wastes, however, especially those considered by the Department of Energy (DOE) and the U.S. Nuclear Regulatory Commission (USNRC) must be isolated for much longer periods of from 100 to over 1,000 years. In developing a cap design for the long-term, designers of DOE type enclosures generally shy away from geosynthetic materials that are assumed to last only decades and intuitively rely on natural materials.

GEOMEMBRANE DURABILITY

In addition to the known physical/mechanical design values used in state of the art geomembrane design, there is always the consideration of product durability and aging over the design life of the containment system. As described earlier, the most frequently utilized geomembrane group in cover applications are the thermoplastics. However, as shown in Table 1, the final compound will include various additives, stabilizers or plasticizer in addition to the predominant resin type. Ideally, each component in the compound formulation must be analyzed when assessing the durability or long-term aging of a particular geomembrane. Those geomembrane polymer systems with very few additives, such as HDPE, are much easier to analyze for durability and aging mechanisms.

According to Koerner (1994), there are six separate primary mechanisms that can be attributed to geomembrane degradation over time by causing polymeric chain scission or bond breaking within the polymer structure:

- ultraviolet degradation
- radioactive degradation
- biological degradation
- chemical degradation
- thermal degradation
- oxidation degradation

Any combination of the above mechanisms can cause synergistic effects due to interaction. In addition, elevated temperatures and stress will accelerate any of the degradation processes. Geomembranes that are exposed to the elements or incorporated in shallow barrier systems (<1 m) with no biotic barrier protection are susceptible to most of the above degradation mechanisms. However, in a buried situation, such as a cap or surface barrier that may be 3-4 m in layered thickness above the geomembrane, most of the primary degradation mechanisms are nonexistent or of little concern. In addition, elevated temperatures and chemical degradation are not an issue. The only liquid that potentially could come into contact with the geomembrane is water due to potential infiltration or soil moisture. The temperature at 3-4 m depth will remain constant over time and probably will not be above 14-18 °C.

Of all of the thermoplastic materials, PVC-soft is the most susceptible to the effects of aging, primarily due to the high content of volatile plasticizer, fillers, and other additives. However, field exhumed samples of 30-year-old PVC geomembrane under shallow burial and saturated soil conditions show an intact geomembrane that is still serving its intended function even with reduced physical/mechanical properties. This same geomembrane will no doubt be in service for an additional 30-100 years.

The semi-crystalline polyolefins, such as HDPE, which are composed of over 95 percent resin are much less susceptible to many of the degradation processes, especially in surface barriers that are 3-4 m in layered thickness.

All geomembranes are subject to the oxidation mechanism. Gases and liquids (water or soil moisture) that come into contact with the geomembrane will cause oxidation and, over time, oxygen will enter the polymer structure. The rate of oxidation reaction is specific to the site and polymer. The reaction is minimized by providing antioxidants in the polymer formulations that neutralize free radicals. Removal of oxygen from the geomembrane's surface (i.e., submerged or covered with waste) will eliminate the potential for oxidation degradation. However, in a deep cover design, the geomembrane will be encapsulated by nonsaturated soil and thus will be susceptible to oxidation. An accelerated test method commonly used in determining reaction time and antioxidant levels is the Oxidation Induction Time (OIT).

Arrhenius Modeling at elevated temperatures currently is being used to determine the projected life of semi-crystalline geomembranes. The Geosynthetic Research Institute (GRI) is currently exposing HDPE samples to superimposed compressive stress, chemical and oxidative exposure, elevated temperature and long testing time. Samples are removed periodically and evaluated for numerous property changes. Deciding on a maximum property change, such as a 50 percent reduction at each temperature allows for the plotting of the Arrhenius curve, which plots inverse temperature against reaction rate. Thus, Arrhenius Modeling can be used for lifetime prediction using elevated temperature aging (Koerner, 1994). The pipe and cable industry have used Arrhenius Modeling methodology extensively in the determination of in-service lifetimes for buried plastic pipe and cable, and much of this documented information is being used to model geomembrane aging characteristics.

SUMMARY

Geomembranes are highly technical engineered systems that form an integral part of today's surface barriers. Obviously, the longevity or durability of a geomembrane is the most important design issue when considering barrier systems that must last 50, 100, 200, or even 1,000 years. However, the primary degradation mechanisms are generally eliminated by burial in long-term cover systems that are in the range of 3-4 m in thickness. In addition, liquid contact, if any, will be limited to potential water infiltration or soil moisture, and thus chemical degradation will not be an issue. Proper stress-limiting design and strict manufacture/installation quality assurance coupled with the use of predictive models, such as the Arrhenius Modeling technique, can be used in helping to assure a long-term barrier utilizing geomembranes.

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EARTHEN MATERIALS IN SURFACE BARRIERS

Craig H. Benson, University of Wisconsin-Madison; and Milind V. Khire, GeoSyntec Consultants, Boca Raton, Florida

INTRODUCTION

The rising cost of remediating contaminated sites has made long-term isolation a viable alternative to the removal and treatment of contaminated soils and wastes. Isolation generally consists of installing subsurface barriers (vertical cutoff walls and/or horizontal liners placed in situ) and caps. In some cases, where lateral and vertical migration of contaminants can be limited by minimizing percolation, isolation may consist of only installing a cap.

The focus of this paper is on caps constructed with earthen materials. Caps may also be constructed with geosynthetic materials (geomembranes, geonets, etc.). However, for large remediation projects, the costs of constructing a cap with geosynthetics may be enormous relative to the cost of a cap constructed with earthen materials. Furthermore, in some regions such as the semi-arid and arid parts of the United States, caps constructed with earthen materials may perform equally as well as caps constructed with geosynthetics.

An effective cap design requires that the cap limit percolation into the underlying soils. For earthen caps, design consists of selecting an arrangement of earthen materials that can be used to divert water away from the cap or store the water until it can be removed later by evaporation and/or transpiration. The materials and their arrangement must be selected in such a way that the cap will minimize percolation and continue to function throughout a long design life.

WATER BALANCE OF EARTHEN CAPS

Earthen caps generally exploit the unique characteristics of unsaturated flow, the storage capacity of fine-grained soils, and the natural capacity of plants to remove water entering the cap during wet periods. These factors are linked by the water balance, which accounts for movement of water into, within, and out of a cap. In algebraic form, the water balance can be described by the following equation:

$$P_r = P - O_f - S - E - T - L_f$$

where P is precipitation in the form of snow, rain, or ice; O_f is overland flow; S is soil water storage; E is evaporation; T is transpiration by vegetation; L_f is lateral drainage; and P_r is percolation from the base (Figure 1). In some cases, evaporation and transpiration are combined as evapotranspiration (E_t).

Design of an earthen cap to minimize percolation entails manipulating soil water storage capacity, lateral drainage, transpiration, and evaporation such that an acceptable value for percolation occurs in a worst-case design year (e.g., year when rainfall or snowfall is abnormally high and temperature and solar radiation are abnormally low). In particular, the cap is designed such that it has adequate capacity to store or divert water that infiltrates during late fall and winter and sufficient vegetation and evaporative potential to remove the stored water during spring, summer, and early fall. Three types of earthen caps that are designed on this principle are (1) traditional resistive barriers, (2) capillary barriers, and (3) monolayer soil barriers. In some cases, these barriers are used in combination.

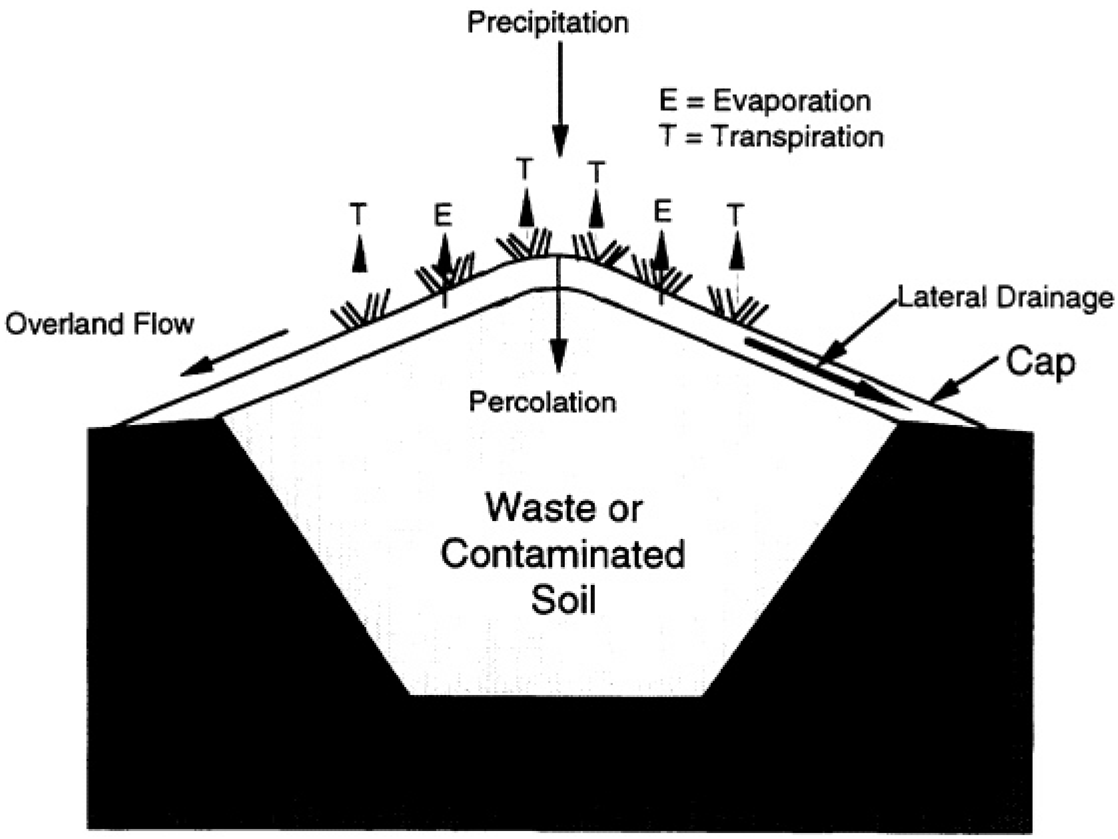

FIGURE 1. Water Balance of a Cap.

RESISTIVE BARRIERS

Principle

Traditional resistive barriers are the most commonly used earthen caps today, even though their cost, performance, and durability can be poorer than other types of earthen caps. A traditional resistive cap consists of a barrier layer of compacted fine-grained soil and a vegetated surface layer (Figure 2). In some instances, a drainage layer is added between the compacted barrier layer and the vegetated surface layer. Also, a "rooting zone" or "weathering layer" may be added beneath the vegetated layer to protect the barrier layer from root intrusion, desiccation, and frost action.

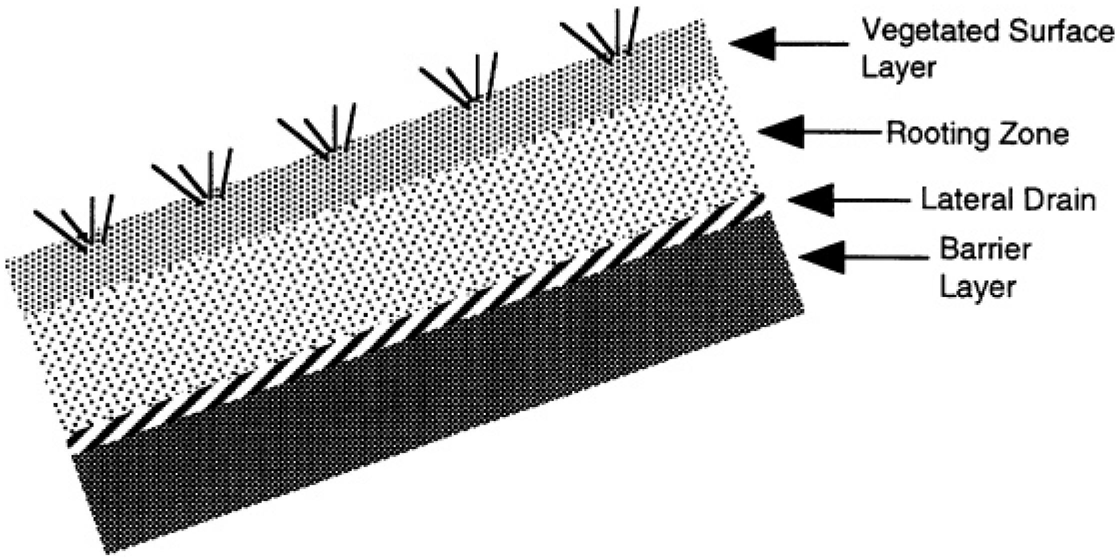

FIGURE 2 Cap with Traditional Resistive Barrier.

Adequate performance of a traditional resistive barrier is predicated on the barrier layer having low saturated hydraulic conductivity. If the barrier layer has low saturated hydraulic conductivity, percolation from the base of the cap is controlled by the saturated hydraulic conductivity of the barrier layer and the gradient imposed by water ponding on the barrier layer. Furthermore, when the saturated hydraulic conductivity is low, water is more likely to be diverted as overland flow or lateral flow in a drainage layer, and thus ponding on top of the barrier layer is minimized. Consequently, the maximum rate of percolation is equal to or slightly larger than the saturated hydraulic conductivity of the barrier layer because the hydraulic gradient is close to unity.

Potential Problems

Resistive barriers are prone to failure because fine-grained barrier layers are easily damaged by weathering and distortion. As a result, their saturated hydraulic conductivity increases significantly and the performance of the cap degrades. For example, Benson et al. (1995) have shown that barrier layers exposed to frost become severely cracked, and their saturated hydraulic conductivity can increase dramatically. In some cases, the saturated hydraulic conductivity can increase as much as four orders of magnitude (Figure 3). Desiccation can also cause extensive cracking of the barrier layer, which results in preferential pathways for flow. A large desiccation crack is shown in Figure 4. This crack formed in a resistive barrier landfill cap within eight years of construction. Similar cracks have been reported by Montgomery and Parsons (1990). Desiccation also causes networks of small cracks in the matrix between the large cracks. These small cracks result in the saturated hydraulic conductivity of the matrix increasing by one to two orders of magnitude (Benson, 1994). Finally, distortion of the barrier layer due to differential settlements also results in cracking, which will degrade the performance of a cap (Jessberger and Stone, 1991).

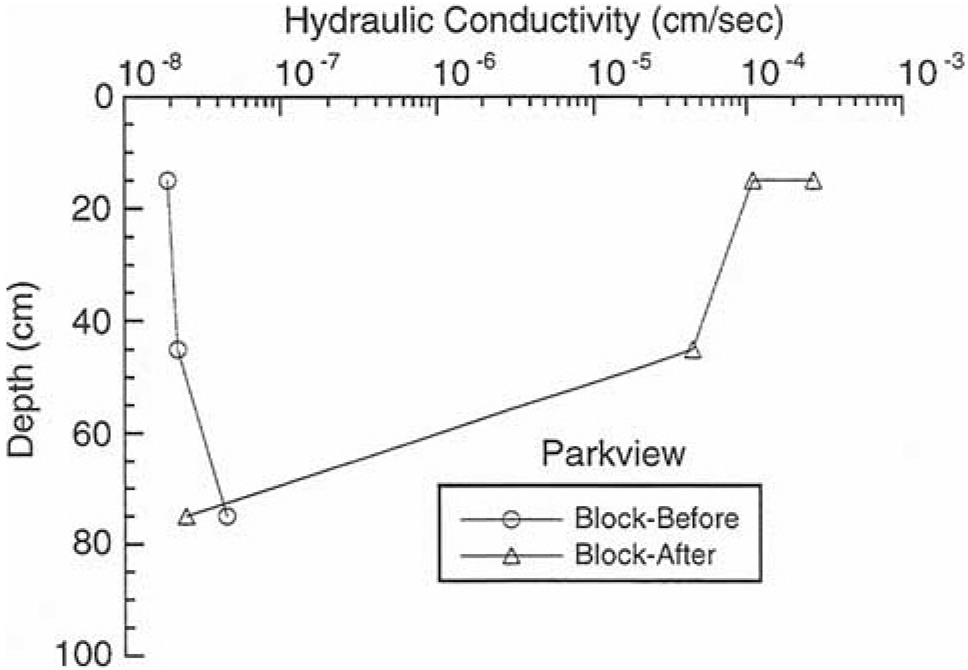

FIGURE 3 Hydraulic Conductivity Before and After Freeze-Thaw (adapted from Benson et al. 1995)

FIGURE 4 Desiccation Crack in Cap Designed as a Resistive Barrier

Case History

Khire, Benson, and Bosscher (1994) show how desiccation cracks can impact the performance of a resistive barrier. A test section was constructed in East Wenatchee, Washington (annual rainfall = 23 cm) to simulate a resistive barrier. The resistive barrier consists of 600 mm of compacted silty clay and 150 mm of vegetated silty topsoil. The test section is instrumented extensively to obtain detailed information about the water balance (Benson et al., 1994).

Cumulative percolation from the test section is shown in Figure 5. The data through Summer 1994 show that a small quantity of percolation is generated each year from late fall through early spring. This percolation is the result of winter rainfalls and snow melts, which saturate the barrier. In 1995, however, the quantity of percolation increased dramatically in the spring, even though the amount of precipitation in Winter 1995 was essentially the same as in Winter 1994 and much less than in Winter 1993. Examination of the barrier layer showed that vertical desiccation cracks had formed during drying the previous summer. These cracks apparently became conduits for preferential flow during winter.

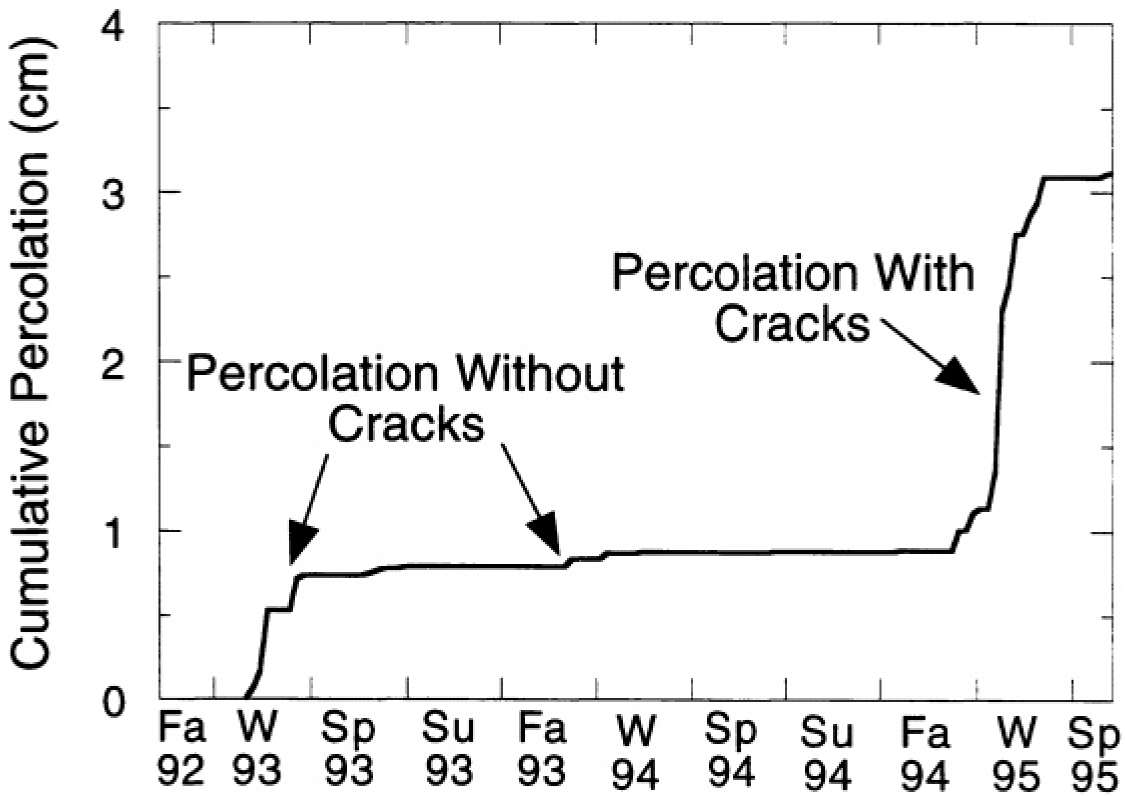

FIGURE 5 Cumulative Percolation from Resistive Barrier with Preferential Flow.

CAPILLARY BARRIERS

Principle

In their simplest form, capillary barriers employ a fine-grained layer over a coarse-grained layer (Figure 6a). Flow across the interface of these layers is restricted under unsaturated conditions because the unsaturated hydraulic conductivity of the coarse-grained layer is much lower than the unsaturated hydraulic conductivity of the fine-grained layer (Figure 6b) (Hillel, 1980). Also, suctions in the fine-grained layer normally exceed suctions in the coarse-grained layer, which results in an upward hydraulic gradient (Khire, *1995).* Thus, the fine-grained soil can store or divert water that infiltrates the cap, and yet flow into the coarse-grained layer is restricted. More elaborate designs employing multiple layers having contrasting grain size are also possible. These caps employ the capillary barrier principle to divert infiltrating water via lateral flow while ensuring that deep percolation does not occur (Nyhan et al., 1993; Yeh et al., 1994).

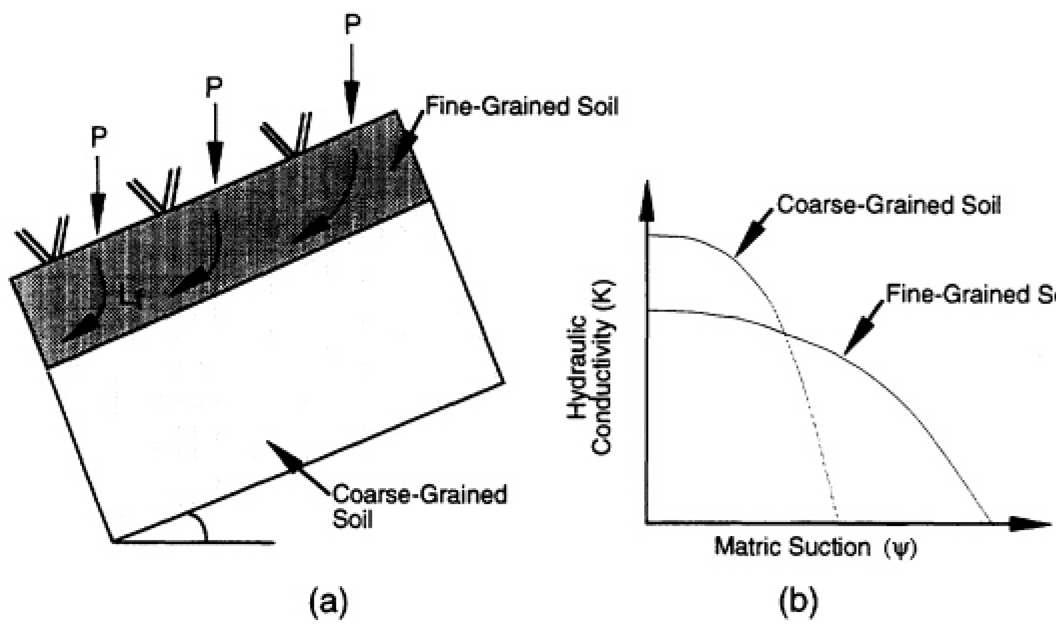

FIGURE 6 Schematic of Capillary Barrier (a) and Hydraulic Conductivity Functions (b).

Potential Problems

Several problems can be encountered with capillary barriers. A particularly problematic condition is when the storage capacity of the fine-grained layer is overwhelmed in regions where significant snow cover accumulates. When the storage capacity is exceeded, rapid breakthrough across the interface between the fine and coarse-grained layers occurs. Percolation during this condition can be significant (Hakonson et al., 1994; Khire, 1995). Defects such as cracks and macropores in the fine-grained layer can also affect the performance of a capillary barrier. If defects exist, water can immediately enter the underlying coarse-grained layer and percolation is generated. Finally, the performance of capillary barriers is sensitive to heterogeneities in the soil layers. Variations in texture or hydraulic properties can result in premature breakthrough across the interface or finger flow in the coarse-grained layer (Khire, 1995). Consequently, percolation can be greater than expected.

Case Histories

Several studies have shown that capillary barriers can be effective in limiting percolation, even in more humid regions. For example, in a study of five test sections located at a landfill near Hamburg, Germany (average annual precipitation is 800 mm), Melchoir et al. (1994) report that capillary barriers, when used in conjunction with clay resistive barriers, can reduce percolation to virtually zero.

Nyhan et al. (1993) also report that capillary barriers can be superior to traditional resistive barriers. In their study, test sections were constructed at Los Alamos National Laboratory. The test sections consisted of the following cap designs: top soil underlain by crushed tuff, an EPA (Environmental Protection Agency)-recommended resistive barrier cap (topsoil layer, sand drainage layer, and a 600-mm-thick soil layer having saturated hydraulic conductivity less than $1 \times 10\text{-}9$ m/s), a loam capillary barrier (topsoil layer, fine sand wicking layer, gravel layer, and coarse sand layer), and a clay-loam capillary barrier (clay-loam layer, fine sand wicking layer, gravel layer, and coarse sand layer). The test sections were monitored for 15 months. Analysis of the data showed that all of the test sections produced percolation, and that the EPA-recommended resistive barrier and the loam capillary barrier produced the largest quantity of percolation (8.5 and 7.4 percent of precipitation, respectively). The clay-loam capillary barrier produced the least amount of percolation (0.7 percent of precipitation).

Hakonson et al. (1994) field tested four designs being considered for a cap at Hill Air Force Base in Utah (precipitation is 510 mm/yr). The first cap consisted of sandy loam overlying a gravel drainage layer, which effectively was a simple, two-layer capillary barrier. The second cap was a resistive barrier and consisted of sandy loam topsoil overlying a sand drainage layer and a layer of clay-loam amended with bentonite. The remaining two caps were capillary barriers having sandy loam topsoil over washed gravel. These two capillary barrier test sections were seeded with different vegetation.

