SECTION 1

INTRODUCTION

Although investigations and efforts to remediate contaminated subsurface zones have been underway in the United States and elsewhere for some two decades, it has become evident that cost-effective clean-up at many sites can not yet be achieved. Factors contributing to this situation include the dearth of cost-effective technologies for *in situ* remediation and the relative ineffectiveness of pump-and-treat approaches at sites where dense nonaqueous phase liquids are present. In view of this, other approaches for controlling subsurface contaminant zones need to be investigated and evaluated.

In 1992, under the guidance of Calvin C. Chien, leader of the Containment and Transport Modeling (C&TM) team, one of five environmental remediation technology application teams established within the DuPont Corporate Remediation Group, an internal study was made of the current status and applicability of barrier containment technologies in remediation. A major internal summary report resulted from that study. Following that internal study, a more exhaustive external investigation was undertaken to further assess the state of the art of containment technologies. This external investigation was conducted under the auspices of the New York State Center for Hazardous Waste Management, located at the State University of New York at Buffalo, with the participation of Calvin C. Chien, Terry D. Vandell, and Richard Landis, all members of the C&TM team. Andrew Bodocsi, David E. Daniel, Jeffrey C. Evans, and James K. Mitchell were invited by DuPont to participate in the investigation. That external investigation resulted in a report to DuPont in 1993. That report was subsequently published (in its entirety) in book form by John Wiley & Sons, Inc. [Barrier Containment Technologies for Environmental Remediation Applications, edited by Rumer and Ryan, 1995].

From the outset of its investigation into containment technologies, the DuPont C&TM team had planned to organize an international workshop as a follow-up to the external report prepared under the auspices of the New York State Center for Hazardous Waste Management [Rumer and Ryan, 1995]. The primary purpose of the planned international workshop was to continue to address the gap between what was known and understood about contamination containment technologies and the level of information needed to support consistent decision making relative to their application in remediation. This was to be accomplished by convening a larger group of experts from academia, industry, and government agencies in the U.S. and abroad to conduct a careful review and discussion of the applicability and reliability of these technologies and to identify information, research, and development needs. Because of mutual interests and similar needs in this area, the U. S. Department of Energy and the U. S. Environmental Protection Agency collaborated with DuPont in organizing and sponsoring the International Containment Technology Workshop, held in Baltimore, Maryland, August 29 - 31, 1995. This publication is a product of that workshop.

Over 100 workshop participants from eight countries were invited on the basis of the knowledge, experience, and perspective each could bring to the discussions concerning the status of containment technologies. Participants were invited from a wide spectrum of endeavors so as to broaden the perspective as well as to facilitate the achievement of the workshop objectives.

The workshop was organized into small working sessions, each dealing with a particular aspect of barrier containment technologies, to provide opportunity for the invited participants to share information, exchange opinions, and achieve consensus on what was known, what was not known, and what was needed concerning the application of containment technologies in remediation. The working sessions dealt with a range of topics, including: design and construction of vertical barrier walls (including sheet piles), barrier floors (indigenous and artificial), caps, geomembrane applications, barrier materials (soil-based and chemical-based), permeable reactive barriers, contaminant transport modeling, performance monitoring, and emplacement verification. Each working session was chaired by a recognized leader(s) in the field who was responsible for preparing the summary report for his or her working session.

This publication contains the edited summary reports from each working session. Although particular individuals have been credited with the preparation of each section, other significant contributors are also identified. It is virtually impossible to acknowledge the specific contributions of all participants attending the workshop, and this is regrettable. However, a full listing of all participants can be found in an appendix to this publication.

Each section of this publication has been reviewed for technical content by James K. Mitchell, Virginia Polytechnic Institute and State University. The initial compilation of the draft report and the final editing were carried out by Ralph R. Rumer, State University of New York at Buffalo. The workshop session leaders participated throughout the editing process.

The responsibility for planning, organizing, and conducting the workshop rested in the hands of an executive committee, co-chaired by: Grover H. (Skip) Chamberlain, Office of Technology Development, U. S. Department of Energy; Randall Breeden, Office of Emergency and Remedial Response, U.S. Environmental Protection Agency; and Calvin C. Chien, C&TM team leader, Corporate Remediation Group, DuPont Company. To continue to provide a forum for future discussions on advancements and experience gained in the application of containment technologies, the three sponsors decided to organize future biannual international conferences, with the first one scheduled for February 1997.

Acknowledgments

The workshop sponsors and all of the contributors to this book express their gratitude to Phyllis Adams, DuPont Co., for her very able assistance throughout the planning and execution of the workshop. The artwork was prepared by Barbara E. Evans, Art and Photographic Services, State University of New York at Buffalo. The design and preparation of this publication was carried out by Sue A. O'Donnell, Peripheral Vision, Buffalo, NY. The index was written by Kathryn W. Torgeson, Ithaca, NY. Loreen Kollar, Florida State University [under a cooperative agreement with the U. S. Department of Energy], handled the travel arrangements for the workshop attendees, assisted at the workshop, and arranged for the printing of this publication.

SECTION 2

SOIL- AND CEMENT-BASED VERTICAL BARRIERS WITH FOCUS ON MATERIALS

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2.1 INTRODUCTION

The soil and cement materials used to construct vertical barriers that include; 1) externally mixed barriers, such as soil-bentonite slurry trench cutoff walls, and 2) *in situ* mixed walls, using deep soil mixing and grouting techniques, are discussed in this section. Slurry trench cutoff walls composed of soil-bentonite, cement-bentonite, plastic concrete, and recent innovations (both proposed and implemented) such as attapulgite based barriers, bioenhanced barriers, and other mineral enhanced barriers are considered. The differences between European and U.S. practices are described. Barrier construction techniques are not discussed, except as necessary to understand differences in barrier material performance. Regardless of construction method, the focus is on the barrier materials used and their properties, as determined by both long and short term performance measures.

Soil-bentonite slurry trench cutoff walls are the most widely used type in the U.S. Their application in remediation is reviewed in Rumer and Ryan (1995). For this reason, they are not given detailed attention here. The construction and performance of soil-bentonite slurry trench cutoff walls are well understood and they are usually considered reliable when used as subsurface vertical barriers for containment.

2.2 BACKGROUND

At virtually all remediation sites, vertical barriers are needed as a component of a constructed containment system. Typically, vertical barriers are used in conjunction with some form of pump and treat remediation. The vast majority of vertical barriers are currently constructed as either soil- or cement-based walls. In the U.S., the vertical barrier of choice is usually a soil-bentonite slurry trench cutoff wall. In Europe, the choice is usually a cement-bentonite slurry trench cutoff wall, often incorporating slag or fly ash in the slurry mix. The experience with soil- and cement-based vertical barriers is extensive and long-term. There is also growing experience with geosynthetics in/as vertical barriers.

Monolithic grout and soil cement barriers may be considered for containment applications below the water table, in the vadose zone, and at arid sites where bentonite-based materials are at risk of desiccation and cracking. Many sites controlled by the U.S. Department of Energy (DOE) are located in such environments. Contaminants of concern at these sites include chemical, radioactive, and mixed wastes. Cementitious barriers are also applicable under saturated conditions.

Subsurface barriers are usually required to exhibit low hydraulic conductivity (<10-7 cm/s), be durable, and have adequate strength. Installed barrier walls may be exposed to leachates, contaminants, and aggressive chemical species that naturally occur in ground water (e.g., high soluble sulfates). Under these conditions, the properties of grout and soil cement barriers can be enhanced through selection of appropriate admixtures .

2.3 STATE OF PRACTICE

The methods used to construct soil- and cement-based vertical barrier walls are well known and well documented. Excavation with clamshells and backhoes, using bentonite-water slurry to maintain trench stability, is well understood and has been widely employed. These excavation techniques allow for the placement of soil-bentonite backfill within the slurry filled trench to form the soil-bentonite slurry trench cutoff wall. Alternatively, the backhoe and clamshell excavation techniques can use a slurry of bentonite-water-cement to maintain trench stability. This slurry can be left to harden in place to form a cement-bentonite slurry trench cutoff wall. Soil- and cement based barrier materials have also been used to construct vertical barriers using *in situ* excavation/placement techniques, such as deep soil mixing and grouting (permeation and jet).

The slurry mix proportions critically affect the short and long term performance of the completed cutoff wall. For soil-bentonite slurry trench cutoff walls, the optimum mix is composed of a well-graded material containing sand and gravel along with 20% to 50% fines of low plasticity/

activity. The mixture typically has a bentonite content of approximately 1%. This bentonite is added to the backfill in the form of bentonite-water slurry used to achieve workability (slump). Although, supplementing the mix with large quantities of bentonite can reduce the initial hydraulic conductivity, mixes with larger proportions of bentonite can be more susceptible to degradation because of incompatibility between the mix and the surrounding contaminated ground water. Non-aqueous phase liquid contaminants pose a particular problem in this regard.

In the U.S., cement-bentonite slurries are typically composed of water, 5% bentonite, and 15% to 25% cement. The resulting hydraulic conductivity is on the order of 10^{-5} to 10^{-6} cm/s. In Europe, slag or fly ash is frequently added to the mix. These latter mixes have a hydraulic conductivity typically less than 10^{-7} cm/s. Cement-water grouts or soil-cement mixtures can achieve a hydraulic conductivity as low as 10^{-10} cm/s by the inclusion of superplasticizers. Water/cement ratio is one of the major factors that determine the final properties of cementitious barrier materials.

2.3.1 Material Choices as Influenced by Construction Method

The cementitious grouting materials chosen for use in a vertical barrier must be compatible with currently available placement techniques. Since numerous techniques (e.g., permeation grouting, jet grouting, soil mixing, etc.) are available, the choice depends on the properties of the site soils, the consistency required for placement by injection or mixing, the required properties of the barrier, and the installed cost of the final product. Also, the grouting material chosen must be non-toxic and acceptable to the Environmental Protection Agency (EPA) and other regulatory agencies. Portland cement-based grouts are generally acceptable. However, some commercially available chemical grouts such as polyacrylamide, polyacrylic acid, phenol-formaldehyde, and urea-formaldehyde may not meet with EPA or other regulatory agency acceptance (Kukacka, 1995).

The mechanical and physical properties of the uncured and cured grouting materials are important. In their uncured state, the materials must exhibit stability and have viscosity compatible with the placement technique to be employed. Bleeding of water and settling of particles should not occur. For a solid grout barrier, the flow properties of the grout should enable it to completely fill the required cavity.

The set time for grouts must be long enough to permit placement and equipment clean up. Long set times are not as critical, unless continuation of work depends on solidification of previously placed material. A period of 12 to 24 hr is generally acceptable. However, placement costs may increase as the cure time is extended.

After curing, the solidified *in situ* barrier materials must exhibit low hydraulic conductivity, low shrinkage, adequate strength, and long-term durability. The strength requirements are typically minimal, since lateral soil pressures are frequently equal on both sides of a vertical barrier wall. Therefore, the design thickness of the barriers is normally controlled by hydraulic conductivity requirements, rather than strength. However, strength and compressibility considerations can be important when other structures are present or in seismically active regions.

Low hydraulic conductivities are needed in vertical barrier walls to control the migration of contaminants through the barriers by advection. EPA 40 Code of Federal Regulations, Part 264, Subpart N (landfills) stipulates that containment barriers for waste impoundments must have hydraulic conductivities less than 10^{-7} cm/s.

Durability of the barrier is essential throughout its service life. Although service life times have not been well defined, a few hundred to a thousand years may be needed in the case of radioactive waste containment. Therefore, the placed barrier material must be stable, and retain the desired mechanical and physical properties for long periods of time.

The method used to mix the formulation before placement may be an important consideration. For example, attapulgite clay cannot be mixed in ponds because it settles too quickly. Both attapulgite and guar gum, used in biopolymer trenches, require high energy mixing for adequate dispersion.

a. Permeation Grouting Materials. Permeation grouting refers to the method of construction by which a barrier is constructed by pumping a grout into the ground to fill the pores and voids of the subsurface soil and rock matrix (Rumer and Ryan, 1995). The grout is often a cementitious material and, once hardened in the voids, seepage and contaminant transport are greatly reduced. For environmental remediation applications, permeation grouting is used primarily in fractured rock stratigraphies, while slurry walls tend to be used for sites underlain by excavatable soils (Karol, 1983; Hausmann, 1990; Xanthakos, et al., 1994).

Use of slag in treatment grouts can contribute benefits such as resistance to sulfate attack. The heat generated during grout hardening may cause cracking, and should be considered. If heat dissipation is inhibited by the surrounding soil, excessive thermal stresses may develop. The addition of slag and optimization of the soil/grout ratio can reduce the risk of thermal cracking.

b. Jet Grouting Materials. Jet grouting employs a very high pressure rotating jet to construct a subsurface barrier. The grout, often a cementitious material, mixes with the native soil materials to form a soil-cement column of low hydraulic conductivity. These columns are constructed sequentially in an overlapped fashion to form the subsurface barrier wall (Xanthakos, 1994; Rumer and Ryan, 1995). For environmental remediation applications, jet grouting can be considered for construction of an artificial bottom barrier, or for a vertical barrier wall when trench excavation is not possible.

c. Deep Soil Mixing Materials. Deep soil mixing is accomplished using a special auger mixing shaft that is rotated into the ground while simultaneously injecting either a bentonite and water slurry or a cement, bentonite and water slurry (Rumer and Ryan, 1995). The advantages of deep soil mixing include reduction in the volume of soil to be disposed, greater stability during construction, structural capability of the completed column, and the ability to carry out soil vapor extraction during the construction process (Day and Ryan, 1995). Sampling of cementitious barrier materials, such as those resulting from the deep soil mixing process, can produce unrealistically high values of hydraulic conductivity when tested. This has been attributed to sample disturbance and is discussed further in Section 3.

Polypropylene fibers can be added to mixes (0.2 % volume fraction) used for producing soil cements by soil mixing, bringing about improved flexural strength and ductility and reduced widths of potential shrinkage cracks. Resistance to wet-dry cycling can also be improved by the addition of fibrillated fibers.

d. Summary of Material Choices for Grouted Barriers. Cementitious grouts selected from mixtures of Portland cement, sand, bentonite, and various additives are excellent choices for barrier wall formulations when using *in situ* placement techniques. These mixtures are low cost compared to chemical grouting materials, compatible with commercially available placement technologies, nontoxic, and, upon curing, yield strong, durable and low hydraulic conductivity products. The cements can be used as bulk materials or can be mixed with on-site soils at ratios up to 5 parts soil to 1 part cement. Hydraulic conductivities in the range of 10^{-10} cm/s may be attainable by adding superplasticizers.

2.3.2 Properties of Cementitious Grouts

In its simplest form, cementitious grout consists of ordinary Portland (ASTM Type I) cement and water. The properties of cementitious grout are controlled by the water/cement ratio, cement type, addition of bentonite or attapulgite clay, partial replacement of cement with mineral admixtures (supplementary cementing materials), addition of retarders or accelerators, and use of water reducing agents. Grouts also may be modified with latex. Sand may be used as a filler material in some situations. Each of these additives has a distinct influence on the properties of the unhardened (uncured) and hardened (cured) grout.

a. Properties of Cementitious Grout in the Unhardened State. The flow behavior of the unhardened grout is critical, since it determines the ability to mix, pump and place the grout using conventional equipment. The important flow properties include; viscosity, gel strength, bleeding, and set time. Specification of flow property values will depend on the placement technique to be used and site conditions.

Viscosity. In addition to the strong influence of the water/cement ratio, the viscosity of grout can be altered using chemical admixtures. Water reducing and superplasticizing agents can be used to minimize the water demand while still maintaining low viscosity (see ASTM C 494 and ASTM C 1017). Reductions in the water/cement ratio are beneficial for properties such as strength, hydraulic conductivity, and durability. Superplasticizers are generally more effective than regular water reducers.

The effectiveness of superplasticizers decreases with mixing time, and the rate of superplasticizer addition can exert a significant effect on grout rheology. An example of this effect is shown in Figure 2-1 (Allan, 1995) which presents a plot of shear stress versus shear rate for a Type I cement grout with a water/cement ratio of 0.4. When superplasticizer was added last, the yield stress decreased at a given shear rate as compared to adding the superplasticizer to the water before mixing. Cement type will also influence grout viscosity. For example, at a given water/cement ratio, a microfine (ultra fine) cement produces a higher viscosity than would be produced using coarser cement.

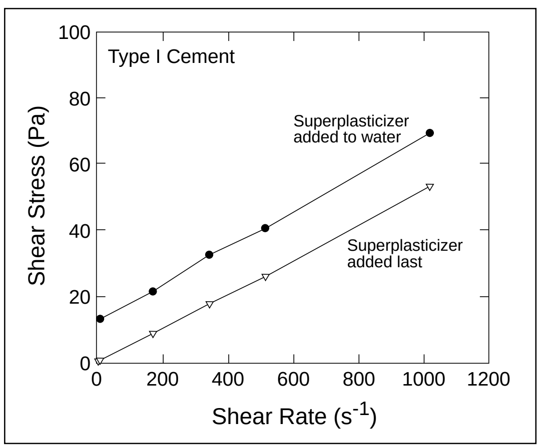

Figure 2-1 Effect of sequence of superplasticizer addition on shear stress-shear rate relationship of Type I cement grout (Allan, 1995)

The most common mineral admixtures used with cement grout include; fly ash (ASTM C 618), silica fume (micro silica), and mechanically granulated blast furnace slag (ASTM C 989). These materials, when used as partial replacements for cement, can be beneficial by reducing heat of hydration and improving durability. Silica fume has a strong effect on rheological properties, while fly ash and slag have less effect. The effect of silica fume replacement (for cement) on the shear stress/shear rate relationship is shown in Figure 2-2 (Allan, 1995). The apparent viscosity at a given shear rate is significantly increased, even at relatively low replacement levels. Thus, the practical cement replacement level with silica fume is limited to 5-10%. Higher replacement levels can be used with fly ash or slag, with less dramatic effects on rheology. This is illustrated in Figure 2-3 (Allan, 1995) for 60% replacement of Type I cement with slag. The type of fly ash used (i.e., pozzolanic or nonpozzolanic) will also influence rheological properties.

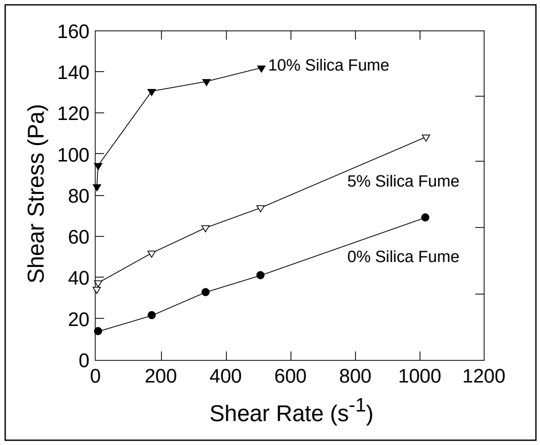

Figure 2-2 Effect of partial replacement of Type I cement with silica fume on shear stress-shear rate relationship (Allan, 1995)

Bentonite is a common cementitious grout additive, used primarily to improve grout stability and reduce bleeding. Mix viscosity increases with increasing bentonite content. Superplasticizers are not effective with high bentonite content grouts. The addition of latex to grout also increases viscosity. Other factors that affect the viscosity of cementitious grouts include; shear and thermal history, temperature, and mixing equipment used.

Gel Strength. When the application of stress ceases and the grout is quiescent, the liquid grout begins to transform to a colloidal gel. Gel strength is a measure of the stress required to re-initiate flow after the grout has been at rest.

The addition of silica fume and bentonite increases gel strength significantly. In contrast, adding slag and fly ash tends to reduce gel strength. Adding superplasticizers also reduces gel strength, the effect again being related to the sequence of addition. By adding superplasticizer after cement, rather than with the mixing water, the gel strength can decrease significantly. Incorporation of latex in grout decreases gel strength.

Bleed. Excessive bleeding (the separation of water from solids) and segregation of grout are undesirable. The most commonly used additive to control bleeding and segregation is bentonite. Silica fume is also highly beneficial in reduction of bleeding. Superplasticizers can act to increase bleed, particularly if overdosed.

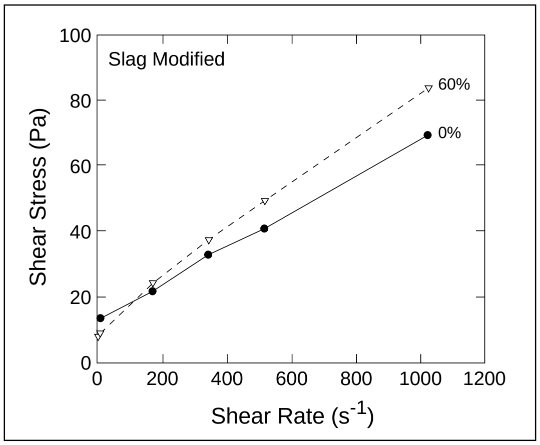

Figure 2-3 Effect of 60% replacement of Type I cement with blast furnace slag on shear stress-shear rate relationship (Allan, 1995)

Set Time. Set time is strongly dependent on temperature, with higher temperatures accelerating the set. Accelerators or retarders can be added to counteract adverse temperature conditions. Common retarders include borax, sugars, and calcium lignosulphonate; while calcium chloride and sodium silicate are common accelerators. Combination types of water reducers-retarders or water reducers-accelerators are also available (refer to ASTM C 494).

Admixtures, used to control other properties, may indirectly affect set time. For example, superplasticizers act as retardants. Fly ash and slag also tend to increase set time. Since it is usually necessary to combine a superplasticizer with silica fume, the set time for such grout mixtures may be retarded. High proportions of bentonite increase set time.

When grout is mixed with soil, the set time may be altered, depending on the soil chemistry. Many contaminants, particularly organics, increase the set time of cement-based materials. Other contaminants may accelerate set. Therefore, if cementitious grout is to be mixed with a contaminated soil, it is essential that the potential for interference with cement hydration and setting be investigated.

b. Properties of Cementitious Grout in the Hardened State. The physical and mechanical properties of hardened grout and grout treated soil (soil and cement) depends on other factors, in addition to the presence of admixtures. Some of these include; mix proportions, curing conditions, age of the hardened grout, temperature, soil type, and presence of contaminants. Mix proportions (i.e. grout to soil ratio) of soil and cement mixtures produced by jet grouting or deep soil mixing can be controlled by the system parameters.

Hydraulic Conductivity. Water/cement ratio is the most significant parameter controlling the properties of hardened cementitious grouts. By reducing water/cement ratio through use of superplasticizers, the hydraulic conductivity can be reduced. The hydraulic conductivity increases with decreasing grout content or with increasing water/cement ratio. This effect on hydraulic conductivity for soil cements produced from superplasticized grout with simulated *in situ* curing is illustrated in Figure 2-4 (Allan, 1995). The results are for laboratory prepared specimens; thus, some deviation from field-placed materials is expected. However, the trends for the different variables are considered relevant.

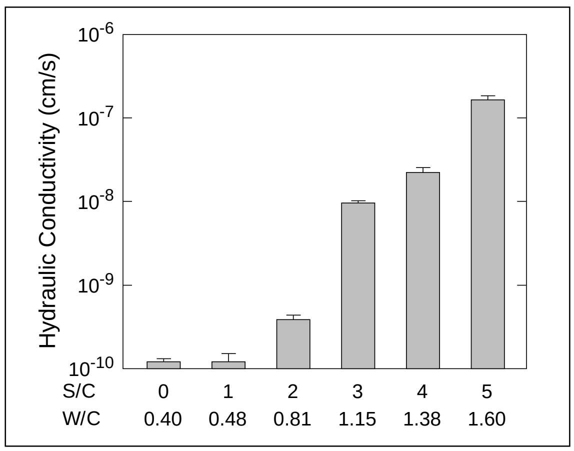

Figure 2-4 Effect of soil/cement (s/c) and water/cement (w/c) ratios on hydraulic conductivity of *in situ* cured soil cements containing superplasticizers (Allan, 1995)

Partial replacement of cement with silica fume, fly ash, or slag generally reduces the hydraulic conductivity of grout, assuming the water/cementitious material ratio remains constant. The use of sulfate resistant cement may also be beneficial in reducing hydraulic conductivity. Figure 2-5 (Allan, 1995) depicts the effect of slag content, water/cementitious material ratio, and soil/cementitious material ratio on the hydraulic conductivity for superplasticized samples made from soil and cement under simulated *in situ* curing conditions in a vadose zone (i.e. dry curing).

Latex reduces the hydraulic conductivity of grouts due to pore blockage by the latex film. Findings for samples made from soil and cement produced using latex-modified grouts have been inconsistent. In general, the hydraulic conductivity of samples made from soil and cement is reduced by latex, but not always. Latex cannot be expected to compensate for very high water/cement ratios.

Strength. Normally, subsurface barriers are not required to possess particularly high strength, and unconfined compressive strengths of 100 to 200 kPa (15 to 30 psi) are usually adequate. If the constructed barrier wall serves a structural purpose, higher strength may be required. The hydraulic conductivity and long-term durability are usually more important than strength considerations. The factors that control hydraulic conductivity and durability also affect strength. Thus, there is an indirect correlation between the strength and the hydraulic conductivity and durability of cementitious barriers.

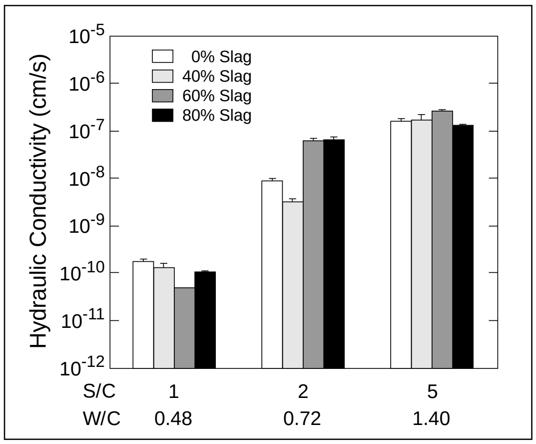

Figure 2-5 Effect of soil/cement (s/c) and water/cement (w/c) ratios and slag replacement level on hydraulic conductivity of *in situ* cured soil cements containing superplasticizers (Allan, 1995)

The strength of cementitious grout barriers made from soil and cement generally exceeds that required in the field. Strength is largely controlled by water/cement ratio and the proportion and type of soil. Reducing the water/cement ratio using water reducers or superplasticizers will result in greater strength. The effects of soil/cement and water/cement ratio on compressive strength of superplasticized materials made from soil and cement are illustrated in Figure 2-6 (Allan, 1995).

The effect of supplementary cementing materials on compressive strength depends on the type of supplementary material, the proportion added, and the curing conditions. Certain materials can increase the strength of cementitious grouts under favorable curing conditions, although the initial rate at which the strength is gained may decrease, particularly with the addition of fly ash materials. When grout is mixed with soil, the effects of supplementary cementing materials on compressive strength tend to diminish as the proportion of soil and/or water increase. An example of the effect of replacing cement with slag on the compressive strength of soil-cement is depicted in Figure 2-7 (Allan, 1995). It is seen that slag has minimal influence on strength at high soil/cementitious and high water/cementitious material ratios, even at replacement levels of 80%. Therefore, high proportions of slag can be used without detrimental effects on the strength properties of the cementitious grout. Similarly, high levels of fly ash can be incorporated in soil-cements. In contrast, the proportion of silica fume that can be added is controlled by the viscosity requirements needed for the placement technique to be used. High proportions of bentonite and other clay-type additives reduce the compressive strength of grouts and soil cements. Latex also causes a decrease in strength, particularly under wet conditions and at high water/cement ratio.

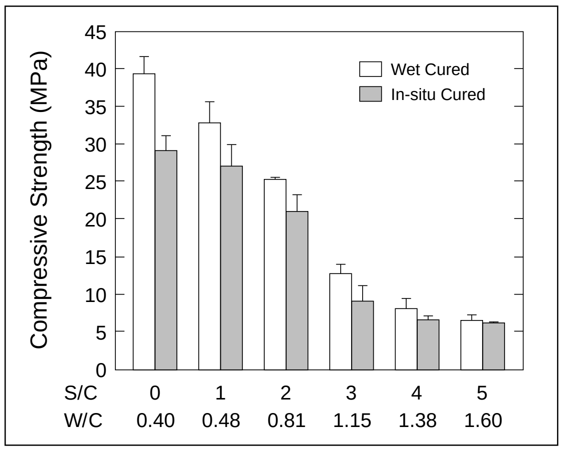

Figure 2-6 Effect of soil/cement (s/c) and water/cement (w/c) ratios on compressive strength of soil cements containing superplasticizers (Allan, 1995)

Although compressive strength is a commonly measured property, flexural strength may also be important. The flexural strengths of soil cements produced from superplasticized grouts can range from 2 to 3 MPa (depending on mix proportions).

Sulfate Resistance. Many ground waters and soils contain sufficiently high levels of soluble sulfates so as to be detrimental to the integrity of Type I cement-based barrier materials. This potential problem can be overcome either through partial replacement of Type I cement with silica fume, fly ash, blast furnace slag, or by substitution with a sulfate resistant cement (Type II or V). Such grout-based materials exhibit improved resistance to sulfate attack. Sulfate resistance is also improved by minimization of water/cement ratio.

Leach Resistance. Soft waters and many leachates, particularly those with low pH, are aggressive towards cementitious materials. Silica fume is highly effective in improving leach resistance.

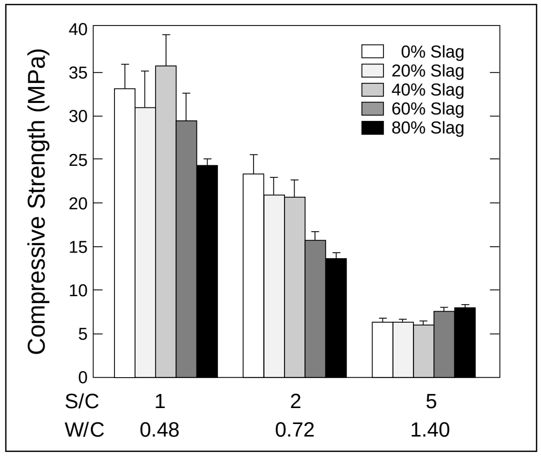

Figure 2-7 Effect of soil/cement (s/c) and water/cement(w/c) ratios and slag replacement level on compressive strength of soil cements containing superplasticizers (Allan, 1995)

Shrinkage. If a barrier shrinks, the resulting void may result in greater flow through the barrier than if the barrier material does not shrink. Shrinkage of cementitious materials can be minimized by using Type K cement or adding calcium sulphoaluminate, calcium sulfate, or gas forming admixtures. These admixtures should be used with caution since continued expansion during the hardened state can result in cracking. Shrinkage is also reduced by lowering the water/cement ratio. Although superplasticizers increase shrinkage, they are less detrimental than a high water/cement ratio. The effect of supplementary cementing materials on shrinkage will depend on the proportions and type used. Sand and other filler materials can reduce shrinkage, but require careful selection of the appropriate size fractions and proportion of filler.

Cracking may occur in hardened or nearly hardened stage when shrinkage is restrained. Crack resistance is enhanced by low water/cement ratios. The widths of cracks developed under restrained shrinkage conditions have been monitored in laboratory tests at BNL. Grouts develop smaller crack widths than soil cements. The addition of fibers to the parent grout reduces shrinkage crack widths significantly, as depicted in Figure 2-8 (Allan, 1995). The fibers used in this example were fibrillated polypropylene.

Fibers also improve ductility, flexural strength, and toughness. Fiber reinforced materials subjected to flexural load or to shrinkage form a network of fine cracks, rather than a single large crack. Even after ultimate load has been reached, fiber reinforced grouts and soil cements remain contiguous due to crack bridging by fibers. Fiber-enhanced grouts are incompatible with jet grouting or permeation grouting, but may have applications in bentonite-based materials placed by backfilling techniques.

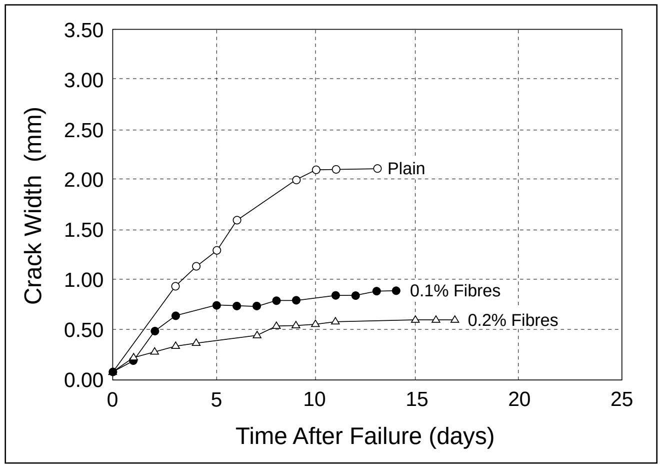

Figure 2-8 Crack widths of plain and fiber reinforced grouts under restrained shrinkage (Allan, 1995)

c. Admixtures and Cementitious Grout Properties. A variety of admixtures can be used to improve the properties of cementitious grouts used to construct subsurface barriers, particularly if conventional materials are unsuited to the site conditions. The most advantageous of these admixtures are water reducers or superplasticizers, which enable reduction of water demand. Reduction of the water/cement ratio results in enhanced strength, lower permeability, and greater durability.

Admixtures cannot compensate for other inadequacies; e.g., poor mix design, unsuitable materials, improper placement techniques for the site conditions, and substandard work practices. Laboratory and field trials are necessary for evaluating the suitability of any admixture for a given application.

In many cases, the addition of admixtures results in increased material costs. Added capital expenses may occur if additional batching equipment is needed. Field trials are required to become familiar with the attributes of specific admixtures. Low bentonite cementitious grouts containing admixtures are probably most applicable to situations in which the performance and durability of soil-bentonite, cement-bentonite, or soil-cement-bentonite materials will not be adequate.

2.3.3 Properties of Soil-Bentonite Materials

a. Hydraulic Conductivity. Soil-bentonite slurry trench cutoff walls can be designed and constructed to have a hydraulic conductivity less that 10^{-7} cm/s. Data compiled by Evans (1993) show that the stress state in the soil-bentonite backfill can have a strong influence on in-service hydraulic conductivity. Also, contaminated permeants can increase the hydraulic conductivity of barrier soils, but the effect is less significant when the soil is under high confining pressure (Acar et al., 1985; Daniel, 1987; and Mitchell and Madsen, 1987). These investigations indicate that the stress state in the soil-bentonite cutoff wall can affect its performance as a containment barrier.

b. Compressibility. D'Appolonia (1980) showed that the compressibility of soil-bentonite backfill mixtures increases as the fines content of the base soil increases, and that the compressibility is greater when the fines in the base soil are plastic. D'Appolonia states that "Compressibility depends chiefly on the percentage of granular bulky-shaped particles in the gradation. Comparatively low compressibility results when there is sufficient granular material in the mix to allow grain-to-grain contact between the granular particles."

Consolidation tests reported by Khoury et al. (1992) exhibited greater compressibility than those of D'Appolonia. It was also found that large amounts of gravel, i.e., at least 30%, would need to be added to the mix to produce significant reductions in compressibility. Consolidation tests performed on laboratory prepared soil-bentonite mixes by Evans et al. (1995) also found that compressibility increased as the fines content increased.

c. Strength. D'Appolonia (1980) presented the results of consolidated-drained and consolidated undrained triaxial compression tests performed on three soil-bentonite mixtures. As shown in Figure 2-9, all three mixes were initially contractive. Mix A, which is the coarsest of the three mixtures tested, showed a tendency for dilation at axial strains above about 3%. The consolidation pressures for the tests were not reported. Effective stress friction angles ranged from 31 to 33 degrees. Undrained shear strengths for mix A ranged from 32% to 70% of the consolidation pressure; for non-dilative mixes B and C, the undrained shear strengths were, respectively, 32% and 40% of the consolidation pressure. Using the normalized plots shown in Figure 2-9, one can obtain approximate values of the initial tangent Young's modulus of the three mixes, expressed in terms of the consolidation pressures listed in Table 2-1.

TABLE 2-1 Approximate Initial Tangent Young's Modulus Values for Soil-Bentonite Mixes (from data presented by D'Appolonia, 1980)

Mix	Approximate Initial Tangent Young's Modulus	
	CU Test	CD Tests
A	$80 \bullet \sigma'$con	$60 \bullet \sigma'$con
B	$40 \bullet \sigma'$con	$50 \bullet \sigma'$con
C	$30 \bullet \sigma'$con	$30 \bullet \sigma'$con

Consolidated-undrained triaxial compression tests performed on samples of soil-bentonite backfill used in the cutoff wall at a contaminated site in Silicon Valley, California, resulted in an effective stress friction angle of 32 degrees and an effective stress cohesion intercept of zero (Filz, 1995, personal communication).

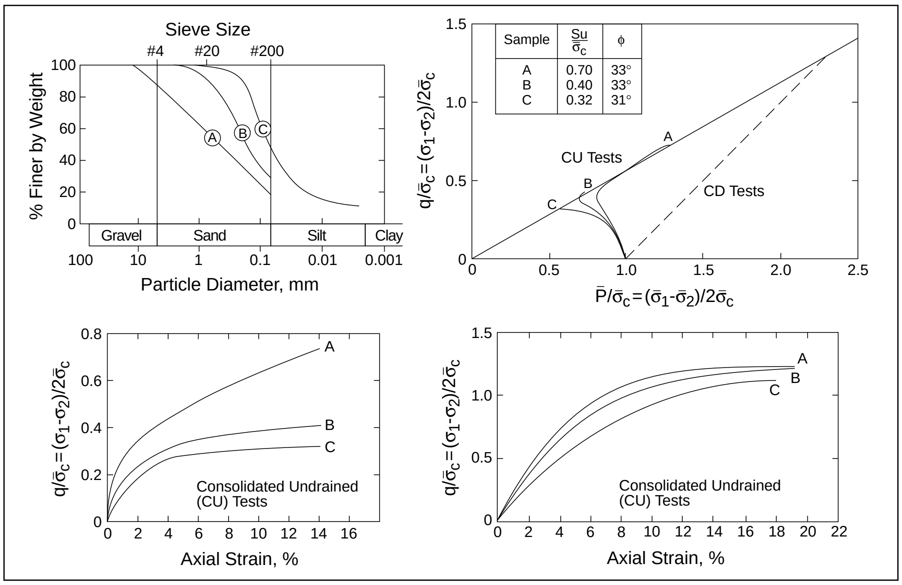

Figure 2-9 Stress-strain and strength behavior for various SB backfill gradations (D'Appolonia, 1995)

2.3.4 Slurry Wall Barriers in the United Kingdom

a. UK Historical Perspective. (Jefferis, 1995). The first cut off walls in the UK were soil-bentonite systems. However, few records exist concerning these walls or the procedures used to form them. One of the few reported walls was installed in 1963 to keep a gravel workings dry. Since 1963 soil-bentonite walls have been used occasionally in the UK - mostly at gravel workings. Overall, it can be said that applications of soil-bentonite walls have passed largely unreported and there is minimal UK literature on such walls. They are certainly not the cut off wall of choice today.

By the late 1960s to early 1970s, cement-bentonite-aggregate (plastic concrete) systems were in use. Little (1975) reports the use of plastic concrete to repair the cores of a number of earth dams. These walls achieved a depth of over 46 m and were two-phase walls excavated in 6 m panels. It is interesting to note that one of the walls was designed as a high pulverized fly ash (pfa) concrete to resist a reservoir water of pH of 3.8. No data are available regarding the performance of this wall over the last two decades.

The first reported cement-bentonite cut off wall in the UK was installed in 1973 to control seepage from an agrochemicals manufacturing plant into a stream. The mix used was a blend of ordinary Portland cement, sodium bentonite (of UK origin and, therefore, a natural calcium bentonite converted to the sodium form with sodium carbonate) and water. The wall successfully reduced the leakage of chemicals from the contaminated soils around the plant into the stream. The 1973 wall was based on a mix design from France where there had been a substantial program of work by Caron and others. However, it was found that the mix design had to be revised considerably to take account of UK materials and, in particular, the cement.

Following the 1973 wall, a research program was initiated at King's College, University of London under the direction of Dr. Stephan Jefferis. This work demonstrated that the properties of cement-bentonite materials could be radically altered if a substantial proportion of the cement in the mix was replaced with ground granulated blast furnace slag, a by-product of steel production. In the UK, such slag is slightly cheaper than Portland cement and is widely used as a cement replacement material in concrete. The effects of adding slag are comparable to those of Portland cement, but the slag has less influence on cement-bentonite mixes. Results of this research were incorporated in the design of a mix for the cut off under the Kielder Dam which created one of the largest man-made reservoirs in Europe. The wall, up to 25 m deep, was constructed in panels using a clam shell grab. The wall was required to have a permeability of less than 10^{-6} cm/s and a strain at failure of greater than 5%. A cement-slag-bentonite wall was used for the dam, the first use of such a mix in the UK. It may also have been the first use of a mix in which the slag-Portland cement ratio had been specifically adjusted to optimize properties of the cut off material. Slag cements had been used in cut offs in Europe prior to this, but at pre-blends prepared for the general construction industry at a slag-cement ratio of the supplier's choice which were often subject to quite wide tolerances. Following the Kielder cut off, there was a slow expansion of barrier work in the UK. The purpose of most of these walls was to reduce seepage around hydraulic structures, rather than prevent contaminant migration. Cement-slag-bentonite materials were used for most of these walls.

In the late 1980s to early 1990s, the emerging issues of contaminated land and gas/leachate control at old landfill sites led to a rapid increase in the use of cut off walls. During this period, a further innovation was the incorporation of jointed geomembranes into the walls. The first actual slurry trench/ geomembrane wall was installed in 1979 in the Jordanian sector of the Dead Sea, as part of a system of ponds for salt extraction by solar evaporation. The geomembrane panels were lapped, not joined. Cement-bentonite-geomembrane walls are now widely used in the UK, especially for the control of gas and leachate migration from old landfill sites.

At the present time, the material used for slurry trench walls is most commonly a cement-slag-bentonite mix, though it is recognized that pfa may give better chemical resistance. A few cement-pfa-bentonite walls have been installed. Slurry trenches in the UK are usually 0.6 m (2 ft) wide. Excavation normally begins using a backhoe (draglines are not used). Below a depth of 20 m, a cable hung or Kelly-mounted grab is used and walls are excavated in panels. In hard grounds, a cutter-miller machine (hydrofraise) may be used.

b. Materials for Barriers Constructed using the Slurry Trench Method. Soil-bentonite, cement-bentonite, cement-pfa-bentonite, plastic concrete, soil-attapulgite, cement-slag-bentonite, and cement-attapulgite have been used to form cut off elements in the UK. The basic materials are bentonite and attapulgite clays, Portland cement, slag, and pfa (in the concrete industry these are often described as cement replacement materials).

Bentonite. In the UK and Europe, almost all the indigenous bentonites have calcium as the predominant exchangeable cation. Calcium bentonite does not readily disperse in water to form the desired viscous, nonsettling suspension at low concentration (perhaps 2% to 6% by weight of water). While it has applications in slurries, it is not a material that can be used to control settlement of cement grains, aggregate, etc. in cut off materials. For this reason, sodium bentonite is used. The main role of calcium bentonites, when included in cut off mixes, is to provide extra clay solids without the resulting mix becoming unacceptably viscous. Depending on the source, sodium bentonites may be used up to a maximum concentration of perhaps 5% to 7% by weight of water without a cement-bentonite mix becoming unworkable. In contrast, calcium bentonite may be used at concentrations over 15%.

Calcium bentonites are rarely used in cut offs in the UK. When used, the indigenous calcium bentonites are pretreated (by the suppliers, rather than on site) with sodium carbonate to precipitate the calcium ions as calcium carbonate and convert the clay to the sodium form. The amount of sodium carbonate used is often above the theoretically required levels and this gives UK and European converted sodium bentonites a pH of 9 to 10.5. The extra sodium carbonate also may slightly flocculate the clay and, as a result, these bentonites may show higher gel strengths than natural sodium bentonites, such as those from Wyoming.

Many bentonites used in UK and Europe have also been treated with polymers to improve their properties (details of the polymer types and quantities added are normally proprietary to the suppliers). It follows that in the UK and Europe, bentonites from different suppliers may exhibit varying properties. Thus, when developing cut off materials, it is necessary to consider the source of the materials to be used. For major cut off projects, trial mixes will be carried out at the mixing plant with the same materials to be used for the main job. In so far as the materials can be specified, there can be problems in ensuring that even a single supplier provides material from a single source throughout a long project.

Portland Cement. Portland cement is a product of clinkering and grinding natural silica and calcium bearing materials. Gypsum (calcium sulfate dihydrate) may be added to moderate the set time. As cement is produced from natural materials, some variation must be expected between projects and even during a single project over time. Portland cements are generally used because of their ability to provide a particular minimum 28-day strength in concrete. Few, if any cements (outside the special products prepared for oil-well cementing), are tailored to applications other than structural concrete. For example, the fluid properties of cement-water systems will be of marginal significance for concrete, since the workability of a concrete will be largely controlled by the aggregate. Thus, the very properties of Portland cements considered essential to grout and slurries, i.e., their early age fluid properties, may be largely ignored by the manufacturers. Cut off wall designers may have to test cements from a number of sources in order to achieve an optimum mix. While not a major issue, additional research is needed on the early age behavior of clay-cement materials.

Portland cement, due to its high calcium content (typically >60%) and its gypsum content (designed to rapidly release calcium sulfate into the solution on wetting), will be a ready source of calcium ions. Therefore, the pore fluid of a cement-bentonite slurry will contain sodium ions from the bentonite and other ions (calcium, sodium, potassium, and other cations) from the cement. The effect of these ions will depend on the particular bentonite and its sensitivity to the ions. However, the calcium ions are likely to dominate the sodium. Thus, some time after mixing, the bentonite in a cement-bentonite mix will be effectively a calcium bentonite. The chemistry of a bentonite first mixed as a sodium bentonite that later converts to the calcium form may be indistinguishable from a calcium bentonite which has never been converted. However, the physical form may be very different and it is this feature that skilled cement-bentonite mix designers recognize and exploit.

The bentonite in a cement-bentonite mix is short lived (Jefferis, 1995). Smectite clays will react with lime that is produced by the cement hydration. X-ray diffraction analysis of old cement-bentonites shows bentonite to be absent (or below detection limits, which can be relatively high). The reaction products of the cement and bentonite are calcium silicate and calcium aluminate hydrates, comparable to those formed in the normal hydration of cement. That cement and bentonite will react is obvious, but the nature of this reaction is not well understood. This aspect is often ignored in the literature, or even worse, special features of the mix are ascribed to the chemistry of the sodium or calcium bentonite used to prepare the mix. Research is needed on this aspect, particularly on the quantity of bentonite that can react with a given quantity of cement.

Ground Granulated Blast Furnace Slag. Slag is a by-product from the blast furnace. To make it hydraulically active, slag must be rapidly cooled after tapping from the furnace. For this reason, not all slag is suitable for use in cement-slag-bentonite mixes. When used as a cement replacement material, it is ground to a fineness similar to or slightly finer than Portland cement. In comparison to Portland cement, slag has a higher silica content and lower calcium content (typically about 40%); thus, it does not release calcium ions into solution at the same rate as Portland cement. Therefore, when slag and bentonite slurries are mixed, there is no immediate stiffening (and subsequent relaxation), which differs from the behavior of Portland cement. Also, the mix shows little bleed and thus the slag does not appear to 'damage' the bentonite dispersion. However, pure slag-bentonite systems set very slowly. The slurries may be still very fluid at 14 to 28 days after mixing. Ultimately, they do set to give high strengths and low hydraulic conductivity. Therefore, it has been standard practice to blend Portland cement and slag in UK cut off mixes. The lime in the Portland cement triggers the slag hydration, although this can also be achieved by the addition of other alkalis. Slag has a number of desired effects when used in cement-bentonite mixes, including; reduction of the bleed (settlement of solids with expulsion of free water), increase in the strength of the mix, and reduction of the hydraulic conductivity of the mix.

The research findings suggest that the effect of the slag is best understood by considering the proportion of slag by weight to the weight of slag plus cement in the mix. Cement-bentonite mixes can then be specified by the total quantity of cementitious material in the mix and the slag replacement level. The effect of the slag on strength may be quite modest up to a replacement level of about 60%; but, above this, the strength substantially increases (see Figure 2-10). Up to 60% replacement, the hydraulic conductivity of a slag mix is generally comparable to that of a pure Portland cement mix. However, between 60% and 80%, the hydraulic conductivity typically will decrease by more than one order of magnitude. Above 80%, there is little further reduction in hydraulic conductivity.

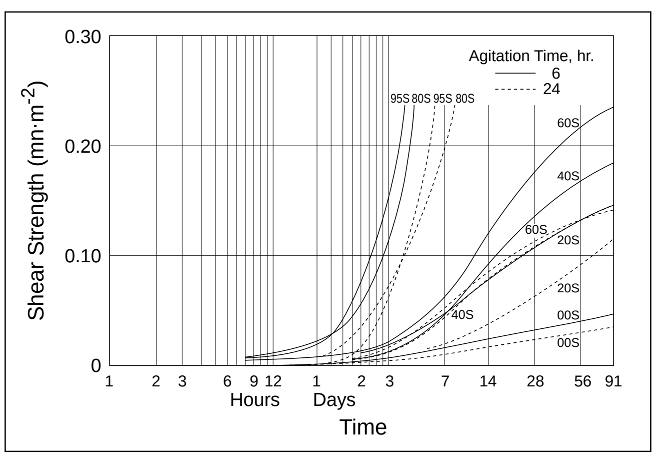

Figure 2-10 Shear strength development in slurries containing slag (Jefferis, 1991)

When cement-bentonite mixes are used, a hydraulic conductivity of less than 10^{-7} cm/s is typically required. To achieve the same low hydraulic conductivity with a pure Portland cement system may require a total cementitious content of over three times that of a slag mix. Therefore, the use of slag can be essential to the economy of the system. However, the resulting increase in strength is regarded as less desirable, a fact that is not always considered.

Pulverized Fuel Ash (pfa). In concrete mix design, pfa and slag are often regarded as comparable replacement materials. Although each has its own particular features, the final concrete mix, containing pfa or slag, may produce comparable values of strength. However, in cement-bentonite mixes, pfa and slag behave quite differently. If used as a cement replacement, pfa does not show the strength increase or reduction of hydraulic conductivity produced by slag. Indeed, if it is used at replacement levels over about 30%, it may lead to very soft mixes. Therefore, it is normally used as an additional material, rather than as a replacement material, and total cement plus pfa contents may be as high as 300 to 400 kg/m^3 of slurry, with pfa as perhaps 50% or more of this. These are much higher solids contents than would be normal for cement-slag-bentonite mixes, where the total cementitious matter seldom exceeds 175 kg/m^3 in the UK. The high solids content of pfa mixes gives them good resistance to drying and the high pfa content means that strength may continue to increase and hydraulic conductivity decrease, at least up to one year after mixing. The early age hydraulic conductivities of pfa mixes are often comparable to those of pure Portland cement mixes. Also, pfa replacement may offer better resistance to chemical attack than slag.

Attapulgite Clay. Attapulgite clay has a structure different from that of bentonite. The particles are lath shaped, rather than plate-like. When mixed with water, the attapulgite laths can tangle, much as a bundle of straw, producing a viscous suspension, although with poorer fluid loss control than bentonite. This effect of attapulgite can be quite limited, but it may be improved by prolonged mixing (tens of minutes to hours) as the laths can split longitudinally to produce more and thinner laths, thus greater tangling. Attapulgite-soil walls and attapulgite-cement walls have both been used in the strongly saline environment of the Dead Sea. Although their use in other situations may be appropriate, arguments to use attapulgite in place of bentonite in a cement-bentonite mix (because of its better resistance to saline environments) would be spurious. As already noted, the bentonite in a cement-bentonite is reacted and removed from the mix to leave cement hydrates. These hydrates also will be present in an cement-attapulgite mix.

Aggregates. In soil-bentonite mixes and in cement-bentonite-aggregate plastic concrete, the aggregate should be graded in order to achieve a dense mix in which the aggregate grains do not settle. Also, the mix should not consolidate unacceptably under the *in situ* soil stresses. For a soil-bentonite, the carrier fluid for the aggregate (the bentonite slurry) will be weak in comparison with most soil stresses; thus, the mix will consolidate until the aggregate experiences grain to grain contact, or until the slurry has consolidated to the point where it has sufficient strength to resist the soil stresses.

In plastic concrete, two types of mix are possible. The mix may be designed so that the aggregate is in grain to grain contact or it may be designed so that the cement-bentonite mix is of sufficient strength to support the aggregate. The latter can have two advantages: 1) it may increase the strain at failure, and 2) the soil stresses will be carried by the cement-bentonite matrix and not the aggregate. This will ensure that some stress is placed on the matrix, thus limiting the damage that may be caused by chemical attack.

In the UK, plastic concrete mixes are rarely used in two-phase walls (separate excavation and replacement processes) since they are more expensive than single phase walls. Two phase walls are used when an excavation is so deep or complex that it cannot be achieved before hardening of the cement-bentonite mix. Such walls are excavated under a bentonite slurry. Plastic concrete backfill is necessary to ensure full displacement of the excavation slurry. Without the aggregate, a cement-bentonite mix may be insufficiently dense to ensure this displacement.

c. UK Specifications. UK specifications include: specifications for materials as supplied, specification of fluid properties, specifications for the hardened properties of the wall material, and durability specifications.

<u>Materials</u>. There are well-tried standards for cement, slag, and pfa in the UK and, although not developed solely for cut off walls, they do provide an acceptable base-line; however, there is a problem for bentonite. Traditionally, the specification used for bentonite has come from the Oil Companies Materials Association Specification, DFCP-4, Drilling fluid materials, Bentonite. However, it has been recognized that bentonites optimized (by the suppliers) for use in civil engineering projects may be outside this specification and to adhere to specifications would not only be costly, but would also impair the material properties required in the civil engineering project.

Unfortunately, the optimum bentonite may depend on the mix formulation, preparation equipment, and site practice. These are normally contractor dependent and outside the control of the owner. This highlights a fundamental issue concerning specifications related to construction work. Should the work be specified by method and/or materials to be used or should the work be specified in terms of the expected performance after completion? In the UK, it is almost universal practice to specify construction work by performance, especially in the case of cut off walls systems. Although, this situation does not apply worldwide, there is interest in moving towards performance specifications in other countries.

Specifications based on method may address only the ingredients, but may not allow the specifier to control the mixing process. Different cut off wall systems may be more or less sensitive to the balance between good ingredients and the skills of a contractor to obtain the optimum mix of the ingredients. Thus, the type of cut off wall used for a specific project may depend on the type of specification used. Experience in the UK suggests that cement-bentonite walls cannot be specified by ingredients. The skills of the contractor, the mixing equipment, the mixing, and special proprietary additives are all key components. In contrast, soil-bentonite walls may be more easily specified by method; i.e., defining soil grading and bentonite content, etc.

<u>Fluid Properties</u>. The fluid properties of a slurry are of little significance, provided that the slurry performs its proper role in supporting the excavation. Nevertheless, measurements of fluid properties are often proposed as a method of quality control for the batching and mixing. As a result, measurements have been made for nearly every property for which a test existed. The range of tests has reduced substantially over the last decade. In the past, Marsh funnel, Fann viscometer, density, bleeding, pH, and filter loss all might be measured, for both bentonite slurries and cement-bentonite slurries. However, it was found that the fluid properties (except density and pH) were strongly influenced by the age and shear history of the slurry. Accurate batching of the raw materials is now regarded as more important than tests on the fluid slurry. Typically, all materials are required to be batched to ±3%. Currently, tests on the fluid slurry are generally limited to the following:

- Marsh funnel flow time (if the slurry will pass through the funnel),
- density, measured to ±0.002 g/ml, using a fixed volume cylinder and an electronic balance, and
- bleeding using a 1000 ml measuring cylinder.

<u>Assessment of the Hardened Material</u>. Ideally, tests should be carried out *in situ*. Some success has been achieved with *in situ* cone penetration tests and, occasionally, techniques based on pore pressure probes inserted into the wall have worked; but the reliability is much less than 100%.

Golder Associates (Eiben and Jefferis, 1994) have developed a procedure for the assessment of walls using two rows of wells on either side of the wall with the wells spaced at intervals of the order of 20 m in each row). The procedure is to inject water into each of the wells, in turn, at a defined rate for a defined period, and to measure the pressure response in the injection well and the adjacent wells. In this manner, the full length of the wall can be inspected, with hydraulic conductivities of 10^{-7} cm/s or greater being measured to within half an order of magnitude .

As *in situ* tests are not yet reliable, specimens for testing the hardened material *in situ* are cast from samples taken from the trench after excavation, but before the slurry hardens. Cored samples from a hardened wall are invariably so damaged as to be useless. Despite this, regular attempts are made to core walls, but the results are invariably of little value.

In general, specifications for the hardened (set) material have required the assessment of three properties of the mix (plus any durability requirements). These properties are; hydraulic conductivity, strength, and strain at failure.

Permeability. For hydraulic structures, a hydraulic conductivity of 10^{-6} cm/s may be acceptable, but for contaminant containment, a hydraulic conductivity of 10^{-7} cm/s is typically specified.

Strength. Specifications call for a strength criterion or a permissible range of strengths. At present, there is no consensus as to what strength level is appropriate (except as it may influence the strain at failure). Typical values might be >50 kPa (7.25 psi), <1000 kPa (145 psi), 100 to 300 kPa (14.5 to 43.5 psi), etc. A common feature of strength tests from many cement-bentonite cut off walls is the wide scatter of the results. The reasons for this are currently the subject of research.

Strain at Failure. A strain at failure of greater than 5% is almost invariably specified in the UK. This specification is seldom justified. Unfortunately, this specification cannot generally be met with cement-bentonite barrier materials except under confined drained triaxial conditions with a confining pressure on the order of 50% to 100% of the unconfined strength of the material. Even under these high confining pressures, the strain at failure is not specified as a strain without increase of hydraulic conductivity (which logically is what is important and for which there is no test, unfortunately). The specified strain is referred to as the strain capacity.

For a cement-bentonite material with a strength of 200 kPa (29 psi), it is unlikely that the 5% strain condition can be achieved, except under a confining pressure of order 100 kPa (14.5 psi). This pressure level will only exist at substantial depth in the ground, if at all. Thus, in the upper region of any cement-bentonite wall, the strain condition will not be achieved and the material is likely to fail at a strain of order 0.2 to 2%. In an attempt to maximize strain capacity, one approach is to specify a maximum strength for cement-bentonite materials. Despite the evident inconsistency of the criterion, this latter requirement persists in many specifications. However, the Draft UK National Specification for Cut off Walls omits this requirement, noting that if high strains are required, special mixes must be designed (although none has been found to date) or a geomembrane must be included in the wall.

Durability. Durability is a major issue for vertical barrier walls designed to prevent the migration of contaminants. The literature on containment systems is replete with references to compatibility tests, which may fail to identify underlying issues. Further, because of general practice to maximize the number of pore volumes passed through a sample in minimum time, many tests have been undertaken at such high confining pressures as to ensure that any serious chemical damage, erosion, or cracking will likely be masked by consolidation (Jefferis, 1992). There is need for much greater guidance on test procedures. Passing contaminants through barriers is a favorite topic of research, but such efforts will have limited value unless standard procedures are developed.

For cement-bentonite mixes, it is just as necessary to consider the effect of water as it is to consider the effect of contaminant chemicals (Jefferis, 1995). Lime (calcium hydroxide) can be leached from cement-bentonite materials. Leaching of the lime (and other soluble alkalis) will reduce the pH of the mix and tend to destabilize the calcium silicate hydrates, causing further release of lime to yield materials of lower calcium to silica ratio, until all the calcium has been removed and only hydrated silica remains. This process has been studied in the laboratory and found to cause significant consolidation of the sample as the loss of lime weakens the cemented structure. However, it does not lead to an increase in hydraulic conductivity, but rather to a marked reduction (even at low confining pressures). The hydraulic conductivity of a cement-bentonite mix may decrease by over two orders of magnitude over a year of permeation. It has been suggested that the hydraulic conductivity of cementbentonite materials will continue to decrease, almost without limit, during permeation. This seems unlikely, since it fails to recognize the leaching effect of water.

In contrast to the leaching power of water for lime (water will dissolve of order grams/liter of lime), the effects of many contaminants dissolved in groundwater, such as heavy metals at low concentrations, may be trivial except over very long times. While the pH of the slurry is high, many metals will be immobilized by precipitation. However, once leaching of lime is complete, they may be re-released unless they remain sorbed in the hydrated silica. There is little data on the effects of organic contaminants on cement-bentonite materials and more research is needed.

Certain materials, such as sulfates, can cause expansion of cementitious barrier materials. This expansion can lead to spectacular damage to unconfined samples, in effect, total disintegration. However, the effects are much more limited if the sample is confined, even under quite modest pressures. As the cement-bentonite material is of only moderate strength, restraint can force the expansive phases (such as ettringite and thaumasite) to expand within the sample rather than to expand the whole sample. Thus, under confined conditions, sulfate appears to cause some softening and only a rather limited increase in hydraulic conductivity.

Under confined conditions, the change of hydraulic conductivity, due to contaminant permeation (with the exception of strong acids), may be limited to the order of a 10 to 50 fold increase, a significant, but often not catastrophic, increase. For many wastes, such as landfill leachate, the immediate effect of the waste may be to substantially reduce the hydraulic conductivity, perhaps one order of magnitude, due to the precipitation of metals, carbonate, etc., in the alkaline-lime rich environment of the cement-bentonite. This reduction in hydraulic conductivity may substantially reduce the advective flux of contaminants; thus, extending the service life of a wall.

The effect of mechanical strain and cracking is also of concern. As already noted, the strain at failure of a cut off wall is likely to be of order 0.2% to 2%. Thus, if ground deformations occur, cracking is possible. In this respect, soil-bentonite materials may offer an advantage over cement-bentonite materials, since they can have a higher strain capacity. However, it should be remembered that cement-based materials have the potential for self-healing (autogenous healing) if the cracking is not so severe as to allow rapid flow of water (seeps of water through concrete retaining walls often block and cease). The blocking mechanism seems to be partly due to deposition of calcium carbonate and partly further hydration/migration of the cementitious phases. This phenomenon has been demonstrated with cement-bentonite materials in the laboratory. In the field, conditions may promote self-healing, since carbon dioxide concentrations in soil are often higher than in air; thus, carbonate deposition will be encouraged. Also, flow through cracks could be slowed by deposition of soil particles.

Drying. Cement-bentonite materials are sensitive to drying and will not swell on re-wetting. This can be expected, given the calcium rich environment of the cement and the fact that some or all the bentonite will have reacted with the cement products to form calcium silicate and calcium aluminate hydrates. Therefore, capping of cement-bentonite walls to prevent cracking needs to be done soon after excavation. Capping also provides some useful effective stress in the upper regions of the trench that will improve durability.

2.3.5 Field Performance of Soil- and Cement-Based Barriers

a. U.S. Field Performance Experience. Soil- and cement-based vertical barrier walls have been successfully constructed to control groundwater seepage and to prevent migration of contaminants from a waste disposal site. For barrier walls constructed to control groundwater seepage and/or provide structural control, their integrity has been demonstrated by safely excavating from within the barrier for many hundreds, if not thousands, of projects. Unfortunately, however, it now appears that reliable estimation of hydraulic conductivities based on post-construction testing of samples from wall constructed of cementitious materials cannot be obtained.

It was the unanimous view and experience of the workshop panel that post-construction sampling and testing of walls constructed of cementitious materials leads to a high estimate of the hydraulic conductivity for the barrier material. It is believed that these high values are due to the mechanical damage resulting from the drilling and sampling process (and the associated stress relief) which alters the microstructure of the cementitious barrier material. For very soft materials, such as soil-bentonite, on the other hand, post-construction sampling and testing is reliable, since the consolidation process during testing appears adequate to overcome any disturbance to the soil-bentonite backfill microstructure.

Soil- and cement-based barrier walls are generally considered compatible with a wide range of organic and inorganic contaminants, if at relatively low concentrations. However, should organic contaminants be present as non-aqueous phase liquids, the potential for wall deterioration increases and should be carefully evaluated. Cement-bentonite and cement-bentonite-slag are subject to cracking, if allowed to dry excessively. Cement-bentonite walls containing fly ash exhibit somewhat better drying resistance. Leaching resistance is related to the hydraulic conductivity, i.e., all else being equal, the lower the hydraulic conductivity the greater the leaching resistance.

b. UK Field Performance Experience. Cement-bentonite based walls are in regular use in the UK. Indeed, at the end of 1994 it was estimated that cut off walls were being installed at of order 4 km/month, with most of these walls constructed at old landfill sites, while very few were constructed for seepage control around hydraulic structures. Earliest barrier walls for environmental remediation applications were installed in the late 1960s and early 1970s. There have been no reported failures in service, even for walls subjected to high piezometric head gradients, and thus subjected to powerful water leaching.

Current wall materials are capable of achieving hydraulic conductivities of order 10^{-7} cm/s using only 40 kg of bentonite and 150 kg of cement (including slag) per cubic meter of slurry, thus yielding economic mixes. Work is underway to develop cement-bentonite mixes with hydraulic conductivities less than 10^{-9} cm/s using cement-slag-bentonite mixes.

Mixes are presently available with hydraulic conductivities below 10^{-9} cm/s and with good resistance to inorganic materials, such as acid and sulfates. These semi-setting materials are based on graded aggregate systems filled with a silicate-silane gel. They have a strain capacity of order 3%. However, the costs are high and, in many circumstances, it may be less expensive to use a geomembrane, unless the diffusion of organics through the membrane is significant.

c. Factors Affecting Field Performance of Soil- and Cement-Based Barriers. The factors affecting the field performance of soil- and cement-based barriers include: curing time, wet-dry cycling, freeze-thaw cycling, fluctuating water table, state of stress, compatibility, and potential construction defects. These are discussed below along with ways to minimize negative impacts. Needs for additional research are also highlighted.

Curing time. There is overwhelming evidence that the hydraulic conductivity of cementitious vertical barrier materials continues to decrease with time over periods of months to years (depicted in Figure 2-11, Jefferis, 1995). The practice (in the U.S.) of measuring the hydraulic conductivity after 28 days of curing will, therefore, result in hydraulic conductivity measurements that will be higher than those after longer curing periods.

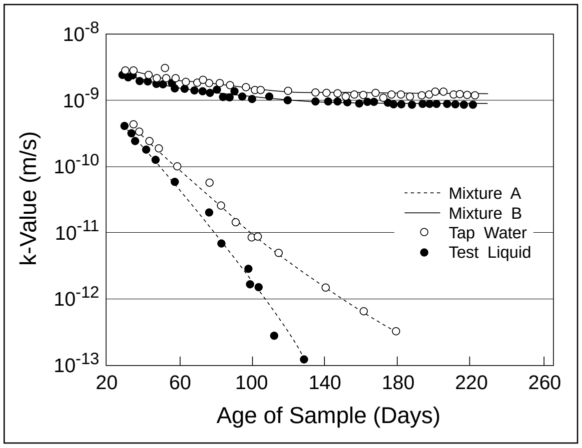

Figure 2-11 Hydraulic Conductivity values of mixtures A and B as a function of age and test liquid (Jefferis, 1995)

<u>Cycles of Wetting and Drying</u>. Little work has been done to investigate the effect of wetting and drying cycles on soil- and cement-based barrier materials. It is known that when a barrier constructed of soil-bentonite is permitted to "dry", the hydrated bentonite shrinks, and hydraulic conductivity increases. In the U.S., little is known about cement-bentonite walls subjected to drying. In the UK, it is believed that cement-bentonite-fly ash cutoff walls are more resistant to desiccation cracking than cement-bentonite-slag walls. For all types of barriers, the height of the barrier extending above the water table may remain intact, if that height is less than the height of capillary rise (assuming the rate of drying is less than the rate at which moisture is replaced by capillary suction).

<u>Cycles of Freezing and Thawing</u>. Cycles of freezing and thawing can cause a general deterioration of vertical barrier materials. However, this deterioration can be avoided by providing a cover, sufficiently thick to protect the barrier wall from freezing temperatures.

<u>Fluctuating Water Table</u>. Although the portion of vertical barriers constructed above the highest water table elevation may be subject to the harmful effects of desiccation, little is known about the portion of the barrier that may be subject to the long-term effects of a fluctuating water table. In virtually all

applications, variations in the elevation of the water table can be expected, primarily due to seasonal and long term variations in climate. The effect of these water table fluctuations on the hydraulic conductivity of the cutoff wall is unknown. Although it may be assumed that there is no effect, there are few data to support or refute this assumption. One sampling study found that the hydraulic conductivity was higher for soil-bentonite cutoff wall material permanently above the water table than that for material permanently below the water table (Evans and Cooley, 1993). Additionally, when the soil sample from above the water table was re-saturated in the laboratory, the original low hydraulic conductivity was not restored. This may not be surprising, given that the effort required to hydrate the bentonite is considerable.

State of Stress. The hydraulic conductivity is influenced by the consolidation stresses during testing; the higher the stresses, the lower the measured hydraulic conductivity. Thus, in order to accurately reproduce these stresses in the laboratory, it is necessary to know the state of stress in the constructed vertical barrier.

Compatibility. The permeation of organic contaminants at low concentrations generally has little negative impact on the hydraulic conductivities of clayey barriers (Mitchell and Madsen, 1987). However, concentrated organic contaminants, e.g., nonaqueous phase liquids, may cause increases in hydraulic conductivity (Day, 1993; Rumer and Ryan, 1995). The permeation of inorganic contaminants, e.g., strong acids or bases, into clayey materials may bring about an increase in hydraulic conductivity.

Construction Defects. The potential for construction defects is of course dependent on the construction technique used, the materials employed, and the QA/QC procedures followed during construction.

2.3.6 Vertical Barriers Enhanced to Impede Contaminant Transport

Recent investigations on different clays have shown that a judicious choice of clay minerals can enhance the adsorption of some contaminants, thus impeding their diffusive transport through the barrier (Amann, et. al., 1994).

At present, containment barriers are not designed to inhibit advective and diffusive transport, nor to treat contaminated water that might permeate the barrier. New types of containment barriers have been proposed that not only impedes contaminant migration, but also treats contaminants permeating the barrier wall. This innovative concept, referred to as an enhanced barrier, combines containment with other technologies, including: stabilization, physical-chemical treatment, and biological treatment.

Two fundamentally different types of enhanced barriers have been proposed. The first, termed a minerally enhanced barrier, must still have a low hydraulic conductivity (as are present barriers), but will also retard organic and inorganic contaminants by adsorption, precipitation, and possibly other chemical reactions. In the second type, a biologically enhanced barrier, the containment barrier material must still have a low hydraulic conductivity and be capable of supporting and promoting the biological degradation of permeating organic contaminants. This second type may also be able to bind organic and inorganic contaminants, similar to the first type.

Enhanced barriers offer improvements over present barriers, which are designed only to prevent or impede the migration of contaminants. Although enhanced barrier technologies are attractive, much research and development remains to be done in order to make them acceptable, cost effective, commercially available, and effective throughout the service life of the constructed containment system. Readers are referred to Sections 10 and 11 for treatments of the mathematical modeling of enhanced barrier systems and of the design and performance of reactive permeable barriers, respectively. [It should be noted that enhanced barriers are meant to continue to impede advective transport, in contrast to the unimpeded flow through permeable reactive barriers discussed in Section 11.]

2.3.7 Basis of Vertical Barrier Design

Vertical barriers are designed to have hydraulic conductivities of 10^{-7} cm/s, regardless of the site specific conditions. However, at some sites, where strength was an also important consideration, cement-bentonite mixes having hydraulic conductivities as high as 10^{-5} to 10^{-6} cm/s have been permitted. A rational basis for design may involve a more detailed consideration of the time dependent contaminant transport through the wall, including retardation mechanisms, biotic reactions, and abiotic reactions. Materials having the same hydraulic conductivity may perform quite differently when evaluated in this manner. Such assessments might lead to evaluation of the time varying reliability of an installed barrier wall (Inyang and Tumay, 1995). The performance of a barrier wall could also be based upon the concept of barrier efficiency, computed by comparing the estimated flux of the contaminant before and after barrier installation. Performance efficiency could also be based on a comparison of the concentration of a contaminant within the containment to the concentration of the contaminant predicted to occur outside the containment at the end of the design service life of the installed barrier wall.

2.4 ASSESSMENT OF SOIL- AND CEMENT-BASED BARRIERS

- For soil-bentonite backfill, it is necessary to select a base soil with a low hydraulic conductivity in order to minimize potential for deterioration in hydraulic conductivity that may develop as a result of the imposed environmental stresses.

- The addition of fibers to a barrier wall formulation improves the post-hardened mechanical properties without significant changes in hydraulic conductivity.
- It is essentially impossible to reliably measure the hydraulic conductivity of core samples obtained from cement-bentonite barrier walls, due to disturbance of the sample. The measured results generally produce high estimates for the hydraulic conductivity.
- The hydraulic conductivity of cementitious materials declines as curing time increases.
- The integrity of vertical barrier systems has been verified by excavating from within the barrier at many projects constructed for ground water and/or structural control. Much less is known about the integrity of vertical barriers installed for environmental remediation applications. Additional monitoring and testing is needed to verify the integrity for these latter applications and to establish if there are differences in reliability for walls constructed of different materials. Suggestions for system performance verification include the use of boxouts, pump tests, injection tests, construction QA/QC, geophysical exploration methods, and post-construction sampling and testing. However, the measured values for hydraulic conductivities obtained from post-construction samples can be higher than the *in situ* values, due to sampling disturbances.
- Although evaluation of the quality of the materials used for the barrier wall is important, quality control of the construction process is equally important.

2.5 RESEARCH AND DEVELOPMENT NEEDS

There is concern about the variation in choice of materials used in vertical barriers for containment (Potter, 1995). The UK experience indicates wide-spread use of cement-bentonite, whereas the U.S. experience indicates soil-bentonite as the material of choice. The basis for this difference in practice needs to be clarified. Such clarification would aid in the selection of one material over another. Also, there remain concerns about the long-term durability of materials used to construct vertical barriers. Thus, better ways are needed to predict the service life of a given barrier that include the effects of environmental stresses such as freeze/thaw cycles, wet/dry cycles, desiccation, dynamic loads (earthquakes), and fluctuating water table. Durability is of particular concern for barriers constructed in the vadose zone.

As noted, the integrity of the vertical barrier has been verified for conventional cutoff walls by excavating from within the cutoff wall, but little has been done to verify the integrity of vertical barriers installed for environmental remediation applications. There is a recognized need for system integrity verification of these barrier walls. Additional monitoring and testing is needed to verify the system integrity (see Section 12) and to establish if there are differences in the reliability of walls constructed from different materials. Research that addresses the issue of system integrity should include: 1) investigations into the cause/impacts of variations between mix batches, including considerations of mixing sequence; 2) study of the long-term performance of cutoff walls constructed in a contaminated environment; 3) study of performance differences between cutoffs constructed using different techniques (e.g., slurry trench, jet grouting, deep soil mixing, etc.) and different materials (soil- or cement-based); and 4) study of the long term effects of cycles of freezing and thawing and cycles of wetting and drying on barrier wall performance and the effects of desiccation of cutoff wall materials.