Hakonson et al. (1994) report that percolation from the resistive barrier was the least (0.006 percent of precipitation), whereas percolation from the simple, two-layer capillary barrier was greatest (24 percent of precipitation). Both capillary barriers produced percolation that was approximately 15 percent of the precipitation. Percolation from the capillary barriers was large because snow accumulated on the test sections, which overwhelmed their capacity for storage and diversion.

Khire et al. (1994) describe the water balance of a capillary barrier test section constructed adjacent to the previously discussed resistive barrier test section in East Wenatchee, Washington. The capillary barrier has a sand layer overlain by a vegetated silty surface layer. The capillary barrier has been effective, limiting percolation to 0.6 percent of precipitation. This is substantially less percolation than has been produced by the resistive barrier test section (4.4 percent of precipitation). Khire et al. (1994) also report that the capillary barrier can be ineffective when snow accumulates on the cap and the subsequent melt overwhelms the storage and diversion capacity of the fine-grained layer.

MONOLAYER BARRIERS

Principle

Monolayer barriers are caps consisting of a thick layer of fine-grained soil. Monolayer barriers exploit two characteristics of fine-grained soil: (1) their large soil water storage capacity when unsaturated, and (2) their low saturated hydraulic conductivity relative to coarse-grained soils. Low saturated hydraulic conductivity limits infiltration through the surface during rainfall or snow melt. High soil water storage capacity provides the capability to store water that does infiltrate until it can be removed by evapotranspiration. The barrier must be sufficiently thick to ensure that changes in water content do not occur near its base, i.e., all changes in soil water storage occur in the upper portion of the barrier (Figure 7). Otherwise, percolation will occur. Monolayer barriers are constructed from silty sands, silts, and clayey silts, and are cost-effective when large quantities of fine-grained soil requiring little processing are available on site.

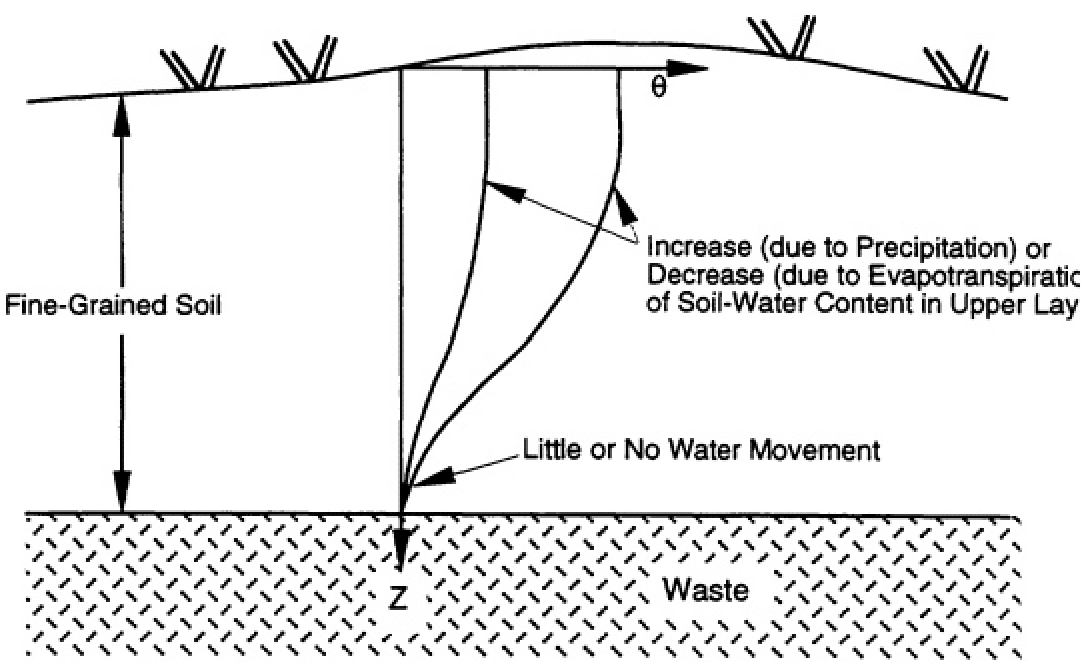

FIGURE 7 Principle of Monolayer Cap.

Potential Problems

The potential problems associated with a monolayer barrier are similar to those for a capillary barrier. Exceeding the storage capacity during snow melt, however, is probably not significant if a monolayer barrier is thick. Monolayer barriers may have problems shedding moisture via evapotranspiration if deep percolation occurs after intense infiltration events.

Case Histories

Morrison-Knudson (1993, 1994) describe the performance of a monolayer barrier proposed as a cap over contaminated soil at Rocky Mountain Arsenal in Commerce City, Colorado. The barrier, which is constructed over trenches containing hazardous constituents, consists of a vegetated layer of topsoil underlain by a compacted layer of fine-grained soil (Figure 8). Analysis of monitoring data has shown that most changes in water content occur in the upper 0.6 m of the cap. However, gradual changes in water content have been observed in the deepest probes (1.2 m). Unfortunately, no percolation lysimeters were installed beneath the cap. Thus, the true performance of the cap cannot be evaluated.

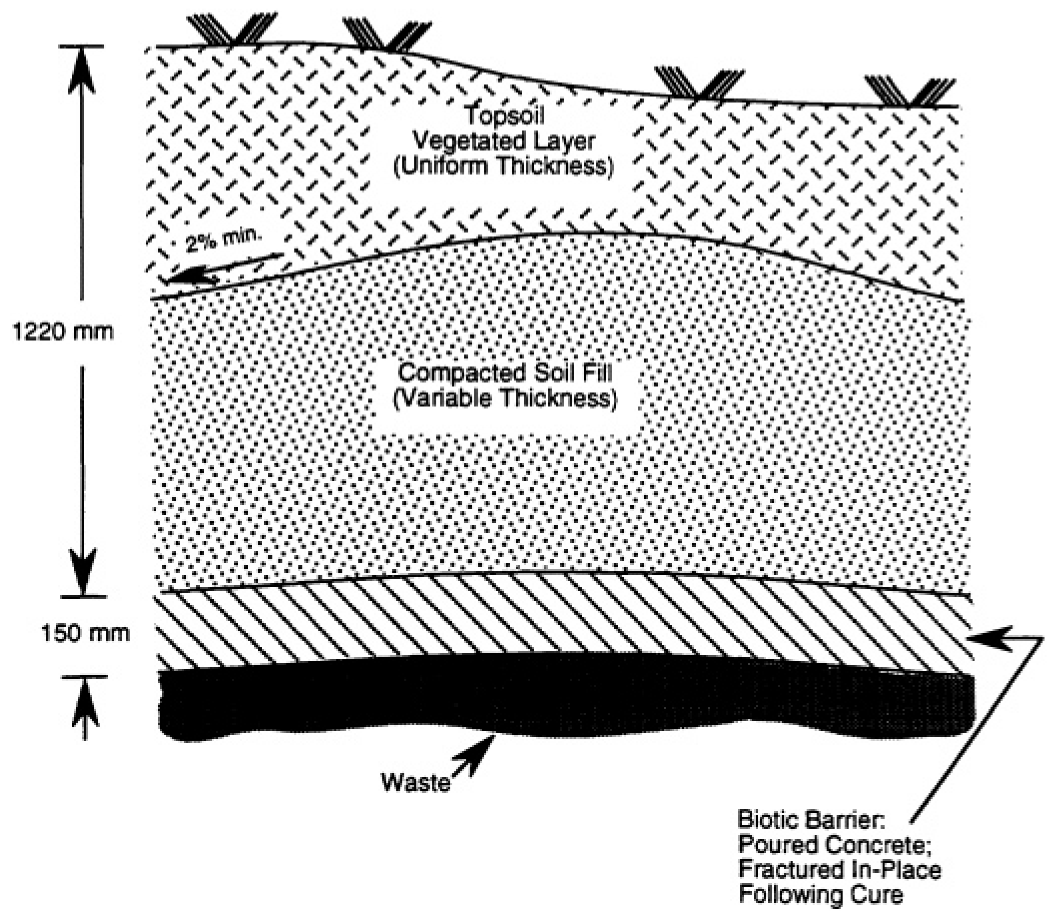

FIGURE 8 Monolayer Cap Considered for Rocky Mountain Arsenal.

GeoLogic Associates (1993) describe a thick soil barrier used as a cap for a landfill in Southern California. The barrier is 2 m thick and is constructed from a clayey silt. To date, water movement has been limited to the upper 0.6 m of soil. The data indicate that the water content of the upper soil layers increases rapidly after rainfall and then gradually decreases as water is removed by evapotranspiration. Unfortunately, percolation is not being monitored, and thus, the actual performance of the cap cannot be determined.

SUMMARY

Earthen caps can be effective barriers to percolation into underlying waste or contaminated soil provided they are designed for the climate in which they will be placed. Currently, three designs for earthen caps are being used: resistive barriers, capillary barriers, and monolayer barriers. Resistive barriers rely on the low saturated hydraulic conductivity of a barrier layer to limit percolation. In contrast, capillary barriers and monolayer barriers generally rely on the soil water storage capacity of fine-grained soils to store infiltrating water until it can be removed by evapotranspiration. Capillary barriers also exploit the contrasting unsaturated hydraulic properties of fine and coarse-grained soils.

Each of the three designs is prone to its own set of problems. Resistive barriers are especially likely to experience degrading performance because it is difficult to protect a barrier layer having low saturated hydraulic conductivity from deteriorating as a result of weathering or settlement. Capillary barriers are not prone to the same problems as resistive barriers but can fail if the storage and diversion capacity of the fine-grained layer is undersized, particularly in regions where significant snow cover accumulates. In contrast, the storage capacity of a monolayer barrier probably will not be overwhelmed, but a monolayer barrier may have difficulty recovering from intense infiltration events. Nevertheless, the problems associated with each of these designs can be avoided by carefully considering the conditions to which the cap will be exposed during its design life.

ACKNOWLEDGMENT

Financial support for a portion of the work described in this paper has been provided by the National Science Foundation through grant number CMS-9157116. This paper has not been reviewed by NSF and no endorsement should be assumed

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COMPARISON OF CLAY AND ASPHALTIC MATERIALS FOR USE AS LOW-PERMEABILITY LAYERS IN ENGINEERED COVERS AT THE ROCKY FLATS ENVIRONMENTAL TECHNOLOGY SITE

M.J. Glade and P.A. Nixon, Parsons Engineering Science, Denver, Colorado

INTRODUCTION

The Rocky Flats Environmental Technology Site (RFETS) located northwest of Denver, Colorado, operated five lined Solar Evaporation Ponds (SEPs) from 1953 until 1986 for the disposal of liquid radioactive and hazardous waste. The U.S. Department of Energy (DOE) has signed an Interagency Agreement (IAG) with the Colorado Department of Public Health and the Environment (CDPHE), and the U.S. Environmental Protection Agency (USEPA), and has agreed to close the SEPs through an Interim Measures/Interim Remedial Action (IM/IRA) accelerated program. Through an alternatives analysis study, DOE selected an alternative to consolidate contaminated pond liners, subsurface soils, stabilized sludge, and miscellaneous contaminated debris from within the operable unit (OU) under an engineered cover.

Based on the Colorado hazardous waste landfill siting regulations (6 CCR 1007-2, part 2), the DOE has designed an engineered cover closure system with an anticipated functional lifetime of 1,000 years. The engineered cover must also meet the Resource Conservation and Recovery Act (RCRA) requirements for the closure of a surface impoundment (40 CFR 265.111 and 40 CFR 265.228). Since regulatory guidance for long-term engineered barriers does not exist, the DOE, CDPHE, and USEPA agreed that synthetic (human-made) materials that have limited long-term performance testing results should not be included as functional components of the engineered cover. Therefore, the engineered cover is designed with natural materials that have expected long-term durability. Figure 1 presents a cross section of the proposed engineered cover. The asphaltic low-permeability layer consists of a composite of a fluid applied asphalt (FAA) over asphaltic concrete.

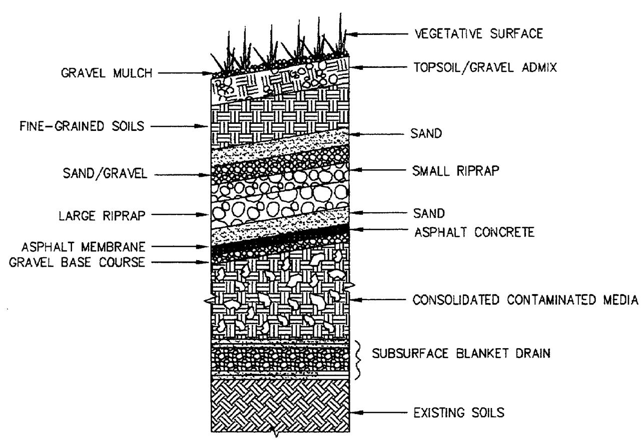

FIGURE 1 Engineered Cover Cross Section.

Low-permeability clay and asphaltic materials (hot mix asphalt concrete and fluid applied asphalt) were compared for use in the engineered cover for the SEPs. Low-permeability clay layers have been used in numerous engineered covers throughout the DOE complex as well as at many other sites. Research has been conducted at the DOE Hanford Reservation, Washington, for 10 years with respect to engineered barriers in semi-arid environments. The Hanford researchers have constructed an asphalt low-permeable layer in a prototype engineered cover with a 1,000-year design life. DOE selected asphalt for the low-permeable layer in the engineered cover for the OU4 SEPs.

This paper presents a comparison of low-permeable clay and asphaltic materials with respect to the specific environmental conditions in Colorado and the design criteria of the engineered cover. The comparison factors include hydraulic conductivity, longevity, engineering characteristics, constructability, and cost.

SELECTION CRITERIA

Selection criteria for the low-permeability layer were established based on the performance requirements, engineering characteristics, constructability, and cost. The following subsections discusses each of the specific selection criteria.

Hydraulic Conductivity

Hydraulic conductivity (permeability) is a material property that is a measure of the ease with which a liquid can be transmitted through a porous material. Clay soils have been used in traditional compacted clay cover systems to meet RCRA requirements and have been shown to have permeabilities below 1×10^{-7} cm/s (USEPA, 1988). However, Day and Daniel (1985) indicated that field-measured permeability values could be 1000 times greater than laboratory measured permeability values possibly due to the numerous hydraulic defects found in field compacted clay liners. Factors that can influence the inherent permeability of a clay material include the void ratio, composition, fabric, and degree of saturation for the material. Field placement factors such as compacted water content, compactive energy, and desiccation can also influence the permeability of a clay material (USEPA, 1991). Large variabilities in the permeability of clays can exist due to all of these factors.

The permeability of asphalt concrete is primarily controlled by the asphalt content, mineral filler content, and air void content, which are determined from the mix design. As a general rule, asphalt type, aggregate type, and batch production variables have only a minor influence on the permeability of the asphalt concrete. However, a well-graded aggregate is preferred. A low air void content will reduce the permeability of the mix. Air void contents are not only controlled by the mix design but also by the energy of compaction. Standard roadway construction equipment may be modified to produce the desired permeability. Asphalt linings 2 inches thick having 4 percent or less air voids will generally have a permeability less than 1×10^{-7} cm/s (Asphalt Institute, 1989).

Freeman, Romine, and Zacher (1994) conducted detailed laboratory and field permeability tests on asphalt concrete samples for the Hanford project. Laboratory tests indicated a permeability range of 1.05×10^{-11} cm/s to 6.96×10^{-11} cm/s for an asphalt content of 6.5 percent, and 2.09×10^{-12} cm/s to 8.25×10^{-11} cm/s for an asphalt content of 7.5-percent. The Hanford researchers also performed field permeability tests on a prototype barrier constructed of two *7.5* cm layers of compacted hot-applied asphalt concrete with an asphalt content of approximately 7.5 percent and compacted it greater than 95 percent of theoretical maximum density (151 lb/ft3). A modified falling head permeability test was conducted at five locations of the prototype barrier, and permeability results ranged from 1.91×10^{-9} cm/s to 1.08×10^{-7} cm/s. The Hanford researchers believe these numbers are conservative due to the fact that the samples were not confined and that the bentonite used to seal the rings of the test was still hydrating and taking up water. Permeability changes of an asphalt concrete after placement are due primarily to an increase in cracks within the asphalt concrete cover. These cracks can be generated by an improper mix design (low asphalt cement content), by freeze-thaw cycles, through the mechanism of asphalt cement stripping by water, by age hardening and embrittlement, or by stress cracks that develop due to consolidation or settlement of underlying materials.

Research was conducted under the OU4 IM/IRA accelerated program to ensure that the proposed Deery Oil, Inc., fluid applied asphalt would perform adequately during actual field loading conditions. Confined permeability tests in accordance with ASTM D-5084 were performed with an applied normal load of 480 pounds per square foot (psf). The FAA was first applied to concrete to the desired thickness and was then covered with a layer of 1/4-inch gravel prior to the application of the normal load. The laboratory permeability test results for both 160-mil- and 87 mil-thick FAA membranes were below 1×10^{-11} cm/s (the limits of the test apparatus) and indicate that gravel embedment in the FAA should not influence the results of permeability testing and field performance dramatically. Freeman et al. (1994) also conducted permeability tests on Deery Oil, Inc., FAA that were removed from an actual placement over asphalt concrete. The results from four samples ranged between 1.18×10^{-11} cm/s and 2.51×10^{-11} cm/s. They concluded that no direct correlation between sample thickness and permeability was apparent.

The IM/IRA accelerated program determined, based on permeability, that the variability of clay materials would make it difficult to predict the field behavior of a compacted clay cover over the 1000-year design life of the system. The local arid climate and the difficulty in locating a borrow source with the desired properties made clay an inappropriate choice based on permeability. Asphalt concrete materials and equipment are available locally. Previous laboratory and field tests confirm that asphalt concrete should meet RCRA requirements for permeability for the closure of a surface impoundment when placed at a higher asphalt content and lower air void content than typical road base designs. The asphalt concrete in combination with the FAA should provide a redundant system with added protection against the development of cracks within the low-permeability layer.

Longevity

The longevity of the clay and asphalt materials were compared with respect to the ability of the materials to retain the desired properties over the 1000-year design life of the cover system. The longevity of a clay soil at the RFETS is expected to be influenced primarily by the ability of the clay cover to retain low-permeability characteristics. No chemical interaction with the contaminated media should be expected within the cover. The major concern with utilizing a clay soil is that desiccation cracks may develop both during construction and after placement. Should excessive differential settlement occur under the cover system, the tensile stains could further lead to cracking of the clay layer. Since the climate is arid, the clay layer could have a relatively low water content, which could enhance the cracking of the cover. Traditional longevity arguments for the use of asphalts have centered around archeological excavation data. However, more specific data on the longevity of "refined" or "modified" asphalts is required. The IM/IRA accelerated program focused on reviewing the research conducted by the Hanford researchers and by the Strategic Highway Research Program (SHRP, 1994).

The SHRP provided important insight into the oxidative aging of asphalts. Excessive oxidative aging can lead to asphalt hardening and embrittlement, which may ultimately contribute to cracking. It was found that the aging of asphalts is a function of their physical state and is highly temperature dependent. Temperature typically determines the rate of oxidation, the amount of oxidation, and the ultimate age hardening of the asphalt. The SHRP aged asphalts under pressure and at temperatures specific to, and higher than, application temperatures. This is an important factor, since the asphalt materials under the cover will not be exposed to UV light. However, tests to determine the mechanism of age hardening were specific to pavement temperatures (140°F) and higher temperatures (266°F). It was found that "asphalts with different component compatibilities may exhibit similar age-hardening kinetics in the low end of the pavement temperature range but quite different kinetics in the high end." The program also determined that long-term changes in the physical properties of an asphalt are a function of temperature and asphalt composition. Plots (Figure 2) of the changes in viscosity with temperature for different asphalts were constructed from samples using aging index data from 144 hour tests. These plots indicated that aging characteristics cannot be interpolated between two different temperatures. Another important trend of this plot is how the aging index converges between the asphalts at the lower test temperatures. It is important to note that the test temperature ranges were 140°F (60°C) to 180°F (80°C). This plot indicates that the lower the actual field temperatures are from the aging test temperatures, the less reliable the aging data. The SHRP concluded that the higher-temperature tests produced more oxidation and more stiffening of the asphalts than when the asphalts were tested at lower temperatures. The SHRP also determined that asphalts that were coated onto aggregates did not demonstrate significantly different aging characteristics from when the asphalts were aged without aggregate. The aggregate type did not appear to alter the aging of the asphalts.

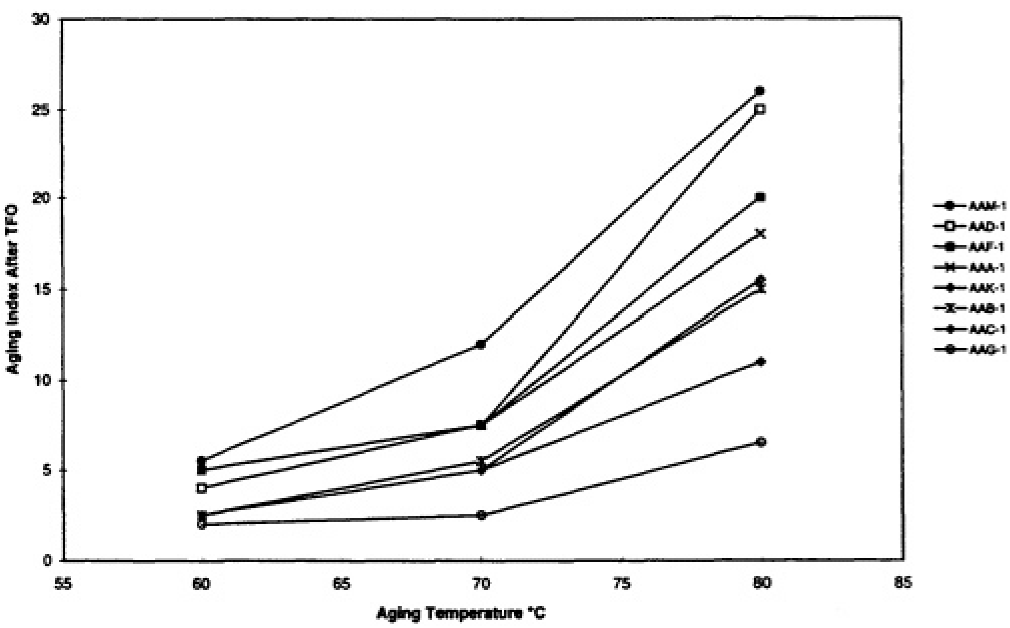

FIGURE 2 Aging Indices After Thin Oven Testing of Core Asphalts Aged by the Pressure Air Vessel Method a Different Temperatures.

Concerns were raised during the IM/IRA accelerated program that "refined" asphalts may age more quickly than "natural" asphalts. At this time, simple conclusions based on the refinement of an asphalt cannot be made. The SHRP utilized a number of asphalts that demonstrated a wide range of behavior patterns. The variability of the aging behavior of asphalts is more chemistry related than process related. The SHRP stated that the physical properties of an asphalt are best described by the effectiveness by which the polar associating materials are dispersed by a solvent moiety consisting of relatively nonpolar aliphatic particles, rather than by strict parameters such as elemental composition. The longevity of asphalt materials is specific to the type and properties of each individual asphalt. Traditional aging indexes (aged viscosity divided by unaged viscosity) may not be relevant for applications involving 1000-year criteria. The SHRP developed effective accelerated aging procedures and used direct rheological measurements (penetration, softening point, viscosity, etc.) that have made aging indexes unnecessary. An accurate assessment of the longevity of asphalt materials can not be made without the benefit of further testing. Furthermore, aging mechanisms at lower temperatures (58°F for buried asphalts) have not been detailed in previous investigations.

Total and differential settlement of the asphaltic concrete and the FAA should be limited due to the degree of compaction of the underlying soils. The maximum estimated total settlement is 4 inches, while the maximum estimated differential settlement is approximately 0.4 percent (about 1/2-inch for every 10 feet). The performance of the hydraulic grade asphalt concrete and FAA should not be negatively impacted (by excessive cracks) due to the estimated settlements. The selection and design of the materials was based on providing an asphalt concrete that would offer some flexibility. In order to reduce the opportunity for cracking caused by settlement, the underlying materials below the cover were specified to be compacted to 95 percent of modified Proctor density.

Some other typical physical factors that may induce failures of normal asphalt concrete applications include:

- freeze-thaw stress changes, which can cause cracking of the asphalt concrete;
- traffic dynamic loading, which can cause vertical and horizontal movements in the asphalt concrete, which can lead to cracking; and
- shrinkage cracks, which can occur in asphalt concrete mixes that have a high content of low-penetration asphalt (Asphalt Institute, 1989).

The forces that produce these physical conditions on an asphalt concrete will not be present after the cover has been constructed.

Engineering Characteristics

The controlling factor for the design of a clay cover at the RFETS includes the 20-percent grade (11.3° slope) of the cover. Clay soils that will meet the permeability requirements may not meet slope stability requirements. RFETS slope stability analysis must include adjustments for seismic loading. The selection of a clay material must account for the large variabilities of friction angels and cohesion found in clays. The changes in the strength parameters due to compactive efforts should also be investigated closely. Effective friction angle differences can be as great as 7 degrees, while the differences in effective cohesion can be as great as 400 psf for compactive efforts ranging between 85 and 100 percent maximum standard Proctor density (Moretto et al., 1963).

The important engineering characteristics to be considered for the design of the asphalt concrete cover include its stability, durability, flexibility, impermeability, and workability during placement. The mixture design may require a trial and error laboratory testing program to determine the optimal design for the RFETS site. Stability of an asphalt concrete is the ability of the asphalt concrete to resist deformation from imposed loads (Asphalt Institute, 1989) and is determined by the internal friction and cohesion of the mixture. Internal friction of the mix is dependent upon the surface texture, gradation of aggregate, particle shape, density of the mix, and quantity and type of asphalt. In order to improve the stability of the mix, 70 percent of the aggregate materials retained on the No. 4 sieve were specified to have a minimum of two mechanically induced fractured faces. Stability is an important consideration, since 11 feet of cover material, which includes large riprap, was designed to be placed above the asphalt composite.

Durability is a property of asphalt that is its ability to resist detrimental effects of air, water, UV exposure, temperature extremes and variations, and dynamic loading. The durability of an asphalt concrete is generally improved by higher asphalt contents, well-graded aggregates, and low air void content. Increased asphalt contents result in thicker film coating of the mix aggregate. Thicker films are more resistant to age hardening (Asphalt Institute, 1989). Lower air void contents seal the mix from exposure to air and water.

Mixtures involving high asphalt contents with low air void contents require care during design. Asphalt contents that are too high produce a thick film thickness that may yield a mixture that is more prone to rutting, shoving, or creep. Constructability of this mix may be difficult on a sloped surface, and stability of the asphalt mix may be reduced due to the asphalt cement acting like a lubricant rather than a binder. Flexibility of an asphalt is required to compensate for minor differential or total settlements beneath the cover system. Higher asphalt contents with smaller size aggregates will increase flexibility within the asphalt concrete.

The engineering and construction properties that are important in asphalts are consistency and purity. The consistency of an asphalt is described as the viscosity or degree of fluidity of the asphalt at a given temperature. Consistency of a paving asphalt is commonly described by a penetration or viscosity test. The asphalt cement selected for RFETS was graded by viscosity methods at 140°F. The grade of the asphalt cement will effect the resistance of the asphalt cement to weather and physical changes and the stability of the asphalt cement to remain stable on significant side slopes. The purity of an asphalt can impact the longevity and performance of an asphalt concrete. Refined asphalts are composed almost entirely of bitumen, with a bitumen content of more than 99.5 percent. The impurities that may be present in the asphalt are usually inert and should not impact the performance or the grading of the asphalt concrete. Hence, asphalts that are graded the same may have dramatically different properties related to the longevity of the asphalt. Natural asphalts, such as Gilsonite or Trinidad Lake asphalts, are not refined but possess characteristics that make them difficult to use in high percentages in a asphalt concrete mix. These materials generally require the addition of a flux oil to soften the mix to produce a usable asphalt cement.

The potential for the FAA to creep was a primary concern with respect to the use of asphaltic materials. Generalized creep testing of materials consists of the application of a constant uniaxial tensile load at a constant temperature sufficiently elevated to cause creep. However, field conditions at the RFETS will not be consistent with standard uniaxial tensile loading of an asphalt membrane at elevated temperatures. Furthermore, the FAA has significant adhesive properties and also will be under a significant normal load that will confine the FAA. Since asphalt materials generally demonstrate classic creep behavior, a creep testing program was initiated during the IM/IRA accelerated program to determine the influence of the field-applied dead load on the creep characteristics of the FAA. The asphalt concrete is not expected to creep significantly under the 11.3° slope and a constant temperature of approximately 58° F. Research conducted by Hills and McAughtry (1986) indicate that creep deformation should be minimal for the RFETS conditions. However, creep deformation may occur during construction operations during the hot application of the asphalt concrete.

The testing program to study the creep behavior of the FAA over the asphalt concrete was initiated to determine if it could affect the integrity of the overlying cover materials (biotic barrier and topsoil). The testing program was developed to incorporate buttressing effects, cover specific materials, and determine the maximum allowable slope of the cover. The test consisted of three large-scale consolidation/creep test frames configured to evaluate downslope creep under an applied dead load on an infinite slope. All tests were performed in an environmentally controlled chamber that was held at a constant 58°F. Each test station incorporated devices to measure vertical and horizontal deflections. The lower slope frame (sloped at 11.3°) was constructed of roughened concrete with the designated thickness of FAA applied to the upper surface of the concrete. The upper loading platen was a custom angled solid aluminum block with a test surface area incorporating a ¼-inch sand representative of the site cushion sand firmly embedded into the surface. The sustained applied dead load for each test was 480 psf, with a minimum testing duration of 40 days. The FAA was placed over roughened concrete samples to simulate the asphalt concrete. Due to the accelerated schedule of the IM/IRA program, not enough time was available to cure asphalt concrete properly prior to the creep tests.

Three tests were initiated under the stated testing conditions. FAA nominal thicknesses of 87, 118, and 160 mil were tested. The 87-mil sample saw an initial embedment of the upper surface platen into the membrane within a period of 16 hours after which a time the platen did not creep. The sample did not demonstrate any creep characteristics over the 40-day test duration. The 118 mil sample appears to have taken longer to become embedded into the FAA. It took approximately 22 days for the sample to become embedded into the FAA prior to the cessation of creep behavior. The 160-mil sample continued to creep at an unacceptable rate for a duration of over 75 days of testing. The rate of creep was estimated to be at 0.07 inches per year. It was determined that over the 1,000-year design life of the cover, this rate would be excessive. The results of the creep testing indicate that a relationship exists between the thickness of the FAA and possibly the angularity and size of the surface aggregate. Surface roughness of the underlying material (roughened concrete) likely influences the creep behavior of the system. Following the results of the creep tests, it was determined that no creep behavior characteristics would be allowed for the FAA materials. Hence, the maximum allowable thickness for the FAA was established at 118 mil.

Constructability

Clay Soils

The difficulty in constructing a clay cover at the RFETS would not be in the construction equipment's ability to perform, but rather in the contractor's ability to control the conditions to which the clay soil is placed and compacted. As discussed in earlier sections, a clay soil requires the application of significant quantities of water to ensure that permeability requirements are met and that desiccation cracking be kept to a minimum. Furthermore, dust suppression is an important issue at the RFETS. The application of water to clay layers during construction by water trucks is generally a "less than scientific" operation. One of the important factors in modeling the leaching of contaminants from under the cover at RFETS was the insurance that excessive water would not exist in the contaminated zone. The introduction of free water that could infiltrate the waste zone is against the design basis for the construction of the cover at the RFETS. Quality Assurance/Quality Control (QA/QC) operations for the construction of a clay cover must be enforced strictly. Areas of the cover that are out of compliance would require significant operations to recompact or replace the clay materials. The use of large rototillers to blend the clay soil could disturb underlying layers of the cover. Lifts of the clay soils that are not correctly bonded, through scarifying or other methods, may provide channels for water migration.

Asphalt Concrete

The construction of the asphalt concrete cover would not require any construction methods that would introduce excessive moisture into the waste zone. Dust suppression would not be an issue during the placement of the asphalt concrete. The difficulty in constructing the asphalt cover would be in the placement of the high asphalt content hot mixture on the sloping surface (11.3° slope). Shoving and inadequate compaction could result if the operations are not performed properly. Excess shoving could lead to the development of transverse hairline cracks. A test pad is recommended for the asphalt concrete and FAA application to confirm the placement methods and any unique behavior properties that may occur during placement.

Cost

The costs for the construction of a clay or asphalt cover will be dependent upon many variables. The estimated in-place cost for the asphalt and base course materials for the currently designed cover of the IM/IRA is $32 per square yard. The total cost of the asphalt for the 12-acre engineered cover would be $1.8 million. Estimates for the in-place cost of a 2-foot-thick clay cover (only the clay portion) are approximately $8 per square yard depending on the borrow location. The total cost of the clay layer for the 12-acre engineered cover would be $465,000.

DESIGN OF SELECTED MATERIAL

The following section discusses the detailed design of the asphaltic low-permeability layer in the engineered cover. The objectives of the total design for the asphalt concrete and FAA were to design a cover that meets RCRA requirements for the closure of a surface impoundment, has the longevity and durability to last the 1,000-year design life of the cover, and be constructable using common construction equipment and techniques.

Asphalt Composite Design

The composite design of the FAA installed above asphaltic concrete was selected as a fail safe measure. It is believed that the FAA will span the cracks in the asphalt concrete that may occur due to settlement. The FAA also appears to have "self-healing" properties, as demonstrated during the performance of the creep tests with sand embedment. The design specifications called for an asphalt concrete that would meet the following requirements as determined by American Society for Testing and Materials (ASTM) D 1559:

- Air Voids: 4 percent maximum
- Asphalt Content: 6-9 percent
- Marshall Stability: 750 pound minimum
- Design compaction by the Marshall Method is limited to 35 blows on each end of the specimen
- Voids in Mineral Aggregate: 15 percent minimum

The asphalt cement was specified as an AC-10 grade. The aggregate material was specified with 100 percent passing the 5/8-inch sieve. These design specifications are subject to change following the performance of more detailed laboratory tests.

The FAA was specified as Deery Oil Co. Membrane 6 or approved substitute. The tests for the IM/IRA project were performed utilizing this material. Specific required tests for the FAA include: softening point (ASTM D 36), specific gravity (ASTM D 70); penetration (ASTM 3407); viscosity (ASTM D4402), and elongation (ASTM D4885).

Long-Term Testing

Current tests that simulate the chemical and physical property changes in asphalts during hot mix plant operations traditionally have been used as indicators for the longevity of an asphalt. However, the SHRP determined that the extended times and high temperatures of traditional tests cause excessive loss of volatiles and the tests are not a reliable source for predicting the behavior of asphalt aging at lower temperatures.

The behavior of asphalt materials over the life of the cover is one of the few factors that is not predictable. Available data on the performance of asphalts does not provide enough detail to confirm that the properties will remain acceptable. It is suggested that accelerated age-testing followed by physical performance tests be conducted on the asphalt samples under "aged" conditions. Freeman and Romine (1994) provide details of a testing program to provide valuable information on the performance of aged asphalts. The Hanford researchers recommended the following activities be conducted to provide data to support the longevity of an asphalt cover system for a 1,000-year design life cycle:

1. Develop a defensible, accelerated aging test procedure to allow for measurements of asphalt barrier properties as a function of age for a minimum of 1,000 years. Accelerated test procedures would be based on the procedures developed by the SHRP. The procedures would be modified to account for the conditions expected in the subsurface conditions versus highway exposures.

2. Age potential the asphalt materials over the conditions expected in the actual subsurface environment.
3. Measure the changes of the asphalt chemical and physical properties using standard and modified testing procedures.
4. Supplement and validate the laboratory aging data by comparing the chemical properties with several hundred- to several-thousand-year-old asphalt artifacts.
5. Estimate the response and performance of aged asphalt materials to a range of performance tests including settlement and permeability tests.

CONCLUSIONS AND RECOMMENDATIONS

Asphalt materials were chosen over clay soils for use as a low-permeability layer to meet RCRA requirements for the closure of a surface impoundment that will have a 1,000-year design life. It is believed that the asphalt materials will outperform clay soils based on permeability, longevity, and constructability considerations. However, the asphalt materials could cost as much as 3-4 times more than clay. Creep testing should be conducted for the design of an engineered cover utilizing asphaltic materials. Creep testing should be performed on the selected asphalt materials and should be a combination of the asphalt concrete and FAA. The FAA will have to be applied at a range of 60-118 mil at the OU4 SEPs, based on the specific side slope and loading conditions. The 60-mil lower-thickness limit was developed from the acceptable permeability test (below 1×10^{-11} cm/s), while loaded at 480 psf. The upper-thickness limit of 118 mil was developed from the performance of the creep tests.

Age testing is necessary in order for asphalt materials to become widely accepted throughout the engineering industry for use as long-term low-permeability layers in engineered covers. The SHRP found that aging kinetics are highly dependent on how temperature will effect the molecular structure of an asphalt. Aging tests need to be specific to the materials and environmental conditions expected within engineered covers. It is important to determine the mechanisms for the aging of asphalts and the subsequent physical and chemical changes in the materials. Realistic tests could determine if premature age hardening may lead to a breakdown in the performance of asphaltic materials.

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ALTERNATIVES FOR GROUND WATER CLEANUP

National Research Council Committee on Ground Water Cleanup Alternatives, Presented by Lynne Preslo

EXECUTIVE SUMMARY

The United States currently faces a very large ground water contamination problem. Although the total number of contaminated sites is unknown, estimates of the total number of waste sites where ground water and soil may be contaminated range from approximately 300,000 to 400,000. Recent estimates of the total cost of cleaning up these sites over the next 30 years have ranged as high as $1 trillion.

Several recent studies have raised troubling questions about whether existing technologies are capable of solving this large and costly problem. These studies focused on "pump-and-treat" systems, which involve installing wells at strategic locations to pump contaminated ground water to the surface for treatment. Pump-and-treat systems are the most common technology for ground water cleanup in the United States. The studies indicated that pump-and-treat systems may be unable to remove enough contamination to restore the ground water to drinking water standards, or that removal may require a very long time, in some cases centuries.

As a result of these studies, there is almost universal concern among groups with diverse interests in ground water contamination-from government agencies overseeing contaminated sites to industries responsible for the cleanups, environmental groups representing affected citizens, and research scientists-that the nation might be wasting large amounts of money on ineffective remediation efforts. At the same time, many of these groups are concerned that the health of current or future generations may be at risk if contaminated ground water cannot be cleaned up to make it safe for drinking. To address these concerns, the National Research Council initiated a study of ground water cleanup systems. The goals of the study were to review the performance of existing pump-and-treat systems, to determine the performance capabilities of innovative cleanup technologies, to assess whether there are scientific and technological limits to restoring contaminated ground water, to consider the public health and economic consequences of contaminated ground water, and to provide advice on whether changes in national ground water policy are needed to reflect the limits of current technology. This report presents the findings of the National Research Council's study.

The study was carried out by the Committee on Ground Water Cleanup Alternatives, appointed by the National Research Council to work under its Water Science and Technology Board and Board on Radioactive Waste Management. The committee consisted of recognized leaders in the fields of environmental engineering, hydrogeology, chemistry, epidemiology, environmental economics, and environmental law and policy. The findings of this report are based on the committee's review of original data from case studies, reports in scientific journals, presentations by experts outside the committee, evaluation of policy documents, and the extensive experience of committee members.

COMPLEXITY OF THE CONTAMINATED SUBSURFACE

Theoretically, restoration of contaminated ground water to drinking water standards is possible. However, cleanup of contaminated ground water is inherently complex and will require large expenditures and long time periods, in some cases centuries. The key technical reasons for the difficulty of cleanup include the following:

- Physical heterogeneity: The subsurface environment is highly variable in its composition. Very often, a subsurface formation is composed of layers of materials with vastly different properties, such as sand and gravel over rock, and even within a layer the composition may vary over distances as small as a few centimeters. Because fluids can move only through the pore spaces between the grains of sand and gravel or through fractures in solid rock and because these openings are distributed nonuniformly, underground contaminant migration pathways are often extremely difficult to predict.
- Presence of nonaqueous-phase liquids (NAPLs): Many common contaminants are liquids that, like oil, do not dissolve readily in water. Such liquids are known as NAPLs, of which there are two classes: light NAPLs (LNAPLs), such as gasoline, are less dense than water; dense NAPLs (DNAPLs), such as the common solvent trichloroethylene, are more dense than water. As a NAPL moves through the subsurface, a portion of the liquid will become trapped as small immobile globules, which cannot be removed by pumping but can dissolve in and contaminate the passing ground water. Removing DNAPLs is further complicated by their tendency, due to their high-density, to migrate deep underground, where they are difficult to detect and where they may remain in pools that slowly dissolve in and contaminate the ground water.
- Migration of contaminants to inaccessible regions: Contaminants may migrate by molecular diffusion to regions inaccessible to the flowing ground water. Such regions may be microscopic (for example, small pores within aggregated materials) or macroscopic (for example, clay layers). Once present within these regions, the contaminants can serve as long-term sources of pollution as they slowly diffuse back into the cleaner ground water.
- Sorption of contaminants to subsurface materials: Many common contaminants have a tendency to adhere to solid materials in the subsurface. These contaminants can remain underground for long periods of time and then be released when the contaminant concentration in the ground water decreases.
- Difficulties in characterizing the subsurface: The subsurface cannot be viewed in its entirety, but is usually observed only through a finite number of drilled holes. Because of the highly heterogeneous nature of subsurface properties and the spatial variability of contaminant concentrations, observations from sampling points cannot be easily extrapolated, and thus knowledge of subsurface characteristics is inevitably incomplete.

Regardless of the remediation technology chosen, these inherent complexities pose major obstacles to ground water cleanup.

PERFORMANCE OF CONVENTIONAL PUMP-AND-TREAT SYSTEMS

The committee found that at the majority of contaminated sites, the complex properties of the subsurface environment and the complex behavior of contaminants in the subsurface interfere with the ability of conventional pump-and-treat systems to achieve drinking water standards for contaminated ground water. The committee reviewed information from 77 sites where conventional pump-and-treat systems are operating. At 69 of the sites, cleanup goals have not yet been reached, although it is possible that they will be reached at some of these sites in the future. The apparent success of remediation at the remaining eight sites suggests that in special circumstances, cleanup in a relatively short time period (less than a decade) may be possible.

Capabilities of Pump-and-Treat Systems

The performance of pump-and-treat systems depends directly on site conditions and contaminant chemistry. As the complexity of the site increases, the likelihood that the pump-and-treat system will meet drinking water standards decreases. Table ES-1, developed by the committee and taken from Chapter 3 of this report, shows the relative ease of ground water cleanup as a function of contaminant chemistry and subsurface hydrogeology. The committee categorized the 77 sites listed in Appendix A according to the rating system shown in this table. The conditions categorized as 1 represent those that will be easiest to remediate, while those categorized as 4 will pose the greatest technical challenge, as shown by the committee's review of the 77 sites:

- Cleanup of sites in category 1: At sites with conditions categorized as 1 according to the table, well-designed pump-and-treat systems generally should be able to restore the ground water to drinking water standards. Such ideal site conditions are rare in the group shown in Appendix A. For example, of the 77 sites listed, only two are categorized as 1; the pump-and treat system reached cleanup goals at one of these sites, a service station where gasoline leaked.
- Cleanup of sites in category 2: Cleanup of sites in category 2 to drinking water standards is also possible but is subject to greater uncertainties than at sites in category 1. For example, 14 of the sites in Appendix A are in category 2, but cleanup goals have yet to be achieved at 10 of these sites, although it is conceivable that goals will be reached in the future.
- Cleanup of sites in category 3: Cleanup of sites in category 3 to drinking water standards is possible but is subject to significant uncertainties; partial cleanup may be a more realistic scenario for many such sites. For example, of the 29 sites in Appendix A in category 3, cleanup goals have been achieved at only three. All three sites were contaminated with gasoline, which biodegrades relatively rapidly, a characteristic that may have accelerated cleanup.
- Cleanup of sites in category 4: Cleanup of sites in category 4 to drinking water standards is unlikely. However, containing the contamination is likely to be possible at such sites. Cleanup goals have not been achieved at any of the 42 sites categorized as 4 in Appendix A.

TABLE ES-1 Relative ease of cleaning up of contaminated ground water as a function of hydrogeologic conditions and contaminant chemistry

	Contaminant Chemistry					
Hydrogeology	Mobile, Dissolved (degrades/ volatilizes	Mobile, Dissolved	Strongly Sorbed, Dissolved (degrades/ volatilizes	Strongly Sorbed, Dissolved	Separate Phase LNAPL	Separate Phase DNAL
Homogeneous, single layer	1[a]	1-2	2	2-3	2-3	3
Homogeneous Multiple layers	1	1-2	2	2-3	2-3	3
Heterogeneous, single layer	2	2	3	3	3	4
Heterogeneous, multiple layers	2	2	3	3	3	4
Fractured	3	3	3	3	4	4

[a] Relative ease of cleanup, where 1 is easiest and 4 is most difficult.

Table ES-1 provides a useful framework for comparing the relative effectiveness of pump-and-treat systems for cleaning up sites with different hydrogeologic and contaminant characteristics. However, it is important to realize that the categories in the table are based on the experience of committee members and a review of preexisting data for sites shown in Appendix A, not on new quantitative analyses. Even more important, the feasibility of cleanup may vary across the site. A single site may contain some regions where difficult-to-extract contaminants remain and continue to dissolve into the ground water and other regions where chemicals are primarily dissolved and no significant long-term contaminant sources are present. The part of the site containing primarily dissolved contaminants might fit category 1 or 2 according to Table ES1, while the part of the site containing entrapped sources of contamination might fit category 3 or 4. Finally, when using a framework such as Table ES-1, it is important to realize that to some extent the feasibility of ground water cleanup depends on the cleanup goals. Returning the ground water to drinking water standards may not be possible at many sites. However, reaching less stringent goals-such as cleaning up areas containing dissolved contaminants and installing containment systems around areas with undissolved contaminants that cannot be-be possible at most sites.

Cleanup Times for Pump-and-Treat Systems

Remediation by pump-and-treat systems is a slow process. Simple calculations for a variety of typical situations show that predicted cleanup times range from a few years to tens, hundreds, and even thousands of years. Some have advocated that ground water cleanup should be considered technically impracticable when the cleanup time is very long. Given the complex policy implications of this issue, the committee defers to the expert agencies in deciding what, if any, limits to set on cleanup time. However, the committee believes that it is important for regulators to recognize that to some extent, cleanup time can be influenced by system design. A system pumping at very low rates may have a very long predicted cleanup time, while one pumping at higher rates may have a shorter predicted cleanup time. In considering the issue of cleanup time, regulators must also be aware that estimating the cleanup time is difficult and is subject to a large number of uncertainties; typical methods used to calculate cleanup time often result in underestimates because they neglect processes that can add years, decades, or even centuries to the cleanup.

CAPABILITIES OF ENHANCED PUMP-AND-TREAT AND ALTERNATIVE TECHNOLOGIES

Numerous innovative technologies exist that have the potential to improve significantly the efficiency of ground water cleanups, especially when technologies suited to specific types of contaminants or specific hydrogeologic environments are combined. While no known technology can ensure the achievement of health-based cleanup goals at complex sites, these innovations nevertheless have the potential to increase the effectiveness and reduce the costs of ground water cleanup. Some innovative technologies-including soil vapor extraction, air sparging, and in situ bioremediation of petroleum products-are already being implemented. However, the use of innovative cleanup methods has been limited by technical, institutional, and economic barriers. As a result, conventional pump-and-treat systems are used at approximately three-quarters of sites with contaminated ground water.

For this report, the committee divided innovative technologies into two categories: enhanced pump-and-treat systems, which require the pumping of fluids, and alternative technologies, which do not require pumping.

Enhanced Pump-and-Treat Systems

Conventional pump-and-treat systems pump relatively large volumes of water with relatively low contaminant concentrations. Because of the slow rates of contaminant desorption and dissolution, these systems must displace many volumes of aquifer water to flush out contaminants. Conventional pump-and-treat systems are therefore an inherently inefficient method for removing contaminants, even if they are effective in some cases. The enhanced pump-and-treat systems listed in Table ES-2 improve the efficiency of contaminant removal and lessen pumping requirements under certain conditions. These technologies can enhance contaminant removal and destruction compared to conventional systems, but each requires pumping fluids (water, air, or water solutions) through the subsurface and will therefore have some of the same limitations as conventional pump-and-treat systems.

TABLE ES-2 Enhanced Pump-and-Treat Systems

Technology	Description	Application	Limitations
Demonstrated technologies[a]			
Soil vapor extraction	Flushes air through soil above the water table to extract volatile contaminants	Extracts volatile contaminants above the water table	Difficulty flushing zones of low-permeability and removing contaminants bound to soil
In situ bioremediation— hydrocarbons	Pumps materials through the subsurface to stimulate growth of organisms that biodegrade contaminants	Removes petroleum products and derivatives above and/or below the water table	Difficulty delivering growth-stimulating materials to zones of lowpermeability; slowed by presence of NAPLs; difficulty delivering adequate oxygen to the organisms
Bioventing	Pumps air through the soil to stimulate growth of organisms that biodegrade contaminants	Removes petroleum products and derivatives above the water table	Similar t those for soil vapor extraction; also, adding nutrients in aqueous solution may inhibit air movement and affect soil's load-bearing capacity
Developing technologies[b]			
Pulsed or variable pumping	Varies the pumping rate to allow contaminants to dissolve, desorb, and/or diffuse from stagnant regions	May improve removal efficiency for sites with NAPLs and other residual contaminants	Increases cleanup time because of reduced pumping rate; other limitations similar to those of conventional pump-an-treat-systems
In situ bioremediation— chlorinated solvents	Pumps materials through the subsurface to stimulate growth of organisms that biodegrade contaminants	Removes chlorinated solvents above and/or below the water table	Similar to those for in situ bioremediation of petroleum hydrocarbons; also, possible accumulation of hazardous intermediate compounds
In situ bioremediation—	Pumps materials through the subsurface to stimulate growth of	Dissolves metals to facilitate extraction or immobilizes them to	Similar to those for the other forms of in situ bioremediation

Technology	Description	Application	Limitations
metals	organisms that change the chemical form of metals	prevent spreading	
Air sparging	Injects air below the water table and captures it above the water table to extract volatile contaminants and promote bioremediation	Extracts volatile contaminants; promotes bioremediation	Difficulty flushing low-permeability zones; difficulty operating at depths below approximately 9 meters (30 ft); difficulty extracting multicomponent mixtures
Steam-enhanced extraction	Injects steam above and/or below the water table to volatilize contaminants	Volatilizes contaminants	Difficulty volatilizing contaminants from low-permeability zones
In situ thermal desorption	Adds heat above the water table by Joule heating, radio frequency heating, or other means to improve removal of contaminants with low volatility	Improves removal of contaminants with low volatility above the water table	Similar to those of conventional pump-and-treat systems; possibility that chemical additives may adversely affect the subsurface
Soil flushing	Flushes surfactants or cosolvents below the water table to enhance recovery of contaminants with low water solubility	Enhances recovery of contaminants with low water solubility	Difficulty delivering chemical to low-permeability zones; possibility that chemical reactions may adversely affect the subsurface; slow reaction rates

[a] For demonstrated technologies, a substantial amount of performance data exists.
[b] Developing technologies require more controlled field studies and large-scale site trials to generate reliable performance data.

Alternative Technologies

Conventional pump-and-treat systems and the enhancements listed in Table ES-2 require a continuous energy input for pumping water or air. The alternative approaches listed in Table ES-3 do not require a continuous energy input and therefore may be less costly. These methods show promise, but they are in the development stage, and their long-term effectiveness has not yet been demonstrated. In addition, some of these methods contain, rather than cleanup, the contamination, and the methods that do result in cleanup may be much slower than the more energy-intensive approaches.