Additional study is needed regarding the analysis of contaminant fate and transport through soil- and cement-based vertical barriers. Enhancements to the vertical barrier (e.g., higher clay fraction in a European wall and zeolitic clays in U.S. studies) can improve wall performance by lengthening the time for contaminant breakthrough and reducing contaminant flux through the barrier. For walls needing added strength, the addition of fibers can be considered. Fiber addition does not increase the hydraulic conductivity of the barrier. Several options exist for enhancing the performance of low hydraulic conductivity walls, including the concept of utilizing the barrier as an *in situ* reactor, capable of biotic and abiotic transformation and degradation of permeating contaminants. Further research, both laboratory and field, is needed to optimize the use of enhancements and to verify expected field performance. Permeable reactive walls have also been proposed (see Section 11). They differ from those discussed in this section, in that they are designed to permit the flow of contaminated groundwater through a permeable reactive media.

Currently, there is no accepted way of testing walls *in situ*. Various techniques are being investigated but have yet to be proven reliable. Thus, there is a serious lack of information on the actual *in situ* behavior of constructed vertical barrier walls at contaminated sites. Data are available for laboratory tests on samples taken from constructed walls, but few walls have been monitored for extended periods of time.

The major issues relating to barrier wall materials include: hydraulic conductivity, strain capacity, durability, cost, and the observed variations of hydraulic conductivities. Hydraulic conductivities of order 10^{-7} cm/s are currently achievable, but the causes of observed variations of measured hydraulic conductivities between batch mixes need to be better understood. Although very low hydraulic conductivities can be achieved with current barrier wall technologies, the incorporation of geomembranes may offer some advantages (see Section 5).

UK specifications often call for a 5% strain without mechanical failure. However, this cannot be achieved except under moderate to high confining pressures. The development of low cost materials with greater strain capacity is of considerable interest, but is unlikely to be achieved with cement based materials, thus requiring new and different cutoff material formulations.

Compatibility testing needs to be standardized. At present, compatibility testing is conducted using a variety of procedures and methods. Additional data are needed to characterize the stress-strain, strength, and compression characteristics of a variety of soil-bentonite mixes. Standardized design procedures are also needed for predicting 1) the stress-state in soil-bentonite backfilled trenches and 2) post-construction adjacent ground movements.

A better understanding of the microstructure of low hydraulic conductivity barrier materials is required. Even with better data on microstructure, long-term durability will remain an issue. Chemical compatibility tests will continue to be essential for some contaminants. Existing walls that have been in service for many years should be investigated to determine the effects of permeation and the fate of the permeated contaminants.

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SECTION 3

DESIGN, CONSTRUCTION, AND PERFORMANCE of SOIL- AND CEMENT-BASED VERTICAL BARRIERS

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3.1 BACKGROUND INFORMATION

Vertical barriers for cutoff of fluid and chemical flow in the ground are frequently used as part of waste containment and remediation schemes at contaminated sites. The purposes of constructing a vertical barrier are to capture, contain, or redirect the flow of clean or contaminated groundwater, soil vapor, or nonaqueous phase liquids (NAPLs). This section focuses on soil- and cement-based vertical barrier technology, including such important aspects as site investigation requirements, analysis, design, construction, and performance. Backfill material issues are discussed where they are relevant to the other topics of this section, but materials are covered more completely in Section 2, "Soil- and Cement-Based Vertical Barriers with Focus on Materials." Excavation/replacement and mixed-in-place technologies, e.g., backfilled trenches and jet-grouting, are discussed; permeation grouting is not covered.

Much information on soil- and cement-based vertical barriers is available from several sources, including Boyes (1975), Evans (1991 and 1993), Jefferis

(1981), LaGrega et al. (1994), Manassero and Pasqualini (1993), Millet and Perez (1981), Ryan (1987), and Spooner et al. (1984). Chapter 3, "Vertical Barriers," in Barrier Containment Technologies for Environmental Remediation Applications (Rumer and Ryan, 1995) provides a comprehensive overview of vertical barrier technology. The intent in the current chapter is to expand rather than duplicate the material in Rumer and Ryan (1995). Accordingly, only brief background material is given here; the purpose being to provide the reader with descriptions of technologies that are discussed in more detail in later sections.

Soil- and cement-based vertical barrier types include soil-bentonite cutoff walls; cement-bentonite cutoff walls; plastic concrete cutoff walls; cutoff walls backfilled with mixtures of cement, bentonite, fly ash, ground granulated blast furnace slag, and/or natural clay; and walls constructed by deep mixing and jet grouting.

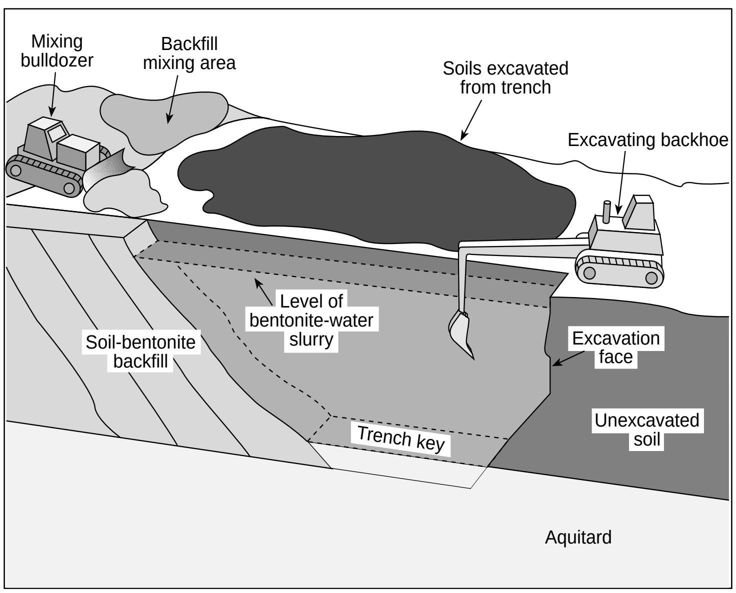

Figure 3-1 Soil-bentonite cutoff wall (Rumer and Ryan, 1995)

As shown in Figure 3-1, soil-bentonite cutoff walls are constructed in long trenches. The trench can be excavated by backhoe, clamshell, milling equipment, or some combination of these devices. Trench stability is maintained by excavating under a bentonite-water slurry, which is maintained in the trench at a level above the adjacent groundwater level. In areas of high groundwater, it may be necessary to construct a berm along the cutoff wall alignment in order to maintain the necessary differential head. The differential fluid pressure between the slurry and the goundwater in the formation creates a tendency for the bentonite-water slurry to flow into the formation; however, the formation filters the bentonite particles from the slurry to create a bentonite filter cake at the trench wall, provided that the void spaces in the formation are not too large. The low hydraulic conductivity of the filter cake causes a large pressure drop to develop across the filter cake. This pressure difference is important in maintaining the stability of the trench wall.

Either the excavated soil, if it is suitable, or imported soil is thoroughly mixed with bentonite and water to form the cutoff wall backfill material. If the backfill is properly designed and mixed, it will have the consistency of a high slump concrete and will exhibit a low hydraulic conductivity. As the backfill is placed in the trench, it displaces the bentonite-water slurry to form the cutoff wall.

Cement-bentonite cutoff walls are constructed by using a cement-bentonite-water slurry to stabilize the excavation. The cement-bentonite-water slurry is left in place to harden and form the cutoff wall. (In Europe, this is referred to as a "one-phase" method, as opposed "two-phase" methods in which the excavation support slurry is replaced by a separate backfill material.) After hardening, the cement-bentonite typically has the consistency of a stiff clay. Since the excavation spoils are not reused as backfill, they must be disposed elsewhere. Material handling is simpler with cement-bentonite cutoff walls than it is for soil-bentonite cutoff walls, and a cleaner operation results. Cement-bentonite cutoff walls can be constructed in long trenches, similar to those used for soil-bentonite cutoff walls, or they can be constructed as a series of overlapping panels.

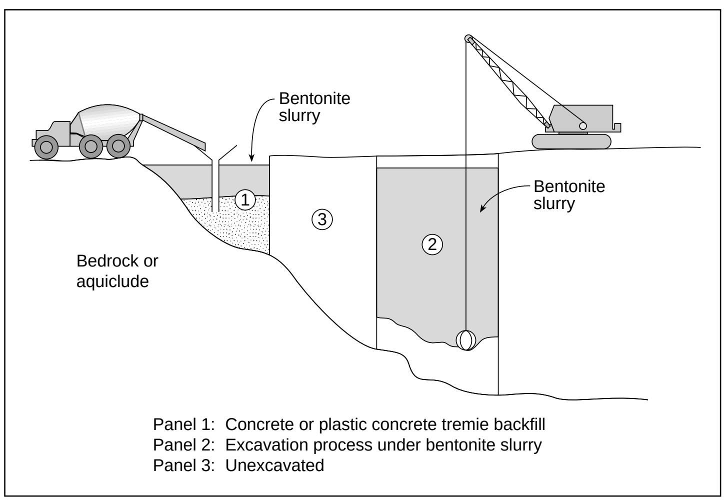

Figure 3-2 Plastic concrete cutoff wall

Plastic concrete is a mixture of cement, bentonite, aggregate, and water. Plastic concrete cutoff walls are usually constructed in panels excavated with clamshells or milling equipment, as shown in Figure 3-2. Panel support during excavation is provided by bentonite-water or cement-bentonite slurry. The plastic concrete is placed by the tremie method to avoid both segregation of the backfill and entrapment of the bentonite-water slurry. Plastic concrete mixes with high unit weights are especially useful when backfilling deep trenches. After setting, plastic concrete walls are significantly stiffer and stronger than cement-bentonite walls.

A recent variation of plastic concrete is soil-cement-bentonite (SCB). SCB walls are usually constructed using the long-reach backhoes typical of soil-bentonite cutoff walls, but with a soil-based (usually clay) backfill containing bentonite and a small proportion (usually less than 10 percent) of cement. The amount of cement is minimized to avoid formation of cold joints during construction. After setting, SCB is similar to cement-bentonite in strength and to soil-bentonite in hydraulic conductivity.

Other backfill mixtures are also used, especially in Europe and Japan. Such backfills have included cement, bentonite, fly ash, slag, and natural clays. When fly ash or slag, for example, are added to cement-bentonite-water slurries, the solids content may increase, which can result in decreased hydraulic conductivity and increased resistance to chemical attack.

The preceding methods may all be termed excavation/replacement methods. Another category of soil- and cement-based vertical barrier construction technology consists of the mixed-in-place methods, which include deep mixing and jet grouting. An advantage of the mixed-in-place methods is that smaller quantities of excavation spoils are created. If excavation spoils are classified as a hazardous waste, special arrangements will have to be made for their safe disposal.

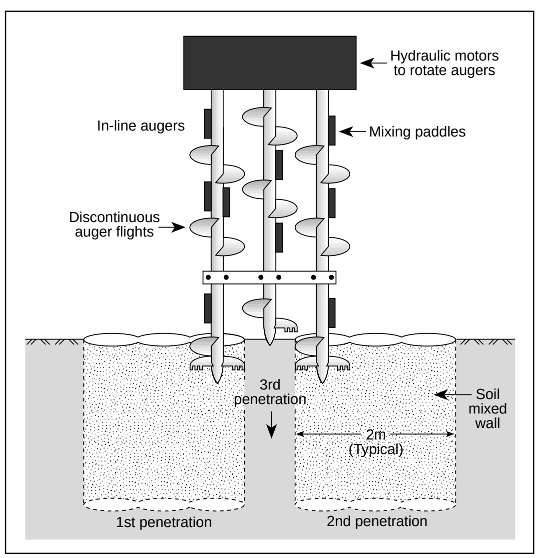

Figure 3-3 Deep soil mixed cutoff wall

Deep mixing (DM) is accomplished by use of a rig fitted with a bank of counter-rotating augers. As shown in Figure 3-3, overlapping panel construction is used to create a continuous cutoff wall. Additions such as cement and/or bentonite are introduced during the mixing process to produce the desired low hydraulic conductivity barrier. Hard ground or the presence of boulders can cause problems for the DM augers. The deep mixing technology is described in more detail by Ryan (1987) and Yang (1994).

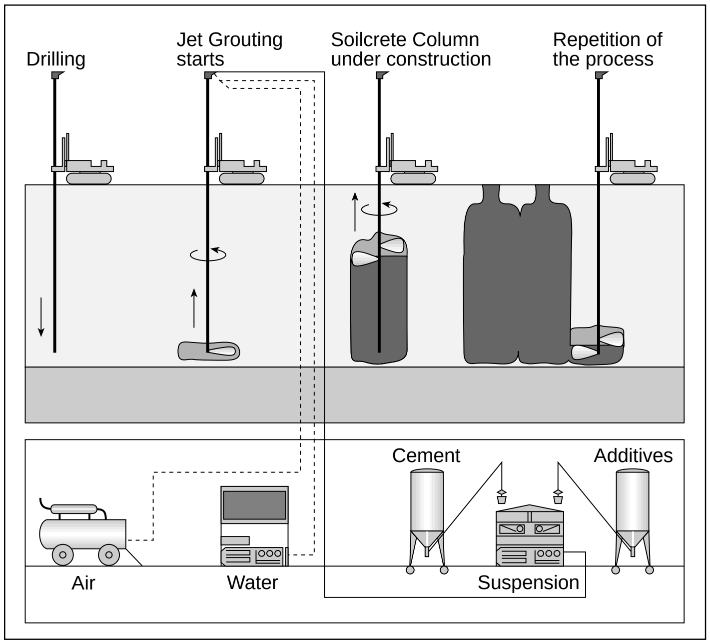

Figure 3-4 Jet-grouted cutoff wall

Jet grouting uses high pressure jets to erode and mix the soil with added cement and/or bentonite. If the jets are rotated, cylindrical piles are formed. As shown in Figure 3-4, cylindrical piles can be constructed in an overlapping fashion to create a continuous wall. Good control of vertical alignment is important to assure wall continuity. Various arrangements of the jets are possible. In the single-rod system, only cement-bentonite grout is pumped out of the jets to accomplish both erosion and mixing. The single-rod system produces a fairly small amount of spoils at the ground surface. In the double-rod system, some jets release compressed air, which can aid the cutting process and increase removal of cuttings from the ground. In the triple-rod system, various jets release compressed air, water, and cement-bentonite grout. The triple-rod system can be operated to accomplish almost complete removal of the soil and replacement with grout. In a relatively recent innovation, the jets are not rotated so that panels from a few to several centimeters thick are formed instead of columns. The jetted panels are angled with respect to one another so that they will intersect and form a continuous cutoff.

The SoilSaw™ is a relatively recent technology that uses a reciprocating beam fitted with jets that erode the soil and mix added bentonite and/or cement to produce a cutoff wall. In effect, the SoilSaw™ is a jet grouting method in which the jets reciprocate rather than rotate. Figure 3-5 illustrates the construction process.

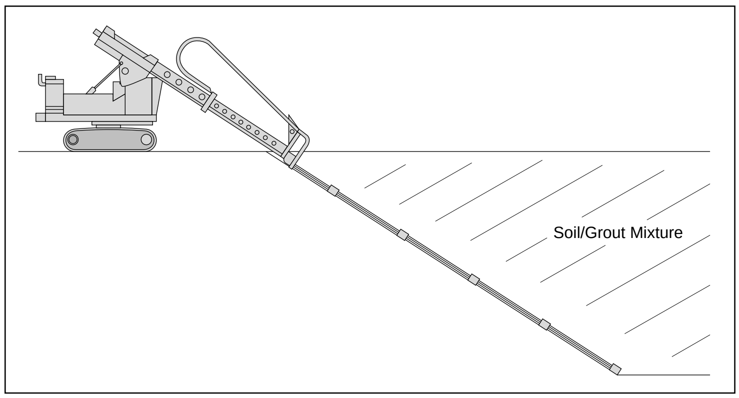

Figure 3-5 Cutoff wall installed using a Soilsaw™

Deep mixing and jet grouting have an excellent history of successful application in conventional civil engineering applications and are now being used in environmental applications. The vendor of cutoff walls constructed using the SoilSaw™ considers this to be a still developing technology.

There is another category of cutoff wall construction that could be termed a displacement method, in which grout is forced into the ground and adjacent soil is displaced. The vibrating beam is one such approach. Displacement methods are not discussed here. Some coverage of the vibrating beam method is provided in Rumer and Ryan (1995).

In the US, soil-bentonite backfilled trenches are common. In Europe, cement-bentonite and slag-cement-bentonite walls are used most often. In the Far East, especially Japan, deep mixing is extensively used.

The following sections present 1) the state-of-practice for design and construction; 2) currently available field performance verification techniques; 3) an assessment of the technology; and 4) research, development, and technology transfer needs.

3.2 STATE OF PRACTICE

3.2.1 Site Investigation Requirements

Subsurface investigations at contaminated sites where vertical waste containment barriers are being considered have several purposes, including to: 1) identify the nature, extent, and distribution of contamination; 2) characterize the geological strata; 3) characterize the hydrogeological conditions; 4) provide geotechnical data for design of the remediation scheme; 5) provide data and samples for the contractor's use; and 6) provide information for use in establishing a performance monitoring program. In seismic areas, it is also necessary to determine the proximity to active faults and to estimate the ground shaking potential.

Site investigation techniques include: 1) reviews of previous studies and reports, 2) geological studies, 3) surface and subsurface geophysical investigations, 4) trenching, 5) drilling and sampling, 6) use of monitoring wells, 7) penetrometer soundings, and 8) *in situ* testing. Several new site investigation technologies are being developed, including probes for detection and analysis of inorganic chemicals and hydrocarbons and *in situ* hydraulic conductivity probes (Stienstra and van Deen, 1994).

Special considerations that apply to investigating contaminated sites include: 1) avoiding sample contamination from the drilling and sampling equipment, 2) avoiding aquifer cross-contamination by proper drilling and abandonment techniques, and 3) health and safety issues.

Typically, a phased approach is used for site investigation, with separate feasibility phase and design phase investigations almost always performed. Additional investigations are sometimes undertaken during construction and post-construction periods.

Site investigation requirements during the feasibility phase include: 1) describing land use; 2) determining site topography; 3) evaluating construction equipment accessibility; 4) drilling at least two to four borings to obtain samples; 5) making soil classifications (the Unified Soil Classification System is preferred); 6) acquiring SPT N-values and/or CPT logs; 7) determining depth to groundwater; 8) evaluating the hydrogeologic properties of the subsurface materials; 9) assessing the likely extent of groundwater level fluctuations; 10) determining groundwater chemistry; 11) locating, identifying, and describing the bottom aquitard in situations where the wall will be keyed; and 12) assessing the suitability of *in situ* materials for reuse as backfill.

Site investigation activities during the design phase should be selected to provide more detailed information needed for final design and construction. In particular, the design phase investigation should include: 1) detailed topography along the wall alignment; 2) borings along the wall alignment spaced at 100- to 1000-foot intervals, depending on the subsurface variability and the length of the wall; 3) Standard Penetration Test blow count determinations (SPT N-values) with Unified Soil Classification System classifications of the soil samples at 5-foot intervals; 4) index property tests on the soil samples, including grain-size distributions, Atterberg limits, water contents, and unit weights; 5) identification of fractures and/or slickensides in clay strata for evaluation of their influence on open trench stability; 6) strength tests for situations in which open trench stability is an issue; 7) consolidation tests for situations in which adjacent ground movements after backfilling are an issue; 8) slurry loss testing in the boreholes if gravel/cobble/boulder zones are encountered; 9) representative samples for mix design and compatibility testing; and 10) structure, depth, thickness, hardness, hydraulic conductivity, etc., of the bottom aquitard in situations where the wall will be keyed. These data will permit evaluation of the impacts of the identified site conditions on construction of alternate containment barrier systems.

The samples and data obtained during site characterization studies should be saved for use during the construction and post-construction monitoring phases of a project. It is especially important that all relevant site data and soil property information be made available to contractors to assist them in bid preparation.

An integrated approach to site characterization is essential. This requires using complementary investigation approaches, including geophysical methods, penetrometer soundings, and conventional drilling and sampling. It is especially important that there be participation by all the relevant disciplines from the very beginning of the site characterization effort. Geologists, geohydrologists, geotechnical engineers, environmental engineers, and others must all be involved. In many past projects, the initial studies have focused only on characterizing the hydrogeologic regime and on identifying the nature and extent of subsurface contamination. While this is clearly very important, other important data that are needed for design and construction can and should be obtained during the early stages of the site characterization studies. The time and effort expended for the development of an integrated interdisciplinary site characterization plan can result in better design, greater construction efficiency, and less total cost to the owner.

There can be no cookbook approach to site characterization. Each site and problem is unique. The expertise of qualified professionals in several disciplines working together is needed to accomplish an appropriate site characterization. Since uncertainty in site characterization can never be eliminated, it is always important that the owner be made aware of the uncertainties and the possible deviations from expected conditions. The need to educate owners, regulators, and the public of the value and, in fact, the necessity of the observational method for site characterization requires continuing emphasis.

3.2.2 Analysis and Design

Clear understanding of the purpose and function of the cutoff wall must be developed prior to designing the wall and other associated components of a waste containment system. For example, a cutoff wall can be constructed to provide full containment, to provide underseepage control, to serve as a seepage diversion wall, or to be part of a temporary dewatering system. A cutoff wall can be either a hanging wall, or it can be keyed into an aquitard. The function of the wall is different in each of these cases, and these differences have impacts on wall design.

A difference in function is also illustrated by the following two hypothetical examples. In both examples, a cutoff wall is part of a complete containment system with a cap and a floor that together enclose an area contaminated with a concentrated, toxic material.

- In the first example, the containment system is operated with an inward gradient maintained by modest groundwater extraction from within the enclosed area. Such a system can limit contaminant fluxes out of the contained region to very small amounts owing to the opposing directions of advective and diffusive flows through the barrier. In such a system both the pumping rates and the treatment costs would be low. It should be noted that even though treatment of the extracted groundwater may be necessary, the groundwater extraction is considered to be a hydraulic control in this case. It is not part of a pump and treat remediation system, because the intent is not to restore a contaminated aquifer to a pristine condition.
- In the second example, the inward hydraulic gradient is not maintained. Instead, the contaminant is allowed to slowly move out of the contained area under the influence of hydraulic and chemical concentration gradients. Outside the contained area, the contaminant is at a low concentration, and natural attenuation processes are utilized to maintain concentrations at safe levels. This example illustrates use of a containment system to extend the range of situations over which the natural attenuation approach is applicable.

The foregoing examples point out that there should not be universally prescribed approaches for the design and operation of waste containment facilities. Once the design function is clearly identified and understood, the following aspects of cutoff wall analysis and design can be undertaken: establishing the wall geometry, considering backfill stresses and adjacent ground movements, selecting the backfill type and mix design, evaluating the suitability of potential construction methods, designing panel joints, evaluating costs, designing a monitoring system, and considering QA/QC requirements. A useful design checklist and reference list for soil-bentonite cutoff walls is available from the US Army Corps of Engineers (1995).

a. Establish Wall Geometry. The aspects of wall geometry that must be designed include alignment, depth, and thickness. Considerations in establishing the wall geometry include containment of the contamination, property ownership, and containment facility function.

The alignment is ordinarily chosen to contain the contaminant within its present boundaries. In some cases, however, contaminated soil from outside the enclosed area may be moved to inside the enclosed area. There may also be benefits in locating cutoff walls a significant distance beyond the present limits of the contamination, provided that property ownership and land use will allow it. These benefits include increased breakthrough time and decreased flux at steady state. However, the potential disadvantages include increased cost of the containment facility, increased volume of contaminated soil and groundwater, and increased land use requirements.

The required cutoff wall depth depends on the depth of contamination, the depth to a suitable bottom barrier layer, and the mode of operation of the facility. If the wall is to be keyed into an aquitard, an adequate embedment depth must be provided, and the seal at the joint between the wall and the aquitard must be evaluated.

Considerations in selection of the thickness of the cutoff wall include wall type, contaminant flux, wall continuity, construction method, and cost. The design basis for cutoff walls in contaminant containment applications should preferably be contaminant flux, rather than the single hydraulic conductivity criterion of $k < 1 \times 10^{-7}$ cm/sec that has been so extensively used in the past. When flux is considered, thick walls will generally be preferred to thin walls, because hydraulic and concentration gradients are lower and because there are more opportunities for chemical adsorption of contaminants by the wall materials. Thick walls may also be easier to construct without defects than thin walls. The practical range of wall thicknesses is often defined by the construction equipment and methods. For example, cutoff walls excavated using a backhoe or clamshell are usually 2 to 3 feet thick, with occasional walls as narrow as 1.5 feet and as thick as 5 feet.

b. Consider Adjacent Ground Deformations and Backfill Stresses. Potential impacts of vertical barrier wall construction on adjacent ground include instability and subsidence. These are more likely when long open trenches are excavated and when non-cementitious backfill is used. These risks should be evaluated by performing stability and stress-deformation analyses.

In addition to providing information about adjacent ground movements, stress-deformation analyses are also useful for estimating the consolidated backfill stress-state. Data compiled by Evans and Cooley (1993) clearly show that the stress state in soil-bentonite backfill has a strong influence on in-service hydraulic conductivity. Similar data for cement-bentonite materials is provided by Jefferis (1992) and Manassero et al (1995). In addition, contaminated permeants can increase the hydraulic conductivity of barrier materials, but their effect is less significant when the material is under high confining pressure (Acar et al., 1985; Daniel, 1987; and Mitchell and Madsen, 1987). Thus, the stress state in the trench has a critical influence on cutoff wall performance.

There are at least two mechanisms important in the development of the final consolidated backfill stress state in soil-bentonite cutoff walls: arching and lateral squeezing. Arching in soils has been described by Janssen (1895), Marston and Anderson (1913), Terzaghi (1945), Blight (1973), and Handy (1985) and has been applied to soil-bentonite backfilled trenches by Evans et al. (1995).

In arching theory, the trench walls are rigid and the backfill is compressible. Consolidation of the backfill results in the deformed shape shown schematically in Figure 3-6. This deformed shape mobilizes shear stresses at the trench walls, thereby providing partial support for the backfill and reducing vertical stresses in the backfill below overburden pressures. The magnitude of the trench wall shear stress depends on such factors as trench depth, trench width, position of the water table, and shear strength at the interface between the backfill and the trench wall. In arching theory, the major principal stress in the backfill at the trench centerline is vertical and the minor principal stress is horizontal. Principal stress directions rotate from this alignment for positions in the trench away from the centerline.

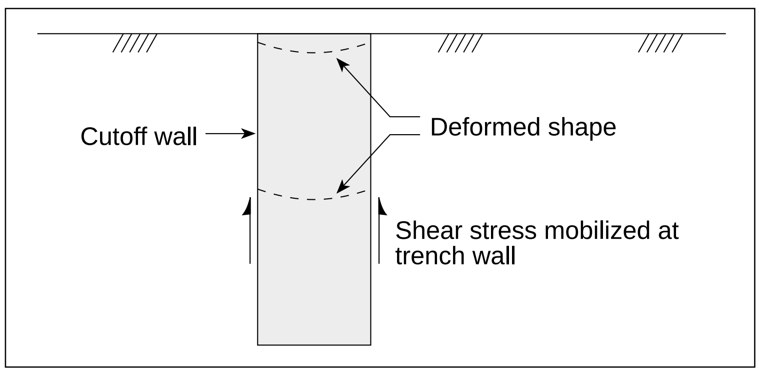

Figure 3-6 Schematic diagram of arching mechanism

The lateral squeezing mechanism for soil-bentonite backfilled trenches (Filz, 1996) is illustrated schematically in Figure 3-7. In this mechanism, the trench walls are not rigid, but can deform as construction proceeds: inward trench wall movement during excavation, possible rebound during backfill placement, and further inward movement during consolidation of the backfill. The backfill is placed as a low-strength slurry and, immediately after placement, excess pore water pressures are very high. As the excess pore water pressures dissipate, the backfill consolidates, and the trench walls move towards the trench centerline. The process can be visualized as if the backfill were being squeezed laterally between parallel plates. For narrow and deep trenches, a reasonable analogy would be to consider the trench backfill as if it were in a 1-D consolidation test laid on its side, with the consolidation load being applied by the inwardly moving trench walls. In the lateral squeezing mechanism, the major principal stress at the trench centerline is horizontal and the minor principal stress is vertical. When this mechanism comes into play, the magnitudes of the stresses in the consolidated soil-bentonite depend on the properties and initial stress state of the adjacent ground, in addition to the factors listed previously for the arching theory.

There is probably a transition and interaction between the two mechanisms, with arching dominant for shallow, wide trenches and lateral squeezing dominant for deep, narrow trenches. Preliminary calculations have been performed to estimate the magnitudes of the major (σ_1') and minor (σ_3') principal effective stresses at the 75-foot depth in the soil-bentonite backfill of a 100-foot deep cutoff wall constructed in a medium dense sand deposit. The results are shown as a function of trench width (B) in Figure 3-8. It can be seen in this figure that the lateral squeezing mechanism dominates for any realistic trench width, under the particular geometric and material assumptions of this example.

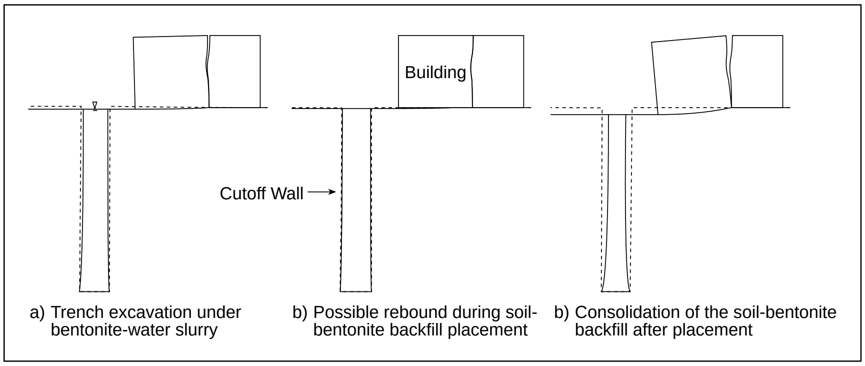

Figure 3-7 Schematic diagram of lateral squeezing mechanism

The sequence of events shown in Figure 3-7 also illustrates the kind of adjacent ground movements that can occur during and after cutoff wall construction, even when there is no risk of instability. The magnitude of adjacent ground movement depends, in part, on the compressibility of the trench backfill. As an example, a soil-bentonite cutoff wall was installed recently as part of a remediation scheme at a contaminated site in Silicon Valley, California. The cutoff wall passed within about 20 feet of an existing building, which was being used for a rather sensitive manufacturing operation at that time. Construction of the cutoff wall caused several inches of settlement, and the building was rendered unusable. A large lawsuit resulted. In this case, it may have been more appropriate to use a cementitious backfill and a panelized construction method.

c. Select Backfill Type and Mix Design. The following backfill types have been used for cutoff walls: soil-bentonite, cement-bentonite, soil-cement-bentonite, plastic concrete, concrete, and other mixtures of cement, bentonite, fly ash, slag, and natural clay. Soil-bentonite and cement-bentonite walls are most common in the USA; whereas, cement-bentonite and slag-cement-bentonite walls are widely used in Europe, and soil-cement-bentonite walls installed by deep mixing have been extensively used in Japan. New backfill materials are being developed to provide some treatment and/or adsorption of chemical contaminants, as well as containment. Selection of material type is influenced by cost, design function, compatibility with the contaminant, construction method, and impacts on adjacent ground.

Selection of the backfill type and backfill mix design are discussed in detail in Section 2, "Soil- and Cement-Based Materials for Vertical Barriers," where the primary focus is on contaminant transport, contaminant compatibility, and durability issues. There are, however, several additional design issues relating to material selection that must be considered. These issues concern stress-strain behavior of the backfill, density, fluidity, and filter criteria, and they have important influences on construction, post-construction performance of the wall, and adjacent ground movements during and after construction.

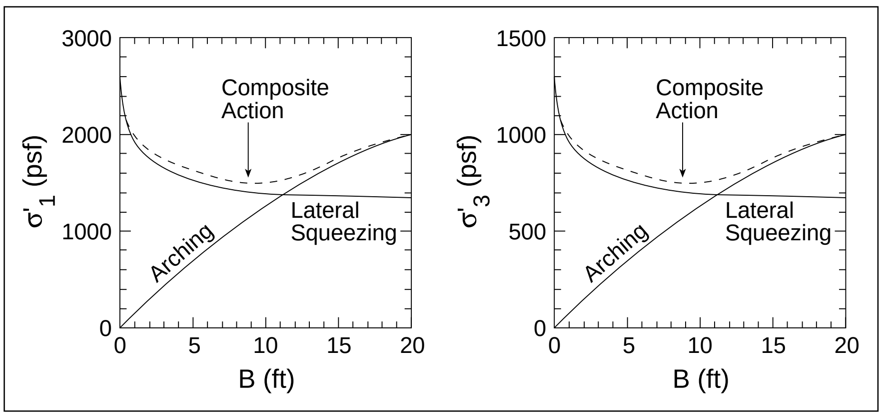

Figure 3-8 Preliminary calculations of the principal stresses in a soil-bentonite cutoff wall at depth 75 feet

As described above, the compressibility of soil-bentonite backfill materials has an important influence on the stress-state that develops in the consolidated backfill. The backfill stress-state, in turn, influences the hydraulic conductivity of the soil-bentonite, the resistance of the soil-bentonite to chemical attack, and the magnitude of adjacent ground deformations.

For walls subjected to movement, e.g., from nearby excavations, adjacent surcharge loads, or seismic shaking, ductile backfill materials can maintain low hydraulic conductivity better than brittle backfill materials, and a soil-bentonite backfill material might be favored over a cement-bentonite backfill. Conversely, if movements of the adjacent ground due to cutoff wall construction are of concern, a panel construction method with cementitious backfill might be favored.

The backfill must have a density sufficient to displace the excavation support slurry, and it should have sufficient fluidity to completely fill the excavated space without bridging across small openings.

The particle size distribution of a non-cementitious backfill should satisfy filter criteria so that it is internally stable and so that it will not be eroded out into the adjacent natural ground under the applied hydraulic gradients.

All of these backfill material issues should be considered during cutoff wall design. In some cases, the design criteria work in opposite directions, as indicated in the following examples:

- A high water content in a soil-bentonite backfill can have the desired effect of making the backfill more fluid, but can also make the backfill more compressible, which could lead to larger adjacent ground deformations.
- A high natural clay content in a backfill can decrease hydraulic conductivity and increase the potential for contaminant adsorption, but can also make the backfill less dense, which could make complete displacement of the excavation support slurry more difficult.
- A high cement content in a backfill will increase strength, but it can also make the barrier less ductile and more likely to crack.

d. Evaluate Suitability of Potential Construction Methods. The suitability of different cutoff wall construction methods depends on such factors as the type and hardness of the formation to be excavated, the required depth, the existence of natural and man-made obstructions, and the desirability of continuous versus panelized construction. For example, chisels or milling equipment may be necessary to excavate through hard materials or to deal with obstructions. Compared to panelized construction, continuous construction methods such as backhoe excavated trenches or the SoilSaw™ reduce the number of joints in the completed cutoff wall. Panelized walls constructed using clamshells, milling equipment, deep mixing, or jet grouting provide an opportunity to use cementitious backfill materials with short set-up times and high strengths. Panelized construction methods open only a small part of the ground at any one time, thereby reducing the potential for adjacent ground instability and movement problems.

e. Design Panel Joints. Depending on the panelized construction technique used, cold joints can be formed either with or without waterstops. For example, waterstops can be used on panels excavated with a clamshell, but not on panels constructed using deep mixing.

The consensus among European experts is that cold joints formed without waterstops during overlapping panel construction do not compromise the integrity or function of panel cutoff walls.

f. Evaluate Costs. A soil-bentonite cutoff wall constructed in a trench excavated by a backhoe is one of the least expensive types of cutoff wall. Costs can increase when cement is used and when other excavation or mixing techniques are employed.

g. Consider QA/QC Requirements. Because the designers have intimate familiarity with the assumptions, analyses, and details of the wall design, they should be involved in establishing the QA/QC requirements for construction.

3.2.3 Construction

a. Mixing Considerations. Almost all soil- and cement-based vertical barriers, except for cement-bentonite, require at least two mixing operations: one to mix the slurry and another to mix the backfill. Even in-situ mixing requires a slurry or grout mixing operation to supply the DM augers. Different mixing methods produce different mixtures, and different materials may require different mixers. As an example, a pugmill supplies a high mixing energy, but mixes only 1 to 2 yd^3 at a time and adds air to the backfill; whereas, bulldozer methods mix hundreds of cubic yards per batch. As another example, attapulgite clay cannot be mixed in ponds because it will settle; a very high energy mixer is needed to develop a high gel strength.

Mixtures containing cement require special consideration, because if they are disturbed after the start of the hardening process, the set will be lost. For soil-cement-bentonite mixtures, even the order of mixing the ingredients is important. Day (1995) cites the following example. Two mixes of sandy clay-bentonite-cement were prepared, each containing 4 percent bentonite and 10 percent cement. In the first, bentonite was mixed with water and added to the soil, with the cement then added dry. The 28-day unconfined compressive strength was 25 psi. In the other, the cement and water were added to the soil as a slurry, and then the bentonite was added dry. The 28-day compressive strength was 249 psi.

For the majority of slurry walls, mixing is done beside the trench. Remote mixing may be required when there is a limited mixing area available near the trench, when borrow is to be used as the backfill, when cement is an ingredient of the backfill, or when unusually strict quality control is necessary. Whatever mixing location is chosen, there must be adequate area available for material storage, equipment, and mixing. Guidelines for soil-bentonite walls are that the working platform should be as wide as the trench is deep, plus 10 ft. For deep mixing, the working platform should be as wide as the pretrench, plus the width of the DM crane. A slurry mixing plant area for cement-bentonite and deep mixing should be about 75 ft by 100 ft.

b. Construction Costs. Many factors influence construction costs including: construction method, backfill material, soil conditions, local labor and materials cost, length and depth of the barrier walls, working room and access, season and weather, contractor experience, and level of personal protection required. Some current installed barrier costs (Day, 1995), exclusive of mobilization, engineering, work platform, and disposal, are summarized in the table below.

WALL TYPE	WIDTH (FT)	DEPTH (FT)	UNIT COST ($/SF)	PRODUCTION RATE PER 10 HRS (SF)
Soil Bentonite	2 - 3	80	2 - 8	2500 - 15000
Cement Bentonite	2 - 3	80	5 - 18	1000 - 8000
Biopolymer Drain	2 - 3	70	7 - 25	1500 - 5000
Deep Mixing	2.5	90	6 - 15	1000 - 8000
DM Structural	2.5	90	15 - 30	1000 - 3000
Jet Grouting	1.5 - 3	200	30 - 80	300 - 2500
Grout Curtain	one row	200	40 - 100	200 - 1000

Another set of estimated costs is provided by Yang (1995):

MATERIAL TYPE	INSTALLATION METHOD	PRICE ($/SF)
Soil-Bentonite	Slurry Trench	3 - 8
Cement-Bentonite	Slurry Trench	6 - 14
Soil-Bentonite	*In Situ* Mixing	5 - 9
Soil-Cement	*In Situ* Mixing	6 - 14

For comparison, the following table provides information about vertical containment barrier costs in Europe (Esnault, 1995).

TYPE	COST ($/SF)	PRODUCTION RATE PER 10 HRS (SF)
Slurry/Backhoe	6 - 10	3000 - 5000
Slurry + Membrane/Backhoe	10 - 16	2000 - 3000
Slurry/Clamshell	16 - 20	1000 - 1500
Slurry + Membrane/Clamshell	18 - 22	800 - 1200
Plastic Concrete/Clamshell	25 - 30	1000 - 1500
Plastic Concrete/Cutter	50 - 80	
Vibratory Beam	3 - 5	3500 - 5000
Jet Grouting	15 - 30	2500 - 3500
Colmix (DM)	12 - 16	250 - 500

3.2.4 Specifications

The technical specifications, along with the drawings, are the means by which the design engineer communicates the location, dimensions, materials, and workmanship of the proposed construction to the contractor. If the specifications are inadequate, the desired integrity and performance of the barrier cannot be assured. The technical specifications may include sections covering the scope of the work, site conditions, required contractor qualifications, submittals, materials, equipment, workmanship, required quality control activities, and measurement and payment.

As more contractors enter the vertical barrier construction market, it becomes increasingly important to require specialized contractor experience. Good general contractors do not necessarily make good specialty contractors. The experience of the contractor's job site supervisor is especially important.

It is important to provide bidders, at least by reference, the data regarding subsurface conditions. As the time set aside for bid preparation is often very short in comparison with the time devoted to the site investigation and design phases, it is unreasonable to expect a bidder to conduct his or her own investigations during the bid period and to be aware of all the conditions that may influence constructability, quality, and costs.

Serious difficulties can occur when both performance (the end result) and procedures (the method) are specified for the same item, as unfortunately often happens. For example, the contractor may be provided a formulation for the barrier, told how to construct it, and then held responsible for the performance of the end product; e.g., attainment of a hydraulic conductivity less than a specified value. Large claims are likely in such cases. Nevertheless, both performance and procedure specifications can successfully be used for different items in the same specification. The objective should be to provide the most flexibility possible for the contractor, while maintaining sufficient control of the materials and construction that the design requirements and the owner's needs are satisfied.

Specifications for mixing are often vague on equipment and mixing quality. Both the designer and the contractor must be aware that different mixing methods can produce different results. In the case of mixing bentonite-water slurry for excavation support, low shear equipment requires use of storage ponds to complete hydration of the bentonite over a 24-hour period; whereas, use of a high shear, colloidal mixer may produce the same quality slurry in 5 minutes, and the 24-hour hydration requirement that exists in some specifications becomes unnecessary.

Most vertical barrier construction specifications will contain at least the following sections:

<u>Introduction</u>	Scope of Work
	Site Conditions
	Definitions

Qualifications	Experience of both the firm and the key personnel
Submittals	Work Procedures QA/QC Procedures
Materials	Requirements for the raw materials should be specified
Equipment	Equipment capabilities should be defined
Construction	Equipment Decontamination Site Work and Site Maintenance Location and Dimensions Methods of Slurry Mixing Slurry Backfill and Mix Properties Clean-up
Inspection and Testing	Should assign roles and responsibilities among contractor and owner/engineer
Health and Safety	An on-site health and safety officer is usually required
Measurement and Payment	For most projects, a unit area basis is used
References	Appropriate ASTM or other procedures should be cited

Examples of guide specifications for slurry walls are available from Evans (1995), the Institution of Civil Engineers (1995), and the US Army Corps of Engineers (1994). Additional information on specifications is presented by Millet et al. (1992).

3.3 FIELD PERFORMANCE OF THE TECHNOLOGY

3.3.1 Construction QA/QC

Construction quality assurance and quality control (QA/QC) activities are critical for success of a cutoff wall project. The purpose of quality control (QC) activities, which are the responsibility of the contractor, are to generate data that the contractor can use to make judgments about the suitability of materials and workmanship, and to modify operations if they are found to be inadequate. Quality assurance (QA), which is the responsibility of personnel independent of the contractor, provides the means by which the owner and the owner's engineer can verify that the contractor is satisfying the requirements specified in the contract documents. In order for quality assurance to be effective, the owner and/or the owner's engineer must possess the knowledge to identify, and the courage to reject, inferior work.

QA/QC activities can include the following types of tests and observations: tests of the excavation stabilization slurry, tests of the backfill mix, observations of the excavation alignment and depth, observations of the backfill mixing and placing operations, observations of ground movements adjacent to the trench, observations of overlap in panelized wall construction, and evaluation of the cutoff wall connection to the bottom aquitard and the overlying cap. Some items of particular importance to the QA/QC program include the following:

- Equipment must possess adequate size and capability to perform the work.
- The excavation support slurry must have sufficient density to support the excavation. It must not be so dense from suspended solids that the solids settle out or that the backfill is not easily capable of displacing the slurry.
- Constant vigilance is necessary with respect to the depth of excavation into the aquitard and clean-out of the key. A seemingly minor deviation can seriously compromise the effectiveness of the barrier. Continuous surveys of the bottom depth are necessary.
- Proper batching and mixing to create a homogeneous backfill are essential. QA/QC activities must include observation of the mixing process and testing of the backfill.
- Backfill placement must be accomplished so as not to entrap pockets of high hydraulic conductivity material or excavation support slurry. Continuous observations of backfill placement procedures and surveys of backfill slope are necessary.
- Post-construction surveys of backfill consolidation should be made. These can occur within a few days for cementitious backfill, but may need to be carried out over a period of several months for soil-bentonite backfill.
- The quality of the connection between the vertical barrier and the cap should be verified.
- Continuous observation of the cutoff wall construction should be made, and all irregularities should be noted on as-built drawings.

Additional description of important QA/QC activities is presented by Tamaro and Poletto (1992).

European and US QA/QC practices are similar in that both the contractor and another agency perform tests and observations of the construction. The specific tests performed are also similar.

Experience has shown that it is virtually impossible to obtain undisturbed samples of low-strength, cementitious backfill material because of the disruptions and stress changes associated with the sampling process. As a result, samples of such material obtained from the completed cutoff wall are likely to exhibit hydraulic conductivities more than an order of magnitude too high because of internal structural changes (Hollenweger and Martinenghi, 1994). Manassero (1994) has proposed use of the piezocone penetration test (CPTU) to assess the *in situ* hydraulic conductivity of cement-bentonite cutoff walls. This method shows promise, but it is still in the developmental stage.

An essential element of the QA operation is that continuity of personnel should be maintained throughout the life of a project. For example, the cutoff wall designer should observe construction activities to determine whether actual conditions deviate from design assumptions sufficiently that modifications to the original cutoff wall design are necessary.

The best assurance of high quality construction is to select qualified and experienced designers and contractors to do the work.

3.3.2 Performance Monitoring

Performance monitoring is essential to establish whether the barrier function of the wall is being met. Although expert consensus is that soil- and cement-based vertical barriers can achieve, and have achieved, a high degree of contaminant containment at many sites, there presently appear to exist fewer well-documented cases of satisfactory wall performance in environmental applications than would be desireable to establish widespread confidence in the technology. On the other hand, documented poor performance at some cutoff wall projects has lead to skepticism by some regulators and members of the public. Such skepticism can only be overcome by performance monitoring of cutoff walls that do their job as designed. Performance monitoring can also serve the following purposes: to provide an early warning system for release of contaminants, to discover when and where to implement corrective repairs, to obtain performance data for future use, and to reduce the costs of remedial activities.

There are important interactions among site characterization, design, construction, QA/QC activities, and performance monitoring. As one example, site characterization studies provide information that is essential for locating performance monitoring equipment. Another interaction is that both QA/QC activities and performance monitoring provide information useful to assess the integrity and effectiveness of a cutoff wall.

Performance monitoring of cutoff walls can include tracer tests, global or local pumping tests, continuity verification, contaminant concentration measurements, and periodic sampling and testing of the wall itself.

Tracers have been used in aquifer studies and other applications, but have been underutilized for cutoff wall performance monitoring due to lack of knowledge of existing tracer technology and insufficient research and development to overcome limitations of the technology. Some of the limitations relate to tracer toxicity, the ability of tracers to simulate contaminants, and the ability of tracer and sensor networks to reliably monitor the overall performance of the large surface area of a typical cutoff wall (Betsill and Gruebel, 1995).

Global pumping tests can be used to test the permittivity of an entire containment facility. While conceptually simple, there are significant difficulties involved in the geohydrologic interpretation of data from pumping tests that influence the vast regions of the subsurface that can be enclosed by cutoff walls. In addition, global pumping tests may not be very useful for determining defect locations.

Pump tests performed in small box-out enclosures improve the ease and reliability of barrier permittivity estimates along specific parts of the wall. However, the results of such tests apply directly only to the boxed-out region. The idea is that if the same construction procedures are used for the rest of the barrier, it should have the same permittivity as the barrier forming the box-out region. Such box-out tests are performed frequently in Europe, and it is recommended that they be used more often in the US.

Local pumping tests involve injecting or withdrawing water on one side of the wall and measuring the response on the other side of the wall (Eiben and Jefferis, 1994). A series of local pumping tests can provide useful information to help locate a defect. Like global pumping tests, the interpretation of local pumping tests is dependent on interpretation of geohydrologic conditions. Slug tests within barrier walls have produced results that are not reliable.

In principle, continuity verification can be accomplished by probing methods. However, a great deal of probing is required to obtain a high probability of encountering a randomly located defect. In addition, probing can potentially damage the completed wall.

Geophysical methods, while attractive from the standpoint of speed, low cost, and their non-intrusive or minimally intrusive character, suffer from imprecision in their ability to detect and locate defects and the ambiguity often associated with their interpretation. The conclusions from an evaluation of geophysical techniques done by Sandia National Laboratories (Borns, 1995) were that high-density networks of sensors, closely spaced vertical and/or directionally drilled access holes, and high frequency geophysical methods are needed to obtain the resolution (e.g., decimeter) needed for containment monitoring.

Contaminant concentration measurements made outside the contained area represent an attempt to directly monitor the ultimate objective of a containment facility, i.e., how much contaminant is actually escaping. While expected statistical variations imply that fixed values at any one location should not be specified, the general principal of monitoring contaminant concentrations outside the enclosed area should be supported. There are, however, very great challenges associated with installing a sensor network capable of reliably monitoring an entire cutoff wall. It should also be noted that there is a Catch 22 in this approach to monitoring, and that is that contamination has breached the barrier when it is detected, and the purpose of the barrier system is to inhibit contaminant escape in the first place.

The preceding considerations are applicable primarily to monitoring below the water table. At present there are no specific regulations under CERCLA or RCRA for vadose zone monitoring. Technical guidance for vadose zone monitoring currently is being developed for RCRA Subtitle C facilities (EPA, 1995) and by ASTM Section D-18.21.02.

The measurement of quantities in addition to chemical concentrations and groundwater conditions can provide valuable information about the condition of the wall and changes that may be occurring with time. These include weather conditions, surface water conditions and drainage patterns, gas accumulations, and temperature. Data on gas generation and temperature distributions, and their changes with time, provide indications of chemical and biological transformations, either of which could impact the barrier effectiveness. For example, high temperatures can adversely impact high water content clay-water mixes through drying and shrinkage cracking. Settlement and inclinometer measurements in the vicinity of slurry walls can provide evidence of ground movements that could impact wall integrity.

Long-term monitoring of a barrier wall by periodic sampling and testing of the wall itself, is, at first glance, an attractive approach for continued quality assurance. There are difficulties in this approach, however, in that the samples can only represent a very small fraction of the total wall volume, thus making it almost impossible to detect gross wall imperfections such as cracks, fissures, and holes. In addition, evidence is now strong that weakly cemented materials are sufficiently disturbed by the sampling process that internal expansion and microcracking can result in unrealistically high values of hydraulic conductivity being measured in subsequent laboratory tests, as mentioned earlier. According to Jefferis (1995), samples cored from a hardened wall are invariably so damaged as to be useless.

3.3.3 Long-Term Performance

Unfortunately, little field information is available that can be used as a basis for definitive conclusions about the long-term durability of soil- and cement-based vertical barrier walls in environmental applications. On the other hand, the results of long-term observations of soils and cement-treated soils in other field applications, as well as the results of chemical compatibility testing in the laboratory, provide some confidence that, except in highly aggressive chemical environments, or when subjected to repeated wetting-drying and freezing-thawing cycles, properly prepared containment wall materials may be able to maintain their barrier function over long time periods. Clays are very stable materials, and their properties are not significantly affected by dilute solutions of organic contaminants. However, these considerations in themselves are not sufficient to assure that initial design criteria will remain satisfied over the full design life of a barrier. Thus, there is continued need for long-term studies of the field performance of soil- and cement-based vertical barriers.

Jefferis (1995) summarized several important aspects of the durability of cement-bentonite walls. Water permeation causes leaching of lime, which in turn can result in weakening of the structure and consolidation, accompanied by a significant decrease in hydraulic conductivity. Sulfates can cause expansion of cement-bentonite which, under unconfined conditions, can result in disintegration of the material; however, under confined conditions, the expansive phases (ettringite and thaumasite) develop within the material rather than expanding it, with only limited effects on the hydraulic conductivity. Jefferis (1995) noted also that changes in hydraulic conductivity as a result of contaminant interaction under confined conditions may be limited to the order of a 10 to 50 fold increase, which while significant, may not be catastrophic. In some cases, interactions between waste and cement-bentonite may cause a decrease in hydraulic conductivity because of the precipitation of metals and carbonates in the high pH environment.

Mechanical strain and cracking of cement-bentonite walls caused by wall movements may be of concern because brittle failure can occur at low strains (0.2 to 2 percent) under confining pressures less than the unconfined compressive strength (Manassero et al, 1995). Jefferis (1995) points out, however, that cement-based materials also have the capacity for self-healing (autogenous healing) provided the cracking does not allow rapid water flow through the cracks. Drying of cement-bentonite walls is also to be avoided, as once dried, they will not swell on rewetting.

Standardized or accepted methods for assessing wall system durability are not currently available. The need for, and approaches to the development of, damage accumulation models are discussed by Inyang (1995), and additional work in this area is needed.

3.4 ASSESSMENT OF THE TECHNOLOGY

The expert consensus is that soil- and cement-based vertical barriers, if properly designed and constructed, can serve a very useful waste containment function. The U.S. Army Corps of Engineers (1995) has used soil- and cement-based vertical barriers extensively as cutoff walls in dams and levees, and has also been very successful in using these barriers for controlling pollutants, contaminated groundwater, and landfill leachate migrating from waste sites. Because of these successes, slurry walls have largely replaced traditional cutoff barriers such as steel sheet pile walls and grout curtain walls at hazardous waste sites. A successful application of a vertical barrier at a contaminated site in Kent, Washington, is described in US EPA (1995b).

Jefferis (1995) states that cement-bentonite cutoff walls are being installed in the UK at a rate of about 4 km per month, with most being for old landfill sites. These types of walls were first used there starting in the late 1960s, and no failures have been reported, even for walls in high water leaching environments. He notes further that mixes are being developed that will have hydraulic conductivities less than 1 x 10^{-11} m/s.

3.4.1 Applications

Soil- and cement-based vertical barriers are potentially applicable wherever it is necessary to contain, capture, and redirect the flow of clean or contaminated groundwater, vapor, and free phase liquids within the ground.

3.4.2 Limitations

Limitations of cutoff wall technologies are principally related to depth, obstructions, site access, ability to excavate the formation, reuse of the excavation spoils, exposure of the excavation spoils leading to contamination in another medium, uncertainties about the long-term properties and integrity of barrier walls, concern about potential incompatibility between the wastes and wall material, and lack of acceptance by regulatory agencies and the public. Most of these limitations can be addressed by proper design, construction, and monitoring, as well as documentation of successful case histories.

3.4.3 Reliability

Reliability issues can be classified in two groups: defects that occur during construction and changes that can occur with time. Defects occurring during construction result from inadequate mixing and placement of materials, and include holidays (gaps in the continuity of the wall), leakage at panel joints, leakage at the connection to the bottom aquitard, and the possible development of a gap between the cutoff wall and cap. Long-term effects can include property changes of the intact barrier material and structural changes, i.e., cracking, in the wall that could be caused by excessive ground movements, drying, and the like. Long-term property changes are primarily materials issues and are covered in Section 2. Design details can be implemented, however, to address long-term property changes caused by desiccation and freeze-thaw cycles, for example. If significant ground movements are anticipated, the wall material type should be selected to accommodate for them.

3.5 NEEDS

3.5.1 Needs for Research and Development

The most important research and development needs for improved acceptance of soil- and cement-based vertical barrier wall technology are in the areas of performance monitoring and evaluation of in-service walls. Important monitoring topics include the improved use of tracers and pumping tests, development of methods for continuity verification, application of new sensor technologies, and the development of geophysical methods that can provide the level of resolution needed for detection and isolation of leaks.

A particularly important need in the current regulatory environment is for reliable methods to determine the *in situ* hydraulic conductivity of the cutoff wall material. As noted previously, samples of cementitious material obtained from barriers after they have set exhibit hydraulic conductivity values that are too high, apparently due to sample structure disturbance that is not corrected by reapplication of *in situ* confining pressures. *In situ* slug tests have also been shown to be less than reliable, which may again be due to disturbance associated with the drilling process. Larger scale field pump test methods avoid disturbance problems, but they are costly and often difficult to interpret. It would be very useful to show that hydraulic conductivity values from laboratory cured specimens of backfill correlate well with *in situ* values, or to develop some other cost effective and reliable means for determining the *in situ* hydraulic conductivity of vertical barriers.

Important related research needs are development of improved QA/QC procedures and development of improved understanding of the long-term properties of barrier materials and damage accumulation under in-service conditions. Advances in both of these areas can increase confidence in the integrity and function of these types of cutoff walls.

Other areas of research need include improved site characterization tools and development of improved analysis procedures to estimate the backfill stress state, adjacent ground deformations, stress-strain behavior, and related hydraulic conductivity.

There is also need for a large-scale model user facility in which construction technologies could be evaluated, panel joints could be studied, and performance monitoring technologies could be developed and tested (Betsill, 1995).

Well-documented and well-monitored demonstration projects would be helpful in the conduct of these types of research and development activities, as well as in building increased confidence in the effectiveness of soil- and cement-based barrier walls as components of environmentally protective waste containment systems.

3.5.2 Needs for Technology Transfer

The major technology transfer need is the communication of well-documented, successful cutoff wall case histories to wall designers, to owners, to regulators, and to the public.

There is also need to communicate the factors that contribute to successful cutoff wall installations. Some of the more important items include the need for: 1) integrated site characterization studies that utilize experts from all the relevant disciplines, 2) cooperation and understanding among the owner, regulatory agency, contractor, and engineer, and 3) consideration of all the important design issues, which can include wall function, wall geometry, backfill type and mix design, contaminant flux, construction feasibility, trench stability, adjacent ground movements, costs, QA/QC requirements, and long-term performance and durability.

3.6 SUMMARY AND RECOMMENDATIONS

Soil- and cement-based vertical barriers are potentially applicable wherever it is necessary to contain, capture, and redirect the flow of clean or contaminated groundwater, vapor, and free phase liquids within the ground. The expert consensus is that these barriers, if properly designed and constructed, can serve very useful and environmentally protective containment functions at contaminated sites. Limitations of cutoff wall technologies are principally related to depth, obstructions, site access, ability to excavate the formation, reuse or disposal of the excavation spoils, exposure of the excavation spoils leading to contamination in another medium, uncertainties about the long-term properties and integrity of barrier walls, concern about potential incompatibility between the wastes and wall material, and lack of acceptance by regulatory agencies and the public. Most of these limitations can be addressed by proper site characterization, design, construction, and the monitoring of successful case histories.

An integrated approach to site characterization is essential. This requires using complementary investigation approaches and participation by all the relevant disciplines from the very beginning of the site characterization effort.

A clear understanding of the purpose and function of the cutoff wall must be developed prior to designing the wall and other associated components of a waste containment system. Important design issues that must be addressed include wall geometry, backfill type and mix design, contaminant flux, construction feasibility, trench stability, adjacent ground movements, costs, QA/QC requirements, and long-term performance and durability.

The quality of the technical specifications have an important influence on the quality of the completed construction. Specification writers must understand not only the technical objectives of the work, but also the construction processes involved. The specifications should provide, at least by reference, the data regarding subsurface conditions.

A soil-bentonite cutoff wall constructed in a trench excavated by a backhoe is one of the least expensive types of cutoff wall. Costs increase when cement is used and when other excavation or mixing techniques are employed.

Well-designed and defined construction quality assurance and quality control (QA/QC) activities are critical for success of a cutoff wall project. Constant vigilance on the part of the QA/QC personnel is necessary in order to achieve and document good quality construction. This objective is best accomplished by selecting qualified and experienced designers, contractors, and QA/QC personnel to do the work.

Performance monitoring is essential to establish whether the barrier function of the wall is being met. Performance monitoring of cutoff walls can include tracer tests, global or local pumping tests, continuity verification, contaminant concentration measurements, and periodic sampling and testing of the wall itself. There are advantages, disadvantages, and limitations to each approach, and further research is recommended to develop improved methods. A particularly important need in the current regulatory environment is for improved methods to determine the *in situ* hydraulic conductivity of the cutoff wall material. In the case of cementitious backfill, for example, experience has shown that it is virtually impossible to obtain undisturbed samples suitable for evaluating *in situ* hydraulic conductivity because of the disruptions and stress changes associated with the sampling process.

Unfortunately, little field information is available that can be used as a basis for definitive conclusions about the long-term durability of soil- and cement-based vertical barrier walls in environmental applications. However, indirect evidence for long-term durability from observations of soils and cement-treated soils in other field applications and from laboratory chemical compatibility testing suggest that these materials are well-suited for cutoff wall applications. The lack of field data from environmental applications highlights the need for long-term performance monitoring studies.

The major technology transfer need is for documentation of successful cutoff wall case histories. There is also need to communicate the factors that are essential for high quality cutoff wall design and construction to owners, designers, regulators, and the public.

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SECTION 4

VERTICAL BARRIERS: SHEET PILES

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4.1 BACKGROUND

Steel sheet pile walls have had limited application for waste containment due to concerns over leakage through the sheet pile interlocks (Rumer and Ryan, 1995). The amount of leakage is greater than typically acceptable for vertical barriers used for environmental containment applications. The principal advantage of sheet piling is its strength, which is useful for retaining earth and water. This high strength provides good resistance to hydrofracturing under high head conditions compared to other barrier materials; however, environmental containment barriers rarely are used in situations where hydrofracturing is a concern. Although the use of sheet piling in environmental applications is likely to be limited until the issue of interlock leakage can be solved, there may be circumstances in which acceptable subsurface containment of contaminants can be achieved using conventional sheet piles.

Interlock leakage was the first issue considered by the workshop panel convened to address the use of sheet piles for environmental containment applications. Attempts have been made to seal the interlocks on sheet pile barriers, and this has introduced the issue of chemical compatibility of the sealing material with the below ground environment.

The issue of corrosion in a potentially chemically aggressive environment has also been expressed as a concern with the use of steel sheet piling for long-term environmental containment applications.

Geologic conditions can limit the use of sheet piling. They cannot be driven reliably in soil deposits having cobbles and boulders. Driven sheet piles can carry contaminants from an upper water bearing zone into and through an aquitard during driving. This may also create a contaminant pathway along the soil/sheet pile interface.

As is the case with most vertical barrier systems, the ability to monitor the integrity of the wall, its depth of embedment into a confining layer or its seal with an underlying bedrock formation are of concern.

Finally, steel sheet piles are generally an expensive alternative compared to other vertical containment barrier types. It is primarily this feature that limits the use of steel sheet piling for containment applications in the United Kingdom.

4.2 STATE OF PRACTICE

4.2.1 General

Sheet pile walls have been used for civil engineering applications for many years and are used frequently in construction. They are a proven technology. However, there are some limitations associated with their use for environmental applications that owners should consider, such as a lack of regulatory acceptance, inadequate QA/QC procedures, high cost (this should include installation, material and disposal costs), and proprietary issues.

Nonetheless, sheet pile walls have many potential uses for environmental applications. They have been used to:

1) reduce leakage from reservoirs,
2) limit settlement of cohesive soils due to ground water seepage during excavation,
3) reduce construction dewatering efforts,
4) contain contaminated ground water,
5) control ground water/surface water interactions,
6) isolate subsurface zones for: performing pilot tests, applying remedial technology, preventing dilution of reagents and controlling ground water flow direction,
7) construct reactive walls (i.e., install the sheets, excavate, dewater, backfill with the reactive barrier material and remove the sheets), and
8) construct temporary enclosures during excavation of contaminated soils.

4.2.2 Interlock Leakage

In civil engineering applications, reducing leakage at sheet pile interlocks first became a concern in cellular cofferdam construction and earth retaining walls, before it became an issue in waste containment applications. The volume of leakage through the interlocks can be observed from within excavated cofferdams, but can not be observed directly in sheet piles that are buried on both sides, such as those used for containment barriers.

The volume of water passing through a cofferdam wall should not be taken as a measure of the volume expected through a sheet pile wall used for a containment application (Tulett, 1995). Retaining sheet pile walls flex from the water or lateral earth pressure applied to them. This flexure tightens the interlocks, making them more water resistant than they might be without these stresses applied, which is the case in cutoff wall containment applications.

One technique that has been used when water leakage through the interlocks of an offshore cellular cofferdam is excessive, is to drop flyash or bottom ash into the water outside the cofferdam at the leaking interlock location. Water flowing to the interlock carries the ash to the leaking location and the ash plugs the leak, Williams and Waite (1993).

Another means for sealing the interlocks on earth retaining sheet pile walls involves drilling holes outside the leaking interlock location and pumping grout into the hole. This approach has limitations in that the flow of grout can not be monitored or controlled, making verification of the integrity of the external grout seal difficult.

A further method for addressing the interlock leakage problem has been to place hot rolled sheet piles horizontally on the ground before driving and to fill the female interlock with an asphalt/grease mixture. The high viscosity of this sealant limits the volume of soil intruding into the interlock when the pile is driven; however, the sealant deforms when the male interlock is driven into it, forming a tight seal. Again, the integrity of the interlock seal can not be inspected *in situ*.

The inability to check interlock seals in environmental sheet pile walls has led to the development of sealable interlocks on sheet piles. One of the first type of sealable interlocks was formed by welding a steel angle section to the sheet pile near the female interlock (see Section A in Figure 4-1) prior to being driven. After driving both sheets, the void created by the angle section can be flushed clean and filled with a sealant.

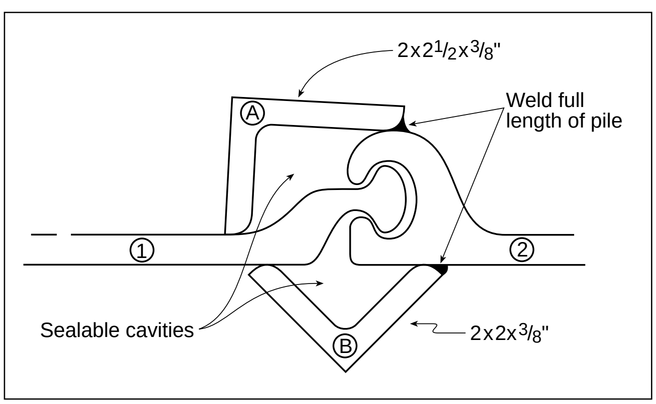

Figure 4-1 Steel angle section welded to sheet pile interlock

Researchers at the University of Waterloo recognized the uncertainty associated with interlock grouting and developed a sealable joint process (Waterloo Barrier™). One of the first experiments at the University of Waterloo involved welding angle sections to the female edges of hot rolled sheet pile interlocks, driving the sheets, washing the cavity left by the angle section, and grouting the cavity with a sealant. A foot plate was welded to the base of the angle section before the pile was driven to prevent intrusion of coarse grained soil particles inside the void formed by the angle section and the sheet pile. The male interlock was driven into the female interlock, and the angle section served to form a cavity around the interlock. The cavity left by the angle section could be filled with a sealant. This created some confidence that the grout would fill the interlock, thus improving the effectiveness of the sheet pile wall as a containment barrier. A second angle section (Section B, Figure 4-1) could be used to create a second sealable cavity, if necessary.

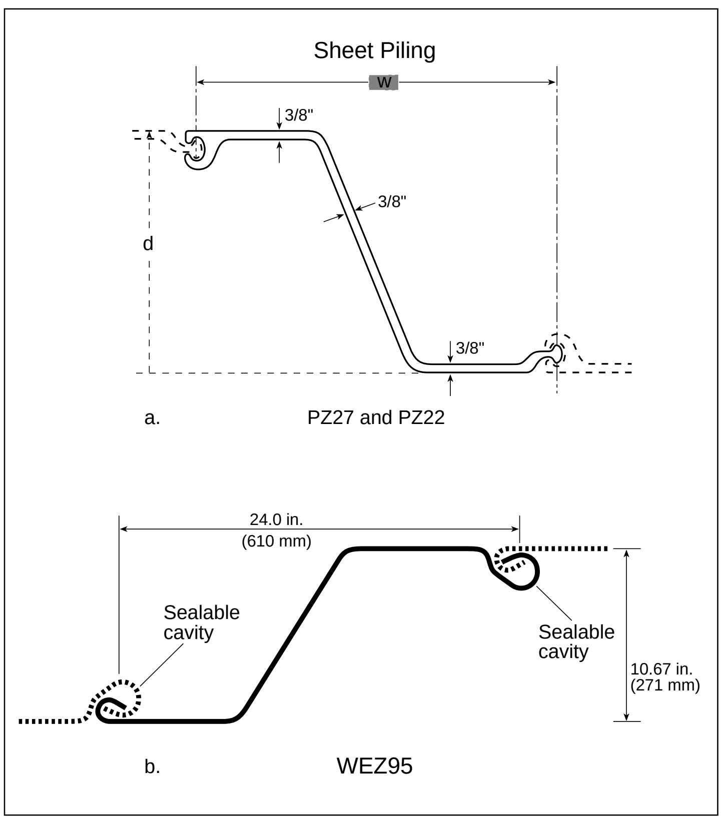

Figure 4-2 Hot-rolled and cold-rolled pile sections. (a) Bethlehem Steel hot-rolled sheet pile, (b) Canadian Metal Rolling Mills cold-rolled sheet pile

This approach led to the development of the Waterloo Barrier™. This system involves driving cold rolled steel sheet piling along the barrier alignment with standard pile driving equipment. The cold rolled sheets have modified loose fitting interlocks which differ from hot rolled sheets, as illustrated in Figure 4-2. This feature produces a small cavity in the interlocks when the piles are driven. Foot plates are attached to the tip of the female interlock to limit the intrusion of coarse grained soil particles into the interlock during driving (see Figure 4-3). The sheet pile is driven to the design depth. Next, the adjacent sheet (with male interlock) is driven into the female interlock, creating a continuous cavity for the length of the sheet piles.

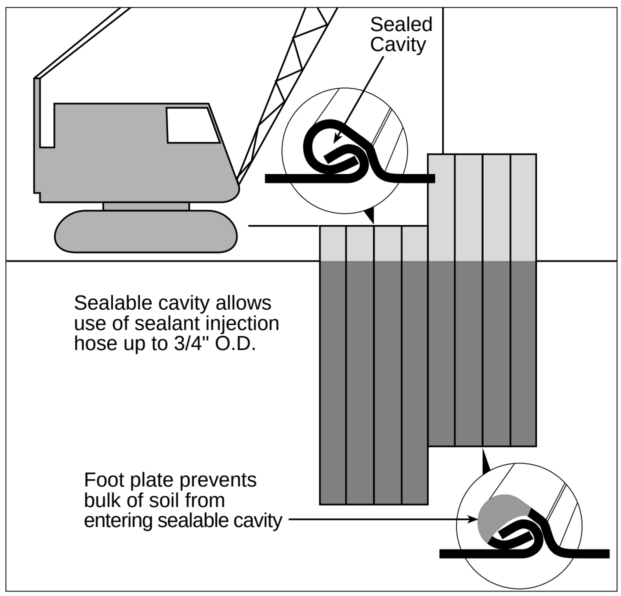

Figure 4-3 Bottom plug for a cold-rolled sheet pile interlock

The cavity is cleaned of finer soil particles by lowering a tube discharging high pressure water into the cavity. The turbidity of the wash water is observed and washing continues until the return water is clear when the pipe has been lowered to the base of the sheet pile. This indicates that the cavity had been cleaned. This interlock cavity can be inspected for defects and plumbness. Downhole cameras have been lowered into the interlock to observe the condition of the cleaned interlock. This can increase confidence concerning the integrity of the barrier installation. A sealant is then placed in the cavity using the tremie method to seal the interlock, see Figure 4-4. Figure 4-4 shows that a second cavity can be created by attaching an angle section to the female interlock.

The sealant type is selected to be compatible with the surrounding environment. The University of Waterloo has experimented with several sealant types, including clay based sealants, cement based sealants, Epoxy, Urethane and others. The criteria used for selecting the sealant include its permeability, pumpability, ability to withstand anticipated differences in hydraulic head across the barrier, ability to be removed, stability, chemical compatibility with contaminants and resistance to variations in temperature.

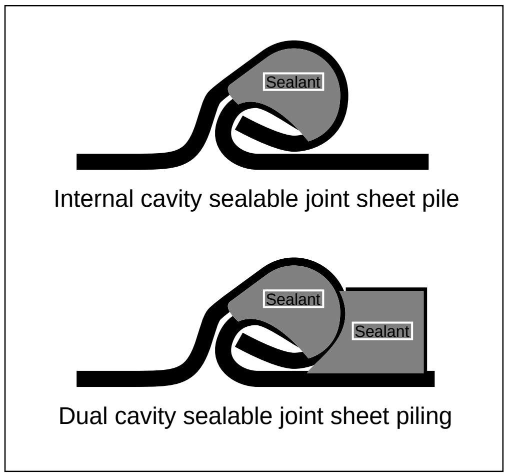

Figure 4-4 Water Barrier™ sealable interlock

Privett, et. al. (1994) describe a process to limit interlock leakage by joining two sheet piles together in the factory and driving the sections in pairs. This technique reduces the number of interlocks requiring field treatment (grouting, sealing, etc.) by a factor of two.

4.2.3 Corrosion

British Steel reports the results of studies on corrosion and protection of steel piling. Corrosion of steel sheet piling in undisturbed soil is considered negligible, regardless of the soil type. This is attributed to low oxygen levels. Piles extracted from aggressive environments (e.g., from a lagoon with a pH of 2.8) show insignificant corrosion. However, a large amount of corrosion is caused by atmospheric exposure, which affects above-ground retaining walls, and by exposure to sea water in marine applications.

Much of the experience with corrosion of steel piling has been gained in marine applications. Marine applications offer several different exposure conditions, each having different corrosion rates (see Figure 4-5). Very little corrosion occurs below the sea bed, where both sides of the sheet pile are exposed to soil. In the continuous sea immersion zone, i.e., the portion of the pile that is always below the water surface but above the sea bed, corrosion is normally retarded due to a blanket of protective marine growth and a lack of oxygen.

The portion of the pile between high and low tides, the so-called tidal zone, is also protected by biological growth, albeit a different type of growth from that occurring in the immersion zone. This biological growth acts to limit pile exposure to oxygen, thus reducing the rate of corrosion.

Highest corrosion rates occur where oxygen is most abundant, i.e., at the low water zone and the splash zone. British Steel recommends design corrosion rates for the various conditions shown in Figure 4-5. The recommended corrosion rates are shown in Table 4-1. British Steel concludes that sheetpile corrosion in undisturbed soils is very slight and recommends a maximum corrosion rate of 0.015 mm/side per year.