Barriers to Implementing Enhancements and Alternatives

A variety of barriers have discouraged those involved in ground water cleanup from assuming the risks associated with using innovative technologies that lack proven track records. The most significant barriers include the following:

- allocation of liability if a technology fails;
- inability to raise sufficient capital for successful commercialization;
- lack of vendors for some innovations;
- federal regulations specifying that any contractor involved in the selection or testing of a technology is ineligible for construction;
- lack of testing facilities;
- lack of cost and efficiency information;
- lack of adequate technical expertise among consultants and regulators; and
- the requirement to construct a pump-and-treat system if the innovative technology fails to achieve cleanup goals.

While the Environmental Protection Agency (EPA), the Department of Energy, the Department of Defense, and others are implementing programs to remove these barriers, the cumulative effectiveness of these efforts is unknown. Mutual risk sharing between the government and private parties would encourage greater use of innovative technologies.

CHARACTERIZING SITES FOR GROUND WATER CLEANUP

The inability of pump-and-treat systems to reach drinking water standards at many sites to date is not just a function of site complexity and technical limitations; it is also a result of insufficient or inaccurate characterization of the problem prior to cleanup. At several sites the committee reviewed, the cleanup systems failed to contain the contamination (much less clean it up) because of poor characterization of the extent of contamination and the locations of contaminant sources. The lack of adequate characterization has often occurred even after huge sums have been spent and considerable time has elapsed in characterizing the site. Thus, whether the technology is conventional or innovative, the design of a strategy for characterizing the site is as important as the design of the cleanup system itself. In brief, site characterization studies must provide the following information:

- the extent of ground water contamination, both horizontal and vertical;
- approximate locations of long-term sources of contamination, including sources near the surface where the contamination originated and sources that developed along the path of contaminant migration (such as residual NAPLs, pools of NAPLs, and metal precipitates);
- characteristics of the hydrogeologic setting important to the design of the remediation system and to the prediction of contaminant migration; and
- data to estimate the site's restoration potential using a method such as that represented in Table ES-1.

In characterizing a site with contaminated ground water, it is important to realize that due to the complexity of the subsurface and the difficulty of observing it, perfection in site characterization is unachievable. The performance of the remediation system itself will provide additional, extremely valuable information on site characteristics that may not be possible to obtain in any other way. Data collection should continue throughout the life of the ground water cleanup system, and these data should be analyzed regularly to determine whether they are consistent with the current understanding of the site and, if not, whether changes in the remediation plan are necessary.

SETTING GOALS FOR GROUND WATER CLEANUP

This report documents that the ability of technology to restore contaminated ground water to drinking water standards is uncertain at many sites. Nevertheless, regulations under the Comprehensive Environmental Response, Compensation, and Liability Act (the Superfund law), the Resource Conservation and Recovery Act (RCRA), and similar state laws require that the water be cleaned up, usually to drinking water standards. The use of drinking water standards as cleanup goals has been questioned by many in the regulated community and others. Critics have long contended that options such as containing the contamination can protect public health, as long as the water in the containment zone is either restricted for use or treated with appropriate technology prior to use. The criticism of using drinking water standards as cleanup goals has increased because of the technical evidence that reaching these standards may not be possible in reasonable time frames at many sites. On the other hand, some people contend that drinking water standards-or stricter requirements-should be maintained as cleanup goals regardless of the capabilities of technology for two reasons: to provide an incentive against further pollution and to encourage development of improved cleanup technologies.

In the debate over ground water cleanup goals, many alternative cleanup goals have been suggested. In broad terms, these alternatives are the following:

- *complete restoration,* or removal of all traces of contamination;
- *nondegradation,* or removal of contamination to natural background levels or to detection limits;
- *health-based standards,* such as the drinking water standards used as cleanup goals at most sites today;
- *technology-based standards,* which would require cleanup to the capabilities of the best available technology;
- *partially restricted use standards,* meaning cleanup to allow nonpotable uses such as irrigation; and

- containment, meaning that contamination remains in-place but systems are installed to prevent contaminant migration off site and, if necessary, to treat the ground water at the point of use.

Each of these options reduces the risk of deleterious impacts due to ground water contamination. However, the magnitude of this risk reduction and the associated economic benefits are difficult-if not impossible-to quantify. The professional community does not agree on the magnitude of health impacts of ground water contamination from hazardous waste sites for many reasons, the most important of which are difficulties in determining the extent to which humans have been and will be exposed to contamination, limitations in extrapolating toxicological effects observed in animal studies to human populations, and uncertainties in the science of epidemiology. Likewise, the total economic value of restoring contaminated ground water is unknown. Thus, a high degree of uncertainty exists, making quantitative assessment of the risks and benefits of various ground water cleanup goals extremely difficult.

Like society as a whole, the committee had diverse views about which of the various alternative cleanup goals is most appropriate and whether the current approach of requiring cleanup to drinking water standards at a large number of sites should be changed. However, the committee strongly believes that because existing ground water cleanup goals cannot be attained in reasonable time frames (decades) at a large number of sites with current technologies, regulators should set short-term objectives for these sites based on the capabilities of current technology. While the long-term goals need not necessarily change, interim objectives are needed to acknowledge current technological limitations. In the recommendations below, the committee outlines a scenario for dividing contaminated sites into three categories, some of which would require interim objectives and some of which would not.

CONCLUSIONS AND POLICY RECOMMENDATIONS

In summary, the committee found that at many sites requiring ground water cleanup, some areas will remain contaminated above drinking water standards for the foreseeable future even when the best available technologies are used. However, the committee also found that cleaning up large portions of these sites is possible, even if limited areas remain contaminated. In addition, a wide range of developing technologies has the potential to improve the effectiveness of ground water remediation. Nevertheless, there are limits to what technology can accomplish, and existing regulatory requirements for ground water cleanup do not adequately account for these limits. The following recommendations provide guidance for modifying policies to reflect the key technical conclusions of this report.

Complexity of the Subsurface

***Conclusion.* Subsurface environments and many common contaminants have properties that interfere with decontamination efforts-regardless of the technology chosen.** These properties make finding the contaminant sources difficult, increase contaminant spreading, and cause contaminants to accumulate in zones from which they are difficult-to-extract. The complex interactions occurring in the subsurface are not fully understood, and therefore the effect of subsurface and contaminant properties on the ability to cleanup ground water is often difficult to quantify.

***Recommendation 1.* The committee recommends that the EPA systematically evaluate its experience in cleaning up sites to improve understanding of factors that prevent achievement of health-based ground water cleanup goals**. The committee suggests that the EPA undertake an annual review of selected pump-and-treat systems based on the experience of EPA project managers throughout the United States. The analysis would be similar to a study of pump-and-treat systems at 24 sites that the EPA conducted in 1992 but would incorporate some of the improvements in analysis suggested in this report (for example, evaluating the number of pore volumes extracted per year).

***Recommendation 2.* The committee recommends that the EPA establish a standardized, centralized, broadly accessible repository for site information**. Currently, accessing the large amount of existing site data from completed and ongoing ground water remediation projects is extremely difficult. To increase the accessibility of data, the EPA could develop suggested formats for collection and analysis of site-specific information. The EPA could also establish an easily used, publicly accessible data base for sites where ground water cleanup is under way.

Performance of Conventional Pump-and-Treat Systems

***Conclusion.* The ability of conventional pump-and-treat systems to reach health-based cleanup goals for contaminated ground water is highly site-specific**. Although cleanup is possible at some sites, properties of the subsurface and the contaminants may make restoring contaminated ground water to drinking water standards technically infeasible with current technology in reasonable time frames (decades) at a large number of sites.

***Conclusion.* Although restoring the total volume of contaminated ground water to health-based standards may not be feasible at many sites, properly designed pump-and-treat systems still provide important benefits, including containment of the contamination, retraction of the plume of dissolved contaminants, and removal of some contaminant mass from the subsurface**. Most sites with contaminated ground water contain two types of problem areas: (1) source areas and (2) dissolved plume areas. Conventional pump-and-treat systems may be effective for cleaning up plumes of dissolved contamination. However, this technology alone will be ineffective for restoring source areas such as those with significant amounts of residual NAPLs, pools of NAPLs, or metals that have precipitated.

***Recommendation 1.* The committee recommends that the EPA's policy for determining whether ground water cleanup is feasible provide for the categorization of contaminated**

sites into three groupings corresponding to the complexity of the site. At one extreme is a group of sites generally represented by category 1 in Table ES-1; cleaning up sites in this group to meet health-based goals should be possible with current technology. At the other extreme is a group of sites generally represented by category 4 in Table ES-1; current technology is highly unlikely to restore sites in this group to health-based standards in reasonable time frames (decades), and therefore these sites may warrant permanent infeasibility waivers with the concomitant selection of a new protective long-term goal. In the middle is a group of sites generally represented by categories 2 and 3 in Table ES-1; for sites in this group, attaining health-based ground water cleanup goals will be difficult or unlikely with current technology but not necessarily impossible over the longterm as technology improves. The long-term cleanup goals for sites in this middle group should be temporarily superseded by interim objectives reflecting the capabilities of existing technologies. (The correlation of the three groupings with the categories of Table ES-1 is only approximate.)

Recommendation 2. **The committee recommends that the EPA assess and develop guidance on institutional strategies for preventing public exposure to contamination over the long-term at sites where reaching health-based cleanup goals is infeasible with the best available technologies**. An institutional structure capable of lasting for several generations will be needed to oversee the large number of sites at which complete cleanup is infeasible with current technologies.

Recommendation 3. **The committee recommends that the EPA and other agencies identify and eliminate disincentives to early implementation of ground water remedial actions.** Ground water cleanup is more likely to be effective if initiated early. Allowing responsible parties to commit to only one-phase of cleanup at a time instead of requiring them to agree to the entire remedy all at once might provide an incentive for early cleanup; the EPA should pilot test this concept to determine whether it results in faster cleanups or whether it slows the process because of the additional negotiations it would require.

Capabilities of Innovative Technologies

Conclusion. **Although innovative technologies for ground water cleanup are subject to many of the same limitations as conventional pump-and-treat systems, many of these technologies can improve the efficiency of ground water cleanup efforts**. However, important technical, economic, and institutional barriers have slowed their development.

Recommendation. **The committee recommends that Congress investigate the possibility of charging an annual "infeasibility fee" to public and private responsible parties at sites where attaining health-based standards is not presently feasible**. Congress could investigate various options for appropriating the funds collected from this fee. The committee sees two options as having special merit. One possibility is to use part of the funds to create an applied ground water research fund to pay for a strong research program for improved ground water cleanup techniques. The other possibility is to use some of the funds to encourage use of innovative cleanup technologies by reimbursing responsible parties for testing these technologies in certain circumstances. Under this scheme, an expert panel would approve use of an innovative technology. In the event that the innovative technology fails to achieve its intended goal and the responsible party is required to construct a backup technology, the responsible party would be able to recoup some or all of its losses from the infeasibility fee fund. If the innovative technology worked, the fund would not subsidize the project. Initially the fund might apply only to Superfund sites, but if successful it might be extended to other types of sites.

Characterizing Sites

***Conclusion.* Optimization of the site characterization and management process could improve the effectiveness of ground water cleanups**. The poor performance of ground water cleanup systems is not solely a function of site complexity and technical limitations; it can also result from insufficient or inaccurate characterization of the problem prior to cleanup, leading to flawed design of the cleanup system.

***Recommendation 1.* The committee recommends establishment of expert panels to evaluate site characterization, remedy selection, and remedy performance at complex sites**. As discussed in this report, a large fraction of contaminated sites fit category 2 or 3 in Table ES-1, and thus design of cleanup systems for many sites will be subject to considerable uncertainties. At present, federal and state regulatory agencies have an insufficient number of technically trained staff members to address the multitude of complex sites. While not a substitute for hiring and retaining technically trained staff, expert panels could provide guidance in addressing the often difficult technical choices at these sites. The panels could also evaluate proposals for using innovative technologies that would be covered under the infeasiblity fee fund discussed above. The panels could be funded by the infeasibility fee and/or by charging those responsible for cleanup at sites where the panels provide advice. The EPA should assess the feasibility of such an expert panel approach to resolving problems at complex sites.

***Recommendation 2.* The committee recommends that the EPA prepare new guidance documents that will lead to improved optimization of the hazardous waste site characterization process and explicitly address factors that will determine whether health-based cleanup goals are practicable**. The EPA should revise existing site characterization guidance for the Superfund and RCRA programs to link the collection of specific characterization information with early action implementation steps. New guidance documents are needed to ensure that factors that may limit the ability to achieve health-based ground water cleanup goals are recognized as early as possible.

Setting Cleanup Goals

***Conclusion.* Existing procedures for setting ground water cleanup goals do not adequately account for the diversity of contaminated sites and the technical complexity of ground water cleanup**. Whether goals established under existing procedures adequately protect public health and the environment, or whether they are overprotective or underprotective, is uncertain, as are the costs to society when these goals cannot be achieved.

***Recommendation 1.* Although the committee recognizes that different agencies must operate under different authorities, all regulatory agencies should recognize that ground water**

restoration to health-based goals is impracticable with existing technologies at a large number of sites. The complexities and limitations that this report describes are functions of the nature of the contaminants and the hydrogeology of the site, not of the identity of the agency or private party attempting to address the problem or the statutory authority or regulatory agencies involved. The EPA and other regulatory agencies should establish consistent mechanisms for deciding the restoration potential of contaminated sites, as indicated by the approach outlined in this report.

Recommendation 2. **The committee recommends that the EPA expand its efforts to inform the public about limitations of existing technologies and capabilities of innovative technologies**. From the perspective of the affected public, the Superfund program has had limited success in responding to community concerns at many sites. Although the ground water cleanup problem is technically complex, the implications of site complexities as well as the promise that innovative technologies hold to improve cleanup should be explained to the affected public. The committee recommends that the EPA include expanded efforts at community relations within the technical impracticability waiver process and revise its community relations guidance documents to include issues of technical impracticability.

CONSTRUCTION OF DEEP BARRIER WALLS FOR WASTE CONTAINMENT

M. Mauro, Rodio, Inc., Boston, Mass.

ABSTRACT

This article reviews the methods that currently are available within slurry wall technology for the construction of deep barrier walls for waste containment. The excavation of cutoff walls using self-hardening slurry is presented and indications of the achievable wall properties are given. The construction of composite walls, with the insertion of high-density polyethylene (HDPE) membrane into the cutoff wall, is described. Lastly, the Hydromill technique, the latest development in slurry wall excavation, is introduced for the construction of cutoff walls with plastic concrete.

INTRODUCTION

The isolation of polluted areas by vertical cutoff walls is one of the current methods used to limit the contamination in the surrounding environment. In order to encapsulate a polluted area (Figure 1), the vertical cutoff wall is generally keyed in a soil layer of lowpermeability of appropriate thickness and continuity. Several technologies currently are available to construct a vertical cutoff wall: slurry walls, jet grouting, soil-mixing, or grout curtains. The selection of the appropriate technology is made considering the geology of the site, the type of contaminant to be intercepted, and the degree of water tightness to be achieved.

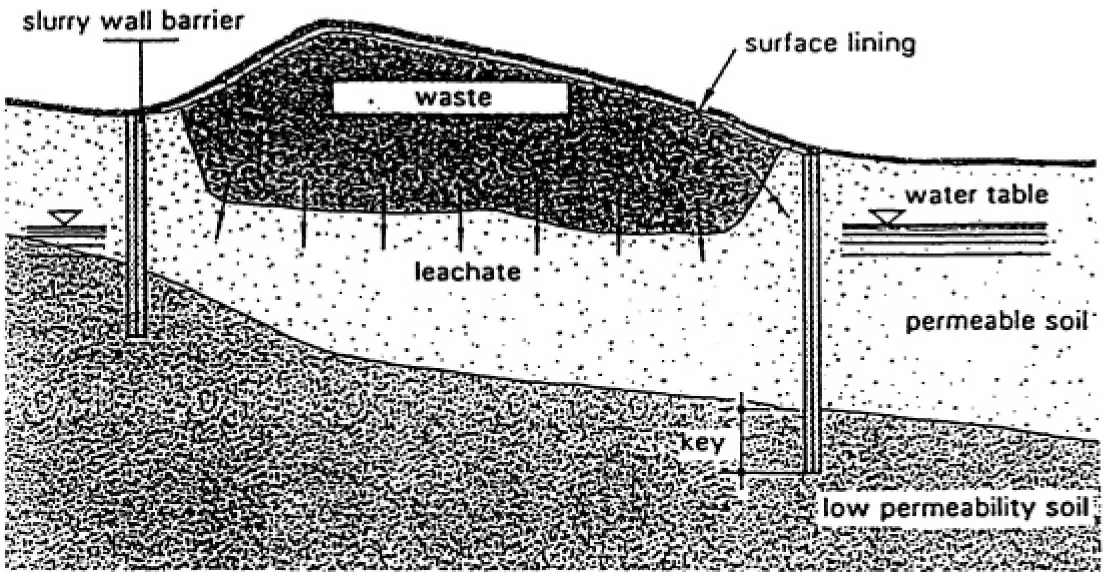

FIGURE 1 Containment of a polluted area.

If the cutoff wall is to reach a considerable depth, typically over 50 feet, slurry walls are one of the technologies most frequently employed. Excavation equipment capable of reaching considerable depth and assuring the continuity of the cutoff walls is available. Significant progress has been made in the design of the mixes used to replace the in-situ soil. In this paper, the following three types of deep cutoff wall constructed with the slurry wall technology are discussed briefly:

- cutoff walls constructed with self-hardening slurry,
- composite cutoff walls, and
- cutoff walls constructed with plastic concrete.

Cutoff Walls Constructed with Self-Hardening Slurry

The construction of cutoff walls with self-hardening slurry takes place with an alternating panel excavation sequence (Figure 2). The in-situ soil is replaced by the self-hardening slurry, a homogeneous mix of water, cement, and bentonite designed to create a very low-permeability barrier. The hydraulic conductivity of self-hardening slurry is typically in the range of 10^{-6} cm/s and can be reduced further with the use of additives. The excavation generally is carried out with mechanical or hydraulic grabs. Mechanical grabs are operated and suspended by cables. Hydraulic grabs are opened and closed by pistons driven by a power pack mounted on the supporting crane. The hydraulic grab can be cable suspended (Figure 3) or connected to a rigid Kelly bar (Figure 4).

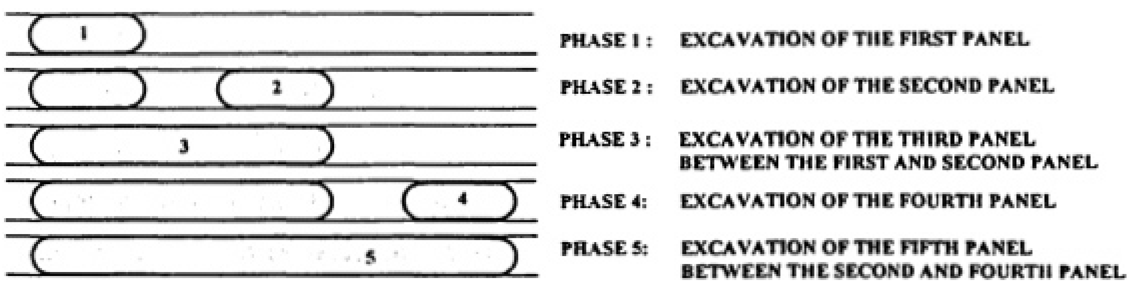

FIGURE 2 Typical excavation sequence of a cutoff wall constructed with self-hardening slurry.

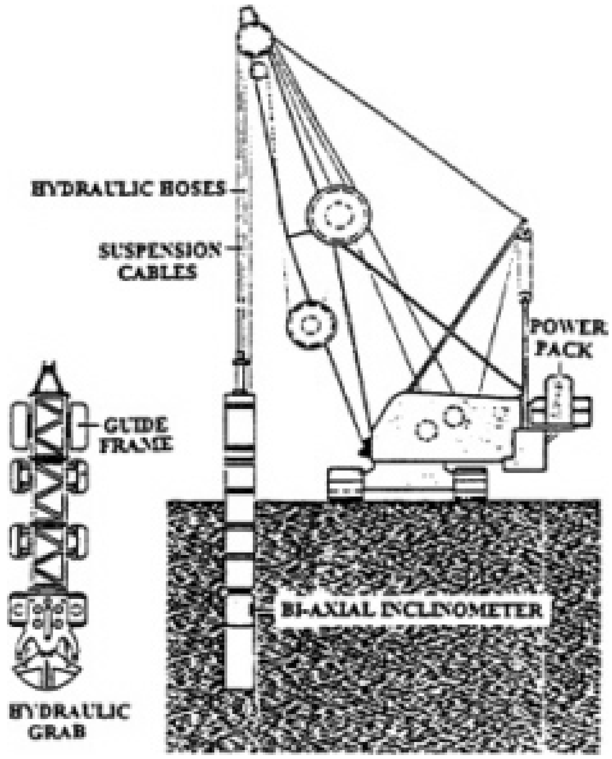

FIGURE 3 Cable-suspended hydraulic grab.

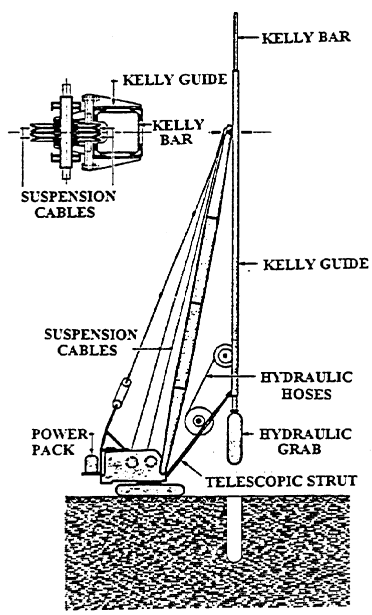

FIGURE 4 Kelly-mounted hydraulic grab.

In the case of cable-suspended grabs, great operator ability is required to maintain the verticality of the excavation. In the case of hydraulic grabs rigidly connected to a Kelly bar, the control of the verticality is in general easier because of the stiffness of the frame guiding the Kelly bar and the large weight of the Kelly bar itself. A considerable advantage of hydraulic over mechanical grabs is that the efficiency and speed of excavation are improved largely.

The continuity of the cutoff wall can be assessed measuring the verticality of each panel during the excavation and checking the overlap between adjacent panels. For the measurement of verticality, an inverted pendulum system or an inclinometer connected to the grab are commonly employed. Readings typically are taken at constant intervals during the excavation. In case of deviation, the modification of the cutting edges of the grab or the use of reamers are usual corrective measures. A deviation from verticality of less than 1.0-2.0 percent can be achieved realistically.

In order to reduce the hydraulic conductivity of the cutoff wall, the choice of the self-hardening slurry composition is very important. The following factors have to be considered:

- The choice of the cement type. In general, better performance, in terms of water tightness and chemical resistance, is obtained using blast furnace or pozzolan cement instead of Portland cement (Figure 5).
- The bentonite content. High bentonite content may be useful in reducing the hydraulic conductivity of the self-hardening slurry and assuring a better uniformity of the cement content in the panel.
- Use of additives. The use of special additives, such as dispersing agents, reduces the viscosity, allowing the use of mixes with higher bentonite or cement content (Figure 5). The development of the hydrosilicate crystals during the hydration of the cement is also improved, with a consequent reduction in the hydraulic conductivity of the wall.
- Several cutoff walls up to 200 feet in depth have been constructed successfully by Rodio using the self-hardening slurry technique.

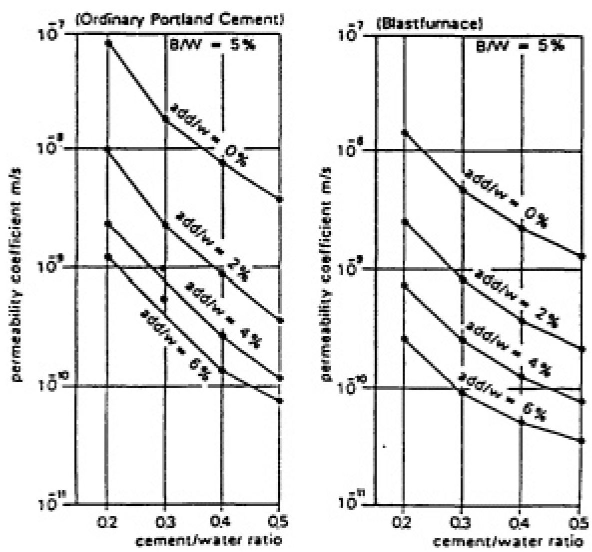

FIGURE 5 Self-hardening slurry permeability coefficient as a function of the type of cement, cement/water and additive/water ratios (De Paoli et al., 1993).

Composite Cutoff Walls

In order to improve the resistance of the cutoff wall to chemical attacks coming from a variety of different agents, or to further reduce the hydraulic conductivity of the barrier, it is often necessary to install a composite cutoff wall. This type of wall is constructed with the insertion of an impervious artificial barrier, such as an HDPE membrane, in a wall excavated with self-hardening slurry (Figure 6). A practically impervious barrier can be created.

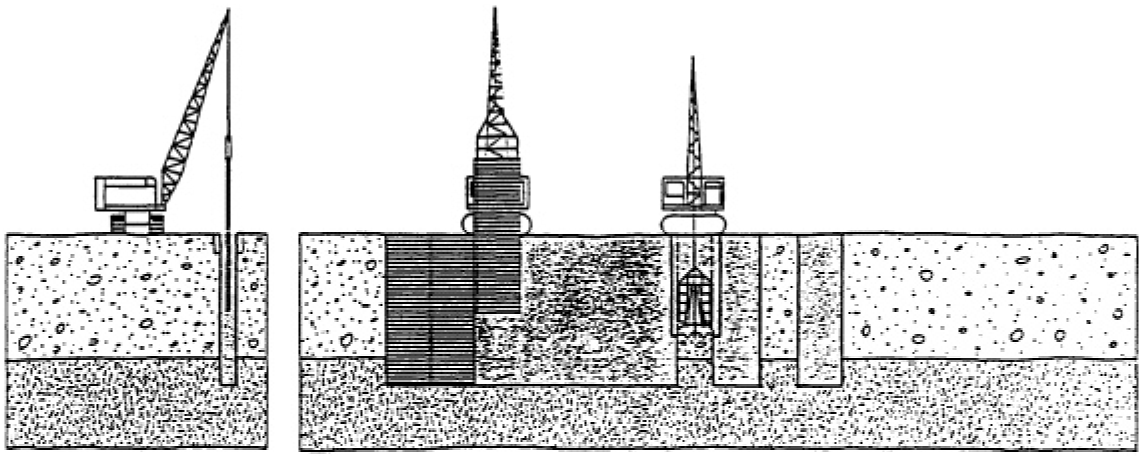

FIGURE 6. Installation of HDPE membrane in a cutoff wall excavated with self-hardening slurry.