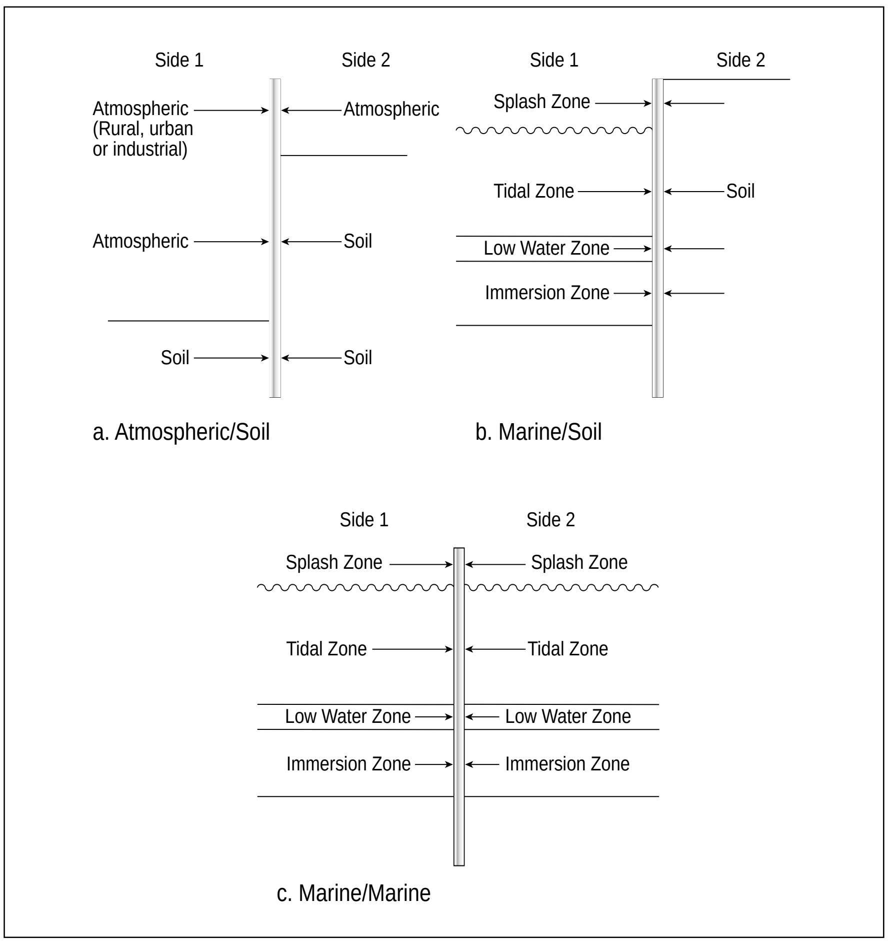

Figure 4-5 Different corrosion environments for sheet piles (British Steel, 1995)

TABLE 4-1 Recommended Corrosion Rates for Steel Surfaces (from British Steel)

Pile Face	Environment						
	Fig. 4-5a	Fig. 4-5a	Fig. 4-5c	Fig. 4-5b	Fig. 4-5c	Fig. 4-5b	Fig. 4-5a
Side 1	Atmospheric	Atmospheric	Splash or Low Water	Splash or Low Water	Tidal or Immersion	Tidal or Immersion	Soil
Side 2	Atmospheric	Soil	Splash or Low Water	Soil	Tidal or Immersion	Soil	Soil
Corrosion* Rates (mm/year)	0.07 (mean)	0.05 (mean)	0.15 (mean)	0.09 (mean)	0.07 (mean)	0.05 (mean)	0.03 (mean)
*Represents total loss of section (Side 1 + Side 2)							

Notes to Table 4-1

1. The corrosion rates quoted are based on investigations carried out by British Steel and others on steel exposed in temperate climates. For most environments, mean values are quoted since they are considered to be most relevant to the design and performance of most sheet piling structures. However, in some circumstances, the designer may wish to take account of higher values. It is suggested that in these circumstances, a reasonable practical upper corrosion rate limit would be that corresponding to a 95% probability value. The mean 95% probability values for various combinations of UK environments are given as follows:

Pile Face	Environment			
	Fig. 4-5c	Fig. 4-5b	Fig. 4-5c	Fig. 4-5b
Side 1	Splash or Low Water	Splash or Low Water	Tidal or Immersion	Tidal or Immersion
Side 2	Splash or Low Water	Soil	Tidal or Immersion	Soil
Corrosion Rate (mm/year) Mean 95% Probability Value Total Loss of Section	0.33	0.18	0.19	0.11

2. For combinations of environments where low water corrosion is involved, in a small number of locations, higher rates than those quoted have been observed at or just below the low water level mark.
3. A maximum value is quoted for soil corrosion and this applies to natural undisturbed soil or well compacted and weathered fill ground where corrosion rates are very low. Recent fill ground or waste tips will require special consideration.
4. Corrosion losses due to fresh water immersion are generally lower than for seawater, however, fresh waters are very variable and general advice can be given to quantify the increase in life.

British Steel lists three methods for extending the design life of steel placed below ground: 1) using a heavier section, 2) using high yield steel, and 3) using organic coatings. Steel piles are normally coated under controlled environment conditions (such as in a fabricating shop), and the applied coating should be damage resistant for transportation and handling. A simple coal-tar pitch mixture has been used for some time, but the coating is soft, thin (up to about 50 microns) and easily damaged. Synthetic resins have been added to the coal-tar to obtain thicker (100 to 250 microns) and harder coatings. British Steel has produced three other coatings: tar vinyl (PC1) which is an aromatic pitch modified with vinyl resins; high-build isocyanate cured epoxy pitch (PC2) which is a coal tar pitch modified with epoxy resins that make it harder than PC1; and colored, vinyl ether primer/finish (PC3).

Coal Tar Epoxy coating has been used in the United States for many years and has been demonstrated to be effective for resisting corrosion in difficult environments, such as in marine applications. Steel sheet piles treated with this process and installed in marine environments 35 years ago have been exhumed recently and have shown no visible defects. This favorable performance attests to both the survivability of the coating during the driving process and its long-term resistance in the marine environment.

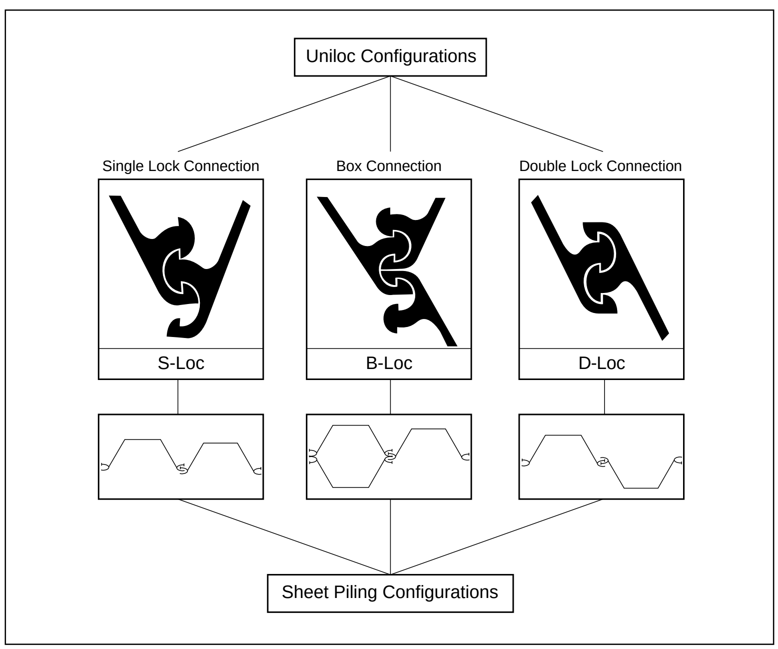

Figure 4-6 Uniloc interlock configurations

The U.S. Army Corps of Engineers has a specification C-200 for Coal Tar Epoxy. Piles coated with this material have shown resistance to chemical solutions, hydrocarbons, acids, alkalis, and alcohols. Newer products have been introduced, but none have the proven performance of Coal Tar Epoxy.

Cathodic protection can be provided to extend the life of underground steel structures, but this is a complicated process that requires on-going maintenance, and it may be operationally expensive.

The concern for steel sheet pile corrosion in a chemically aggressive environment has lead to the development of the "Uniloc" high density polyethylene sheet pile by Spencer White & Prentis, DuPont and Polymer Construction Products, Ltd. The material used is DuPont's Zemid PG which is highly resistant to chemical degradation. The Uniloc system consists of trapezoidal-shaped sheet pile sections having a 3/8 inch wall thickness. The interlocks differ from those found on steel sheet piles (see Figure 4-6) in that the flow path through them is longer, thus producing a lower leakage rate for the same hydraulic head difference. The technique for installing Uniloc sheets is under development. A hydrostatic pile driver may be used. This would involve a hydraulic piston alone or a piston with a vibratory hammer (in conjunction with water jet techniques).

In addition to the Uniloc system, there are other products being advertised, such as the GeoGuard™ vinyl sheet piling, which is promoted as a corrosion resistant sheet piling.

4.2.4 Sealant Compatibility

For sheet pile barriers with sealants, the sealant used to treat the interlocks will depend on site-specific circumstances, and it must be compatible with the anticipated subsurface environmental conditions. Candidate sealants include clay based (bentonite and attapulgite) grouts, cement-based grouts, epoxy polymers, urethane polymers, and others. If field circumstances dictate, different sealants can be used at different elevations within the interlock. The sealant selected must also be compatible with the hydraulic head applied, must be pumpable, have a satisfactory permeability once placed, and must have thermal expansion properties compatible with the interlock. The sealant material selected may influence the cost of a project; however, because the volume of the sealable cavity is small relative to the size of the wall, it may be feasible to select effective sealants for sheet pile barrier wall interlocks that would be impractical or too expensive for other types of barrier walls, e.g., slurry walls.

4.2.5 Construction Monitoring

A detailed subsurface characterization of the site is necessary for monitoring the key of a sheet pile wall into an aquitard. This includes making test borings along the barrier alignment at intervals necessary to fully define changes in subsurface conditions.

Another method to verify that the sheet piles are keyed into the appropriate aquitard calls for monitoring the effort required to advance the piles with depth. Advancement rates differ in different soil types. Monitoring the driving rate (blow counts) of impact-type pile hammers provides quantifiable data; whereas, monitoring vibratory hammer performance is more qualitative.

On occasion, downhole cameras have been dropped into the interlock (after cleaning) to observe its condition. In situations where the integrity of the sheet may be jeopardized during placement, observation tubes can be welded to the sides of selected sheet piles (between interlocks) prior to placement, thus providing a means for later inspection of the sheet.

4.2.6 Costs

Costs for sheet pile barriers depend on many factors, e.g., the distance to mobilize equipment, size of the project, type of sealant used, the use of a coating, special installation conditions, weight of the section, etc. Approximate costs range from about $15 U.S. to $40 U.S. per square foot for the sealable interlock joint sheet pile wall (Smyth, et. al., 1995).

4.3 FIELD PERFORMANCE OF THE TECHNOLOGY

The bulk hydraulic conductivity of sheet pile barriers is an appropriate property to use in comparing sheet pile barriers to other barrier types, because this property considers the discontinuities in the sheet pile wall. Field studies of sheet pile barriers report this property.

Another appropriate property is the barrier permittivity (the ratio of hydraulic conductivity to the barrier thickness), because this considers not only the bulk hydraulic conductivity, but the actual potential for flow through the barrier, i.e., the flux. For example, a soil-bentonite wall, two feet thick, with a hydraulic conductivity of 10^{-7} cm/s has a permittivity of 1.6×10^{-9} sec^{-1}. The bulk hydraulic conductivity of a sheet pile wall might be 10^{-9} cm/s (two orders of magnitude less than the soil-bentonite wall), but the permittivity is 1×10^{-9} sec^{-1}, which is comparable to the soil-bentonite wall.

The University of Waterloo has conducted field tests on more than 20 research test cells and on commercial installations. Hydraulic tests have been undertaken on concentric test cells that extended into an underlying aquitard to measure the bulk hydraulic conductivity of the sheet pile wall. In these cells, a constant head of water was maintained in the moat between the two cell walls and a higher head was applied within the interior cell (see Figure 4-7). The difference between the two heads was monitored with time.

One test case was discussed by Smyth, et. al. (1995) in which a bentonite slurry was used as the interlock sealant. Accounting for evaporation and assuming that no seepage occurred through the aquitard, the bulk hydraulic conductivity of the cell double-wall system was calculated to be 6×10^{-9} cm/

sec. An organic polymer sealant when similarly tested resulted in a bulk hydraulic conductivity of less than $1x10^{-9}$ cm/sec. If these walls were 0.375 inches thick, their permittivity would be 6.3 x 10^{-9} sec^{-1} and 1 x 10^{-9} sec^{-1}, respectively.

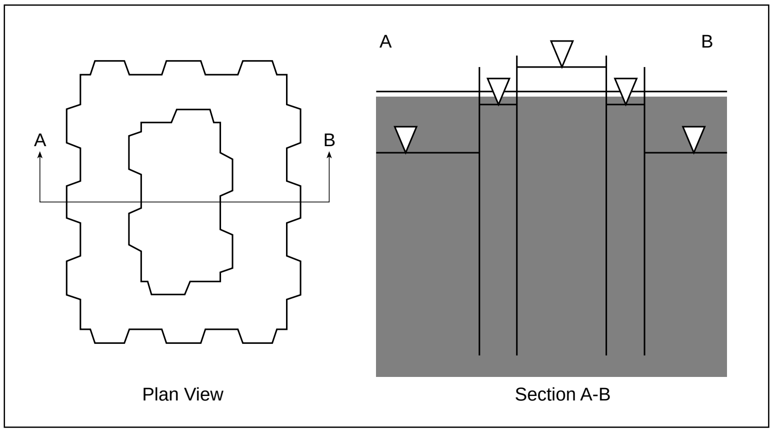

Figure 4-7 Plan and cross section of Waterloo barrier test cell for hydraulic conductivity (Smyth, et al., 1995)

A field test program was executed for DOW Chemical Company in Freeport Texas, ENSR (1993). In this field test, Arbed AZ18 piles were selected to withstand the difficult driving conditions. These sheet piles did not have sealed interlocks, but the interlock configuration differs from the Bethlehem Steel interlock shown in Figure 4-2. The Arbed interlock is illustrated in Figure 4-8.

To conduct the field test, a pumping well was installed (with four observation wells) and pumped to measure the hydraulic characteristics of a silty sand deposit that overlayed a clay deposit. A sheet pile enclosure, 50 feet x 50 feet, was constructed around the pumping well and additional observation wells were installed inside and outside the test facility. A second pump test was undertaken to measure the effects of the sheet pile enclosure. The data indicated that the sheet pile barrier had an estimated permeability of $7x10^{-7}$ cm/sec. Since the piles were driven into a bottom clay deposit, the observed leakage was assumed to have flowed through the sheet piles and not under the piles.

A separate study was undertaken to address the concern for the potential of flow along the sheet pile/aquitard interface, Hayman, et. al. (1993). In this case, a laboratory study was initiated to explore the potential for flow along the interface of piles penetrating an unconfined aquifer overlying an aquitard and a confined aquifer. This study was carried out to simulate conditions in which piles are driven through interbedded sand and clay strata and to determine the potential for creating a pathway for contaminant migration.

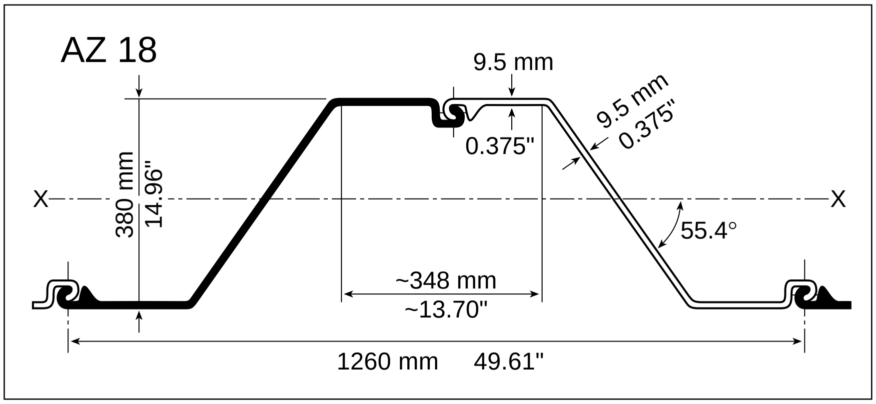

Figure 4-8 Arbed sheet pile interlock

Test vessels were constructed with a sand layer over a clay layer which was over another sand layer. The researchers saturated the lower sand layer with deionized water and then consolidated the clay layer to represent a selected overburden pressure. The upper sand layer was saturated with a dense non-aqueous phase liquid (DNAPL) and a pile was driven through the soil layers. The deionized water in the lower layer was purged and replaced with another volume of deionized water. The first volume was sampled and analyzed for the DNAPL. This purging and sampling process was repeated with time after completion of the pile driving.

It was found that DNAPL was carried down with the pile during driving, since DNAPL was detected in the initial samplings of the deionized water in the lower layer. However, the DNAPL concentration decreased in subsequent samples, indicating that the DNAPL was being purged from the lower sand layer. It was concluded that DNAPL was introduced into the lower layer only during driving and that there was no detectable leakage along the pile/ clay interface after the sheet pile had been placed.

4.4 ASSESSMENT OF THE TECHNOLOGY

4.4.1 General

As with any technology, there are advantages and limitations associated with the use of steel sheet piles for environmental containment applications.

4.4.2 Advantages

1. No excavation required - Sheet piles are driven into the ground and (usually) excavation of potentially contaminated soils is not required.

2. Minimum waste disposal - Since no excavation is required for construction of the sheet pile barrier, disposal of excavated contaminated soil is not required. The cleansing of interlocks prior to sealing produces potentially contaminated water which must disposed of properly.

3. Reduced health and safety concerns - Since there is no excavation of potentially contaminated soils, the risk of exposure to contaminated soils is lessened, with a correspondingly reduced concern for worker health and safety.

4. Joints can be resealed (depending on the sealant used) - If there is concern for the integrity of a sealed interlock, some sealing materials can be removed and the process repeated to provide a better seal.

5. Sheet piles can be removed later if required - Some environmental remediation projects require temporary containment. Sheet piles can be removed using conventional techniques to restore the site to the pre-construction condition.

6. Standard construction equipment can be used - Sheet piles have been driven for many years. This technology uses standard equipment to install the barrier.

7. Rapid installation - Since sheet piling involves standard procedures and equipment, installation can be rapid.

8. Topography and depth to water have little impact - These site-specific features affect the construction of many barrier types. For example, the ground surface must be flat to construct a slurry wall and the ground surface usually must be above the ground water level to assure trench stability. This may involve raising ground surface elevations along the barrier alignment to provide these conditions. These conditions are not required for construction of sheet pile barriers.

9. Diffusive transport through the barrier is greatly reduced - Properly sealed steel sheet piles are barriers to diffusion, which may be an important contaminant transport mechanism. Furthermore, they do not sorb contaminants.

10. Irregular enclosure shapes are possible - This offers an advantage when constructing a containment system near buildings, utilities, and at small properties.

11. Costs may be reduced - Many other barrier types are installed by specialty contractors. Construction at sites located some distance from these contractor's

locations will incur expensive mobilization costs. Since sheet piling installation is a common technology, many local contractors can reliably construct this barrier type without the expensive mobilization costs. Additionally, smaller walls can be cost-competitive with other barrier wall types. Many wall types require construction of temporary facilities that increase the unit prices on small projects (fixed costs divided by a small quantity). An example is the construction of slurry ponds for a slurry wall. These temporary facilities are not required for sheet piling and can make them cost competitive, especially on smaller projects.

4.4.3 Limitations

1. Noise and vibration - Sheet piles are driven with either vibratory or impact hammers that have associated side effects. Impact hammers create noise and vibratory hammers impart vibrations to the ground that can densify loose, saturated sand deposits resulting in settlement of the ground surface. The vibrations can also cause damage to adjacent structures, depending on their location, construction type, and proximity to the vibration source.

2. Not suitable for stiff clay or soils containing cobbles and boulders - Boulders and cobble deposits resist penetration by sheet piles regardless of the hammer type used. Predrilling may be necessary for advancing piles through these soil types. Stiff clay deposits can also introduce difficulty for driving piles. The potential for the piles fissuring stiff clay deposits should be evaluated. Vibratory pile hammers are not usually suitable for driving through clay soil deposits.

3. Keying into rock is not possible - Most rock deposits are too hard for sheet piles to penetrate, making keying into rock impractical.

4. Depth penetration limited - There are limitations regarding the depth that sheet piles can be driven. The maximum depth depends on the soil conditions, hammer type and the sheet pile section. It commonly is about 30 to 45 meters, Smyth, et. al. (1995). Predrilling along the alignment can be undertaken to extend this, but this will add to the project cost.

5. Some technologies are proprietary - Proprietary information sometimes introduces difficulties in competitive bidding situations.

4.5 NEEDS

There is a need to collect information on the long-term performance of sheet pile walls used as barriers in containment applications. The potential for corrosion of the steel requires further evaluation of the damage to coatings caused during installation and from exposure to different chemicals encountered at waste sites.

The long-term stability of interlock sealants needs to be studied, both with regard to exposure to different chemicals and to the effect of fluctuating hydraulic gradients.

Published case histories and demonstration results with accurate performance measurements are needed before this technology will gain acceptance by regulatory agencies. Several projects have been implemented and others are planned, so a better prognosis may be available in the near future. Quality control procedures need to be developed to measure the bulk hydraulic conductivity and to establish the locations of defects in the field. However, several of the quality control procedures currently available for sealable joint sheet piling are as rigorous or better than some of the quality control measures applied to other barrier wall construction techniques.

4.6 SUMMARY/RECOMMENDATIONS

Field demonstrations using both conventional and sealable joint sheet piling have shown that sheet piling is a viable barrier option in containment applications. Although not appropriate for all barrier applications, sheet piling is worthy of consideration in the project planning stage because it may be the appropriate technology.

Leakage through the interlocks of conventional sheet piling is piling dependent, as the joint tightness varies among different pile types. Concerns about leakage decrease if the containment wall is used with other remediation techniques, such as pump and treat ground water systems.

Some project experience is available, and the results are promising. Longer term trials at contaminated sites are required. These should involve both hydraulic monitoring and observations of contaminant migration in the evaluation of containment performance.

Monitoring of sites on a large scale is necessary to develop data on the success of this technique. This will help to make identification of gross leakage possible, but detection of small leaks will continue to be difficult, as is the case with all barrier wall types.

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SECTION 5

VERTICAL BARRIERS: GEOMEMBRANES

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5.1 BACKGROUND

Background information on geomembranes for use as vertical barriers in waste containment facilities is summarized in the following paragraphs.

5.1.1 Overview and General Schematics

The essential reason for using a geomembrane as a vertical barrier is to assure complete continuity by means of an extremely low permeability material. Such continuity and relative impermeability might be compromised in soil-bentonite (SB), soil-cement (SC), cement-bentonite (CB), soil-cement-bentonite (SCB) walls and other related vertical barriers due to the occurrence of unobservable factors during construction, such as;

- collapse of trench walls during construction or backfilling,
- sand deposits during pauses in backfilling,
- sand or debris on the bottom of slurry supported trenches leading to ineffective embedment and
- improperly mixed backfill material containing granular soil or large obstructions, e.g., rock or construction debris.

Furthermore, the low permeability of SB, SC, CB or SCB materials and other related vertical walltypes might be further compromised due to factors such as;

- changes in the quality of backfill material during field mixing,
- discontinuities at joints and locations of work stoppage during the backfilling process,
- drying and desiccation above the water level or during water table fluctuations,
- freeze and thaw cycling in the upper portions of the wall, and
- chemical incompatibility with the leachate that the site contains.

Geomembranes have been used as vertical barriers either alone or in conjunction with other relatively impermeable materials for approximately 10 years, with the earliest attempts being made in 1980. Applications of the technology have included the following;

- the geomembrane by itself,
- the geomembrane with SB, SC, CB or SCB backfill forming a composite barrier, and
- a double geomembrane with a sand leak detection layer between.

The technology is also available to provide the following;

- a double geomembrane with a geonet leak detection layer between, and
- a double geomembrane with geonet between backfilled with SB, SC, CB or SCB forming a RCRA Subtitle "C"-type liner system used as a vertical wall.

Geomembranes used for vertical walls are almost always made from high density polyethylene (HDPE); however, other polymers can also be used. The geomembrane sheets can be continuous, but usually finite length panels interlocked by a number of possible connections are used. The depth of the final wall can be essentially unlimited, but the design depth will dictate the method of installation. A number of different installation procedures are available.

The top of the geomembrane sheets in the vertical wall can be seamed to the geomembrane in the containment cover making a mechanical seal as shown in Figure 5-1(a). Such a system can be monitored using conventional CQC/CQA procedures. Alternatively, the geomembrane in the cover can be extended horizontally or dropped down vertically making an overlap seal with respect to the geomembrane in the vertical wall. This is the more common arrangement, as shown in Figure 5-1(b).

The toe of the geomembrane sheets can be keyed into an aquitard (if one is available) or can be taken to an adequate depth where under-seepage becomes negligible per the site-specific design, as depicted in Figures 5-2(a) and 5-2(b), respectively. The latter case is frequently referred to as a "hanging wall".

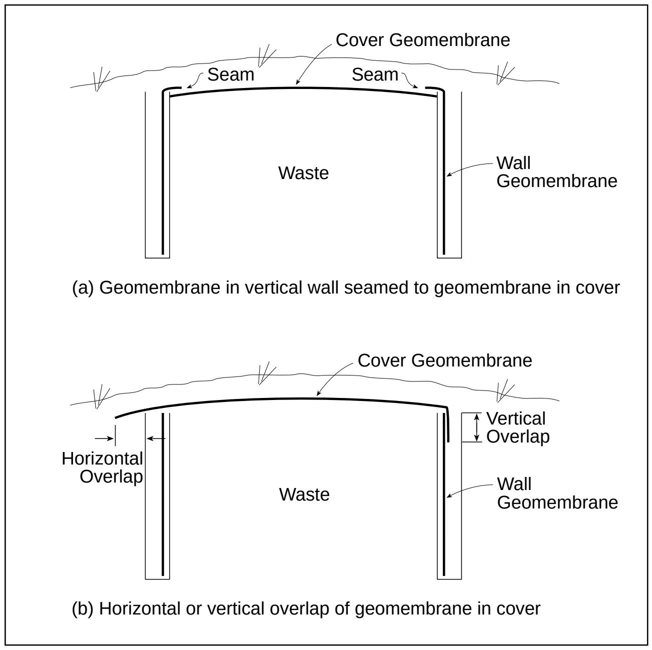

Figure 5-1 Termination of the top of geomembrane vertical barriers

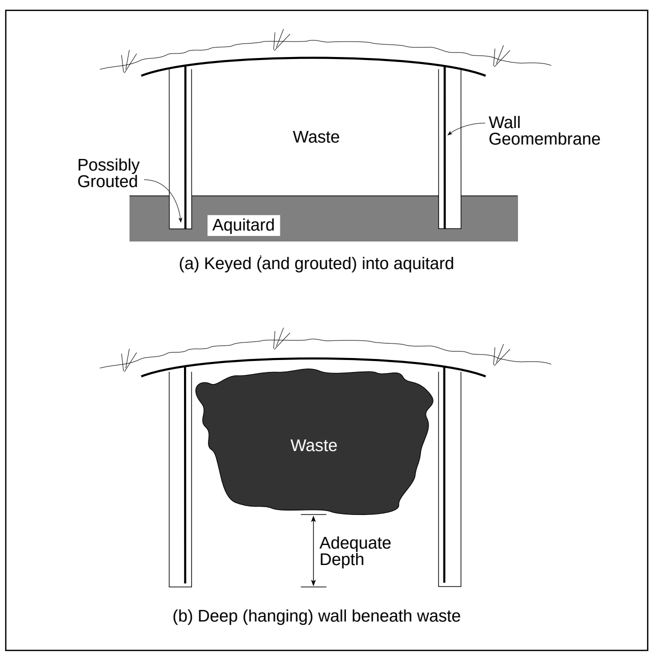

Figure 5-2 Termination of toe (bottom) of geomembrane vertical barriers

Vertical barrier walls with geomembranes are available in a number of different forms and have been used to the point where their technical feasibility is well established. CQC and CQA are regularly practiced during the installation. The state-of-the-practice, field performance, assessment of the technology, needs for further implementation, and references that are available in the open literature are presented in this section.

5.1.2 Types of Geomembranes Used

Essentially all barrier walls constructed to date using geomembranes have been made from high density polyethylene (HDPE). In this regard, HDPE is a logical polymer choice for the following reasons;

- HDPE is chemically resistant to a wide variety of chemicals, including organic solvents,
- HDPE is readily extruded into sheets and into detailed connection fittings for joining the sheets together,
- the sheets and the connection fittings can be seamed together in the factory or in the field using readily available welding equipment. They are then referred to as "panels",
- variable sheet thicknesses, panel widths, and panel lengths can be manufactured,
- the cost of HDPE is currently very low, traditionally being the least expensive polymer of all geomembrane polymer types, and
- a sufficient number of commercial systems are available, such that a generic specification can be generated and bid upon accordingly.

HDPE geomembranes consist of a formulation of three components as shown in Table 5-1. The final geomembrane density is typically 0.941 to 0.943 g/cc, placing it in the high density category of linear polyethylenes; hence, its identification as HDPE.

TABLE 5-1 Typical HDPE Formulation

Component	Density (g/cc)	Percentage (by weight)	Comments
resin	0.934 to 0.940	97 - 97.5	virgin resin, i.e., no post consumer recycled resin
carbon black	2.5 to 2.8	2.0 - 2.5	for ultraviolet stability and durability
antioxidants	1.5 to 2.5	1.0 - 0.5	for processing and long-term durability

While stiff with respect to physical handling, HDPE sheets or panels can not be directly driven into native soil using conventional pile driving equipment. Even vibratory pile hammers would deliver excessive stresses to the sheeting causing buckling, folding or tearing of the geomembrane.

If direct driving is a required feature, a stiffer or (higher modulus) polymer is required. Polyvinyl chloride (PVC) has been used in this regard. Several PVC sheet products, replicating steel sheet piling, have appeared on the market that are capable of direct driving. Their primary use appears to be in the construction of small walls and bulkheads, but they could conceivably be used for vertical containment walls. Other rigid polymer alternatives exist in this category as well.

Due to their widespread use in containment walls, however, HDPE sheets and panels will be the focus of the remainder of this section.

5.1.3 Intrinsic Impermeance of Geomembranes

HDPE has an extremely low "permeability". Note that the permeability of a geomembrane is not hydraulic conductivity (as used for analysis of SB, SC, CB or SCB vertical barriers), but is a diffusion-related transport property. The standard test for its determination is a water vapor diffusion test performed in accordance with ASTM E96. A typical value for 2.5 mm (100 mil) thick HDPE geomembranes is approximately 0.006 g/m^2-day ($\simeq$ 0.006 gal/acre-day). While the calculations are a bit spurious, this value converts to approximately 1 x 10^{-13} cm/sec of equivalent Darcian type hydraulic conductivity, Koerner (1994). Haxo (1988) has determined comparable vapor transmission values for a range of solvents, see Table 5-2. Note that the values

TABLE 5-2 Solvent Vapor Transmission Values of HDPE after Haxo (1988)

Property	0.80 mm (32 mil)	2.6 mm (102 mil)
Solvent vapor transmission, g/m2-day		
Methyl alcohol	0.16	—
Acetone	0.56	—
Cyclohexane	11.7	—
Xylene	21.6	6.86
Chloroform	54.8	15.8

are dependent on thickness and the type of solvent vapor. They are related to the solubilities of the permeating liquids, which in these cases were 100% neat organic solvents. While these vapor-transmission values are lower than any other available polymer barrier system, they apply only to the geomembrane sheet and not for the interlocks. The interlocks used to join the individual panels are of the following three different types;

- a hydrophilic gasket,
- grouted, with a variety of sealants, or
- mechanically welded.

Note that values shown in Table 5-2 can refer to interlocks that are mechanically welded, but not to gasket or grouted type interlocks. The gasket or grouted interlock types are generally used in vertical barrier wall construction. Owing to their importance, interlocks will be treated in more detail later in this section.

5.1.4 Scope of Section 5

The state-of-the-practice as presented by the eight containment technology workshop panelists is central to the text that follows. Field performance via case histories will also be presented. Emphasis is on those case studies that are presented in the literature rather than internal reports or manufacturer's brochures.

The technology of geomembranes as vertical barriers is assessed in light of costs for the various installation techniques and in comparison to other types of vertical barrier systems. Emerging trends and technologies are also noted. Lastly, the research, development, and implementation needs related to the use of geomembranes as vertical barriers is assessed. As will be seen, although the technology already exists, greater exposure is needed in the technical literature so that the advantages and disadvantages can be assessed for the wide variety of site-specific conditions encountered in practice.

5.2 STATE OF PRACTICE

This section reviews the current practices, materials, designs, and details relative to the use of geomembranes as vertical barriers and concludes with a set of case histories illustrating the various nuances of the technology.

5.2.1 Installation Methods

Early applications of geomembranes as vertical barriers utilized a slurry supported excavation with the inserted geomembrane displacing the slurry in a progressive manner. One installation technique, called Envirowall®, employed a folded U-shaped geomembrane that covered both the sides and bottom of the trench. The space within the U-shaped geomembrane was backfilled with sand, as installation proceeded. This procedure resulted in a double lined vertical wall with a sand leak detection system between the two geomembranes, see Koerner (1986). Unfortunately, construction of this system was difficult. With the slurry at a density of approximately 1.2 g/cc and the HDPE geomembrane at 0.94 g/cc, it was difficult to submerge the folded geomembrane beneath the slurry and to hold it in place until sufficient sand mass had been placed to counterbalance the buoyancy. Other early efforts used a single geomembrane against the side of the trench, with the bottom weighted down with pipes, rails, etc. Most of these early techniques were rather unwieldy.

TABLE 5-3 Current Installation Methods for Geomembrane Vertical Walls

Method No.	Method or Technique	Geomembrane Configuration	Trench Support	Typ. Trench Width mm (in.)	Typ. Trench Depth m (ft.)	Typ. Backfill Type
1	trenching machine	continuous	none	300-600 (12-24)	1.5-4.5 (5-15)	sand or native soil
2	vibrated insertion plate	panels	none	100-15 (4-6)	1.5-6.0 (5-20)	native soil
3	slurry supported	panels	slurry	600-900 (24-36)	no limit, except for trench stability	SB, SC, CB, SCB, sand or native soil
4	segmented trench box	panels or continuous	none	900-1200 (36-48)	3.0-9.0 (10-30)	sand or native soil
5	vibrating beam	panels	slurry	150-220 (6-9)	no limit	SB, SC, CB, SCB slurry

Since these early attempts, many clever installation methods have been developed. Five newer methods which have been developed and used on a regular basis are described in Table 5-3. Each will be described separately.

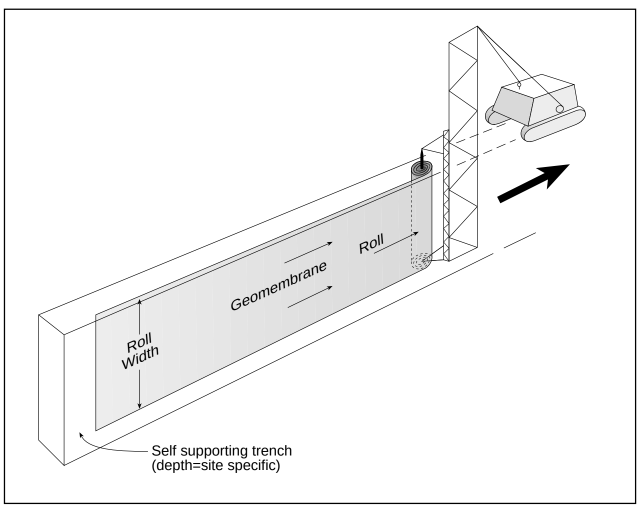

Figure 5-3 Continuous geomembrane in self-supporting trench excavated by a trenching machine

Figure 5-3 presents the essence of the <u>trenching machine</u> installation method. A large bucket trencher or disc cutter is used to excavate an unsupported trench in the ground. The geomembrane roll is lowered vertically into the trench and progressively unrolled. The geomembrane can also be unrolled at the ground surface, but the sheet must be distorted into an S-configuration in order to reach its final position. A major advantage of this method is that no seams are necessary (until the end of the roll is reached). Native soil is commonly used for backfill, since a visual inspection can be made of the completed installation, at least to the level of water in the trench. Alternatively, a sand backfill with a pipe leak detection system can be used. A major disadvantage is that the unsupported trench depth is limited by site-specific circumstances.

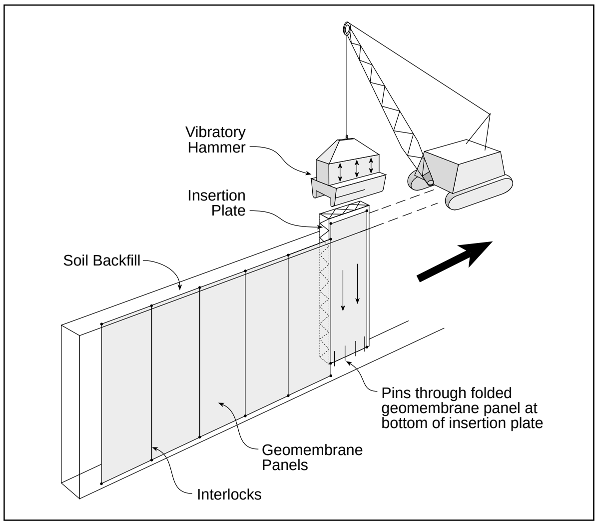

Figure 5-4 Vibrated insertion plate method with geomembrane panel attached.

The vibrated insertion plate method utilizes geomembrane panels approximately 3 m (10 ft.) wide on the side of a steel truss "insertion plate", see Figure 5-4. The geomembrane is fixed at its base with pins protruding from the bottom of the insertion plate. The geomembrane panel is folded around the bottom of the insertion plate for penetration by the pins. A vibratory pile hammer forces the entire assembly (insertion plate plus geomembrane panel) to the desired depth. The insertion plate is then withdrawn, leaving the geomembrane panel behind. Interlock connections are required to join one geomembrane panel to the next. This relatively fast method requires neither trench support nor special backfill. It can also be used in running sands, which may be problematic with other methods. A disadvantage is that only certain soft and/or loose soils can be penetrated to substantial depths. Also, damage to the geomembrane panel during insertion may be a concern with certain soil strata.

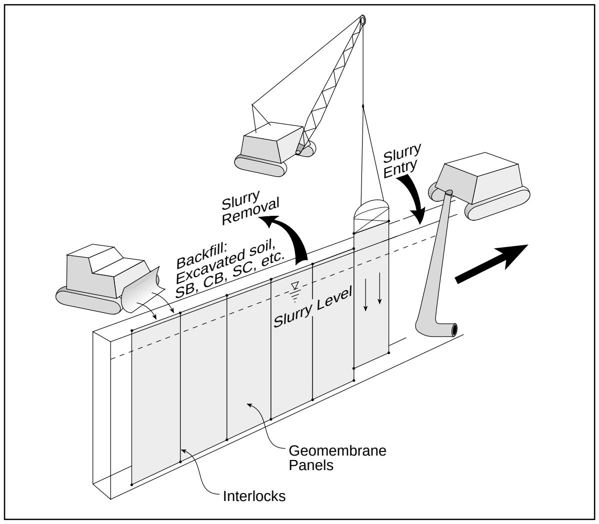

Figure 5-5 Backhoe excavated, slurry supported method with geomembrane panels and engineered backfill as secondary operation.

The slurry supported installation method begins by using a conventionally excavated and slurry supported trench, see Figure 5-5. The geomembrane panels are then inserted into the slurry to the desired depth utilizing a steel frame. The panels are held in their final position with weights or toed into the subsoil stratum and interlocked to form a continuous liner. The backfill is usually SB, SC, CB or SCB materials. There are other options, including low permeability native soil. If geomembranes are placed on both sides of the trench, sand is used as the intermediate backfill, creating a double lined vertical wall with intermediate leak detection. Depth is limited only by the stability of the trench, and the deepest vertical walls have been constructed by this method. The maximum depth constructed to date is 30 m (100 ft.), but considerably greater depths are being considered.

The segmented trench box method uses a modified steel trench box to support the side walls of the trench immediately following excavation, see Figure 5-6. It is moved laterally within the trench which is excavated by a backhoe or other construction equipment. The geomembrane panels are placed between the two outer segments of the box. Backfill is usually native soil, but other materials such as sand with a perforated pipe collection system can also be used. The technique is depth limited, hence quite site specific. Although untried, the technique could possibly be used with a continuous geomembrane roll being fixed to the advancing end of the trench box and unrolled as the trench box is moved along, thus avoiding the need for interlocks.

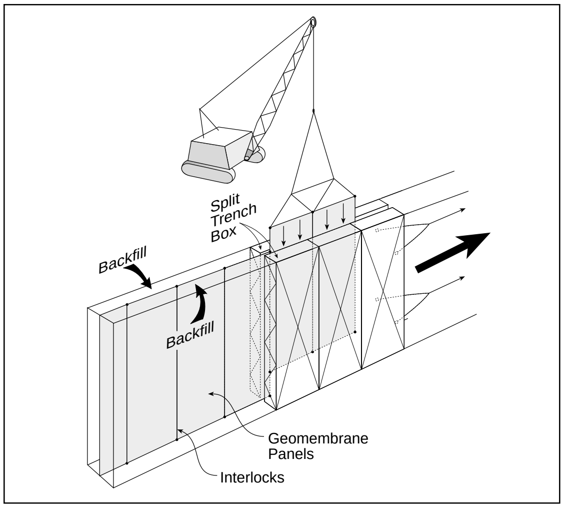

Figure 5-6 Trench box method with geomembrane panels and subsequent backfill

The vibrating beam method utilizes a modified form of the slurry-supported trenching method, see Figure 5-7. The slurry is introduced as the vibrating beam is inserted. Once the trench is slurry supported and the beam removed, the geomembrane panels are installed as a secondary operation. The usual backfill for the relatively narrow trench is a SB, SC, CB or SCB slurry. The technique is commonly practiced but is somewhat dependent on site-specific soil conditions.

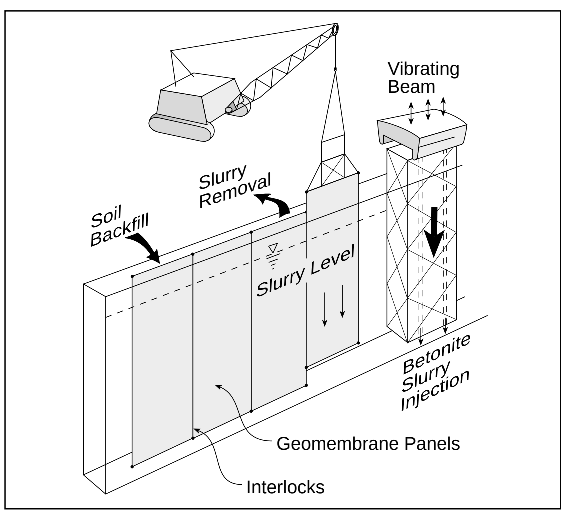

Figure 5-7 Vibrating beam/slurry trench support method with geomembrane panels as secondary operation

There may be other methods for vertical geomembrane wall installation, but those discussed above appear to be the most commonly used at present.

5.2.2 Types of Interlocks

Three types of interlocks are used to connect the geomembrane panels (or rolls) together. These are:

- hydrophilic gasket types,
- grouted chemical or tube types, and
- continuous seaming by welding.

They generally require a special extruded HDPE shape to be seamed to the edges of the geomembrane sheets. Quite often they are lock-and-key mated, requiring the alternating type to be seamed on opposite sides of the panels. Seaming is done in the factory by wedge welding. Either single or double wedge welds are made, the latter having the capability for air pressure testing before the panels are shipped to the field, EPA (1993).

a. Hydrophilic Gasket Interlocks. Figure 5-8 illustrates four different interlock configurations, each of which requires a hydrophilic gasket for liquid tightness of the installed panels. The gaskets are either circular or rectangular in cross section. They are threaded into their groove or slot during connection in the field. The length of gasket that is fed into the interlock from the ground surface provides reasonable assurance that the interlock connection has been made down to the bottom of the previously placed panel.

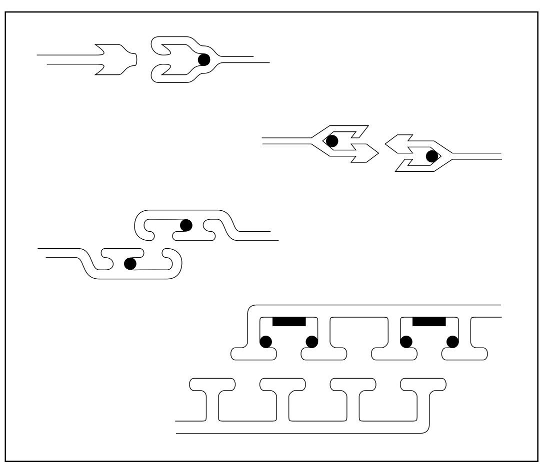

Figure 5-8 Various hydrophilic gasket type geomembrane panel interlocks

Although the gasket material is usually proprietary, most are made from rubber (chloroprene or neoprene) formulated with a hydrophilic polymer.

Unconfined swelling in water is five to eight times the original volume. Chemical compatibility of the gasket can be evaluated by standard laboratory techniques, e.g., the modified EPA 9090 test procedure.

b. Grouted Interlocks. Alternatively to a gasket and for certain types of interlocks, a liquid-tight interlock can be made using various slurries or grouts, see Figure 5-9. Here the pumped slurry or grout flows down one channel or tube, and, after reaching the bottom of the interlock returns to the surface in an adjacent channel or tube. The grout can be any flowable material, e.g., cement-bentonite or polymers of a wide variety.

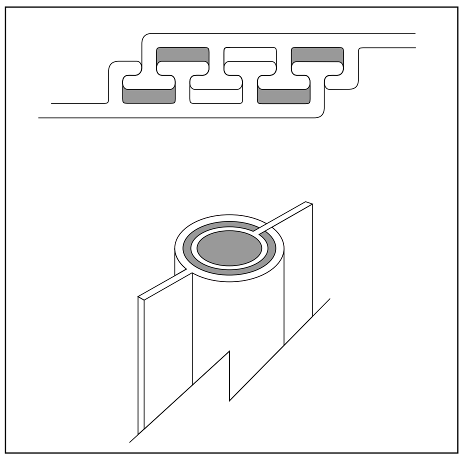

Figure 5-9 Various grouted geomembrane panel interlocks

c. Welded Interlocks. Lastly, it might be possible to create a completely seamed system by *in situ* welding of the sheets or panels. This possibility is currently being investigated. To date, there has been only one field installation of this type. This interlock category is added for completeness and as an indication of what might become more established.

5.2.3 Connections to Covers and/or Floors

The end connections (top and bottom) of geomembrane vertical barriers are important in providing a completely contained system. The vertical wall must be installed first; then the cover (or cap) is placed on the contained waste. Depending on the details, in particular the layering of geosynthetics in the cover, there are several connection possibilities (see Section 6 on caps for details).

Since there is usually a geomembrane in the cover, the possibility of mechanically attaching the geomembrane in the cover to the geomembrane in the vertical wall should be considered, as was shown in Figure 5-1. The two geomembranes must be compatible insofar as their seaming is concerned.

Cover geomembranes are often designed for out-of-plane flexibility; thus, very flexible polyethylene (VFPE), flexible polypropylene (fPP) or polyvinyl chloride (PVC) are generally used. While difficult to accomplish, HDPE can be seamed to VFPE and fPP. HDPE cannot be seamed to PVC. For the polyolefins (VFPE and fPP), extrusion fillet welding is required, a difficult process that requires welding over the interlocks. However, it is possible, as depicted in Figure 5-1(a). Alternatively, the cover geomembrane can be extended horizontally or dropped down vertically over the outside of the geomembrane vertical wall as shown in Figure 5-1(b).

The termination of a geomembrane vertical wall at the base, and in particular its connection to the lower aquitard or confining layer, is important, especially since the termination cannot be visually inspected. If a low permeability aquitard is available, an adequate geomembrane penetration depth into such a layer should be specified as depicted in Figure 5-2(a). The backfill material at the base of the geomembrane is important and a low hydraulic SB, SC, CB, SCB, or polymer grout, should be considered. In the absence of an aquitard, a hanging wall can be created, as depicted in Figure 5-2(b). The depth of a hanging wall is determined on the basis of a seepage analysis to limit flow beneath the bottom of the geomembrane. This depth is somewhat subjective and is essentially a design/regulatory decision.

The bottom of a geomembrane vertical wall can also be keyed into an artificially placed floor. In general, the artificially placed floor should be constructed before the geomembrane vertical wall is installed.

5.2.4 Case Studies

Several significant case histories have been described in professional journals and at conferences that illustrate the use of geomembrane vertical barriers. See Table 5-4 for an overview.

Burnette and Schmednecht (1994) describe a geomembrane vertical barrier that was constructed around a Great Lakes chemical plant to prevent off-site migration of contaminated groundwater. The vertical barrier was constructed through layers of sand, gravel, cobbles, and boulders. The vibrating beam installation method with a clay/cement slurry was used to construct a 120 mm (5 in.) wide trench. HDPE panels, 1.2 m (4 ft) wide and 2.0 mm (80 mil) thick, were inserted into the trench with a steel template. Trench depths were up to 10 m (35 ft). Jet grouting was used to construct vertical barriers at the locations with underground or above ground interferences that could not be rerouted. Average daily production for the entire project was approximately 100 sq. m (1100 sq. ft), with an improved average of approximately 160 sq. m (1700 sq. ft) during the second half of the project.

Bliss and Burnette (1995) describe 20 km (12.5 mi) of existing earthen dike embankments in Arizona that were modified with a geomembrane vertical barrier to provide both a seepage barrier and a filter zone. A 600 mm (24 in.) wide trench was excavated using a biopolymer slurry for stability.

Large, 2.0 mm (80 mil) HDPE geomembrane panels, 7.5 m (24 ft) wide by 16.5 m (55 ft) long, were inserted into the trench with a large steel frame. Granular backfill material was tremied into the trench as a drainage layer. A full-scale test was conducted to evaluate the constructability and effectiveness of the design. A 15 m (50 ft) deep by 340 m (1100 ft) long test section was constructed. Quality assurance verification of all joints was accomplished with the electronic circuit method described later in this section. Instrumentation was installed and the test section impounded with water for 30 days. Seepage results based on several flow rate monitoring stations indicated the geomembrane provided an excellent vertical barrier.

TABLE 5-4 Case Studies Utilizing Geomembrane as Vertical Barriers

Reference	Type of Contained Waste	Type of Installation	Depth of Wall		Length of Wall		Type of Interlock	Backfill Material
			(m)	(ft)	(km)	(mi.)		
Burnette & Schmednecht	hazardous waste	vibrating beam	10	35	0.3	0.2	Curtain Wall Interlock	clay/cement slurry
Bliss & Burnette	earth dam cutoff	slurry supported	15	50	20	12.5	Curtain Wall Interlock	sand
Burnette & Pierce	petroleum wastes	slurry supported	4.5	15	0.4	0.2	Curtain Wall Interlock	sand
Burnette & Pierce	hazardous wastes	slurry supported	14	45	0.5	0.3	Curtain Wall Interlock	sand
Scuero, et al	earth dam cutoff	slurry supported	9	30	0.1	0.1	Geolock	CB slurry
Michalangeli	ash & MSW	slurry supported	various		various		grouted	CB slurry
Hansen & Crotty	contaminated drilling waste	trenching machine	3	10	34	21	welding	natural soil

Scuero, et al. (1990) describe a 2.0 mm (80 mil) HDPE vertical barrier that was constructed in southern Italy at the upstream toe of a 110 m (360 ft) long earth dam to limit seepage under the dam. A 500 mm (20 in.) wide by 9 m (30 ft.) deep trench was excavated and a geomembrane installed and keyed to a minimum of 1 m (3 ft) into an underlying confining stratum. Cement-bentonite slurry was used to seal the geomembrane into the key. Concrete was placed at the top of the HDPE vertical barrier to provide a water tight connection between the vertical barrier and the PVC liner on the downstream slope of the earth dam.

Michalangeli (1995) describes a series of four case histories using geomembrane panels as vertical barriers around landfill sites in Italy. In all cases, 2.0 mm (80 mil) thick HDPE geomembranes were used with grouted-type interlocks. The projects were extremely large, up to 130,000 sq. m (155,000 sq. yd).

Hansen and Crotty (1995) describe the installation of 34 km (21 mi) of geomembrane vertical barrier in Alaska's North Slope oil fields to contain drilling waste fluids. Various geomembranes were used, including; HDPE, reinforced PVC, and reinforced polyurethane. Single and double lined systems were installed. Double lined systems comprised about 85% of the installations. A trencher was used to excavate a 300 mm (12 in.) wide trench. Typical depths were 3.0 m (10 ft) and the barriers were keyed into permafrost. Geomembranes were rolled out in the trenches and roll ends were seamed by a thermal fusion method. Penetrations through the vertical barrier by underground utilities were sealed with boots and welding. The liners were installed in temperatures as low as –42°C (–44°F). Anticipated ground temperatures were –32°C (–26°F). Excavated soil was used for backfill. Production was about 150 m (500 ft) per 10 hour shift in spite of snow, extremely cold weather, and wind.

5.3 PERFORMANCE OF THE TECHNOLOGY

This section on the performance of geomembrane vertical barriers discusses both laboratory and field issues. It is focused completely on HDPE, due to its general use in vertical barriers.

5.3.1 Durability of HDPE

The chemical resistance of HDPE geomembranes is well established, based on thousands of EPA 9090 compatibility tests. A wide range of leachates have been evaluated for landfill/waste pile/heap leach materials. For this reason, "blanket approval" of HDPE has often been requested of the regulatory community and, while not granted at the federal level, many states currently sole source HDPE geomembranes for the containment of all types of solid waste. It is used for 80 to 90% of the liners beneath nonhazardous and hazardous landfills, waste piles, and surface impoundments. It is generally thought that low level radioactive waste poses no degradation concern; however, high level radioactive waste must be evaluated on a site specific basis.

The service lifetime of HDPE geomembranes should be considered for the containment of all waste materials, especially for low-level radioactive, low-level radioactive mixed, and hazardous wastes. The common mechanism for long-term degradation of HDPE is chemical oxidation. Several studies have been conducted to assess the lifetime of HDPE polymer, particularly in the gas transmission pipe, electrical cable shielding, and geomembrane application areas. It is generally considered that HDPE passes through the following three different stages during its service lifetime:

- depletion of anti-oxidants,
- an induction time preceding the onset of degradation, and
- time for degradation of 50% of a relevant engineering property, such as strength or elongation (the so-called half-life).

These three stages are shown in Figure 5-10 as A, B and C, respectively. Since field degradation of buried HDPE geomembranes has not been reported since their introduction as lining materials in 1978, direct field data are not available. Thus, emphasis must be placed on laboratory simulation and testing. (Note that uncovered geomembranes will degrade more rapidly when exposed to ultraviolet radiation and/or high temperatures; hence, this discussion pertains to covered, or buried, geomembranes).

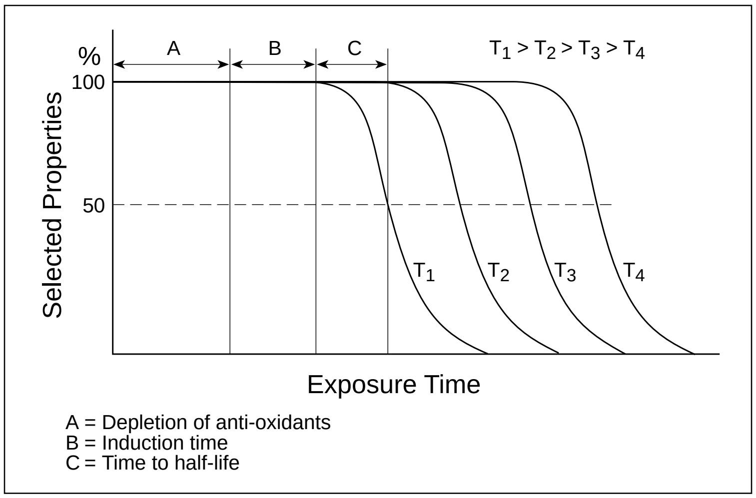

Figure 5-10 The stages of lifetime of HDPE geomembrane utilizing time temperature superposition technique

The accepted method for lifetime simulation of polymer property behavior is based on time-temperature testing at laboratory-controlled temperatures combined with Arrhenius modeling (fitting an exponential-type curve to the experimental data) and using the model to estimate the anticipated response at a site-specific temperature. The procedure involves incubation of samples at several elevated temperatures, determination of the time-dependent behavior of the selected polymer property, and extrapolation of the behavior to the site specific temperature. The estimated lifetime of the sample property is predicted by this procedure.

Work at the Geosynthetic Research Institute has resulted in experimental estimates of the time for antioxidant depletion between 50 to 150 years, an unknown incubation time (perhaps decades), and half-life degradation times of 200 to 750 years, Koerner, et al. (1990) and Hsuan, et al. (1993). The half-life degradation estimates utilized activation energies from the literature, see the references cited in the above articles.

As a result of the above laboratory simulation studies, in-service lifetimes for buried HDPE could approach 1000 years.

5.3.2 Diffusion through HDPE

Diffusion of water or solvent through HDPE geomembranes can only occur in a vapor state. Upon adsorption of the liquid phase into the geomembrane as a vapor, it enters the polymer structure and diffuses through the amorphous phase. After diffusing through the geomembrane, the vapors condense to liquid on the opposite side of the geomembrane.

HDPE has the lowest diffusion rate of any commonly used polymer for geomembranes. Vapor diffusion rates for a number of solvents were given in Table 5-2. Note that solvent vapors may have higher or lower diffusion rates compared to water vapor. HDPE sheets used in vertical wall panels are typically 2.0 mm (80 mil) thick, i.e., thicker than most geomembranes used for landfill liners and covers.

While there is certainly a finite diffusion rate of solvents through HDPE geomembrane walls, Park, et al. (1995), it is so low that the focus of attention is usually placed on the integrity of the interlocks.

5.3.3 Interlock Continuity and Testing

Geomembrane panels must be joined together by interlocks, with the most common type being the hydrophilic gasket type shown in Figure 5-8. To test the continuity of such interlocks, it is possible to place a contact element and conductive wire on each paired interlock, Bliss and Burnette (1995). The contact elements are positioned at the bottom of the interlocks so as to touch one another when the two panels are properly installed and fully mated to one another. A battery is connected to the wires at the ground surface, creating an electrical circuit. Measuring the resistance in the open versus closed circuit verifies the continuity of the interlock at the intended depth.

Field testing for interlock liquid tightness is very difficult; however, a laboratory test is available. The experimental test setup is shown in Figure 5-11, Gundle (1994). Test results at pressures up to 800 kPa (100 lb/in^2) exhibited only nominal seepage.

The testing of grouted interlocks as shown in Figure 5-9 can be accomplished by observing the flow of grout coming out of the adjacent channel or outer pipe, respectively. Although not known to have been performed in the field, a volumetric check probably could be developed.

5.3.4 CQC and CQA

In addition to the techniques described above, both construction quality control (CQC) and construction quality assurance (CQA) should be employed during installation of geomembrane vertical barriers. These procedures are well advanced in the landfill liner and cover application areas, EPA (1993). A certified field inspectors' program exists through the National Institute for Certification of Engineering Technologists, NICET (1992) based on experience.

Level I (entry), Level II (2-year experience) and Level III (5-year experience) examinations, resulting in individual certifications are available for both CQC and CQA personnel. A certification program is not yet available for inspection of geomembranes in vertical barriers, the possible reasons being;

- the field is too new,
- the field is not large enough, or
- a demand has not existed.

It should be possible to expand the existing certification program to include geomembrane vertical barriers. CQC/CQA for geomembrane vertical walls is similar in concept to the installation of geomembranes in caps. However, it may be more difficult and require a greater level of effort than when the geomembrane is horizontal or on a slope, but certainly proper CQC/CQA is possible.

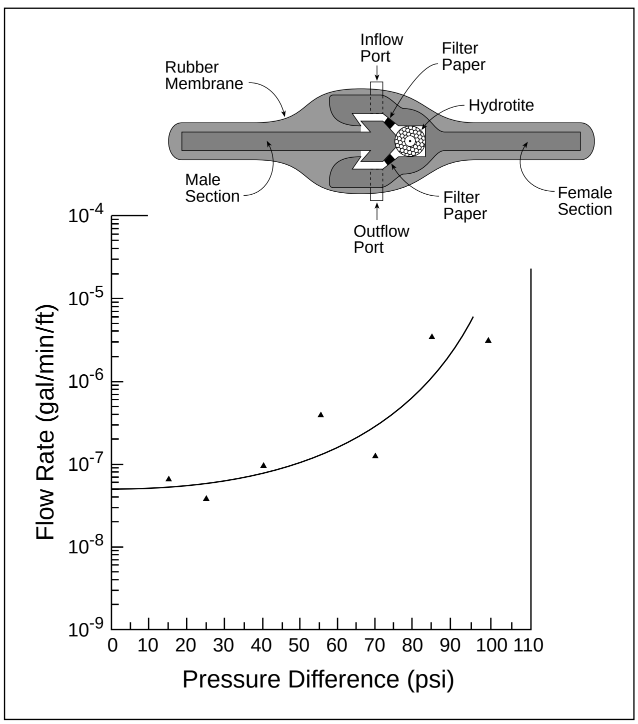

Figure 5-11 Hydrophilic gasket interlock test developed by GeoSyntec for Gundle Lining Systems, Inc.

5.3.5 Long-Term Monitoring

There has only been indirect evidence of the effectiveness of geomembrane vertical walls based on long-term monitoring. For example, downstream monitoring wells have not indicated the presence of contaminant plumes emanating from landfills contained by geomembrane vertical walls. More direct evidence could be provided by lysimeters located downstream of the geomembrane. Construction of a double walled geomembrane with a leak detection layer between the two geomembranes, as shown in Figure 5-12, would provide even better direct evidence of the effectiveness of a geomembrane vertical wall. Possible arrangements include geomembranes being placed on both sides of the excavated trench and sand filled between, or with a geomembrane/geonet/geomembrane composite being installed as a self contained unit. To our knowledge, the latter has not yet been implemented.

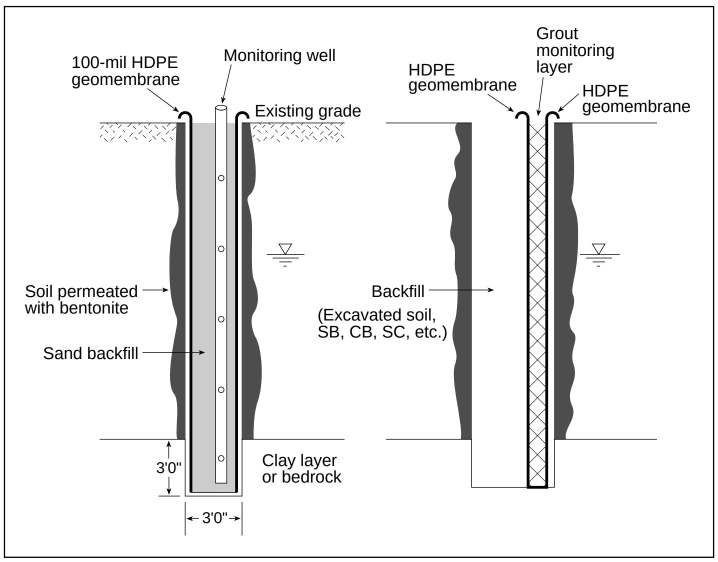

Figure 5-12 Two possible doublewall schemes for leak detection monitoring

5.3.6 Warrants

Geomembrane manufacturers and installers do give warrants on materials and workmanship, but it must be recognized that the duration of the warrant will affect the project cost. This is not because of trepidation on the part of the manufacturer and/or installers, but because funds will have to be escrowed to insure the warrant.

The service lifetime of HDPE will extend long beyond the normal 20-year warrant. Furthermore, any installation flaw should evidence itself soon after completion of the project; thus, a long-term installation warrant is generally not necessary. For example, a hole or broken interlock in a geomembrane vertical wall caused during construction will leak immediately, rather than after many years of service.

Thus warrants, while available, are felt to be of limited value, especially since they are pro-rata and are reflected in the bid cost of the installation.

5.4 ASSESSMENT OF THE TECHNOLOGY

An assessment of geomembrane vertical barriers, as presented by the workshop panelists, is presented utilizing a generalized table, with approximate costs, followed by miscellaneous items, including emerging technologies.

5.4.1 General Assessment, Including Estimated Costs

An overall assessment of geomembrane vertical barrier technologies is given in summary form in Table 5-5. It follows the order of the five installation methods presented earlier in this section. The methods have been placed in the approximate order of increasing cost per unit area of wall. Table 5-5 also includes some comments concerning advantages or disadvantages of each method. It should be noted, however, that the prices and associated comments reflect site specific information and are very generalized.

TABLE 5-5 General Assessment of Geomembranes as Vertical Walls

Method No.	Method Name	Typical Cost		Some Advantages	Some Disadvantages
		$/ft^2	$/m^2		
1	trenching machine	2-5	20-50	•no seams •rapid installation •no slurry	•depth limited •soil type limited •trench stability necessary
2	vibrated insertion plate	3-7	30-70	•rapid installation •narrow trench •no material spoil •no slurry	•soil type limited •possible panel stressing •bottom key is a concern
3	slurry supported	5-15	50-150	•no stress on panels •conventional method •choice of backfill	•requires slurry •buoyancy concerns •slow process
4	segmented trench box	16-18	160-180	•can weld seams •visual inspection •no stress on panels •no slurry	•depth limited •soil type limited •slow incremental process
5	vibrating beam	18-25	180-250	•narrow trench •no material spoil •no stress on panels •usually CB slurry	•requires slurry •slow incremental process •soil type limited

5.4.2 Leak Detection Methods

As with most vertical barriers, post-construction leak detection is performed via downgradient monitoring wells. The number and spacing of these wells is a critical decision. It is, perhaps, a more critical decision with geomembranes as vertical walls as compared to other wall types, since leakage would probably be from a single (or few in number) point source where a flaw is located. Thus, the plume from such a leak would not be spread over a large area where it could be detected by widely spaced monitoring wells. The actual situation is obviously very site specific.

For critical applications, however, a different strategy for leak detection should be considered when using geomembranes as vertical walls. Two geomembranes should be employed, with an intermediate sand or geonet drainage layer acting as a leak detection system, as shown in Figure 5-12. With sand as the leak detection drainage layer, the geomembranes would be placed on each side of the excavated trench. With a geonet as the drainage layer, the composite system (geomembrane/geonet/geomembrane) is placed against one side of the trench and the remainder backfilled as desired. For such a double liner system placed against the side of the trench facing the waste and the opposite side backfilled with a low hydraulic conductivity soil, cement, or grouted material, one has the vertical equivalent of a hazardous waste liner system, i.e., Subtitle "C" type. Such double liner systems can be the most secure of the various alternatives within the vertical wall category.

5.4.3 Geomembranes vs. Other Vertical Barriers

In comparing geomembrane vertical walls to other vertical walls described in this report, many considerations must be assessed. For example, the following aspects must be considered;

- hazard potential of the waste,
- presence (or not) of an aquitard,
- site stratigraphy,
- depth of wall,
- configuration of wall,
- space considerations/site availability,
- allowable hydraulic conductivity through barrier,
- anticipated ground movements,
- allowable diffusion through barrier,
- allowable leakage (if any),
- disposal of excavated material from trench,
- durability (aging) of system,
- capability of long-term monitoring,
- risk assessment of the selected system,
- availability of materials, installers and equipment, and

- cost of the installed system.

With this many considerations, it is impossible to prioritize or even compare many of the alternative strategies for design, construction and monitoring of different types of vertical walls. It is, however, relevant to note that for critical containment applications, a double wall with leak detection capability and a low permeability soil (or other material) backup can provide the most secure containment of the various types of wall systems that are available.

5.4.4 Emerging Technologies

Emerging geomembrane vertical wall technologies can be grouped according to materials, installation methods, and interlock connections. As a material, HDPE dominates the currently available systems. On the basis of cost and availability it is likely to continue to do so in the future. This is not to say that other, better systems might not become available. For example, high modulus, high strength polymers would permit direct driving (like steel sheet piles), providing an additional installation method to those described in this section. For those situations where diffusive transport is of concern, a cross-linked HDPE is a distinct possibility as is the potential for using geosynthetic clay liners, Heerten, et al. (1996). Even further, a geomembrane cured *in situ* is conceivable. Fiber optic sensors within the geomembrane, i.e., the use of "smart geomembranes", are being evaluated for both leak detection/location and estimation of material stresses.

There are many possible installation methods in addition to the five described in this chapter. Undoubtedly, others will appear in the future. The new excavation method of soil sawing is interesting in this regard and could be considered as a subset of the trenching method depicted in Figure 5-3. The future may bring about full fusion welding of connections in the trench to join the individual panels together. The segmented trench box method described by Figure 5-6 provides such an opportunity. Welding may even become possible while in the slurry of the supported trench. Such methods, however, are in the development stage at this time.

5.5 NEEDS

In assessing the "needs" within the state-of-the-practice of geomembranes used as vertical barriers, the Panel did not call for laboratory tests nor additional R & D to be initiated and performed. The general feeling of the Panel is that the technology exists and is available for application by the owner/user community. The question was raised as to why geomembrane vertical barriers are not used more than currently. Clearly, part of the answer is the added cost compared to vertical barriers without geomembranes. This cost can range from a 10 to 30% increase above the cost of a conventional SB, SC, CB or SCB backfilled vertical wall without a geomembrane. However, the added environmental safety and security that a continuous barrier, even a composite barrier, offers may justify such an increase in cost.

What is needed for increased use of geomembrane vertical barriers is greater exposure of the technology. Such exposure can come about from published case studies, of which there are relatively few in the published literature, (see Table 5-4). The development and eventual publication of performance case studies should generate increased interest, especially if the related advantages and disadvantages of the system are noted for the particular situation.

Increased exposure through publications would eventually bring about performance based designs. With such design methodologies, direct comparisons could be made between geomembrane vertical barriers and other vertical barriers without the inclusion of geomembranes. Such a methodology would likely be a significant factor in decision-making by owners, designers and regulators alike.

Finally, a few full scale demonstration projects would be significant in illustrating the potential for application of geomembrane vertical barriers in environmental remediation. Such demonstrations have not been undertaken to date. Double geomembrane technology combined with leak detection capability between the two geomembranes offers an attractive option for construction of containment barriers.

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SECTION 6

CAPS

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6.1 BACKGROUND

Caps, also called "cover systems" or "surface barriers", are one component of the engineered systems used to manage wastes at modern waste disposal units and, not infrequently, at contaminated sites as part of remedial actions. The purpose of a cap is to perform one or more of the following functions (Rumer and Ryan, 1995):

- minimize percolation of water into the underlying contaminated materials;
- raise the ground surface and provide appropriate slopes to promote surface-water runoff;
- control the release of gas from the contaminated materials; and
- separate the contaminated materials from humans, animals, and plants.

Caps are composed of six basic components, from top to bottom: surface layer, protection layer, drainage layer, barrier layer, gas collection layer, and foundation layer. The layers are illustrated in Figure 6-1, and the primary functions and potential materials for each layer are listed in Table 6-1. All layers must have adequate durability so that they function over the design life of the cap and adequate shear strength so that the cap surface slopes are stable. Some layers may contain several materials. For example, a hydraulic barrier layer may consist of a geomembrane upper component and a compacted clay lower component (i.e., a composite). Not all layers are needed for all sites. For example, a drainage layer may not be needed at an arid site. All caps, however, require a surface layer.

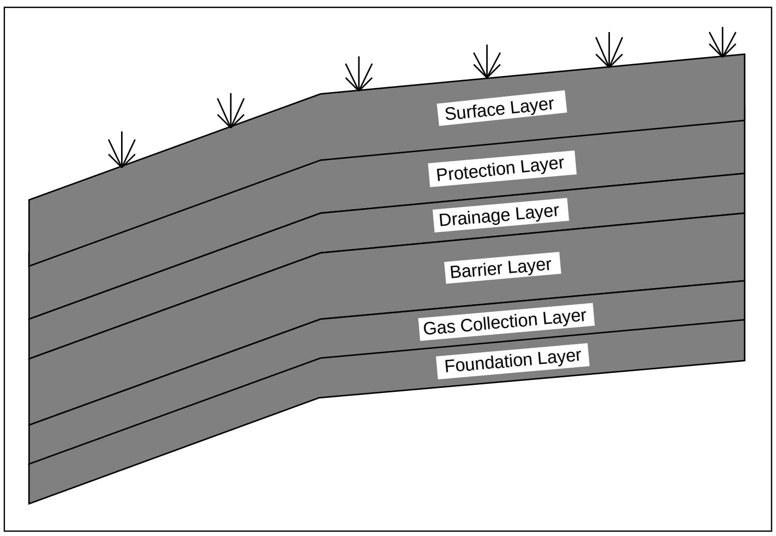

Figure 6-1 Cap cross section

The current knowledge and practices for cap technology have been recently reviewed and evaluated (Rumer and Ryan, 1995). The review included an evaluation of the materials that are used to construct caps, a description of construction quality control procedures used for barrier layers, and an assessment of factors that can impact the overall performance of caps. The primary factors that influence cap performance were identified as:

- layer types included in the cap, layer materials, and layer thicknesses,
- annual precipitation,
- surface slope angle of the cap,
- compressibility of the underlying waste,
- gas generation, and
- presence of burrowing animals.

TABLE 6-1 Cap components

Layer	Primary Functions	Potential Materials
Surface Layer	•Separate underlying layers from the ground surface •Resist wind and water erosion •Reduce temperature and moisture extremes in underlying layers	Topsoil (vegetated) Geosynthetic erosion control layer over topsoil (vegetated) Cobbles Paving material
Protection Layer	•Store water that has infiltrated the surface layer until the water can be removed by evapotranspiration •Separate the waste from humans, burrowing animals, and plant roots •Protect underlying layers from wetting-drying cycles, which may cause cracking •Protect underlying layers from freezing-thawing cycles, which may cause cracking	Soil Cobbles Recycled or reused waste (e.g., fly ash, bottom ash, and paper mill sludge)
Drainage Layer	•Reduce the head of water on the barrier layer, thereby decreasing water percolation through the cap •Reduce pore water pressures in the overlying cap layers, thereby increasing slope stability •Reduce the time the overlying layers are saturated following rainfall events, thereby decreasing erosion	Sand or gravel Geonet or geocomposite Recycled or reused waste
Hydraulic Barrier Layer	•Impede percolation of water through the cap •Restrict outward movement of gases from the waste	Compacted clay Geomembrane Geosynthetic clay liner Recycled or reused waste Asphalt Sand or gravel capillary barrier
Gas Collection Layer	•Collect and remove gases to reduce the potential for uncontrolled gas migration •Collect gas for energy recovery	Sand or gravel Geonet or geocomposite Geotextile Recycled or reused waste
Foundation Layer	•Serve as a foundation for the cap, especially during construction of layers requiring compaction (e.g., compacted clay barrier layer)	Sand or gravel Soil Recycled or reused waste Select waste

In that review, it was noted that little published information was available concerning the performance of caps. The performance data are limited because modern multicomponent caps have only been in existence for about a decade, giving insufficient time for much data to be gathered. Also, it is difficult to quantify the percolation of water or the migration of gas through a cap on the basis of the available data because the data are not normally collected for the purpose of evaluating the performance of the cap.

The purpose of this section is to review the current technological status of caps, including material performance, design methods, field performance (i.e., cap failures), and field monitoring. The focus is on field performance and field monitoring, as this aspect of caps was identified as an area requiring further research in the earlier publication by Rumer and Ryan (1996). To this end, researchers and practitioners in the U.S. and Europe involved in the conduct of field evaluations were asked to share both published and unpublished data. These data are summarized in this section.