Typically, the HDPE is lowered in the trench in sheets from 7- to 25-feet wide. Since the density of the membrane is lower than the density of the self-hardening slurry, a special heavyweight guide frame is used to ensure a proper installation. The frame is withdrawn after the installation of the HDPE sheet in the wall.

Continuity between the HDPE sheets is guaranteed through special joints (Figure 7). The type of joint and the method used to splice the HDPE sheets in the field must be selected carefully.

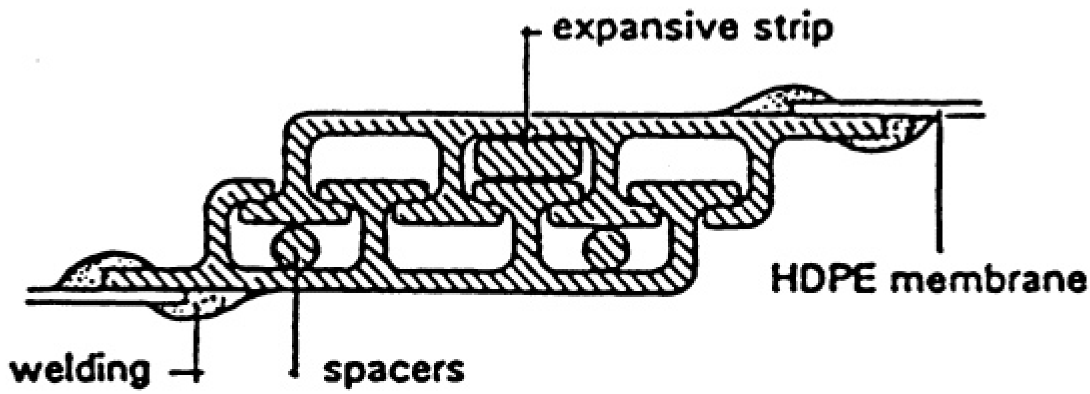

FIGURE 7 SLT interlocking joint between HDPE sheets.

It is very important that the self-hardening slurry maintains appropriate rehological properties long enough to allow the installation of the membrane in the cutoff wall and the interlock of the sheets at the joints. With the addition of dispersing agents, the workability time of ordinary self-hardening slurry can be substantially increased.

Until a few years ago, the construction of composite barriers was limited to depths shallower than 50 feet. Recently, a composite barrier with a depth of 100 feet was constructed by Rodio for the containment of a landfill close to Florence, Italy.

Cutoff Wall Constructed With Plastic Concrete

For the excavation of very deep cutoff walls, typically over 100-150 feet, or in special circumstances, like the presence of rock or hard layers, cutoff walls can be excavated under bentonite slurry with the Hydromill equipment. The excavation is subsequently backfilled with plastic concrete, i.e., a mix of coarse aggregate, sand, cement, and bentonite slurry that can be designed to achieve very low-permeability, usually in the range of 10-7 cm/s.

The Hydromill trenching equipment (Figure 8) is equipped with two cutting wheels able to cut the soil and rock. A submerged pump located in the frame of the equipment sends the spoilladen slurry from the bottom of the trench to the slurry treatment plant through a pipeline. At the plant, the spoils are separated from the slurry by means of a series of vibrating screens and cyclones, and the "clean" slurry is sent back to the top of the panel under excavation through another pipe line. The transport of the slurry from the bottom of panel, its treatment, and the return to the panel all take place in a closed loop (Figure 9).

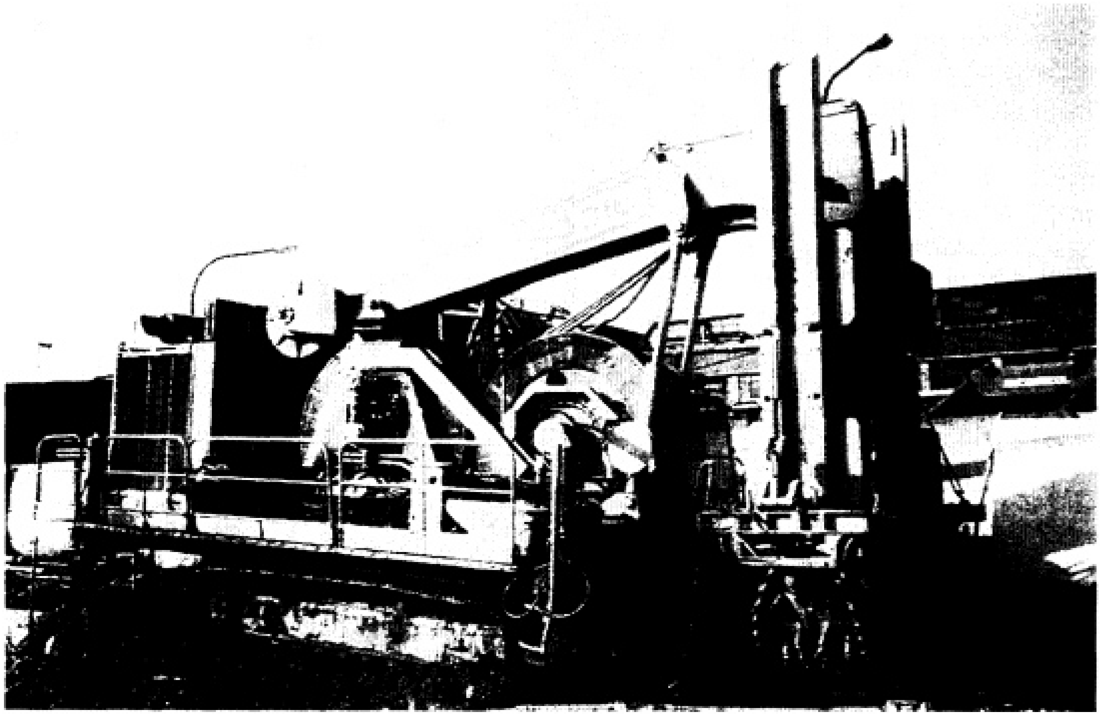

FIGURE 8 Hydromill, model "Latina," for low headroom conditions.

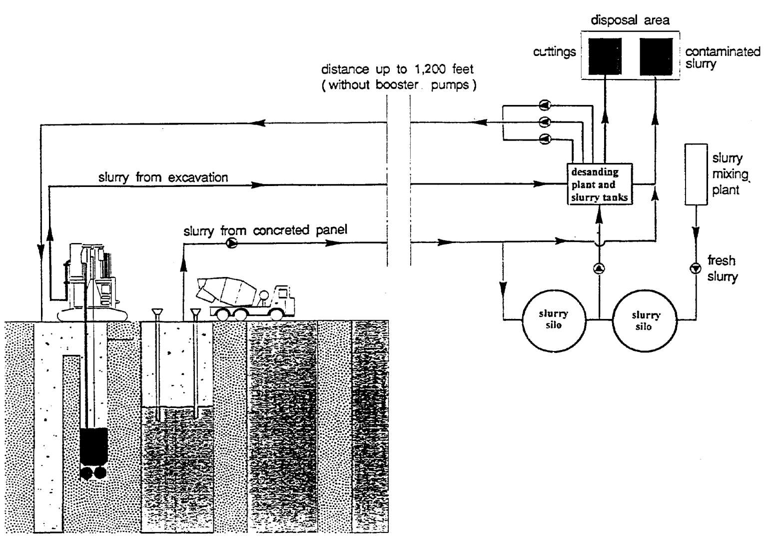

FIGURE 9 Hydromill excavation - Bentonite slurry flow chart.

A primary-secondary panel sequence is usually employed to excavate the cutoff wall. During the excavation of the secondary panels, a portion of the adjacent primary panels is cut by the Hydromill. The result is the creation of a rough contact surface between the panels that provides excellent wall continuity and water tightness.

The verticality of the trench is monitored constantly during the excavation with two highresolution inclinometers, mounted on the frame of the Hydromill trencher and connected to a readout unit located in the operator cab. The operator of the Hydromill can take corrective action at the slightest sign of deviation as transmitted by the inclinometers.

If the deviation occurs perpendicular to the wall axis, the deviation can be corrected by inclining the cutters' frame with respect to the main frame of the trencher, or by moving the shield connected to each side of the main frame of the trencher. If the deviation takes place longitudinally, the corrective action generally consists of varying the relative speed of the cutters and the power delivered to them. In normal operating conditions, the Hydromill equipment is capable of achieving a vertical tolerance of 0.3-0.5 percent in both the longitudinal and perpendicular direction.

The Hydromill technology has been applied by Rodio to excavate panels up to a depth of 330 feet (Bruce, et al., 1989). Rock with unconfined compressive strength of 4,000 psi has been excavated.

CONCLUSIONS

The selection of the most suitable method for the construction of deep barrier walls for waste containment is to be made considering the following factors:

- geological conditions of the site;
- type of soil, or rock, to be excavated;
- type of contaminants present in the soil;
- degree of water tightness to be achieved; and
- depth to be reached.

The methods presented in this paper can be employed economically and effectively to construct deep barrier walls for waste containment.

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SELF-HARDENING SLURRIES AND STABLE GROUTS FROM CEMENT-BENTONITE TO IMPERMIX®

Gilbert Tallard, President EnviroTrench Co., Pelham N.Y.

ABSTRACT

Conventional concretes and grouts, with which most engineers are familiar, are materials with a water/cement ratio generally below 0.5. The strength of these materials is in the tens of MPa (thousands of psi). The structural strength of these materials is their primary quality. A less familiar group of materials, particularly in the United States, consists of very watery slurries that set to form a solid of relatively low strength ranging from 0.1 to 3.5 MPa (15-500 psi). The water content of such materials may vary from 250 to over 500 percent, with a water/cement ratio as high as 10 or more. At these water content levels, in order to have a homogenous material, it is necessary to suspend the cement with a viscosifier showing thixotropic properties. The viscosifier is traditionally some kind of clay, generally bentonite. The notion of stabilizing a cement suspension with a tixothropic clay has made stable grouts possible. The degree of stability of such grouts is indicated by the amount of bleed or free water at set time. The essential purpose of these materials is to provide an economical means of controlling ground water migration in the ground, be it soil or rock. Whether such materials are called a slurry or a stable grout depends on the application. A grout is used when the application is localized; a slurry is used when the application is in an open excavation, although the term slurry grout, meaning watery stable grout, is often used. The application field is below the ground water table and slightly above. Given the high water content, the dehydration and destruction of the hardened slurry material would occur if exposed to the elements, without any kind of protection. Conserved in water, these materials have an infinite life. Originating in natural ground water seepage control in the dam construction and repair technology, the control of contaminated ground water migration called for similar technology. The chemistry of the environment and the service under which these materials are to perform has brought forth certain shortcomings. The strongest limitation has been the lack of water tightness of mixes using Ordinary Portland Cement with respect to the regulatory conductivity threshold of 10-7 cm/s. Bentonite clay loses some of its properties in the presence of calcium in the cement, and as a highly dispersive clay, it is subject to shrinkage in the presence of certain organic chemicals. In view of these limitations, a different pair of constituents has been found and performs surprisingly well, from both chemical and permeability standpoints. Named IMPERMIX®, this flexible formulation is based on attapulgite or sepiolite clays and finely ground blast furnace slag cement. Slag cement's long setting time is taken advantage of in some applications like slurry trenching in difficult ground, or shortened when strength buildup is necessary, as in sludge solidification. Given the low-viscosity and low-solids content of these suspensions, a very penetrating environmental grout for sealing contaminated fractured rock can be found in IMPERMIX– Whereas cement-bentonite slurries have found limited applications in environmental remediation, due to excessive permeability, the permeability threshold of self-hardening slurries has been lowered with IMPERMIX® by one and two orders of magnitude and occasionally more. With the fact that an IMPERMIX® formulation can be tailored to satisfy a specific chemical condition, it is now possible to engineer and control, with a high degree of quality assurance, the self-hardening slurry application in site-specific conditions. Contaminated ground water barriers have been created by

the one-phase slurry trenching technique using backhoe and clamshell tools, the injection vibrating beam method, and jet grouting. Future use for in-situ fixation by soil-mixing, pipe line abandonment, contained sludge in-situ solidification, and horizontal jetted barriers are a few future obvious applications. Solidifying low-level radioactive aqueous waste is a more engaging prospect.

Definitions

Slurry: suspension of solids in a large amount of a liquid-generally water-and not necessarily stable.

Thixotropy: a physical phenomenon occurring when a liquid builds up rigidity at rest, while reversing to a low-viscosity fluid in a dynamic condition; such liquids are also known as shear sensitive.

Mineral grout: suspension of mineral powders in water, not necessarily stable, and containing at least one cementitious ingredient to create an eventual set after injection into receiving ground.

Stable grout: suspension of at least one clay mineral and one cementitious powder capable of developing sufficient viscosity to maintain all particles in suspension with a minimum of bleed water.

Self-hardening slurry: a diluted, stable grout used in a one-step construction process in which the slurry in liquid phase is the slurry stabilizing the walls of a trench and in a set-in-place solid phase providing the desired end-product.

Bentonite: a highly dispersive montmorillonite clay used extensively for viscosifying water-based drilling or trenching fluids, and for the preparation of common stable grouts; viscosifies by adsorbing water between its structural platelets; high ion-exchange capability.

Attapulgite: a clay mineral of the palygorskite family with a needle-like crystalline structure; viscosification of water occurs by mechanically stacking the needles and trapping water between them; very little ion-exchange capability by comparison to bentonite.

Cement: common term for Ordinary Portland Cement, the most common cementitious binder used in the world; sets by forming at least three distinct chemical compounds and some noncombined salts.

Slag cement: finely ground blast furnace slag cement is a hydraulic binder manufactured from a residue of modern steel making; molten slag is blown into a cold-water spray, with the slag cooling into glassy pellets; once finely ground in a cement mill, an industrial pozzolanic material is obtained.

Cement-bentonite (CB): a generic mixture of Portland cement and bentonite, with a solids content of generally less than 35 percent, mixed into a stable grout and used for alluvium and fractured rock grouting as well as a self-hardening slurry for cutoff wall construction.

IMPERMIX®: a proprietary mixture of attapulgite clay and blast furnace slag cement and other ingredients used as a self-hardening slurry, typically at a maximum solids content of 22 percent and exhibiting very low-permeability, high strength, and very good stability under chemical attack.

INTRODUCTION

Chronologically, self-hardening slurries have their origin in stable grouts. Barrier technology, as understood in pure geotechnical construction, consists of controlling ground water flow under or through a man-made structure. The technology started with grout curtains and cutoff walls having application to embankment dam construction and repair. In the waterproofing of porous granular soils, stable grouts do a very good job in sealing these formations, by comparison with neat cement grouts. Cement-bentonite (CB) grouts have been used extensively in alluvium grouting around the world, except in the United States. The best permeability to be expected for the grouted soil is in the 10^{-5} cm/s range. However, with grout cutoffs up to 25-feet thick, a rather low gradient is at play, and the safety of the system can be seen as higher than in the case of the thin cement-bentonite cutoff wall subject to a much higher gradient. This construction method was developed in the 1970's, when tools and evolution in the rheology of the self-hardening slurries permitted the construction of cutoff walls 2-feet thick and over 100-feet deep.

LIMITATIONS OF CEMENT-BENTONITE

For better understanding, economy, and quality assurance, design engineers generally prefer trenched cutoff walls to grout curtains. When designing water reservoir embankments or contemplating the repair of a leaky earth dam, a conductivity (K) in the range of 10^{-6} cm/s is quite satisfactory in reducing the residual seepage to an acceptable value concurrently achieving a total cessation of internal erosion. Cement-bentonite self-hardening slurry cutoff walls installed in the United States since the mid-1970's for the purpose of impeding the migration of a contaminated aquifer had to get a variance from regulatory agencies with respect to the coefficient of permeability above the norm as established by the soil-bentonite slurry trench bench mark. Some more stringent agencies, such as the California Water Board, require both compatibility and a permeability coefficient of less than 10^{-7} cm/s, with no exception. In this context, 10 years ago the Los Angeles County Sanitary District was looking to use both self-hardening slurry technology and to meet the $K < 10^{-7}$ cm/s criterion for their numerous barriers to be installed across canyons and gullies confining its landfills. This writer was hired to formulate a cement-bentonite slurry that could perform reliably in the field and provide a lowpermeability. After a few months of testing, a cement-bentonite slurry prepared with 4.5 percent bentonite, 25 percent Portland cement, and 15 percent fly ash fluidified and retarded with an amount of lignosulfonate satisfied the criteria. Permeabilities at 60 days in the low 10-8 cm/s range were achieved. Practically, nearly a doubling of ingredient solids over a normal formulation were called for to reduce the permeability by one order of magnitude. The added cost for materials and preparation on-site was quite significant. At these solid levels, the cost for fluidifier/ retarder is equal to that of the bentonite. The recourse to a high dosage of fluidifier for viscosity control can lead (and did) to defects by excess dosage, causing suspended soil particles to settle in discrete bedding patterns of higher permeability. This solution could not be generalized to the uncontrolled hazardous waste remediation industry at large.

MATERIALS

As a combination, Wyoming bentonite, which is highly dispersive and prone to ion-exchange, and Portland cement, which exhibits an early false set and some leachability, are not the best partners for creating a low-permeability, high-water content, self-hardening slurry material. Hydrated bentonite will readily flocculate at the first encounter of a little amount of Portland cement. The flocculation causes a drastic change in the filtration characteristics of bentonite slurry, going from as low as 13 cm^3 on an American Petroleum Institute (API) standard test (100 psi for 30 min) to over 80 cm^3. Once the cement is mixed into the bentonite slurry, the filtrate, under API standard, becomes total (250-300 cm^3). It is clear that a head differential of 210 feet of water is unrealistic for most projects and the API standard test should be seen only as a qualitative procedure. Concurrently with the flocculation and filtrate loss, a significant increase of viscosity occurs.

The Europeans, who are not blessed with our resources in Wyoming and South Dakota, have to permutate natural calcium bentonites into sodium bentonite. This industrial process allows the production of all kinds of bentonites and, in particular, clays much less sensitive to calcium ion-exchange. Also, the number of cement types available in Europe is far greater than here, and certain blended cements offer a better match to bentonitic clays, as well as intrinsically higher chemical stability.

CHEMISTRY

When addressing hazardous waste barriers, the chemistry of the contaminant can become a major factor in the barrier's durability. Particularly with certain volatile organic compounds, such as benzene, xylene, or methanol, the water adsorbed between the bentonite mineral platelets suffer a change in dielectric constant (whether in a soil-bentonite or cement-bentonite configuration) and causes the bentonitic gel to contract by loss of dispersive energy. This causes an increase of conductivity that can be over an order of magnitude. The purpose of compatibility permeability testing under a higher than service gradient is to determine, in a few months, the degradation potential over the life of the barrier. When permanent enclosures are involved, a 50-year service life is far from overly ambitious.

QUALITY CONTROL/QUALITY ASSURANCE

This writer has always been adverse to the use of soil-bentonite barriers for permanent enclosures. A coarse technology and a crude construction method cannot provide the level of quality control (QC) and quality assurance (QA) necessary to satisfy all parties having a vested interest in the enclosure. On the other hand, the self-hardening slurry approach to the construction of a barrier allows the sophistication of an engineered project: at the design stage, during construction QC stage, and at the post-installation QA stage. With cement-bentonite self-hardening slurries, only a number of sanitary landfill barriers could be satisfied with a K above 10^{-7} cm/s with no adverse consequences, which meant practically the end of the road for cement-bentonite in hazardous waste remediation in the 1980's, a profound disappointment to this writer.

IMPERMIX®

The history of IMPERMIX® starts with this frustration and builds on a multiplicity of past experiences and a taste for pluridisciplinarity. As the steel mills of this country were closing in the 1970's, so were the sources of blended cement such as Portland/ground blast furnace slag cement blends. Today, very few sources are producing a pure blast furnace slag cement product that is sold to the ready-mix concrete industry. A remarkable characteristic of hardened blast furnace slag cement in very high water/cement ratio slurry is the strength, which is many times that of Portland cement mixed at the same ratio. In conventional concretes, the gain in strength is only 5-15 percent, with the permeability and resistance to salts improved noticeably.

When, 10 years ago, this writer constructed his first soil attapulgite slurry trench at a chemical facility on the Gulf of Mexico, actually using seawater to prepare the slurry, a good clay mate for the slag cement was recognized. A clay that viscosifies mechanically instead of swelling and doesn't blink at the sight of electrolytes appeared to be a good candidate indeed. The concept of a controlled initial viscosity based on mixing energy was extremely appealing.

Further research determined the range of interesting proportions. Surprisingly, a total of 15 percent solids by weight of water could produce a hardened slurry with a coefficient of permeability less than 10-7 cm/s and a strength substantially above 100 psi. From this research, IMPERMIX® was born, and the base formulation was set at one part attapulgite to 2 parts of slag. At the present time, it is possible to target 10-9 cm/s. Any increase in slag content is matched to a certain increase in clay to assure stability and chemical balance. Any slag proportioning above 15 percent involves some structural design consideration.

Once permeability to water was assured, compatibility with common pollutants was addressed. Organic chemicals, whether aqueous or nonaqueous, heavy metals, strong alkalis and acid solutions to a pH level of 2, all caused a long-term decrease in permeability with no degradation. Addition of fly ash, silica fume had no positive effect on the permeability. Addition of ground lime appeared to provide a positive decrease in permeability on well-cured samples, although initially retarding the set. Addition of filtrate reduction polymers proved very effective from a technical standpoint, although not always cost effective.

Applications

With a natural setting time of 70 hours (at room temperature, shorter in warm weather), the working time of an IMPERMIX® slurry is unusually long, allowing for the construction of practically jointless barriers. The long setting time coupled with high strength (for a self-hardening slurry, 150-250 psi) has allowed the construction of support of excavations where structural elements are placed and suspended inside the fluid trench until the slurry is hardened. A number of shoring systems can take advantage of this technique.

The high fluidity of IMPERMIX® mixes, which have proven to be non-shrink, opens a market for backfilling abandoned utility ducts or sewers where only cellular concrete could do the job at a much higher cost. A means of accelerating the setting of IMPERMIX® has opened a new field of application for light-weight backfill material. IMPERMIX is now being used to pretrench along the alignments of the slurry walls supporting the excavation for the Central Artery in Boston. In this application, the IMPERMIX® slurry is used as the trenching support slurry to minimize the size of the excavation required to remove obstacles such as granite blocks or timber cribbings, inherited from the rich history of this New England capital city. The elimination of dewatering and shoring is a major simplification and a much safer procedure. In lieu of backfilling with a conventional lean mix, the modified IMPERMIX® sets in-place overnight and hardens enough in 2 days to permit trenching of the slots for the guide wall construction ahead of the slurry wall panels excavation proper.

IMPERMIX–'s fluidity is also an asset when using jet grouting techniques to install a barrier under utilities and connecting to conventional trenched cutoffs or soil-displacement narrow walls.

The potential for soil-mixing in situ exists and has been demonstrated in the laboratory. The potential for sludge fixation also exists with the smallest increase in volume. One must bear in mind that IMPERMIX– may represent only 13 percent of solids; 87 percent is water (water content > 600 percent). It is perfectly conceivable to mix the contents of a leaky tank almost full of contaminated water (e.g., low-radiation water) into an IMPERMIX® grout that will not leach and to consider the problem solved satisfactorily over the long-term, as long as the tank remains buried. IMPERMIX® will only release water by drying. In a submerged condition, despite the already high water content, the water content of IMPERMIX® tends to increase.

Research Needs

There is still much that needs to be learned about IMPERMIX® before we start to understand why the performance of this combination of materials is so good. No electron microscope observation has been made, and no mercury permeation has been attempted to determine the system's porosity structure. An answer needs to be found to why IMPERMIX® is capable of combining, after set, more water than its already extremely high water content. A low-gradient permeability test had water going in for a month with no water coming out at all and no change in volume. From a practical standpoint, when performing a compatibility test, the practice with materials having a substantial solids content is to percolate a multiple of the pore volume, the latter being defined by the ratio of dry weight and the initial sample weight. In the case of self-hardening slurries, this definition results in the pore volume being equal to the water content. With the very low-permeability exhibited, it is clear that most of the water content is part of the structure and not of the pore space; therefore, the need arises to redefine "the pore volume" for realistic compatibility testing.

A review of compatibility testing procedures in a flexible wall triaxial cell permeameter should be made to compare the typical method using unrealistically high gradients (50-200) through fairly thick samples (4 inch) and methods using low gradients through a large-diameter and thin sample in an oversized triaxial permeameter cell (Haug, 1980). A new test for very low-permeability materials should be developed to establish threshold gradients below which the material can be considered truly impermeable.

Permeability is not the only concern when dealing with hazardous waste. Sorbtive characteristics and electric diffusion are areas that deserve investigation. Sorbtive properties of attapulgite clays are well known, and slag cement/lime combinations have been tested successfully for chromium. No evaluation of IMPERMIX® sorbtive properties has been undertaken yet, nor has diffusion been studied.

A better understanding of the material will permit the introduction of specific ingredients to improve chemical retention. This writer has started investigating the benefits of adding activated carbon to the mix. The chemical activation of the slag itself has caused a reduction in permeability for low solids IMPERMIX® formulations, an important improvement when trying to minimize the volume increase in fixation projects. Despite the unglamorous sulfur odor of fresh IMPERMIX® samples, sulfur contained in slag may be a factor in explaining the exceptional strength and should be investigated.

CONCLUSIONS

Cement-bentonite slurries and grouts have opened many avenues in geotechnical engineering and construction. Lack of choice in better local ingredients and chemical and regulatory thresholds have limited the full use of these products in the hazardous waste remediation business despite their "engineering" values. With contamination eventually migrating from soils into the rock basement, barriers must address both media, and self-hardening slurries are a good tool for dealing with both. A new pair of clay-cement ingredients, bearing the flag of IMPERMIX®, is a good candidate to take over the self-hardening slurry technology to carry it to the hazardous waste environment. In times when remediation funding is getting tighter, the return to permanent vertical and horizontal enclosures is in order, where possible. The laxity in slurry trench barrier technology (soil-bentonite) that has prevailed up to now because of the planned temporary nature of the barrier or because of the creation of negative gradient barriers should give way to barriers that are engineered and built to be barriers that act like barriers. The bureaucratic observance of performance criteria is often limited to a coefficient of permeability threshold without even defining the laboratory testing criteria. This attitude does not seem to factor in the constructability versus end-product quality elements that are essential to achieving one's objectives and has led to risky situations (not to mention law suits) that would not be tolerated in a permanent enclosure project. Some engineering and construction rigor will have to be reintroduced in the process and, if this is possible from an overall business/legal consideration, then long-term in-situ bioremediation (Nature's work) will take place safely at a fraction of the cost of the present approaches in areas where land can be furrowed for an extensive period of time.

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A FIELD TEST OF PERMEATION GROUTING IN HETEROGENEOUS SOILS USING A NEW GENERATION OF BARRIER LIQUIDS

G. J. Moridis, P. Persoff, J. A. Apps, L. Myer, P. Yen, and K. Pruess, Earth Sciences Division, Lawrence Berkeley National Laboratory

ABSTRACT

A field demonstration of permeation grouting was conducted at a gravel quarry near Los Banos, California, with the purpose of demonstrating the feasibility of the concept. Two grouts were used: a form of colloidal silica that gels after the addition of a gelling agent, and a polysiloxane that polymerizes after the addition of a catalyst. Both create relatively impermeable barriers in response to the large increase in viscosity during gelation or polymerization, respectively. The grouts were successfully injected at a depth between 10 and 14 feet. Subsequent exhumation of the injected gravels revealed that both grouts produced relatively uniform bulbs. Laboratory measurements of the grouted material retrieved from the field showed at least a four order of magnitude reduction in permeability over the ungrouted material.

INTRODUCTION

The development of in-situ contaminant containment technologies is necessitated by (a) the need to control and/or suppress the release of contaminants from buried sources, (b) the need to prevent the spread of existing plumes, and (c) the difficulty and cost associated with the recovery of contaminants from the subsurface by conventional means. The activities described in this paper advance the technology of permeation grouting, which will ultimately lead to powerful and more economical containment methods with broad applicability to a large variety of sites and a diversity of contaminant problems.

APPROACH

The basis for permeation grouting is to inject low-viscosity liquids into the subsurface to produce impermeable barriers through a chemically or physically induced substantial increase in viscosity. Appropriate emplacement of these liquids can contain a contaminated zone by entrapping and immobilizing both the contaminant source and the plume. The application of two general types of barrier fluids are described in this paper (Moridis et al., 1993, 1994; Persoff et al., 1994). The first is colloidal silica (CS), which consists of an aqueous suspension of silica microspheres in a stabilizing electrolyte. It has excellent durability characteristics, poses no health hazard, is practically unaffected by filtration, and is chemically and biologically benign. The increase in viscosity of the CS following injection is due to a controlled gelation process induced by the presence of a neutralizing agent or a concentrated salt solution, either of which are added immediately prior to injection at ambient temperatures. The CS has a tendency to interact with the geologic matrix, and therefore, special formulations or techniques are required to minimize or eliminate the impact of such interactions.

The second type of barrier fluid is an organic liquid belonging to the polysiloxane (PSX) family, chemically and biologically inert silicon-based chain polymers. PSX increases in viscosity through to a vulcanization-like process in which a catalyst induced cross-linkage of the polymer chains forms a high viscosity elastic product. The cross-linking process is controlled by the quantities of the catalyst, crosslinker, and (occasionally) retardant added to the PSX prior to injection. PSXs are largely unaffected by aquifer or waste chemistry.

Permeation grouting technology can be applied in three ways. The first, conditions permitting, results in permanent immobilization of the contaminants in the affected aquifer region by sealing and entombing them in a "monolith" of grout. In the second option, an impermeable container is created to surround and isolate the contaminated region for treatment at a later time. Finally, the third option allows sealing of permeable aquifer zones, thus confining the effects of traditional cleanup techniques (such as pump-and-treat) to less permeable zones.

Substantial preparatory work was conducted to ensure the success of permeation grouting technology in the field. The work included identification and characterization of promising materials, evaluation of their containment potential by means of laboratory and pilot-scale experiments, and the development of appropriate numerical simulators. Many institutional issues involving interactions with regulatory agencies and industry partners also required resolution.

Lawrence Berkeley National Laboratory (LBNL) staff completed a wide search for fluids with desired properties and identified CS and PSX as promising candidates. The rheological and wettability properties of these barrier fluids were measured. Laboratory studies of barrier fluid flow and emplacement in porous media were conducted, and it was determined that both CS and PSX are effective in sealing porous media. Alternative processes were developed to alleviate possible effects of the soil chemistry on the CS gel times, and ways to control the gel time and the texture of the gels were identified. Protocols for the sequential injection of CS were established, and it was demonstrated that hydraulic conductivities could be reduced to less than 10^{-8} cm/s after two injections. Processes to control the viscosity and gel time of PSX were also identified. PSX cross linkage times are far less sensitive to the soil chemistry than CS gelation. Furthermore, hydraulic conductivities could be reduced to 10-10 cm/s after a single injection.

In collaboration with the manufacturers, new CS and PSX formulations were developed to meet barrier fluid requirements (the CS formulation selected being unaffected by the soil chemistry, and the new PSX formulation having an initial viscosity low enough to allow injection using existing equipment). A series of laboratory tests were conducted to investigate the barrier performance of the selected CS and PSX formulations at all length scales of interest: from submillimeter (pore micromodels) to one-dimensional experiments (column studies) to two dimensional studies (ranging from 1 feet × 1 feet × 1 feet to 7 feet × 6 feet × 0.5 feet). Preliminary waste compatibility tests were conducted, and it was concluded that both CS and PSX are not significantly affected by a wide range of wastes contained in the buried tanks at the Hanford Reservation, Washington.

The general-purpose TOUGH2™ model (Pruess, 1991), was appropriately modified to predict the flow and behavior of gelling/cross-linking fluids when injected into porous media, (Finsterle, Moridis, and Pruess). The expanded TOUGH2™ was used to design the laboratory experiments (one- and two-dimensional) of barrier fluid injection, and to conduct a sensitivity analysis of the relevant parameters (Finsterle, Moridis, and Pruess).

In interactions with industry and regulatory agencies, LBNL developed an agreement with Bechtel to collaborate in the area of barrier fluid emplacement. LBNL also signed a confidentiality agreement with Dow Coming, the manufacturer of PSX, as a result of which Dow Coming made available to the project the new low-viscosity PSX used in the experiments and the field test. Agreements for possible applications of the barrier technology at a number of potential sites were concluded and a Categorical Exclusion under National Environmental Protection Act (NEPA) regulations for the first-level field test was obtained, due to the environmentally benign nature of the barrier fluids.

In preparation for the field test, LBNL staff developed a design package for the application of the barrier fluid technology using TOUGH2™, completed a preliminary evaluation of geophysical techniques for monitoring barrier performance and emplacement, identified a local site in California with a subsurface geology similar to that at Hanford, and obtained permission from the owner and the regulators to conduct the first-level test at that site. Following the signing of the Host Site Agreement, the field test was conducted in January 1995.

THE FIRST FIELD-LEVEL DEMONSTRATION

In the following sections, various aspects of the field demonstration are described. These include the objectives of the demonstration, a site description, specification of the barrier liquids, and the four stages in executing the demonstration: (a) well drilling and permeability measurements, (b) barrier fluid injection, (c) grouted bulb (plume) excavation and sample recovery, and (d) laboratory investigations of grouted samples.

Objectives

The objectives of the test were to demonstrate the ability to:

- inject colloidal silica and polysiloxane using standard permeation grouting equipment;
- track the grout fluid movement using tiltmeter measurements of ground surface deformation;
- control of the grout fluid gel time under in-situ chemical conditions;
- create a uniform grout plume in very heterogeneous matrices including cobbles, gravels, sands, silts and clays;
- create intersecting/merging plumes of grout; and
- decrease the permeability of the grouted soils.

The demonstration was not intended to prove the creation of continuous and/or impermeable barriers. Such an effort would be significantly larger in scope and involve merging and overlapping the injected barrier liquid plumes, as well as multiple injections.

The Site

The test site is located in central California in a quarry owned by the Los Banos Gravel Company. The quarry is situated along the western flank of the San Joaquin Valley, adjacent to the eastern margin of the central California Coast Ranges. The quarry exploits river gravels in a 100km^2 alluvial fan generated by Los Banos Creek at the foot of the California Coast Range. The deposits exposed at the quarry are primarily coarse sands and gravels, deposited on a distributary lobe of Los Banos Creek adjacent to its present channel. They are internally heterogeneous, with discontinuous and lenticular coarser and finer strata, and occasional lenses of well-sorted crossbedded sands. Large gravel and cobble clasts are commonly set in the sandy matrix, and range between 1 and 10 cm and sometimes larger. The matrix is predominantly coarse sand (0.5-1 mm), and comprises varicolored lithic fragments, along with grains of feldspar, quartz, and quartzite. Induration, where present, is caused by infiltration (illuviation) of clay into pores between sand grains; a fine film of yellow-brown clay can be seen binding the sandy matrix in most samples.

Prior to development of the Los Banos quarry, the area was under agricultural use. Upon development of the quarry, the uppermost soil layers were partially stripped and staged in piles away from the area of gravel excavation.

Barrier Liquids

The barrier fluids selected for injection included one type of PSX (2-7154-PSX-10, hereafter referred to as PSX-10; Dow Corning, Midland, Mich.) and one type of CS (Nyacol DP5110; PQ Corporation, Valley Forge, Pa.). In preliminary experiments, other variants of PSX and CS products were also tested. All the barrier fluids tested are environmentally benign and carry no warning label requirements.

Nyacol DP5 110 is a CS, in which silica on the particle surfaces has been partly replaced by alumina; its solid content is 30 weight-percent, and its pH is 6.5. A technical grade aqueous solution of $CaCl_2$, HB-23 (Hill Bros. Chemical, San Jose, Calif.) was used to promote gelation for the final tests and the field demonstration. The concentration of the solution was nominally 35 weight-percent (4 mol/L) $CaCl_2$.

PSX-10 is a polydimethylsiloxane, divinyl terminated to provide active sites for cross-linking. It is formulated by the manufacturer with a cross linker (a small cyclic siloxane molecule) that can react with the terminations of the long chains, in the presence of small concentrations an organically coordinated platinum catalyst. The polydimethylsiloxane and crosslinker are delivered already mixed, but unreacted. A platinum based catalyst is added by the user at the level necessary to achieve the desired gel-time.

Well Drilling and Permeability Measurements

Four injection and four observation wells were drilled with a layout shown in Figure 1 The injection wells were drilled to a depth of 16 feet, while the observation wells were drilled to depths ranging between 12 and 20 feet. Following well completion, all the wells were fitted with appropriate tubing, and probes were punched through the bottom of the wells for air permeability measurements.

Air permeability measurements included static single point permeameter tests using constant head air injection tests, and a new dual probe dynamic pressure technique developed at LBNL for measurement of air permeability between wells (Garbesi, 1994). The latter uses a sinusoidally varying pressure with a mean near-atmospheric pressure at the injection well. Pressure responses are continuously monitored at several observation wells. The single point permeameter technique provides information on the permeability immediately surrounding each well, while the dual probe technique provides information on the permeability between wells.

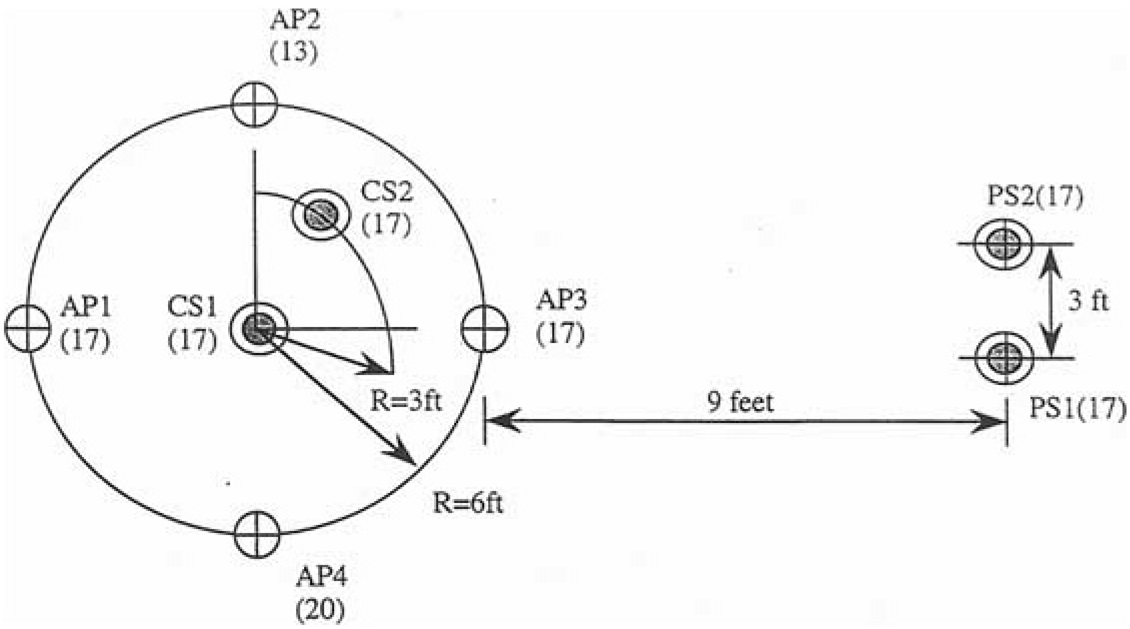

FIGURE 1 Plans of well locations at the injection site.

The static permeability measurements, conducted in all eight wells, indicated permeabilities ranging from a high of 1.0×10^{-10} m^2 to a low of 3.6×10^{-13} m^2. For all but two wells, the values ranged from 5.6×10^{-11} to 8.1×10^{-11} m^2

Injections into holes AP1, CS1, and CS2 using the new dual probe dynamic pressure technique, yielded inter-hole permeabilities between 3.510^{9} m^2 and 110^{-11} m . These permeabilities are between 1 to 2 orders of magnitude higher than those obtained using the static technique. The apparent lack of agreement is due to conceptual differences between the two approaches: the static technique in essence measures the permeability at the point of injection, whereas the dynamic technique measures the mean permeability between a source and a receptor well along paths that are not necessarily the shortest. Though the magnitudes of the static and dynamic measurements differ, trends are consistent between the two techniques. These observations substantiate the validity of the two methods, and support the hypothesis that the differences between static and dynamic values are due to scale effects.

After completing the air permeability tests, all observation wells were plugged to prevent barrier liquids from flowing into the observation wells and bypassing the area to be grouted. The bottoms of the injection wells were also plugged.

Barrier Fluid Injection

The barrier liquids were injected through three ports in each well (at depths of 10, 12, and 14 feet) using the tube-à-manchette technique. Approximately 400 gallons of CS grout was injected into two wells, CSI and CS2. About 120 gallons of PSX-10 was injected into a single well, PS1.

The smaller scale of the PSX-10 injection test was dictated by budget considerations, as it is still a developmental product and economies of scale in its production have not yet been realized.

The barrier liquids (CS and $CaCl_2$ brine, PSX-10 and catalyst) were premixed at the surface using the agitators of the mixing tank and the recirculation equipment of the grouting system. For the CS injection, food-color dye was added to enhance its visibility during subsequent excavation of the site. Green dye was added to the batches injected into CS 1, and purple dye into the CS2 batches. The same quantity of barrier fluid (66 gallons for CS, 40 gallons for PSX-10) was injected at each depth. Standard chemical grouting equipment was used for delivering the barrier fluids to the hole. The procedure for injection followed those typically used in tube-à-manchette grouting. The injection sequence was carried out in order to maximize complete permeation of the soil in the vicinity of the wells. Thus, injection began at the lowest port (14 feet), followed by injection through the uppermost port (10 feet) and, finally, injection through the intermediate depth port (12 feet).

The barrier fluids were injected without any significant rise in pressure (which would have indicated premature gelling). During injection, the volume of injected grout and injection pressure were monitored. Average values of injectivity, a measure of the apparent permeability at each injection port, decreased with depth with values at the 14 feet depth an order of magnitude or more lower than those at shallower depths.

Eight tilt meters were installed at the injection site. The tiltmeter array recorded ground movement every 60 seconds throughout the test, and was able to detect movement of the injected fluids. Tiltmeters measure the angle of deviation of the land surface from the vertical axis. Because the deformation detected by tiltmeters is minuscule (nano- to micro-radians), LBNL staff decided to apply this technology to track the swelling and uplift at the earth's surface due to the intrusion of the barrier liquids.

Deducing the movement of fluids through the subsurface from surface tilt requires the solution of an inverse problem, which cannot presently be conducted in the field in real time, although such is anticipated with the rapid advancement of computer technology.

Excavation and Visual Inspection

The excavation of the grouted plumes was facilitated by the proximity of the wells to the exposed face of the quarry (20 feet) and the use of heavy earth moving equipment. The ground was excavated to a depth of up to 21 feet. Both CS and PSX-10 had gelled/crosslinked in the subsurface satisfactorily. Despite the extreme soil heterogeneity, both the CS and the PSX-10 created fairly uniform plumes, indicating that the potential problem of flow along preferential pathways of high permeability (such as a gravel bed overlying a tight silty or clayey zone) can be overcome.

The CS grouted and sealed fractures and large pores in the clays. In open zones (such as gravels with centimeter-sized pores) it did not fully saturate the voids but appeared to have sealed access to them. CS did not impart substantial structural strength to the matrix but permitted vertical sections of the matrix (with the exception of very loose and friable materials) to stand, as shown in Figure 2.

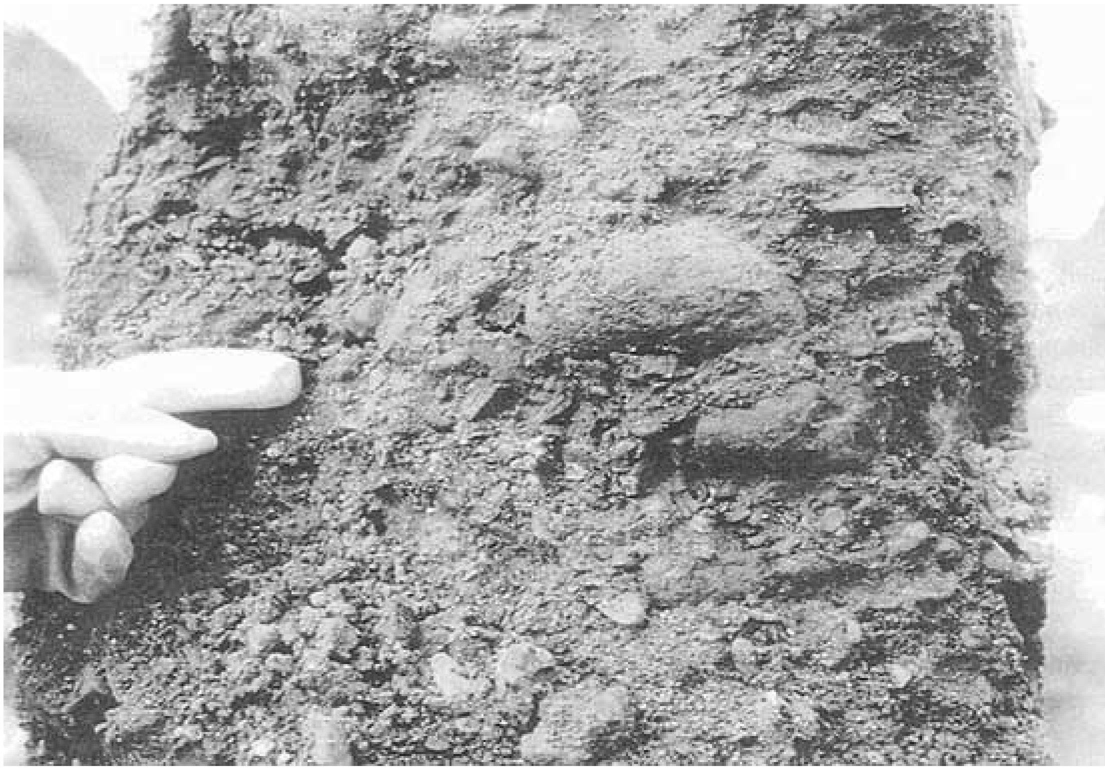

FIGURE 2 Excavated portion of the CS grouted soil.

PSX-10 was singularly successful in grouting the extremely heterogeneous subsurface at the site. PSX-10 created an almost symmetric plume, grouting and sealing gravels, cobbles, sands, silts, and clays. PSX-10 filled and sealed large pores and fractures, as well as accessible small pores in the vicinity of these pores/fractures. In extremely large voids in open zones, it coated the individual rocks in the gravel and sealed access to and egress from these zones. PSX-10 also invaded clays and silts (Figure 3), which is unusual. The mechanism through which this penetration is achieved has not been determined, but is under investigation.

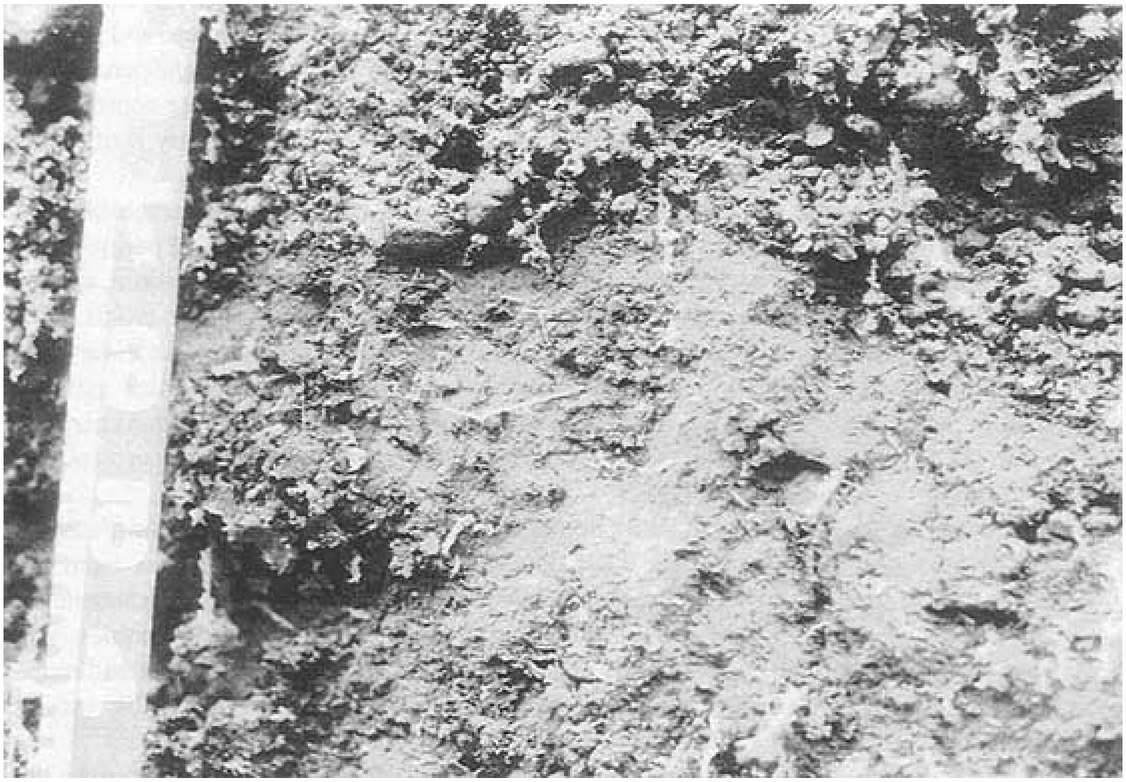

FIGURE 3 PSX-10 grouted soil at the interface of sandy and argillaceous zones.

PSX-10 is relatively easy to identify in the subsurface. Unlike CS, PSX-10 imparted structural strength and elasticity to the grouted soil volume, and gave sufficient strength to incoherent gravels to permit vertical walls to stand. It fully penetrated clean sands, which resisted disaggregation due to its considerable elasticity.

Post-Excavation Analyses

The grouted plumes were excavated primarily to determine the volumetric extent of the grouted zone. LBNL staff also took advantage of the excavation to recover boulder-size chunks of grouted sand from which smaller samples could be taken for permeability measurement in the laboratory. After excavation, grab samples of ungrouted matrix were taken at various depths from locations adjacent to the grouted bulbs. Both moisture content and material gradational analyses were performed on these samples.

The moisture content of the ungrouted soil was very low, but increased with depth from about 2.5-5 weight-percent, with most of the increase occurring at depths of 10 feet and greater. The gradational analysis showed an increase in fines with depth from 1-2 to 8-9 weight-percent. An abrupt increase in fines is seen at depths greater than 10 feet. A correlation in moisture content with fines would be expected. The gradational analysis also correlated with the injectivity profile and visual observations that the amount of fines increased with depth.

The permeability of grouted sand depends primarily upon two factors: the permeability of the grout itself, and the degree of grout saturation in the pore space. The lower limit of permeability is achieved when the pore space is completely filled with grout. To estimate this lower limit, special samples were prepared by a method in which sand is poured into liquid grout in molds. This method ensured a complete filling of pore space by the grout and resulted in an absolute lower limit of permeability that is unattainable with a single injection under field conditions. Other samples were prepared in the laboratory by injecting grout upward into sandpacks in order to minimize the amount of trapped air. Samples prepared in this manner represent the lower limit of permeability that could be achieved by injection in the field.

The permeabilities of the grouted sand samples were measured using a Wykeham-Farrance flexible wall permeameter (Humboldt Equipment, Durham, North Carolina). Samples from the field were cored or carved from the boulder-sized chunks for insertion into the permeameter. Coring using a soil-sampling tube was possible only with a material containing no pebbles. The extreme heterogeneity of the formation at the Los Banos site made it difficult to sample and make permeability measurements. Hence, the number of field samples subjected to permeability testing was limited.