It should be recognized that much of the current knowledge about caps has been developed from landfill capping applications rather than from experiences from containment of contaminated lands. It is important, however, to learn from the landfill capping experience, since this is the principal source of the best available information.

6.2 STATE OF PRACTICE

6.2.1 Introduction

The design of a cap for a specific application depends on the requirements of the application. While the cap requirements for hazardous waste landfills may differ from those for radioactive waste landfills or municipal waste landfills, the procedures used to design caps for different applications are generally the same. The current state of practice for the design of "conventional" caps for municipal solid waste (MSW) landfills has been described by Othman et al. (1995). As outlined in their paper, major design aspects to be considered relate to: (i) flow of water into and through the cap; (ii) impacts of waste settlement on the cap performance; (iii) static and dynamic stability of the cap; and (iv) surfacewater management and erosion control. Other considerations in cap design include: (i) gas collection and removal; (ii) the need for bio-barriers; and (iii) long-term durability of cap materials.

Several issues related to cap materials and cap design, namely, the service life of high density polyethlyene (HDPE) geomembranes, internal shear strength of geosynthetic clay liners (GCLs), properties of the alternative barrier layer materials asphalt and paper mill sludge, slope stability of caps, and water balance modeling, are reviewed in this section.

6.2.2 Service Life of HDPE Geomembranes

Polyethlyene was first synthesized in 1933; however, it was not used in liners until the 1960s. HDPE geomembranes have not been used long enough to field test their durability. Nevertheless, the service life of HDPE geomembranes can be estimated using Arrhenius modeling to interpret the results of laboratory tests conducted at elevated temperatures (see Section 5). Research on the service life of HDPE geomembrane is currently being conducted by R.M. Koerner and Y. Hsuan, both of the Drexel University Geosynthetic Research Institute (GRI). The approach is described by Koerner, et al. (1992). They identified eight factors influencing degradation of HDPE geomembrane: (i) oxidation; (ii) chemical attack; (iii) hydrolytic effects; (iv) ultraviolet radiation; (v) nuclear radiation; (vi) biological attack; (vii) stress effects; and (viii) temperature effects. Of these factors, oxidation, hydrolysis, stress, and temperature are being considered in their study. Koerner and Hsuan defined service life as the time for depletion of antioxidants, initiation of degradation (i.e., induction time), and degradation to half-life of relevant strength properties. All of these factors may affect the effectiveness of a geomembrane as a barrier component. Strength properties are used rather than permeation properties because loss of antioxidant has little effect on permeation characteristics but can cause embrittlement and major changes in strength and elongation characteristics. The study is scheduled to be conducted over a 10-year time period. Based on the approximately three years of data currently available, depletion of antioxidants from HDPE geomembrane may take from 45 to 115 years and the geomembrane service life may be about 250 to 900 years, depending on the specific product and in-place conditions.

6.2.3 Internal Shear Strength of GCLs

As described by Daniel and Koerner (1993), a cap incorporating a GCL barrier layer can outperform a cap with a compacted clay layer. While a clay layer will frequently crack due to differential settlement, freezing-thawing cycles, and wetting-drying cycles, a GCL possesses some self-healing capability when subjected to these stresses. Also, a GCL is easier to install and to repair if damaged. One of the primary concerns with using GCLs in caps is the relatively low shear strength at mid-plane, as reported for hydrated specimens of some GCLs when tested in the laboratory (Rumer and Ryan, 1995). GCLs fabricated with internal reinforcing (e.g., needlepunched GCLs) have a higher peak strength than unreinforced GCLs, but, have been found to exhibit a significant decrease in strength when subjected in the laboratory to relatively large shear displacements. A field-scale study is currently underway to: 1) evaluate the mid-plane shear strength of different types of GCLs, 2) verify that caps incorporating GCLs will remain stable on 3H:1V slopes, and 3) verify that stable caps have a factor of safety of 1.5 or greater for slope stability. This study, which is being conducted jointly by the University of Texas, Drexel University, and GeoSyntec is described below.

Fourteen cap test plots, 20 m (66 ft) long by 3 m (10 ft) wide, were constructed with slopes of 3 horizontal: 1 vertical (3H:1V) and 2H:1V. The cap cross sections consist of, from top to bottom:

- 0.9 m (3 ft) thick cover soil layer, covered with a geosynthetic erosion mat and grassed;

- sand (0.3 m, or 1 ft thick) or geocomposite drainage layer;
- textured geomembrane/GCL composite or GCL barrier layer; and
- subsoil.

The GCLs used in the study were geotextile-encased and needlepunched (i.e., Bentomat, Bentofix I, and Bentofix II), geotextile-encased, adhesive-bonded, and stitched (i.e., Claymax), and a geomembrane-bentonite composite (i.e., Gundseal). To try to force shear failure through the GCL, all geosynthetic layers above the mid-plane of the GCL, including the upper geotextile of geotextile encased GCLs, were cut at the top of the slope. Also the cover soil was removed from the toe of the slope to eliminate toe buttressing. The deformations and moisture contents of the GCLs are being monitored at selected depths using tell-tales and fiberglass resistance cells, respectively. Two cap plots on 2H:1V slopes failed as blocks at 21 and 52 days after the cover soil was placed. In both cases, the GCLs were internally reinforced, with the failure surface located between the textured geomembrane and the woven geotextile of the GCL, rather than within the GCL. Based on the test results to date, the following observations have been made:

- The factor of safety against mid-plane failure of GCLs on 3H:1V slopes has been at least 1.5, based on the fact that 2H:1V slopes have been stable. (Theoretically, if a 2H:1V slope has a factor of safety > 1.0, then the factor of safety for a 3H:1V slope composed of the same materials is > 1.5.)
- For reinforced GCLs, the mid-plane strength shortly after construction has been greater than the interface strength between the woven geotextile of the GCL and the textured geomembrane.
- Bentonite sandwiched between two geomembranes was found be partially hydrated at one test plot. The mechanism for this wetting is unclear, though it may be due to water migration along the tell-tale and fiberglass resistance cell cables. At the two other test plots with a similar GCL type, the bentonite between the two geomembranes has remained dry. The GCLs in the remaining 11 test plots became hydrated within several months after construction as a result of absorption of moisture from the underlying, wet subsoils.

6.2.4 Properties of Asphalt

A composite asphalt barrier, composed of a 5-mm (200-mil) thick fluid applied asphalt membrane (FAM) overlying a 0.15-m (0.5-ft) thick hot-mix asphalt concrete (HMAC) layer, is being considered for the cap at the Hanford site as an alternative to traditional barrier layer materials (Freeman et al., 1994). Asphalt may be preferable to traditional barrier materials, such as geomembranes and compacted clay layers, since asphalt appears to have a longer service life than those materials. The expectation for long service life is based on the fact that asphalt occurs naturally and is known to have existed naturally in the subsurface for millions of years. Also, asphalt artifacts, up to several thousand years old, have been found by archaeologists. Asphalt has also been demonstrated to have a low permeability and be effective at controlling radon emissions. Asphalt barriers have been installed at several sites. However, in many cases, asphalt may not be selected over other barrier materials because of its relatively high cost [e.g., \$ 96/m^2 (\$ 80/yd^2) for the asphalt barrier proposed for the Hanford site compared to less than \$ 7/m^2 (\$ 6/yd^2) for installation of a 1.5-mm (60-mil) thick HDPE geomembrane]. Two concerns with asphalt use in a barrier are: (i) the potential for asphalt to creep; and (ii) asphalt cracking due to age hardening. Asphalt studies are currently underway at the Pacific Northwest Laboratory (PNL). Two of these recent asphalt studies are summarized below.

A 0.6 hectare (1.5 acre) prototype cap incorporating the composite asphalt barrier and an adjacent 18.6 m x 8.6 m (61 ft x 28 ft) barrier test pad were constructed during 1994 at the Hanford site, located near Richland, Washington. Laboratory tests conducted on five core samples from the HMAC component of the barrier gave an average hydraulic conductivity of 1.3×10^{-9} cm/s. Field falling head tests conducted at five locations and sealed double ring infiltrometer (SDRI) tests conducted at two locations gave average hydraulic conductivity values of 3.7×10^{-8} cm/s and 1.1×10^{-8} cm/s, respectively, for the HMAC. The hydraulic conductivity of the FAM component of the barrier, as measured in the laboratory for four field samples, was on the order of 1×10^{-11} cm/s. It should be noted that the potential exists for preferential flow through the HMAC layer. The highest HMAC hydraulic conductivity of 1.1×10^{-7} cm/s was found for a core sample taken along a vertical asphalt seam. Also, the SDRI test conducted over a vertical seam gave a somewhat higher hydraulic conductivity than the SDRI test performed over unseamed HMAC. Lateral flow of water within the asphaltic concrete, between horizontal asphalt seams, was observed during the SDRI tests.

Accelerated aging tests have been developed at PNL, permitting the rheological and chemical properties of asphalt to be determined as a function of age. The procedure is being validated by comparison with ancient asphalt artifacts, ranging in age from 500 to 4000 years, and asphalt from naturally occurring seeps. Accelerated aging tests still need to be completed on a number of asphalts to allow the long-term performance of composite asphalt barriers to be predicted.

6.2.5 Properties of Paper Mill Sludge

Paper mill sludges have been used since 1995 as the barrier layer for some caps constructed in Massachusetts and Wisconsin (Moo-Young and Zimmie, 1995). However, there is little information in the literature on the engineering properties of this sludge. Moo-Young and Zimmie performed laboratory tests to determine water content, organic content, specific gravity, permeability, compaction, consolidation, and strength for seven paper mill sludges. They found that the sludges had a high initial water content ranging from 150 to 270 percent, an initial hydraulic conductivity ranging from about 1 x 10^{-7} to 5 x 10^{-6} cm/s, and behaved similarly to a highly organic soil.

Moo-Young and Zimmie (1995) also performed laboratory tests on six samples of a sludge used as the barrier layer material in a cap. Three samples were obtained shortly after construction and the other three samples were taken at nine, 18, and 24 months after construction. The results of the laboratory tests on these undisturbed samples indicated that the water content and hydraulic conductivity of the sludge decreased somewhat over time as the sludge consolidated and biodegraded (i.e., mineralized to become more like a soil). The depth of frost penetration in the sludge barrier layer has been monitored since 1992. To date, the frost layer has not penetrated into the sludge layer due to the protection provided by the overlying cap layers and the high water content of the sludge. Based on the results of laboratory tests over a range of water contents, if a sludge layer is subjected to freezing and thawing cycles, the hydraulic conductivity of the sludge may increase by one to two orders of magnitude (Moo-Young and Zimmie, 1995).

6.2.6 Slope Stability

Static and seismic slope stability analyses are typically carried out as part of cap design. The general procedures for evaluating the static and seismic stability of landfills caps have been summarized by Othman et al. (1995). One of the more critical parts of the analyses involves the selection of the appropriate shear strength values to use for the cap materials and material interfaces. While there are published shear strength values, it is recommended that project-specific testing be conducted under the expected field conditions (e.g., soil moisture content and unit weight, consolidation load and time, interface wetting conditions, normal stresses, shear direction, shear displacement rate and magnitude). Other factors which may affect the long-term shear strength properties of caps, such as freezing-thawing cycles, heating-cooling cycles, and creep, should also be considered. However, no consistent standard of practice currently exists for directly addressing the potential effects of these factors on slope stability. They may be accounted for indirectly through use of a higher safety factor. Other measures, such as increasing the thickness of cover soils above the critical layers to provide thermal insulation and isolation from the environment, may be beneficial in some ways but detrimental in others.

Seismic design of caps has been brought to the attention of the engineering community with the promulgation of the Resource Conservation and Recovery Act (RCRA) Subtitle D regulations for municipal solid waste landfills (40 CFR 258) which became effective in 1993 and with the observed field performance of several landfills during recent earthquakes in California. Under the Subtitle D regulations, the performance of landfills located in seismic impact zones must be evaluated. A seismic impact zone is an area with 10 percent or greater probability that the maximum horizontal accelerations in bedrock will exceed 10% of the earth's gravitational pull within 250 years. With this definition, seismic impact zones cover most of the western U.S. and part of the central and eastern U.S. Overall, these zones cover approximately 40 percent of the U.S., including areas where seismic design may not have been considered in the past. Furthermore, under Subtitle D regulations, a geomembrane/compacted clay composite barrier layer be incorporated in caps over certain landfills, e.g., landfills containing composite bottom barriers. Incorporating a geomembrane (as well as other geosynthetics) in the cap, however, may make the cap more susceptible to instability and deformations induced by seismic loading due to relatively low interface shear strengths.

Observations of the seismic performance of numerous (over 30) landfills in California, based primarily on field inspections after recent earthquakes, indicate that, in general, the landfills performed well but that some limited damage did occur to soil covers (Anderson and Kavazanjian, 1995). Typical impacts of earthquakes on landfills included: cracking of interim cover soils, some downslope movement of the cover soils, and disruption of gas collection systems. No observations are available for landfills with geosynthetic final caps. However, significant displacements could occur along interfaces between geosynthetics and other geosynthetics or soils.

The state of practice for evaluating the stability and deformation of landfills under seismic loading has recently been summarized by Anderson and Kavazanjian (1995). As described in their paper, the seismic design of caps involves four steps:

- characterization of the ground motions for the design earthquake,
- evaluation of the response of the landfill to the design earthquake,
- calculation of the stability and deformation of the entire waste mass, and
- evaluation of the ability of the cap to maintain its integrity when subjected to the calculated ground motions.

An important consideration relative to seismic loading is the potential for the waste mass to amplify free-field ground accelerations. Amplification is known to occur in soil deposits and has been observed at the OII Landfill near Los Angeles, the one landfill at which instrument measurements are available. A threefold amplification of peak ground acceleration from the base of the landfill to the crest was recorded during the 1992 Landers earthquake at the landfill. Amplification can be predicted analytically if the design ground motions and the dynamic properties of the waste are known. However, in the central and eastern U.S., there is considerable uncertainty about the nature of the design motions. There is also considerable uncertainty about the dynamic properties of wastes, especially municipal solid waste. Factors influencing seismic resistance of the landfill cover are the yield acceleration of the cover system and the allowable seismic deformations. Yield acceleration depends on the shear strength of the considered failure mass. For geosynthetic materials, it is not clear when peak, residual, or deformation-compatible strengths should be used. Additionally, most interface strength tests involving geosynthetics are run at displacement rates well below the rates occurring during seismic loading. The interface shear strength may be higher or lower at the actual displacement rate caused by the earthquake, depending on the material used and on moisture conditions. It is extremely difficult to design an unconditionally stable cap using geosynthetic materials in areas of high seismicity. Even in areas of low to moderate seismicity, unconditional stability may be difficult to obtain for covers containing geosynthetics that are steeper than 3H:1V. Allowable seismic deformations of the cap are based on practical considerations rather than rigorous analysis. For noncritical caps, seismic deformation of the cover may be handled as a maintenance issue.

6.2.7 Water Balance Modeling

One of the principal functions of a cap is to limit percolation of water into underlying contaminated materials. The design of the cap to limit percolation of water through it requires a model that can predict water percolation through caps. Such models are called "water balance models". Because water balance models are essential to performance-based design, they received considerable attention and discussion at the workshop.

Several computer models are available to evaluate the hydraulic performance of landfill caps. The most widely used model is the USEPA Hydrologic Evaluation of Landfill Performance (HELP) model (Schroeder et al., 1994a,b). The HELP model has the advantage over other models in that it contains default climatic and material properties data, is relatively easy to use, and is accepted by the regulatory community. The HELP model, however, contains a number of simplifying assumptions that make the model inappropriate in certain cases. For instance, the HELP model, which generally assumes a unit gradient to model vertical drainage, cannot be used to simulate the performance of partially saturated layers in very dry environments. Another model that has been used to model caps is UNSAT-H. This model, which was developed by Pacific Northwest Laboratory by Fayer and Jones (1990), does not contain the default data of the HELP model and requires extensive computer time to conduct simulations. However, UNSAT-H performs a much more rigorous analysis of unsaturated flow by solving the relevant partial differential equation, rather than assuming unit gradient as the HELP model does. A limited number of field studies and analytical assessments have been performed to evaluate the reliability of models as tools to predict trends and magnitudes of the different landfill water balance components. However, the findings from these studies are not in general agreement. For example, some of the studies found that the models overpredicted percolation in humid climates and underpredicted percolation in arid climates, while other studies concluded the opposite. In many cases, the models were unable to predict short-term trends. However, for a number of cases they appeared to give reasonable predictions of cumulative water balances. The developers of the HELP model report that the approximate annual errors in water balance components calculated by the HELP model are:

- runoff = ± 25 percent of actual value or ± 2 percent of precipitation,
- evapotranspiration = ± 10 percent of actual value or ± 6 percent of precipitation,
- percolation = ± 10 percent of actual value or ± 0.3 percent of precipitation, and
- lateral drainage = ± 7 percent of precipitation.

In a study of field-scale caps [30 m x 30 m (100 ft by 100 ft)] in Atlanta, Georgia and East Wenatchee, Washington, incorporating compacted clay barrier layers, the measured water balances of the caps were compared to the simulated water balances using the HELP and UNSAT-H models (Khire, 1995). The caps were constructed with two layers: (i) a 60- or 90-cm (2- or 3-ft) thick compacted clay barrier layer; and (ii) a 15-cm (0.5-ft) thick vegetated surface layer. Collection of climate, runoff, soil water content, and percolation data for each cap began in 1992 and is ongoing. Runoff and percolation are collected in tanks and measured, while soil moisture is measured using time domain reflectrometry. The cumulative percolations measured for the caps in Georgia and Washington were about 24 cm (9.4 in.) and 3.1 cm (1.2 in.), respectively. Khire found that for the Georgia cap, HELP overpredicted percolation by about 320 percent and and UNSAT-H underpredicted percolation by about 24 percent. The relatively large error in percolation predicted by the HELP model was attributed primarily to the underestimation of runoff. Both models overpredicted percolation by about 43 percent for the cap in Washington. This deviation was believed to have been caused by preferential flow through cracks and animal borrows observed in the barrier layer. Khire also found that while both models captured seasonal variations in runoff, evapotranspiration, soil water storage, and percolation, UNSAT-H simulated the variations more accurately.

6.3 FIELD PERFORMANCE OF THE TECHNOLOGY

Some cap failures and findings from other field studies being conducted in the U.S. and Europe are described in this section.

6.3.1 Cap Failures

There have been a number of documented cases of cap failures, with most of the failures occurring during or shortly after construction and resulting from excessive erosion, build up of pore water pressures in the cap layers, lack of a drainage layer, a drainage layer with insufficient capacity, or incorrect "estimation" of the shear strength between the cap layers (Boschuk, 1991). While most of these failures did not involve rupture of the barrier layer, they were costly to repair. In one recent case, severe erosion problems developed because the cap slopes were relatively long (180 m (600 ft)) and the cap drainage layer was designed with an outlet only at the toe. At some locations, the cap had eroded to the top of the clay barrier layer. The erosion problem was exacerbated in some cases because the drainage layer outlet at the toe of the slope had not been constructed. In these cases, the trapped water eventually caused pore pressures to become excessive, causing sloughing of the overlying soil layers at the toe of the slope. At another landfill, a gabion-lined channel for surface water slid down the cap slope due to liquifaction of the fine sand beneath it brought about by high pore pressures. It was noted that most of the failures occurred in states with relatively restrictive, prescriptive cap designs rather than more flexible performance objectives, suggesting that, in these states with prescriptive cap designs, greater attention may have been given to regulatory compliance than to the design itself.

6.3.2 Field Study in Hamburg, Germany

Six test caps [10 m (33 ft) wide x 50 m (160 ft) long] were constructed in 1987 to evaluate the field performance of different cap configurations (Melchior and Miehlich, 1989; Melchior et al., 1994). The caps were constructed with a 0.75-m (2.5-ft) thick sandy loam topsoil layer, underlain by a 0.25-m (0.82-ft) thick fine gravel drainage layer. The drainage layer was underlain by one of four barrier layer types: 1) a 0.60-m (2.0-ft) thick compacted clay layer, 2) a HDPE geomembrane/clay composite layer with welded geomembrane panels, 3) a geomembrane/clay composite layer with overlapped geomembrane panels, and 4) a compacted clay layer overlying a 0.60-m (2.0-ft) thick fine sand wicking layer and a 0.25-m (0.82-ft) thick coarse sand/fine gravel capillary barrier.

Each of the four cap configurations was constructed on 4 percent or 20 percent slopes, and several configurations were constructed for both slopes. Climate, lateral drainages from the topsoil and drainage layers, runoff, and percolation data are being collected. Soil moisture data are also being collected from several test caps using neutron probes and tensiometers. The preliminary findings of this field study are summarized below.

For the caps with the compacted clay barrier layer, little percolation was observed for the first 20 months after construction. Beginning in August 1989, percolation began to increase and show a correlation with precipitation events. The summer of 1992 was very dry, and tensiometers indicated that the clay layers had undergone more drying than usual. This drying resulting in an almost tenfold increase in percolation measured during the fall of 1992 over percolation recorded a year earlier. Flow through the capillary barrier under one of the compacted clay layers was first observed during this time. When excavations were made into the caps in 1993, the clay layers were found to have small fissures and contain plant roots. Since 1993, the network of plant roots has developed further, contributing to preferential flow paths and desiccation cracking. Percolation through the compacted clay layers is still increasing and was about 200 mm (8 in.) in 1994.

The caps constructed with a composite barrier layer performed much better, and no percolation has been observed. However, during the summer and fall months, the matric potential in the clay layers increased and drainage from the clay layer was recorded. This drainage, which had typically been less than 1 mm/yr (0.04 in./yr), has been attributed to thermal gradients. During the summer and early fall, the temperature at the top of the clay layer has been greater than that at the bottom of the clay layer, and water likely flows in liquid and vapor phase from the hotter to cooler regions resulting in the measured drainage. The water loss caused by thermal gradients has not caused shrinkage of the clay, although the potential for future shrinkage exists.

6.3.3 Field Study in Beltsville, Maryland

Six lysimeters [14 m (45 ft) wide x 21 m (70 ft) long x 3.0 m (10 ft) deep] with 20 percent side slopes were constructed between May 1987 and January 1990 to evaluate caps incorporating either a compacted clay barrier layer, a rock capillary barrier, or bioengineering management, which combines enhanced runoff and plant transpiration (Schultz et al., 1995). The bioengineering management option used alternating aluminum and fiberglass panels as the surface layer over about 90 percent of the cap with moisture-stressed vegetation (i.e., Pfitzer junipers) located along gaps in the panels. This latter option requires periodic maintenance and is intended to be used when significant subsidence of the underlying waste is expected. The six lysimeters were constructed with the following caps: (1) bioengineering management, with the initial water level 90 cm (35 in.) above the bottom of the lysimeter; (2) bioengineering management, with the initial water level 190 cm (75 in.) above the bottom of the lysimeter; (3) reference lysimeter similar to lysimeter 1, except without the surface panels and vegetated with fescue grass; (4) rip-rap surface layer and gravel drainage layer over a compacted clay layer; (5) vegetated soil surface layer, gravel drainage layer, and compacted clay layer over a gravel capillary barrier; and (6) vegetated soil surface layer and gravel drainage layer over a compacted clay layer. All of the caps were constructed over native soil. Rainfall, runoff, deep drainage, and soil moisture content data were collected for the lysimeters.

The data collected through 1994 reveal that initially ponded water in lysimeters 1 and 2 was removed by the plants within two years after construction. The soils in these lysimeters have generally become drier over time. The initial water level in lysimeter 3 rose until it was near the surface of the lysimeter and the water had to be pumped out. Deep drainage has been measured for this lysimeter every year of the study. Except for the deep drainage from lysimeter 5, which occurred during 1994, deep drainage has not been observed from lysimeters 4 to 6. It has been noted that the moisture content of the clay layer in lysimeter 4 has been increasing, indicating the possibility of future seepage through the clay. The moisture contents of the clay layers in lysimeters 4 and 6 show some seasonal cycling, with the lowest moisture contents being measured in the summer.

6.3.4 Field Study in East Wenatchee, Washington

Two caps [30 m x 30 m (100 ft x 100 ft)], one with a 60-cm (2-ft) thick compacted clay barrier layer overlain by a 0.15-m (0.5-ft) thick vegetated soil surface layer and the other with a 75-cm (2.5-ft) thick sand capillary barrier layer used in lieu of a clay resistive barrier, were constructed and monitored (Khire, 1995). Climate, runoff, percolation, and soil moisture data have been continuously collected since November 1992. The collected data show that cumulative percolations through the resistive and capillary barrier layers have been 3.1 and 0.5 cm (1.2 and 0.2 in.), respectively. Most of the water movement through the capillary barrier occurred in winter 1993, primarily due to snowmelt from the relatively high snowfall (169 cm (66.5 in.)) occurring that year.

6.3.5 Field Studies in Richland, Washington

Since 1985, PNL and Westinghouse Hanford Co. have been working to develop a cap design for the Hanford site. Field tests have been conducted for over the past seven years using lysimeters to evaluate the performance of different cap materials and configurations (Petersen et al., 1995). Currently, 24 lysimeters are being monitored to assess the effects of varying precipitation, surface soil, and vegetative conditions. No drainage has been measured from lysimeters with vegetated or nonvegetated silt-loam surfaces under normal precipitation conditions; however, some drainage has occurred from lysimeters with nonvegetated silt-loam surfaces under extreme precipitation conditions (i.e., three times normal). Significant quantities of water have drained from lysimeters with gravel and sand surface layers. The performance of one lysimeter with a 1.5-m (5-ft) thick layer of nonvegetated silty loam was modeled over a six-year period using the HELP and UNSAT-H models. The HELP model simulation prediction was 1800 percent greater than the observed drainage, while the UNSAT-H model simulation prediction was at 52 percent of the observed drainage.

A prototype cap [0.6 hectare (1.5 acre)] was also constructed at the site in 1994 using the following layers, from top to bottom (Gee et al., 1994; Wing and Gee, 1994; Peterson et al., 1995):

- 1.0-m (3-ft) thick silt loam/admix gravel surface layer,

- 1.0-m (3-ft) thick silt loam protection layer,
- 0.15-m (0.5-ft) thick sand filter,
- 0.30-m (1-ft) thick gravel filter,
- 1.5-m (5-ft) thick fractured basalt riprap capillary barrier and biobarrier,
- 0.3-m (1-ft) thick gravel cushion and drainage layer,
- 0.15-m (0.5-ft) thick composite asphalt barrier layer,
- 0.10-m (0.3-ft) thick top course, and
- compacted soil foundation.

The composite asphalt barrier layer was discussed previously. The water and wind erosion, biointrusion, revegetation success, and water balance of the cap continue to be monitored. Water balance components being recorded include: precipitation, runoff, snow depth, soil moisture, and percolation. The water balance is being evaluated under normal and stressed (i.e., irrigated) conditions.

6.3.6 Field Study in Idaho Falls, Idaho

A replicate field test program is underway at Idaho National Engineering Laboratories to compare the hydraulic performance of four caps designs: (1) 1-m (3-ft) thick vegetated soil layer over a geomembrane/ compacted clay composite barrier layer; (2) 2.5-m (8.2-ft) thick vegetated soil layer with a 0.5-m (1.6-ft) thick biobarrier located within it at 0.5 m (1.6 ft) below the ground surface; (3) the same design as cap 2, except the biobarrier is located 1 m (3 ft) below the ground surface; and (4) 2.0-m (6.6-ft) thick vegetated soil layer. The test plots were constructed in 1993. Two vegetation types have been used, a native mixed plant community and a monoculture of crested wheatgrass. Both vegetative covers were considered since planted monocultures may be reinvaded by native plant species in the future, and a mixed native plant community may be more resilient to environmental fluctuations. The test plots will, at times, be subjected to burrowing animals, ants, and high levels of irrigation. Climate, soil moisture, and percolation data are being collected. Soil moisture is being measured using a neutron probe and time domain reflectometry. No data are available.

6.3.7 Field Studies in Albuquerque, New Mexico

A large-scale field test program is being conducted at Sandia National Laboratories to compare the performance of three caps [each 13 m (43 ft) wide and 100 m (330 ft) long]; one incorporates a compacted clay, another a geomembrane/clay composite, and a third has a geomembrane/GCL composite barrier layer. The test caps were constructed and instrumented during 1995 (Dwyer, 1995). The hydrology and the erosion of the caps are being monitored. Another three caps are scheduled to be constructed in 1996, each having capillary barriers with one vegetated to enhance evapotranspiration. No data on performance had been collected at the time of the workshop. Monitoring is expected to continue for the next several years.

In another field study under limited evapotranspiration conditions, the performances of two gravel capillary barriers were compared (Stormont, 1995). When a uniformly-graded sand layer was placed between the gravel barrier and the overlying silty sand, the lateral drainage above the barrier increased while drainage through the gravel decreased.

6.4 ASSESSMENT OF THE TECHNOLOGY

Several aspects of cap materials and design, namely the service life of HDPE geomembranes, the internal shear strength of GCLs, the properties of asphalt and paper mill waste, the slope stability of caps, and water balance modeling, have been reviewed. The findings from this review are presented below:

- Preliminary results from long-term laboratory testing of HDPE geomembranes indicate that service life of the geomembranes should be at least several hundred years, depending on the specific product and the in-place conditions. If HDPE geomembranes provide this length of service, they appear to be a good investment relative to other more costly barrier layer materials such as compacted clay.
- Based on ongoing field-scale tests of prototype caps incorporating GCLs, the factor of safety against mid-plane failure of GCLs on 3H:1V slopes has been observed to be 1.5 or greater. These results are encouraging since a cap with a GCL is a more effective barrier against infiltration than a cap with a compacted clay layer. If GCLs remain stable on cap slopes, they appear to be preferable to a compacted clay layer.
- Asphalt and paper mill waste have been used as the barrier layer material for caps. However, the service life of these materials is uncertain. While asphalt may have a longer service life than traditional barrier layer materials, it is significantly more expensive. Also, the significance of flow along asphalt seams is unknown. Paper mill waste involves reuse of a waste product; however, it may be affected by some of the same processes (e.g., freezing/thawing cycles) that adversely impact compacted clays.
- The shear strength properties of cap components should be evaluated by conducting project-specific tests under the expected field conditions. The effect of freezing-thawing cycles, heating-cooling cycles, and creep on the long term shear strength properties should also be considered by using a higher safety factor or by increasing the thickness of cover soils above the critical layers to provide thermal insulation and isolation from the environment.
- Observations of the seismic performance of several landfills have indicated that, while landfills have performed relatively well, cracking and some

downslope movement of interim cover soils and disruption of gas collection systems occurred. The effect of the seismic motions on the integrity of caps incorporating compacted clay layers or geosynthetics has not been well documented. However, there is concern that compacted clay layers could be prone to cracking and that displacements may occur along interfaces between two geosynthetics and between a geosynthetic material and soil.
- There is considerable uncertainty about the dynamic properties of wastes, especially unconsolidated municipal solid waste, and about the appropriate shear strength values to use for geosynthetic materials under dynamic loading.
- Allowable deformations of caps under seismic conditions are based on practical considerations rather than rigorous analysis. For non-critical caps, seismic deformation of caps may be handled as a maintenance issue.
- Water balance models are used to predict the performance of caps in terms of water percolation through the cap. Available water balance models that are applied to caps typically contain numerous simplifying assumptions and have been inadequately verified by field data. Since these models provide wide-ranging estimates of water infiltration under site-specific conditions, their main value may be to compare alternative designs utilizing different cap configurations and materials.

Based on the field monitoring of test caps and cap failures, the following remarks are made regarding cap performance:

- Several examples were given of inadequately protected compacted clay barrier layers that degraded after a few years as a result of desiccation, root penetrations, or both. Although covering a clay layer with a geomembrane provided greatly improved protection, one case suggested that thermally induced flow could eventually desiccate even a geomembrane-covered clay barrier layer.
- There are few data on the performance of caps incorporating geomembranes and compacted clay layers and even fewer data for caps that include capillary barriers or employ surface vegetation to enhance evapotranspiration. Therefore, we are unable to document how well existing caps are performing.
- The performance life of caps has not been established. Although the service life of some components of the system can be estimated, the functional life of the surface layer and barrier layer are not well documented and the long term performance of constructed cap systems has not been adequately documented.
- A number of cases of cap failures have been documented; however, most of these failures could have been prevented through proper design and construction. It is felt that some failures may have resulted from preoccupation with regulatory compliance rather than engineering design considerations.

The primary factors adversely affecting cap performance for each of the six basic layer components are summarized in Table 6-2.

TABLE 6-2 Factors Affecting Cap Performance

Layer	Factor
Surface Layer	• Erosion • Evapotranspiration • Native versus Exotic Vegetation • Appropriate Armoring for Side Slopes at Arid Sites
Protection Layer	• Erosion • Slope Failure Due to Pore Pressure Buildup • Animal Burrows
Drainage Layer	• Clogging • Insufficient Capacity • Insufficient Drainage Layer Outlets
Barrier Layer	• Cracking due to Desiccation, Deformations From Waste Settlement, or Seismic Motions (Clay, Paper Mill Sludge) • Root Penetration • Resistance to Gas Migration (GCLs) • Stability • Creep (asphalt) • Service Life
Gas Collection Layer	• Adequate Cover Over Waste
Foundation Layer	• Adequate Strength

6.5 NEEDS

The following needs related to cap technology are presented based on the assessment of caps presented in this section.

- Data are available that demonstrate that the performance of compacted clay barrier layers in caps will deteriorate over time. Even so, compacted clay barrier layers are still being used in caps, primarily because they are specified in regulations. There is a perception that it may be difficult to obtain regulatory approval to use alternative barrier materials to a clay layer. This situation could be improved if guidelines were available for demonstrating the equivalency of performance among the different options for cap components.
- Few data are available concerning the hydraulic performance of traditional caps with resistive barriers. There are even fewer performance

data for caps containing capillary barriers. More data need to be collected to assess cap performance. Data are especially needed to bring about regulatory and community acceptance of alternative cap configurations. While some of these data can be collected from currently instrumented sites, other additional sites will probably need to be monitored.

- The expected performance life of caps is uncertain. Studies are underway to assess the service life of some individual cap components. However, the long term service life of these components in a constructed surface barrier system has not been adequately studied.
- There have been a number of documented cases of cap failures; however, most of these failures could have been avoided through proper design and construction. There is a need for more guidance on cap design and for independent peer review of completed designs prior to construction. Compliance with regulations is not a sufficient check on a completed design.
- More field observations on the effect of seismic motions on the integrity of caps incorporating compacted clay layers and/or geosynthetics need to be made.
- The shear strength at interfaces between materials is known to be an important factor affecting the physical stability of caps on slopes. The shear strength is also affected by freezing-thawing cycles, heating-cooling cycles, and creep. Standard procedures for evaluating interfacial strength need to be developed.
- More information is needed on the dynamic properties of wastes, and on the appropriate dynamic shear strength values to be used for geosynthetic layer materials.
- Other alternative barrier layer materials, such as asphalt and paper mill waste, appear promising for future use. However, more information needs to be collected on the long-term performance of these materials.
- Available computer models for simulating the hydrologic and hydraulic performance of caps need to be verified by comparison with field data, and modified as necessary.

6.6 SUMMARY/RECOMMENDATIONS

The purpose of this section was to define the current technological status of waste containment caps, including: material performance, design methods, field performance, and field monitoring. With respect to material performance and design method, the following issues were considered: service life of HDPE geomembranes; internal shear strength of GCLs; properties of asphalt and paper mill sludge; slope stability of caps; and water balance modeling. Cap failure cases and study results from field monitoring at sites in arid and temperature climates have been reviewed. Based on these considerations and review, it is concluded that:

- The service life of caps is uncertain. The performance of a compacted clay barrier layer in a cap will likely deteriorate over time, while an HDPE geomembrane barrier layer may perform satisfactorily for several hundred years. The long-term performance of cap components, and caps as systems, needs to be studied further.
- The slope stability aspects of caps are complex. The shear strength properties of cap components and component interfaces may be impacted by moisture conditions, stress, creep, temperature, and seismic motions. However, there are no standard procedures for evaluating interfacial strength or accounting for these effects. In addition, the dynamic properties of buried wastes are generally not well understood. Under certain conditions, the waste may amplify seismic-caused ground movements which can be transmitted to the cap. Standard procedures need to be developed for evaluating the shear strength of cap materials and additional research needs to be performed on the seismic properties of various types of buried wastes.
- There are few published data on the field performance of constructed cap systems. More data need to be collected. The collected hydraulic data can also be used to verify currently used models for simulating the hydraulic performance of caps.
- Over emphasis on regulatory compliance inhibits innovative and creative cap design, particularly with regard to the selection of alternative materials for cap components. Greater emphasis needs to placed on how the design will affect cap performance. Technical guidelines for demonstrating alternative cap equivalency need to be developed.
- There have been a number of documented cases of cap failures; however, most of these failures could have been avoided by proper design and construction. This problem may be minimized by independent peer review of design and more rigorous QA/QC during construction.
- Nontraditional materials, such as asphalt, paper mill sludge, capillary drains, capillary barriers, plants with high tranpiration capability, and GCLs have undergone limited testing in the field but look promising for future use in caps.

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SECTION 7

FLOORS AND BOTTOM BARRIERS: INDIGENOUS

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7.1 OVERVIEW

Natural subsurface barriers can be utilized as the floor of a waste containment system. However, the effective utilization of a natural or indigenous barrier depends on a good understanding of the overall site characteristics, including the integrity of the indigenous formation and how it can be integrated into an engineered containment facility. The objective is to eliminate or limit contaminant flux through the natural and constructed barriers that form the containment structure.

Generally, engineered containment barriers are designed, constructed, and then subjected to quality assurance (QA). Natural barriers, on the other hand, simply *exist*. Their use as part of a containment system depends on a determination of their material and physical properties, which may vary from point-to-point within an otherwise apparently consistent stratigraphic unit. For European perspectives on this topic, see DalPra and Magnini (1993) and Dorhofer (1993).

Writing on the art of war in 2500 BC, the Chinese general Sun Tsu stated that the first requirement of a successful campaign was to know the enemy. Similarly, the designer of a containment system must know the site. Evaluation of a specific site depends on careful exploration, guided by a thorough understanding of transport mechanisms under saturated and partially saturated conditions. This chapter on indigenous (or naturally occurring) barriers addresses most of the key issues that must be considered in the decision to utilize an existing indigenous formation as an integral component of a containment system. These include:

- characterization of the site,
- determination of the natural barrier transport properties, and
- evaluation of potential barrier property changes resulting from exposure to the contaminant and other environmental changes.

7.2 DETERMINING STRATIGRAPHY

7.2.1 Background

Stratigraphic studies should be directed towards finding and evaluating the conduits, or preferential migration paths, in natural formations that are under consideration as components of a containment system. Stratigraphy evaluation is, therefore, ultimately aimed at a determination of the distribution of hydraulic conductivity in the proposed formation.

A well-executed program to determine site stratigraphy consists of several elements, including: (1) a review of available literature to understand the regional and general site stratigraphy; (2) a definition of the objectives of the stratigraphic characterization (i.e., what sorts of decision making will be made based on the stratigraphic interpretation); (3) the development of a conceptual model of the site stratigraphy; and (4) a field program consisting of one or more exploration methods to generate stratigraphic data for testing and refining the stratigraphic model. The preferred procedure is for the field program to be phased, with each phase of testing leading to further refinement of the evolving stratigraphic model.

7.2.2 State of Practice in Determining Stratigraphy

a. General Guidelines. In most cases, multiple methods should be used in stratigraphic determinations. This provides greater confidence in the final stratigraphic model. It is also essential that each phase of the investigation have well-defined objectives. These objectives should be made clear to all personnel, including the drillers and instrument technicians involved with the investigation. Furthermore, all personnel should be qualified and experienced in their portion of the field work, or at least *directly supervised in the field* by experienced persons.

b. Intrusive Methods. Most intrusive methods have been used long enough so that the geoscience professions have confidence in the procedures and results. However, intrusive methods produce penetrations/perforations into or through potential barrier strata which must then be sealed. The primary sealing method is to grout from bottom-to-top with a cement/bentonite or high-solids bentonite grout. Field data are available from sites where monitoring wells were double-cased through contaminated zones into uncontaminated zones, indicating that sealing is effective under even severe conditions.

Soil Borings. This well-established technology is still the ultimate geologic tool (the relevant ASTM standards for many of the procedures include D1452, D1586, D1587, D2113). Although good drillers and loggers are essential, they must be fully aware of the investigation's objectives. Sampling boreholes in the normal manner can be slow, allowing time for small amounts of contaminants to transfer from one stratum to another. The amounts transferred, although small, may affect groundwater chemical analysis results. Therefore, multiple-use boreholes (e.g., used also as monitoring wells) are to be avoided where possible. Continuous sampling is needed, especially through critical zones containing discontinuities such as lenses of non-matrix material or fractures. Stress relief is a problem in laboratory testing of borehole samples. In highly overconsolidated deposits, such as shales, stress relief effects can invalidate strength or hydraulic conductivity test results.

Test Pits. The advantage of a test pit is that it provides direct observation of a larger-scale cross-section of the formation. Structural bracing may be required for safety, or even special protective apparatus may be needed in some cases.

Cone Penetrometer Testing (CPT). While CPT technology is mature and accepted (ASTM D3441), new downhole sensing devices are being developed rapidly. The capabilities of the CPT system are limited by the normal rod sizes; only a certain amount of sensor cable can be used. An advantage of CPT (or other push technologies) is that few, if any, cuttings (which can be expensive to dispose) are produced. Disadvantages include the potential for carrying contaminants down during the advancement of the cone and the need for proper grouting. Downhole bottom-to-top grouting without prior rod removal is recommended for contaminated areas, especially.

CPT has been used to delineate ionic and radioactive contaminant plumes. The methodology is ideal for downhole ultrasonics because of the intimate contact of soil and sensor. It is good for DNAPL interface sampling. However, if exploration is to proceed below a DNAPL or other contaminated zone, it is necessary to case off the contaminated zone. Thus, deeper exploration requires casing or nested rod sizes. Further discussion of current trends in CPT Technology is given in Shinn, et. al., 1995.

Downhole Geophysics. In most cases, geophysical methods can supplement but not replace detailed intrusive sampling. Although most of the downhole geophysical methods are well established in geophysics, they have been sparingly used on remedial site investigations. The techniques and equipment, originally developed using analog data reduction, need upgrading for digital computation. In general, gamma logs provide better data than neutron logs. Shear-wave and ultrasonic logging are good for finding discontinuities such as fractures. Ultrasonic logging requires a borehole caliper log for accuracy. These units can be pulled through a slanted or horizontal hole, but at a higher risk of losing the (expensive) equipment because of hole collapse. Cross-hole techniques improve the resolution of downhole geophysics. The ideal situation would involve an integrated sensing network, but this is not currently done. Other useful downhole techniques include temperature gauges, real-time groundwater pressure transducers, and flowmeters for detecting both vertical and horizontal flows.

c. Non-Intrusive Geophysics. Currently, surface geophysical techniques are capable of delineating only gross features. They are useful for evaluating continuity between intrusive sampling points. Various geophysical methods that apply to environmental work are described by Benson and Yuhr, 1995.

<u>Seismic Reflection and Refraction</u>. This technique has been successful in the upper units of the clay-sand geology in the Gulf Coast area. However, it should be considered qualitative, not quantitative. Data interpretation is currently quite subjective.

<u>Electromagnetic Induction</u>. This methodology has been successful in delineating strongly differing strata, such as clays, sands, and limestones. It has poor resolution for minor subsurface feature changes, but can locate "windows" in barrier strata if they are large enough and at a depth that is relatively shallow compared with their lateral extent. The method has been used with good success as a tool for interpolating between intrusive sampling points. When this method is combined with regression analysis on the "hard data" points, depths to strata changes can be determined to within about ±10% to ±20% (Lawrence and Boutwell, 1990). Electromagnetic induction has been used to locate fractures in limestones. It is fast, and large areas can be surveyed relatively quickly.

<u>Electrical Resistivity</u>. The statements made above for electromagnetic induction also apply for electrical resistivity. However, the electrical resistivity method is generally much slower unless specialized equipment is used.

<u>Ground-Penetrating Radar</u>. This method has been very successful in sands above the groundwater table. However, heavy clays and groundwater reflect the signal, preventing deeper penetration. Where applicable, it is excellent for delineating irregularities such as faults, fracture zones, etc.

d. Interpretive/Presentation/Predictive Methods. In every subsurface exploration, the first step should be development of a simple conceptual model of the subsurface conditions based on available data from geologic publications, previous exploration work, and observation of strata exposed in existing cuts, etc. A few simple cross-section profiles may suffice to construct the first version of the model. This model should be continually reviewed and updated as more information becomes available.

At many remediation sites, investigations may extend over many years and have been performed with varying objectives and/or with different levels of expertise. This frequently leads to data inconsistencies, especially concerning the minor features that govern contaminant transport. Eventually, the amount of data becomes so great that a computerized data management system is required for efficient interpretation. Unfortunately, there can be a tendency to be overly dependent on the interpretative capabilities of the available computer software, with too little independent thinking by trained persons. Still, computer techniques are useful in the relatively rapid production of visual presentations, such as isopach maps, cross-sections, and isometric profiles.

Experience with geostatistical methods, such as kriging, although mixed, has been generally poor. The levels of confidence achieved can easily be an artifact of some necessary assumption made in the analyses.

7.2.3 Field Performance of the Technology

The investigative tools currently available to the geoscience professions are adequate for determining stratigraphy at a point. Except in the case of fractured media, the interpretive methods also appear adequate. There have been numerous cases where it has been possible to compare stratigraphy, as revealed in excavations, with predictions based on the findings from typical field explorations. The correlations have been found to be satisfactory when a proper field investigation had been made. Discrepancies (which can be significant in transport analyses) generally occur in complex and/or fractured strata situations, especially when only a minimal investigation has been made.

7.2.4 Assessment of the Technology

The selection of exploration methods for determining stratigraphy at a site will be based on a variety of considerations. Although the circumstances will vary from site to site, the following three considerations tend to be common to all:

- the anticipated geologic setting, in particular the types of lithology and their stratigraphic sequence, will generally indicate the preferred exploration methods;
- the decision to implement the indicated exploration methods will be influenced by the relative costs of implementation, which, in turn, will be dependent on the scale of the project; and
- the objectives of the stratigraphic characterization (e.g., contaminant delineation, hydrogeologic modeling, consolidation analyses, stability evaluations, or other) will also affect the selection of exploration methods to be used.

Early development and repeated refinement of the site conceptual stratigraphic model is essential, particularly for large, multi-phased exploration programs. Development of a stratigraphic model early in the project is helpful in the process of defining the objectives of more detailed stratigraphic characterization. Ongoing testing and refinement of the stratigraphic model during large field programs helps to ensure that a project team is reviewing data from a completed phase of a field program before proceeding to the next phase. A review of this type can:

- enhance the performance of field staff by providing them with a better understanding of the types of field data that are important to a particular stratigraphic characterization effort; and
- confirm success in a stratigraphic characterization effort: if data from later phases of a field program do not exhibit increasing agreement with the conceptual stratigraphic model of the site, this may indicate the need for a different model.

Multi-phase field programs, which can be expensive, ultimately stand a better chance of being performed efficiently and completely if the data from each phase is carefully reviewed before embarking on the succeeding phase.

In recent years, the compilation, interpretation, and presentation of stratigraphic data, as well as decision-making based on the interpretation, have been greatly facilitated by the advent of software applications that can manage three dimensional earth science data sets, enabling interpolation (and extrapolation) of data based on a variety of geostatistical routines. These applications tend to be used on large, long-term projects, both because these systems are relatively more economical at a large scale, and because large scale projects (e.g., several hundred borings over decades) are difficult to manage in any other way. Despite the advantage these applications offer, they must be used with care, keeping in mind that the compilation and interpretation of data are only as good as the quality and comparability of the underlying data. This applies not only to the accuracy and consistency of data, e.g., a CPT trace or geologist's classification of soil, but also to the accuracy of the site survey which provides spatial control for such data. Professional judgment is important in the application of geostatistical procedures. The modeling approach needs to be compatible with the character and variability of site stratigraphy. Field data must be collected with a frequency and spatial distribution that enables robust interpolation. Finally, the results of interpolation must be checked against localized stratigraphic irregularities (e.g., faulting, buried stream channels, fill, etc.). Notwithstanding these caveats, these geostatistical applications offer enormous promise for cost-effectively managing large stratigraphic data sets and will aid in the development and ongoing checking and refinement of the stratigraphic model.

7.2.5 Needs

Non-intrusive site exploration methods need further development to improve the level of confidence in quantifiable data. In interpreting data, current quantification methods by statistics lack field-proven statistical parameters, e.g., correlation lengths.

7.3 DRIVING POTENTIALS

7.3.1 Background

While contaminant transport is *impeded* by the geologic strata, it is *driven* by spatial variations (gradients) of potential energy. For a given situation, the gradient of potential energy is the key factor affecting the direction and rate of transport. Fluid potential differences drive advective flow. The most common analysis, that for advective groundwater flow, is based on gradients of piezometric head (sum of pressure head and elevation). However, the piezometric head at a point can be modified by the presence of chemicals, water of varying density, and localized heat sources. Fluids also flow in the gas phase, driven by variations in gas pressure (pressure gradients). Gas flow examples include methane migration in/from degrading waste masses and steam or air stripping of soils. Thermal gradient situations involving waste containment facilities are not simply limited to high-level nuclear sites; other examples include chemical reactions (or even fires) in buried waste masses and in subsurface storage of hot hazardous fluids.

7.3.2 State of Practice of Investigative Methods

a. Point Measurement vs. Field Distribution. The geoscience professions have relatively adequate instrumentation for determining piezometric head, chemical concentration, and temperature at a given point. The challenge is in determining the spatial distribution of these potentials, i.e., the potential field.

b. Hydraulic (Liquid) Potential. For simplicity, the term "hydraulic" in this discussion will refer to all fluids, not water alone. The basic technique is simple: measure the piezometric head at various locations and depths. The normal direct methodologies involve some form of intrusive sensor placement. Among these are well and/or piezometer clusters and piezocone CPT measurements. Measurement methods are covered in ASTM D4750. Transducers in wells, etc., can determine the potentiometric level to ±0.001 feet (±0.3 mm) in water and to ±0.01 ft.(±3 mm) in nonaqueous liquids (NAPLs). The results from sonic units or tape probes exhibit less accuracy and consistency. Indirect methods, such as electrical resistivity, seismic reflection, and ground-penetrating radar, can be helpful; but their current accuracy level is poor. More work is needed if these methods are to become reliable in a quantitative sense. Water levels in nearby water bodies, such as streams or lakes, can be useful references when conducting a survey of piezometric head potential.

While the actual data obtained from direct measurement techniques may be precise, the interpretation of the results may be less accurate. The potential (piezometric head) distribution is greatly influenced by the distribution of hydraulic conductivity in the formation being investigated. This has often been noted in vertical gradient studies, where low-hydraulic conductivity zones may cause perching. In fractured materials, local changes in hydraulic conductivity can significantly alter the head distribution over short distances.

There are many cases where the vertical gradient of piezometric head reverses over a distance of just a few feet. The potential head distribution within a fractured mass can be quite complex and easily misinterpreted when based on a few scattered measurements. There can also be temporal variations due to precipitation or barometric events, and these must be considered in potentiometric mapping.

To aid in data interpretation, a recommended technique is to concentrate point measurements in the pervious zones, where head variations are typically small. These values serve to define the boundary conditions for the natural barrier formations fairly accurately, and the gross potential head difference across (but not necessarily within) the aquitard can then be evaluated.

Overall, the hydraulic head potential distribution can be defined more accurately than can the distribution of hydraulic conductivity; however, this may not be true in the case of sparsely fractured materials.

c. Gas Potential. Experience has been limited in measuring gas pressure potentials, implying that potential fields in gas have not been studied in detail. There have been some cases of "breathing" in the unsaturated zone, where cyclical air pressure changes cause transport of gaseous contaminants into or from the subsurface. These barometric pressure changes can be on the order of ±2% to ±3% of atmospheric pressure.

d. Chemical Potential. The chemical potential field is usually determined by tests for chemical concentrations at various points. The usual technique involves analytical chemistry testing on samples taken from wells or boreholes. These tests are quite precise in determining the chemical concentrations in the *sample*. However, they may be misleading in an overall sense unless the sample is representative of the field conditions. Field methodologies include scan chemistry (organic vapor analyzers), pH and/or electrical resistivity, CPT probes, etc. These are far less precise but, because they test larger volumes, are often more accurate than analytical chemistry. Downhole analytic chemistry is available today for volatile organics and petroleum products using CPT (push) technology. Also, downhole optic fiber sensors and portable analytical chemistry instrumentation are being developed for on-site determination of chemical concentrations (Magee, 1996).

Generally, delineation of chemical potential fields is based on the general pattern of point chemical concentrations, modified (if possible) to account for chemical reactions. Local variations from the general pattern will exist, but complete definition of these variations is normally not achievable without excessive effort (and cost).

As in the case of hydraulic head potentials, chemical potentials can be determined to greater levels of accuracy than can be achieved in determining the transport parameters. While improvements in chemical potential characterization are desired and possible, limited resources might be better expended on research and development that improves characterization of the transport parameters of the aquitard.

e. Thermal Potential. The thermal potential field is studied through measurement of the subsurface temperature distribution. The normal methodology involves intrusive placement of temperature sensors. Temperature sensors can be permanently emplaced with remote reading. Temperature profiles can be obtained using available CPT downhole sensors. Precision levels for temperatures measured utilizing direct methods are quite good (tenths to hundredths of °C).

The field distribution of temperature is subject to considerations similar to those of hydraulic head distributions (for saturated conditions). However, significant latent heat can be stored or released during phase changes due to vaporization or freezing. Overall, however, temperature distribution determinations have been adequate, as has been modeling of heat flux in the subsurface using heat conservation equations. Since advective transport tends to smooth thermal gradients in permeable units, bounding conditions for aquitard units can be established fairly accurately. Absent phase changes, the confidence level in the determination of thermal potential distributions is similar to that obtained for hydraulic head and chemical potential distributions.

7.3.3 Assessment of the Technology

The tools available to the geoscience professions have more than adequate levels of precision in measuring piezometric head, chemical concentration, and subsurface temperature *at a given point* in a geohydrologic system. It is a more difficult task to define the potential field based on a limited number of point measurements. In spite of this difficulty, the accuracy of potential field prediction far exceeds the level of accuracy in the determination of the transport parameters (e.g., hydraulic conductivity).

7.3.4 Needs

The primary needs in potential field determination involve the improvement of interpretive methods and the development of improved surface scanning techniques for spatial interpretation of the potentials based on point measurements.

7.4 LIQUID TRANSPORT PROPERTIES - SATURATED CONDITIONS

7.4.1 Background

The primary transport parameters under saturated conditions are: hydraulic conductivity, effective porosity, diffusion/dispersion coefficients, and partition coefficients that account for adsorption. These transport parameters determine the capabilities of a natural stratum to serve as an indigenous barrier. Determining representative values of these parameters within acceptable levels of accuracy is the key to adequate transport analyses.

7.4.2 State of Practice

Current methodologies for determining transport parameters are discussed below.

a. Hydraulic Conductivity. Laboratory Methods. Laboratory tests to determine hydraulic conductivity can be conducted in either rigid or flexible-walled permeameters. Daniel (1994) presented an extensive summary on the state-of-the-art of these tests and concluded that flexible wall tests are best for fine-grained deposits. The results obtained from these tests are critically dependent upon the selection and preparation of test specimens. Both constant or falling-head tests can provide satisfactory results. However, head change must be small if the test specimen is compressible in order to avoid densifying the specimen, which reduces the sample hydraulic conductivity. Liquids other than water can be used in the test if the specimen is adequately backflushed so that chemical equilibration is reached. Other possible problems with these tests include wall effects (excessive flow near the wall) and entrapped air (which reduces the effective porosity). The major problem is sample size; the sample is frequently too small to accurately represent the effects of macropores on the flow system.

Field Methods. Selection of a field hydraulic conductivity test method depends on whether the test can be conducted at the exposed barrier floor or if boreholes must be used. Trautwein and Boutwell (1994) present an overview of both types of methods. Comparisons of slug test data on indigenous fine-grained soils based on permeameter test data from samples of the same soils show that slug tests generally produce higher values of hydraulic conductivity. This has been attributed to the larger scale of the slug tests (Herzog and Morse, 1986; Bradbury and Muldoon, 1990). Tests on properly compacted clay liners have not shown this difference.

Borehole methods include slug tests, which are summarized by Herzog (1994), and the two-stage borehole (TSB) test, which is described by Trautwein and Boutwell (1994). Borehole tests in fine-grained deposits are best conducted in open boreholes created using a thin-walled sampler to eliminate complications in the analysis due to the high hydraulic conductivity of sand pack and from side wall smearing. Herzog reviewed four methods for analyzing slug test data by Cooper et. al. (1967), Bouwer and Rice (1976), Hvorslev (1951), and Nguyen and Pinder (1984). Although slug tests primarily measure horizontal hydraulic conductivity, tests conducted in angled holes have been used to estimate the vertical component of hydraulic conductivity (Herzog and Morse, 1986). All four analytical methods produced similar values of hydraulic conductivity for low hydraulic conductivity deposits, but the values calculated for an individual test tend to diverge at higher hydraulic conductivities. The Nguyen and Pinder method provides a composite hydraulic conductivity value, since it is based on the three-dimensional form of the flow equation. Unfortunately, the method is not recommended because of an error in the solution (Butler and Hyder, 1994). There is a need for an improved 3-D method to evaluate hydraulic conductivity from slug test data.

The TSB test, a variation of the slug test, uses the Hvorslev method of analysis to provide estimates of both the vertical and horizontal hydraulic conductivities. Borehole methods can provide information on the spatial variation of vertical and horizontal hydraulic conductivities, and such testing can be conducted on slopes or at depth. ASTM is preparing standard methods for conducting the slug and TSB tests.

b. Porosity. For saturated materials, total porosity can be determined by the conventional oven-drying method (ASTM D2216). A microwave method (ASTM D4643) can be used if results are needed more quickly. The microwave method requires the use of a calibration curve that correlates the values obtained from the microwave method with those obtained using the conventional oven-drying method for each soil to be tested (Krapac et al., 1991).

Effective porosity represents the volume of void space available for fluid flow and is more difficult to measure. Peyton et al., (1986) studied the effective porosity of fine-grained deposits and concluded that it depends on the size of the molecules being transported compared with the size of the passages connecting the pores. He also found that water could pass through these passages in a lacustrine sediment, so that effective porosity and total porosity were identical for water in that case. In contrast, Horton and Thompson (1986) found the effective porosity of loess and glacial till to be approximately ±10% to ±20% of total porosity. The difference between these two findings has not been resolved and requires further study.

c. Diffusive Flux. In low hydraulic conductivity environments, such as exist in waste containment barriers, diffusive flux can be more significant than advective flux (see Section 10). No standard laboratory or field method exists for the measurement of diffusive flux parameters.

Laboratory Methods. Both steady-state and transient methods are available for determining diffusion in the laboratory. Shackelford (1991) presents a comprehensive review of these methods and a summary of published values for effective diffusion coefficients in low hydraulic conductivity soil formations. Techniques include column, half-cell, double and single reservoir, and chemical digestion methods. No standards have been published for these methods. The degree of saturation is a major factor affecting the measured value of the effective diffusion coefficient in a porous medium. Reported values for nonreactive and reactive solutes in saturated soils are up to 10 to 20 times greater than the corresponding values in unsaturated soils. Thus, degree of soil saturation is critical in the determination of reliable effective diffusion coefficients. Diffusive transport rates of nonreactive solutes may appear to be much greater (up to 5000 times greater) than rates obtained for reactive solutes, unless reversible sorption reactions are taken into account for reactive solutes.

d. Adsorption. Laboratory Methods. Adsorption coefficients must be determined for specific soils and solutes. The batch-adsorption or static-equilibration technique is the most common laboratory method for testing the capacity of geologic materials to adsorb chemicals. The U.S. EPA has published a Technical Resource Document (Roy et al., 1991) on this method, which is also addressed in ASTM standard D4646. Procedures have been developed and tested for both inorganic and organic solutes. Although the techniques are relatively simple, the results are dependent on several experimental parameters, including: contact time, solution pH, hydrolysis, and the presence of other dissolved constituents in the aqueous solution, in addition to the solute of interest.

Adsorption of organic solutes is affected by the following: presence of dissolved organic carbon, adsorbate volatility, photodegradation, biodegradation, and compound stability. German researchers have found that, in general, retardation coefficients calculated from batch tests are distinctly higher than those obtained from diffusion and percolation experiments (Czurda and Wagner 1991). The reasons for these differences deserve further attention.

Field Methods. No field methods have been developed for measuring adsorption parameters.

7.4.3 Field Performance of the Technology

Hydraulic conductivity can be adequately measured at a point, but the accuracy of an overall stratum conductivity estimate is probably only correct to within one order of magnitude. In fractured media, the accuracy may be poorer. There are few field data that can be used to evaluate the accuracy of estimates for the other transport parameters.

7.4.4 Assessment of the Technology

As stated above, the actual level of accuracy is within about an order of magnitude in the determination of hydraulic conductivity and is essentially unknown for the other transport parameters.

7.4.5 Needs

The predominant need relative to the determination of transport properties for indigenous barriers is the need for a better understanding, definition, and measurement of the *in situ* values of these transport properties. Some progress has been made in accounting for specimen scale size in the measurement of hydraulic conductivity. Analogous advances are needed in the evaluation of diffusion and adsorption coefficients, including their relationship to effective stress and contaminant chemistry.

7.5 LIQUID/GAS TRANSPORT - UNSATURATED CONDITIONS

7.5.1 Background

Currently, all U.S. regulations and most of the commonly used analytic/finite element models for contaminant transport are based on saturated flow conditions. However, the saturated condition does not always exist, especially at sites in the western U. S., requiring that the more general (and more complex) situation of contaminant transport under unsaturated conditions be considered. Clay barriers are a good example. Although the *saturated* hydraulic conductivity is generally specified, the common field tests (Sealed, Double-Ring Infiltrometer and Two-Stage Borehole Test) involve flow into an unsaturated medium (Trautwein and Boutwell, 1994).

The concepts applied to barriers in the saturated case are no longer valid in the unsaturated regime. Hydraulic conductivity, for example, becomes a function of the degree of saturation and the driving potential for transport may be dominated by suction pressures rather than by hydrostatic pressures. Also, effective porosity varies with the degree of saturation and with the concentration and type of ions present in the pore water. Diffusive flux ceases to be characterized using the aqueous solution value of the diffusion coefficient, and may be better evaluated in terms of gas flow.

Fredlund and Rajardo (1992) suggest that at saturation levels of 85% or more, transport tends towards (but not exactly) saturated behavior, i.e., liquid flow dominates; while at saturation levels less than 15%, soil behavior tends toward dry behavior, i.e., gas flow dominates. Between these two saturation levels, a full partially saturated analysis is required.

7.5.2 Role of Diffuse Double Layer

In clayey soils, most of the soil moisture is contained within the diffuse double layer. The moisture content in soils at low levels of saturation exists as thin films on the particles, rather than as sparsely distributed three-dimensional pockets of moisture. Film thicknesses are on the same order as the diffuse double layer thicknesses, so that the soil transport properties are influenced by the electrical fields. High soil densities are often associated with highly over-consolidated natural clays. Under these conditions, particle spacings may be so small that the electric fields influence transport in the ‘virtually saturated state’ just as they do in the unsaturated state.

These electrical fields affect transport of dissolved ionic contaminants more than they do the bulk water flow, acting to trap “conservative tracers”, such as chloride and bromide. Thus, pure advective transport does not occur in soil pore systems dominated by the presence of diffuse double layers. Adsorption of ionics must also be considered. This sorption depends on the ions’ charge density (valence per unit volume of the ion). High charge densities produce high “sorption coefficients” and thus, high retardation of ionic contaminant migration. Anion adsorption is usually low, on the order of 1% to 5% that of cations.

7.5.3 State of Practice

a. Effect on Conventional Transport Parameters. The level of saturation influences the magnitude of the conventional transport parameters; i.e., hydraulic conductivity, effective porosity, diffusion coefficient, and retardation coefficient.

<u>Hydraulic Conductivity</u>. While unsaturated flow of liquids may still be analyzed utilizing Darcy’s Law, the hydraulic conductivity in unsaturated flow varies with the degree of saturation. The lower the degree of saturation, the lower will be the hydraulic conductivity. On the other hand, suction pressure increases with decreasing levels of saturation. Therefore, the material with the lowest hydraulic conductivity will also exhibit the highest suction. The changes in transport resulting from changes in degree of saturation can be very large, varying by orders of magnitude. Further, the relationships between hydraulic conductivity and degree of saturation are nonlinear and exhibit hysteresis (not single-valued). The relationship differs when a soil is dried from the wet state as opposed to when it is wetted from the dry state (hysteresis effect). Determining this relationship, even in the laboratory, poses a significant undertaking for each soil type to be studied.

Hydraulic conductivity is thus a strongly nonlinear function of both the degree of saturation and the piezometric head potential. Sands drain at lower potentials than do clays. As a consequence, while sands have higher hydraulic conductivities at full saturation, they generally have lower conductivities at low levels of saturation. The relationship between hydraulic conductivity and the degree of saturation is routinely determined by measuring the hydraulic conductivity under saturated conditions and the moisture characteristics curve, thus enabling the calculation of the hydraulic conductivity as a function of level of saturation.

Effective Porosity. Effective porosity indicates the percentage of the total subsurface volume that is actually available for fluid flow. Practical experience indicates that the "effective" porosity is generally less than the total porosity. The actual portion of the pore system void space available for transport affects solute breakthrough times in laboratory column experiments, as well as solute travel times in the field. An exception may be the transport of tritium, where the effective porosity is frequently the same as total porosity. For anions that are excluded from the diffuse double layer, effective porosity appears related to free water content. Total porosity is normally determined by ovendrying a saturated sample. However, the drying process involves the addition of heat energy to the water. Each bound layer requires a different heat energy for vaporization. Thus, pore water evaporates sequentially, corresponding to these energy levels. Free water is lost at moderately low temperatures, followed by the diffuse double layer, and then by interlayer water as the temperature increases to the ASTM D2116 standard of 105^{o}C. Thus, the measured porosity becomes a function of drying temperature. Effective porosity is essentially a measure of the free water (unbound) and probably could be determined by drying at moderate temperatures. Currently, there is no standard method for this procedure.

Diffusion Coefficient. Diffusion of solutes in unsaturated materials can not be described assuming that one diffusion coefficient is applicable, since there is simultaneous diffusion in the bulk liquid, the liquid film, and the gas phases. Diffusion of ionics is also a function of charge density. Because of this, the apparent diffusion coefficient is observed to have a dependence on the degree of saturation, similar to hydraulic conductivity. Most organic contaminants and CO_2, both of which partition into both the liquid and gas phases, tend to show an apparent diffusion coefficient intermediate between that obtained under full saturation and that obtained in the gas phase alone. Therefore, under partially saturated conditions, the apparent diffusion coefficient for a specific contaminant depends on the degree of saturation, as well as Henry's law constant, which determines how the contaminant partitions between the liquid and gas phases. As a "rule of thumb", the apparent diffusion constant may be taken as being somewhere between the gas value and the liquid value. Although this relationship depends on the degree of saturation, an average of these two limiting values is frequently used.

Retardation Coefficient. Ionics can be either retarded (anions) or accelerated (cations) by the electrical fields of the double diffuse layers. Thus, the retardation factor can be greater than one (anions) or less than one (cations). If the double layers do not overlap, ions tend to be accelerated. Overlapping double layers can result in retardation of the ions dissolved in the solvent (water), i.e., producing retardation factors greater than one. Anions and cations can not be easily separated due to the charge imbalances that would be created. Slight charge separation does cause streaming potentials, but cation and anions tend to move together. In many experiments, the observed retardation coefficients for anions are less than one.

Non-Polar Molecules. The transport behavior of organic compounds in unsaturated materials has not been studied extensively, except in vapor extraction systems. Therefore, little is known on this topic. Most of the transport of volatile organics occurs in the vapor (gaseous) phase of an unsaturated soil. Studies have been conducted on vapor extraction systems and air flow through porous media. In many extraction systems, approximately 50% of the total costs can be attributed to the treatment of the extracted contaminants. Subsurface aeration can enhance subsurface biodegradation of the contaminant, potentially lowering surface treatment costs.

b. Implications for Design. The purpose of a barrier is to prevent or inhibit migration of contaminants, *not* simply to comply with some stated regulatory value for the saturated hydraulic conductivity. In the relatively dry condition (low degree of saturation), the hydraulic conductivity of fine materials is *greater* than that of coarse materials; the exact opposite to the saturated condition. Thus, under relatively dry conditions, fine materials may serve as drainage layers and coarse materials become barriers. This phenomenon has been used as a basis for constructing barrier systems in arid climates. A fine sand becomes the leachate collection system and coarse sands, gravels, or even concrete blocks become the barrier to migration.

c. Monitoring. Monitoring procedures in the (unsaturated) vadose zone depend upon the specific contaminant being monitored. If aqueous phase contaminants are of concern, suction lysimeters are often used to sample soil water. If gas phase contaminants are an issue, soil gas is commonly monitored with multiport sampling systems. Barometric pressure fluctuations may also be important in contaminant movement and can be monitored with pressure transducers.

Moisture content is typically monitored by neutron probe measurements. However, time domain reflectometry and capacitance measurements are becoming more popular. Use of these latter devices requires training and experience in order to obtain accurate moisture content data over time.

Tensiometers are often used to monitor hydraulic head potentials near saturation. At lower head potentials (1/3-5 bar), head dissipation probes are generally the tool of choice.

The make-up of lysimeter samples, obtained by suction, can depend on the suction level used in the lysimeter. At lower suctions, a larger fraction of free water and higher ionic concentrations are obtained. At higher suctions, water adjacent to the double layer and lower ionic concentrations can be sampled. Thus, the analytical chemistry results can depend on the suction level and may not reflect contaminant concentrations in the free, advecting water. Also, the (necessarily) small pores of the lysimeter will preferentially transmit small ions and exclude or retard large ions. A truly representative sample of ionic contaminants can be difficult to obtain.

7.5.4 Field Performance

Vadose zone monitoring can provide early detection of contaminant movement, thus offering an early opportunity for intervention and the possibility of preventing groundwater contamination. Since natural vadose zones frequently exhibit preferential flow paths and heterogeneities, much of the data obtained from point measurements (e.g., tensiometers) or scans (neutron probe logs) may not capture important three-dimensional transport. Nielsen et al., (1990) have concluded "The efficacy of accurately predicting the attenuation and eventual location of solutes or constituents in the vadose zone remains undeveloped". For these reasons, site monitoring will remain a requirement for the foreseeable future.

7.5.5 Assessment of the Technology

Vadose zone contamination problems must be approached with a sound understanding of unsaturated soil phenomena. Better use could be made of our current understanding of transport processes and of the available exploration technologies. While there are limitations in the ability to characterize, monitor, and predict transport in the vadose zone, additional vadose zone work continues to be undertaken because of the possibility of preventing groundwater contamination and possibly reducing the ultimate costs associated with remediation. For volatile contaminants, field monitoring in unsaturated formations will be of increasing importance. In engineered systems, soil physical properties can be manipulated to enhance the effectiveness of point or scan measurements of hydraulic properties. Engineered vadose zones offer some practical advantages for both containment and monitoring. While this knowledge exists, it is often not considered in a facility design.

7.5.6 Needs

Tools are being developed to improve the ability to 'see' vadose zone processes. These tools range from geophysical techniques, such as electrical resistance tomography, to improved capacitance and time domain reflectometry probes. The understanding of contaminant transport processes in unsaturated soil materials needs improvement, as does the understanding of and the ability to predict transport of moisture in the vadose zone. The effects of temperature gradients, diffuse double layers, and sorption processes are not well understood. As one example, many vadose zone data sets show large variations in ionic concentrations; but few models offer an adequate explanation as to why this occurs. More effort needs to be made in educating the environmental community regarding vadose zone phenomena, since incorrect assumptions and incorrect installations of monitoring equipment can lead to costly consequences. Such educational efforts would eliminate many common mistakes made in vadose zone hydrology.

7.6 EFFECT OF FRACTURES ON TRANSPORT

7.6.1 Background

Fractures (including joints, faults, bedding-plane separation, etc.) can occur in any cohesive or lithified geologic material. Fractures can often be seen in outcrops or core samples, but it is generally not possible to visually determine whether they are significant pathways for contaminant transport. The presence of alteration features such as oxidation staining, in-filling, and root casts, are indications that the fractures were "open" at some time in the past. However, open fractures can be "closed" as a result of changes in the surrounding stress-field or by in-filling of the fracture with low hydraulic conductivity material. On the other hand, inactive fractures may become active if the in-fill material is dissolved and washed away. Such dissolution can result from both natural or man-induced changes in geochemistry of water. Finally, pre-existing fractures with no visible alteration can act as significant pathways for flow and such fractures are very difficult to distinguish from man-induced fractures caused by drilling or excavation.

7.6.2 State of Practice

a. Cubic Flow Law. Flow through a fracture network is controlled by a number of factors, including: aperture (or separation distance between the walls of the fracture), fracture density (i.e., spacing between fractures), length of fractures, and fracture orientation. Fracture spacing, length, and orientation control the degree of interconnection of the fracture network and can often be characterized by visual mapping. Fracture aperture is more difficult to measure because of the small values (typically on the order of 10 to 100 micrometers) and because stress-relief in excavations or boreholes affects the aperture value. Aperture values in low-hydraulic conductivity materials are generally calculated indirectly from measurements of bulk hydraulic conductivity and fracture spacing, length, and orientation using the "cubic-law" (Snow, 1969). The cubic-law is based on the relationship between hydraulic gradient and discharge for a *viscous fluid* flowing between two parallel *smooth* surfaces; hence, aperture values derived from this relationship are some sort of average for the entire fracture network. In reality, the aperture can vary greatly within a single fracture, which leads to larger aperture "channels" controlling much of the flow. There have been a number of theoretical investigations of the possible influence of aperture channeling on contaminant migration, but it is very difficult to get reliable field data. Thus, the most common method for estimating fracture aperture values is still based on the cubic-law.

b. Matrix Diffusion. Migration of a solute in a fractured porous medium occurs by advection through the fractures, advection in the matrix, and especially diffusion into the matrix. In most low hydraulic conductivity materials, advection in the matrix is negligible (especially if the system is fully-saturated) and diffusion controls the transfer of solutes from the fast moving water through the fractures into the relatively immobile pore water of the matrix. This process of "matrix diffusion" can theoretically retard the overall migration of a solute by many orders of magnitude relative to contaminant transport in the fracture alone. The effectiveness of this retardation process is largely related to the porosity of the matrix, with the effect being much greater in high porosity clays and shales than in low porosity rock. In recent field and lab experiments in fractured glacial clays (McKay et. al., 1993 and Hinsby et. al., in-press) non-reactive solutes (bromide and chloride) were retarded by a factor of up to 100 relative to the transport of colloidal virus tracers, which were too large to enter the matrix pores, and hence were not retarded by matrix diffusion. Modeling simulations by Sudicky and McLaren (1992) showed that in a scenario which included matrix diffusion, a nonreactive solute (e.g. chloride) migrating downwards through a 25 μm aperture fracture in a glacial clay would take about 50 years to reach the underlying aquifer at 10m depth. For the same scenario, but with no matrix diffusion, the solute would reach the aquifer in about a half a day. Field and laboratory investigations by Birgersson and Neretnieks (1990) have shown that solute diffusion in low porosity granitic rock, although much slower than in the above fractured clay examples, can be significant. As a result, it is possible that in some scenarios (e.g., radioactive waste migration in deep fractured granites), matrix diffusion may be a significant retarding mechanism even in very low porosity fractured rocks.