In Table 1, the three types of samples are represented: (i) samples prepared by pouring the sand into the grout, (ii) samples prepared by laboratory injection into sandpacks, and (iii) field samples. These three types of samples have increasing ungrouted voids. Because the field samples are expected to have the greatest amount of ungrouted voids, multiple injections will be required to achieve permeability reductions of type (ii) in field applications (Moridis et al., 1993). This goal was not pursued in the first-level field injection, as the reduction of permeability to a near-zero level was not among the objectives of this field demonstration for the reasons discussed earlier.

A review of the hydraulic conductivity data confirms that it increases with the increase of ungrouted voids. In comparing the laboratory prepared samples with nearly complete grout saturation (i), those grouted with PSX-10 had lower hydraulic conductivity than those grouted with CS. Sands with an initial hydraulic conductivity on the order of 10^{-4} m/s, can attain an ultimate hydraulic conductivity of 10^{-10} m/s level after grouting with CS, while PSX-10 reduces hydraulic conductivity even further to 10^{-12} m/s. These differences reflect the different permeabilities of the grout materials. CS gel contains a significant volume of water, and diffusion of dye through the aqueous component can be observed in a matter of hours in a plug of gelled CS, indicating a potential for diffusive transport. No such diffusion occurs in PSX-10.

Table 1 Hydraulic Conductivity Measurements on Laboratory and Field Samples of Grouted sand

Table 1. Hydraulic Conductivity Measurements on Laboratory and Field Samples of Grouted sand

Sample	Sample Type	Sample Length *(in.)*	Hydraulic Gradient (-)×10^3	Cell Bias Pressure *(psi)*	Hydraulic Conductivity *(m/s)*
Hanford sand, PSX-10,#1	laboratory injection	4	69.767	14	4.08×10^{-12}
Hanford sand, DP5110, #1	laboratory injection	2	13.953	20	1.03×10^{-09}
		2	13.953	40	6.33×10^{-10}
		2	13.953	60	4.60×10^{-10}
		2	41.86	60	4.20×10^{-10}
Los Banos sand, PSX-10, #1	cored field sample	3	9.302	5	2.28×10^{-6}
		3	9.302	10	1.52×10^{-6}
		3	9.302	20	1.14×10^{-6}
		3	27.907	20	1.24×10^{-6}
Los Banos sand, PSX-10, #2	cored field sample	3	4.651	10	4.52×10^{-6}
		3	4.651	20	2.75×10^{-6}
		3	4.651	40	2.15×10^{-6}
Hanford sand, DP5110, #2	sand added to DP5110	3	9.302	5	6.48×10^{-10}
		3	9.302	10	3.39×10^{-10}
		3	9.302	20	2.02×10^{-10}
Los Banos sand, DP5110, #1	carved field sample	2	6.977	5	3.96×10^{-6}
		2	6.977	10	3.07×10^{-6}
		2	6.977	20	2.59×10^{-6}
Los Banos, DP5110, #2	carved field sample	2	6.977	5	6.02×10^{-6}
		2	6.977	10	3.63×10^{-6}
		2	6.977	20	2.85×10^{-6}
Hanford, PSX 10, #2	laboratory injection	3	46.512	10	2.90×10^{-6}
		3	27.907	20	3.37×10^{-7}
		3	27.907	40	1.70×10^{-8}
		3	55.814	40	1.18×10^{-8}
		3	55.814	60	6.03×10^{-9}

The Hanford-PSX-10 #2 sample shows unusually high hydraulic conductivities for laboratory-grouted cylindrical samples, which can be due to an imperfect outer cylindrical surface that allowed flow between the rubber membrane and the grouted core. With increasing confining pressure, the hydraulic conductivity decreases, confirming the visual observation of surface imperfections. Such side-flow effects are expected to be far more pronounced in the cored or carved field samples.

In the case of field-grouted sand and pebbles, the observed hydraulic conductivities reflect incomplete saturation of the pore space. Damage to samples during recovery, transport, storage, and trimming to fit the apparatus could also have contributed to increases in hydraulic conductivity. Similar values were observed whether CS or PSX-10 grout was used, but this may not mean anything since they were different samples from different locations and with different soil textures. Partial saturation of pore space is also suggested by the observation of the larger than expected plumes. This supports the view that grout desaturation occurred due to plume spreading. LBNL's plume emplacement model predicts that this phenomenon will always occur in the vadose zone.

The problem arising from plume spreading and incomplete sealing can be solved by multiple, sequential injections of grout. Moridis et al. (1993) demonstrated this technique in sandpacks. Because plume spreading does not occur in sandpacks, the desaturating effect was achieved by saturating the sandpack with grout and then blowing air through the sandpack to displace the grout. Hydraulic conductivities ranging from 3×10^{-7} to 1×10^{-5} m/s were observed after the first injection, which are similar to the values of order 10^{-6} m/s observed in Los Banos field samples. After two or three such injections, hydraulic conductivity was reduced to $1x10^{-10}$ m/s, i.e. close to the type (i) laboratory result.

The grouted Los Banos material is 2 orders of magnitude less permeable than the ungrouted sand fraction of these materials. The sand fraction is less permeable than the actual soil due to its finer texture. Compared to the field measurement of air permeability, these samples indicate a permeability reduction by 3 to 4 orders of magnitude. In that respect, the results are very encouraging.

Data from the tiltmeter measurements was inverted in order to relate the tiltmeter measurements to the shape and extent of the injected grout plume. Based on the inversion results, the ground motion due to injection could be predicted. The peak vertical displacement of the land surface due to injection of CS was found to be 0.18 micrometers. The preliminary work suggests that tilt measurements can be used to monitor subsurface injections. However, further refinement of the technique is required for future application.

SUMMARY AND CONCLUSIONS

A first-stage field injection of colloidal silica and polysiloxane grout was completed successfully. The fluids were injected at depths of 10-14 feet in a heterogeneous unsaturated deposit of sand, silt and gravel, typical of many arid DOE cleanup sites and particularly analogous to the conditions of the Hanford Reservation. Both grouts effectively permeated gravel and sand beds. Despite the extreme heterogeneity, both the CS and the PSX-10 created fairly uniform plumes. Within the grouted plumes, both large and small pores were grouted. The CS grouted plume did not have substantial cohesiveness or strength, but allowed vertical sections of the soil to be exposed. Unlike CS, PSX-10 imparts structural strength and elasticity to the grouted soil. PSX-10 is relatively easy to identify in the subsurface and gave sufficient strength to very loose gravels without any cohesiveness to form vertical walls. Characterization of in-situ permeability at the site was carried out using both single hole and dual probe dynamic pressure air permeability methods. The dual-probe technique, sampling a larger volume of material, gave permeabilities at least an order of magnitude higher than the single hole measurements. Tiltmeters were used successfully to monitor surface displacements during grout injection. The resulting data was then inverted to model the shape of the subsurface plume, which would have produced the observed surface displacement. In conclusion, LBNL staff believe that the first field test was an unqualified success, and that the objectives were achieved.

ACKNOWLEDGMENTS

The authors are indebted to their colleagues, Drs. Stefan Finsterle, Pat L. Williams, and Don W. Vasco, and to members of the LBNL Geophysical Measurements Facility team (Jim Dougherty, Ray Solbau, Phil Rizzo, and Don Lippert) for the planning, preparation, support, and flawless execution of the field test. Thanks are due to Mr. Gordon Mills, General Manager of the Los Banos Gravel Company, for his generous help in locating an appropriate site for the field test and facilitating its execution. This work was supported by the U.S. Department of Energy, Office of Environmental Management, Office of Technology Development, under Contract No. DE-AC0376SF00098.

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SEALABLE JOINT STEEL SHEET PILING FOR GROUND WATER POLLUTION CONTROL

David J.A. Smyth and John A. Cherry, Waterloo Centre for Ground water Research (WCGR), University of Waterloo, Waterloo, Ontario, Canada; and Robin J. Jowett, Waterloo Barrier, Inc., Rockwood, Ontario, Canada

ABSTRACT

The Waterloo Barrier™ (patents pending) system employs modified steel sheet piling for use in the control and containment of subsurface contamination. The interlocking joints of adjacent sheet piles have been modified to incorporate a cavity that can be flushed and filled with a sealant subsequent to installation of the sheet piles in the ground. Field hydraulic tests indicate that bulk hydraulic conductivity values of the barrier wall system of less than 10^{-8} cm/s can be achieved using a variety of sealant materials. The installation and sealing process affords opportunities to ensure that the integrity of the barrier wall is good. The technology was developed recently; it has been used to construct test cells for field research pertaining to the behavior and remediation of contamination in ground water and also has been applied commercially for ground water pollution control at industrial, military and waste management sites.

INTRODUCTION

Ground water contamination arising from inappropriate handling and disposal practices for industrial chemicals, products, and wastes has been identified as a problem at tens of thousands of commercial, industrial, military and government agency sites across North America and Europe. More than a decade of experience and expenditures of billions of dollars have demonstrated that ground water remediation is a difficult task and that ground water remediation programs, particularly in cases where full restoration of the ground water system to a condition suitable for unrestricted water-supply use is required, have generally fallen short of expectations. The apparent failures in subsurface restoration have resulted from a lack of clear recognition about the nature and characteristics of the contamination problem and from limitations or inappropriate applications of technologies used in the remediation process.

Subsurface contamination problems generally have two components: a zone or plume of dissolved contaminants emanating with ground water flow from a source zone, where the contaminants were introduced and continue to exist in the subsurface. The characteristics of the source zone are dependent on the contaminant type. For many inorganic contaminants, the source zone may contain solid, soluble materials. Industrial organic contaminants may be present as immiscible-phase liquid pools and residual within the geologic media, or as organic vapors in the zone above the water table. For LNAPLs (light, non-aqueous phase liquids) such as petroleum hydrocarbons, contamination in the source area is generally confined to the vadose and shallow ground water zones. DNAPLs (dense, non-aqueous phase liquids), such as chlorinated solvents, may penetrate to significant depths below the water table, so contaminant distribution in the source zone may extend from the vadose zone to significant depth within the ground water zone.

The solubility of some of the industrial organic compounds, including most of the volatile organic compounds associated with typical LNAPLs and DNAPLs, and inorganic contaminants in water may exceed the levels corresponding to regulatory human health and environmental criteria. In their dissolved form, these contaminants may also be quite mobile in ground water. In general, the mass of contaminants present in the source zone far exceeds that present in the dissolved-phase in the associated plume, although the plume may be spatially more extensive. Potential risks and impacts to human health and the environment, however, are often more immediate for the dissolved-phase contaminants in the plume than for the source zone.

The most common approach to ground water remediation has been based on ground water extraction by wells or drains, with subsequent treatment of the contaminated water prior to its ultimate discharge back to the environment. This pump-and-treat approach can be effective in the control or containment of plumes, but as indicated by Mackay and Cherry (1989), it generally requires long-term operation. Heterogeneities of geological materials within the ground water system may prolong the time frame required for removal of dissolved-phase contaminants. Further, if subsurface sources of contaminants are present, it can be anticipated that pump-and treat control will be required for decades and longer, and dissolved-phase plumes will be reestablished if pump-and-treat operations are terminated.

The growing recognition of the limitations and inefficiencies of pump-and-treat has provided strong impetus to develop alternate approaches and new technologies for ground water remediation. Cherry, Feenstra, and Mackay (1992) and Mackay, Feenstra, and Cherry (1993) outline an approach to ground water remediation that recognizes the distinct implications of contamination within the source zone and the plume. They suggest three levels of remediation, including:

- plume containment, which leaves the subsurface source in-place, but leads to no further expansion of the plume;
- partial aquifer restoration, which involves long-term isolation of the source zone in combination with remediation of the plume;
- aquifer restoration, which involves full remediation of both the plume and source zone. In cases involving DNAPL contamination, full aquifer restoration may be an impractical goal using current technologies.

Cherry et al. (1992) and Mackay et al. (1993) further suggest four approaches to source zone isolation. As shown in Figure 1 for the DNAPL case, source zone isolation may be provided by containment within a low-permeability cutoff wall or barrier, long-term hydraulic control using an active pump-and-treat system, or in-situ treatment of contaminants emanating from the source zone using permeable reaction curtains and funnel-and-gate systems. Each of the approaches will have their merits and limitations for different contaminant problems in different hydrogeological settings; however, in many circumstances, there may be good opportunities for using vertical barriers for contaminant source zone isolation.

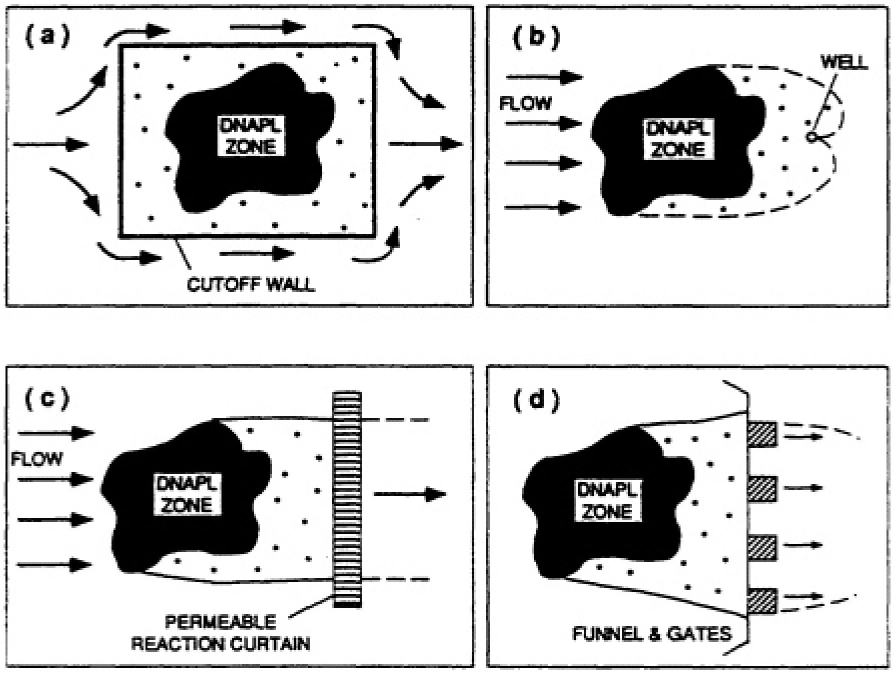

FIGURE 1 Contaminant source zone isolation using (a) low-permeabilitybarrier enclosure, (b) hydraulic containment by pump-and-treat, and contaminant containment by (c) permeable reaction curtain and (d) funnel-and-gate system.

Starr and Cherry (1992) and Mutch, Ash, and Cavalli (1994) provide discussions pertaining to the use of low-permeability barriers for control of ground water contamination. Hydraulic performance and the degree of containment provided by an enclosure can be optimized in situations where the barrier can be keyed into an underlying aquitard of low-permeability geological materials beneath a source zone. In circumstances where this is not possible due to the absence of such conditions, significant containment can be achieved by an enclosure that extends to depths beneath the source zone but that is not keyed into an underlying aquitard if a pump-and-treat system is operated within the enclosure. The presence of the enclosure will reduce significantly the volumes of water that must be pumped to maintain hydraulic control in comparison to systems where an enclosure is not used, and hence will also reduce operational and treatment costs for the contaminated water. It is also conceivable that restoration of a source zone using chemical flushing technologies for enhanced removal or in-situ destruction of contaminants will be more efficient in cases where an enclosure is present.

As Mutch et al. (1994) indicate, there has been a resurgence in research, development and application of various barrier wall construction technologies within recent years. This resurgence has resulted in the development of capabilities to improve the hydraulic performance and extend the depths to which barrier walls can be constructed. Given the variety of construction techniques available, it is reasonable to assume that there will be technical and cost advantages of using particular barrier walls in different situations. Conventional construction techniques include compacted clay barriers and slurry trenches, which typically incorporate soil-bentonite, soil attapulgite, and cement-bentonite mixtures in the barrier. New and developing technologies for barrier construction include vibrated beam cutoff walls, deep soil-mixing or auger cast walls, jet grouted walls, geomembrane barriers, and sealable joint steel sheet piling.

The remainder of this paper provides an overview of the development and application of sealable joint steel sheet piling (Waterloo Barrier ™ -patents pending) for barrier wall construction.

Waterloo Barrier™

The initial concept and field applications of the Waterloo Barrier ™ arose from a requirement for secure test cells for contaminant-related ground water research at the University of Waterloo (UW) in the late 1980's and early 1990's. Field research involving the controlled introduction of DNAPL chemicals to a shallow sand aquifer was being conducted at Canadian Forces Base Borden approximately 100 km northwest of Toronto, Ontario, Canada. The sand aquifer is underlain by a clay-rich aquitard at depths ranging from several meters to in excess of 10 m below ground surface. A trial application of jet grouting technology for construction of a test cell proved to be unsatisfactory. Slurry wall barriers were also considered. The required test cells were quite small, involving total wall lengths of up to 40 m, and projected costs were high, primarily as a consequence of the costs associated with the mobilization of the construction equipment. Thus, other construction options were sought.

Some preliminary experimentation was undertaken using conventional steel sheet piling. It was soon recognized that the leakage of water through the joints of conventional sheet piling may not always be suitable for contaminant-control applications. The search for methods to improve the seal between adjacent sheet piles ultimately led to the development of a sealable cavity at the joints. Although the initial version involved the modification of conventional sheet piling with an angle-welded sealable cavity at each joint, the capability to produce a special cold-rolled sheet pile with the sealable cavity joint incorporated directly in the production process was developed in cooperation with Canadian Metal Rolling Mills of Cambridge, Ontario, by 1991.

The essential components of the Waterloo Barrier™ are shown in Figure 2. Barrier construction employs conventional sheet piling installation equipment. As described above, the unique feature of the barrier system is the sealable cavity at each joint. The configuration of the bottom of the cavity largely prevents pebbles and debris from entering the cavity as the piles are driven. Subsequent to installation of the barrier, soil that does enter the cavities is removed by jetting with water. Following this process, the integrity of the joints throughout their entire length can be assessed and any imperfections or blockages noted. This inspection process has been enhanced recently through the use of downhole camera techniques. Once the cleaning and inspection of the cavities has been completed, the sealant can be emplaced from bottom to top in each cavity.

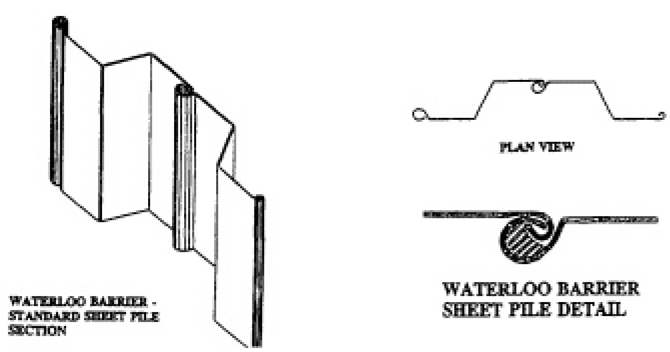

FIGURE 2 The Waterloo Barrier™ system, showing interlocking steelsheet piling and modified joint with the sealable cavity.

A variety of joint sealant materials can be used. Selection of sealant will be based on project requirements. The types of issues that may be considered in the sealant selection process may include sealant/contaminant compatibility, the presence of unusual ground water chemistry conditions such as high salt content, the ability of the sealant to withstand the anticipated differences in hydraulic head across the barrier, the amenity of the sealant to removal of the barrier system following a specified period, permeability characteristics, pumpability characteristics, thermal expansion characteristics of the sealant, design life of the system, and cost. The types of sealants available include clay-based grouts such as bentonite and attapulgite, cement-based grouts (modified with expanding agents), epoxy polymers, urethane polymers, and miscellaneous sealants such as vinyl esters, polysufides, swelling gaskets and bituminous grouts.

Hydraulic Testing

In excess of twenty test cells have been constructed for field research purposes, and several of these cells have been designed in a manner that facilitates rigorous hydraulic testing. Some of these cells have been constructed using concentric double walls, such that the hydraulic head in the moat bounded by the two walls can be maintained at a constant level. Further, these cells all penetrate to an underlying aquitard.

Figure 3, from Starr et al. (1992), shows a schematic diagram of such a cell. In this case, the cell extends through a surficial aquifer of approximately 12 m in thickness and terminates in a clay aquitard at a depth of approximately 14.7 m. The cavities were sealed with a bentonite slurry. Hydraulic testing was conducted by elevating the hydraulic head within the internal cell, maintaining a constant hydraulic head within the moat, and monitoring the decline in relative difference between the hydraulic head measurements with time. The plot of this decline, accounting for losses by evaporation, is shown in Figure 4, also from Starr et al. (1992). In applying the analytical solution, all water flux from the cell was attributed to leakage through the internal cell wall, and the clay aquitard was assumed to be impermeable. In reality, some leakage would have occurred through the aquitard at the base of the cell. This assumption aside, the bulk hydraulic conductivity of the cell wall was calculated to be 6×10^{-9} cm/s. Similar tests in other cells, one of which was sealed with an organic polymer sealant, have indicated that bulk hydraulic conductivities of less than 10^{-9} cm/s can be achieved.

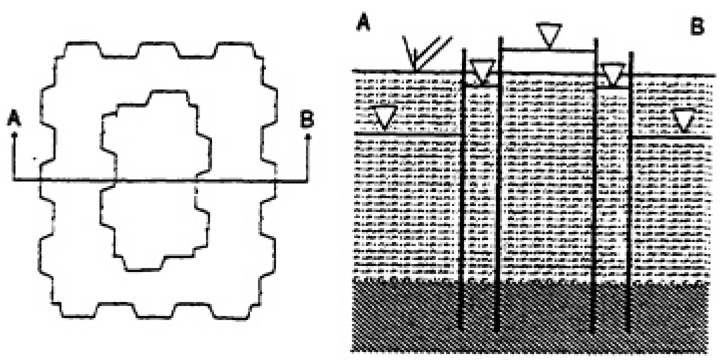

FIGURE 3 Plan and section view of test cell used to conduct hydraulic testing (after Starr et al., 1992). Figure 3 Plan and section view of test cell used to conduct hydraulic testing (after Starr et al., 1992).

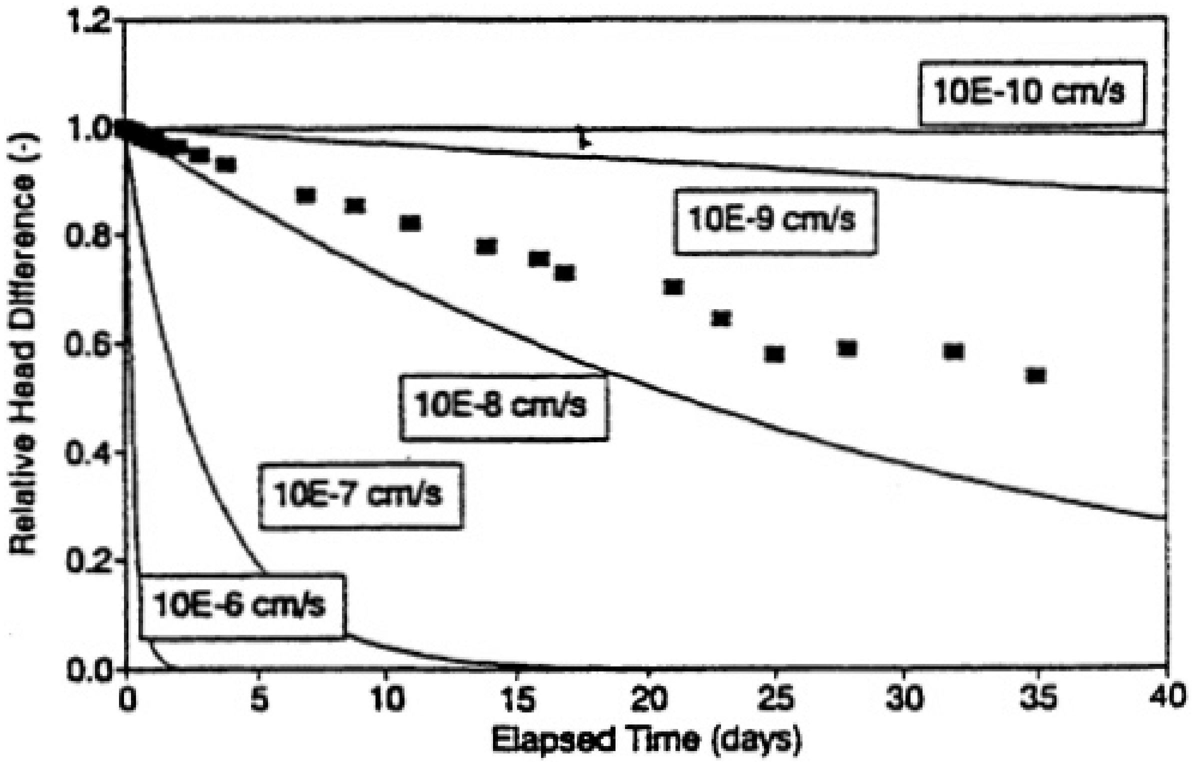

FIGURE 4 Observed response of representative hydraulic test oftest cell showing bulk hydraulic conductivity of barrier wall (after Starr et al., 1992).

Applications and Commercialization

Commercialization rights for Waterloo Barrier™are held under license from UW by Waterloo Barrier, Inc. of Rockwood, Ontario. A sub-license agreement for the production of the modified sheet piling has been made with Canadian Metal Rolling Mills (CMRM). CMRM currently produces a cold-rolled 7.5-mm (0.295-inch) section incorporating the modified cavity; a 9.5-mm (0.375-inch) section will be available by the last quarter of 1995. To date, three companies have been issued sub-licenses covering supervision of installation, joint sealing, and quality assurance/quality control measures. The companies include: C^3 Environmental of Breslau, Ontario; Slurry Systems, Inc., of Gary, Indiana; and RCI Environmental, Inc. of Kent, Washington.