The downward migration of DNAPL's in fractured media (Kueper and McWhorter, 1991) usually occurs fairly rapidly after a spill and is unaffected by diffusion, since the penetration is controlled primarily by fracture aperture and the properties of the DNAPL (density, interfacial tension, and viscosity). However, once the DNAPL has become immobilized, diffusion of DNAPL contaminants into the matrix pores will make it much more difficult to remediate.

c. Assessment Methods. Methods of assessing the significance of fractures in soil or rock fall into the following categories:

1) geologic methods
   - mapping of fractures in outcrop or core samples
   - downhole camera and geophysical surveys
   - local or regional faulting, folding, and/or other indicators of stress history
2) hydraulic methods
   - direct measurements of field or lab hydraulic conductivity
   - monitoring of variations in hydraulic head profile with depth due to seasonal or episodic changes in head (or recharge)
   - borehole flow meter surveys
3) geochemical and isotopic methods
   - isotopic indicators of groundwater age (^{3}H, ^{2}H, ^{18}O, ^{14}C, ^{85}Kr)
   - chemical indicators of recent recharge (CFC's, dissolved solvents, or pesticides; i.e., man-made chemicals)

Each of the above methods has varying degrees of reliability, and sometimes different methods of investigation can produce contradictory results. In general, the geochemical/isotopic methods tend to be more reliable, but use of multiple methods of investigation is recommended.

7.6.3 Field Performance

Geophysical methods, such as ground penetrating radar, are capable of locating fractures and fault zones. However, methods for determining transport properties through a fractured mass in the field need to be further developed to become an engineering tool.

7.6.4 Assessment

The capability exists for integrating the various stratigraphic, hydrological, and chemical data in the prediction of transport of a specific contaminant. However, this is a complex task and field verification is difficult. Thus, the current ability to assess the capacity of indigenous layers (especially if fractured) to serve as natural barriers is still unsatisfactory. Basic questions remain concerning fractures that require further intense investigation on such topics as: the role of confining stress in fracture opening and closure due to excavation and construction; the extent of irreversibility of such opening and closing and the related question of fracture "healing"; fracture opening/closure in response to pore pressure cycles; the role of in-filling material, which may be dissolved or precipitated by concentrated chemical contaminants; the role of matrix diffusion in retardation of different types of dissolved contaminants; the factors influencing penetration of DNAPL's in fractures;

and the factors influencing attenuation of colloidal contaminants. Current research models that attempt to simulate these processes are generally too complex and user-unfriendly for application in the field.

7.6.5 Needs

From an applied perspective, significant improvements are needed in the following areas: development of methodologies to detect hydraulically-conductive fractures, particularly in settings where the fractures are widely-spaced (meters to tens of meters); development of user-friendly computer models that incorporate fractures and matrix diffusion; and finally, presentation of rigorously investigated case histories that illustrate the integration of stratigraphic, hydraulic, geochemical/isotopic and geophysical information combined with the use of computer models in the assessment of contaminant migration in different types of fractured, low hydraulic conductivity deposits. Overall, the biggest need is for field verification of the processes controlling contaminant transport and continued development of predictive models.

7.7 EFFECTS OF CHEMICALS

7.7.1 Background

The natural geochemistry of clay minerals, excluding the potential impacts of synthetic chemicals, is itself a complicated subject. Clay mineralogy textbooks, such as Grim (1968), address this natural geochemistry; but, understanding subsurface geochemistry at both a micro and macroscopic level remains a difficult task. Since the geochemical equilibrium in some clay minerals is often a delicate balance, most aggressive chemical conditions are likely to alter this equilibrium which, in turn, may alter both the physical and chemical performance of any clay mineral. Aggressive chemicals may also alter the physical behavior of both cohesive and cohesionless soils, which both represent potential transport pathways in geologic media.

Much of the available information concerning the effects of synthetic chemicals on geologic media is based on laboratory research (Acar, et. al., 1985, Anderson, et al., 1985, Hermann, 1985). Much less has been reported on field observations of the effects of synthetic chemicals on geologic materials (Hermann, 1978; 1985). Chemical effects include a variety of chemical reactions and physico-chemical processes which may alter the clay mineralogy, as well as the physical properties of the material. For example, in the case of a secondary fracture system that dominates the flow characteristics of a natural geologic material, the shrink/swell characteristics of clay minerals in or adjacent to the fracture will affect the hydraulic conductivity of the overall stratum.

The ability to measure chemical effects on geologic material is enhanced in the laboratory where conditions can be more tightly controlled than in the field. However, indigenous barrier applications are concerned with *in situ* conditions and the *in situ* effects of synthetic chemicals on the hydraulic conductivity, clay mineralogy, stratification, and natural geochemical equilibrium. In this regard, low temperature aqueous geochemistry is an emerging field that has become particularly important in the prediction of contaminant transport of highly concentrated chemicals contained by indigenous earth barriers.

7.7.2 State of Practice

Chemical effects on indigenous barriers are, in principle, similar to their effects on man-made barriers. The results of laboratory compatibility experiments have been reviewed in depth by Alther, et. al. (1985), Mitchell and Madsen (1987), and Shackelford (1994). The principal mechanisms of contaminant influences on soils are summarized as follows: *brackish-brine salts*: precipitation, dehydration, cation exchange, density gradients (Alther, et. al., 1985); *acids*: dissolution, geochemistry changes; *alkalis*: precipitation, geochemistry change (Peterson and Gee, 1985); *organics (non-polar)*: dehydration, water miscibility of the contaminant. The most important clay transport property is hydraulic conductivity. Threshold contaminant concentrations required to produce an order of magnitude or greater impact on hydraulic conductivity are: brine concentrations greater than 100,000 ppm (NaCl and other salts), pH less than or equal to 1 or 2, and non-polar organic concentrations greater than 50-80%.

Hydraulic conductivity changes due to permeation by non-polar organics depend on the kind of clay and the kind of organic chemical (mainly with respect to their dielectric constant and miscibility). Typically, the changes are related to the following mechanisms: dehydration, swelling, flocculation, and macroscopic cracking (see e.g. Anderson et al., 1985, Daniel et al., 1986). A 70% concentration of an organic liquid in the pore water can increase hydraulic conductivity by two to four orders of magnitude, if no confining stress is applied. If there is a compressive stress applied, there can be a two order of magnitude reduction in hydraulic conductivity (Bowders and Daniel, 1987, Quigley et.al., 1988, and Fernandez and Quigley, 1991). High effective stress may play a beneficial role in healing possible damage to a clay liner caused by a catastrophic inflow of a highly concentrated low dielectric constant contaminant. The level of stress necessary to prevent or remediate a hydraulic conductivity increase is an important technical parameter in landfill design.

7.7.3 Field Performance

Most of the data concerning synthetic chemical effects on hydraulic conductivity and other properties of clays have been obtained in laboratory studies. The few reported field observations have tended to support the laboratory findings in a qualitative sense, but quantitative data is lacking.

7.7.4 Assessment

The effects of chemicals on hydraulic conductivity are progressive and develop over a long time. Predictions based on short term laboratory and field experiments may be misleading. In the field, observations based on the transit time of a tracer may require very long monitoring periods, on the order of decades. Subtle changes in the clay geochemistry equilibrium may affect long-term performance.

7.7.5 Needs

Changes in the natural geochemical equilibrium can alter the hydraulic conductivity of an indigenous barrier, sometimes drastically. Clay-filled secondary structures (fractures) can cause defects in the barrier when attacked by contaminants. The available data on the effects of chemical attack are almost exclusively from laboratory studies. Field data are scarce and poorly documented. Efforts should be made to publish existing field data, despite their sensitive nature. The response of field systems to the presence of chemicals should be studied from both the experimental and modeling points of view. Special study is recommended on determining which chemically-induced changes in hydraulic conductivity are reversible and which are irreversible.

7.8 THERMAL AND RADIATION EFFECTS ON TRANSPORT

7.8.1 Background

This section covers two related topics: thermal and radiation effects. Thermal effects are important in conventional practice, as they can adversely affect the barrier properties of a stratum. Both thermal and radioactive effects are critical in design of high-level radioactive waste facilities. Each is discussed separately below.

7.8.2 Elevated Temperatures

a. State of Practice. Most of the recent experience in Europe with thermal effects on soils at elevated temperatures (50-150° C) has been gained in the course of developing technologies for disposal of high level radioactive waste (HLW) in subsurface clay formations (in Belgium, Italy, Spain, UK, France and Switzerland). In the commonly accepted concept of a containment system (consisting of waste materials, the engineered barriers of backfill and buffer, and a host clay formation) the man-made barriers are assumed to fail, releasing the contaminant within 300 yr. Thus, the ultimate barrier is the indigenous clay formation component of the containment system. No actual repository is scheduled for construction before 2020.

The principal design consideration in these technologies is the rising temperature within the soil mass. Temperature increases the pore pressure during heating. Thermally induced pore pressure gradients may substantially increase the natural water flow, enhancing potential contaminant transport if an early breakdown of the inner engineered barrier occurs. Temperature and pore pressure gradients determine the spacing of tunnels and/or boreholes, a principal design parameter. Contaminant transport in clays is assumed to be mainly by advection with water. Therefore, the hydraulic conductivity of the clay mass is another critical factor for the design. Current design methodologies are based on predicting temperature and flow using a linear heat diffusion law, advection using Darcy's law with constant hydraulic conductivity, dispersion, and mass decay.

b. Field Performance. There are few field data reported on thermal effects, other than the effects on foundation behavior. A 2.5-year *in situ* heating test has shown a pore pressure buildup to 0.75 MPa around the heat source (Picard et al., 1994). Other *in situ* experiments and studies of natural analogs have revealed zones of microfractured clay around heat sources (Leone et al.,1986).

c. Assessment of Current Knowledge. Thermally-induced pore pressure excess measured in the laboratory under undrained isotropic conditions ranged between 1.5 to 5 MPa per 100 ˚C (Del Olmo et al., 1994). At deviatoric stress, the thermally-induced growth of pore water pressure may lead to failure and/or hydraulic fracturing (Hueckel and Pellegrini, 1992). Formation of such fractures can produce preferential transport paths for radionuclides during the radionuclide release phase, long after the thermal phase is finished.

The pore pressure excess results from the difference between the thermal expansion of pore water and the thermal volumetric strain of the skeleton. While the pore water expansion is poorly known, the thermal strain during heating depends critically on isotropic effective stress: it is expansive and reversible for very low effective stresses, but contractive and partly irreversible for high effective stress. Water content and hydraulic conductivity are additional factors affecting this process.

The hydraulic conductivity caused by a temperature of 150˚ C after one half year of heating in the presence of potassium in artificial clays was observed to increase by a factor of 100. Mineralogically, thermally driven dehydration produces illitization of smectites. This generates larger voids between the collapsed stacks, depending on the effective stress. Studies on natural analogs reveal that similar processes may occur in indigenous formations (Pusch and Guven, 1990). The presence of carbonates affects thermo-mechanical behavior of clays by increasing their apparent overconsolidation, rigidity and brittleness, and pore pressure buildup. The variability in carbonate content in natural clays makes site characterization a particularly complex task (Hueckel, 1995).

The assumptions of linear heat diffusion and Darcian hydraulic flow are limitations of the current evaluation methods. Non-(or de-) saturated and fractured or faulted clays may yield substantial departures from the linear predictions. The effect of heat on diffusive transport is not well understood.

Numerical predictions using two-phase, hydro-thermo-plasticity models have revealed that while the thermal phenomena are limited to a narrow zone around the heat source, the increased water pressures affect a larger area. During unloading in the course of cooling, clay may reach failure stress in tension. However, thermal effects generally terminate before the time the engineered barrier is expected to fail resulting in radionuclides being released to the host clay. Therefore, the most relevant issues of the whole thermal phase are the irreversible changes (dehydration, fracturation, strain), because of the impact these irreversible changes have on the long term performance of the repository (Baldi et al., 1987).

d. Other Contexts. In other applications outside of the context of high level radioactive waste disposal technology, one may envisage having to deal with chemically active liquids at elevated temperatures. No systematic expertise exists in this area, and condition specific response of soil is expected. In general, one would expect that elevated temperatures might enhance a possible destructive chemical effect on hydraulic conductivity, mechanical strength, and deformation of indigenous clay barriers. However, it is anticipated that long term thermal exposure, comparable to that with radioactive waste, is unlikely to occur.

7.8.3 Radiation Effects

During the waste containment phase, the radiation effects (mainly due to gamma radiation) will be limited to a radius less than 1m. Limited experiments with Belgian clay show the formation of radiolitic gases: H_2, O_2, H_2O_2, CO_2, and CH_4. Migration of these gases will depend on the reaction rates, generated gas pressure, and the formation hydraulic conductivity. Generally, interest is focused on gas migration phenomena and possible desaturation effects. During the release phase, clay moisture may enhance the radiolysis, increasing the production of gas (Henrion et al., 1988).

7.8.4 Low Temperature Effects

a. State of Practice. The focus in this area has been on freeze/thaw effects on the hydraulic conductivity of compacted natural clays used as landfill liners and covers, liners for ponds, etc. However, the concepts also apply to natural barriers. The technology involved is limited and consists of demonstrating a sufficiently low hydraulic conductivity, even after a number of freeze and thaw cycles. The understanding is that pore water during freezing expands over 9% forming ice lenses which generate cracks and cause suction, shrinkage, and cracking in the unfrozen zone. After a few cycles, the network of cracks form a highly conductive system of channels yielding up to three orders of magnitude increase in hydraulic conductivity. Effective stresses applied to the soil during freeze/thaw cycles (and to a lesser degree after the process), visibly attenuate the increase of hydraulic conductivity. High rates of freezing worsen the situation (Othman et al., 1994). Earlier studies of freezing zone propagation and frost heave support this understanding.

b. Field Performance. Freeze-thaw effects on hydraulic conductivity have been found to be significant in large-scale testing (Benson and Othman, 1993).

c. Assessment of Current Knowledge. Current knowledge concerning the freeze/thaw effects is mainly phenomenological. Little effort has been made to quantify the thermo-mechanical phenomena involved, to predict the outcome, and possibly control it. Some earlier developments exist on frost penetration during monotonic cooling, but outside of the context of concern about hydraulic conductivity changes. Any possible effects of concentrated chemicals on freeze/thaw changes on hydraulic conductivity are unknown. How diffusive transport is affected by freeze/thaw cycling is also an open question.

d. Needs. A better understanding of the basic mechanisms of thermally-induced pore water pressures and ice pressure development needs to be achieved. This includes reversible and irreversible macro- and micro-structural thermally- induced changes in clay, such as fracturing and increase in hydraulic conductivities. The role of effective stress in controlling the changes in hydraulic conductivity should be better understood. This effort should be addressed through academic research involving highly specialized mineralogical labs. Thermal effects in pre-fractured and in thermally fractured media should be studied in the lab, and because of scale effects, also studied with *in situ* field tests or on large models. Numerical models should be developed for quantitatively predicting and aiding in the control of thermally-induced changes in indigenous barriers.

7.8.5 Modeling: Coupled Thermo-Hydro-Mechanical and Chemo-Hydro-Mechanical Effects

a. State of Practice. The predominant practice in modeling combines linear heat diffusion and thermal expansion with the advective flow equations for non-deformable media for radionuclides. For modeling transport of chemical contaminants, the advective flow equations for non-deformable media are applied, accounting for diffusive transport and adsorption. An important step forward would be consideration of hydraulic conductivity in the non-saturated zone for contaminant transport. However, the role of effective stress on the hydraulic conductivity is rarely taken into account. Similarly, physico-chemical phenomena and chemical reactions are frequently not included.

b. Assessment of Current Knowledge. Contemporary knowledge of the mechanics of two-phase transport in porous media combined with numerical methods offer the possibility to improve the prediction of contaminant transport beyond that provided by simple linear models. The most serious obstacle in the development of more sophisticated models is still the limited understanding of the mechanisms and processes involved and the lack of quantitative data needed to evaluate material properties. These shortcomings constitute a challenge for both experimentalists and theoreticians.

There are number of analogies to be explored between thermal and chemical effects on indigenous barriers. They are not simply limited to the nature of heat diffusion and contaminant transport, but also include: the physico-chemical changes affecting the adsorbed water during a thermal process and the permeation by organics; the nature of thermally and chemically induced strain, thermal expansion, chemically induced swelling, thermal consolidation, and chemical consolidation under high effective stress. Also, there may be an analogy regarding the nature of the effective stress dependence and changes in hydraulic conductivity. The thermal expansion of pore water has an analogy in the permeant density change during contamination.

However, there are also differences between mechanisms and processes related to chemical and to thermal effects: temperature is a single agent, while chemicals in a leachate present an enormous diversity. Chemical reactions, e.g., dissolution, precipitation, etc., have no analogy in the thermal process. Despite these differences, the basic structure of the model for thermo-mechanical and chemo-mechanical clay behavior may be based on the same principles.

<u>Modeling of coupled thermo-hydro-stress effects</u>. Following the above strategy, a well-established soil mechanics Cam-clay model has been extended to include thermal conditions, experimentally corroborated, and coupled to the flow equations for pore liquids in a manner similar to consolidation theory. Most importantly, the plastic yield limit has been assumed to decrease with increasing temperature. The latter assumption allows for such phenomena as thermal decrease of peak strength, thermal generation of overconsolidation, and for thermal consolidation strain at high stress (Baldi et al., 1987).

<u>Simulations of high level radwaste disposal in boreholes in clays</u>. Using the above described model, clays from Belgian, Italian, and Spanish experimental candidate sites have been characterized. Numerical simulations indicate that while the higher temperatures affect only the close vicinity of a borehole, the elevated pore pressures have a much larger influence and may affect

neighboring boreholes. The effective stress evolution close to the heat source indicates a possibility of soil failure during cooling. In the failed zone, a preferential flow path could arise along the containers. In another simulation, a significant thermally-induced increase in the hydraulic conductivity was found within a radius of 5 m around the heat source (Ma and Hueckel, 1992).

Modeling of coupled stress-chemical-hydraulic effects on contaminant flow. To model the above coupling, one may employ the same strategy as in the case of thermal effects. Chemical reactions and effects resulting from sequential miscibility can be taken into account by introducing additional constitutive equations. To effectively model the response of an indigenous barrier using such models, specialized experiments are required to provide material parameters. Calibration of such a model is underway utilizing a data set generated in the laboratory using a Canadian liner clay subjected to flow of ethanol (Hueckel, 1995). This work is still in progress.

Simulation of coupled chemical-transport effects in clay layers. Limited simulations of purely advective flow with concentration-sensitive hydraulic conductivity have shown that the total discharge is largely unaffected by the changing hydraulic conductivity for contaminant permeation distances less than half the thickness of the layer. However, for permeation beyond the half thickness, contaminant transport is accelerated and the breakthrough occurs significantly earlier than when chemical permeation is absent (30 yr. in the discussed case). There is further treatment of modeling presented in Section 10.

c. Needs. There is a need for basic experimentation combined with simulation modeling in the study of the role of effective stress and coupled chemical and mechanical effects on contaminant transport in indigenous barriers. Also, the role of effective stress on the hydraulic conductivity of fractured media needs to be studied, both in laboratory and in the field.

7.9 EFFECTS OF MAN-MADE DISCONTINUITIES

7.9.1 Background

Man's activities can affect the properties of natural indigenous geologic formations. One man-made activity involves the perforation of aquitards with holes (such as borings or wells) or objects (such as foundation piling). The integrity of indigenous barriers (relative to vertical contaminant transport) is threatened by: pre-existing wells (known or unknown); wells and borings drilled for site characterization, monitoring, and/or remediation; deep structures, such as foundation piles; and stress relief effects from nearby deep excavations. Investigations have been conducted on methods to locate abandoned wells and borings and on ways to minimize the potential for promoting contaminant migration as a result of drilling or well construction activities. The flow through an open man-made hole that penetrates an aquitard can dwarf the advective flow through the aquitard itself. For example, the discharge through a 10 cm hole in a 6-meter thick clay stratum is on the same order as the advective flow through 100,000 square meters of the intact clay (under similar hydraulic gradients). However, wells and boreholes are not the only potential pathways for contaminant migration. There are numerous instances of utility trenches and pipe trenches acting as conduits for lateral movement of fluids. Even foundation units have been suspected as providing pathways for contaminants.

7.9.2 State of Practice: Wells and Boreholes

a. Problem Sources. The major problem is from wells abandoned without proper sealing. The problem is not new; it has been known for almost 90 years. For example, Bowman (1906) describes how upward and downward cross-contamination of aquifers can result from dissolved chemicals entering improperly abandoned borings and deteriorated well casings. He recommended the following: (1) plugs and packers be used to prevent cross-contamination along wells or borings; (2) accurate geologic logs be made for every well drilled; (3) the yield and quality of groundwater from each zone be examined; (4) the hydraulic heads in each water-bearing zone be determined; (5) a competent casing should be installed and maintained over time, using suitable experiments as needed to check well integrity; (6) consideration be given to potential casing deterioration when changes in water quality or hydraulic head are observed to occur; and (7) laws be established to regulate well construction and to minimize the potential for cross-contamination. Bowman's recommendations remain valid today.

During the past 20 years, the consequences of contaminant migration through known/unknown pre-existing wells and through wells installed for site characterization, monitoring, or remediation have been realized at many sites (U.S. EPA, 1977; U.S. EPA, 1992). Modeling studies have demonstrated that aquifers can be contaminated by preferential vertical migration at wells or along other similar pathways (Harrison et al., 1992). The significance of such contaminant migration depends on a variety of factors, including: dispersion/dilution of the contaminant and the mobility, persistence, and the toxicity of the migrating contaminant. The potential magnitude of contamination associated with preferential migration along vertical conduits can be evaluated by monitoring and through the use of simulation models. Overall, the most important well defects include:

- well left open,
- well annulus or bore improperly grouted, and/or
- chemical attack on the well casing or the grout material.

The first two defects commonly occur in heavily industrialized areas where shallow groundwater contamination is more prevalent. The third defect is more rare, but has been observed in PVC wells that extend through hazardous waste cells. In one such case, the well was virtually destroyed in a matter of weeks. Borehole television was used to verify the condition of the well.

b. Locating Abandoned and Missing Wells and Borings. Regulations dealing with the underground injection of liquid waste require that all abandoned wells be located within an "area of review" to minimize the risk of contaminant short-circuiting. Many remediation sites are in old industrial areas where the well records, if any, cannot be found. The wellheads are frequently covered, and relocating the well can be difficult if not impossible. Airphotos, geophysical surveys, and test pits are the primary location techniques available. Well location methods described by Aller (1984) and are pertinent to shallow contamination investigations. At many sites, however, the locations of all pre-existing wells and borings cannot be determined, even after using these methods. This is often the case at sites where tens to hundreds (or more) of borings and wells have been drilled since the 1970s. Well inventory and integrity management is a significant concern at most sites.

c. Recent Evaluations of Methods and Grouts for Sealing Wells and Boreholes. Whether plugging and abandoning an old well or installing a new well, the primary objective is long-term integrity. This is achieved through proper construction of the well, installation of a full, low-permeability seal, and regular maintenance of the seal in the presence of a potentially damaging environment. Recognition of the potential for cross-contamination has spurred research into methods and materials to adequately seal wells and borings. The sealing characteristics of selected grouts used for well construction were recently investigated using a large-scale laboratory model by Edil et al. (1992). Similarly, Lutenegger and DeGroot (1994) examined methods and materials used for borehole abandonment. Synopses of these studies are provided below.

The annular spaces between well casings and adjacent formations must be properly sealed to eliminate the potential for preferential contaminant transport along the annulus. Edil et al. (1992) used a laboratory physical model to assess several popular sealing grout mixtures. The sealants tested included: (1) Neat Cement (one 94-lb bag of C-150T Type I cement mixed with 5.5 gallons of water); (2) Bentonite-Cement (5 lbs of Quick-Gel© mixed with 6.5 gallons of water and to this slurry, one 94-lb bag of Type I Portland cement was added); (3) Volclay© (2.1 lbs of Volclay© was mixed with each gallon of water to which 2 lbs of magnesium oxide powder was added to each 50 lbs of slurry as a setting initiator); and (4) Benseal©-Bentonite Slurry (125 lbs of Benseal© was mixed into a slurry containing 30 lbs of Natural Gel© bentonite and 100 gallons of water).

Edil's physical model was constructed using a moist sand-filled plexiglass container, 1.5 ft. x 6 ft. x 6 ft. deep (30 x 180 x 180 cm). Completely mixed batches of drilling mud sealant materials were placed in the annular space between simulated 4-in (10 cm) ID steel well casing pipes and outer 8-in (20 cm) ID PVC casings. The sealant materials were either injected at the bottom of the annular space using a tremie tube, or, for the thicker mixtures (bentonite-cement and Benseal-bentonite grout slurries), simply poured in the annular space. After grout placement, the outer PVC casings were slowly lifted from the sand tank allowing direct contact of the grout slurries with the sand. A short section of 8-in PVC casing pipe was then pushed one foot below the surface of the sand at the sand-sealant interface. Infiltration tests were subsequently conducted (in part using Rhodamine dyed water) for 17 weeks by allowing water to seep down from the short PVC pipe into the grout column. At the end of the infiltration tests, the model was taken apart and the sealants were examined slice by slice to investigate dyed water movement and the condition of the sealants.

Based on the measured infiltration rates and a finite-element seepage analysis of the experiments, Edil et al. (1992) found that the hydraulic conductivities of all of the grouts, except for the Volclay©, were on the order of 10^{-7} cm/s. The infiltration rate into the Volclay© grout seal was two orders of magnitude greater, apparently due to a separation between the Volclay© and the steel casing, which occurred as the Volclay© shrank away from the well casing. Edil et al. (1992) concluded that: (1) the effectiveness of a well sealant depends on its structural stability, adherence, and hydraulic conductivity; (2) the Benseal©-Bentonite grout adheres to steel and PVC pipes and provides an excellent seal because of its low hydraulic conductivity, good swelling characteristics, and flexibility; (3) neat cement and bentonite-cement grouts form rigid seals with low hydraulic conductivity and high durability, however, they allowed some limited infiltration at the seal-casing interface; (4) the Volclay© grout does not adhere sufficiently to the well casing; and, (5) Quik-Gel© bentonite slurries of various viscosities and sand contents, like the Volclay© grout, form poorer seals compared to Benseal©Bentonite, neat cement, and bentonite-cement slurry grouts.

Lutenegger and DeGroot (1994) reviewed materials and practices employed to seal/backfill abandoned boreholes, and characterized several sealant materials on the basis of laboratory tests. Materials examined included compacted soil cuttings, compacted soil and bentonite mixtures, and bentonite and cement grouts and mixtures. Based on their evaluation of practices and materials, Lutenegger and DeGroot (1994) concluded: (1) the seal should have a hydraulic conductivity that is less than the native soil, on the order of 10^{-7} cm/s to reduce contaminant movement by advection; (2) the seal material must have sufficient intrinsic structural integrity to prevent seal loss into the native host soils; (3) the seal material should be compatible with the native host soil in order to provide a satisfactory bond at the soil/seal interface; (4) the seal material should be compatible with instrumentation hardware materials left in the borehole in order to provide a satisfactory bond at the instrument/seal interface; (5) the seal should be of sufficient length to ensure that contaminant transport is reduced to an acceptable; and (6) the seal should be considered a permanent repair.

Several practical considerations should be taken into account when selecting a borehole seal. The seal must be constructible by drillers and drillers' helpers using readily available equipment and materials. Sealing materials must be emplaced at appropriate intervals. Generally, it will be important to place sealant materials in stratigraphic units that act as natural groundwater protection layers (e.g., aquitards). As a practical matter, it is probably easier to place the seal over the entire length of the borehole rather than to attempt to construct a seal over an isolated zone. A longer seal length will serve to reduce diffusive transport of the contaminant. Normal grout mixes may not set in highly contaminated situations, especially those involving high concentrations of dissolved salts. There have been numerous cases where dissolved salts (ions in solution) prevented hydration of the bentonite and/or cement. Such conditions require special additives to the mix; proven additives are available commercially. Finally, material and construction costs are always important considerations.

In general, the bentonite and cement grouts and mixtures generally used to decommission boreholes are capable of providing low permeability seals. Lutenegger and DeGroot (1994) concluded that: (1) native cuttings may be used to seal boreholes provided that sufficient compaction effort is used or provided that sufficient bentonite is added (note that this recommendation may not be suitable at some contamination sites); (2) commercially available bentonite chips and pellets commonly used to seal boreholes have similar hydraulic characteristics and can successfully be used provided that they can be placed within the hole and that sufficient water is available to ensure hydration; and (3) bentonite slurries, mixed to proportions in the range recommended by manufacturers (i.e., 20% to 40%), do produce tight seals.

d. ASTM Guides for Decommissioning Wells and Borings and for Monitor Well Construction. ASTM D5299-92 is the standard guide for decommissioning wells and boreholes. The standard lists criteria for plugging materials specify that such materials: (1) must not react adversely with contaminants, groundwater, or geologic media; (2) should have hydraulic conductivities that are similar to, or less than, the adjacent native formations; (3) must have sufficient structural strength to withstand anticipated pressures from native conditions; (4) must maintain sealing characteristics and not degrade due to chemical interaction, corrosion, dehydration, or other processes; (5) should not be susceptible to cracking or shrinkage; (6) should be capable of being placed at the position in the well or borehole where they are needed and must have properties to reduce their unintended movement vertically or horizontally; (7) must be capable of forming a tight bond and seal with well casing and the formation; and (8) must have properties that eliminate leaching or erosion of the plugging material into the formation.

Several procedures are recommended for well and / or borehole plugging. Selection of a particular method depends, in part, on review and inspection of site-specific well construction, depth, formations present, and other factors. Well casings can be removed by either pulling or overdrilling. Depending upon construction, it may be necessary to leave the casing in place and create perforations in the screen and casing to allow for plugging material to penetrate the annular space and the formation. If the grout in the annular space can be verified to be in good condition, the well can be decommissioned by cutting the blank casing and filling the screened interval with grout. If a filter pack is present, it may be necessary to remove the filter pack after perforating the casing, either by washing or overdrilling. If well construction conditions are not adequately known, and the site contains hazardous material, it may not be appropriate to remove the casing and screen, as this may increase the mobility of the contaminants.

The well may require preconditioning to reduce borehole sloughing. This usually involves removing mud from the borehole walls (when drilling mud was used) or stabilizing the borehole prior to plugging (using a drilling fluid). To achieve an effective seal, the borehole should be free of debris and foreign matter that may restrict the adhesion of plugging materials to the borehole wall. One method used to prepare the borehole is to circulate clean water or a high-quality bentonite slurry through a grout pipe set at the bottom of the boring until blockages are removed and the formation is stabilized. As soon as the borehole is prepared, the plugging materials should be slowly injected via a grout pipe placed at the borehole bottom, forcing fluids upward until undiluted grout reaches the surface. The grout pipe can be raised at this point, or when the pumping pressure increases significantly due to the rising grout column. Grouting of shallow auger holes and other shallow borings can be accomplished by placing grout through a side discharge grout pipe that has a funnel attached to the top. As grouting progresses, the pipe is slowly raised.

ASTM D5092-90 is the standard guide for the design and installation of groundwater monitoring wells. It provides recommendations on materials and procedures for installing annular sealants. The guide recommends the placement of bentonite pellet or slurry seals, with a thickness of 3 to 5 feet, above the screen interval filter pack, using a tremie pipe if possible. A 6-in. to 1-ft. filter pack can then be placed over the bentonite seal, and grout fill is then injected under pressure from the bottom up in one continuous operation until the full strength grout flows onto the ground surface without evidence of drill cuttings or fluid. The guide also provides guidance on constructing telescoped, multi-casing wells.

e. Boreholes. All the problems given above for wells, and their solutions, are also applicable to exploration boreholes. A special problem with geotechnical and seismic boreholes is that sealing was not required until the past 10 to 20 years, despite Bowman's (1906) warnings. Therefore, many sites, especially those in industrial areas, have many deep, unsealed old boreholes. Relocation of these old boreholes is virtually impossible.

The are several detailed summaries of drilling, sampling, and well construction methods at contamination sites (Aller et al., 1989; Driscoll, 1986; Hackett, 1987, 1988; Smolley and Kappmeyer, 1991). Drilling at a contaminated site is most commonly done using hollow-stem augers with splitspoon sampling (Riggs and Hatheway, 1988). Despite the advantages of drilling with hollow-stem augers, DNAPL can flow down through the disturbed zone along the outside of the augers and/or possibly enter the augers through joints and sink to the boring bottom. Drilling in rock typically poses a significantly greater risk of promoting vertical DNAPL movement than drilling in unconsolidated media. This is due to the brittle and heterogeneous, fractured nature of rocks. Fracture networks in rocks are usually ill-defined. When drilling in rock, DNAPL can enter and exit the borehole unpredictably by way of fractures. Drilling can also create or widen fractures in the near-well environment. There is some risk of vertical DNAPL migration associated with all drilling methods.

Care should be exercised to avoid causing downward movement of mobile, perched, DNAPL, or DNAPL-contaminated soil while drilling through an indigenous barrier layer. Similarly, DNAPL may sink preferentially along the inside or outside of a well. Specific conditions that may cause downward DNAPL migration include: (1) an open borehole during drilling and prior to well construction; (2) an unsealed or inadequately sealed borehole; (3) a well screen that spans a barrier layer and connects an overlying zone with a perched DNAPL to a lower transmissive zone; (4) an inadequately sealed well annulus that allows DNAPL to migrate through the well-grout interface, the grout, the grout-formation interface, or along vertically-connected fractures in the disturbed zone adjacent to the well; and (5) structural degradation of bentonite or grout sealant, or well casing, due to chemical deterioration by DNAPL or the groundwater environment (Cohen and Mercer, 1993).

To minimize the risk of inducing DNAPL migration as a result of drilling, site investigators should: (1) avoid unnecessary drilling within the DNAPL zone; (2) minimize the time during which a boring is open; (3) minimize the length of hole which is open at any time; (4) use telescoped casing drilling techniques to isolate shallow contamination zones from deeper zones; (5) utilize a site conceptual model (knowledge of stratigraphy and contaminant distribution), and carefully examine subsurface materials brought to the surface as drilling progresses, to avoid drilling through a barrier layer beneath DNAPL; (6) consider using a dense drilling mud to prevent DNAPL from sinking down the borehole during drilling (care must be taken to avoid deleterious mud recirculation); (7) consider drilling horizontal wells; (8) consider using less invasive site investigation techniques, such as use of the cone penetrometer; (9) select optimum well materials and grouting methods based on consideration of site-specific chemical compatibility; and (10) if the long-term integrity of a particular grout sealant is questionable, consider placing alternating layers of different grout types and sealing the entire distance between the well screen and the surface to minimize the potential for vertical migration of DNAPL.

At many sites, the DNAPL or highly-contaminated zones can be adequately characterized while limiting drilling to shallow depths. Characterization of deeper units can be accomplished by locating deeper borings and wells outside the highly-contaminated zone.

7.9.3 Field Performance

There are numerous cases where unsealed wells or boreholes have been pathways for significant or even massive contaminant transport (USEPA, 1977). However, there is little field documentation on the performance of properly sealed wells and boreholes. What documentation exists suggests that the techniques discussed above are adequate for preventing significant contaminant transport along these man-made conduits.

7.9.4 Assessment

As indicated above, the sparse available data indicates that current technology is adequate for installing proper seals on boreholes and wells. The problem lies in finding old wells and boreholes. The current technology for locating old boreholes and wells is inadequate.

7.9.5 Needs

The primary needs within are for improved techniques for detecting and locating boreholes and wells and for greater documentation of field studies that describe sealing behavior.

7.9.6 Foundation Units

Deep foundation units, e.g., drilled shafts and driven piles, can and often do penetrate into or through aquitard strata (potential indigenous barriers). Although these deep foundation units have been suspected by some regulatory authorities as providing pathways for vertical migration, there is little knowledge of this occurring in the field. A literature search yielded one oblique reference, where such a pathway was suspected, but the migration was definitely from a nearby pond (Campbell, et.al., 1984). A model study by Hayman, et.al., 1993, indicated no migration along steel or concrete piles and virtually no contaminants carried along with the piles. Untreated wood piles were found to be subject to "wicking" the contaminants. Other experiments (A.D. Little, Inc., 1981) have shown that stainless steel may be preferentially wetted by organic DNAPLs (thus, possibly facilitating DNAPL movement along steel pilings or casings). It should also be noted that the very action of driving piles sets up high lateral forces against the sides of the piles which tend to close potential pathways. Generally, a remolded zone is created around each pile, destroying any secondary structure in clays.

Drilled shafts, however, are often emplaced by open-hole drilling. This technique reduces stresses in the soils near the shaft, which can open secondary structure in the adjacent formation. Although the shaft forms a conduit while it is open, concrete filling should seal against the soil unless there is excessive shrinkage of the concrete. If the latter occurred, the installed piles would not develop the skin friction on which their capacities depend. Shafts installed using mud-drilling techniques might be more appropriate in known or suspected soil contamination zones.

For the most part, driven piles (other than untreated wood) can be considered a safe technique in contaminated zones. The situation for deep foundation units constructed by augering or similar methods is more ambiguous. The primary need under this subtopic is for field verification of the conclusions given above.

7.10 SUMMARY AND CONCLUSIONS

The performance of an indigenous barrier depends primarily on:

- barrier stratigraphy, which defines likely flow paths,
- barrier material properties, which resist contaminant flux, and
- the potential field, which drives the flux.

7.10.1 Stratigraphy

Stratigraphic characterization at a site usually consists of grouping the material types by their hydraulic conductivities (e.g., sands, clays, etc.). A major thrust of any investigation is the determination of the presence or absence of transmissive defects in the bottom barrier strata (aquitards). Small defects, such as pervious layers, fractures, etc., can dominate the flux characteristics of the barrier.

Current technology for field characterization includes boreholes, cone penetrometer testing, etc., and are generally adequate for defining gross stratification. The interpretive methods, which include computer-assisted data basing, are also adequate for the gross situation. The major problem is in defining the small defects; additional work is needed in this area.

Overall, a thorough investigation can usually characterize stratigraphy to an adequate degree. The effect of stratigraphy on flux can probably be evaluated to within ±20% to ±30% in unfractured media, with less accuracy (say ±50% to ±100%) in fractured media.

7.10.2 Material Properties

The material properties which influence contaminant flux are hydraulic conductivity, diffusion coefficients, and retardation coefficients. Hydraulic conductivity can be directionally dependent in anisotropic media and also highly variable and scale dependent in heterogeneous media. Most of the knowledge regarding diffusion and retardation coefficients is based on laboratory studies; field data are scarce. The major development needs are to acquire more field data relating to these properties, including how they are affected by the subsurface chemical regime.

Definition of the material properties is the least accurate of the three major factors affecting the performance of indigenous barriers. The level of accuracy is probably within one order of magnitude for hydraulic conductivity, even in the absence of chemical or thermal effects. The level of accuracy in the determination of the diffusion and retardation coefficients is unknown, due to lack of field verification.

7.10.3 Potential Fields

The driving forces for advective and diffusive transport are the gradients of hydraulic head, chemical concentration, and temperature. Current technology is adequate for defining these potentials at a point. However, definition of potential fields is less accurate. Nevertheless, potential fields can be determined to a far better level of accuracy than can material properties. The accuracy is probably similar to that of stratigraphy characterization; i.e., within ±20% to ±50% of the actual situation.

7.10.4 Man-Made Discontinuities

Man-made defects in natural (indigenous) bottom barriers can provide contaminant flux pathways which virtually negate the use of a naturally occurring strata as a physical barrier. The major man-made problem is improperly sealed wells and/or exploratory borings. Although sealing methods are adequate, the problem is in detecting perforations. The existing technology for detection of perforations through indigenous barriers is not adequate; work is necessary in this area.

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SECTION 8

ARTIFICIALLY EMPLACED FLOORS AND BOTTOM BARRIERS

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8.1 BACKGROUND

Although emplacement of floors and bottom barriers is a potentially viable technical option for environmental remediation throughout the United States (U.S.), there is little experience and few techniques are available for the construction of floors and bottom barriers. Potential functions for these barriers include the prevention of contaminant migration from subsurface contaminant sources, e.g., underground storage tanks, landfills, soil columns, trenches, and at inactive hazardous waste disposal sites. There are challenges to constructing bottom barriers (depicted in Figure 8-1) that include: heterogeneities in the subsurface; limited site characterization data; buried structures, e.g., buried debris, engineered landfills, tanks, and utilities; complicated site geology and hydrology; limited access; limited available technologies for construction; and difficulties in verifying the integrity of the constructed bottom barrier.

Important technical parameters affecting the successful installation of a bottom barrier include: (1) defining the extent of contamination (for establishing the barrier emplacement depth and length); (2) determining soil porosity, hydraulic conductivity, and moisture content; (3) evaluating site geology to assess drilling and emplacement requirements; (4) selecting the barrier material, whether it is soil and cementitious grouts, chemical grouts, or frozen soil moisture; and (5) establishing barrier performance requirements. In addition, many applications will require that the bottom barrier be interlocked with a vertical barrier. These technical considerations are addressed in other sections of this report (primarily Sections 2, 3, 5, and 9) and the information presented in those sections must be considered along with the techniques presented in this section to produce a competent barrier system. There is very limited field experience with the emplacement of these barriers and with the verification of their construction and performance.

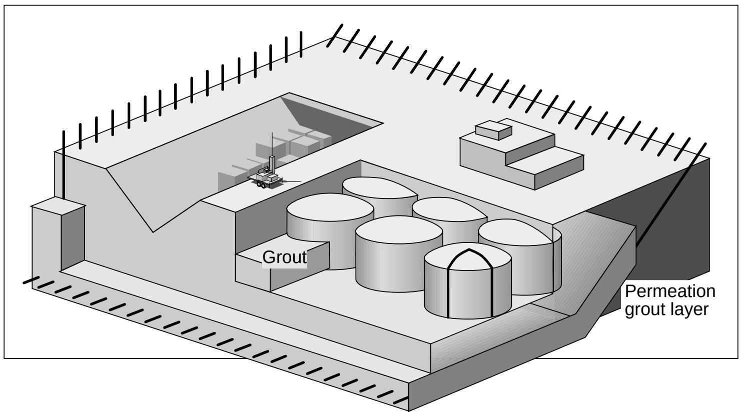

Figure 8-1 Tank farm with grout enclosure barrier.

Techniques for creating a bottom barrier include: permeation grouting, jet grouting, directional drilling, hydrofracturing, block displacement, sheet pile cutoff combined with injection grouting, the Züblin System, microtunneling with injection grouting, and rectangular microtubes pushed side by side (Rumer and Ryan, 1995). Recently, other techniques have been identified and advancements in jet and permeation grouting have occurred. These recent advancements in the state-of-the-practice for emplacement of bottom barriers are described in this section.

8.2 STATE OF PRACTICE

Advancements of the state-of-the-practice can be attributed to the development of new grout materials, the recognition that technologies for establishing bottom barriers warrant development, and the establishment of linkages between U.S. and international industrial expertise. New grout materials (see Sections 2 and 9) have been developed with controllable flow properties and solidification times, making them more compatible with the expected site-specific variations in soil characteristics. Advancements have been made in the techniques for jet grouting and permeation grouting, making them more suitable for bottom barrier construction. However, the key technical and cost issues affecting the implementation of these techniques are related to the drilling of the boreholes. A key to successful directional drilling is the ability to accurately place the boreholes. A guidance system for monitoring and controlling subsurface boreholes is available and has been used in the oil fields and for river crossings. The availability of such guidance systems is an important element in the creation of bottom barriers.

8.3 GROUTED BOTTOM BARRIERS

8.3.1 Construction of Bottom Barriers by Permeation Grouting

Recently developed grout materials, specifically chemical or particulate gelling agents, have significantly advanced the technology of permeation grouting. Previous attempts at solidification, encapsulation, and barrier construction have been hampered by the inability of grout materials to flow into fine media, such as fine-grained sand, silt, and clay. Due to subsurface heterogeneities or stratification, radial grout travel could vary from location to location. Because of the apparent lack of control in the placement of conventional permeation grouts, it has been less favored as a means for constructing a barrier at hazardous waste sites. Factors affecting the penetration of permeation grouts include: grout viscosity, gel-time control, injection pressure, host material grain size, and porosity.

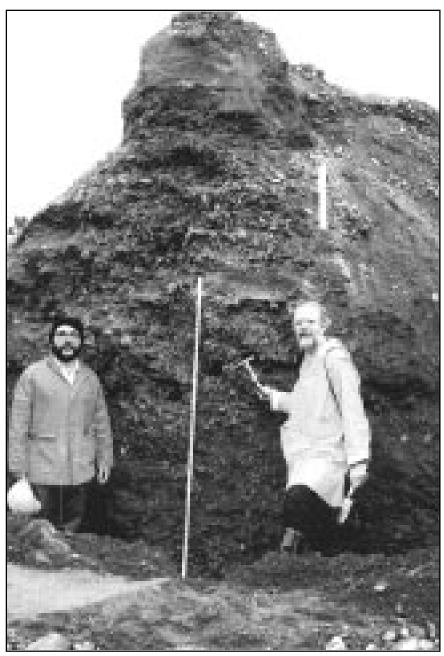

Figure 8-2 Exposed polysiloxane (PSX) grouted plume. The whitish strands of material in the solidified plume indicate the cross-linked solid PSX.

Specific new grout materials that have advanced this technology include polysiloxane and colloidal silica. These materials have a viscosity less than that of water, and testing has demonstrated long-term stability in a wide range of chemical environments. Their low viscosity (< 1 cP) permits these grouts to travel outward as a wetting front, independent of injection pressure. The low viscosity, together with controlled gel-times ranging from several minutes to over 24 hours, enables more efficient placement, by using fewer drill holes than in normal construction practice. The limited field testing completed to date has produced a uniform solidified grout bulb (up to 9 ft in diameter) in heterogeneous alluvial material. The solidified grout bulb is shown in Figure 8-2. This testing demonstrated the ability to permeate fine sands, silts, and voids in clays. Measurements made in the laboratory have shown that hydraulic conductivities of 10^{-8} cm/s were achieved. This is significant because previously, permeation grouting was only able to achieve hydraulic conductivities of 10^{-5} cm/s. The next phase of testing aims to emplace a bottom barrier in heterogeneous alluvial materials using permeation grouting and these grout materials. [See European Chemical News, 1995 and Environmental Engineering World, 1995 for descriptions of this technology.]

8.3.2 Construction of Bottom Barriers Using Directional Drilling and Jet Grouting

The concept of a thin diaphragm wall system with various inclines, interlocks, and column elements was developed in Italy and represents an advancement in the application of high pressure jet grouting. The coupling of this system with directional drilling has enabled the emplacement of three experimental floors. This technology is depicted in Figure 8-3. Using this technique, a vertical, angled, or curved barrier can be created at a depth or length up to or greater than 1000 m. Both cylindrical and planar barrier shapes can be created using either single- or multiple-rod systems. It uses standard pumping equipment, but requires a dedicated directional drill rig and a location/guidance system. A wire-based location and steering system has been used in the past. The operation involves the drilling of a horizontal or angled borehole up to the depth or the length required. The rod-string (generally 60 to 90 mm in diameter) is equipped at the bottom with the drill bit and a nozzle holder device. Initially, the rod-string is extended through the subsurface to the ground surface at the other end of the target zone for the emplacement of a floor or to the target depth, when a complete floor is not needed. Next, the rod-string is extracted at a constant speed, without rotation, while pumping the grouting material through the nozzle at the required pressure. Rotational movement can be added at any time during the extraction, if the application requires it. The excess fluid and soil is forced to the surface around the drill rod. The nozzles are 160° apart, thus enabling the creation of two thin diaphragm walls roughly 6 ft long. The pilot boreholes are repeated on centers of approximately 10 ft to create intersecting thin diaphragm walls that form a containment floor or wall.

The basic elements of this containment system are the panels which can be interlocked with other panels to form horizontal and inclined curved thin diaphragm wall barriers. Because of the greater length achievable with this technique, a barrier can be emplaced under and around a large contaminated area without drilling through the contaminated zone. The barrier emplacement may terminate below the surface at an intermediate layer or may extend to the ground surface. Single- and double-rod systems are used extensively and quite effectively in horizontal and sub-horizontal applications. The radius or depth of penetration is increased with the double-rod system.

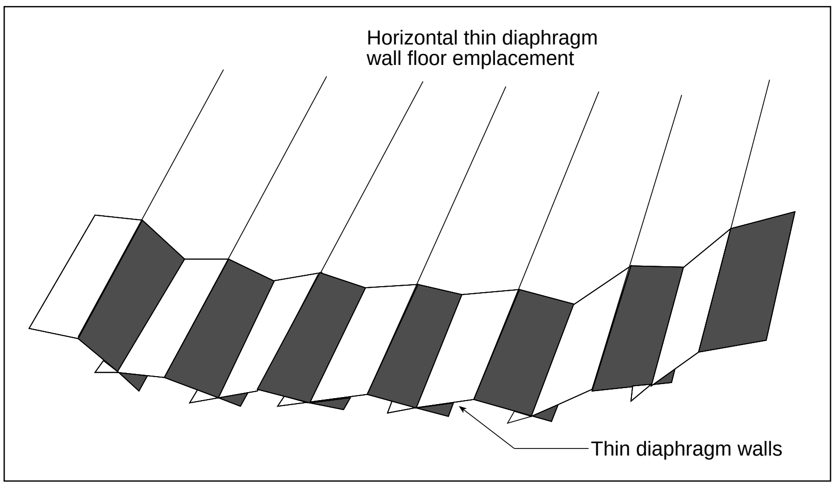

Figure 8-3 Creation of a bottom barrier using thin diaphragm walls and directional drilling technique.

a. Advantages. The advantages of using this technique are as follows:

- The technique utilizes the soil at the site to form the barrier. The only secondary wastes produced are the soil spoils exiting around the rod string during the treatment. However, this material will have been already partially treated with the grout material and therefore, is likely to be less hazardous.
- The high pressure system allows the injected volume of grout and the shape of the barrier to be controlled. This is an advantage over permeation grouting techniques.
- This technique is flexible and enables the creation of barriers over specified distances in the subsurface.
- This system can be used to repair cracks, failures, or defects in the barrier, as well as to seal leaks in other barriers or structures. The barrier is repaired

by drilling a small diameter hole in the area of the failure, followed by the creation of either a column or wall to seal or close the crack. The same approach may be used in the future to increase the depth or extend the barrier.

- The directional drilling technique is a dedicated system and is able to achieve considerable distance, allowing for emplacement of a barrier outside the contaminated portion of the site.
- With modifications, the double-rod system could simultaneously inject two chemical reagents that are designed to react only after injection into the subsurface.
- The double-rod systems can be modified to carry the wires and transmission system for the control/guidance system needed to guide directional drilling.

b. Limitations. The limitations to this technique are as follows:

- The efficiency and continuity remain dependent on site-specific soil conditions. The technique is feasible in loose soil (silts, sands and gravels) and is less feasible in rocks and fractured systems. Pure gravels are not suited for using the directional drilling system.
- The presence of subsurface obstacles or structures require special attention, both in the drilling phase and the injection phase, in order to ensure proper drilling and sealing with the appropriate barrier shape.
- As with any subsurface barrier, the consistency, dimensions, and continuity of the barrier usually cannot be directly observed. Therefore, verification is based on results from field tests, the recording and evaluation of data during operations, and the post operational testing, including coring and geophysical techniques (sonic, radar, etc.)
- Using the control/guidance system, the actual tolerance depends on the precision of the electromagnetic system used and the number of intermediate checkpoints.
- The rod-string may bind and become stuck in the soil, depending on the friction generated between the soil and the rod string, the length of the borehole, the type and stability of drilling fluid, and the soil response to the high pressure injection. This problem can be overcome by increasing the pull force of the drill rig, casing the hole to ensure circulation, and creation of an auxiliary pressure relief hole. These solutions can increase placement costs and may extend construction time.

8.3.3 Construction of Bottom Barriers by Non-Directional Jet Grouting

Conventional jet grouting has been identified as one technique for creating a bottom barrier (Burke and Brill, 1993 and Burke and Welsh, 1995). Conventional jet grouting refers to techniques which use non-directional drilling to emplace grout columns. Barriers produced during jet grouting can differ greatly by varying the many parameters that are part of the grouting process, including: the drilling parameters (drill hole annulus size, lift speed and consistency, and rotation speed and consistency); injection parameters (number of injection nozzles and their size, velocity of injected fluids, and volumes of injected fluids); and grout properties (type of grout, consistency of the mix, and viscosity of the mix). One technique to create bottom barriers using conventional jet grouting is depicted in Figure 8-4.

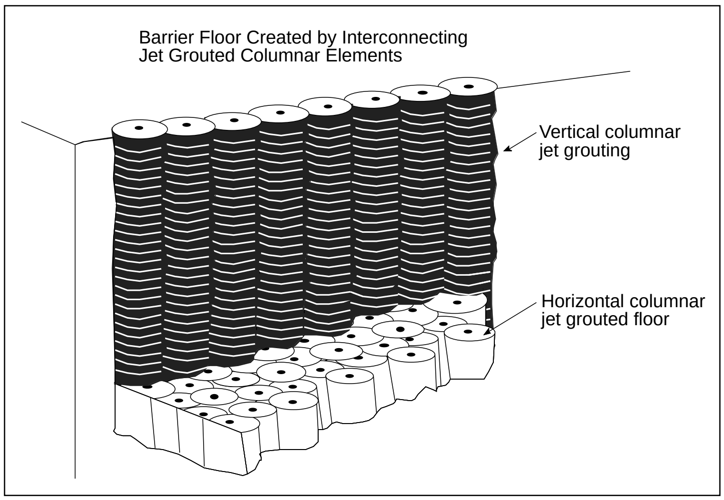

Figure 8-4 Creation of a bottom barrier using conventional jet grouting technique.

In addition, the mono-directional thin diaphragm technique can be employed to create a slanted wall beneath a contaminated zone (shown in Figure 8-5). This technique was developed in Italy and is a variation of the technique described in the previous section. Using this technique, an angled, but straight, structure can be created to a depth or length of 20-30 m. The barrier shape can be either cylindrical, planar, or some combination. The monodirectional thin diaphragm wall technique uses the double-rod system and standard drilling and pumping equipment. The operation involves drilling the vertical or angled borehole using a single-rod string. Once the required depth or length is reached, the grout mixture is ejected through the nozzles. The position of the nozzle is confirmed and extraction of the nozzle then begins at a constant withdrawal speed with no rotation. Nozzle rotation can be added to the technique at any point during extraction, depending on the specific requirements of the project. The monodirectional thin diaphragm wall technique is used for applications requiring hydraulic control (seepage and filtration) without any substantial static requirements. Therefore, the important performance parameters include barrier continuity, competent joints between thin diaphragm wall elements, and optimization of the thickness and amount of injected material. The double-rod system will maximize the dimensions of the barrier without using water. This enables a good sealing effect between the thin diaphragm wall elements. It avoids the clear and defined cutting lines usually associated with the disruption action of the water in the triple-rod system.

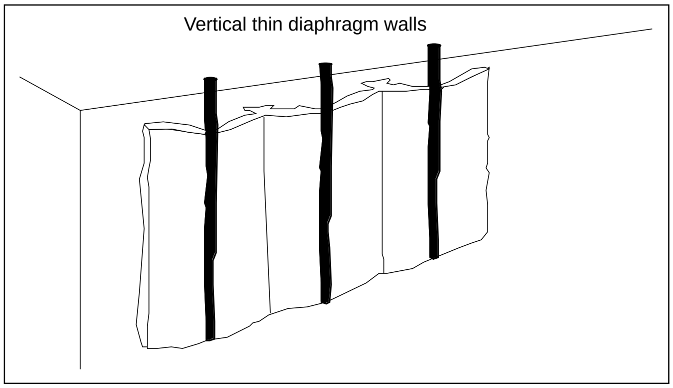

Figure 8-5 Creation of a slanted wall using the mono-directional thin diaphragm wall technique.

The basic elements of the monodirectional system are panels that can be interlocked with other panels for the formation of an inclined thin-wall barrier. During the extraction phase, the monodirectional technique can be alternated with the standard rotational technique so that a continuous structure can be formed with both panel and column elements using the same borehole. The formation of columns and panels can be alternated on different holes creating a mixed structure of full height single panels interlocked with full height single columns. This technique allows for the enclosure of a contaminated zone with a "V"-shaped barrier or connection with a bottom barrier which will broaden the potential applications for bottom barriers.

a. Advantages. The advantages of using conventional jet grouting include the following:

- It is effective in a wide range of soil types, at great depths (up to 150 ft), and above and below the water table.
- The process can be adjusted to compensate for heterogeneities and stratified soil conditions.
- The size of equipment can vary, depending on the needs of the project.
- Unconfined compressive strengths of 10-1000 psi and hydraulic conductivities of 10^{-5} to 10^{-7} cm/s can be achieved.
- The geometry of the barrier can take many shapes and lends itself to repair, when the damaged portion of the barrier has been located.

Additional advantages for using the thin diaphragm wall technique were provided in the previous section.

b. Limitations. The limitations for using conventional jet grouting are identified below.

- It is only cost effective on soils that are easily erodible such as sands, silty sands, etc.
- A mixture of some soil and grout returns to the surface which is a waste by-product that may require disposal if the barrier is constructed in a highly contaminated area.
- The waste return is essential for the creation of uniform soilcrete. If the return is compromised for even short periods, it is likely that hydrofracturing will result instead of erosion, causing inconsistent barrier quality and geometry.
- The sequence and timing of soilcrete formation are very important.
- For applications that do not use the control/guidance systems, the tolerance for drilling the boreholes must comply with specifications established for the project to ensure the interlocking of the thin diaphragm wall elements.

Additional limitations for using the thin diaphragm wall technique were provided in the previous section.

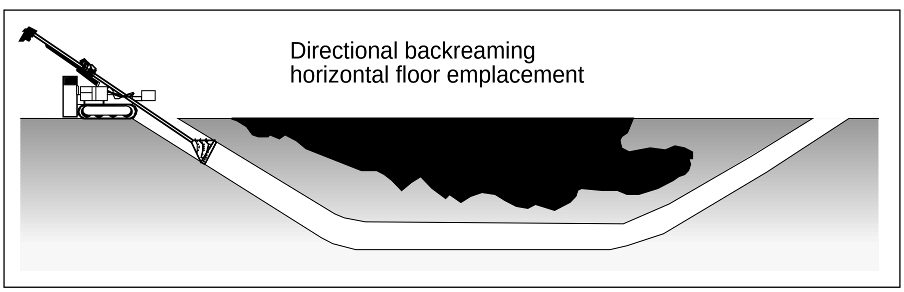

Figure 8-6 Creation of a bottom barrier using directional drilling and scarifying technique.

8.3.4 Construction of Bottom Barriers by Directional Drilling and Scarifying

Horizontal directional drilling is a technology used by the oil, gas, petrochemical, utility, and construction industries for placing underground pipelines, conduits, and utilities under rivers and other locations where conventional trenching methods are unacceptable. It has been used to install small and large diameter pipe over short to long distances in a variety of geologic formations. In construction applications, directional drilling has been used in tunnelling projects where the walls and ceiling of a tunnel are formed by drilling a series of parallel and overlapping holes horizontally to overcome the problems of caving in of soft or unconsolidated formations. The application of this technology is depicted conceptually in Figure 8-6. To emplace a floor, a pilot hole would be drilled beneath the target area, exiting at the ground surface on the other side of the target area. To create a grouted barrier, the drill bit would be removed and replaced by a reaming device that would blend the grout with the soil as the reamer is pulled back under the target zone. Borehole spacing of approximately 5 ft would be required to create a continuous barrier. Although technically feasible, this technique may not be economically feasible because of the large number of directionally drilled holes.

8.4 FROZEN SOIL BARRIERS

Ground freezing is used in civil engineering applications to provide load-bearing strength to soils during construction and to prevent groundwater seepage into excavated areas. Frozen soil barriers have the potential to prevent the migration of contaminants. The system for freezing soil consists of subsurface pipes spaced evenly beneath and around the contaminated zone to be isolated. Inside each pipe is a concentric "feed-pipe" which supplies a coolant, such as calcium chloride or ethylene glycol. The chilled coolant flows through the annulus formed by the two pipes, and then back to the refrigeration plant through the concentric inside pipe. The cold exterior pipe freezes the adjacent soil, forming a frozen soil mass around the freeze pipe over its full length. The thickness of the frozen soil mass increases with time at a rate depending on the soil, soil moisture, and thermal conditions at a given site. A series of freeze pipes, properly spaced and aligned, can form a continuous frozen barrier when the frozen soil mass around one pipe merges with that from an adjacent pipe.

The frozen soil method offers two approaches for containing a subsurface contaminated zone. One approach would be to freeze the entire contaminated zone into a solid mass of frozen soil, thus solidifying the contaminants and preventing their migration. For small areas where the contaminated soil does not include vessels that could rupture from frost heave, this approach may be acceptable. A second approach would be to create a frozen soil containment barrier around the contaminated zone, as shown in Figure 8-7. Ground freezing may have limited application in environmental remediation due to the operation and maintenance costs associated with the need to continually circulate refrigerant to maintain the barrier. The applicability of ground freezing in the environmental marketplace may be limited to emergency responses or interim containment during source removal activities. The technology has been field tested at an uncontaminated site near Oak Ridge, Tennessee.

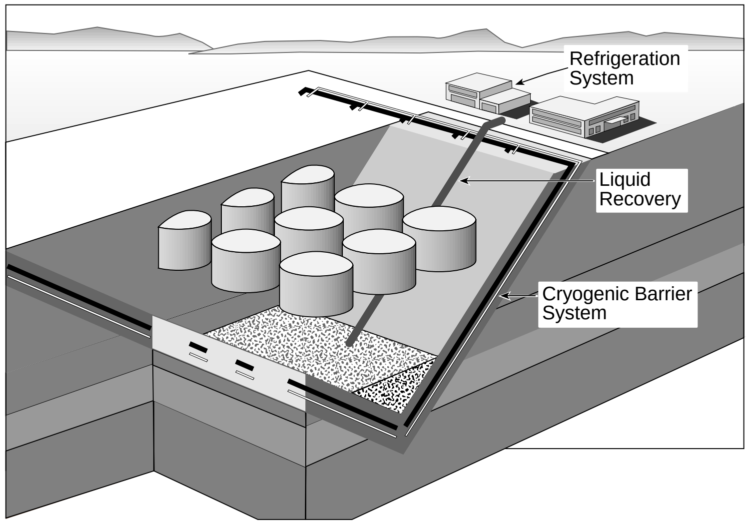

Figure 8-7 Creation of a bottom barrier using the ground freezing technique.

8.4.1 Advantages

The advantages of frozen soil barriers for the confinement of hazardous waste include:

a. Flexibility/Additional Uses

- Frozen soil barriers can be installed in all types of soils where greater than 20% soil moisture is available.
- The barrier can be frozen into any configuration or depth, so long as the freeze pipes can be placed. The containment system can be formed by: (1) placing evenly spaced vertical pipes around the perimeter of the contaminated zone to a depth where they intersect an impervious horizontal strata; (2) inclining the pipes to intercept an impervious barrier, such as another line of freeze pipes, thus forming a "V" trough-type enclosure; or (3) by arranging the pipes to form a "U" shape containment zone, accomplished by directional drilling.
- Openings in the barrier can be closed by adding freeze pipes to the areas in question.
- As with other barrier technologies, the contaminated zone isolated by freeze barrier technology becomes an *in situ* reactor chamber where other remediation technologies can be applied without concern for contaminant migration.
- The thermal gradients induced by the slow freezing process may cause

migration of soil moisture, thus offering the possibility of concentrating dissolved contaminants around the frozen mass.

b. Environmentally Friendly

- Except for the freeze pipes and associated instrumentation, nothing is added to the subsurface to create the barrier.
- Minor cracks in the barrier are self-healing, if sufficient moisture is available to freeze within the crack and seal it.
- The thickness and extent of the barrier can be controlled by adjusting the coolant temperature, by the number and location of freeze pipes used, and by installing heating pipes where necessary to protect utilities located adjacent to or within the barrier confines from freezing.
- The frozen barrier can serve as a structural retaining wall, if excavation and processing of contaminated soil becomes necessary.

8.4.2 Limitations

The application of ground freezing technology is subject to the following limitations:

- Sufficient groundwater must be available so that, after freezing, the ice fills the soil voids sufficiently to create an impermeable barrier.
- If the barrier is to be installed in the saturated zone, groundwater velocities (on the order of one meter per day) may prevent adjacent freezing soil masses to merge and form a continuous barrier.
- The mechanical refrigeration systems require continuous inspection and maintenance.
- Frost heave pressures may cause structural failures in adjacent facilities if the barrier is not properly designed.
- When the barrier is thawed during decommissioning, adjacent facilities may settle causing structural failures if precautions are not taken.
- Frozen barriers may not effectively isolate all dissolved contaminants, especially those that are known to lower the freezing point of water.
- The cost of directional drilling may limit the application of this technology to create barriers beneath large contaminated sites.
- There are few commercially available and reliable sensors for detecting barrier failures.

8.5 GUIDANCE SYSTEMS FOR DIRECTIONAL DRILLING

The most practical technology for monitoring or controlling the trajectory of a subsurface borehole is the magnetic sensor used in conjunction with a gravity sensor. The current state-of-the-art permits the application of magnetometers and accelerometers with only small errors in orientation, azimuth, and inclination. In order to obtain best results, the sensors must be properly mounted in a common probe, properly calibrated, and used with good practice.

The advantages of this guidance technology include: 1) a high state of development; 2) good accuracy; 3) rather robust, reliable instrumentation as presently applied; and 4) relatively low power requirements. Current systems are designed for ease in use. The software supplied (i.e., for the all-angle steering tooled system) has been widely accepted as error-free and user-friendly. Nearly all earth materials are virtually transparent to magnetic fields, certainly so to gravity fields. The few materials in nature that affect magnetic guidance, such as magnetite, do so on a relatively short-term basis in terms of hole length. In addition, a method is available for checking calibration in the field, without the need for other instruments.

Those who have utilized magnetic instruments for directional control have developed an operating protocol for using these instruments that avoids magnetic influences capable of causing unacceptable errors in azimuth readings. The protocol requires the use of a sufficient length of nonmagnetic materials around, above, and below the magnetometers such that the maximum error produced is within tolerable limits. The requirement for nonmagnetic material around the magnetic sensors currently limits the application of this technology.

When directionally drilling at river crossings (trajectories under rivers or other obstacles) or establishing boreholes for strategic placement of drain holes or instrumentation, magnetic interference often is encountered (or is suspected to be present) for which the operator has no control. In some of these cases, precise placement of the boreholes may be of greatest importance. For these applications, where magnetic anomalies are suspected or increased accuracy is needed, Tensor has developed a steering tool system augmented by a surface coil or coils (called TruTracker™) that can be used to generate a predictable magnetic field. The TruTracker™ technology is currently used by approximately 25 licensees. TruTracker™ (shown in Figure 8-8) has been used to place boreholes across river crossings with great accuracy, particularly as the exit stake is approached, even when little or no magnetic interference is expected.

The TruTracker™ technology is used by first placing a coil at the ground surface and extending it along the borehole right-of-way. The width of the coil should be approximately equal to borehole depth. After placing the coil, the location of the corners are stored in the computer coil input file. At intervals during drilling (usually when drill pipe is added), two sets of readings are taken, one set with current flow in the forward direction, the second set with current flow through the coil in the reverse direction. The electrical current is accurately measured and the value entered into the computer. The computer then subtracts one set of readings from the other, thus removing the earth's magnetic field and interference fields. What remains is the field generated by the coil. The algorithms produce a model of the field distribution in terms of field direction and intensity in the plane at right angles with the desired trajectory. By comparing actual readings to the model output, a coordinate position is determined that would bring the difference between the actual reading and the model output to within the desired margin of error. Thus, the steering tool probe is reported to be located at X distance right or left of the centerline and Y distance below the entry-to-exit connecting line elevation.

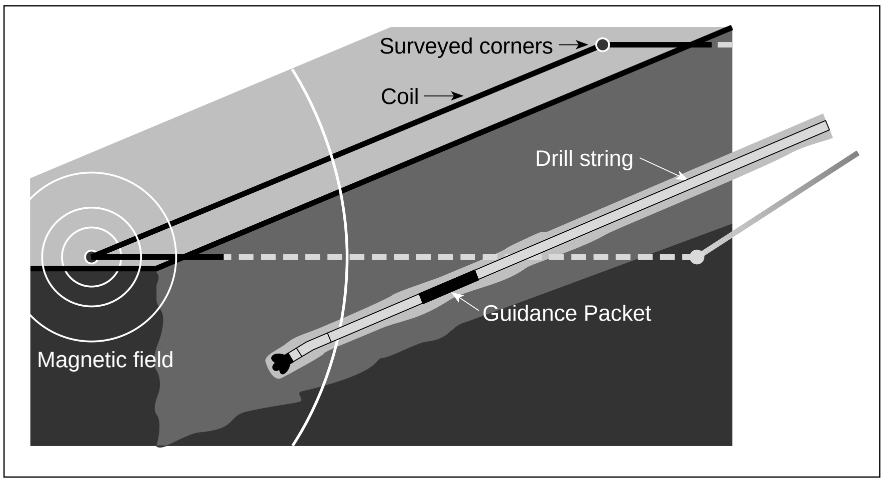

Figure 8-8 Schematic of the TruTracker™ technology for steering the drilling of boreholes.

The TruTracker™ method has been used to place boreholes between adjacent pipelines, under runways containing reinforcing steel, and below multi-story building foundations. Several case studies are briefly discussed below.

- Boreholes for three pipelines were "stacked" with vertical spacings of 10 ft, alongside three others in a limited right-of-way, under a ship channel. Total width of the right-of-way was 40 ft.
- A borehole was placed parallel and between corrugated sheet metal pilings 30 ft apart, under a canal. The total distance was approximately 450 ft.
- A borehole was placed approximately 20 ft under 250 ft of tarmac and 180 ft of runway to exit at a prescribed point within 4 ft.
- Boreholes were placed at approximately 4 ft intervals around the circumferences of a pair of tunnels. The boreholes were placed with an accuracy of +/- 1.5 ft. Their purpose was to reinforce the circumference of the tunnels by grouting prior to boring the tunnels. The trajectory of a section of the tunnels, under the downtown area of a large city in Southeast Asia, was continuously curving, starting side by side at one end, and one above the other at the other end.

8.6 FIELD TESTING OF BARRIER SYSTEMS

Field testing of bottom barrier technologies is necessary to acquire the experienced-based knowledge needed to apply the technologies under varying site-specific conditions. Unfortunately, only a few field tests have been completed for creating bottom barriers.

8.6.1 Construction of Bottom Barriers Using Jet Grouting to Create Thin Diaphragm Walls

The thin diaphragm wall system with various inclines, interlocking, and column elements was developed for seepage, infiltration, and erosion problems on the River Po in Northern Italy. A field test was conducted near the town of Cremona that demonstrated the ability to create a vertical monodirectional thin diaphragm wall in sandy soils. A water/cement/bentonite mix injection material was used. In this field test, a 2.5 m to 4 m radius was achieved which resulted in the treatment of 5 to 7 m^2 per borehole for every 1 m of nozzle movement. The test also showed that the monodirectional and rotational system could be used in combination. A continuous barrier was formed by interlocking the panels and columns. An inclined thin panel was created with just 1 m of cover by orienting the nozzle in the downward direction.

The first full scale application of a thin diaphragm wall occurred in Italy to alleviate a deep seepage problem along a flood protection levee at the confluence of the River Oglio and the River Po. A 3 km-long, 20 m-deep interlocked thin diaphragm wall barrier was formed on the toe of the levee embankment in a predominantly sandy soil. After four years and at least one major flood each year, the seepage problem has not re-occurred.

A similar seepage problem in Italy has recently been solved in the same manner by constructing an 800 m-long, 15 m-deep, 60 cm-diameter interlocked columnar system in a predominantly gravelly soil. This jet-grouted barrier was installed to remedy an emergency deep filtration problem on the River Po embankment near the Ticino River.

Directional drilling using thin diaphragm wall technology was tested in Germany by an Italian-German research group (Fondazioni Speciali - Parma/FlowTex-Ettlingen). The test, conducted in the Bitterfield area, demonstrated the feasibility of constructing a thin diaphragm wall structure using a mineral wax (Montan wax) grout material. A small tub structure was successfully created.

In Roblingen, an 80 m directional drilling thin diaphragm wall barrier was successfully created in conjunction with the formation of a 200 m column. Both single and double fluid jet techniques were used and the operational difficulties encountered were successfully overcome.

8.6.2 Frozen Soil Field Test

The U.S. Department of Energy Office of Science and Technology performed a subsurface groundfreezing demonstration at the Scientific Ecology Group (SEG) facilities in Oak Ridge, Tennessee. The primary goal of the field test was to demonstrate the frozen soil barrier could be used to prevent migration of radioactive and hazardous contaminant plumes.

The ground freezing demonstration was initiated in March 1994 using a configuration shown in Figure 8-9. The site contained a fine clay with chert fragments, typical of the Oak Ridge area. The moisture content was approximately 27%. The demonstration configuration consisted of a double outside ring of freeze pipes and a single inside ring of heat pipes that formed a V-shaped enclosure. The outside and inside dimensions were 56 ft x 56 ft and 33 ft x 33 ft, respectively. The freezing pipes were installed at an angle of 45° and extended to a depth of 28 ft below the ground surface. A 750 gal steel tank was located near the center of the enclosed unfrozen soil. This tank was used to release water and tracers to test the hydraulic conductivity of the barrier. Sensors were placed in the barrier and the adjacent area for recording temperatures, soil conductivity, hydrostatic pressures, and movement of the tracers. Observation wells were also installed.

Two refrigeration units were used, each with a refrigeration capacity of 40 tons utilizing recycled R-22 coolant. Both units were operated during the initial 41-day barrier formation. After formation of the frozen barrier containment system, only one unit was needed part-time to maintain the barrier thickness.

The demonstration showed that a frozen soil barrier can be created that is impervious to water and the tracers used. The two outer rings of freeze pipes formed a 10- to 12-ft thick frozen barrier. The innermost ring of heat pipes circulated warm brine and was effective in controlling the inner growth of the frozen barrier. The power requirement for maintenance of the frozen barrier system was approximately 31% of that needed to initially form the barrier. To the extent that convective heat transfer coefficients could be quantified and the thermal properties of the soil could be characterized, the finite element heat transfer analysis provided an effective analytical tool for predicting the time it would take to develop the frozen soil barrier to the design thickness and the shape desired. The results showed that finite element analysis can be a useful tool for developing freeze wall designs and selecting refrigeration equipment.

A series of tests was performed during installation of the freeze pipes, barrier formation, and maintenance operation. These tests are described below.

- A barrier diffusion test was conducted to measure the diffusive transport of water-borne contaminants through the frozen soil barrier. Fluorescein and Rhodamine tracers, at a concentration of 200 ppm, were used. Fluorescein was released prior to the freezing to assess the hydraulic flow between the release point and the recovery well. Fluorescein was found to be distributed both inside and outside the barrier.

- Approximately 750 gal of water with Rhodamine were released from the buried tank to simulate a sudden release of liquids from a tank and assess the ability of the frozen barrier to contain the liquid. Rhodamine was found only inside the barrier.
- Soil movement was continuously measured after initiation of refrigeration. Theoretical calculations of soil upward movement during testing predicted 1.2 ft to 2.24 ft displacement during the first 70 days. The actual movement measured in the field was 1.65 ft. The maximum heave during steady state operation was measured to be 2.25 ft, which is within the range typically seen during civil engineering applications of this technology.
- The strain gauge measurements were used to analyze the mechanical stresses produced on the tank as a result of soil movement during freezing. The observed maximum stress was 4000 psi, well below the allowable stress of 12,000 psi for carbon steel.

The findings from the field test were promising, but longer tests would be beneficial. The tracers used in the future to quantify diffusive transport through the frozen barrier need to be more representative of the contaminants to be encountered in the field.

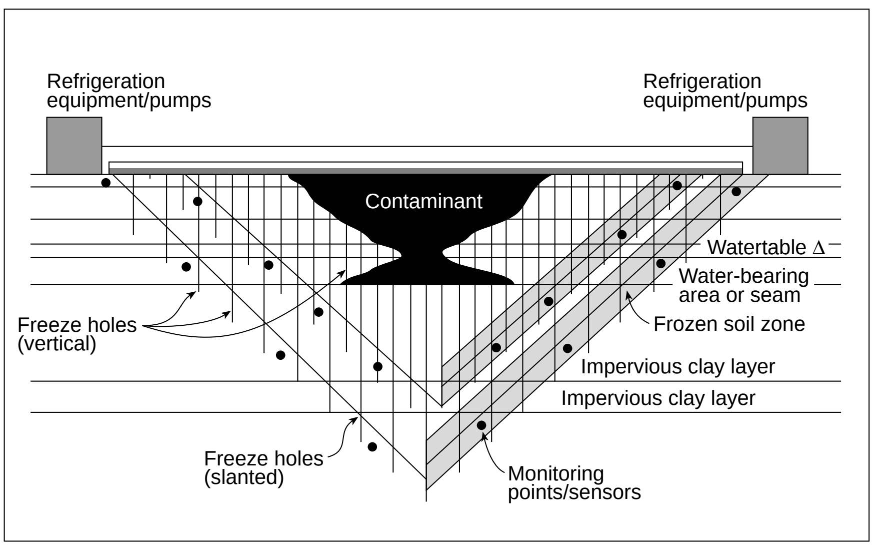

Figure 8-9 Configuration of demonstration ground freezing system at Oak Ridge, TN.

8.7 ASSESSMENT OF THE TECHNOLOGY

8.7.1 Thin Diaphragm Wall Technology

The monodirectional thin diaphragm wall technology is ready for immediate application at sites with limited drill distance and where no active steering device is required. The directional drilling thin diaphragm wall technology, although still in the testing phase to determine the extent of its application, is ready for use in applications where the maximum distances are in the range 200-300 m.

Single-rod systems have been tested and used with cement- based grout mixtures. The system has been technically proven and can be used to pump fluids that are rheologically similar to cement slurries. Mineral wax slurries that have been previously tested can also be used. Application of more aggressive chemical mixtures in the slurry need further study, including evaluation of the appropriate rubber and sealing packs required in the high pressure pump circuits.

Double-rod systems enable the use of a wider range of possible grouting materials because of their ability to separately inject two different grouting components and mix them in the subsurface. These components could be two liquids or a liquid and a gas. Hard soil also can be treated with appropriate spacing of the barrier elements. The triple-rod system, with its water cutting action, may be required for some soil types.

The verification techniques and the characteristics of the barriers produced using these techniques are described below.

a. Emplacement and Performance Verification. There are two phases to quality control during emplacement of the barrier. The first phase of construction quality control is to ensure that the boreholes are in the correct location and that any deviations are within predetermined tolerances.

For the monodirectional thin diaphragm wall system, there is no active guidance control system used during operation because of the relatively short drill length. Precision of placement is based on an accurate “on hole” mast orientation, stable “on hole” drill rig positioning, and rigidity of the drill rod string. Before injection, the borehole deviation can be verified using an inclinometer. Depending on the project requirements, deviations normally should not exceed 2% of the hole depth.

For directional drilling, a guidance control system is used. The accuracy of positioning the borehole is based on the precision of the guidance/control device used during the drilling phase, the position and realization of the reference magnetic field (from the surface with a circuit in the ground, etc.), and the type of drilling technology. The deviation should not exceed 15 to 20 cm, independent of the length of the borehole.

The second phase of construction quality control involves verification of the injection of the grout mixture. This is done by monitoring the rheological and physical properties of the grout mixture being pumped as well as other injection parameters, e.g., pump outlet pressure, nozzle outlet pressure, flow rate, extraction and rotation speed, and pump rotational speed. After emplacement, further verification can be accomplished by coring the barrier or by using non-intrusive geophysical techniques, e.g., cross-hole tomography and seismic reflection-refraction instrumentation. Also, a series of leachate detection borehole devices can be placed around and beneath the barrier to monitor long-term performance.

b. Emplaced Barrier Characteristics. The physical characteristics of the constructed barrier depend on the specific grout mixture used and site-specific soil conditions. The constructed barrier is usually characterized by the compressive strength, hydraulic conductivity, and elasticity modulus. Compressive strengths can range from 2 to 15 MPa (290 to 2,175 psi) for cement-based mixtures in soil formations ranging from clays to gravel, respectively. For Montan wax based mixtures, compressive strengths range from 0.2 to 1 MPa (29 to 145 psi). Hydraulic conductivities can range from 10^{-7} to 10^{-9} cm/sec for panels formed using cement/bentonite mixtures and from 10^{-8} to 10^{-10} cm/sec for panels formed using Montan wax based mixtures. The elasticity moduli generally range from 1000 to 5000 MPa (1.45 x 10^5 to 7.25 x 10^5 psi). Barrier shrinkage during hardening is a concern.

During construction of the thin diaphragm walls and columns, a minimum radius or depth of penetration of the jet stream is required to achieve continuity and interlocking of the elements. For the formation of thin diaphragm walls, the minimum depth of penetration is usually 1.5 m to 2 m, guaranteeing at least 3.0 m^2 to 4.0 m^2 of thin diaphragm wall formation. For the construction of columns, the minimum radius is usually 0.4 m to 1.0 m, guaranteeing at least 0.8 m^2 to 2.0 m^2 of column formation. Under standard operating conditions (two injection nozzles and a 350 hp pump), the minimum barrier thickness is usually 8 to 10 cm near the drill string and 20 cm to 30 cm at the end of the panel. By modifying the nozzle, the nozzle position, and increasing the size of the pump, the minimum thickness can be increased to approximately 15 cm to 20 cm. Depending upon the specific project requirements, thin diaphragm walls are typically spaced 1 m to 3 m apart and columns are typically spaced 0.5 m to 0.8 m apart.

8.7.2 Frozen Soil Barriers

The verification techniques and the characteristics of constructed barriers using this technology are described below.

a. Emplacement and Barrier Verification. Quality control during construction of the frozen barrier begins by ensuring that the freeze pipes have been installed in the proper location followed by the use of temperature probes and conductivity sensors to confirm the extent of the frozen barrier mass.

The location of the freeze pipes in the subsurface is determined by passing a slope inclinometer down each pipe. At the present time, defects in a frozen barrier are detected by analyses of continuously recorded temperature, conductivity, piezometer, and readings taken at critical locations within the barrier and adjacent areas. In addition, the temperature of the liquid coolant is monitored.

b. Frozen Soil Barrier Characteristics. The hydraulic conductivity of frozen soil depends on the type of soil and its temperature, when near the freezing point of water. Andersland, et al. (1995) found that, at a temperature of -10 °C, the hydraulic conductivity of a frozen saturated sand was below 10^{-5} cm/sec, the limit of the accuracy of the equipment used. The thickness of a frozen soil barrier can be controlled to meet the requirements of the site conditions. Thicker barriers provide greater resistance to chemical erosion and offer added resistance to contaminant migration. A typical single row of freezing pipes spaced at about 1.5 m can create a frozen barrier with an effective thickness of about 1.3 m in about 45 days (Sanger and Sayles, 1979). Multiple rows of freezing pipes can be used to form a thicker barriers during the same time period. The demonstration of ground freezing at Oak Ridge (reported by SEG 1995) developed a barrier thickness of about 12 ft in using two layers of freezing pipes.

8.7.3 Cost Analysis

The construction of a barrier floor beneath a subsurface zone of contamination is a relatively unproven technology. Construction approaches can employ conventional civil engineering methods, sometimes using innovative combinations of these methods. For example, conventional high pressure jet grouting technology could be used to create a relatively small slanted "V" trough or conical shaped containment floor. Alternatively, conventional high pressure jet grouting could be innovatively coupled with directional river crossing technology to construct a much larger barrier containment floor. Since construction of artificial bottom barriers is a relatively new field, very few field demonstrations have been conducted, resulting in a paucity of actual emplacement cost data. Nevertheless, reasonable estimates of costs to construct bottom barriers can be based on the operating experience from conventional applications.

For construction of large artificial containment floors, most technologies will likely rely heavily on directional river crossing technology and guidance techniques. It is estimated that the construction technologies currently of interest will require spacings of 6 ft to 10 ft between the directionally drilled boreholes. Directional drilling of a pilot hole typically costs roughly in the range of \$50/ft to \$150/ft for relatively shallow, somewhat cohesive soils. Once the pilot hole is in place, back reaming or scarifying the pilot hole costs roughly \$50/ft to \$75/ft. This does not account for the directional drilling costs to drill beneath the target zone, mobilization and demobilization costs at the site, disposal costs for wastes produced during construction, and contingency costs, e.g., when adverse soil conditions are encountered.

a. Jet Grouting. For conventional civil engineering applications, high pressure jet grouting typically costs $15 to $20 per square foot for columnar walls. For thin diaphragm walls using jet grouting, costs are estimated at $10 to $15 per square foot. For estimating purposes, it is assumed that the typical high pressure jet grouted wall has a diameter of roughly 6 ft and the thin diaphragm wall length is 5 ft in both directions. In addition, the cost of directional drilling is estimated at $8 to $25 per square foot for columnar walls drilled on 6-ft centers and $5 to $15 per square foot for thin diaphragm walls drilled on 10-ft centers. The installed cost of a jet grouted columnar floor is estimated at $23 to $45 per square foot and, for a thin diaphragm wall floor, $15 to $30 per square foot. Not included in these estimates are the costs of the grouting materials, waste disposal, and contingencies.

b. Directional Drilling. Directional drilling using river crossing technology can currently scarify a hole to a 60-in. diameter in one operation after the pilot hole has been drilled. Conventional directional river crossing technology typically costs $50 to $150 per linear foot to drill the pilot hole and $50 to $75 per linear foot to backream or scarify the hole to a large diameter. On a square foot basis, the estimated cost is $17 to $38 per square foot for directionally drilled holes on 6-ft center lines, with a scarified hole diameter of up to 8 ft. Not included in these estimates are the costs of waste disposal, grouting materials, and contingencies.

c. Frozen Soil. Conventional civil applications of frozen soil technology, when used to stabilize soil walls for deep excavations, typically costs $60 per square foot to emplace and roughly $2 per square foot for operation and maintenance. The $60-per-square-foot cost estimate is applicable for smaller installations where standard non-directional drilling technology is used. For applications requiring directional drilling, the costs, based on 10-ft spacings, are estimated at $65 to $75 per square foot. Not included in these estimates are the costs of waste disposal and contingencies.

d. Permeation Grouting. The cost of directional drilling can dominate the overall cost for emplacing a large containment floor using permeation grouting. Similarly, but to a lesser degree, conventional vertical or slant drilling costs can dominate the overall cost for emplacing a smaller containment floor, as compared to the cost of the grout. Based on spacings of 10 ft between boreholes, directional drilling costs are estimated at $7 to $17 per square foot. Not included in this estimate are the costs for grouting materials, waste disposal, surface support equipment, and contingencies.

8.8 NEEDS

8.8.1 General

The following general needs have been identified relative to the construction of artificial floors:

- advanced technologies are needed to verify the emplacement and the performance of the installed bottom barriers;
- new barrier materials need to be developed and tried; and
- improved economic analyses are needed to better estimate the cost of these technologies.

The lack of good economic analyses is partly due to the lack of practitioners of the technology. Well planned and integrated field demonstration tests may provide the most effective means to acquire good cost data.

The major cost and performance factors are associated with directional drilling. Although directional drilling guidance technologies are currently available, their accuracy needs to be improved for ensuring cost-effective emplacement of containment floors. Improved accuracy could reduce the number of boreholes required, thus reducing emplacement costs. Also, many potential applications are in high risk areas where drilling accuracy is critically important.

Currently, there are no cost-effective methods available to verify the emplacement and continuity of an artificial containment floor. Easily detectable chemical additives to the grout may be a potential technique for verifying that a barrier has been properly installed. Also, additional field experience is needed with the new slow setting or reactive grout materials.

Performance standards need to be developed. The development of such standards could greatly impact the costs for emplacement, verification, and monitoring of an artificial containment floor. Will performance standards be based on estimates of contaminant mass flux passing through the barrier during its service life, on the absence of defects within the containment floor, or will they be based on a requirement that the measured bulk hydraulic conductivity in the field meet a project specification? Finally, there are no standards for determining when a bottom barrier would be required under varying site-specific conditions.

8.8.2 Technology Specific Needs

The following technology development needs have been identified and apply to both the directional and non-directional drilling with jet grouting to create thin diaphragm walls technologies:

- Testing is needed under a variety of site-specific soil conditions to develop

methods and hardware modifications that will prevent rod-string binding and sticking.
- A system needs to be developed for detecting the position of the nozzle during the extraction phase of grout emplacement.

For directional drilling with jet grouting to create thin diaphragm walls, the technology development needs include:

- Improvements in the measurement of the alignment deviation of a borehole. A tolerance in the range of 10 cm is desired for all applications (open-hole, blind-hole, extended depth, etc.).
- Ability to directional drill into gravels, cobbles, and noncohesive soils.
- Advancement of guidance control systems. Current systems slow down both the drilling and injection phases of the operation. Wireless systems appear attractive if they can comply with the requirements for accuracy.

For construction of bottom barriers by conventional jet grouting, technology needs include:

- Technical improvement of standard drilling systems and directional drilling systems to improve alignment and increase reachable lengths.
- Improvement of the operational efficiency and mechanical and electronic links between a directional drill sector and high pressure injection sector.
- Development of the ability to inject separately two chemical reagents which are designed to react *in situ* and form the barrier after injection.
- Techniques to verify the accuracy of barrier geometry and hydraulic conductivity, at a macroscopic scale on the order of a meter.

For improved application of soil freezing technologies, the following needs have been identified:

- An effective field method is needed for injecting water into dry soils near the freezing pipes while freezing of partially saturated soil is in progress. This must be developed in order to create an impervious barrier.
- An acceptable performance standard for the frozen soil barrier needs to be developed. For example, will the standard vary depending on the nature of the contaminants being contained?
- The rates of diffusion of different contaminants through the unfrozen water films in frozen silt and clay soil need to be determined.
- The chemical deterioration of the frozen soil barrier needs to be assessed for chemicals that the barrier is likely to encounter. This includes the effect of brine, nonaqueous phase liquids (NAPLs), and radionuclides on the unfrozen water films and the ice in the frozen barrier.
- A reliable method needs to be developed for detecting and locating defects within the frozen barrier. Possibilities include use of Frequency-

Modulated Continuous Wave (FMCW) radar imaging techniques, acoustic imaging techniques, and Time Domain Reflectometer (TDR).

- Additional field data are needed to evaluate and validate the mathematical models used for design and prediction of rates of freezing.

8.9 SUMMARY/RECOMMENDATIONS

Artificially emplaced bottom barriers are viewed skeptically by the public and the regulatory community because verification of emplacement and performance is done indirectly. However, the techniques employed have been routinely used in the construction industry for critical and complex applications. The acquisition of field experience in constructing bottom barriers and the establishment of successful performance by well-documented case histories are key factors to the acceptance and application of these technologies. As field experience is gained, the techniques for construction, verification, and monitoring of emplaced bottom barriers will evolve and be developed for the soil conditions and constraints that exist at hazardous waste sites. Additional research and development are required to provide cost-effective techniques for verifying barrier emplacement and performance. The current interest in bottom barrier technologies by the federal government and industry may aid the formation of joint field demonstration projects, thereby expediting the development and acceptance of this technology. These field projects should be designed to integrate all the elements (grout materials, emplacement technique, monitoring technologies, etc.) required to create a competent bottom barrier system that is acceptable to the regulatory community and the public.

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SECTION 9

CHEMICAL-BASED BARRIER MATERIALS

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9.1 BACKGROUND

Chemical-based barrier materials may provide performance improvements over soil- and cement-based materials. Chemical grouts offer the potential to:

- lower hydraulic conductivity,
- minimize the effects of wet-dry cycling,
- improve resistance to degradation by contaminants,
- minimize diffusive transport of contaminants, and
- enable easier placement due to reduced viscosities and/or little or no particulate matter.

However, chemical-based grouts are usually more expensive than soil-based materials, so these potential improvements in performance must be weighed against the added costs on a case-by-case basis.

Chemical grouts have been emplaced using conventional permeation grouting techniques for decades. However, in recent years, the interest in jet grouting for environmental remediation applications has grown. Jet grouting can provide better control of placement and the specifications on set time may not be as stringent as is the case with permeation grouting. Permeation and jet grouting emplacement methods have been described in Rumer and Ryan (1995). Also, see Sections 2, 3, and 8 in this book.

Selection of the appropriate chemical-based barrier material for a given site depends on factors such as:

- hydraulic conductivity requirements,
- compatibility with soil chemistry,
- compatibility with the contaminants being contained,
- soil moisture content, and
- emplacement method to be used.

9.2 ASSESSMENT OF THE TECHNOLOGY

In this section, the following chemical-based barrier materials are reviewed.

- sodium silicate
- acrylate gel
- colloidal silica
- iron hydroxides
- montan wax
- sulfur polymer cement
- epoxy
- polysiloxane
- furan
- polyester styrene
- vinylester styrene
- acrylic

Sodium silicate and acrylate grouts have been used for decades in field applications such as construction and water sealing. They can provide good performance at a modest cost. Colloidal silica, iron hydroxides, and montan wax have been identified more recently as potentially useful materials for environmental remediation applications. These materials are derived from inorganics or naturally occurring materials and may offer improved performance compared to sodium silicate and acrylate grouts. The remaining materials provide excellent performance, even in aggressive chemical environments, but they are expensive. Sulfur polymer cement has low material costs, but emplacement requires heating of the soil.

Important parameters or properties affecting the performance of chemical-based barrier materials are listed below. Each material will be described in terms of these parameters.

• *Viscosity:* As discussed in Rumer and Ryan (1995) the viscosity requirement for permeation grouting depends on the soil being grouted. With jet grouting, a viscosity close to that of water is needed for more efficient energy transfer and to enable the free flow of excess materials to the surface.

• *Set time:* Set time must be controllable. Requirements depend on the emplacement method. Set time must be long enough for grout to reach its target location and short enough for grout to set before it travels beyond this location. Depending on the emplacement method, a time lag (to account for possible operator error) may be needed to avoid premature setting.

• *Hydraulic conductivity:* The primary function of a barrier material is to prevent or inhibit seepage. Thus, barrier materials are selected for their low hydraulic conductivities. Hydraulic conductivities of 10^{-7} to 10^{-9} cm/s are commonly attained with soil-based grouts. Because they are more expensive, chemical-based barrier materials should exhibit hydraulic conductivities at least equal to, and preferably lower than, those of soil-based grouts. The hydraulic conductivity of a barrier will depend on the soil conditions, the grout material used, the extent to which the grout fills the pore space, and on changes to the soil matrix, e.g. compaction, that occur during grouting.

• *Wet-dry cycling:* Soil-based barrier materials and many of the gel-type chemical grouts require saturated conditions around the barrier to remain intact. Saturation is easily maintained below the water table. Moisture loss can be greatly reduced in vertical barriers by capping them. Barrier materials used at contaminated sites in arid climates must tolerate low soil moisture and the fluctuations in soil moisture that result from precipitation events.

• *Resistance to chemicals: acids, bases, organics:* Barrier materials used in environmental applications may be exposed to a wide range of chemical contaminants. These chemicals may degrade the barrier properties of soil-based grouts. Some chemical grouts will maintain good barrier properties, despite exposure to contaminants.

• *Resistance to irradiation:* U.S. Department of Energy has many sites with radioactive wastes. The ability to withstand exposure to radiation is important for barrier materials that may be used in these sites.

• *Reduction of diffusive transport:* As discussed in Chapter 10, contaminants can be transported through a barrier by diffusion, as well as by advection. The ability of barriers to resist diffusive transport may be critical to long term containment of hazardous materials. Diffusive transport depends on the solubility and molecular diffusivity of the contaminant, as well as on the tortuosity of the barrier pore system and the sorptive properties of the barrier materials. The sorptive properties of many chemical grouts have not been measured.

• *Expected lifetime:* The expected lifetime of chemical grouts depends on the durability of the emplaced grout at the application site. For example, gels will have longer effective lifetimes in the saturated zone than in the vadose zone, with shorter effective lifetimes in arid regions. Data for predicting lifetimes are limited. Sodium silicate and acrylate gels are the only materials that have been used as grouts long enough for lifetimes to be estimated from field performance. For engineered polymers, there are no natural analogs and, while lifetime is expected to be long, testing methods have not been developed that enable accurate predictions of service life in the field. Expected lifetimes of greater than 25 years have been assigned to materials that are expected to last indefinitely.

• *Repairability:* It is expected that containment barriers may sometimes need repair. Some materials may be more easily repaired than others.

• *Safety, toxicity, regulatory acceptability:* These factors must be determined for resins, catalysts, and the final set grouts.

• *Cost of materials:* The total cost of a barrier system includes the costs of both materials and emplacement. Only material costs are compared in this report. The amount of grout needed depends on soil type, emplacement method, and needed barrier properties. Estimated costs are based on a grout-to-soil ratio of 1-to-3. More grout may be needed in some situations, especially when grout flows past its destination during emplacement.

For comparison with other containment methods, e.g., sheet piling and geomembranes, costs have also been calculated per square meter of barrier wall. This is done by assuming a 1 m wall thickness for barrier materials with hydraulic conductivity 10^{-8} cm/s or greater, and a 0.3 m wall thickness for materials with hydraulic conductivity less than 10^{-8} cm/s. For materials of extremely low hydraulic conductivity, thinner walls may provide adequate containment, but they may require more grout per volume of soil. Costs for other wall thicknesses or grout-to-soil ratios can be calculated from the data provided in Tables 9-1 to 9-12.

• *Extent of application:* Many of the materials described have not been field tested. Only a few have been commercially applied in the field.

• *Commercially availability:* This question will be answered for each material.

9.2.1 Sodium Silicate

Sodium silicate is the first chemical grout whose use in soil was documented. In 1886, Jeziorsky was granted a European patent based on the injection of concentrated sodium silicate into one hole and a coagulant into another nearby hole (Karol, 1990). Until the early 1950's, sodium silicate was the only chemical grout in field use. Today, it accounts for 60 to 80% of the total chemical grout volume used in the United States. It is widely used in construction and generally emplaced using permeation grouting. It can be used in combination with cementitious grouts.

Sodium silicate, n–SiO_2 • Na_2O, is commercially available as an aqueous colloidal solution. The silica/alkali ratio should be between 3 and 4. Solutions are typically composed of 20-30% SiO_2, 510% Na_2O, and 60-70% water. When the colloid is mixed with a concentrated solution of the appropriate acid, polyvalent cation, or specific organic reagent, there is nearly instantaneous gelation. The earliest successful field process is credited to Joosten and uses $CaCl_2$ as the salt causing gelation. The reaction is given as:

$$SiO_2 \bullet Na_2O + CaCl_2 + H_2O \longrightarrow Ca(OH)_2 + SiO_2 \bullet 2NaCl$$

This process is virtually unused today. Today, ethyl acetate and formamide are most commonly used to initiate the gelling of the sodium silicate. (Karol, 1990)

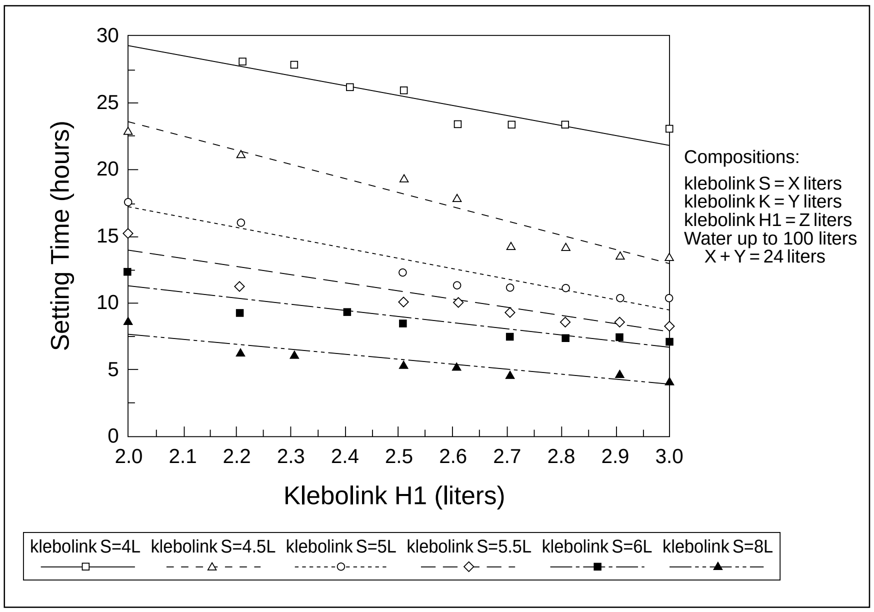

Figure 9-1 Set times of glyoxal-modified sodium silicate (Golder Associates, 1993)

Golder Associates (1993) conducted tests with a glyoxal-modified sodium silicate (GMSS) grout developed by the French chemical company, Societe Francaise Hoechst. This material was extensively tested in the laboratory at the Hoechst facilities in Paris and at the Technical University of Clausthal. It has been used extensively in Europe and the United States as a soil strengthener in unconsolidated soils. GMSS is composed of the following four components:

- water,
- 4-8% Klebolink S, an alkaline liquid with SiO_2, NaO_2, and water,

- 2-3% Klebolink H1, an acidic liquid with glyoxal and additives, and
- 16-20% Klebolink K, an aqueous suspension of non-agglomerated silica particles in an alkaline medium.

The set time depends on the additives used (see Figure 9-1) and can be controlled well for set times of about several minutes, but is poorly controlled for set times of several hours. Set time increases when the material is flowing.

Golder Associates (1993) measured hydraulic conductivities resulting from pumping GMSS through soils taken from DOE's Hanford and Sandia sites. Ungrouted soils had hydraulic conductivities of 10^{-2} to 10^{-3} cm/s. Treated soils had hydraulic conductivities 10^{-5} cm/s. This is consistent with other literature on sodium silicate and glyoxal-modified sodium silicate grouts (Bodocsi et al., 1988).

Sodium silicates are primarily used in the saturated zone. GMSS samples left to desiccate in petri dishes became dehydrated and fragile in 1 day.

TABLE 9-1 Overview of Sodium Silicate

Viscosity: GMSS:	2-5 mPa•s; 5-50 mPa•s for many other products
Set time:	depends on additives*
Hydraulic conductivity measured:	10^{-5} cm/s*
Wet-dry cycling:	crumbles when dried*
Resistance to chemicals: acids:	fair*
bases:	poor*
organics:	fair*
Resistance to irradiation:	unknown
Reduction of diffusional transport:	unknown
Expected lifetime:	10-20 years
Repairability:	good
Safety, toxicity, regulatory acceptability:	non toxic
Cost of materials:	\$0.50/l, \$130/m^3 (25% grout), \$130/m^2 (1 m thick wall)
Extent of application:	used commercially for decades for construction
Is material commercially available?	yes
Unusual strength or weakness:	hydraulic conductivity not very low
* see text for additional discussion	

To test for resistance to chemicals (ASTM C267-82), cylindrical samples of grouted silica sand were immersed for 28 days in each of the following chemical solutions (Golder Associates, 1993):

- 4N hydrochloric acid,
- 6N sodium hydroxide,

- 20% cupric sulfate,
- 100% methanol,
- 100% ethylene glycol,
- 100% aniline, and
- 100% xylene,

In most cases, there was a minimal amount of pitting, as indicated by the presence of sediment in the bottoms of the test beakers. The exception was sodium hydroxide. This sample completely disintegrated within one hour of immersion.

9.2.2 Acrylate Gel

Acrylate grouts appeared in the commercial grouting market in the early 1980's, in response to an industrial need for a less toxic substitute for acrylamide. Acrylate grouts are widely used for seepage control, e.g. sealing storm and sewer lines (Karol, 1990). AC-400, a product made by Geochemical Corporation, is a commonly used, relatively nontoxic, proprietary mixture of the following:

- acrylate monomers,
- methylenebisacrylamide cross-linking monomer,
- ammonium persulfate initiator,
- triethanolamine accelerator, and
- potassium ferricyanide inhibitor.

As seen in Figure 9-2, the gel time depends on the concentrations of the initiator (KFe) and the accelerator (or catalyst).

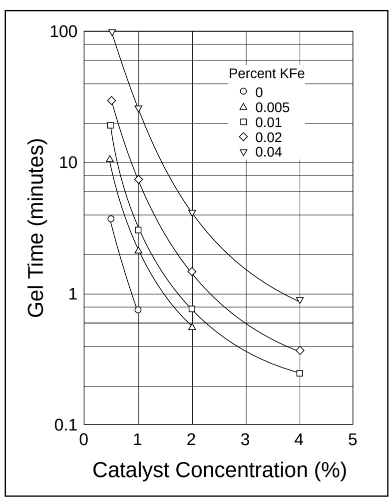

Figure 9-2 Set times of AC-400 acrylate grout (Krizek et al., 1992)

Injection of AC-400 into columns of Ottawa 20-30 sand yielded hydraulic conductivities of less than 10^{-7} cm/s (Siwulo and Krizek, 1992). Hydraulic conductivities of various acrylate grouts without soil have been measured to be 5 x 10^{-9} cm/s (Clarke, 1982) to 5 x 10^{-10} cm/s (Bodocsi, 1988).

Acrylate gels are intended for use in the saturated zone. Burial of a grouted sand sample in saturated sand resulted in sample weight increases of 1-4%. In an arid environment, acrylate gels will dry and lose their barrier properties (Krizek, 1992).

TABLE 9-2 Overview of Acrylate

Viscosity:	2 mPa•s
Set time:	depends on additives*
Hydraulic conductivity measured:	10^{-7} to 10^{-9} cm/s*
Wet-dry cycling:	crumbles when dried*
Resistance to chemicals: acids:	poor*
bases:	good*
organics:	fair*
Resistance to irradiation:	unknown
Reduction of diffusional transport:	unknown
Expected lifetime:	10-20 years
Repairability:	good
Safety, toxicity, regulatory acceptability:	low toxicity
Cost of materials:	\$4/kg, \$3/l 40% solution, \$1/l mixed to 12%, \$230/m^3 (25% grout), \$230/m^2 (1 m thick wall)
Extent of application:	used commercially for 10-15 years for water sealing
Is material commercially available?	yes
Unusual strength or weakness:	
* see text for additional discussion	

Acids and bases were pumped through AC-400-grouted columns of Ottawa 20-30 sand. For solutions with pH as low as 4 and as high as 10, hydraulic conductivities remained less than 10^{-7} cm/s (Siwulo and Krizek, 1992).

The following hydraulic conductivities were measured when other solutions were pumped through various acrylate grouts without soil (Bodocsi, 1988):

- 1N HCl: 10^{-5} cm/s, pH 0;
- 6N NaOH: 10^{-10} cm/s;
- 20% $CuSO_4$: 10^{-7} cm/s;
- 25% acetone: 10^{-8} cm/s;

- 100% acetone: 10^{-4} cm/s;
- 100% aniline: 10^{-5} cm/s;
- 100% ethylene glycol: 10^{-8} cm/s;
- 25% methanol: 10^{-9} cm/s;
- 100% methanol: 10^{-3} to 10^{-5} cm/s; and
- 100% xylene: 10^{-7} cm/s.

9.2.3 Colloidal Silica

Colloidal silica (Moridis et al., 1995b) is produced from saturated solutions of silicic acid by formation of Si-O-Si (siloxane) bonds. Repeated accretion of molecules by this mechanism results in the formation of particles, the size of which can be controlled in the range of 2-100 nm. Uncombined SiOH groups remain on the particle surface. Gelling of colloidal silica particles occurs when the particles approach each other, and siloxane bonds are formed between them (see Figure 9-3).

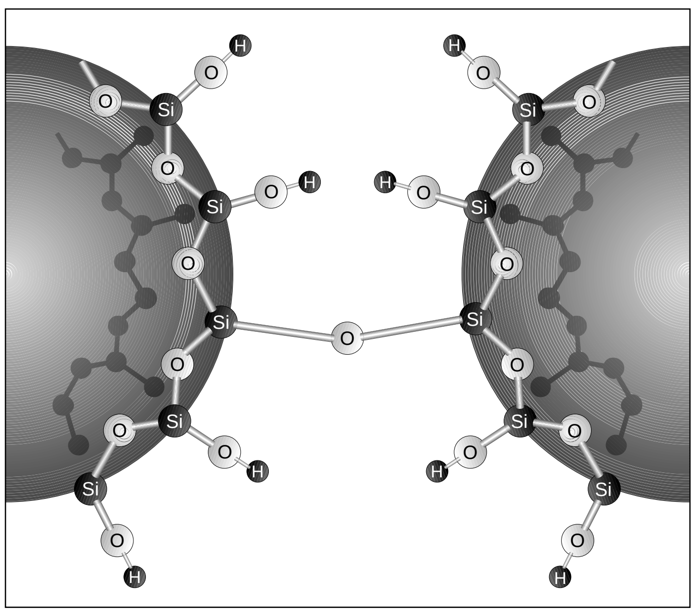

Figure 9-3 Formation of siloxane bonds as colloidal silica particles gel (Moridis et al., 1995)

The colloid is stabilized to prevent gelling during storage and shipment by producing a charge on the particles. This is typically done by raising the pH of the colloidal suspension. In this basic environment, a double-layer of counter-ions, usually Na^+ or NH_4^+, causes a repulsion that prevents particles from approaching each other closely enough to initiate gelling. For controlled gelling, the repulsive forces must be carefully reduced so that random motion of colloidal particles results in collisions and siloxane bond formations at the desired rate. Some ways of accomplishing this are to reduce the pH, to increase the ionic strength, or to introduce multivalent cations.

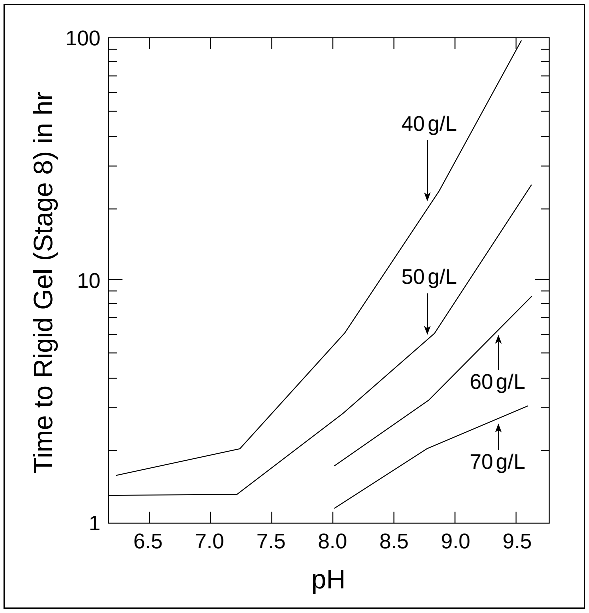

Figure 9-4 Set times of Nyacol 1440 colloidal silica (Moridis et al., 1995)

Figure 9-4 shows sample set times measured at Lawrence Berkeley National Laboratory. Nyacol 1440, a product of PQ Corporation, was mixed with various concentrations of sodium chloride in a 5-to-1 colloid-to-brine ratio. Because injected grout will be buffered by the soil, LBNL staff decided to use ionic strength as the primary means of controlling set time.

Set time of colloidal silica can be accelerated when soils are present, because of the ionic content of the soil. Such rapid gelation can be prevented by (1) preflushing the soil with a 4% NaCl solution to remove multivalent ions, or (2) by using a newly developed generation of colloidal silica products, e.g., Nyacol DP 5110, in which some surface Si has been replaced with Al.

In laboratory tests, Hanford sands of hydraulic conductivity 10^{-2} cm/s were injected with DP 5110 colloidal silica, which is 30 wt % silica. Hydraulic conductivity dropped to 10^{-8} cm/s. For comparison, samples were also prepared in which Hanford sand was added to the colloidal silica solution. This represents a best case scenario, because, unlike with the injection tests, it is certain that pore spaces are filled with grout. These tests also yielded hydraulic conductivity of 10^{-8} cm/s.

In other tests (Noll et al., 1992), sandpacks were grouted with DuPont's Ludox SM colloidal silica diluted to 5 wt %. Hydraulic conductivities before grouting were from 8 x 10^{-5} to 9 x 10^{-4} cm/s. After grouting, hydraulic conductivities ranged from 4×10^{-8} to 5×10^{-7} cm/s.

TABLE 9-3 Overview of Colloidal Silica

Viscosity:	5 mPa•s
Set time:	depends on pH, ionic strength, multivalent ions*
Hydraulic conductivity measured:	10^{-8} cm/s*
Wet-dry cycling:	crumbles when dried quickly
Resistance to chemicals: acids:	expect to be good
bases:	expect to be poor
organics:	expect to be good
Resistance to irradiation:	expect to be good
Reduction of diffusional transport:	immobilizes leachable metals (Noll et al. 1992)
Expected lifetime:	> 25 years
Repairability:	good*
Safety, toxicity, regulatory acceptability:	non-toxic
Cost of materials:	30 wt %: \$1/kg, \$1/l, \$250/m^3 (25% grout), \$250/m^2 (1 m thick wall); 5 wt %: \$40/m^3 (25% grout), \$40/m^2 (1 m thick wall)
Extent of application:	laboratory and field tested
Is material commercially available?	yes
Unusual strength or weakness:	
* see text for additional discussion	

Observations in sandbox experiments at Lawrence Berkeley National Laboratories indicate that a not-yet-gelled colloidal silica plume merged seamlessly with a previously gelled plume, suggesting good repairability.

9.2.4 Iron Hydroxides

Methods for precipitating metals *in situ* have long been investigated for enhanced oil recovery applications. Environmentally benign methods for *in situ* generation of iron hydroxides and oxyhydroxides from low cost iron waste streams are currently being developed by DuPont (Hapka et al., 1995). High concentrations of iron are kept in solution at moderate pH (e.g., pH = 3) using chelating agents. These solutions can be neutralized by (1) mixing with base during jet grouting or (2) using premixed additives that cause a pH increase at a controlled rate following emplacement by permeation grouting.

Results indicate that mixtures of iron, citric acid, urea, and urease could be permeation grouted. The citric acid is used to keep 6 %wt iron in solution at pH 3. Urease, a naturally-occurring enzyme, breaks down the urea. Urea breakdown results in generation of ammonia, which causes the pH to increase.

Set time depends primarily on initial pH and amount of urease. Set times of 5 minutes to 5 hours have been measured in the laboratory. It is expected that longer set times could be obtained by using less urease.

Columns of saturated, graded sand have been grouted in the laboratory by pumping iron solution through them. Measurements using a flexible wall permeameter indicate that the hydraulic conductivity of the sand dropped from greater than 10^{-2} cm/s before grouting to 10^{-7} cm/s after grouting.

Diffusion of contaminants through iron-based grouts has not been measured. However, there is extensive literature concerning sorption of metals to iron minerals such as goethite (e.g. Rodda et al., 1993). Sorption will retard contaminant transport through an iron-grouted barrier.

Long-term tests have not been conducted. Nevertheless, because this grout mimics naturally occurring mineral materials, durability is expected to be very good in some environments.

TABLE 9-4 Overview of Iron Hydroxides

Viscosity:		1 mPa•s
Set time:		depends on initial pH, amount of urease*
Hydraulic conductivity measured:		10^{-7} cm/s*
Wet-dry cycling:		unknown
Resistance to chemicals:	acids:	expect to be poor
	bases:	expect to be good
	organics:	expect to be good
Resistance to irradiation:		unknown
Reduction of diffusional transport:		expect adsorption of metals*
Expected lifetime:		> 25 years*
Repairability:		expect to be good
Safety, toxicity, regulatory acceptability:		non-toxic*
Cost of materials:		permeation grouted: \$0.70/l, \$180/m^3 (25% grout), \$180/m^2 (1 m thick wall)* jet grouted: \$0.20/l, \$50/m^3 (25% grout), \$50/m^2 (1 m thick wall)*
Extent of application:		under development in the laboratory
Is material commercially available?		no
Unusual strength or weakness:		can be made from low cost industrial by products
* see text for additional discussion		

The primary components of the iron solutions tested (i.e. iron, citric acid, urea, and urease) are all naturally-occurring and non-toxic. However, one of the attributes of this grout is that it can be made from low cost industrial by-products. These by-products may contain other materials as well. These other materials will need to be tested for safety and effectiveness on a case-by-case basis.

Because iron-based grouts can be made from inexpensive by-products, the primary costs involved are transportation and additives. For permeation grouting, urease is the most expensive component. The costs given in Table 9-4 may be further cut as the grout formulation is refined. For jet grouting, urea and urease are not needed, so, as listed in Table 9-4, the material is much less expensive.

9.2.5 Montan Wax

Montan wax is a fossilized plant wax extracted from coal or peat deposits. It is a hard, high melting point, non-toxic material that has been used commercially for over 100 years in polishes, carbon paper, as a dispersing and lubricating agent, and as a mold release agent. 80% of the world's supply of montan wax comes from the Romanta plant in Amsdorf, FRG. It has been used in Germany and tested in the United States by Golder Associates Inc. (1993).

Montan waxes are mixtures of pure wax (70%), resins (20%), and asphalt-like materials (10%) composed of C-24 to C-32 carbon chain esters of long chained acids and alcohols. Montan wax grout is a suspension-type grout, consisting of a stable emulsion of montan wax (20%), water (78%), and an emulsifier (2%).

To break the emulsion, 2-5 %wt sodium or calcium bentonite clay is added just prior to injection. The clay binds the emulsifier and causes the emulsion to break. The resulting wax/bentonite mixture is highly viscous and can significantly reduce the hydraulic conductivity of a soil matrix.

The viscosity before setting depends on the amount of montan wax and the amount of bentonite. The set time depends on the type of bentonite used, the amount of mixing, and the soil type. In laboratory tests, rapid mixing results in much faster setting. Set time is difficult to control, ranging from about 20 minutes to several hours.

Hydraulic conductivity was measured by pumping montan wax through soils from DOE's Hanford and Sandia sites. Hydraulic conductivities of soils from the Hanford site changed from 3×10^{-2} cm/s without grouting to 2×10^{-4} to 5×10^{-5} cm/s with grouting. Hydraulic conductivity of soils from Sandia dropped from 1×10^{-3} cm/s to 6×10^{-6} to 6×10^{-8} cm/s after grouting.

Montan wax samples that were left to desiccate in petri dishes lasted several days before dehydrating and becoming fragile.

To test for resistance to chemicals (ASTM C267-82), cylindrical samples of grouted silica sand were immersed for 28 days in one of the following chemical solutions:

- 4N HCl,
- 6N NaOH,
- 20% $CuSO_4$,
- 100% methanol,

- 100% ethylene glycol,
- 100% aniline, and
- 100% xylene.

The xylene and aniline had no observable effect on the grouted sand. The acid, base, and cupric sulfate resulted in some softening at the surface that seemed to progress very slowly. The ethylene glycol and methanol caused the development of a disturbed and loosened zone in the top portion of the sample.

TABLE 9-5 Overview of Montan Wax

Viscosity: depends on composition:	montan wax %	bentonite %	water %	viscosity mPa•s
	20	0	78	5-7
	18	2	78	10-20
	16	5	77	20-50
Set time:	very sensitive to type of bentonite used and amount of mixing*			
Hydraulic conductivity measured:	10^{-4} to 10^{-7} cm/s			
Wet-dry cycling:	crumbles when dried*			
Resistance to chemicals: acids:	fair*			
bases:	fair*			
organics:	fair*			
Resistance to irradiation:	unknown			
Reduction of diffusional transport:	sorbs organics; attenuation measured for TCE			
Expected lifetime:	25 years			
Repairability:	expect to be good			
Safety, toxicity, regulatory acceptability:	non-toxic*			
Cost of materials:	\$1.50/kg, \$1.10-\$1.30/l, \$300/m^3 (25% grout), \$300/m^2 (1 m thick wall)			
Extent of application:	commercially used in the field in Europe			
Is material commercially available?	yes			
Unusual strength or weakness:	set time is difficult to control			
* see text for additional discussion				

Montan wax is nontoxic. There may be some residual toluene, but in laboratory tests it did not leach. Recently, evolution of methane gas has been encountered during long term storage of Montan wax. The methane is thought to be the result of anaerobic activity, which seems to occur with certain emulsifiers when stored at high temperatures.

9.2.6 Sulfur Polymer Cement

Sulfur polymer cement (SPC) was developed by the U.S. Bureau of Mines to utilize surplus sulfur. It is formed by reacting elemental sulfur with dicyclopentadiene and oligomers of cyclopentadiene. Since it is a thermoplastic material, there is a reversible change of viscosity when heated. It has been used for construction of chemical vats and for road repairs. Brookhaven National Laboratories has tested Martin Chemicals' Chement 2000 product for environmental remediation applications (Heiser and Milian, 1994).

SPC must be heated above its melting point (119 °C) to be mixed with soil. At 135 °C, its viscosity is 28 mPa•s (5.85×10^5 lb-s/ft^2). SPC sets when cooled below its melting point, so its set time depends on the rate of cooling.

To emplace SPC, soil must be preheated. Local heating could be accomplished using modified jet grouting equipment. Such equipment might inject steam from a nozzle just above the nozzle from which grout is injected. As the stem is lifted from the soil, soil would be heated and grouted in close succession.

Laboratory tests were conducted by mixing SPC and soil, heating to 124 °C, and pouring into sample holders. Three soil types were tested; blended sand, blended sand and coarse stone, and Hanford soil.

When immersed in ionized water, SPC-grouted Hanford soils cracked after 3 to 5 days. This was probably because of swelling clays present in the soil. When the soil was initially heated to 124 °C, residual soil moisture would have been driven off, and clays would have shrunk to their minimum volume. On immersion in water, certain clays would swell and induce tensile stresses, which could cause the cracking observed in the samples. Therefore, SPC can not be used in clay-containing soils that will get wet.

Grouted samples were tested for resistance to chemicals by immersion for 30-90 days in:

- pH 2 aqueous nitric acid solution,
- pH 12.5 aqueous sodium hydroxide solution, and
- trichloroethylene (TCE)- saturated water.

Samples were removed every 30 days and tested for compressive strength (ASTM-C39).

Resistance to acids was excellent. There were no visual or dimensional changes observed. Composites showed downward trends in the average strength, but the strengths were within one standard deviation of the baseline.

Resistance to bases was poor. No visual or dimensional changes were observed, but the composites showed downward trends in the average strength. SPC is known to be attacked by bases, and so deterioration was expected.

Resistance to TCE was fair. The surface developed a "drip-like" pattern and blotchy discoloration. Strength loss was measured after 90 days of immersion.

TABLE 9-6 Overview of Sulfer Polymer Cement

Viscosity:	28 mPa•s at 135C*
Set time:	depends on cooling*
Hydraulic conductivity measured:	10^{-10} cm/s*
Wet-dry cycling:	swells and cracks when immersed in water*
Resistance to chemicals: acids:	good*
bases:	poor*
organics:	fair*
Resistance to irradiation:	good*
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	by remelting
Safety, toxicity, regulatory acceptability:	not regulated
Cost of materials:	\$0.31/kg, \$630/m^3 (25% grout), \$190/m^2 (0.30 m thick wall) note that emplacement would be more expensive than usual, because of the need to heat the ground
Extent of application:	laboratory tested
Is material commercially available?	no
Unusual strength or weakness:	emplacement requires preheating of soil
* see text for additional discussion	

To test for resistance to radiation, samples were dosed with a total of 1 x 10^8 rads at a rate of 1 x 10^6 to 4 x 10^6 rad/hr. The samples showed no visual or dimensional changes. Destructive tests for compressive strength (ASTM D-695) showed no changes of strength.

9.2.7 Epoxy

CARBRAY 100 is an alcohol-based elastomeric epoxy available from Carter Technologies. It was designed as a two component epoxy for use with conventional jet grouting equipment to encapsulate buried radioactive waste. It may also be used for construction of thin barrier walls. It hardens to a tough rubber-like material resembling a soft polyurethane. Extending the product with water produces a softer product with physical properties similar to polyacrylamide gel polymers. It can also be blended with particulate matter as an extender.

CARBRAY 100's viscosity of 5 to 20 mPa•s (1 to 5.22 x 10^5 lb-s/ft^2) is very low compared to other epoxies. It was designed for jet grouting, but it can also be used for permeation grouting. The grout will fully wet most soils at mix ratios up to about 1 part grout to 4 parts soil. Jet grouting will usually produce lower mix ratios (closer to 1 to 1) and a correspondingly more elastic final product.

TABLE 9-7 Overview of Epoxy

Viscosity:	5-20 mPa•s*
Set time:	8 hours; can be cleaned with soap and water 6 hours after mixing
Hydraulic conductivity measured:	expect to be < 10^{-10} cm/s
Wet-dry cycling:	shrinks upon drying
Resistance to chemicals: acids:	expect to be good
bases:	expect to be good
organics:	expect to be good
Resistance to irradiation:	expect to be good
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	good
Safety, toxicity, regulatory acceptability:	non-toxic
Cost of materials:	\$8/l, \$1600/m^3 (25% grout), \$490/m^2 (0.30 m thick wall)
Extent of application:	limited laboratory testing
Is material commercially available?	yes
Unusual strength or weakness:	unusually low viscosity for an epoxy
* see text for additional discussion	

CARBRAY 100 costs \$8/l for pure resin. In a Soil Saw™ application in damp to wet sandy soils averaging less than 20 blows/foot, resin without water could form a 0.3 m wide wall using a mix ratio of 25% CARBRAY by volume. The materials cost for this wall would be about \$490/m^2. A jet grouted diaphragm panel of thickness 0.15 m and 1-to-1 mix ratio would also cost about \$490/m^2.

9.2.8 Polysiloxane

Polysiloxanes are chemically and biologically inert silicon-based chain polymers. Lawrence Berkeley National Laboratories (Moridis et al., 1995b) has conducted laboratory and field tests with Dow Corning products. PSX-527 was used in laboratory tests. 2-7154-PSX-10 was used in field tests after being developed especially for this application. These grouts consist of the following five components:

- di-vinyl-terminated polydimethylsiloxane polymer,
- di-hydrogen-terminated polydimethylsiloxane polymer,
- a cross-linker,
- a catalyst, and
- an inhibitor which can be employed to control cross-linking after the catalyst is added.

The lengths of the polymer chains determine the viscosity. The cross-linker and catalyst are present in very small amounts. Cross-linking develops when reactions occur between the H terminations and the cyclic or terminal vinyl groups. Several hydrogen-terminated chains can link to a single cyclic molecule to form a complex polymer network.

TABLE 9-8 Overview of Polysiloxane

Viscosity:	PSX-10: 10 mPa•s; PSX-527: 35 mPa•s
Set time:	depends on catalyst*
Hydraulic conductivity measured:	10^{-10} cm/s
Wet-dry cycling:	expect little effect
Resistance to chemicals: acids:	expect to be good
bases:	expect to be good
organics:	may swell*
Resistance to irradiation:	expect to get stronger
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	good*
Safety, toxicity, regulatory acceptability:	non-toxic; inert
Cost of materials:	\$20/kg, \$20/l, \$5000/m^3 (25% grout), \$1500/m^2 (0.30 m thick wall); when mass produced, may cost \$4/kg, \$4/l, \$1000/m^3 (25% grout), \$300/m^2 (0.30 m thick wall)
Extent of application:	laboratory and field tested
Is material commercially available?	yes
Unusual strength or weakness:	high cost
* see text for additional discussion	

Set time of PSX is controlled mostly by varying the amount of catalyst, but there is a lower bound to the catalyst amount, below which a solid phase is not formed. Should longer set times become necessary during application, the retardant can be added to effect an additional means of control. Soil seems to have a slightly accelerating effect on the gelling of PSX, but the degree of acceleration can be easily compensated for by adjusting the catalyst concentration.

In laboratory tests at Lawrence Berkeley National Laboratories, Hanford sand columns of hydraulic conductivity 10^{-2} cm/s were injected with PSX-527. The resulting hydraulic conductivities were 10^{-10} cm/s.

Resistance to chemicals has not been measured. Other polymers similar to PSX sometimes undergo swelling and changes in elastic properties when exposed to solvents, but they do not deteriorate.

Merging of setting PSX plumes has been tested in the field and appears to have been successful. Quantitative measurements are needed to demonstrate the imperviousness of the plume interface.

9.2.9 Furan

Brookhaven National Laboratory (Heiser and Milian, 1994) has tested a commercial, furfuryl alcohol based furan polymer, FA-Rok 913, manufactured by QO chemicals. The furfuryl alcohol resin is biodegradable, nonflammable, and water-soluble. Furans have been used in the fabrication of polymer concrete pipes, as an organic cementing and sand consolidating material in oil wells, as floor coatings, and as chemically resistant containers.

Polymerization of furan occurs through a condensation reaction using a strong acid catalyst. Brookhaven National Laboratories used FA-Rok catalysts. Polymerization is inhibited by basic reagents. Basic soil consumes the acid and prevents polymerization.

Set times and exotherms can not be controlled as well as in other polymer systems. Free floating resin will polymerize very quickly without the heat sink that the aggregates provide. Care should be taken to avoid free standing catalyzed resin in order to prevent localized excessive exotherms.

Laboratory tests were conducted using a grout having a one to two hour gel time. Samples were prepared by mixing grout with soils and pouring into sample holders. Two soil types were tested: blended sand, and blended sand with coarse stone.

Hydraulic conductivity measured using a flexible wall permeameter (ASTM D-5084) for the furan/sand/stone composite was 5×10^{-11} cm/s. Hydraulic conductivity for the furan/sand composite was 3×10^{-8} cm/s.

Furan grouts showed excellent performance when subjected to combined wet-dry cycling and temperature cycling. Samples were cycled 12 times, from 60 °C dry to 20 °C wet (ASTM D-4843). Samples were weighed between cycles and compression tested (ASTM D-0695) after the final cycle. Weight changes of grouted samples averaged 0.23%. Strengths increased by an average of 57%. There may have been further curing induced by the elevated temperature during the dry cycle.

To test for resistance to chemicals, grouted samples were immersed for 30-90 days in:

- pH 2 aqueous nitric acid solution,
- pH 12.5 aqueous sodium hydroxide solution, and
- trichloroethylene (TCE)- saturated water.

Samples were removed every 30 days and tested for compressive strength (ASTM-C39).

Resistances to acid, base, and TCE were excellent. Immersion showed no visual or dimensional changes. No loss of strength was measured.

To test for resistance to irradiation, samples were dosed with a total of 1 x 10^8 rads at a rate of 1 x 10^6 to 4 x 10^6 rad/hr. No visual or dimensional changes were observed.

TABLE 9-9 Overview of Furan Polymer

Viscosity:	3-8 mPa•s
Set time:	not well controlled; builds heat easily*
Hydraulic conductivity measured:	10^{-8} to 10^{-10} cm/s*
Wet-dry cycling:	little effect*
Resistance to chemicals: acids:	good*
bases:	good*
organics:	good*
Resistance to irradiation:	good*
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	new polymer adheres to old
Safety, toxicity, regulatory acceptability:	non-toxic polymer; non-toxic, non-flammable, water soluble, biodegradable resin
Cost of materials:	\$2.35/kg, \$2700/m³ (25% grout), \$800/m² (0.30 m thick wall)
Extent of application:	laboratory tested
Is material commercially available?	yes
Unusual strength or weakness:	inflammable, biodegradable resin; does not set in basic soil
* see text for additional discussion	

9.2.10 Polyester Styrene

Polyester styrenes (PES) are among the most widely used thermosetting resins. PES polymers are a mixture of a linear polyester resin and styrene monomer. Chemical and physical characteristics of the final polymer depend on the ratios of polyester resin to styrene. A need for strong alkali led Brookhaven National Laboratory (Heiser and Milian, 1994) to study a modified bisphenol fumarate resin distributed by Reichhold Chemicals using the tradename Atlac 4010A. A 6% solution of cobalt naphthenate was used as the promoter. Methylethyl detone peroxide was added as the initiator.

Laboratory tests were conducted using a grout having a 1 to 2 hour set time. Samples were prepared by mixing grout with soils and pouring into sample holders. Three soil types were tested: blended sand, blended sand and coarse stone, and Hanford soil.

PES grouts showed excellent performance when subjected to combined wet-dry cycling and temperature cycling. Samples were cycled 12 times, from 60 °C dry to 20 °C wet (ASTM D-4843). Samples were weighed between cycles and compression tested (ASTM D-0695) after the final cycle. Weight changes of grouted Hanford soil averaged -1.2%. Grouted sand/stone had weight change of 0.004%. There was no change of strength.

TABLE 9-10 Overview of Polyester Styrene

Viscosity:	300 mPa•s
Set time:	depends on promotor-catalyst combination, amount and ratio*
Hydraulic conductivity measured:	10^{-10} cm/s*
Wet-dry cycling:	little effect*
Resistance to chemicals: acids:	good*
bases:	good*
organics:	fair*
Resistance to irradiation:	gets stronger*
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	new polymer adheres to old
Safety, toxicity, regulatory acceptability:	non-toxic polymer; toxic, flammable resin
Cost of materials:	\$3.10/kg, \$3200/m^3 (25% grout), \$950/m^2 (0.30 m thick wall)
Extent of application:	laboratory tested
Is material commercially available?	yes
Unusual strength or weakness:	flammable resin
* see text for additional discussion	

To test for resistance to chemicals, grouted samples were immersed for 30-90 days in the following:

- pH 2 aqueous nitric acid solution,
- pH 12.5 aqueous sodium hydroxide solution, and
- trichloroethylene (TCE)- saturated water.

Samples were removed every 30 days and tested for compressive strength (ASTM-C39).

Resistance to acid and base was good to excellent. Immersion showed no visual or dimensional changes. No loss of strength was measured.

Immersion in TCE resulted in moderate degradation characterized by surface blistering, flaking, small depressions, blotchy and "drop-like" color patterns, granular texture, and grout dissolution. Strength dropped slightly after 30 days, but remained constant thereafter. To test for resistance to irradiation, samples were dosed with a total of 1×10^8 rads at a rate of 1×10^6 to 4×10^6 rad/hr. Samples changed colors from beige to yellow after irradiation. No other visual or dimensional changes were observed. Destructive tests for compressive strength (ASTM D-695) showed significant strength increase after irradiation. This effect is attributed to additional cross-linking of the polymer chains.

9.2.11 Vinylester Styrene

Vinylester styrene (VES) polymers have been used to encapsulate radioactive wastes and in a wide variety of applications calling for resistance to harsh chemicals. Brookhaven National Laboratory (Heiser and Milian, 1994) tested a product of DOW Chemicals with the tradename, Derakane 470-45. It is an epoxy novolac-based vinyl ester resin dissolved in styrene. Dimethylaniline promoter was used in conjunction with a 40% benzoyl peroxide catalyst system. The polymerization occurs through an oxidation-reduction reaction. This particular Derakane was formulated to exhibit good resistance to chemicals, retention of properties at high temperatures, and low viscosity.

Laboratory tests were conducted using a grout having a 1 to 2 hour set time. Samples were prepared by mixing grout with soils and pouring into sample holders. Three soil types were tested: blended sand, blended sand and coarse stone, and Hanford soil.

Hydraulic conductivity measured using a flexible wall permeameter (ASTM D-5084) ranged from 1×10^{-10} to 7×10^{-10} cm/s.

VES grouts showed excellent performance when subjected to combined wet-dry cycling and temperature cycling. Samples were cycled 12 times, from 60 °C dry to 20 °C wet (ASTM D-4843). Samples were weighed between cycles and compression tested (ASTM D-0695) after the final cycle. Weight changes of grouted Hanford soil averaged -0.7%. Grouted sand and sand/stone had weight change of less than 0.2%. There was no change of strength.

To test for resistance to chemicals, grouted samples were immersed for 30-90 days in the following:

- pH 2 aqueous nitric acid solution,
- pH 12.5 aqueous sodium hydroxide solution, and
- trichloroethylene (TCE)- saturated water.

Samples were removed every 30 days and tested for compressive strength (ASTM-C39).

Resistance to acid, base, and TCE was good to excellent. Immersion showed no visual or dimensional changes. No loss of strength was measured.

To test for resistance to irradiation, samples were dosed with a total of 1 x

10^8 rads at a rate of 1×10^6 to 4×10^6 rad/hr. The samples showed no visual or dimensional changes. Destructive tests for compressive strength (ASTM D-695) showed significant strength increase after irradiation. This effect is attributed to additional cross-linking of the polymer chains.

TABLE 9-11 Overview of Vinylester Styrene

Viscosity:	100 mPa•s
Set time:	depends on promotor-catalyst combination, amount and ratio*
Hydraulic conductivity measured:	10^{-9} to 10^{-10} cm/s*
Wet-dry cycling:	little effect*
Resistance to chemicals: acids:	good*
bases:	good*
organics:	good*
Resistance to irradiation:	gets stronger*
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	new polymer adheres to old
Safety, toxicity, regulatory acceptability:	non-toxic polymer; toxic, flammable resin
Cost of materials:	\$3.50/kg, \$3600/m^3 (25% grout), \$1100/m^2 (0.30 m thick wall)
Extent of application:	laboratory tested
Is material commercially available?	yes
Unusual strength or weakness:	flammable resin
* see text for additional discussion	

9.2.12 Acrylic

Methacrylate monomers (acrylic) are a commonly used family of polymers. Brookhaven National Laboratory (Heiser and Milian, 1994) investigated the use of a series of acrylics manufactured by the 3M company under the tradename 3M 4R Concrete Restorer. It is a modified high molecular weight methacrylate. The system consists of dicyclopentadienyl methacrylate and isooctyl acrylate. It is polymerized using a cobalt octoate promoter (reducing agent) and cumene hydroperoxide initiator. The laboratory test results presented are for the low modulus product 3M 4R 5741. Based on work with Brookhaven National Laboratory, 3M has recently produced the 5750 series of resins, which is less than half the cost of the original formulations.

Set time depends on the promoter-catalyst combination, amount, and ratio. It can vary from minutes to days. Laboratory tests were conducted with a cobalt naphthenate and cumene hydroperoxide system having a 30-60 minute gel time (24C). Field tests were conducted with a TEABPO system having a 90-100 minute gel time (24C). It is expected that set times as long as one day are possible.

Laboratory samples were prepared by mixing grout with soils and pouring into sample holders. Three soil types were tested: blended sand, blended sand and coarse stone, and Hanford soil.

Hydraulic conductivity measured using a flexible wall permeameter (ASTM D-5084) ranged from <2 x 10^{-11} to 4 x 10^{-9} cm/s.

TABLE 9-12 Overview of Acrylic

Viscosity:	5-10 mPa•s
Set time:	depends on promotor-catalyst combination, amount and ratio*
Hydraulic conductivity measured:	<10^{-9} to 10^{-11} cm/s*
Wet-dry cycling:	little effect*
Resistance to chemicals: acids:	good*
bases:	good*
organics:	fair*
Resistance to irradiation:	gets stronger*
Reduction of diffusional transport:	unknown
Expected lifetime:	>25 years
Repairability:	new polymer adheres to old
Safety, toxicity, regulatory acceptability:	non-toxic polymer, toxic resin
Cost of materials:	3M 4R Concrete Restorer 5741: $14/kg, $15000/m^3 (25% grout), $4400/m^2 (0.30 m thick wall); 5750 series costs less than half this; further reductions are possible
Extent of application:	laboratory and field tested
Is material commercially available?	3M 4R Concrete Restorer 5741 is; 5750 is not
Unusual strength or weakness:	very expensive
* see text for additional discussion	

Acrylic grouts showed excellent performance when subjected to combined wet-dry cycling and temperature cycling. Samples were cycled 12 times, from 60 °C dry to 20 °C wet (ASTM D-4843). Samples were weighed between cycles and compression tested (ASTM D-0695) after the final cycle. Weight changes of grouted Hanford soil averaged 1.0%. Grouted sand and sand/stone had weight changes of less than 0.2%. Acrylic/Hanford composites showed slight increases of strength; acrylic/sand and acrylic/sand/stone composites showed no change of strength.

To test for resistance to chemicals, grouted samples were immersed for 30-90 days in the following:

- pH 2 aqueous nitric acid solution,
- pH 12.5 aqueous sodium hydroxide solution, and
- trichloroethylene (TCE)- saturated water.

Samples were removed every 30 days and tested for compressive strength (ASTM-C39).

Resistance to acid and base was good to excellent. Immersion showed no visual or dimensional changes. No loss of strength was measured.

After immersion in TCE, there was moderate degradation characterized by surface blistering, flaking, small depressions, blotchy and "drop-like" color patterns, granular texture, and grout dissolution. Strength dropped slightly after 30 days, but remained constant thereafter.

To test for resistance to irradiation, samples were dosed with a total of 1 x 10^8 rads at a rate of 1 x 10^6 to 4 x 10^6 rad/hr. The samples showed no visual or dimensional changes. Destructive tests for compressive strength (ASTM D-695) showed significant strength increase after irradiation. - P^{a}effect is attributed to additional cross-linking of the polymer chains.

9.3 FIELD PERFORMANCE

9.3.1 Permeation Grouting Using Colloidal Silica and Polysiloxane

Lawrence Berkeley National Laboratory conducted a field test near Los Banos in the San Joaquin Valley, California in which unsaturated soils in a quarry were permeation grouted with colloidal silica (CS) and polysiloxane (PSX) (Moridis et al. 1995a). The field test was designed to demonstrate various aspects of grout emplacement via permeation. It was not intended to achieve the creation of a continuous impermeable barrier.

The colloidal silica used was Nyacol DP5110, manufactured by PQ Corporation. This colloidal silica has alumina replacing some of the silica on particle surfaces. Its solid content was 30 wt %, and its pH was 6.5. Before injection, it was mixed with a 35 %wt solution of $CaCl_2$ (a 4M solution). The polysiloxane used was a Dow Corning product, 2-7154-PSX-10.

The soil was heterogeneous with discontinuous and lenticular coarser and finer strata, and occasional lenses of well-sorted cross-bedded sands. The matrix was predominantly coarse sand (0.1-1 mm) with large gravel and cobble clasts (mostly 10-100 mm) and a fine film of yellow-brown clay binding much of the sandy matrix. The moisture content of the ungrouted soil varied from 2.5 wt% to 5 wt%. Soil was more moist at depths below 3 m, due to an increase in fines content below 3m.

1500 liters of CS were injected in two wells, one meter apart. 450 liters of PSX were injected in another well. In each well, the liquid grouts were injected through 3 ports at depths 3.0, 3.7, and 4.3 m, using a tube-a-machette technique. During injection, there was no significant rise of pressure, which would have indicated premature gelling.

After grout placement, soil was excavated around the grouted plumes to a depth of up to 21 feet. Both CS and PSX had set satisfactorily. Despite the extreme soil heterogeneity, both CS and PSX created fairly uniform plumes of solidified grout.

The CS grout sealed fractures and large pores in the clays. In zones with gravels and pores on the order of 10 mm, CS did not fully saturate the voids, but appeared to have sealed access to them.

PSX filled and sealed large pores and fractures, as well as accessible small pores located in the vicinity of the larger pores and fractures. In extremely large voids, it coated the individual rocks in the gravel and effectively isolated these zones. PSX also permeated clays and silts, which is unusual. The mechanism through which this penetration was achieved is under investigation.

Cored or carved samples were taken from the boulder-sized grouted chunks for insertion into a flexible wall permeameter to determine hydraulic conductivity. Coring could only be done in material containing no pebbles.

Grouted materials were found to have hydraulic conductivities two orders of magnitude less than the ungrouted sand fraction, which, in itself, was less permeable than the actual geologic matrix. The final hydraulic conductivities of cored field samples grouted with PSX ranged from 1×10^{-4} cm/s to 3×10^{-4} cm/s. The carved field samples grouted with CS had final hydraulic conductivities of 2×10^{-4} to 6×10^{-4} cm/s.

The hydraulic conductivities of the field samples were substantially higher than had been measured in the laboratory. This could be attributed to incomplete saturation of the pore space and damage to the samples during recovery, transport, storage, and trimming. Better sealing may be achievable by multiple, sequential injections of grout.

9.3.2 Permeation Grouting Using Colloidal Silica

DuPont has conducted a field test in which a saturated, fine- to medium-grained sand was permeation grouted with colloidal silica (CS) (Noll, 1993). In one phase of this test, grout was injected to simulate the stabilization of a contamination hot spot. In another phase, the construction of a horizontal floor was attempted.

The material used was Ludox® SM colloidal silica from DuPont. The Ludox® was mixed with HCl and NaCl and diluted to 5 %wt colloidal silica. The mixes were designed to have 40 and 72 hour set times for the hot spot and horizontal floor tests, respectively. Eight batches were mixed during 38 hours of injection for the hot spot stabilization. Thirteen batches were mixed during the 72 hours of injection for the horizontal floor.

Pump tests indicated the sand had an initial hydraulic conductivity equal to 1.3×10^{-2} cm/s. Computer modeling was used to design the most efficient injection and extraction well systems for the two tests.

For the hot spot stabilization test, CS was injected in a single vertical well surrounded by 6 vertical extraction wells, located at a radial distance of 3 m from the injection well. The pumping rate for the injection well was 12 l/min and for each extraction well, 2 ml/min. Tests indicated that NaCl tracer, introduced at the injection well, reached the extraction wells in about 12 hours. However, when CS was injected, CS reached one extraction well in 17 hours and had not reached the other five wells after 36 hours. Ground penetrating radar and soil borings both indicated that CS travelled radially 2 - 3 m from the injection wells and to a depth of up to 4 m. Slug tests conducted after emplacement of CS indicated a hydraulic conductivity of 3×10^{-6} cm/s.

For the horizontal floor test, the objective was to create a bottom barrier, thus completing the construction of a containment system formed by the floor and four vertical cement-bentonite barrier walls that had been previously constructed. The four barrier walls formed a 4 m x 4 m square. CS was injected in a horizontal well parallel to and near the bottom of one barrier wall. Additional horizontal and vertical wells were used to inject and extract water, thereby creating paths by which the CS might flow to the bottom of the opposite barrier wall. Fluids were injected into and extracted from the horizontal wells at 6 l/min. However, CS did not reach the opposite slurry wall, as revealed by both ground penetrating radar and soil borings following the emplacement. Pump tests also suggested discontinuities in the CS floor.

CS did not travel as far as expected in either of these tests. Possible reasons for this include the following.

- When constructing the horizontal floor, increased calcium content in the mix water caused later batches of CS to have shorter set times.
- The vertical component of CS flow was not accounted for during the computer modeling; thus, some of the injected CS may have flowed vertically, rather than horizontally as intended.
- There may have been unidentified soil heterogeneities.

Attempts at sample collection for laboratory determination of hydraulic conductivity were unsuccessful. However, pump tests indicated that hydraulic conductivity was substantially reduced in the hot spot stabilization test.

9.3.3 Permeation Grouting Using Sodium Silicate and Montan Wax

Golder Associates Inc. has conducted two sets of field tests of permeation grouting using sodium silicate and montan wax. The sodium silicate tested was a glyoxal-modified sodium silicate grout developed by the French chemical company, Societe Francaise Hoechst. The Montan wax grout was a mixture of mineral wax and clay developed by the German companies, Vereinigte Mitteldeutsche Braunkohlenwerke AG and DBI Gas-und, Umwelttechnik GmbH.

The first set of tests (Golder Associates, 1994) was designed to evaluate the permeation characteristics of the grouts in a range of soil types and conditions. Single-borehole injection tests were performed at the MWLID site at the Sandia National Laboratories. Grouts were injected at multiple levels within each of four boreholes in soil types ranging from low hydraulic conductivity silts to high hydraulic conductivity coarse sands and gravel deposits. Grout was injected using the tube-a-manchette method.

The sodium silicate grout mix was comprised of two components: a mixture of 3% Klebolink H1, 8% Klebolink S, and water; and 16% Klebolink K. The grout mix had a viscosity 2 to 5 mPa•s (4.2 to 10.4 x 10^5 lb-sec/ft^2). The two components were mixed together using a dual-phase system that simultaneously drew fluid in specified proportions from two reservoirs and mixed the components at the wellhead, just prior to injection.

Three montan wax grout formulations were tested. Montan wax content ranged from 9 to 18%. Bentonite content varied from 1 to 2 %. The montan wax grout was manually mixed and injected in batches, because the prehydrated bentonite was viscous and difficult to pump. The grout was continuously stirred with a recirculating fluid pump to minimize plugging in the injection line.

Geophysical methods were used to monitor movement of grouts, including measurements of electromagnetic resistance, moisture content, and temperature. These methods gave some indication of the arrival of the grouts in the vicinity of boreholes, but none of the methods gave consistent indications of grout permeation.

Soil samples were collected using a Shelby tube, but samples taken for gas permeability tests were disturbed during collection. Soil samples were manually inspected to assess the presence of grout, by sight and smell.

The grout penetration distances typically ranged from 1 to 2 m, for both the sodium silicate and montan wax grouts. Grout penetration appeared to have been either asymmetric, heterogeneous, or both.

These test results suggest that both the sodium silicate and montan wax grouts can be injected in unconsolidated soils using conventional grout permeation methods, provided the soils have sufficiently high hydraulic conductivity. The minimum hydraulic conductivity for permeation grouting, using either sodium silicate or montan wax grout, appears to be around 5 x 10^{-4} cm/s. Permeation grouting was not possible within the fine-grained materials at the site, due to low hydraulic conductivities.

A second set of tests (Voss et al., 1995) was undertaken to demonstrate the feasibility of constructing subsurface barriers under field conditions. The primary goal was to produce a continuous horizontal barrier with a hydraulic conductivity of 10^{-6} cm/s. Multiple-borehole tests were conducted near Richland, Washington.

The site had laterally continuous, poorly-graded medium grained sands at a depth of 5 m below the ground surface. Beneath the medium sands were finer sand and silt. The hydraulic conductivities of the medium sand and fine sand were 9 x 10^{-2} cm/s and 8 x 10^{-3} cm/s, respectively. The water table was about 38 m below surface.

Grouts were injected between 3 and 5 m deep, using tube-a-manchettes. Geophysical monitoring was used to monitor resistance and moisture.

Permeation grouting using the montan wax grout was largely unsuccessful. Grout was observed flowing at the surface along or near the borehole annulus, despite various attempts to reseal the borehole, inject additional boreholes, or install sleeves with additional locations to inject annular sealing material. Hydraulic fracturing of the soil was suspected.