Waterloo Barrier™ has been applied for contaminant-control purposes in more than twenty experimental cells as large as 10 × 10 m at several Canadian sites in association with the ground water research program at UW. The maximum depth of these applications is approximately 15 m, and the sealants used have included bentonite grout and an organic polymer.

Waterloo Barrier™ systems have also been installed at seven sites on a commercial basis. These installations have included test cells for trial applications of various remediation technologies at Hill Air Force Base (Utah) and Dover Air Force Base (Delaware); enclosures around contaminant source zones at industrial facilities in Toronto, Ontario, and in the Seattle area; a barrier system used in conjunction with a methane gas collection system at a municipal landfill in Kitchener, Ontario; and barrier walls used in conjunction with pump-and-treat systems at an industrial facility in Vermont and a military facility in Colorado. The size of the barrier systems have ranged from approximately 5,500 m (60,000 ft2) to 275 m^2 (3,000 ft2). The design depths of the installations have ranged from approximately 5 to 15 m. Various sealants have been used, including bentonite grout, cementitious bentonite and attapulgite grouts, and epoxy polymers. Rigorous joint-inspection procedures have been followed in all the projects, and sealant operations generally have proceeded without serious impediments. Only one of the commercial applications has been amenable to hydraulic testing, and in this instance, a bulk hydraulic conductivity for a barrier wall cell was estimated to be less than 10^{-8} cm/s. Overall costs for these projects have ranged from \$160.00 to \$430.00 per square meter (\$15.00 to \$40.00 per square foot) of barrier.

DISCUSSION

The field applications have confirmed several advantages of the Waterloo Barrier™ system, including:

- Clean and flexible installation. There are modest wastes generated during installation of the barrier system, thus problematic and potentially expensive issues associated with the handling or treatment of wastes are generally avoided.
- Site-specific custom design. The design and installation of the barrier system can, within reason, accommodate some unusual requirements arising as a consequence of buildings or facilities on site. In one application, a barrier was installed through the floor of an existing building and involved the sequential driving and welding of up to three vertical sections of pile.
- Detailed quality assessment/quality control (QA/QC). During pile driving and the joint flushing process, it has been possible to provide very detailed monitoring and inspection services. This has facilitated the preparation of excellent documentation records, which may be quite advantageous in assurance of compliance with regulatory requirements.
- The systems following installation look to be fundamentally sound. Based on the joint inspection and sealing activities, and hydraulic testing on enclosures where such testing has been feasible, statements regarding the expected hydraulic performance and integrity of the barrier system can be made with some confidence.

Like all engineered controls, installation of an effective barrier system using the Waterloo Barrier™ may not be the most appropriate selection of barrier system for all applications. It can be anticipated that Waterloo Barrier™ may have limitations in some circumstances including:

- The general depth and installation limitations associated with conventional sheet piling. In bouldery and rocky terrain, and in areas of dense unconsolidated sediments, the use of sheet piling will not be possible. Even in apparently appropriate media there will be limitations to the depth to which sheet piling can be installed. This depth will vary, but it is not unreasonable to assume that applications may be restricted to depths of less than 30-45 m or so. Although installation capabilities for sheet piling might be enhanced by using features such as water jets at the leading edge of the pile as driving occurs, or by resorting to measures such as pre-drilling along the footprint pattern of the barrier, these will add to project costs.

- Keying systems to bedrock underlying unconsolidated deposits. Although techniques have been developed for sealing the base of Waterloo Barrier™ system to underlying rock formations, special precautions will be necessary, and effectiveness of the sealing techniques may be difficult to confirm.
- **Vibration and noise associated with piling installation.** Although all construction may disrupt normal activities in the vicinity, the installation of sheet piling generates loud noises, and the level of vibration induced by pile driving may not be acceptable in some urban environments. The installation of sheet piling may result in some compaction and subsidence of adjacent soils, which also can be a concern. It is also worth noting, however, that all barrier construction techniques will have similar drawbacks associated with their implementation.

Based on development, testing, and application to date, the potential utility of Waterloo Barrier™systems in the control and containment of subsurface contamination has been demonstrated. The technology has been commercially available for only less than two years, so it is anticipated that further development will occur. It is also anticipated that the full capabilities, including the advantages and limitations of the technology, will become more clear. Additional experience is also necessary to better define the range of costs for projects involving application of Waterloo Barrier™ systems.

ACKNOWLEDGMENTS

The authors acknowledge the contributions of Sam Vales (UW), Robert Starr (UW), Jack Hammill (CMRM), and Cam Wood and Murray Gamble (C^3 Environmental) to the development of Waterloo Barrier™ technology. Research funds were provided by the University Consortium Solvents-in-Ground water Research Program, which has been sponsored by The Boeing Co, Ciba-Geigy, Dow, Eastman Kodak, General Electric, Laidlaw, Mitre Corporation, Motorola, PPG Industries, United Technologies Corporation, and the Canadian (NSERC) and Ontario (URIF) governments and the Ontario Environmental Technologies Program.

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ARTIFICIALLY FROZEN GROUND AS A SUBSURFACE BARRIER TECHNOLOGY

Steven A. Grant and Iskandar K. Iskandar, Cold Regions Research and Engineering Laboratory, U.S. Army, Hanover, New Hampshire

INTRODUCTION

Frozen water-saturated ground has great mechanical strength and very low-permeability. These properties are exploited by construction ground freezing, a mature civil engineering practice that has long been used in North America, Europe and Asia, usually to restrict ground water seepage or stabilize the walls during excavations. Construction ground freezing is often competitive economically with other barrier methods; its principle disadvantage is the long time it can take to form a frozen wall. Recently, artificial ground freezing has been used to form barriers at contaminated sites. The principle advantages of artificial ground freezing for this application, which we designate here as "environmental ground freezing," are thought to be its ability to form impermeable barriers across transitions from soil to bedrock, the minimal amount of solid material brought to or taken from the site during barrier construction or decommissioning, and the great flexibility in designing or adjusting the barrier's size and shape (Iskandar and Jenkins, 1985). Some believe that, as a natural process, soil freezing is likely to find public and regulatory acceptance, although the technique has been implemented at too few contaminated sites to judge this belief.

PHYSICS, CHEMISTRY AND MECHANICS OF FROZEN GROUND

Frozen ground is composed of four phases: gas (often as entrapped bubbles), liquid aqueous solution, solid ice, and solid mineral matrix. A schematic of this assemblage is presented in Figure 1. The amount of liquid aqueous solution remaining at a particular temperature is determined by two physical-chemical phenomena: interfacial forces caused by the distortion of the ice-solution interface as it is forced to adjust itself to the geometry of the ground's mineral matrix; and colligative properties of the water as a solvent, that is, the freezing-point depression of the pore solution. Of the two, interfacial forces are the more important in determining the amount of liquid aqueous solution persisting at subzero temperatures in ground. As would be expected, the larger the specific surface area of the minerals, the larger the fraction of water remaining unfrozen at subzero temperatures. Figure 2 presents the calculated volumetric liquid water content of a water-saturated Royal sandy loam as a function of temperature. Due to freezing-point depression, the presences of solutes in the pore solutions will cause the amount of liquid present at a given subzero temperature to be increased. This can become an important consideration when ground saturated with seawater is being frozen.

The amount of liquid water remaining in frozen ground is not solely of academic interest. In many cold regions, it is this liquid fraction that wicks water from deeper in the soil profile to the ground's freezing front, causing frost heaves--a serious engineering problem. Liquid water contents of frozen ground affect the ground's strength, its permeability, and the diffusion rates of solutes in it. All of these could be factors in determining the suitability of environmental ground freezing at a given site.

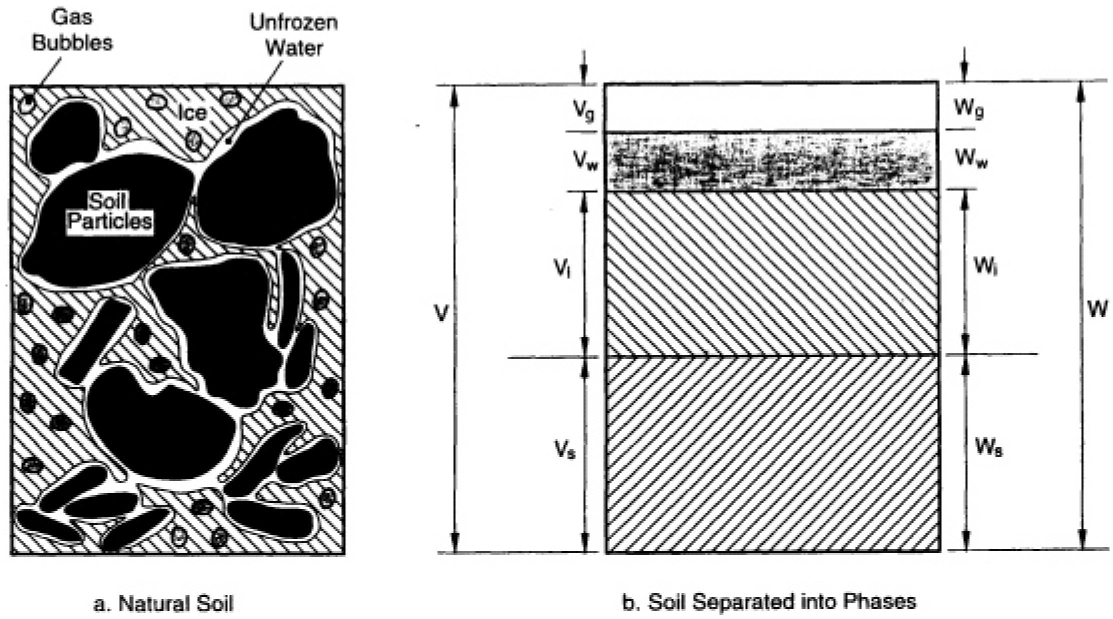

FIGURE 1 Schematic drawings showing the phase distribution in frozen ground.

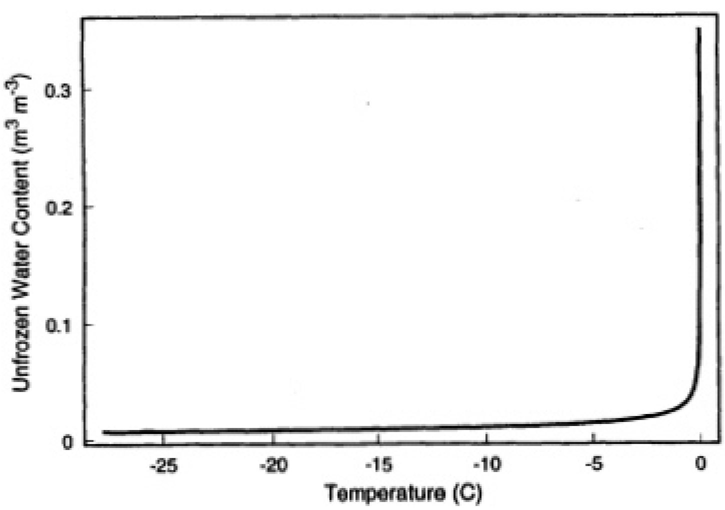

FIGURE 2 Calculated relationship between temperature and liquidwater in a frozen Royal sandy loam.

Mechanical Properties

The strength of frozen ground is due to the strengths and volumetric fractions of its constituents: ice, mineral grains, and liquid water. By most measures, frozen ground can be a very strong material. The measured unconfined compressive strength of frozen Ottawa sand at -50°C was 40 MPa (Sayles, 1988). The corresponding strengths of competent granite without macroscopic flaws are between 150 and 250 MPa. The resistances of frozen ground to load and deformation are affected by its mineral composition, water content, temperature, previous loading, and rate of loading. The effect on the proportion of water is presented in Figure 3. As a general rule, the resistance of frozen ground to load and deformation increases with decreasing temperature but decreases with time. These two tendencies are shown in Figure 4, which is reproduced from Sayles (1988).

Permeability

The permeability of frozen ground can be estimated based on the ground's degree of saturation by liquid pore solutions. Figure 5 presents the calculated permeabilities of a Royal sandy loam as a function of temperature.

ARTIFICIALLY FROZEN GROUND AS AN ENGINEERING TECHNOLOGY

Many, including practicing civil engineers, are unaware of construction ground freezing, yet it is hardly a secret. Perhaps the most famous structure to be affected by construction ground freezing is the tower in Pisa, Italy, the foundation of which is being stabilized by the technique while a more permanent solution to its inclination is being constructed. Perhaps the largest construction project to use construction ground freezing was the Tokyo Metro. A total volume of 38,000 m^3 of material was excavated (Jessberger, 1980).

In 1883, the technique was patented by F.H. Poetsch in Germany as a mine-shaft construction method. While improvements have been made, the techniques used today are essentially those of Poetsch. Energy is removed from the ground by circulating cold liquids through pipes drilled into the ground. The two refrigeration approaches are: using a mechanical refrigeration plant through which a heat-transfer fluid is circulated to freeze pipes, or pumping expendable cryogenic liquids through the freeze pipes. Freeze pipes, which consist of two nested pipes for supply and return of the heat-transfer fluid (usually $CaCl_2$ or $MgCl_2$ brines) or cryogenic liquid (typically liquid nitrogen), are drilled into the ground at regular intervals, typically 1 m. In principle, any soil can be frozen. Shuster (1980) noted that successful implementation of ground freezing may be limited as follows:

1. The degree of saturation of the ground by water must be high enough to bind the soil grains when frozen. Generally, fully water-saturated ground is preferred.
2. The concentrations of solutes in pore water solution should not be too high. This is a consideration, for example, when freezing marine sediments. The melting point of sea ice is lower and its strength less than that of fresh-water ice. (There have been instances of catastrophic failures

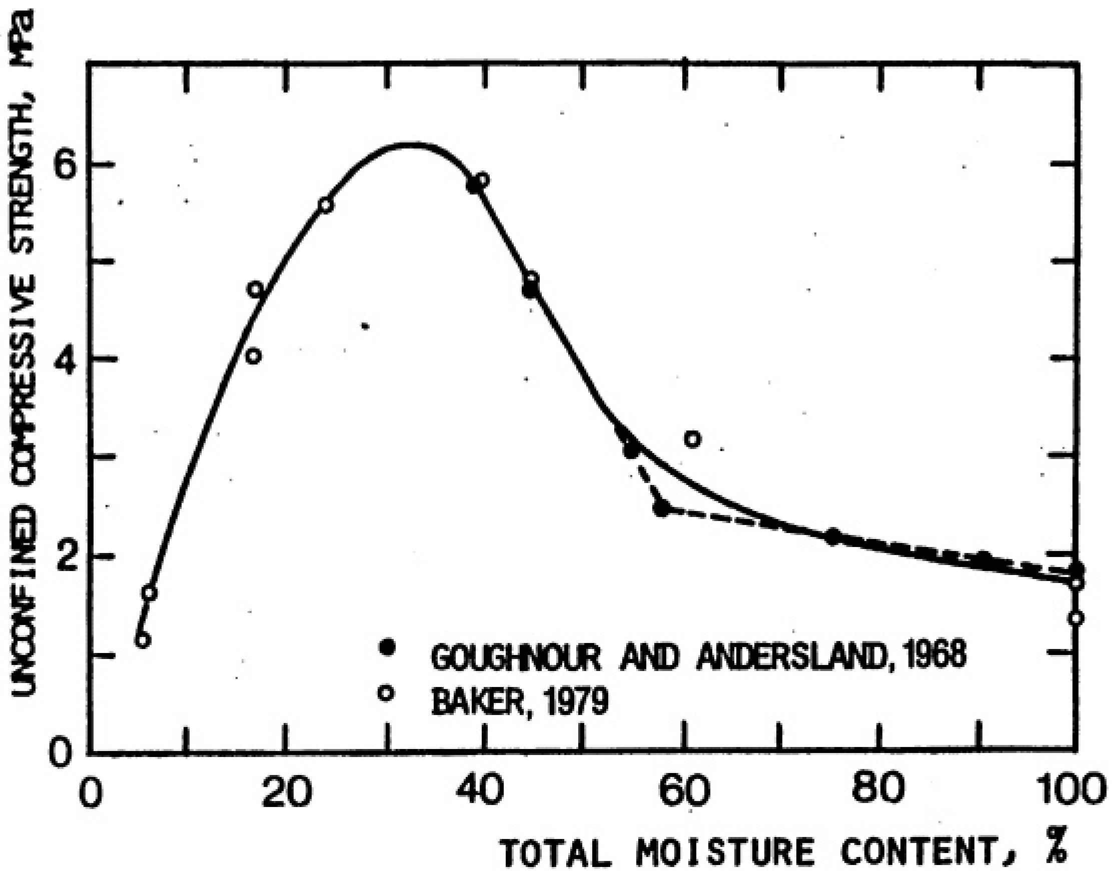

FIGURE 3 Effect of total moisture content on unconfined compressive strength of a frozen fine sand at -12ºC and at a strain rate of 2.2 × 10^{-6} s^{-1} (from Andersland and Ladanyi, 1994).

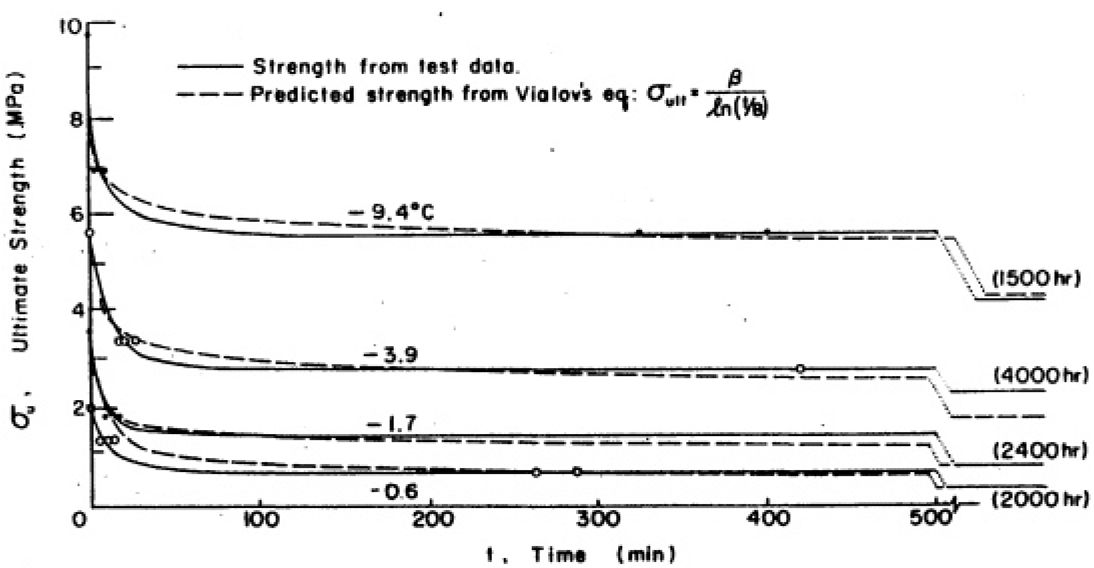

FIGURE 4 Calculated relationship between temperature and hydraulic conductivity of a frozen Royal sandy loam.

of frozen walls formed in salt-water-saturated ground [E.J. Chamberlain, personal communication, 1994]).

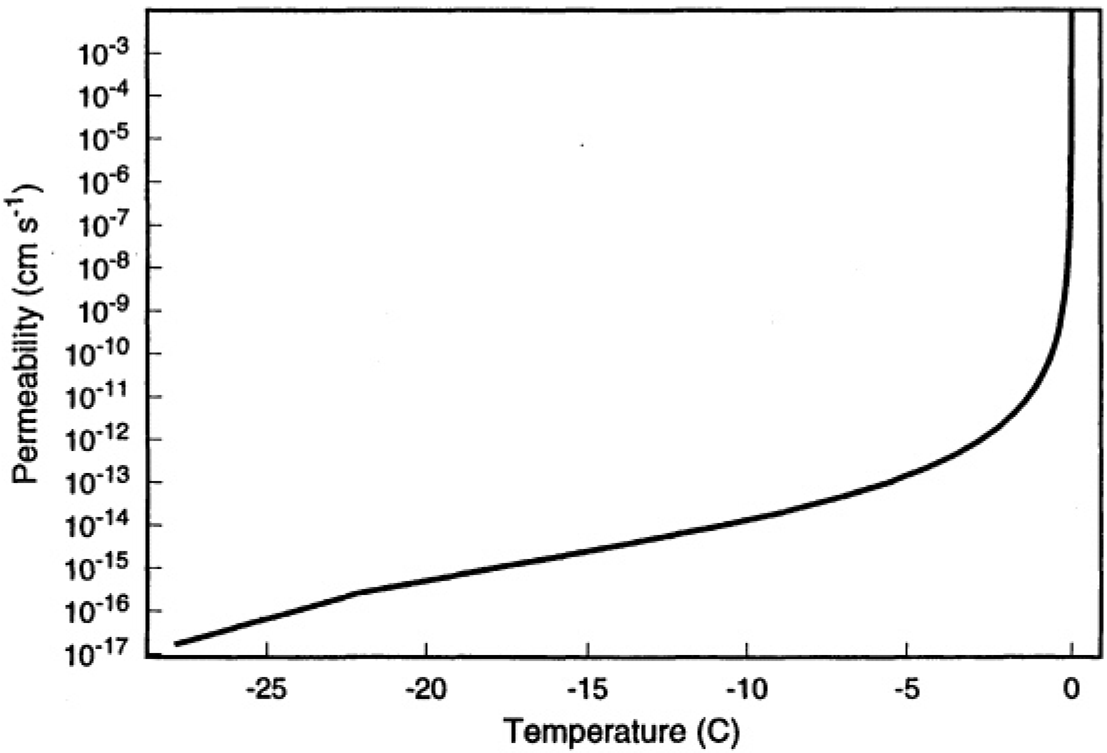

FIGURE *5* Calculated relationship between temperature and hydraulic conductivity of a frozen Royal sandy loam.

3. Ground water seepage does not bring to the forming barrier more energy than can be removed by the refrigeration plant. Calculations have shown that flows should not be greater than 2 m/day. The ground has comparatively high heat capacity and low thermal conductivity (Sullivan and Stefanov, 1990). Once formed, a frozen ground barrier effectively eliminates energy loss due to convection and can be maintained indefinitely with low power consumption.

There is a small, but stable market for construction ground-freezing services; four firms in the United States provide the service.

ARTIFICIALLY FROZEN GROUND AS A WASTE-CONTAINMENT TECHNOLOGY

We are aware of two contaminated sites at which artificial ground freezing is being used for waste containment. The first is a National Priority List (NPL) site in the eastern United States situated next to a river. Frozen ground walls are being formed in conjunction with sheet-pile walls to limit the migration of contaminants off-site to the river. This site will be excavated after the walls are formed. The second site is a petroleum-contaminated site on permafrost in Alaska. Here a frozen ground barrier has been formed to prevent snowmelt-runoff driven horizontal flows of dissolved petroleum products as the active layer above the permafrost thaws.

Ground freezing has been proposed, though not as yet adopted, at two DOE reservations: Oak Ridge and Hanford. A successful field trial was conducted at Oak Ridge in 1994. The soils of the Solid Waste Storage Areas within the Oak Ridge Reservation (ORR) are fairly shallow (3-9 feet), weakly developed, and overlay weathered rock (saprolite). Environmental ground freezing is seen as having two potential advantages at ORR. First, the barrier can be formed continuously across the transition from soil to the underlying saprolite. Second, when compared with other methods, less earth will be excavated in installing a frozen ground barrier. Environmental ground freezing has also been proposed at the Hanford Reservation, although an unrealistically long time may be needed to develop a reliable technique to saturate and cool an arid soil simultaneously, without water loss. Results of bench-scale experiments, however, are encouraging (Andersland et al., 1994; Dash et al., 1994).

SCREENING MATRIX

Development Status

Construction ground freezing is routine, but environmental ground freezing is in its infancy. The firms that participated in the environmental ground freezing projects and demonstrations have been able to meet the design criteria without difficulty. A principle difficulty appears to be that there is no in-situ sensing technology for continual assessment of the integrity of the barrier. As with other barrier technologies, sensing leaks from beneath the barrier are problematic.

Availability

As noted above, a limited number of firms provide this service, which is as much an art as a construction practice. Were the demand for environmental ground freezing to increase markedly, experienced practitioners could be fully booked. This appears to be a hypothetical, not imminent, problem.

Implementability

At sites that meet the criteria for successful implementations of construction ground freezing, one would expect that environmental ground freezing could be implemented straightforwardly.

Reliability/Maintainability

Based on experience with construction ground freezing, the technology should be reliable and easily maintained when installed professionally at an appropriate site.

Secondary Waste Produced

Intrinsically, very little secondary waste is produced with environmental ground freezing. Since the method typically relies on water in the ground, only that material brought to the surface for drilling holes for freeze pipes may constitute a secondary waste. Once refrigeration is stopped, the ground returns largely to its original condition, although frozen ground may consolidate while thawing.

Implementation Costs

These tend to be high but are competitive with other barrier technologies (Sullivan, Lynch, and Iskandar, 1984).

Operating Costs

Although dependent on the intensity of on-site monitoring, these costs should be modest.

Regulatory Acceptance

As an organization that supports environmental research, the U.S. Environmental Protection Agency (EPA) has funded some studies and demonstration projects, but it has not indicated how readily it will accept environmental ground freezing.

Public Acceptance

Environmental ground freezing has been implemented at very few sites. To our knowledge, no studies of public acceptance have been made. It is assumed that environmental ground freezing should not incite public resistance since it is an adaptation of a natural process.

Natural Resources Impact

Extended periods at subzero temperatures would kill many of the flora and fauna remaining in the frozen ground barrier. As noted above, forming the frozen ground barrier takes roughly a month; presumably, most of the burrowing animals would vacate the barrier as it cools. Rapid rehabitation from the surrounds after decommissioning of the barrier would be anticipated.

Risk-Management Reduction

It is expected that the risk-management effects of environmental ground freezing would be on par with other barrier techniques.

CONCLUDING REMARKS

Environmental ground freezing has long been a promising waste isolation technology but one that was all too rarely fielded (Bovay Northwest, Inc., 1992). Recent evaluations and implementations at contaminated sites may yield a sufficient body of experience so that the technique may be implemented more often.

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