The set time of the montan wax emulsion is highly sensitive to the type of bentonite clay, the amount of bentonite clay, and the amount of mixing that occurs. The presence of natural clays in the soil can also affect set time. Under field conditions, it was very difficult to control either the mixing time or the amount of mixing. As a result, the break time was likely to have varied and the lack of permeation may have been due to grout setting soon after it was injected into the soil. Similar problems have been encountered in the German montan wax grout program, where permeation grouting has largely been abandoned in favor of other application methods, such as jet grouting.

The sodium silicate grout was successfully injected, but placement direction and distribution were uncertain. There were two occurrences where grout flowed to the surface, but these situations were corrected by resealing the borehole annulus. Geophysical methods indicated that grout flowed radially over 1 m from the point of injection, but there was no indication that the target penetration distance of 3 m was achieved. This may have been due to significant vertical grout movement, or that the geophysical tools failed to detect the presence of the grout.

The grouted soils have recently been excavated, but a full report is not yet available. It is doubtful that a continuous layer was made. Penetration was sometimes uniform, but sometimes there was much fingering.

9.3.4 Jet Grouting Using Montan Wax

FlowTex®, a German company, has been testing their Flowmonta technology for creating horizontal barriers with montan wax (Sass, 1995). By constructing a bowl-shaped barrier beneath a waste site, the uncertainties associated with keying vertical walls into a natural floor are avoided.

Four methods have been proposed for verifying the integrity of a constructed bowl-shaped barrier:

- pump tests,
- horizontally drilled wells, utilizing fiber optics and geoelectric seismic topology,
- core sampling, and
- utilization of a geoelectric direct control system to measure changes in electrical resistivities between overlapping borings (still under development).

A combination of horizontal directional drilling and jet grouting is used to emplace the barrier. Drilling begins at one side of the waste site, passes beneath the waste, and exits at the other side. At the exit side, a jetting tool is attached and jet grouting proceeds while the tool is pulled back to the beginning side. Grout is jetted in only two directions, to create overlapping panels. Figure 9-5 illustrates how a system of overlapping panels could be used to create a bowl-shaped barrier.

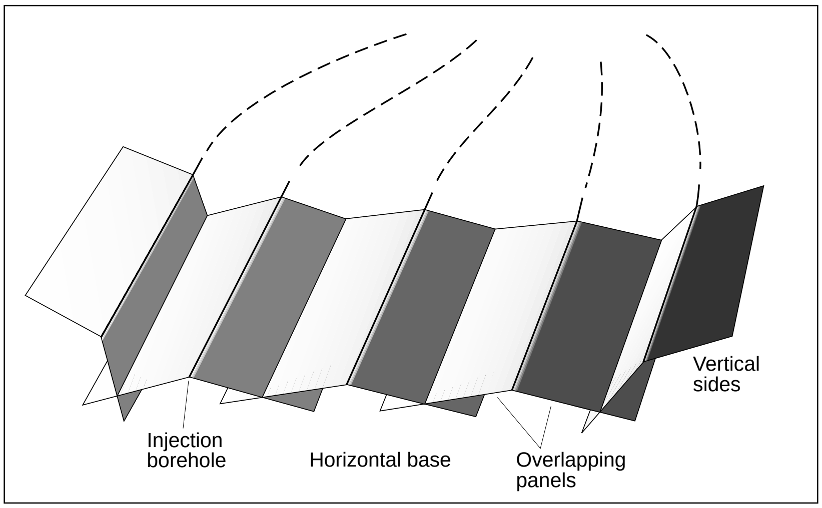

Figure 9-5 FlowTex® barrier system (Sass, 1995)

In preparation for constructing a bowl-shaped barrier, grouts of different viscosities were tested by injecting 200 m long columns. Based on the observed performance, it was concluded that it would be possible to construct 1000 m columns. Drilling accuracy is within 0.1 m over 100 m.

The bowl-shaped barrier was to be 9 m deep and 100 m long. The soil was an overconsolidated heterogeneous glacial till with clay and sand intercalations. Overconsolidation and shallow depth made fracturing to the surface a possibility. The soil hydraulic conductivity was as low as 10^{-5} cm/s, two orders of magnitude below that normally desired for jet grouting.

A mixture of montan wax and cement bentonite was injected. The cement bentonite was prepared and mixed with montan wax. The grout did not appear to follow preferential paths. Its movement was predictable for at least 1.5 m from the points of injection. After penetration over several meters, horizontal fracturing and uncontrolled flow occurred.

Post-placement coring has shown grout to be located where expected. Currently, additional tests are being conducted, including: pump tests, tests for radon penetration, and tracer gas tests using carbon dioxide and helium. At the completion of these tests, the site will be excavated to examine undisturbed samples.

This test has revealed the need for well-controlled gel times and better site characterization, especially when heterogeneous soil conditions are present.

9.3.5 SoilSaw™ Emplacement of Montan Wax

SoilSaw™ has been used to emplace montan wax in combination with 4% pre-hydrated bentonite (E. Carter, 1995). SoilSaw™ is a modified form of jet grouting, designed to make continuous, uniform, barrier walls at speeds higher than other methods. It consists of a heavy beam that is reciprocated in the ground by a crawler mechanism. Several jet nozzles are located along the length of the beam. As the beam is passed through the soil, the injected grout disrupts and mixes with the native soil to form backfill in a single step. SoilSaw™ operates at ten times the flow rate of typical jet grouting. See Section 3 of this report for more discussion of the SoilSaw™ method.

The montan wax-bentonite combination produced a soft, plastic material. Upon examination, the side cuts looked good. However, the montan wax did not provide any significant performance improvement over bentonite alone.

Difficulty was encountered in obtaining undisturbed samples from the field for laboratory analysis. A method was developed for hand trimming samples in the laboratory, which seemed to improve things.

9.3.6 Jet Grouting Using Acrylic

At Idaho National Engineering Laboratory, a test was conducted in which acrylic was jet grouted to stabilize a waste pit (G. Loomis, 1995). A two-component resin system was used, one half had catalyst and one half had promoter. The two components were held in separate tanks and emplaced through a double-wall drill stem having two nozzles. The two components did not mix until injection into the soil. By this method, a grout with a short set time of 90 minutes could be used. Short set times are useful when the soil is very coarse, since grout would pass through the coarse soil if it did not set quickly.

In this test, 35 columns were constructed using two different resin blends. Gel times in the field matched laboratory measurements. Further results are not yet available.

9.4 NEEDS

9.4.1 Assessment of Field Performance

Extensive laboratory testing has been completed for a wide range of chemical grouts, but only a few chemical grouts have been tested in field studies. For the field tests that have been conducted, evaluation of placement methods was emphasized, rather than barrier performance. Additional field tests need to be conducted with more attention given to barrier performance.

In many of the field tests, investigators found or suspected deterioration of the collected field samples, making laboratory verification of field performance difficult. Better sampling methods need to be developed and disseminated to address this problem.

Based on the findings from additional field tests conducted to evaluate barrier performance, it may be possible to design laboratory tests that will better simulate field performance. Field performance can be affected by many factors, including site soil chemistry, overburden pressure, and the emplacement method used. In studies using sodium silicate, strengths of grouted soils measured in the field were about three times those measured in laboratory-prepared samples. In tests with jet grouting, lower hydraulic conductivities have been measured in the field than in the laboratory (possibly due to sample disturbance). Although laboratory testing is much faster and less expensive than field testing and it allows the variation of more parameters, care must be given to assure that laboratory test results are true indicators of field performance.

9.4.2 Assessment of Long Term Performance

Service life expectancy of containment systems for environmental remediation applications is an important performance criterion. Service life expectancy will remain uncertain until constructed grouted barriers have been in the field for extended periods of time. However, laboratory testing can be used as a basis to estimate life expectancy. Such laboratory tests are not presently being conducted. A focused effort should be made to adapt testing methods from other disciplines to testing of barrier materials. In other areas of material science, tests are routinely conducted to predict long term performance using the results from short-term tests.

Better test methods are needed for determining the long-term performance of emplaced chemical grouts exposed to contaminants. For acrylate grouts, test findings indicate that hydraulic conductivity changes when permeated by various chemical solutions. For many other chemicals, grouted samples were immersed in chemical solutions for 30-90 days, with assessment of resistance to chemicals made visually and by strength testing the samples. Pumping chemical solutions through grouted samples is a better simulation of long term exposure, and the effect on hydraulic conductivity is of great importance in the design of environmental containment systems. However, such testing can be time consuming for very low hydraulic conductivity materials. Better test methods are needed for these materials.

As discussed in Section 10, the migration of contaminants through barrier materials can occur by diffusive transport, as well as by advective transport. Few measurements have been made on the diffusive transport or sorptive properties of chemical-based grouts. In the absence of measurements, estimates may be possible for some materials based on available published data. Continued evaluation of diffusive transport and advective transport through these materials may indicate the point at which further reduction of hydraulic conductivity has little effect on contaminant transport. For some of the chemical grouts having extremely low hydraulic conductivity, it may be possible to develop less costly formulations that yield comparably low contaminant transport rates.

9.4.3 Properties Affecting Verification

As with all containment systems, verification of emplacement and performance monitoring are critical. Since chemical grouts can be custom-engineered, it may be possible to develop materials with specific properties that make detection in the field easier. For example, if the emplaced barrier material has an electrical resistance that is dramatically different from the surrounding soil, detection of defects would become more feasible. Developers of chemical grouts need to work closely with developers of performance monitoring systems to determine the grout properties that may make verification and monitoring easier.

9.4.4 Performance Standards

This review has show that there are many chemical grouts presently under study, primarily in the laboratory. The development of performance standards will make it easier to determine which materials deserve continued study, and the grout properties warranting more aggressive research and development. Some of the materials presently being studied may perform better than needed. Noll et al. (1992) showed that good performance can be attained when colloidal silica is diluted by a factor of six, greatly reducing material costs. Identification of performance needs may make it possible to develop more cost-effective materials.

9.5 SUMMARY

Table 9-13 presents in summary form some of the key properties of the materials that have been reviewed. Selection of chemical grouts will depend on the site conditions and containment needs. Traditional materials, e.g., sodium silicate and acrylate gel, provide modest performance at modest cost. Other more recently identified materials based on inorganics (e.g., colloidal silica and iron materials) and naturally-occurring materials (e.g., montan wax) provide improved performance in specific areas. The remaining materials either require special emplacement methods (e.g., sulfur polymer cement) or utilize engineered polymers. These remaining materials may provide excellent performance for a wide range of conditions, but at a high cost.

Data obtained from field tests and demonstrations using these newly developed chemical grouts are limited. There are some indications that field performance has been poorer than expected, based on laboratory measurements. To further evaluate the performance of chemical-based grouts, additional field tests are needed as well as improved methods for retrieving field samples for laboratory testing. Based on these additional field tests, the relationship between field and laboratory performance can be better assessed, and improved laboratory test methods developed, as necessary.

TABLE 9-13 Summary of Key Properties of Chemical Grouts*

Grout	Hydraulic Conductivity (cm/s)	Resistance to Acids	Resistance to Bases	Resistance to Organics	Expected Lifetime (Years)	Cost** ($/m^3)	Cost*** ($/m^2)
sodium silicate	10^{-5}	fair	poor	fair	10-20	130	130
acrylate gels	10^{-7}-10^{-9}	poor	good	fair	10-20	230	230
colloidal silica	10^{-8}	good	poor	good	>25	60-330	60-330
iron hydroxide	10^{-7}	poor	good	good	>25	50-180	50-180
montan wax	10^{-4}-10^{-7}	fair	fair	fair	25	300	300
sulfur polymer cement	10^{-10}	good	poor	fair	>25	630	190
epoxy	10^{-10}	good	good	good	>25	1600	490
polysil-oxane	10^{-10}	good	good	good	>25	1000-5000	300-1500
furan	10^{-8}-10^{-10}	good	good	good	>25	2700	800
polyester styrene	10^{-10}	good	good	fair	>25	3200	950
vinylester styrene	10^{-10}	good	good	good	>25	3600	1100
acrylics	10^{-9}-10^{-11}	good	good	good	>25	7000-15000	2000-4400

* See 9.2 for discussion of properties
** Cost of grout, per unit volume of grouted soil
*** Cost of grout, per unit area of barrier wall

Better laboratory tests are needed for estimating the long term performance of chemical grouts. Testing is especially needed to determine:

- barrier performance in the absence of unusual stresses,
- barrier performance when exposed to chemicals, and
- diffusive transport of contaminants.

Performance standards are needed to guide the development of cost-effective materials. For materials with extremely low hydraulic conductivity, lower cost versions of these materials may also provide adequate performance.

Although chemical grouts have been used for decades in construction and water sealing, development of chemical grouts for environmental remediation applications is a relatively new field. Chemical grouts can be made from a wide range of materials, offering the option of customizing to meet specific performance requirements. Further laboratory testing, field testing, and performance standards are needed to develop materials that will provide long term, cost-effective performance.

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SECTION 10

CONTAMINANT TRANSPORT MODELING

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10.1 BACKGROUND

10.1.1 Overview

Mathematical modeling is a well-established tool for the analysis and design of systems for environmental remediation. Due to the lengthy time scales for transport processes and the anticipated long decision horizons, modeling is expected to play a significant role in the analysis of containment systems. A number of aspects of barrier performance are amenable to modeling analyses, including:

- conceptual design of low-permeability barriers;
- conceptual design of permeable reactive walls;
- performance assessment of barrier systems; and
- risk assessment for systems that include barriers.

The focus of this section is on the analysis of barrier performance with respect to contaminant transport; other issues, such as the structural behavior of barriers, are considered only as factors that might influence contaminant migration. The barrier performance analysis can be referenced to a local scale (e.g., transport through a low-permeability wall) or a regional scale where the entire barrier system is considered. Except where noted, barrier systems are treated as porous media, under the assumptions generally applied in groundwater modeling (e.g., Bear, 1979), including the continuum hypothesis, conservation equations, validity of empirical flux laws (Darcy's Law, Fick s Law), and applicable constitutive relationships.

A considerable body of research has developed related to contaminant transport in porous media. The vast majority of this work, however, has been applied to systems in which advective transport is the primary concern. As pointed out by several researchers, however, under design conditions, molecular diffusion is expected to be much more significant than advective transport in barrier systems (e.g., Goodal and Quigley, 1977; Crooks and Quigley, 1984; Gray and Weber, 1984; Johnson et al., 1989). This distinction is important, and must be reflected in the modeling strategy applied.

Modeling of subsurface transport phenomena may be conducted at various levels of sophistication, ranging from simple analytical calculations to complex numerical simulations that stretch the limits of the fastest supercomputers. A premise underlying this section is that the current interest in long-term containment as a strategy for subsurface remediation will result in a demand for user-friendly models that can be applied by practitioners, including designers and regulators. The discussion is therefore focused on the development and application of relatively simplified models that are intended to balance the competing concerns of accuracy, conservatism, and computational requirements. This focus should not be construed as a de-emphasis of the need for continued development of sophisticated numerical models. On the contrary, complex numerical models provide an avenue for validating and exploring the limitations of simpler analytical models and, as discussed subsequently, they may be needed in practical applications where an assessment of the viability of a proposed remedial strategy is highly sensitive to factors that are not adequately represented by simpler models. Complex models are also valuable tools for exploring the relative importance of various transport processes.

Modeling applications considered in this section are primarily referenced to the consideration of engineered low hydraulic conductivity barriers installed to limit the migration of dissolved contaminants by advective and diffusive transport under saturated groundwater flow. Models for permeable reactive walls are conceptually similar and are discussed in this context. Much literature exists regarding landfill liners (e.g., Rowe et al., 1995) and a detailed treatment of this topic is therefore avoided. While modeling is also commonly used in the analysis of caps (e.g., Schroeder et al., 1994), the focus of such efforts is on the prediction of a water balance and infiltration rates, rather than contaminant migration, and as such this subject is not considered here.

10.1.2 Mathematical Model

The analysis of contaminant transport in groundwater systems is generally expressed in terms of aqueous phase concentrations. In the models discussed herein, the contaminant is assumed to be present only in the dissolved and sorbed phases. While the behavior of nonaqueous phase liquid (NAPL) contamination is of importance in the analysis of remedial systems, horizontal NAPL migration into and/or across a barrier system is unlikely to occur, due to the large pressure gradients required to overcome the capillary forces typical of fine-grained materials. For a discussion of conceptual and mathematical models for multiphase contaminant transport, the reader is referred to Abriola and Pinder (1985) and Cohen and Mercer (1993).

In the discussion that follows, the contaminant mass flux across a containment barrier is considered the dependent variable of concern. Contaminant mass flux is preferred, rather than contaminant concentration, because: (1) under diffusion-dominated conditions, mass transport is driven primarily by concentration gradients, rather than the actual concentration; (2) for some one-dimensional conceptual models of interest, the concentration of contaminant at the barrier/aquifer interface is assumed to be negligible in order to produce conservative estimates of mass flux; and (3) correspondence between simplified one-dimensional barrier models and global or field-scale models can be more readily established by comparing calculated mass fluxes at the barrier/aquifer interface.

The flux of a single mobile liquid phase contaminant of uniform density through a saturated porous media with unidirectional groundwater flow in the x-direction is given by

$$J = n\upsilon C - nD\frac{\partial C}{\partial x} \tag{10-1}$$

where J is the flux of contaminant mass per unit area at a specified time t, x is the direction of flow, n is the porosity of the porous media, C is the aqueous phase contaminant (or solute) concentration, v is the seepage velocity (average interstitial) of groundwater in the x direction, and D is the longitudinal dispersion coefficient.

A common approach is to evaluate the above expression for contaminant transport after solving the mass conservation equation for the solute, henceforth referred to as the advective-dispersive reactive equation (ADRE)

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x_i}\left(D_{ij}\frac{\partial C}{\partial x_j}\right) - \frac{\partial}{\partial x_j}\left(\upsilon_i C\right) - \frac{\rho_b}{n}\left(\frac{\partial S}{\partial t}\right)_{srp} + \frac{q_s}{n}C_s - \lambda_a C - \lambda_s\frac{\rho_b}{n}S \tag{10-2}$$

where i and j are subscripts that indicate the direction associated with the Cartesian coordinate system, x is a spatial coordinate, v is the seepage velocity,

D is the dispersion coefficient, q_s is a fluid sources/sink, C_s is the contaminant concentration for the source/sink, ρ_b is the media bulk (dry) density, S is the solute mass fraction in the sorbed phase, the *srp* represents a sorption reaction, λ_a is a first-order decay constant for the aqueous phase, and λ_s is a first-order decay constant for the solid phase.

The seepage velocity is related to the piezometric head gradient by Darcy's law [Eq. (10-4)]. The mass conservation equation for the bulk fluid [Eq. (10-3)] is given as

$$\frac{\partial}{\partial x_i}\left(K_{ij}\frac{\partial h}{\partial x_j}\right)+q_s=S_s\frac{\partial h}{\partial t} \tag{10-3}$$

$$v_i=-\frac{K_{ij}}{n}\frac{\partial h}{\partial x_j} \tag{10-4}$$

where K is the hydraulic conductivity, h is the piezometric head, and S_s is the media specific storage.

The dispersion tensor is commonly represented as consisting of two components

$$D_{ij}=D_{h_{ij}}+D* \tag{10-5}$$

where D_h represents (mechanical) hydrodynamic mixing and D^* is the effective molecular diffusion coefficient for the solute of interest.

In Eq. 10-5, D_h is considered to be a function of the seepage velocity and the pore system geometry of the porous media. The reader is referred to recent textbooks (e.g., Zheng and Bennett, 1995) for further discussion of dispersion in field-scale models. For barrier systems, the seepage velocity is expected to be very small, frequently negligible; therefore, molecular diffusion is considered to be the dominant transport process. The solute diffusivity in porous media (at the macroscopic scale) is commonly related to the solute diffusivity in liquid by a relationship similar to the following

$$D*=\frac{D_l}{\tau} \tag{10-6}$$

where D_l is the liquid diffusivity and τ is a tortuosity factor.

As presented here, the "tortuosity" factor is a lumped parameter that incorporates several mechanisms that inhibit the diffusion of a solute in porous media, including constrictions and extended flow paths related to the pore structure. Shackelford and Daniel (1991) referred to this lumped parameter as an "apparent" tortuosity factor (τ_a). The definition and interpretation of tortuosity and "effective" diffusion coefficients vary throughout the literature, and the reader should be careful when comparing results presented by different researchers. In particular, the definition of tortuosity used in this report does not reflect the fractional reduction of the cross-sectional area for diffusion represented by the media porosity or retardation due to sorption, as these factors are explicitly accounted for in Eq. 10-2. Note also that the tortuosity appears in the denominator of Eq. 10-6 and thus is understood to be greater than unity. An alternative definition of tortuosity, as a multiplicative factor less than unity, also has been used extensively (e.g., Shackelford and Daniel, 1991).

In Eq. 10-2, it is assumed that only two contaminant phases (sorbed and aqueous) are present. To close the system, an additional equation must be provided to represent the distribution of the contaminant between the dissolved and sorbed phases. The term "sorption" as used here, encompasses a variety of processes, including: hydrophobic partitioning into soil organic matter, ion exchange and other surface reactions, and diffusion into the solid matrix and/or immobile fluid. Most transport models rely on a phenomenological approach in which the solid phase is treated as a single compartment and an isotherm expression is used to define the relationship between the dissolved and sorbed phases at equilibrium. A commonly employed relationship is the linear isotherm

$$S = k_d C \tag{10-7}$$

where k_d is the solute distribution coefficient.

A nonlinear sorption equilibrium relationship may be described by the Freundlich isotherm

$$S = K_F C^{n_F} \tag{10-8}$$

where K_F and n_F are the empirical Freundlich parameters. The Freundlich isotherm is one of several commonly applied nonlinear isotherms (e.g., see Kinninburgh, 1986, for discussion of other approaches).

If the sorption process is fast, relative to the transport processes, the local equilibrium assumption is commonly employed. When a linear isotherm is appropriate, the governing ADRE reduces to

$$R_f \frac{\partial C}{\partial t} = \frac{\partial}{\partial x_i}\left(D_{ij}\frac{\partial C}{\partial x_j}\right) - \frac{\partial}{\partial x_j}\left(\upsilon_i C\right) + \frac{q_s}{n} C_s - \left(\lambda_a + \lambda_s \frac{\rho_b k_d}{n}\right) C \tag{10-9}$$

$$R_f = 1 + \left(\frac{\rho_b k_d}{n}\right)$$

where R_f is the retardation factor.

Although commonly employed because of its mathematical convenience, the linear local equilibrium assumption may not be appropriate for all systems. In particular, much research suggests that nonequilibrium sorption can play a significant role in contaminant transport and remediation when advection is the dominant transport process. Models for describing nonequilibrium sorption are numerous, and a detailed review is beyond the scope of this report (see discussions by Brusseau et al., 1989; Haggerty and Gorelick, 1995). In general, nonequilibrium models require identification of distinct solid phase compartments, the solute mass distribution among the compartments at equilibrium, and rate expressions for mass exchange across compartments. Equilibrium expressions may be linear or nonlinear. The form of the rate expression may vary depending on whether the governing theoretical model is based on assumptions relevant to mass transfer, diffusion, or chemical kinetics. In general, models that contain more compartments provide for more flexible data-fitting, but are difficult to parameterize independently and may be computationally demanding. For this reason, simpler "lumped" parameter models are often used in practice. As an illustration, a commonly applied two compartment, linear, first-order model is given as

$$\frac{dS}{dt} = \alpha \left(k_d C - S \right) \tag{10-10}$$

where α is the sorption rate coefficient.

The other reaction process explicitly represented in Eq. 10-2 is first-order decay. This reaction term may be interpreted to represent a number of processes including radioactive, abiotic, and biotic transformations. The decay constant is related to the contaminant "half-life" by

$$\lambda = \frac{\ln 2}{t_{50}} \tag{10-11}$$

where t_{50} is the half-life.

a. One-dimensional Geometry. Consideration of a containment barrier system in the context of the surrounding aquifer requires a complex model resolved in sufficient detail to represent the considerably different conditions that influence the contained area, the barrier itself, and the adjoining aquifer. For this reason it is often convenient to treat the barrier as a separate one-dimensional system, with the containment zone and surrounding aquifer represented as boundary conditions. If source/sink terms and reactions (other than decay and sorption) are neglected, the one-dimensional form of Eq 10-2 is given as

$$\frac{\partial C}{\partial t} = -\upsilon \frac{\partial C}{\partial x} + D\frac{\partial^2 C}{\partial x^2} - \frac{\rho_b}{n}\left(\frac{\partial S}{\partial t}\right)_{srp} - \lambda_a C - \lambda_s \frac{\rho_b}{n} S \tag{10-12}$$

where x is distance measured from the containment zone side of the barrier in the direction normal to the barrier-containment zone interface (the dimensional subscripts are henceforth dropped).

The seepage velocity is related to the hydraulic gradient across the barrier

$$\upsilon = -\frac{K_E \, \Delta h}{nL} \tag{10-13}$$

where K_E is the effective hydraulic conductivity in the direction normal to the plane of the barrier, L is the barrier thickness, and Δh is the difference in piezometric head between the inside and outside of the contained area, with a positive difference representing a larger head on the outside of the barrier.

According to classical dispersion theory (e.g., Bear, 1972) the dispersion coefficient in Eq. 10-12 can be represented as Eq. 10-14 where a_l, the longitudinal dispersivity, is a property of the barrier media. Although independent measurements of a barrier dispersivity have not been reported, the effect of the hydrodynamic mixing represented by this term is generally believed to be much less significant than diffusion for the low flow condition associated with barrier systems (e.g., Rowe et al, 1995).

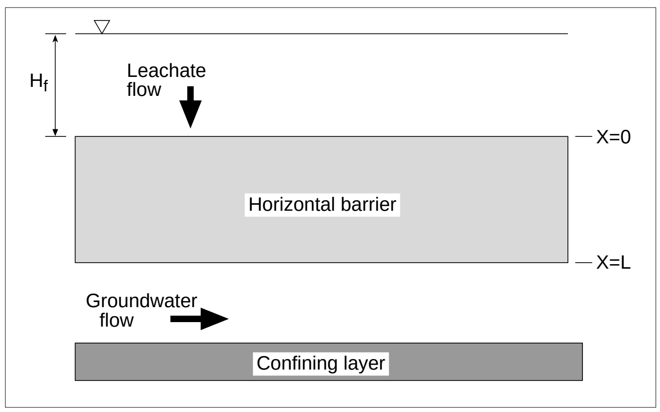
$$D = a_l \, \upsilon + D^* \tag{10-14}$$

Two representations of this conceptual model are shown in Fig. 10-1. Fig. 10-1a is based on a conceptual model described by Rowe et al. (1995) for a bottom liner, while Fig. 10-1b is based on a similar geometry applied to a vertical barrier. While similar in form, distinctions are noted between the two models:

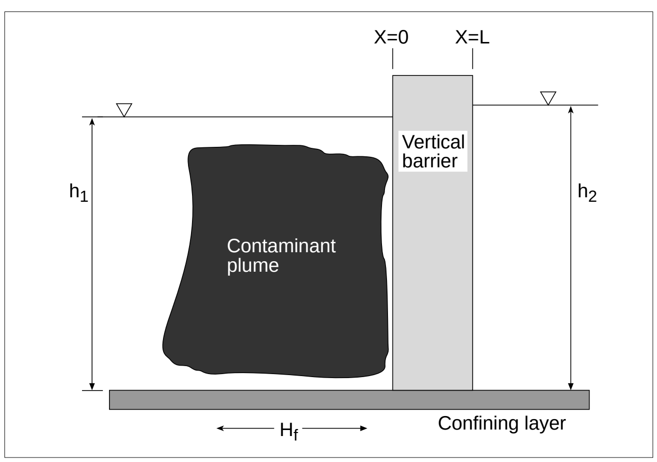
- Each model represents the contaminant zone as a completely mixed region of uniform concentration immediately adjacent to the barrier. For the commonly applied landfill model, the contaminant zone is understood to consist of leachate mounded above the bottom barrier, while for the vertical barrier, the contaminant zone is generally a dissolved contaminant plume of specified width, frequently assumed to extend over the full depth of the barrier.

- A number of conceptual models for the exit side of the barrier are possible, depending upon the boundary condition specified. For bottom barriers, one approach is to consider the aquifer as a finite, completely mixed region characterized by lateral groundwater flow. For vertical barriers, this assumption is unrealistic, although various degrees of contaminant "flushing" may be represented by adopting the mathematical form of the finite aquifer condition.

- Advective transport is driven by the piezometric head gradient across the barrier. Although this gradient is frequently assumed to be in the direction of the uncontaminated aquifer for bottom barriers, the gradient may be in either direction for both bottom and vertical barriers.



(a) bottom barrier



(b) vertical barrier wall

Figure 10-1 Schematic diagram of barrier configurations

Application of these conceptual models is discussed in more detail below.

b. Auxiliary Conditions. The form of the solution to the ADRE depends on the auxiliary boundary conditions specified. For engineered barriers, it is reasonable to assume that the barrier is free of contaminant at initial time, t = 0; i.e.,

$$C(x,0) = 0 \tag{10-15}$$

The form of the specified boundary conditions may greatly influence the solution, particularly for diffusion-dominated transport. For a one-dimensional representation of a barrier system, boundary conditions must be applied at the contaminated side of the barrier *(x=0),* denoted here as the barrier "entrance", and at the less contaminated side of the barrier *(x= L)*, denoted here as the barrier "exit". In general, boundary conditions are classified as: (1) first type or Dirichlet conditions, in which the value of the concentration is fixed at the boundary; (2) second type or Neumann, in which the gradient of the concentration at the boundary is specified; and (3) third type or mixed conditions, in which the concentration gradient at the boundary is a function of the concentration.

The discussion that follows deals with the specification of boundary conditions for field applications. For analysis of laboratory column experiments, different boundary conditions may apply (e.g., Shackelford, 1994a) and caution is urged in generalizing results between the laboratory and the field. For simplicity, solute decay is neglected. For a more detailed discussion of field scale boundary conditions the reader is referred to Rowe et al. (1995) and Rabideau et al. (1996).

<u>Entrance condition</u>. The most common and conservative condition applied to one-dimensional problems is the first type boundary condition; i.e.,

$$C(0,t) = C_o \tag{10-16}$$

where C_o is assumed to be a known constant concentration.

Eq. 10-16 is popular because it often leads to closed-form solutions of the ADRE. The fixed concentration condition is conservative in that it does not provide for the development of a concentration gradient within the contaminant zone, nor does it account for the reduction in contaminant concentration that could result from remedial actions or from contaminant mass transport into the barrier. Consideration of a time-dependent Dirichlet boundary condition usually increases the computational burden associated with the solution. Such an effort may be appropriate, however, if time-dependent data are available, as might be the case in the retrospective analysis of historical performance of a system.

An alternative conceptualization of the entrance boundary is the "finite mass" condition described by Rowe and Booker (1985) [mathematically, this condition may be considered a third type condition]. Physically, the condition can be interpreted to represent a known initial mass of contaminant (difficult to estimate) within a completely mixed contaminant zone. The uniform concentration in the contaminant zone is then reduced to account for transport into the barrier

$$C(0,t) = C_o - \frac{1}{H_F}\int_0^t \left[n\upsilon C(0,\zeta) - nD\frac{dC}{dx}(0,\zeta) \right] d\zeta \tag{10-17}$$

where H_F is the leachate height (if applied to a landfill liner) or the width of the contaminant zone normal to the orientation of a vertical barrier.

The application of the finite mass condition results in a smaller amount of mass transported across the barrier for a specified initial boundary concentration, and thus is less conservative, but possibly more realistic, than the constant concentration boundary condition. The sensitivity of the predicted flux to the width of the contaminant zone, H_F, is discussed by Rowe et al. (1995) for landfill systems and by Rabideau et al. (1996) for vertical walls. In general, when H_F is on the order of the barrier thickness, L, a significant reduction in flux is noted compared to the constant concentration condition. However, when H_F exceeds the barrier dimension by an order of magnitude or greater, there is little difference between the two conditions when diffusive transport dominates (Rabideau et al., 1996).

Other third-type entrance conditions have been proposed for application to laboratory test columns, including

$$\upsilon C_o = \upsilon C(0,t) - D\frac{dC}{dx}(0,t) \tag{10-18}$$

Sound arguments have been advanced by Parker and van Genuchten (1984b) for the applicability of Eq. 10-18 to advection-dominated laboratory conditions (based on the conservation of mass flux across the entrance boundary). However, this boundary condition produces unrealistic flux predictions when applied under the very low-flow conditions anticipated in the vicinity of low-permeability barriers (Rabideau et al., 1996). For permeable reactive walls expected to operate under advection-dominated conditions, either Eq. 10-18 or Eq. 10-16 may be appropriate, depending upon the dispersive properties of the treatment wall and, if comparisons are made with field data, the manner in which the field samples are collected (see discussion by Parker and van Genuchten, 1984b).

Exit condition. One-dimensional transport is sometimes modeled as a "semi-infinite" system in which the exit boundary condition is defined at $x = +\infty$, rather than at the barrier/aquifer interface, i.e.,

$$\frac{\partial C}{\partial x}(\infty, t) = 0 \tag{10-19}$$

Although this boundary condition implies that the transition at the barrier-surrounding aquifer interface does not influence contaminant transport within the barrier, it can facilitate the derivation of closed-form solutions to the ADRE. For laboratory studies of advection-dominated transport, Eq. 10-19 is commonly applied. However, for field conditions where diffusive transport is dominant within the barrier, it is likely that, in the adjoining aquifer outside the barrier, groundwater flow will result in more rapid removal of the contaminant exiting the barrier. This scenario may be represented by the third type boundary condition [proposed by Rowe and Booker (1985) for describing conditions at the base of a landfill liner],

$$C(L,t) = \int_0^t \left[\frac{n\upsilon C(L,\zeta)}{n_b h_b} - \frac{nD}{n_b h_b}\frac{dC}{dx}(L,\zeta) \right] d\zeta - \int_0^t \left[\frac{\upsilon_b C(L,\zeta)}{W} \right] d\zeta \tag{10-20}$$

where n_b is the porosity of the adjoining aquifer, h_b is the adjoining aquifer dimension normal to the barrier, v_b is the velocity of groundwater flow parallel to the barrier in the adjoining aquifer, and W is the width of the barrier.

For diffusion-dominated transport, Eq. 10-20 is more conservative than the semi-infinite condition, since it results in a higher concentration gradient, and consequently greater diffusive flux, at the barrier exit. In the limit, Eq. 10-20 reduces to a first-type exit condition

$$C(L,t) = 0 \tag{10-21}$$

For diffusion-dominated transport, Eq. 10-21 is the most conservative exit condition, since it implies instantaneous removal of the contaminant exiting the barrier. It is arguably the most appropriate condition for one-dimensional models of low hydraulic conductivity vertical barriers. This boundary condition is henceforth referred to as the "perfect flushing" condition.

For laboratory studies in which advective transport is predominant, a second type boundary condition has been proposed (e.g., Danckwerts, 1953)

$$\frac{\partial C}{\partial x}(L, t) = 0 \tag{10-22}$$

Eq. 10-22 is clearly inappropriate for diffusion-dominated transport in that it implies zero diffusive flux at the barrier exit. Surprisingly, in practice, laboratory results from advection-dominated studies are often extrapolated to the field without modification of boundary conditions (e.g., Acar and

Haider, 1990). The extrapolation of such results should be interpreted with caution.

c. Composite Barriers. The one-dimensional mathematical framework described above is based on the assumption of uniform barrier properties throughout the system, that, although difficult to verify in the field, is consistent with the design of most engineered systems. A notable exception to this assumption concerns the case of a composite barrier system, in which different materials are placed side by side in successive units, for the purpose of enhanced resistance to contaminant transport or improved reliability of the system. Conceptually, a composite barrier may be represented schematically as a layered system with each layer characterized by different properties. Composite systems could include the use of a geosynthetic liner on either or both sides of a vertical wall, or alternating horizontal layers of earthen and/or geosynthetic barriers with differing properties. The one-dimensional mathematical framework is applicable to the composite system as long as the differing barrier properties are properly represented. The solution of the governing equations for a composite system will, in general, require greater computational effort.

In computing the seepage velocity through a composite system, the following two modifications must be made to Eq. 10-13: (1) the effective barrier hydraulic conductivity, $K_{E,}$ is computed as the weighted harmonic mean of the layer hydraulic conductivities (weighted by layer thickness), and (2) the seepage velocities through the individual layers are computed using the respective layer porosity.

Some composite systems may be represented as an "equivalent" homogeneous system, particularly if the contaminant flux or concentration is desired at a single point in space (e.g., at the barrier exit). If the porosities, diffusion coefficients, and sorption properties of the individual layers are similar, then an equivalent one-dimensional barrier may be represented using the computed effective velocity. Similarly, if advection is negligible and layer porosities are similar, an effective diffusion coefficient may be computed using a weighted harmonic mean of the layer values. When both advection and diffusion are significant for a composite system, specification of an "equivalent" barrier may not be feasible, and more sophisticated solution techniques may be required.

10.1.3 Solutions

Since the governing processes and boundary conditions applicable to a barrier system will be site specific, some published solutions to the ADRE may be applicable. Based on the mathematical models presented and site-specific field conditions, a useful solution to the one-dimensional ADRE for application to contaminant transport through containment barriers should incorporate the following features:

- diffusive transport;
- advective transport in either direction;
- first-order decay; including, where appropriate, parent-daughter chains;
- linear or nonlinear equilibrium sorption;
- nonequilibrium sorption;
- a constant concentration, time-varying concentration, or finite mass entrance condition;
- a semi-infinite or flushing exit condition;
- spatial variation in parameters (i.e., multiple layers); and
- temporal variation in parameters (e.g., variation in hydraulic conductivity due to damage accumulation, consolidation of the barrier matrix, and/or accumulation of biomass, etc).

The extent to which the above features need to be represented in the ADRE dictates the solution technique and computational effort required. In general, solutions may be divided into three categories: analytical, semi-analytical, and numerical. The distinction between analytical and semianalytical is somewhat arbitrary, as some semi-analytical techniques (e.g., truncation of an infinite series) may be readily performed by hand calculation; while implementation of some closed-form solutions is sufficiently tedious that computers are routinely used. A thorough review of published solutions is beyond the scope of this report; however, a brief summary is provided with an emphasis on solutions that incorporate as many subsets of the above features as possible.

a. Analytical and Semi-analytical Solutions. Closed-form solutions or solutions requiring numerical integration, root finding, or approximation of an infinite series are included in this category. A large collection of solutions related to diffusion phenomena are presented by Carslaw and Jaeger (1959) and Crank (1975), including some that incorporate spatially and temporally varying diffusion parameters. Solutions including both advection and diffusion, first-order decay, and a constant concentration entrance boundary were summarized by Bear (1972) for the semi-infinite exit condition, and by Owen (1925) for a zero-concentration perfect-flushing exit. The latter, which is included in Carslaw and Jaeger (1959), is particularly appropriate for low hydraulic conductivity barriers and is reproduced in an appendix to this section, along with the appropriate form of Eq. 10-1 (the flux equation) for this solution.

Most published analytical solutions that include both advective and diffusive transport assume spatial and temporal uniformity of parameters. Linear equilibrium sorption may be accommodated through appropriate modification of parameters in the concentration-based solution utilizing a retardation factor. However, the calculation of contaminant flux using Eq. 10-1 is based on the nonretarded velocity and diffusion coefficient. An inward hydraulic gradient may be represented by specifying a negative velocity.

Semi-analytical solutions incorporating a constant concentration entrance condition, a semi-infinite exit condition, and nonequilibrium sorption according to the linear, first-order model (Eq. 10-10) are described by van Genuchten and Wieranga (1976). These and other concentration-based solutions have been implemented in a useful set of computer codes that include parameter estimation routines (Parker and van Genuchten, 1984).

Calculation of contaminant flux from concentration-based solutions may be accomplished by differentiation and algebraic manipulation according to Eq. 10-1; flux-based extensions for some of the solutions listed here are discussed by Rabideau et al. (1996).

b. Laplace-transformed Solutions. A powerful semi-analytical solution technique incorporates the numerical inversion of a Laplace-transformed solution, as described by Rowe and Booker (1985) and implemented in the POLLUTE code (Rowe and Booker, 1985). This approach has several advantages, including: (1) ability to incorporate both the finite mass entrance and the flushing exit boundary conditions; (2) ability to represent composite barrier systems; and (3) generally superior computational efficiency compared to numerical models. A drawback to this solution approach is its complexity; however, the POLLUTE code is available commercially, and although designed for application to horizontal barrier systems, it may also be applied to vertical barrier systems after reinterpretation of parameter definitions. Extension of the finite layer approach to incorporate nonequilibrium sorption is discussed by Rabideau and Khandelwal (1996).

c. Numerical Solutions. A detailed discussion of numerical solutions to the ADRE is beyond the scope of this report. In general, numerical solutions provide greater flexibility in the specification of boundary conditions and permit the incorporation of such processes as nonlinear/nonequilibrium reactions, as well as spatial variability of parameters within the barrier. The merits of various numerical schemes for advective-dispersive transport are discussed by Zheng and Bennett (1995). One dimensional solutions to the ADRE are generally accomplished by traditional Eulerian methods (e.g., finite difference or finite element). Common numerical difficulties related to sharp fronts are less likely to be significant in the analysis of low hydraulic conductivity barriers, due to the reduced role of advective transport.

The inclusion of additional nonlinear and/or nonequilibrium reaction terms in a numerical solution of the ADRE can be addressed through a variety of strategies; in particular, split-operator methods (e.g., Kaluarachchi and Morshed, 1995) have seen considerable application in simulating reactive transport in porous media. However, the use of split-operator techniques requires careful selection of the time step. While several recent studies have addressed the selection of time steps under advection dominated conditions (e.g., Miller and Rabideau, 1993; Morshed and Kaluarachchi, 1995), there are few, if any, published studies of this method applied to transport through low hydraulic conductivity barriers. Therefore, additional numerical experimentation is essential to determine appropriate temporal and spatial discretization for application of numerical models to reactive barrier systems.

10.2 STATE OF PRACTICE

10.2.1 Contaminant Transport in Low Hydraulic Conductivity Barriers

a. Field-scale Models. Incorporation of a containment system into a field-scale transport model requires either (1) appropriate spatial mesh refinement and the ability to represent regions with differing chemical and hydraulic properties, or (2) treatment of the contained contaminant zone as a time-dependent source region within the larger model domain. The initial step in analyzing the field problem is to simulate the groundwater flow system with special attention to the influence of the containment system. This is accomplished utilizing a groundwater flow model, for which data are more readily available and for which there may be greater confidence in computed results relative to transport models. Considerable insight can be developed from flow modeling prior to solute transport modeling.

Several commercial three-dimensional transport codes are currently used by practitioners, including the Princeton Transport Code (Babu and Pinder, 1984) and MT3D (Zheng, 1992). Incorporation of low hydraulic conductivity barriers in such models is straightforward from the standpoint of the groundwater flow system. A detailed treatment of the diffusion-dominated transport region associated with a containment barrier itself, however, would normally require use of a very fine discretization in the vicinity of barrier. The commonly employed multidimensional transport codes also typically rely upon a simplified representation of reaction processes (generally limited to first order decay and equilibrium sorption) to reduce the computational burden.

A considerable amount of recent research has addressed the development of numerical models for simulating transport at the field scale in conjunction with more sophisticated descriptions of reaction processes . While models of this nature are currently not routinely used, there are models available that incorporate three-dimensional heterogeneity and various reaction processes, often in conjunction with high-performance computers. For research applications or for projects where considerable resources are available to support simulation studies, such a detailed modeling approach is likely to provide the most satisfying results, assuming that adequate data are available for parameterization.

An alternative approach for field-scale modeling would be to treat the contained contaminant zone as a set of source nodes within the larger discretized domain. While such an implementation is possible with most transport codes, it may not be straightforward to represent the dynamic interaction between the barrier and the surrounding aquifer. A reasonable approach to this problem would be to couple a separate model for the barrier system with the field-scale model. The output from the barrier sub-model would provide the source term for the field-scale model, while the field-scale model would provide updated boundary condition information to the barrier sub-model. The dynamic coupling of the two models could be accomplished using either an iterative approach or a sequential solution over small time steps. The barrier submodel could be represented as a single or multiple one-dimensional model(s) of sufficient complexity to handle time-varying boundary conditions and the appropriate chemistry. While this coupled approach is conceptually straightforward, its implementation has not been reported.

b. One-dimensional Models. Except for landfill liners, published applications of contaminant transport models to low hydraulic conductivity barrier design and analysis has been limited. With appropriate treatment of boundary conditions, however, the one-dimensional framework may be applied to vertical barriers and/or emplaced bottom barriers. Closed-form and semi-analytical solutions for many of the applicable models are available and may be implemented in spreadsheets or simple programs (Shackelford, 1990). Commercial codes are also available for one-dimensional transport models (see review by van der Heijde and Elnawawy, 1993). The International Groundwater Modeling Center, Colorado School of Mines, serves as a clearinghouse for a variety of groundwater models.

The commercial POLLUTE code implements several solutions for the one-dimensional conceptual models discussed earlier (Rowe and Booker, 1985). Although structured for application to liner systems, POLLUTE may also be applied to the analysis of low hydraulic conductivity walls (with appropriate assumptions). Useful features of POLLUTE include: the ability to represent composite barriers; the inclusion of a parameter estimation algorithm that may be applied to the analysis of experimental data; and the recent addition of graphical user interface. Published applications of the POLLUTE code are numerous (see summary by Rowe et al., 1995). Further examples of the conceptual implementation of one-dimensional models and application to barrier systems are provided by Shackelford (1989) and Rabideau et al. (1996).

10.2.2 Groundwater Flow Modeling for Low Hydraulic Conductivity Barriers

Under some circumstances, insight into the potential effect of a barrier containment system on a proposed remedial action may be obtained by considering simply the influence of the barrier system on groundwater flow patterns. Such modeling is implemented using a variety of available numerical and semi-analytical groundwater flow models. For example, the widely used

MODFLOW code (McDonald and Harbaugh, 1988) is adaptable to the inclusion of low hydraulic conductivity barriers. MODFLOW has recently been expanded to include a module specifically designed to incorporate vertical barriers that are much thinner than the model discretization (Hsieh and Freckleton, 1993). For scenarios where NAPL contamination is present, a multiphase flow model may be required.

The following case studies illustrate the application of models to evaluate the impacts of low hydraulic conductivity barriers on remedial activities and the regional groundwater flow system.

a. Gilson Road, New Hampshire. The Gilson Road site, located in Nashua New Hampshire, was the site for the first cooperative agreement signed under the CERCLA program. As part of the remediation system a soil/bentonite cutoff wall was used to isolate a contaminant plume from the surrounding aquifer and nearby surface water. The cutoff wall extended approximately 4000 feet in length and up to 110 feet in depth. The Gilson Road project is significant in that it represents what is believed to be the first slurry wall designed for hazardous waste containment, and the project was therefore subjected to extensive quality assessment (Ayres et al., 1983; Schulze et al., 1984; Barvenik et al., 1985; Barvenik et al., 1986; Barvenik and Ayres, 1987).

Following the installation of the containment system in 1982, a series of post-construction verification tests was conducted, including a pumping test designed to determine the transmissivity of the fractured bedrock that formed the bottom of the contained area. An extraction well was located inside the contained area adjacent to the slurry wall and operated for 10 days. Drawdowns were measured at over 70 monitoring wells, and the data were used to calibrate a three-dimensional model of groundwater flow for the site. The calibrated model (Trescott, 1975) was then used to analyze the performance of the containment system, including sensitivity analyses to establish a range for the bulk average hydraulic conductivity of the slurry wall. Although the design hydraulic conductivity of the wall was 10^{-7} cm/s, the results of the numerical simulations suggested that the effective hydraulic conductivity of the wall could as high as 10^{-5} cm/s before significant change would be noted in the predicted hydraulic head distribution. The value of 10^{-5} cm/s was therefore adopted as an "upper bound" for the hydraulic conductivity of the installed cutoff wall, and used to obtain conservative estimates of leakage across the wall.

b. Lipari Landfill, New Jersey. At the Lipari landfill Superfund site, an areal groundwater flow model (Trescott et al., 1986) was applied to evaluate various remedial actions (Andersen et al., 1984; Mercer et al., 1987). The numerical model was used to simulate existing flow conditions at the site and provide initial conditions for a series of sensitivity analyses, including simulations designed to evaluate: (1) a slurry wall, (2) location of a drain system, (3) drain depth, and (4) a clay cap.

The Lipari Landfill is a 6-acre former gravel pit and industrial chemical dump located in Gloucester County near Pitman, New Jersey. On September 1, 1983, this site was placed at the top of the Superfund Priority Cleanup List (Russakoff, 1983). Model calibration for the Lipari site consisted of matching observed water-level data and surface water discharge data, and the numerical model was determined to be adequately calibrated so that it could be used for conceptual design purposes. After calibration, sensitivity simulations were performed to evaluate various proposed remedial actions. Numerous simulations were performed for various configurations of a slurry wall, drain system, and clay cap. In these simulations, the finite difference block representing the wall was assumed to have a hydraulic conductivity six orders of magnitude smaller than that of the aquifer. The drain was approximated by treating the finite-difference block containing the drain as a constant-head node. Full and partial caps were simulated by setting the recharge to zero in the finite-difference blocks representing the cap.

The results from model simulations were used to assess the predicted performance of various components of the remedial system. For example, the type of cap (full versus partial) was shown to have little effect on the predicted discharge to the drain system. The findings from the study were considered along with economic and engineering factors in the selection of the final remedial system.

c. Love Canal, New York. For the Love Canal site, a vertical cross section, variably-saturated flow model (Trescott et al., 1976) was applied to evaluate various proposed corrective actions (Cohen and Mercer, 1984; Mercer et al., 1987; Cohen et al., 1987). Approximately 22,000 tons of chemical wastes were buried in a 3-city block long, 18-m (60-ft) wide excavation (known as Love Canal) in Niagara Falls, New York between 1942 and 1953. In the mid-1970s, chemical seepage and odors were observed in the basements of many homes adjacent to the site. Remedial work conducted in 1978 and 1979 included the construction of a French drain completely around the site and a clay cover over the landfill. Following declaration of a state of emergency at the site on May 21, 1980 by President Carter, the U.S. EPA undertook a major study of the Love Canal environment (U. S. EPA, 1982). Based on the findings by EPA and others, the New York State Department of Environmental Conservation (NYSDEC) directed that further monitoring be undertaken and corrective actions be considered at the Love Canal hazardous waste site.

Additional proposed corrective actions included encapsulation of the site with a concrete cut-off wall and a synthetic cover. The impact of these proposed additional remedial actions on groundwater flow at the site were evaluated for the following cases: (1) no additional corrective actions, (2) synthetic cover only, (3) synthetic cover and cut-off wall, and (4) addition of a second French drain. The potential effectiveness of these measures were evaluated with regard to: (1) dewatering the shallow flow system, (2) French drain flux, and (3) reversal of vertical hydraulic gradients.

Calibration of the groundwater flow model was achieved in a series of steps, including: (1) simulating the general steady-state water-table pattern prior to installation of a French drain and clay cap, (2) matching the drain flux trend observed between August 1979 and June 1982, and (3) matching water levels measured in a series of piezometers in November 1982.

Effects of the proposed synthetic cover and concrete cut-off wall were evaluated by comparing 50-year simulations initiated at 1365 days (after drain and clay cap installation) in which: (1) no additional corrective actions were taken, (2) only a synthetic cover was included, and (3) both a cover and cut-off wall were included. Several additional simulations were conducted, including: (1) a sensitivity analysis of the shallow system hydraulic conductivity; (2) a simulation in which a second French drain was installed beneath the synthetic cover just inside the concrete wall; and (3) a simulation in which a second French drain was installed at the location of and in place of the concrete wall.

The synthetic cover was treated in the model as an impermeable barrier that produced a net recharge of zero. It was also assumed that runoff from the membrane was properly managed, causing no extra net recharge to the area beyond the cover. The concrete cut-off wall was assigned a hydraulic conductivity of 3×10^{-7} m/d (3.47×10^{-10} cm/s) for the entire 50-year simulation period. For the purpose of analyzing the hydrogeologic effects of the proposed corrective actions, it was assumed that the concrete cut-off wall and the synthetic cover achieved and maintained their design specifications. Given these assumptions, numerical simulations of existing and proposed corrective actions at Love Canal were used to assess various aspects of system performance, including the flux to the drain system under various conditions, the time to attain the maximum flux to the drain, and the amount of flow from various locations within the site that contributed to the drain discharge.

Construction of a synthetic membrane cover has been completed; however, on July 28, 1983, the New York State Department of Environmental Conservation announced that the cut-off wall would not be constructed as earlier planned (NYSDEC, 1983), citing observations from the modeling studies which suggested that the wall would have little effect on the volume of leachate collected and would impede collection of contaminants from outside the wall.

d. S-Area, New York. At the Occidental Chemical S-Area site, a multiphase flow model was applied to evaluate remedial actions (Faust, 1984; A.D. Little, Inc., 1983; Cohen et al., 1987), including flow reversal of chlorinated hydrocarbons present as a dense non-aqueous phase liquid (DNAPL). The S-Area Landfill is located on the southeast corner of Occidental Chemical Corporation's Buffalo Avenue Plant in Niagara Falls, New York.

Approximately 63,100 tons of chemical wastes were deposited at the site. A major concern at the landfill are discontinuities in an underlying confining bed that allowed DNAPL to enter a bedrock aquifer. Proposed actions included containment of liquids present in the landfill utilizing an integrated system of barrier walls, plugs, drains, and a cap designed to prevent off-site migration. The multiphase flow model was used to establish the magnitude of an upward hydraulic gradient needed to prevent downward migration of DNAPL.

10.2.3 Permeable Reactive Walls

For permeable reactive walls, modeling is needed to evaluate: (1) the degree of treatment attainable within the wall, and (2) the degree to which barriers are needed and their effectiveness in directing contaminated water through the treatment zone. Conceptual models for permeable reactive barriers that assume horizontal flow in the vicinity of the treatment zone may be modeled using the one-dimensional framework discussed previously. However, funnel and gate systems will concentrate and increase flow velocities in the treatment zone (Starr and Cherry, 1994). For most systems, the governing conceptual model is a plug-flow reactor with dispersion. Because advective transport is expected to dominate, the predicted effluent concentration distribution will be less sensitive to the form of the specified boundary conditions and reasonable accuracy should be obtainable by assuming idealized conditions that lead to closed-form solutions. Accurate representation of the reaction term is essential, since the objective of the model will be to determine the dimensions of a treatment wall necessary to achieve the desired reduction in contaminant levels (via the *in situ* reaction).

As the development of permeable reactive walls is a relatively new concept, published modeling studies of actual installations are not available. A likely application, however, is the consideration of a treatment wall in which organic contaminants undergo first-order decay in the presence of a reactive agent. Since the design time frame of such a system is long, a steady-state solution of the ADRE is suitable for design purposes. Under the assumption of negligible sorption, the solution for the effluent concentration, C, relative to the entrance concentration, C_o, is given by (e.g., Tchobanoglous and Schroeder, 1987)

$$\frac{C}{C_o}=\frac{2aP_e}{(1+a)^2\exp(0.5aP_e)-(1+a)^2\exp(-0.5aP_e)}$$
$$a=\sqrt{1+4\lambda_t P_e} \qquad (10\text{-}23)$$
$$P_e=\frac{vL}{D}$$

in which P_e denotes the Peclet number as defined, L is the length of the treatment zone, D is the dispersion coefficient for the permeable wall, and λ_t is the reaction decay rate constant. When the velocity, dispersion coefficient, and decay rate constant for the system are known, Eq. 10-23 may be used to adjust the length of the treatment zone, *L*, to achieve the desired reduction in contaminant concentration. Similar equations may be developed for a variety of reactive systems, such as sparge gates and sorption barriers. More complex reaction mechanisms may require the use of a numerical model and/or time-dependent analyses.

Regional flow system models may be used to verify that the contaminated groundwater passes through the treatment zone and to determine the advective velocity at the wall. Starr and Cherry (1994) demonstrate the use of a groundwater flow model to analyze the performance of a funnel-and-gate permeable reactive wall system. Because regional hydraulic gradients often exhibit seasonal variation in both magnitude and direction, a simulation model can provide useful insight into the trade-offs between system design parameters, e.g., funnel length, and the confidence in the ability of the system to maintain the desired routing of the contaminant plume.

An important concern, however, relates to site characterization and assumption of homogeneity in a groundwater flow model when applied to the analysis of a permeable reactive wall. Because advective transport dominates these systems, the influence of heterogeneity within the surrounding aquifer may be significant, particularly when the reactive wall is not keyed into an underlying confining layer. Teutsch and Schad (1995) describe an example where incorporation of observed heterogeneity into a three-dimensional flow model of a permeable wall suggested that bypassing of the barrier due to vertical flow could potentially occur. Such a condition could also occur if temporal changes in the pore system of the reactive wall (e.g., due to biomass growth and/or chemical precipitation) cause a reduction in hydraulic conductivity. For these reasons, careful consideration should be given to the use of more sophisticated numerical models, incorporating three dimensional resolution, density driven flows, and variations in hydraulic conductivity due to heterogeneities, for the analysis of permeable reactive walls.

10.2.4 Parameter Estimation

Application of mathematical models to barrier systems requires estimation of the numerous parameters contained in the governing equations described above. Parameter estimation for transport models can be accomplished by a number of strategies including: (1) empirical or theoretical correlation with other known system properties; (2) use of literature values; (3) contaminant/media specific laboratory experimentation; and (4) calibration using field data. For engineered barrier systems, calibration based on field data will generally not be an option and some combination of the other approaches will be required.

Values for the barrier parameters (e.g., barrier thickness, porosity, and density) can be specified as part of the design process and are therefore determined. When advection is included in the model, estimates of the hydraulic conductivity and piezometric head gradient are also required. Detailed discussions of hydraulic conductivity estimation are available elsewhere (e.g., Rumer and Ryan, 1995). The pre-construction hydraulic gradient can be determined through field measurement and may be subject to control by the designer through the specification of pumping rate(s). Groundwater flow modeling may be needed to predict hydraulic gradients after installation of the containment barrier system. The focus of the remaining discussion is on the estimation of diffusion coefficients, decay rate constants, and reaction parameters.

a. Liquid Diffusivity. As indicated in Eq. 10-5, the maximum expected value for a contaminant diffusion coefficient is the free liquid solution diffusivity. Literature values are available for many contaminants of interest (e.g., Cussler, 1988; Shackelford, 1991; Shackelford and Daniel, 1991). For other compounds, a variety of correlations are available relating diffusivities to molecular properties. For organic compounds, a review of applicable correlations is provided by Lyman et al. (1990). An example of a commonly applied relationship is that proposed by Wilke-Chang (1955),

$$D_l = \frac{(7.4x10^{-8})(\phi_W M_W)^{1/2} T}{\mu_W V_B^{0.6}} \tag{10-24}$$

where ϕ_W is the solution association constant, M_W is the molecular weight of water, μ_W is the dynamic molecular viscosity of water, T is absolute temperature, and V_B is the molar volume of the solute at boiling point.

For inorganic compounds, estimation of the liquid solution diffusivity is less straightforward, due to a variety of electrochemical factors as discussed by Shackelford et al. (1989). A reasonable starting point is the Stokes-Einstein equation (Reid et al., 1977).

$$D_l = \frac{RT}{6\mu_\omega r_B} \tag{10-25}$$

where R is the ideal gas constant and r_B is the solute molecular radius.

It is noted, however, that the Stokes-Einstein assumptions of a single dissolved species and infinite dilution are rarely appropriate for estimating effective diffusivities of inorganic species in subsurface systems.

b. Tortuosity. As noted in Eq. 10-6, the liquid solution diffusion coefficients for both organic and inorganic compounds are likely to be reduced by effects related to the structure of the porous media. Various terms have been introduced in the literature to describe these effects; however, in this section, porous media influences will be grouped into a single "tortuosity" parameter, greater than unity, that is used to scale the liquid diffusivity as expressed in Eq. 10-6. In doing this, tortuosity is understood to represent the combined effects of the tortuous flow pathways through the pore system geometry, the steric hindrance or anion exclusion effects that exclude solute molecules from the smaller pores, and the increases in fluid viscosity that may occur near surfaces. The tortuosity parameter discussed here does not include the effect of the reduced macroscopic cross-sectional area relevant to porous media, since this effect has been accounted for explicitly in Eq. 10-1 and Eq. 10-2.

The value of the tortuosity factor is difficult to determine directly and is usually inferred by comparison between a measured porous media diffusion coefficient for a conservative solute with that obtained in free liquid solution. A number of correlations have been developed that relate the tortuosity, τ, to the media porosity (e.g., Millington, 1959). These correlations can be expressed in the form

$$\tau = n^{\varepsilon} \tag{10-26}$$

where ε is an empirically-determined exponent, typically between -2 and -1/3, although there are few data for low hydraulic conductivity materials.

Application of a correlation similar to Eq. 10-26 generally is based on the assumption that the reduction of the effective diffusion rate is primarily related to the geometric tortuosity of the pore system geometry, rather than to various constrictions that might occur as the size of the solute molecule nears the average dimension of the pore system voids (see Mott and Weber, 1991a, for a discussion of these effects). In experiments conducted with a standard soil/bentonite slurry wall mixture, Mott and Weber (1991a) found that an exponent of -1/3 provided good agreement with experimental data, and attributed the majority of the tortuosity effect to the extended diffusion path associated with the pore structure. Theoretical considerations suggest that the applicability of Eq. 10-25 would likely produce a range of values for the exponent, μ, considering the wide range of barrier material types.

c. Experimental Determination of Diffusion Coefficients. The most straightforward means of estimating effective diffusion parameters is through laboratory experimentation. Shackelford (1991) and Rowe et al. (1995) have provided reviews of relevant experimental techniques. Laboratory measurements can provide an estimate of an "effective" diffusion coefficient that incorporates the influence of media tortuosity and chemical reactions. If experiments are performed with a conservative solute, the observed effective diffusion coefficient may be compared with the theoretical free liquid solution diffusivity to provide an estimate of the media tortuosity. This information may then be combined with independent experiments (e.g., sorption isotherms) to yield a complete set of parameters for reactive contaminants.

The selection of the appropriate experimental technique is dependent upon several factors including: (1) nature of the contaminants; (2) availability of skilled laboratory personnel; (3) ability to obtain independent estimates of sorption parameters; and (4) time frame available for measurement. For strongly sorbing contaminants, it may not be feasible to utilize methods that call for measurement of spatial or temporal breakthrough curves, because of the long time frames required (e.g., see Shackelford and Redmond, 1995). Also, volatile and biodegradable contaminants may require that special attention be given to experimental procedures and data analysis.

d. Sorption Parameters. Estimation of sorption equilibrium constants is usually accomplished through independent batch isotherm experiments. Much literature exists concerning the measurement of sorption isotherms, and discussion of this topic is therefore omitted here (e.g., U.S. EPA, 1991). Several correlations have been developed for hydrophobic organic compounds that relate k_d to the soil organic carbon fraction and the contaminant hydrophobicity, as expressed by the aqueous solubility or octanol-water partition coefficient (e.g., Fetter, 1994). Most published correlations for sorption equilibrium parameters are applicable only for dilute solutions of hydrophobic organics. Reliable methods for a priori estimation of sorption parameters for other organics and inorganics are not available, and laboratory experimentation is recommended.

Nonequilibrium sorption may be attributed to mechanisms such as chemical kinetics, physical diffusion, or both. Most model descriptions of nonequilibrium sorption are regarded as quasi-empirical, with the model coefficients determined by calibration using laboratory or field data. Examples of recent experimental studies can be found in Harmon and Roberts (1994) and Brusseau et al. (1989). Limited efforts have been made to develop and apply correlations for determining sorption rate coefficients (e.g., Brusseau and Rao, 1989a; Mott and Weber, 1992). Such correlations may not be widely adopted, due in part to the diversity of mechanisms influencing sorption rate behavior. If barrier performance is expected to be influenced by nonequilibrium sorption, laboratory kinetic experiments are recommended.

e. First-order Decay. In the ADRE, contaminant disappearance through first-order decay is generally understood to include the combined effects of a number of reaction processes, including biotransformation, hydrolysis, and radioactive decay. The most straightforward approach to the estimate of first-order decay constants is through batch experiments performed under conditions as similar as possible to those likely to be encountered in the field. A drawback to this approach is that for compounds that decay very slowly, the duration of experiments will be long and strict controls will be required to obtain accurate results.

An important class of reactions likely to influence aqueous solutions of organic compounds is hydrolysis, which is generally represented as a first-order process. The nature and analysis of hydrolysis reactions, including correlations for estimating rate coefficients, are discussed by Lyman et al. (1990). Published hydrolysis rate measurements are available for many compounds of interest (Mabey and Mill, 1978). A summary of measured radioactive decay rates for radionuclides is provided by Moody (1982). Observed rates for some other abiotic reaction processes that influence organic compounds are presented by Vogel et al. (1987). Few data are available for transport experiments conducted in low hydraulic conductivity porous media. Consequently, potential mass transfer effects on rate coefficients are not accounted for in the available measured data, and the use of published correlations and rate constants should be viewed as a last resort in situations where no media-specific experimental data are available. Furthermore, for radionuclides, inclusion of decay reactions without consideration of daughter-product formation may produce unrealistic results.

10.3 ASSESSMENT

10.3.1 Field-scale Models

The term "postaudit" has been used to describe studies in which the validity of model predictions is tested by comparing model output with field data collected long after the model simulations were first performed. There have been few published "postaudit" studies validating groundwater flow or contaminant transport models. In a review of five published postaudits, Anderson and Woessner (1992) noted that, in all cases, the models gave poor predictions. The primary explanations offered for these poor predictions were (1) an inadequate representation of the subsurface stratigraphy resulting in an inadequate representation of the hydraulic conductivity (e.g., too few layers, incorrect boundary conditions), and (2) an inadequate representation of time-varying operating conditions (e.g., pumping cycles, recharge, etc.).

Since engineered subsurface barrier systems are generally designed on the basis of parameter conformance to a regulatory standard (e.g., $K < 10^{-7}$ cm/s), it is probable that the reduced role of advection will result in minimal errors due to misjudgments in regional groundwater flow boundaries and stresses. On the other hand, the engineered component of these systems introduces added uncertainty due to the potential for irregularities introduced during the construction process, and the possible time-evolution of barrier properties. The latter are considered in a subsequent section.

The rationale supporting the use of sophisticated numerical models in the analysis and design of remedial systems is based on their ability to produce detailed and accurate predictions, assuming that the necessary parameters and contaminant distributions are known. The merits of this rationale depend on the objectives of the modeling exercise. As an example, consider the use of site-specific numerical models to predict cleanup times for pump-and-treat systems. Accurate predictions of cleanup times require: (1) delineation of the contaminant distribution with respect to the phases present and spatial variation; (2) knowledge of the spatial structure of hydraulic conductivity; and (3) knowledge of rate coefficients for reactions (e.g., dissolution, desorption, etc.). The determination of these factors is often difficult and may not be feasible for some applications. For this reason, while solute transport models may be appropriate for some remedial applications (e.g., delineation of extraction well capture zones), predictions for cleanup times and long-term system performance are considered to be highly uncertain.

Modeling of containment barrier system performance is subject to considerations that are both similar to and distinct from those relevant to pump-and-treat. For low hydraulic conductivity barriers, if the one-dimensional framework is adopted, concerns related to characterization of contaminant distribution and aquifer heterogeneity become less important. The integrity of the constructed barrier is a prime concern, however, as is the role of reactions that influence transport within the barrier. Similarly, for permeable barrier walls, the initial distribution of contaminant within the plume is less important than the accurate representation of system hydraulics and the reactions occurring within the treatment zone, as well as any long-term effects such as clogging and/or reductions in reaction rates. Therefore, assessment of model output needs to be referenced to the site-specific situation and the objectives of the modeling exercise.

10.3.2 One-dimensional Models for Low Hydraulic Conductivity Barriers

Ultimately, application of the one-dimensional modeling should be decided on the basis of whether model output can give an accurate prediction of contaminant transport under conditions consistent with those likely to found in the field. Because the relevant field data generally are not available, the decision to apply a one-dimensional model is likely to be based on (1) the ability of models to describe the results of well-controlled laboratory studies, (2) the consistency of model predictions with field observations of transport in clay landfill liners, (3) insights into the behavior of one-dimensional models obtained through sensitivity analysis, and (4) a critical assessment of the underlying assumptions built into the modeling framework.

a. Agreement with Laboratory Studies. Although there is a large body of literature describing laboratory studies of advection-dominated transport in porous media, few studies have been conducted using low hydraulic conductivity materials. Shackelford (1991) and Rowe et al. (1995) provide reviews of experimental results related to the measurement of diffusion coefficients in various soils. However, experiments of this nature do not provide a good basis for evaluating the ability of models to accurately predict contaminant transport under diffusion-dominated conditions. There have been few studies designed to directly address this issue. The work of Mott and Weber (1991a, 1991b, 1992) is a notable exception in that independent quasi-steady-state and transient experiments were used to test the ability of diffusion models to predict contaminant distributions. While some column experiments have been performed using barrier materials (e.g., Acar and Haider, 1990; Bierck and Chen, 1994), few experiments have addressed the low-flow conditions likely to be encountered in the field [the recent work of Shackelford and Redmond (1995) is an exception].

Laboratory soil-column experiments are typically conducted using a subset of the boundary conditions applicable in the field. A more general set of experiments has been described by Hensley and Schofield (1991), employing a centrifuge and a physical model designed to mimic the third type boundary conditions used in the POLLUTE model (finite mass entrance, flushing exit). Based on their studies and preliminary results of ongoing research sponsored by DuPont, Inc. (Khandelwahl, 1995, unpublished data), the following observations are offered:

- measured contaminant concentration profiles or breakthrough curves have generally been consistent with the hypothesis that diffusive transport dominates under the low flow conditions associated with barrier materials;
- observed effective diffusion coefficients for organic compounds are generally consistent with theoretical predictions when barrier materials are not strongly sorbing;
- difficulties in predicting breakthrough curves for inorganics have been noted and attributed to a variety of chemical processes not well represented by the local equilibrium assumption;
- for some barrier materials, particularly those amended with organic matter to enhance sorption, the retardation factor approach may be inadequate due to isotherm nonlinearity and possible nonequilibrium effects; and
- good agreement has been noted between the centrifuge experiments and predictions generated by the POLLUTE code.

b. Field Studies of Landfills. There have been several investigations of contaminant distributions within compacted clay landfill liners over time (e.g., Johnson et al., 1989; studies summarized by Rowe et al., 1995). While these studies do not carry the weight of a postaudit, they do provide insight into the applicability of advective-diffusive transport theory for the analysis of transport in low hydraulic conductivity materials. Based on the findings from these studies, the following observations are offered:

- as expected, the measured contaminant profiles were consistent with the hypothesis of diffusion-dominated transport;
- measured profiles for nonreactive substances could be reasonably well predicted by mathematical models using values for diffusion coefficients obtained in the laboratory;

- the observed permeation of various reactive organic and inorganic species varied, depending on the compound, but in general did not exceed the permeation of chloride;
- independent predictions of contaminant distributions based on hypothesized reaction mechanisms were in general unsuccessful; and
- no appreciable increases in hydraulic conductivity with time were noted for the clay liners associated with municipal solid waste (MSW) landfills; in some cases, a gradual decrease in hydraulic conductivity was noted.

Although the ability to describe reactive transport was limited for these field studies, the systems under consideration present a particularly difficult challenge due to the complex chemical interactions between MSW leachate and compacted clay, as well as uncertain temporal variability in leachate conditions. Systems that involve fewer contaminants and/or engineered materials may be more amenable to predictive modeling of reactive transport. In general, however, the results of these studies suggest that for some systems, modeling of nonreactive solute transport may be sufficiently conservative, making more detailed simulations of reactive contaminant transport unnecessary.

c. Sensitivity Analysis. While the one-dimensional framework may be suitable for design and analysis of many barrier systems, the required computations may not be trivial in some cases. In the application of one-dimensional models, the following factors must be considered:

- the selection of processes to be included (e.g., advection, decay, sorption, etc.);
- the time frame of concern (i.e., steady-state versus transient, design service lifetime);
- the governing boundary conditions; and
- parameter estimation.

Each of the above factors may be considered at various levels of complexity. A decision regarding the complexity of model formulation should be referenced to the modeling objective and/or performance criteria associated with the particular barrier application. A common approach to evaluating the relative importance of various transport processes is to perform sensitivity analyses using a model that incorporates all of the relevant mechanisms. In the discussion that follows, unless indicated otherwise, the results of computer simulations are taken from recent work sponsored by DuPont, Inc. (Rabideau et al., 1996; Rabideau and Khandelwal, 1966). Modeling results are expressed in terms of dimensionless groupings referenced to a "characteristic diffusion time" $T = L^2/D$ which is on the order of 30 to 300 years for a typical slurry wall (L = 1 meter, $K = 10^{-7}$ cm/s, $D = 10^{-6}$ to 10^{-5} cm^2/s). Note that, due to the negligible role played by mechanical dispersion, for these simulations, the dispersion coefficient (D) is equivalent to the effective molecular diffusion coefficient (D^*).

A conservative estimate of the steady-state contaminant flux, J, through a barrier wall may be obtained by neglecting advection and reaction and assuming a "flushing" boundary condition; resulting in the following expression

$$J = \frac{nDC_o}{L} \tag{10-27}$$

where D is the effective diffusion coefficient as defined in Eq. 10-6, C_o is the contaminant concentration at the barrier entrance, and L is the barrier thickness.

Eq. 10-27 ignores the early-time transient behavior of the system, and does not consider advection, sorption and first-order decay. Incorporation of these processes in a time-dependent solution, while retaining the conservative boundary conditions, results in the more complicated expression for instantaneous flux (see appendix to this section). The model equations described in the appendix to this section provide a more realistic representation of system dynamics and, although more cumbersome than many common solutions of the ADRE, may be evaluated using a spreadsheet or simple computer program. An example of an application of a barrier design procedure utilizing a transient solution to the ADRE is given by Shackelford (1990).

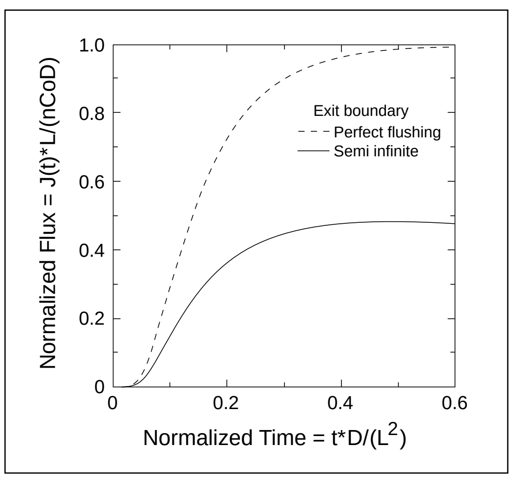


Figure 10-2 Pure diffusive flux predictions for different exit boundary conditions

Boundary Conditions. When advective transport in the direction of the chemical gradient is negligible, Eq. 10-27 provides a conservative estimate of contaminant flux because of the form of the boundary conditions and the assumption of negligible reactions. The effect of assuming a flushing type boundary, as opposed to a semi-infinite boundary, is shown in Fig. 10-2. Differences are noted in the shape of the flux profiles as well as the peak flux values. The most significant difference between the two model applications is that the flushing boundary results in a fairly rapid rise to the steady-state condition given by Eq. 10-27, attaining 90% of the steady-state value in less than half the characteristic diffusion time, while the semi-infinite boundary condition produces a slow rise to a smaller peak, followed by a slow decrease in the gradient-driven diffusive flux as the contaminant accumulates in the vicinity of the barrier exit. While some portions of a containment system may be reasonably well-represented by the semi-infinite condition, it is likely that the flushing condition is more realistic, and is clearly more conservative.

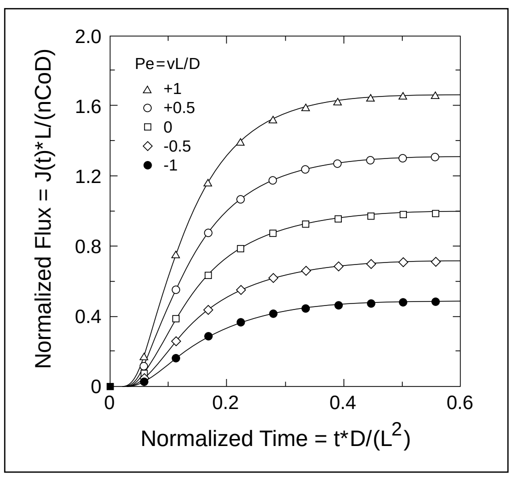


Figure 10-3 Flux predictions with varying amounts of advection

Advection. When a positive (outward) hydraulic gradient exists, the steady-state diffusive flux, as given by Eq. 10-27, will no longer provide a conservative estimate of contaminant flux. Alternatively, if pumping is implemented on the inside of a contained area causing an inward hydraulic gradient, Eq. 10-27 may produce excessively conservative estimates of contaminant flux, since advective transport will be oppositely directed to diffusive transport. The relative importance of advection may be represented by the macroscopic Peclet number $Pe = vL/D$, which may be considered as the ratio of the advective transport to the diffusive transport. The results of sensitivity analyses are shown in Fig. 10-3, with a positive *Pe* corresponding to an outward hydraulic gradient, and a negative Pe corresponding to an inward gradient. Note that even relatively small changes in *Pe* result in significant changes in the predicted steady state flux, due primarily to the role of advective transport (i.e., hydrodynamic dispersion is small relative to molecular diffusion). Based on the definition of seepage velocity expressed in Eq. 10-13, it is seen that *Pe* is directly proportional to the hydraulic gradient across the barrier. For the vertical wall parameters used in the simulations ($K = 10^{-7}$ cm/s, $D = 10^{-5}$ cm^2/s, $n = 0.4$, $L = 100$ cm), $Pe/i = 2.5$. Thus, the results shown in Fig. 10-3 suggest that advection might be reasonably neglected under a typical regional gradient of $i = +0.05$, while a large induced inward hydraulic gradient $i = -1.0$ would produce a significant reduction in the predicted contaminant flux through the barrier. Similar results have been shown by Gray and Weber (1984).

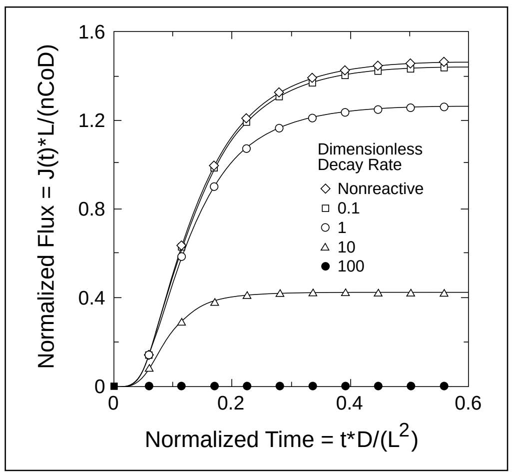


Figure 10-4 Flux predictions for various contaminant decay-type reactions

Reactive Transport: First-Order Decay. Since biotic and abiotic reactions tend to reduce the rate of contaminant transport through a barrier, their neglect generally produces conservative estimates for contaminant flux, assuming suitable advection and diffusion parameters have been specified. However, when a reactive barrier is specifically being considered as a means of reaching performance criteria, barrier chemistry is important and needs to be included in the analysis of contaminant flux.

A number of processes may result in the transformation of contaminants, including radioactive decay, biodegradation, and abiotic reactions. Inclusion of a first-order decay term in the ADRE is a convenient means of representing these processes, although often neglected in practice. Because of the long residence times for contaminants in barrier walls or floors, the effect of decay-type reactions may be significant, as illustrated in Fig. 10-4. In general, when the value of the dimensionless rate coefficient $\lambda L^2/D$ exceeds 1.0 (half life of approximately 20 years for typical slurry wall parameters: $D = 10^{-5}$ cm^2/s, $L = 100$ cm), there can be significant reductions compared to steady-state contaminant flux. Decay processes should be included in models (if they exist) when design calculations, based on neglecting decay, do not meet performance criteria or when greater accuracy in model predictions is desired. It can be seen that as the dimensionless rate constant approaches 100, reactions within the barrier wall may be sufficient to negate the outward diffusive transport, virtually eliminating contaminant flux.

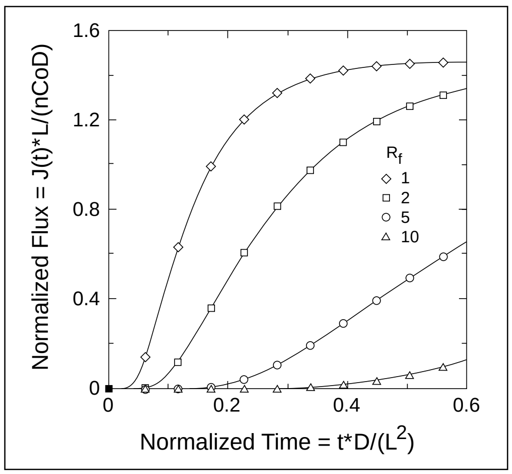


Figure 10-5 Flux predictions for various contaminant retardation factors ($Pe = 0.1$)

Reactive Transport: Sorption. In the absence of reactive decay processes, the effect of equilibrium (i.e., rapid) sorption in a barrier represented by a flushing exit condition is to lengthen the time required to reach steady-state. However, the magnitude of the peak flux may not be significantly reduced, as illustrated in Fig. 10-5. If reactive decay processes are present, sorption will still increase the contaminant residence time in the barrier and result in a further reduction of contaminant flux.

A difficulty encountered in the modeling of reactive transport occurs when there is nonlinearity in the sorption isotherm (e.g., Brusseau and Rao, 1989b; Weber et al., 1991). Incorporation of a nonlinear isotherm in Eq. 10-2 generally necessitates the use of a numerical solution technique. To facilitate the use of available analytical solutions, the nonlinear isotherm may be linearized through one of several approaches, as discussed by Brusseau and Rao (1989b). If the form of the governing nonlinear isotherm is known, a simple strategy is to equate the linear and nonlinear expressions at some reference concentration, typically the entrance boundary concentration, Co. The distribution coefficient, k_d, is then computed in terms of the nonlinear isotherm parameters and the reference concentration. Linearizing an isotherm in this fashion generally results in conservative predictions of solute transport (i.e., longer times for contaminant breakthrough) if the governing isotherm expression is concave in shape (e.g., Freundlich exponent less than 1.0), as is common for many contaminants. However, the distribution of contaminants within the barrier generally may not be accurately portrayed when a nonlinear isotherm is linearized (Shackelford, 1993).

The specification of the appropriate reaction model is particularly important when organic matter is added to barrier materials for the purpose of enhancing sorption. In such cases, a sorption isotherm can be determined from batch experiments and incorporated in the appropriate ADRE. However, if the sorption process is slow and local equilibrium was not achieved, modeling predictions will not be conservative. In Fig. 10-6, the results of sensitivity analyses are shown for a system described by the two-compartment first-order sorption model (Eq. 10-8). As the sorption rate constant is increased, the curves move from a shape consistent with negligible sorption to one consistent with the local equilibrium assumption. Significant deviations from the equilibrium model are noted when the dimensionless sorption rate constant $\alpha L^2/D$ is below approximately 100, a result consistent with similar analyses conducted for advection-dominated transport (e.g., Jennings and Kirkner, 1984). These results reveal that the local equilibrium assumption may be inappropriate for some sorptive barrier materials.

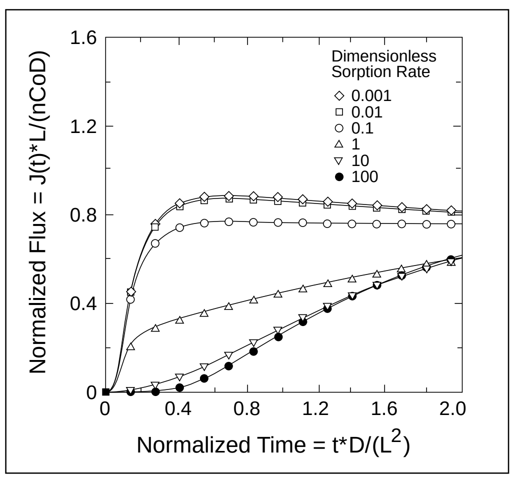


Figure 10-6 Flux predictions for various sorption rate constants (semi-infinite exit boundary, $R_f = 10$, $Pe = 0.1$)

Few data are available to evaluate the validity of the local equilibrium assumption for low permeability materials. In the studies of Mott and Weber (1992), good agreement with measured contaminant profiles was noted using a retarded diffusion model, combined with independent measurements of sorption and diffusion parameters for a nonamended soil/bentonite (SB) mixture. However, when flyash was added to the mixture to enhance its sorption capacity, models based on independent isotherm experiments and the equilibrium assumption significantly underpredicted the penetration of the contaminant by diffusion into the barrier material. Possible nonequilibrium effects were postulated as an explanation for the observed deviations.

Bierck and Chang (1994) added granular organic carbon to promote enhance sorption in soil/bentonite mixes. However, experiments conducted with a hydrophobic organic contaminant produced less retardation than would be consistent with equilibrium sorption based on independent isotherm experiments (Rabideau and Khandelwal, 1996).

The results described above suggest that nonequilibrium effects may influence the sorption of hydrophobic organic compounds in amended soil-bentonite material. For these systems, application of the local equilibrium assumption may be inappropriate. Development of a model that includes rate limited sorption is recommended (e.g., Rabideau and Khandelwal, 1996).

d. Parameter Estimation for Barrier Design. For permeable reactive barriers, it is expected that detailed laboratory and pilot scale studies will be required to support design and analysis. For low hydraulic conductivity barriers, however, the degree of effort needed for parameter estimation will depend upon the modeling objectives and the acceptable level of conservatism. Laboratory studies for evaluating diffusion and sorption parameters are, by nature, time-consuming and difficult. Because the observed range for diffusion coefficients is small for saturated fine-grained soils (10^{-6} cm^2/s < D^* < 10^{-5} cm^2/s) relative to other sources of uncertainty, for design applications it may be adequate to use literature values, particularly if reaction processes are neglected in the analysis. A conservative value for D^* is 10^{-5} cm2/s, which is towards the high end of the range for observed free liquid solution diffusivities. For more accurate results, the methods described in the previous section should be applied; it is expected that laboratory experiments with the contaminant/barrier materials of interest should produce the most reliable results. However, to account for mass transfer effects during contaminant migration through barrier materials may require long-term testing under low flow rates (Shackelford and Redmond, 1995).

Although the neglect of first-order decay in transport modeling results in conservative predictions of contaminant flux, such an approach may not provide sufficient accuracy, particularly for radionuclides. As discussed previously, the determination of first-order decay parameters in general is not straightforward and is best accomplished by laboratory experimentation under conditions as similar as possible to the expected field conditions. Published studies of transport and decay in low hydraulic conductivity media are not readily available, and it is difficult to generalize about the appropriateness of using published values in transport simulations.

In the absence of decay, the effect of sorption is to retard migration but not necessarily reduce the peak flux. Thus, for mildly sorbing barrier materials, a retardation factor of 1.0 is conservative and not unreasonable. If more accurate results are required, the methods described in the previous section are applicable, with direct laboratory experiments recommended if resources permit.

10.3.3 Deviations from Model Assumptions

The contaminant transport models discussed in this report are based on the applicability of traditional advective-dispersive-reactive transport theory. For low-permeability materials, however, some deviations from fundamental assumptions commonly applied in the analysis of porous media may be expected. Several concerns are mentioned briefly here: the applicability of Darcy's Law, the assumptions of homogeneity and saturated conditions, and the role of "coupled flow" processes. Although Darcy' s law does cease to be valid for nonlinear laminar flows or for turbulent flow in porous media, neither of these flow conditions can be expected to occur in low hydraulic conductivity fine grained materials (Bear, 1979). In references cited by de Marsily (1986), observations of a hydraulic conductivity "threshold" and a nonlinear relationship between observed flow rates and low hydraulic gradients for compacted clays are discussed. However, there is little unambiguous or incontrovertible evidence to negate the validity of Darcy's law at low hydraulic gradients in fine-grained materials (Neuzil, 1986).

The use of a one-dimensional modeling framework implies that the properties of the barrier are invariant in the other directions. Although this may be an appropriate modeling framework, the existence of a uniform hydraulic conductivity throughout a low-permeability barrier has not been substantiated by field data. Studies of hydraulic conductivity heterogeneity for other subsurface remedial systems have suggested that even relatively small variations in hydraulic conductivity may significantly impact the predicted system performance (Rabideau and Miller, 1994). While numerical modeling tools exist for incorporating heterogeneity, such tools require greater user sophistication, compared to one-dimensional models. The ability to construct a homogeneous low hydraulic conductivity barrier and the effects of heterogeneity on transport in barriers are currently not well understood and deserve further study.

Vertical barrier walls are commonly analyzed assuming fully saturated conditions. For an unconfined aquifer, however, seasonal water table fluctuations may produce dewatering in a top portion of the wall, with two implications: (1) wet/dry cycling may affect barrier properties and performance, and (2) vapor phase diffusion of contaminant may occur. Because vapor phase diffusion coefficients are generally much higher than liquid diffusivities, contaminant transport may be greater in the unsaturated portion of the barrier wall. While some concerns have been noted with regards to maintaining saturated conditions in earthen walls (Rumer and Ryan, 1995), the potential impacts of water-table fluctuations on long-term barrier performance need to be better understood.

At the low flow rates expected for low hydraulic conductivity barrier systems, "coupled flow" processes may play a significant role, as demonstrated in recent efforts to develop remedial technologies suitable for fine-grained soils (Mitchell, 1992). For example, osmotic and electrokinetic transport may occur in response to both hydraulic and chemical gradients. The extent to which these processes occur in low-permeability barrier systems is relatively unknown, and further research is needed to delineate the conditions under which these processes should be incorporated into the ADRE.

10.3.4 Long-Term Performance

Generally, groundwater flow models for contaminant transport in porous media assume that media properties remain constant with time. Although some models do attempt to simulate changes in flow and transport conditions due to the presence or removal of contaminant (e.g., NAPL dissolution, biofouling, etc.), certain fundamental media properties, e.g., total porosity and intrinsic permeability, are usually assumed to be non-varying. Engineered barriers, however, are of a fundamentally different nature, and it is reasonable to assume that barrier properties are subject to change over time. This applies both to low hydraulic conductivity barriers and permeable reactive walls. Since the design service lifetime for containment systems may range from decades to centuries, more consideration needs to be given to the possible evolution of barrier properties during this period.

Engineered systems have been analyzed using risk-based or reliability approaches for predicting the time-dependent probability of failure for an engineered system (Freeze et al., 1990). The Weibul probability model has been commonly used (Inyang, 1995). Certain questions arise when using the reliability approach, including: (1) "What constitutes "failure" of a barrier?"; (2) "How can failure probabilities and the barrier properties causing failure be quantified?" and assuming that these questions can be answered, (3) "What combination of stochastic and deterministic modeling techniques might be used to produce a realistic scenario for contaminant transport over the lifetime of a barrier?" For example, using Monte Carlo techniques (Zheng and Bennett, 1995), output from repeated simulations could be combined to produce a probability distribution for contaminant flux that incorporates the failure information.

One approach to modeling a "damaged" low hydraulic conductivity barrier would be to consider it as a fractured, rather than a porous, medium. A number of strategies have been proposed for modeling transport in fractured media (Pinder et al., 1993). Simplified equations for describing a regularly fractured medium have been described by Rowe et al. (1995), and a multilayered system that includes fractured zones can be modeled with the POLLUTE code.

Probabilistic modeling approaches, as described above, have not been employed in the analysis of containment barriers, and the current knowledge regarding failure mechanisms and model parameters is insufficient to support the use of this approach. Nevertheless, it may be equally inappropriate to rely on deterministic models to predict the performance of barriers over their design lifetimes, particularly when they do not incorporate the possibility of barrier failure due to damage accumulation. The development of appropriate long-term modeling frameworks and identifying the supporting data needed should be a research priority.

Potential causes of long-term damage to barriers include: freeze-thaw cycling, wet-dry cycling, thermal effects, chemical action of leachates, weathering, earthquakes, land subsidence, and, in the case of horizontal barriers, bearing capacity failure. Thermal cracking, as related to the disposal of radioactive materials, is a primary concern (see Section 7). In experimental studies conducted by Britto et al. (1989), burial of high-temperature wastes in sediments caused fractures that led to increased contaminant transport. Although high-temperature materials will not generally be placed in the immediate vicinity of engineered barriers, additional experimental work is still needed to delineate scenarios under which heat-related damage may occur.

The chemical compatibility of barrier materials with the permeating solution is another concern (Shackelford, 1994b). Experimental studies have identified strong acids and bases, strong solutions of electrolytes, alcohols, and nonaqueous phase liquid (NAPL) organics as contaminants of concern, in this regard. The chemical compatibility of a leachate with a barrier material can be investigated in the laboratory by passing several pore volumes of the leachate through a permeameter. Manassero and Shackelford (1994) proposed a "compatibility index" for identifying potential compatibility problems. These approaches have tended to identify compatibility problems in terms of a "yes/no" evaluation. From a modeling standpoint, however, quantitative functional relationships are needed relating *in situ* hydraulic conductivity to chemical concentrations, if long-term chemical damage is to be incorporated into ADRE models. Further research is needed in this area, particularly for inorganic compounds. In the case of NAPLs, it is unlikely that hydraulic gradients would be large enough to produce significant NAPL penetration into an engineered barrier.

In summary, while there is an appreciation of the mechanisms that may produce damage to engineered barriers, there are no mathematical models presently available that incorporate these mechanisms. Yet, if containment is to be accepted as a viable option in remediation strategies, new models must be developed that predict long-term behavior by including mechanisms that cause time dependent damage accumulation. In the near-term, hydraulic control (i.e., pumping to create inward gradients) may represent the best strategy for addressing the uncertainties associated with damage accumulation to engineered barriers. In conjunction with physical barriers, such control should be relatively cost-effective, compared to conventional pump-and-treat systems.

10.4 NEEDS

10.4.1 Performance Criteria for Low Hydraulic Conductivity Barriers

Although clean-up criteria in environmental remediation are frequently concentration-based standards, in the case of engineered containment barriers, flux-based standards (that provide acceptable risk levels external to the containment system) would be more appropriate. This statement is based on the following:

- Since transport in low hydraulic conductivity barriers is likely to be diffusion-dominated, attainment of a low concentration at the barrier-aquifer interface does not, by itself, ensure that contaminant flux out of the containment barrier system is insignificant. Over time, a significant amount of contaminant mass may be transported across a barrier due to diffusion, even when low concentrations exist at the barrier-aquifer interface.
- The recommended boundary condition (flushing condition) for one-dimensional analysis of a low hydraulic conductivity barrier does not give meaningful predictions of contaminant concentration at the barrier-aquifer interface; however, it does yield meaningful and conservative flux predictions.

Currently, both flux and concentration criteria are under consideration as elements of proposed regulations related to landfill design in Germany (Teutsch, personal communication, 1995). While adoption of a flux-based standard represents a conceptual shift from the traditional approach, it would facilitate the use of tractable models and produce conservative system designs. In some situations, it may be advisable to supplement flux-based criteria with a requirement for some specified level of hydraulic control. In the future, such control may become unnecessary as more data become available and models are developed that more confidently predict the long-term performance of engineered barriers.

10.4.2 Software Development

An attractive approach to modeling barrier systems that would enable the consideration of the barrier system in the context of a larger field-scale model is the "coupled" approach in which a local (e.g., one-dimensional) model of reactive transport within a barrier system is linked as a source term to a larger-scale multidimensional model of flow and transport. Because the conditions in the surrounding aquifer influence the boundary condition at the barrier, a dynamic linkage of the two models would be most desirable, enabling a realistic treatment of the barrier system while maintaining the larger discretization and simpler chemical model appropriate for the field scale. Such a linkage, while conceptually straightforward, has not been implemented in practice, and should be a priority for future model development activities.

10.4.3 Better Understanding of Reactive Transport

In general, accurate modeling of multispecies reactive transport is poorly developed. For some low-permeability barrier systems, neglecting the role of chemical reactions may be conservative and appropriate. The development of reactive low-permeability barriers may be a promising alternative, however, to the construction of the massive barriers that would be necessary to limit diffusive flux over time frames of interest. If such an approach is to be successful, development of appropriate and reliable models for reactive transport is essential. Currently, the relative importance of various reaction process is poorly understood, particularly for systems that contain mixtures of contaminants. Because engineered barriers will typically exhibit chemical behavior that is different from the surrounding media, a solid understanding of system behavior is needed to support innovative design.

10.4.4 Field Data for Model Testing

Confidence in model predictions can only be established through documentation of successful simulations in a post-audit mode. In particular, considerable uncertainty is associated with modeling of low-permeability barrier systems, due to:

- the need for modeling predications over time horizons much longer than any experimental or historical observations of similar systems;
- difficulty in conducting laboratory experiments that are representative of field conditions;
- uncertainty regarding the correspondence between low-permeability barrier systems as designed and constructed; and
- the lack of information regarding time-evolution of barrier properties.

For these and other reasons, it is crucial that coordinated efforts be initiated to develop a database for field applications that includes spatial and temporal measurements of parameters influencing contaminant transport.

10.4.5 Long-Term Performance

From conceptual standpoint, engineered barriers are unlikely to perform as intended over the design lifetime of the system, and realistic models should include consideration of the time-evolution of barrier properties. In practice, however, neither an appropriate modeling framework for barrier systems nor data for parameterization of failure and damage accumulation models are available. While there are analogues in other disciplines, the unique nature of subsurface porous media systems complicate the application of models incorporating failure in a realistic manner. If predictions of long-term barrier performance are to be credible, a framework must be developed for dealing with the anticipated long-term evolution of barrier properties. Along with the acquisition of field data, the development of such a framework should be a research priority and a subject of ongoing dialogue among modelers and all affected parties. In the short term, performance criteria may need to include some level of hydraulic control to compensate for uncertainty regarding long-term performance.

10.5 SUMMARY

10.5.1 Background

The primary focus of the Contaminant Transport Modeling session was on simulation of groundwater flow and advective-dispersive-reactive contaminant transport, as applied to low permeability barriers. There was general agreement regarding the applicability of the fundamental theories commonly applied to porous media and the availability of a variety of suitable analytical and numerical models. The primary focus of the discussion was on identifying appropriate modeling strategies for “applied” modeling and decision making, and in this regard a major concern was the establishment of correspondence between modeling objectives, the degree of detail needed in a predictive model, and the availability of data for parameter estimation.

10.5.2 State of Practice

The assessment of the state of practice varied according to the modeling objective under consideration. In general, it is believed that low-permeability barriers can be readily incorporated into groundwater flow models through well-established procedures; the popular MODFLOW model, for example, has recently been expanded to include a barrier module. Numerous examples are available in which modeling studies had been used to analyze the impact of low-permeability barriers on groundwater flow.

With regards to contaminant transport, it suggested here that the inclusion of a low permeability barrier in a regional or field-scale model, while tractable, is unrealistic for routine applications, due to the grid resolution needed to handle the discontinuity in media properties between the barrier and adjoining aquifer. A “coupled” modeling approach is suggested, in which a local model of the low-permeability barrier system would be solved separately and coupled with a field-scale transport code (e.g., MT3D) as a source term. While this coupling is expected to be straightforward, it has not been implemented.

A considerable amount of discussion addressed the appropriate application of one-dimensional transport models. One-dimensional contaminant transport models for horizontal low-permeability barriers have been developed and widely used, and with appropriate modification of the conceptual model may be readily applied to vertical low-permeability barriers. In general, modeling predictions would be best expressed in terms of flux from within a contained area, rather than concentrations at a point outside the barrier.

Performance criteria for low-permeability barrier systems should be referenced to the flux concept; recent consideration of this approach in Germany for landfill design was cited.

10.5.3 Assessment

In general, the use of one-dimensional models for low-permeability barrier systems was viewed as reasonable approach when the barrier performs as designed, i.e., for diffusion-dominated transport with constant, spatially uniform barrier properties. A variety of appropriate published solutions are available for various boundary conditions and reaction scenarios. For a conservative analysis, a constant-concentration entrance boundary and a zero-concentration exit boundary are recommended. The Appendix contains an example of a useful solution that includes these conditions and the processes of advection, linear equilibrium sorption, and first-order decay.

A number of areas have been identified in which one-dimensional transport models may be inadequate. In particular, the importance of a realistic representation of chemical reactions is highlighted. Since most relevant reactions result in reduced contaminant flux, however, neglecting reaction terms may result in conservative predictions and may, therefore, be acceptable for routine applications. For cases in which reactive low-permeability barrier materials were specified as a means of meeting performance criteria, a more realistic treatment of reaction terms is essential. For example, the validity of the local equilibrium assumption has not been established for earthen barrier materials amended to promote sorption of contaminants. Other areas of concern include an inadequate understanding of coupled flow processes, and a lack of experimental studies performed under the low flow conditions and long time frames relevant to low-permeability barrier systems.

Problems were noted with respect to the use of simplified groundwater flow models (e.g., two dimensional, homogeneous) in the design of permeable reactive barriers. For this application, it is important to consider the effects of hydraulic conductivity heterogeneity, as even minor variations in spatial and temporal flow conditions could result in contaminants bypassing the treatment zone. The use of multi-dimensional numerical models and a refined computational grid was recommended for this application.

Discussion regarding parameter estimation focused on the measurement of diffusion and sorption coefficients. While laboratory methods for estimating these parameters are well established, the use of literature values may be appropriate and conservative for many design applications. As noted previously, more precise estimates may be required for reactive low-permeability barriers.

An unresolved issue relates to the consideration of barrier damage accumulation. Engineered barriers are unlikely to maintain constant properties over the course of decades or centuries, but a consensus has not been reached regarding how to incorporate damage accumulation and failure into predictive models. Discussion of this topic among committee members should continue.

10.5.4 Recommendations

With regards to future research efforts, three needs are highlighted:

- implementation of well-instrumented, long-term field studies, in which data are collected over an extended time period;
- additional laboratory studies of contaminant transport in low-permeability materials, especially those involving reactive contaminants and low-flow conditions; and
- further development of a conceptual approach for modeling damage accumulation and failure in barrier systems, eventually leading to an established procedure for incorporating these considerations into predictions of long-term performance.

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10.7 APPENDIX

Transport Equation/Solution for One-Dimensional Transport of a Miscible Contaminant in a Low Hydraulic Conductivity Barrier

A useful model for describing the transport of a miscible contaminant in a one-dimensional barrier system is

$$R_f \frac{\partial C}{\partial t} = -\upsilon \frac{\partial C}{\partial x} + D \frac{\partial^2 C}{\partial x^2} - \lambda C$$

where C is the volume-averaged aqueous phase contaminant concentration, t is time, x is the distance from the entrance side of the barrier, v is the interstitial fluid velocity, D is the dispersion coefficient (includes hydrodynamic dispersion and molecular diffusion), λ is the first-order decay coefficient, and

R_f is the retardation factor.

As discussed in the text, a conservative set of auxiliary conditions for predicting contaminant flux under diffusion-dominated conditions is

$$C(x,0) = 0$$

$$C(0,t) = C_o$$

$$C(L,t) = 0$$

where L is the barrier thickness and C_0 is the constant contaminant concentration on the entrance side of the barrier. A solution to the above system is provided by Owen (1925), modified here to incorporate the retardation factor.

$$C = C_o \left\{ \begin{array}{c} \dfrac{\sinh\left([\mathrm{L} - \mathrm{x}]\sqrt{\upsilon^2 + 4\lambda D}\right)}{\sinh\left(L\sqrt{\upsilon^2 + 4\lambda D}\right)} \exp\left[\dfrac{x\upsilon}{2D}\right] \\ + \left(\dfrac{2\pi}{L^2}\right) \exp\left[\dfrac{x\upsilon}{2D}\right] \displaystyle\sum_{m=1}^{\infty} \dfrac{(-1)^m m \sin\left(\dfrac{m\pi x}{L}\right)}{\dfrac{\lambda}{D} + \dfrac{\upsilon^2}{4D^2} + \dfrac{m\pi^2}{L^2}} \exp\left[-\left(\dfrac{\lambda}{R_f} + \dfrac{\upsilon^2}{4DR_f} + \dfrac{Dm\pi^2}{R_f L^2}\right)t\right] \end{array} \right\}$$

The concentration-based solution may be modified to represent the contaminant flux normal to the barrier according to

$$J = n\upsilon C - nD\frac{\partial C}{\partial x}$$

where J is the contaminant flux and n is the porosity.

The development of the flux-based solution is straightforward, as presented by Rabideau, et al. (1996)

$$J=nC_o\exp\left(\frac{x\upsilon}{L}\right)\left\{\begin{array}{l}\left(\frac{3\upsilon}{2}\right)\frac{\sinh\left([L-x]\sqrt{\upsilon^2+4\lambda D}/2D\right)}{\sinh\left(L\sqrt{\upsilon^2+4\lambda D}/2D\right)}\\[2ex] +\left(\frac{3\pi\upsilon}{L^2}\right)\sum_{m=1}^{\infty}\frac{(-1)^m m\sin\left(\frac{m\pi(L-x)}{L}\right)}{\frac{\lambda}{D}+\frac{\upsilon^2}{4D^2}+\frac{m^2\pi^2}{L^2}}\exp\left[-\left(\lambda+\frac{\upsilon^2}{4DR_f}+\frac{Dm^2\pi^2}{R_fL^2}\right)t\right]\\[2ex] +\frac{\sqrt{\upsilon^2+4\lambda D}}{2}\frac{\cosh\left([L-x]\sqrt{\upsilon^2+4\lambda D}/2D\right)}{\sinh\left(L\sqrt{\upsilon^2+4\lambda D}/2D\right)}\\[2ex] +\left(\frac{2D\pi^2}{L^3}\right)\sum_{m=1}^{\infty}\frac{(-1)^m m^2\sin\left(\frac{m\pi(L-x)}{L}\right)}{\frac{\lambda}{D}+\frac{\upsilon^2}{4D^2}+\frac{m^2\pi^2}{L^2}}\exp\left[-\left(\lambda+\frac{\upsilon^2}{4DR_f}+\frac{Dm^2\pi^2}{R_fL^2}\right)t\right]\end{array}\right\}$$

The net flux of contaminant crossing the barrier at a specified time may be determined by evaluating the above expression at ($x=L$).

10.8 LIST OF SYMBOLS

- a_l Longitudinal dispersivity (L)
- C Aqueous phase solute concentration (M/L^3)
- C_o Known constant concentration (M/L^3)
- C_s Concentration of source/sink (M/L^3)
- D Dispersion coefficient (L^2/T)
- D_h Hydrodynamic mixing component of dispersion coefficient (L^2/T)
- D^* Effective solute diffusion coefficient (L^2/T)
- D_l Solute liquid diffusivity (L^2/T)
- h Piezometric head (L)
- h_b Dimension of adjoining aquifer normal to barrier (L)
- Δh Difference in piezometric head between the inside and outside of a barrier wall (L)
- H_F Dimension of contaminated area inside barrier (L)
- J Solute flux (M/L^2-T)
- K Hydraulic conductivity (L/T)
- k_d Sorption distribution coefficient (L^3/M)
- K_F Freundlich isotherm parameter (L^3/M)^{n_f}
- K_E Effective hydraulic conductivity (L/T)
- L Barrier thickness (L)

M_W Molecular weight of water (M/L^3)
n Porosity
n_f Freundlich isotherm exponent
P_e Peclet number
q_s Fluid source/sink (L^3/T)
r_B Solute molecular radius (L)
R Ideal gas constant (0.082 atm-L/mol-K)
R_f Retardation factor
S Solute sorbed phase mass fraction (M/M)
S_s Specific storage (1/L)
T Absolute temperature (°K)
t_{50} Half-life (T)
v Seepage velocity (L/T)
v_b Velocity of groundwater in aquifer adjoining the barrier (L/T)
V_B Solute molar volume at boiling point (L^3)
W Width of barrier normal to fluid/concentration gradients (L)
x Spatial coordinate (L)
α Sorption rate coefficient (1/T)
ε Exponent used in correlation between tortuosity and porosity
λ_a First-order decay constant for the aqueous phase (1/T)
λ_s First-order decay constant for the solid phase (1/T)
λ_b Lumped first-order decay constant (1/T)
ρ_b Bulk density (M/L^3)
ϕ_W Solution association constant in Wilke-Chang correlation
τ Tortuosity factor
μ_W Dynamic viscosity of water (FT/L^2)
ζ Integration variable

SECTION 11

PERMEABLE REACTIVE BARRIERS

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11.1 BACKGROUND

A permeable reactive barrier ("PRB") consists of a permeable curtain containing appropriate reactive materials, generally constructed to intercept the path of a contaminant plume. As the contaminated groundwater passes through the curtain, the contaminants are removed through chemical, physical, or biological processes (Gillham,1995). PRBs may also be installed downgradient of contaminant sources to prevent plumes from developing. Although permeable reactive walls are generally vertically oriented, horizontal applications have been considered for controlling the downward migration of contaminants. This latter application may prove feasible in some situations. Discrete areas may be treated by the use of fencepost-style reactive zones placed into the natural groundwater flow system (Puls et al., 1995). In addition, PRBs may be used in low permeability strata when the local hydraulic gradient must be supplemented with another driving force, such as electroosmosis (Orth, 1995).

PRBs are comprised of a reactor system and an optional control system to guide the groundwater flow through the PRB. The control system may be advantageous in situations where the contaminant plume is so large that the dimensions of a continuous PRB would be economically impractical. In these cases, sheet piling or low permeability slurry walls may be employed to funnel the plume more efficiently through a small reactor system. Under favorable hydrogeologic conditions, the natural groundwater flow system may require no control, and a continuous permeable wall reactor system may be employed.

The reactor system contains a porous reactive medium designed to treat the dissolved contaminants as the groundwater flows through the PRB. The reactive media may be permanent or replenishable, depending on the nature of the site geochemistry, reaction mechanisms, contaminant loading, degree of contaminant reduction desired, and design lifetime of the PRB. Reactors may extend the full width of the contaminant plume (as in the case of a continuous barrier) or be combined (singly or in multiples) with low hydraulic conductivity barrier walls (funnel elements). Also, PRBs containing different reactive porous media may be constructed in series to treat individual contaminant constituents.

PRBs can utilize a wide variety of reactive materials to effect contaminant removal. Removal mechanisms can be biotic or abiotic and include: sorption, precipitation, dehalogenation, oxidation-reduction, fixation, and physical transformation. PRBs can be installed by excavating a trench and emplacing the porous reactive medium, by flushing-type emplacement and replenishment of the reactant using injection wells, or by other specialized means discussed in this section.

PRBs can be "passive" or "active", based on the need for post-construction operations at the site. The "passive" PRB system utilizes natural groundwater flow to effect treatment. The "active" system requires postconstruction operation of extraction and injection wells to effect treatment. For example, operation of a series of air sparging wells or continuous in-well aeration is considered "active". For the most part, this section focuses on passive PRB technology.

11.2 STATE OF PRACTICE

11.2.1 Introduction

For an *in situ* reactor system to be effective in removing groundwater contaminants, three basic requirements must be met. First, the time required for the desired reactions to occur must be less than the time required for the contaminated groundwater to pass through the reactor. If the reaction rates are too slow, only partial transformation and removal of the contaminant will occur, leading to incomplete treatment, and possible failure of the system to meet regulatory compliance goals.

The second requirement is that the emplaced reactive treatment medium perform satisfactorily for an economically viable period. Contaminant plumes can persist for decades or longer (especially when DNAPL source zones are present), requiring the treatment system to be effective for a corresponding period. Under some geochemical settings and contaminant loadings, the selected reactive materials may require replacement during the design lifetime of the PRB. Design of a PRB for a given application may call for optimization in the selection of the reactive media so that it is sufficiently reactive to effect treatment during the design flow residence times, yet sufficiently stable to be effective for an economically viable period.

The third requirement is that the reactive medium itself not introduce contamination that would be unacceptable in the downgradient groundwater (Blowes, et al., 1995a). In a poorly designed reactor system, reaction by-products might be formed that are less desirable than the original contaminant. The goal in most cases is to achieve risk-based groundwater cleanup requirements; for example, U.S. EPA maximum contaminant levels (MCLs) in the case of a drinking water aquifer, or levels consistent with surface water protection where groundwater is not used directly.

Thus, in the design and construction of a permeable reactive barrier, due consideration needs to be given to the requirements discussed above. The basic elements which factor into the design and construction of PRBs are:

- selection of the proper treatment technology/medium,
- system design protocol, including
  - site characterization in advance of the design,
  - design of the optional control system, when necessary,
  - design of the reactor system,
- emplacement/construction of the permeable reactive barrier, and
- performance monitoring.

11.2.2 Permeable Barrier Treatment Technologies

The selection of a treatment medium is based primarily on its effectiveness to treat site-specific contaminants. Treatment schemes can be based on organic, inorganic, and combined (organic and inorganic) reaction processes. Frequently, several different contaminants can be treated utilizing the same or related reaction processes. For example, sorption may be effective in removing hydrocarbons and certain metals from contaminated groundwater. For this reason, reaction mechanisms are discussed in terms of their applicability to treat specific types of contaminants. Table 11-1 presents an overview of various treatment media considered for PRBs, the target contaminants they address, and the current stage of development (i.e., laboratory study stage, field demonstration stage, etc.). Brief descriptions have been provided for the types of reactions used in PRBs.

a. Chemical Precipitation. Precipitation of dissolved inorganic species (e.g., heavy metals) may be accomplished by using a slightly soluble material containing an ion which forms an insoluble salt (such as a phosphate, sulfate, hydroxide, or carbonate metal salts) with the contaminant. The material itself should be nontoxic, have a solubility higher than that of the precipitate to be formed, but sufficiently low so that the material continues to be effective for the desired period of time.

TABLE 11-1 Treatment technologies applied in permeable reactive barriers

Treatment Medium	Target Contaminants	Technology Status
zero valent iron	halocarbons	commercially applied
zero valent iron	reducible metals (Cr+6, U)	field demonstration
limestone	metals, acid waters	in practice (mining)
precipitation agents (gypsum, hydroxyapatite)	metals	lab studies
sorptive agents (Fe hydroxide, GAC*, zeolites, coal)	metals and organics	field demonstration and/or lab studies
reducing agents (organic compost, dithionite, hydrogen sulfide)	reducible metals	field demonstration
metal couples**	halocarbons	lab studies
biologic electron acceptors (ORC***-oxygen source, nitrate)	BTEX	field experiments

* GAC = granulated activated carbon
** coupled oxidation of metal and reduction of halocarbon to produce chloride and Fe+2 in solution
*** ORC = oxygen release compound

Other reactants can be added to promote precipitation of insoluble mineral phases or to promote precipitation through adjustment of pH. Limestone (calcium carbonate) has been used for many years to neutralize acid and to precipitate metal contaminants; e.g., to construct reactive flow-through treatment barriers to neutralize acid mine drainage from coal and metal ore mining sites. Similarly, hydroxyapatite (calcium phosphate) has been used as a source of phosphate to precipitate lead from groundwater in the form of lead phosphate. Morrison and Spangler (1993) investigated the use of hydrated lime as a precipitation agent in chemical barriers.

b. Oxidation-Reduction Reactions. Oxidation-reduction ("redox") reactions can be used to change the valence state of an inorganic contaminant, thus reducing solubility and enhancing precipitation (Blowes and Ptacek, 1994). An example involves the reduction of hexavalent chromium (Cr+6) to the trivalent form (Cr+3), with subsequent precipitation as an oxide. Redox reactions can involve chemical, electrochemical, and biochemical removal processes. Treatment media which have been applied include such reductants as zero-valent metals, hydrogen sulfide, sodium dithionite, and degradable biomass.

Blowes, et al. (1995a) and Thomson (1995) reported on the use of biomass to promote redox removal of inorganics. Biological reduction of sulfate to sulfide by sulfate-reducing bacteria can be used to remove metals from mine tailings' water through precipitation as insoluble metallic sulfides. Also, a biological approach can be used to remove nitrate from groundwater through the process of denitrification. Generally, these processes employ the addition of a substrate to the PRB that promote biodegradation using indigenous bacteria.

Redox reactions also can be used to convert organic contaminants to nontoxic by-products. For example, the strong oxidant potassium permanganate has been investigated as a possible oxidizing agent for remediation of chlorinated hydrocarbons (Farquhar, 1992).

c. Zero-Valent Metal Dehalogenation. Zero-valent metals (e.g., granular iron) can be used to promote reductive dechlorination of chlorocarbons, e.g., PCE and TCE (Gillham, et al., 1995). The zero-valent iron reaction involves oxidation (corrosion) of the metal by the chlorocarbons, with the metal serving as a source of electrons for chlorocarbon reduction. The reduction step removes chlorine atoms from the chlorocarbon molecule, releasing chloride and ferrous iron (Fe+2) into solution. The process appears to be surface-mediated, meaning that direct contact with (and possibly sorption onto) the iron surface is necessary for the reaction to take place, in that the rate of reaction appears to be directly proportional to the surface area of granular iron present. The end-products of PCE and TCE dehalogenation are primarily ethene and ethane. However, a small proportion of the initial PCE/TCE can appear as partially dechlorinated "daughter products" (i.e., cis 1,2-dichloroethene and/or vinyl chloride) if reaction time is insufficient. Thus, a primary design goal using reactive iron is to size the PRB to ensure complete dehalogenation. Table 11-2 (Gillham, et al., 1995) provides a summary of contaminants that have been tested with granular iron, along with reported degradation half-lives based on iron surface area normalized to 1.0 square meter per milliliter of solution.

Various enhancements to the standard granular iron degradation process are being examined. Orth (1995) investigated the use of iron plated with metals having higher reduction potentials (e.g., copper) and found significantly greater rates of dechlorination compared to iron alone, apparently due to the corrosion-inducing effect of the plated metal. Korte, et al. (1995) had similar results using palladium plated on granular iron ("palladized iron"). In addition to increasing the rate of dechlorination, chlorinated daughter products were not detected. The addition of sulfur-containing compounds such as pyrite may help to stabilize the iron surfaces, thus prolonging reactivity (Holser, et al., 1995).

d. Biological Degradation Reactions. Modifying redox conditions can increase the rates of biodegradation of some common aromatic hydrocarbons, such as benzene, ethylbenzene, toluene, and xylenes (BETX). These compounds generally are more readily biodegraded under more oxidized conditions than under highly anaerobic conditions (Wilson and McNabb, 1983). Modifying redox conditions involves the addition of a suitable electron acceptor (nitrate or oxygen). The rate of biodegradation can be enhanced by supplying the PRB with rate-limiting nutrients, e.g., nitrogen and phosphorus.

TABLE 11-2 Compounds tested and half-lives normalized to 1 m^2 iron surface per ml solution (from Gillham, 1995)

Organic Compound		Pure Iron $t_{1/2}$ (hr)	Commerical Iron $t_{1/2}$ (hr)
Methanes	Carbon Tetrachloride	0.20[a], 0.003[f], 0.023[h]	0.31-0.85[b]
	Chloroform	1.49[a], 0.73[f]	4.8[b]
	Bromoform	0.041[a]	
Ethanes	Hexachloroethane	0.013[a]	
	1,1,2,2-Tetrachloroethane	0.053[a]	
	1,1,1,2-Tetrachloroethane	0.049[a]	
	1,1,1-Trichloroethane	0.065[a],1,4[g]	1.7-4.1[b]
Ethenes	Tetrachloroethene	0.28[a],5.2[g]	2.1-10.8[b]
	Trichloroethene	0.67[a],7.3-9.7[f]	1.1-4.6[b],2.8[e]
	1,1-Dichloroethene	5.5[a],2.8[g]	15.2[e]
	trans 1,2-Dichloroethene	6.4[a]	4.9[b],7.6[e]
	cis 1,2-Dichloroethene	19.7[a]	10.8-33.9[b]
	Vinyl Chloride	12.6[a]	10.8-12.3[b],4.7
Other	1,1-2-Trichlorotrifluoroethane (Freon 113)	1.02[b]	
	1,2,3-Trichloropropane		24.0[c]
	1,2-Dichloropropane		4.5[c]
	1,3-Dichloropropane		2.2[c]
	1,2-Dibromo-3-chloropropane		0.72[b]
	1,2-Dibromoethane		1.5-6.5[b]
	n-Nitrosodimethylamine (NDMA)	1.83[b]	
	Nitrobenzene	0.008	
No Apparent Degradation	Dichloromethane [af,g], 1,4-Dichlorobenzene[g], 1,1-Dichloroethane[g], 1,2-Dichloroethane[b], Chloromethane[b]		

[a] Gillham and O'Hannesin (1994)
[b] Unpublished Waterloo data
[c] Focht (1994)
[d] Agrawal and Tratnyek (1994)
[e] Mackenzie, et al. (1995)
[f] Matheson and Tratnyek (1994)
[g] Schreier and Reinhard (1994)
[h] Lipczynska-Kochany, et al. (1994)

Kao and Borden (1992) proposed the use of a two-layer PRB, composed of a nutrient-laden (nitrogen and phosphorus) concrete briquette layer followed by a layer of peat. Nitrate is released as the contaminated water passes around the nutrient briquettes, thus stimulating BETX degradation through denitrification.

Bianchi-Mosquerat, et al. (1993) describe the use of a solid-phase oxygen release compound (ORC) to enhance degradation of BETX compounds. Devlin and Barker (1992) proposed the use of a semi-passive PRB to facilitate nutrient addition for stimulating bioremediation processes. They envisioned a PRB with a hydraulic conductivity substantially greater than the surrounding geologic material, through which a nutrient-amended groundwater stream would be periodically circulated using injection and withdrawal wells. The injected amended groundwater stream would then be carried downstream of the wall by the natural groundwater flow. Periodic injection of the amendment to the PRB would result in a series of pulses that, while migrating downstream, would mix (by dispersion) with the contaminated groundwater, blending to form a continuous treatment zone of nutrient-amended groundwater.

e. Sorption Reactions. Sorption reactions may be divided into hydrophobic, hydrophilic, and ion exchange types. Many materials have been investigated and/or used to sorb dissolved organic and inorganic species from aqueous solutions. Since sorption reactions typically equilibrate in a short period of time, sorption media are generally well-suited for application in PRBs. However, sorption media have finite capacity to sorb; therefore, contaminant breakthrough occurs when this capacity is exceeded. Thus, in order to avoid eventual breakthrough of the contaminant, a means to remove and replenish the treatment medium must be provided in a sorption PRB system.

f. Sorption of Organics. Sorption may be well suited for strongly sorbing organic compounds that have relatively low water solubilities, hydrophobic character, and are not easily amenable to biodegradation (such as polynuclear aromatic hydrocarbons). These compounds are known to partition from the water phase to the solid phase organic carbon of geologic materials. Hence, a possible approach for the *in situ* removal of these contaminants from groundwater might be to increase the organic carbon content of the aquifer material in the path of the contaminant plume.

Teutsch and Grathwohl (1995) discuss the application of permeable sorptive walls for treatment of hydrophobic organic contaminant plumes. Potential materials applicable to sorption of these organics include: granular activated carbon, peat, coal, and organic-rich shales. Again, attention needs to be given to the sorptive capacity of the selected treatment material. Modification of sorptive materials by surfactant application can increase sorption capacity and selectivity. The capacity of a porous medium to sorb hydrophobic organic solutes may be enhanced by injecting a cationic surfactant solution into the subsurface. The ionic end of the surfactant molecule is sorbed by the mineral surfaces, while the organic solutes are sorbed at the hydrophobic end of the surfactant molecules.

Immobilized (sorbed) organics may be subsequently biodegraded through the upstream injection of nutrients (Teutsch and Grathwohl, 1995). This combined treatment approach might be particularly effective for organics that are readily biodegradable, such as aromatic hydrocarbons (e.g., BETX).

g. Sorption of Inorganics. Inorganics, e.g., metals, are well suited to hydrophilic and ion exchange sorption reactions. Materials suitable for sorbing metals include organic carbon, zeolites, aluminosilicate clays, iron oxyhydroxides, and other mineral materials.

The most important factors affecting metal adsorption onto organic carbon are: the carbon type, pH of the contaminated groundwater, and surface loading rate. The adsorption reaction occurs between the metal ion and their hydroxy species with the carbon surface to form hydrogen bonded surface complexes. Sorption of metals on carbon follows the Irving-Williams order of complex formation as follows: Pb > Cu > Ni > Zn = Mn = Cd = Co; where all the metals are in the two valence state.

Zeolites have been widely used and studied as ion exchange media. Their ion exchange capacity depends on the substitution of alumina for silica in parts of the zeolite structure, resulting in a net negative charge on the mineral. Sodium, calcium, potassium, and other exchangeable cations balance the charge by occupying the open channels in the structure. Larger cations, with low energies of hydration, form the strongest bonds in the zeolite structure. Factors affecting the sorption of metals by zeolites include the initial form of the zeolite, (e.g. sodium saturated), the ionic radius and energy of hydration of the sorbing cation, and the presence of complexing agents in solution. Unmodified zeolites sorb only cationic metals (i.e., oxyanionic forms such as arsenate and chromate are not sorbed) and the sorbed metals may be displaced with concentrated sodium solutions. Clinoptilolite, chabazite, and other zeolites have been used to sorb inorganic contaminants.

The use of organo-zeolites (formed by reacting natural zeolite with a cationic quaternary amine surfactant) has been investigated for the sorption of inorganic oxyanions, such as chromate, selenate, and sulfate (Haggerty and Bowman, 1993). The precipitated organo salt is stable in the hydrophobic environment created by the surfactant. Sorption of TCE by a synthetic hydrophobic zeolite has been investigated by Alvarez-Cohen, et al. (1993) using a two stage process in which the hydrocarbon is first sorbed onto the zeolite and subsequently desorbed and biodegraded.

While aluminosilicate clays are suitable ion exchangers and are known to sorb organics and heavy metals, their small particle size (typically less than five microns) and associated low hydraulic conductivity excludes them for use in PRBs.

Iron, aluminum, and manganese oxyhydroxides can adsorb divalent metals such as cadmium, cobalt, copper, lead, and zinc. Adsorption of heavy metals onto oxyhydroxides depends on pH, (i.e., sorption increases with increasing pH). The affinities of metal ions for the oxide surface decrease in the order: Cu > Pb > Zn > Co > Cd. Morrison and Spangler (1996) report on the laboratory study of amorphous iron oxyhydroxides (AFO) to sorb a wide variety of metals, including the oxyanions chromate and arsenate.

Oxide surfaces can be modified by polyphosphates to enhance sorption. The polyphosphates do not form insoluble metal salts, but they will sorb to the oxide surfaces. Once sorbed, they may enhance the sorption of metals by forming metal-polyphosphate-surface complexes. Excessive amounts of polyphosphate (beyond sorption capacity) will cause desorption of metals from the oxide surface.

Sorption of organics by organo-clay complexes has potential application in low organic carbon aquifers. Aquifer mineral phases can form organo-mineral complexes that increase the adsorption capacity of the aquifer materials. The organo complexes are formed by reaction of the aquifer solids with various surfactants. Surfactant micelles adsorbed to the aquifer solid surfaces attract dissolved organics. The strongly bound surfactants can substantially increase the sorption capacity of the aquifer materials.

TABLE 11-3 Summary of reported permeable reactive barrier materials for inorganic contaminants* (from Morrison and Spangler, 1995)

Exp. No.	Contam-inant	Material	Maximum Capacity (mol/mol)	Reference
1	Cr	Fe^o	5×10^{-4} mol Cr/mol Fe	Cantrell, et al. (in press)
2	Cr	Fe^o	3×10^{-6} mol Cr/mol Fe	Blowes and Ptacek, 1992
3	Cr	Fe^o	1×10^{-2} mol Cr/mol Fe	Powell, et al. 1995
4	Cr	AFO	6×10^{-3} mol Cr/mol Fe	Zachara, et al. 1987
5	Cr	HDTMA Zeolite	3×10^{-4} mol Cr/mol Si	Haggerty and Bowman, 1994
6	Cr	H_2S	1×10^{-2} mol Cr/mol S	Thornton and Jackson, 1994
7	Cr	BrY bacteria	1×10^{-6} mol Cr/10^6 Cell	Gorby, et al. 1994
8	U	Fe^o	1×10^{-4} mol U/mol Fe	Kaplan, et al. 1994
9	U	AFO	1×10^{-4} mot U/ mol Fe	Morrison, et al. 1995
10	U	Peat	2×10^{-3} mol U/mol C	Morrison, et al. 1992
11	U	$Ca(OH)_2$	1×10^{-4} mol U/mol Ca	Morrison, et al. 1992
12	Mo	Fe^o	3×10^{-4} mol Mo/mol Fe	Cantrell, et al. (in press)
13	Pb	Phosphate rock	2×10^{-2} mol Pb/mol P	Ma, et al. 1995
14	Sr	Zeolite	9×10^{-3} mol Sr/mol Si	Cantrell

*Some estimates of chemical compositions (e.g. of peat) and porosity were made.

Table 11-3 (from Morrison and Spangler, 1995) presents a summary of PRB materials suitable for inorganic contaminants, including several sorption materials. Removal capacities (based on lab studies) are listed for the various materials. The reported removal capacities range over several orders of magnitude, and should be used with caution. For example, the values of chromate retention in experiments 1 and 2 represent minima because the granular iron capacity had not been depleted in the experiments. In addition, solution composition (e.g., hardness, pH, concentration of competitive ions) will affect the degree of sorption. For these reasons, PRB designs should be based on treatability studies using the sorptive media and groundwater from the site.

11.2.3 System Design Protocol

a. Site Characterization. PRB systems are designed by focusing on the factors that most affect their performance. Characterization of the remediation site is an essential element in the system design protocol. (Since site characterization is treated elsewhere in this report, discussion here is limited to PRB applications.) In many respects, the considerations given to the placement of a low hydraulic conductivity barrier are also applicable to the placement of a PRB. Since it is desired that all of the contaminated groundwater flow through the PRB, careful attention needs to be given to development of the groundwater flow model for the site. The groundwater flow model enables evaluation of: 1) the effectiveness of various PRB configurations to intercept the contaminated groundwater flow and 2) the need for hydraulic control augmentation.

A hydrogeologic conceptual model is developed early in the process to aid in making large scale decisions concerning the optimal deployment of a PRB. Simply put, a conceptual site hydrogeologic model consists of defining the basic dimensions of the contaminant plume and defining the rates and directions of plume movement within the context of the site stratigraphy. The conceptual model helps in developing the initial concept of how a PRB would best be used and also serves as a basis for initial assessment of technical and cost feasibility. In addition, the model serves as a framework for developing more detailed information on the site which is needed during the final design.

The most obvious constraints to emplacement and effective treatment using a PRB are the geologic setting of the contamination (i.e., aquifer type) and the depth of contaminated groundwater. The PRB typically will be constructed through the entire thickness of a contaminated aquifer. Ideally, the barrier would be keyed into an underlying low hydraulic conductivity layer, e.g., a clay aquitard or sound bedrock. For buoyant or sufficiently shallow plumes, a hanging wall system may be considered.

Once the preliminary conceptual model is completed and a PRB concept (or set of PRB options) developed, a detailed geologic investigation is conducted in the immediate vicinity of the proposed PRB. Geologic parameters to be determined include lithology, stratigraphy, grain size distribution, and structural relationships. A set of geologic cross sections is developed for the PRB site showing details of the stratigraphy at least as deep as the first aquitard beneath the contaminant plume.

The site groundwater gradient, flow direction, hydraulic conductivity, and water balance are developed using standard methods. Uncertainties in these parameters can be particularly significant in the design of a PRB. Measurements of piezometric head may not be a sufficiently reliable basis for estimating of flow direction due to small head differences displayed by wells located in close proximity. Groundwater flow directions can vary due to seasonal recharge effects and heterogeneity and anisotropy of the aquifer, which may cause local variations in flow rate and contaminant flux, often by an order of magnitude or more from one point to another. Aquifer anisotropy is particularly critical when a "hanging gate" PRB is being considered. PRB designs must account for these uncertainties. Alternatively, the designer may build in physical features or safety factors to compensate for these uncertainties.

Delineation of the plume boundaries and the location of zones with high concentration are important factors in determining if a PRB will be a cost-effective remedial technology. The spatial distribution of the contaminant must be determined prior to design, as well as contaminant properties (solubility, vapor pressure, specific gravity, partitioning, etc.), including their chemical relationship to site geology and geochemistry.

Groundwater flow models are useful in optimizing the PRB design. Two-dimensional plan view simulations are appropriate for PRBs that penetrate the entire thickness of relatively homogeneous aquifers. Systems that extend only partially through an aquifer (hanging wall systems) are best described by three dimensional simulations, since low hydraulic conductivity lenses can significantly affect PRB performance, potentially limiting overall system efficiency.

Geochemical characterization of site groundwater and host aquifer materials is critical to proper evaluation of PRB performance. Naturally occurring groundwater constituents may compete or interfere with sorption and precipitation reactions. Chemical reactions between treatment reactants, groundwater contaminants, and the natural groundwater constituents may also cause formation and reaction zone plugging. Chemical interactions of the barrier reactants with natural groundwater constituents can cause barrier aging and failure. For example, amorphous ferric oxyhydroxide undergoes a transformation to the crystalline goethite form of iron oxide, occurring over a period of weeks to years. Both iron species will adsorb contaminants from groundwater but at different reaction rates. In addition, aquifer materials can buffer groundwater equilibrium reactions in ways that are unexpected and deleterious to the PRB's performance.

b. PRB Hydraulic Control Systems. Basic types of PRB hydraulic control systems that have been studied for application to contaminant plume control include: funnel and gate PRBs, continuous wall PRBs, and injected treatment zone PRBs.

<u>Funnel and Gate PRBs</u>. Funnel and gate PRBs (Starr and Cherry, 1994; Smyth, et al., 1994; Smyth, Cherry, and Jowett, 1994) utilize an impermeable barrier (funnel) placed in the path of the contaminated groundwater flow (see Figure 11-1). The funnel shape guides the flow through the permeable reactive zone (gate). Types of impermeable barriers currently used in practice include slurry walls, sheet piles, or soil admixtures formed by soil mixing or jet grouting. The function of the funnel is to widen the capture zone by diverting the groundwater flow through the relatively small permeable gate. The presence of the impermeable barriers alters the local piezometric head distribution which in turn controls the groundwater flow in the vicinity of the funnel and gate. The funnel must be placed to sufficient depth and width to produce a zone of capture which encompasses the entire plume and directs it through the permeable reactive gate. Keying the bottom of the funnel and gate into a lower aquitard is the typical concept considered.

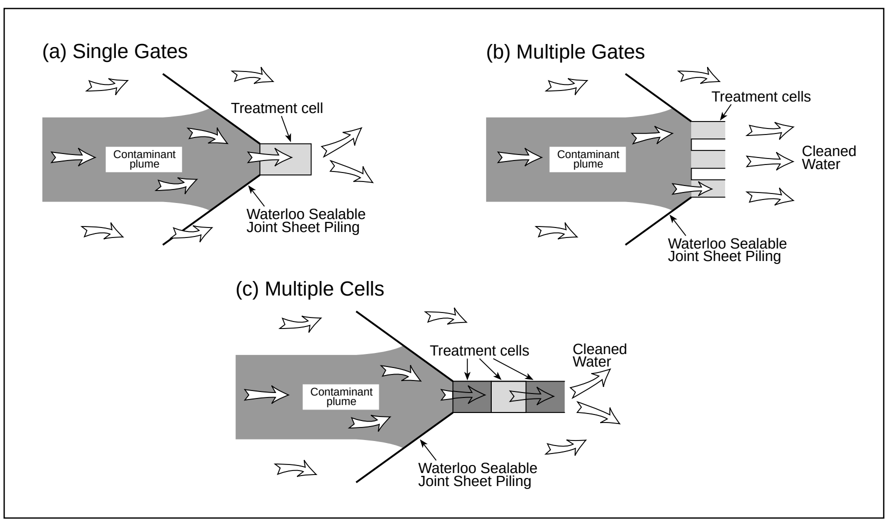

Figure 11-1 Conceptual configurations of funnel-and-gate systems in plan view: 11-4a shows a simple system with a single-cell gate; 11-4b shows a system with three single-chamber gates in parallel; and 11-4c depicts a system with a sequence of treatment steps incorporated in the gate. (Smyth, Cherry, and Jowett, 1994)

The funnel and gate system can have one of two basic configurations: (1) retrievable, or (2) non-retrievable. The retrievable system consists of the following:

- a porous structural member with possible surrounding gravel pack designed to separate aquifer soils from the treatment medium yet allow

transmission of water through the media (e.g., a well screen with gravel pack), or
- porous treatment medium contained in a removable container such as a filter fabric bag, wick drain, or metal cassette.

The possible advantage of a retrievable gate is that the treatment medium can be removed and replaced, once its useful life has been reached. However, care must be given to ensuring that the effective hydraulic conductivity of the retrievable gate is comparable to that of the surrounding aquifer. Excessive head losses in the vicinity of the gate will raise local piezometric head levels, resulting in the potential diversion of contaminated groundwater flow around, over, or under the gate. In addition, such systems have yet to be tried at full-scale, and retrievable/replaceable materials could prove difficult in practice.

Non-retrievable reactive zone gates place the reactive medium in direct contact with aquifer soils or with a filter medium (e.g., gravel pack) between the reactive medium and aquifer soils. One disadvantage of this approach may be a reduced capability to replace the reactive medium if the design life is not sufficient for long-term plume control. It remains to be seen if this is a true limitation, since various methods of replacing or regenerating materials in place can be envisioned (e.g., soil mixing, vacuum trucks, regenerative reagent flushing, etc.).

In a funnel and gate system, the resultant zone of capture may be small in comparison to the required funnel dimensions. The following are potential concepts to enhance the zone of capture of funnel and gate systems (Phifer, 1995):

- multiple funnel and gate systems placed in series,
- downgradient pumping to steepen the hydraulic gradient and increase groundwater flow through the treatment gate,
- creation of enhanced permeable zones parallel to the impermeable barrier on both the up- and downgradient sides of the barrier, and tied into the treatment gate.

The use of downgradient pumping to increase the hydraulic gradient through the gate has the disadvantage of bringing groundwater to the surface which will require handling. Since the water theoretically will be clean, reinjection upgradient of the funnel and gate may be acceptable. However, routine operation will be required, so that the system is no longer strictly passive (a definite disadvantage).

Starr and Cherry (1994) provide a comprehensive modeling study of various alternative funnel and gate systems and offer guidance on optimizing the design of funnel and gate systems.

<u>Continuous Wall PRBs</u>. As described by Phifer (1995), a continuous wall PRB consists of the placement of a permeable reactive zone perpendicular to the

path of the contaminant plume across its entire depth and width. Reactive media is placed in direct contact with aquifer solids or a gravel pack may be placed between these elements. As discussed in a later section, continuous wall PRBs could be emplaced by excavation/backfill or soil displacement methods.

The continuous wall has the conceptual advantage of creating less disturbance to the natural groundwater flow pattern than the "funnel and gate" system (described above). Also, the system may be more easily designed to achieve the long-term life requirements for addressing groundwater plumes. The following considerations are important factors in the selection of a continuous wall PRB versus a funnel and gate system:

- up-front cost of the treatment media compared to the on-going costs of replacing treatment media in a funnel and gate system,
- irretrievability of the treatment media; a new wall would need to be emplaced if plume life exceeded the reactive life of the continuous wall PRB,
- volume of contaminated soils excavated during PRB emplacement (depending on the emplacement method selected and ability to replace soils on-site),
- potential formation disturbances resulting from PRB emplacement methods.

<u>Injected Treatment Zone PRBs</u>. An alternate form of PRB involves the injection of the reactant into the soil or aquifer to form a reactive treatment zone. The reactant may be delivered into the aquifer as a dissolved phase by injection through wells. The injected solution then reacts with and coats the aquifer solids, thereby creating the PRB. For example, Burris and Antworth (1992) discuss the formation of a sorbent PRB within an aquifer by injection of an aqueous solution of cationic surfactants through a set of injection wells located downgradient from a contaminant source. Morrison and Spangler (1993, 1995) proposed the injection of a ferric chloride solution into calcite-bearing aquifers to precipitate amorphous ferric oxyhydroxide on the aquifer solids as a sorbent PRB. Alternatively, the reactive media may be injected as a slurry (suspended solid phase media) through the process of hydraulic fracturing.

A potential advantage of this approach over the funnel and gate and continuous wall PRBs is the ability to more cost-effectively emplace materials at depths greater than are accessible by standard excavation methods. A potential disadvantage is the relative lack of control during the emplacement process. These issues are discussed in more detail later in this section under "Emplacement Methods".

<u>Miscellaneous Factors Affecting Hydraulic Performance</u>. *Designing the PRB to Address Non-Uniform Flow.* Nonuniform flow can result when regional groundwater flow directions are not perpendicular to the orientation of the

treatment wall (Gallinatti and Warner, 1994). Also, heterogeneities in the aquifer and/or in the reactive barrier itself can result in nonuniform flow. The implications of nonuniform flow in the vicinity of the permeable barrier are serious, since the treatment of the contaminated groundwater flow may also be nonuniform.

Groundwater flow that is not perpendicular to the orientation of the permeable barrier may cause groundwater flow to converge within the reactive zone causing local increases in velocity. Heterogeneities may create preferential flow paths that also result in locally high groundwater velocity zones. Two methods for reducing the potential effects of nonuniform flow are: 1) extending lateral barriers downgradient from the permeable wall to create a downgradient "chute" that causes flow convergence downgradient rather than within the treatment zone, and 2) sandwiching the treatment zone between zones of very high hydraulic conductivity material; the nonuniform flow components tend to occur within the higher hydraulic conductivity zones rather than within the treatment zone (Warner, et al., 1995).

Effect of Reactor Hydraulic Conductivity. The hydraulic conductivity of the permeable reactive barrier should be as high as possible so as to minimize local groundwater head buildup upstream of the reactor. However, maximizing reactor conductivity must be tempered by the kinetics of the reaction mechanism, whose rate may be dependent on surface area considerations (generally low for large grain-size porous media). Discharge through the reactor increases with increasing hydraulic conductivity, but there is relatively little increase for conductivities greater than 10 times that of the aquifer. Thus, selection of a properly graded porous reactive media is an important consideration in balancing hydraulic conductivity and reaction kinetics. In general, a poorly graded (well-sorted) granular backfill is likely to be best in practice.

Enhanced Permeable Zones. In the funnel and gate PRB system, enhanced permeable zones upstream and/or downstream of the gate can be used to direct the flow of groundwater more efficiently through the reactive zone, thereby increasing the size of the capture zone. Enhanced permeable zones can also be used to redistribute groundwater downstream of the gate to alleviate hydraulic head buildup in the funnel area upstream of the gate (Phifer, 1995). Enhanced permeable zones can also be used to reduce the need for more elaborate funnel configurations and to reduce the effects of subsurface heterogeneity.

c. PRB Reactor System Design. The major elements of reactor system design include: (1) selection of the reactive medium, including the chemical makeup, particle size distribution, and proportion of admixtures, if needed; (2) setting the dimensions of the reactor to provide the necessary retention/reaction time; and (3) sizing the reactor to provide for adequate treatment over the required design lifetime. Treatability studies (described briefly below) using the candidate reactive media and site groundwater form the basis for primary design. However, a number of other related activities should be considered in the reactor design, including: (1) examining the potential effects of flow dispersion in the reactor, and (2) applying a groundwater flow model to examine the effects of the proposed PRB on piezometric heads and gradients. Finally, evaluating and mitigating potential adverse effects of the PRB reactions on downgradient groundwater chemistry should be an integral part of the reactor design.

Determining Reactor System Residence Time. The relationship between the residence time of contaminated groundwater in the PRB and the rate of the contaminant degradation reactions is of critical importance in designing the PRB. The necessary residence time depends on the influent contaminant concentration, the required effluent concentration, and the reaction rate, which in turn may depend on the seepage velocities through the PRB.

In simplest terms, the mean residence time in the PRB reactor can be calculated by dividing the pore volume of the PRB by the discharge through the PRB. For degradation processes that can be described in terms of first-order reaction kinetics, the residence time necessary to reduce concentrations a given amount can be calculated using:

$$N^{1/2} = [\ln (C_{effluent} / C_{influent})] / \ln (1/2)$$

where $C_{effluent}$ is the concentration at the effluent, or downstream side of the reactor; $C_{influent}$ is the concentration at the influent, or upstream side of the reactor; and $N^{1/2}$ is the number of half-lives required.

Reduction in effluent concentrations can be achieved by either increasing reaction rates or lengthening retention times. In some cases, it may be possible to increase the reaction rate by increasing the surface area of the reactive material or by using a higher proportion of reactive material in the reactive porous mix. Alternatively, longer residence times can be obtained by decreasing the discharge through the reactor, which causes a corresponding reduction in the size of the capture zone, or by increasing the size of the reactor.

The reactor volume can be increased by making the reactor wider (or the wall thicker). In a funnel and gate design, gate thickness (in the direction of flow) has a minor effect on discharge through the gate, so retention time increases almost linearly with gate thickness. The ultimate manifestation of increasing gate width is the resulting continuous wall PRB design, where the wall width exceeds the plume dimension. At this point, further increases in the reactor width causes no further increase in retention time. Long residence times may also be achieved by completely surrounding a contaminant source zone with cutoff walls, except for a gap that contains a PRB in the downstream wall. In general, capture zone size and residence time are inversely related.

It should be noted that plumes are highly variable in concentration (by orders of magnitude) and often occupy heterogeneous media. For this reason, calculation of average residence time across a plume/PRB system probably is not of much value for design. Of greater relevance to design is the "worst case" combination of plume concentration and groundwater velocity, yielding the greatest mass flux of contaminant which must be addressed by the PRB.

Treatability Studies for Reactor Design. Typically, reactor system design is based on results from treatability studies which help in estimating the reaction rates achievable in the field. Standard treatability test protocol involves batch reaction tests and lab or field flow-through column tests. At this stage in PRB development, no standard treatability test protocols have been developed. However, a number of references contain test methodology applicable to various treatment approaches (for example, Gillham and O'Hannesin, 1993; Matheson and Tratnyek, 1994; Burris and Antworth, 1992; Xu and Schwartz, 1992; Morrison and Spangler, 1993).

Prior to initiating the treatability studies, a concept of the reaction process should be known. The concept should consider the type of reactions involved, the sites where various reactions take place (i.e., on reactive particle surfaces; in solution, etc.), and the likely kinetic rate limitations. One also should consider whether certain geochemical factors may affect reactivity, such as dissolved oxygen concentration, pH, Eh, temperature, and/or concentration of various anions and cations that may compete for reaction sites. Also, the model should consider the effect of initial contaminant influent concentration on the rate of reaction.

In general, to ensure representative results, laboratory treatability studies in support of PRB design should be conducted with actual groundwater taken from the application site or, if necessary, with a synthesized contaminated groundwater that has been matched to the site. When taking field samples, it is often not possible to avoid a degree of sample disturbance in the course of transport to the lab. For example, dissolved oxygen incursion, Eh and pH shifts, and temperature variation may cause the sample to be less than completely representative of the *in situ* groundwater. The reaction process concept can be used to predict the potential unrepresentative elements and help in interpretation of results.

The designer should also consider the potential for competing pathways or mechanisms of degradation. Of special concern is the potential for alternate degradation pathways to form intermediates that are more toxic than the contaminants originally of concern (Burris, 1995).

Batch reaction tests involve reacting a known quantity of contaminated groundwater with a known quantity of reactant in a closed container. The container typically is agitated to facilitate contact between groundwater and reactant. A series of tests may be performed in which the proportion of groundwater to reactant is varied and reaction times are also varied. Initial and final contaminant concentrations are compared to evaluate important measures of reactivity (e.g., degradation half life, sorption isotherms, etc.) useful for reactor design. Reactivity is expressed in general terms (such as mass of contaminant sorbed per mass of reactant).

Flow-through column tests have been applied both in the laboratory and in the field in support of PRB design. These tests are conducted by packing a column with the reactive media and passing the contaminated groundwater through the column. Influent and effluent concentrations are recorded to evaluate the effectiveness of treatment. Column tests can be set up to directly model the flow-through process in a PRB. Flow rate can be chosen to simulate groundwater velocity and reactor residence time, and the column length can be scaled to mimic the reactive wall dimension. In some cases, a number of researchers have devised field columns or canisters that can be operated at the site, circulating actual contaminated groundwater pumped from wells in the plume. A number of such ex situ tests are discussed in Section 11.3.

Geochemical interactions between the reactant and the natural groundwater system can affect the performance of PRBs. Buffering can affect the solubility of solid and aqueous species. Precipitation of minerals can cause plugging at the entrance of the reaction zone, porosity reduction within the reaction zone, thereby inhibiting the performance of the PRB (Mackenzie et al., 1995). One purpose of performing column tests is to identify such potential problems in advance.

Detailed micro-scale characterization of reactants, as an adjunct to standard treatability testing, may contribute to the understanding of the mechanisms responsible for the degradation of organics and/or fixation of inorganics (Paulson and Petrie, 1995). Tools available to perform the detailed characterization of reactant materials include: optical light microscopy, scanning electron microscopy, electron probe microanalysis, and x-ray diffraction analysis. Since oxide layers and reaction rims on the surfaces of the barrier materials can have significant influences on reaction rates, micro-scale characterization studies also provide information on PRB long-term performance factors such as precipitate formation, effective barrier life, and potential plugging problems.

<u>Effects of Flow Dispersion in the Reactor</u>. Emplacing mixtures of reactive materials that are chemically and physically homogeneous is difficult. As reactions occur, mixtures that were homogeneous initially can develop preferential flowpaths over time. For these reasons, predicting the long-term efficiency of contact between contaminated groundwater and reactive material is difficult. Diffusional processes or flow in secondary and higher-order channels may be sufficient to utilize the majority of reaction sites, but mass transfer rates may become too slow to efficiently reduce contaminant concentrations. Since the performance of PRBs is scale dependent, accurate assessment of reaction efficiency is best derived from field data. Unfortunately, performance of a field pilot in advance of every PRB project is not necessarily practical or economically justified. As an alternative, safety factors (e.g., increasing PRB wall thickness to increase residence time beyond the design requirement) can be included in the design as a way to account for

uncertainties such as dispersion.

Effects of the Reactor Medium on Groundwater Quality. Barrier reactants may have a deleterious effect on downstream groundwater quality. For example, the use of hydrated lime to precipitate metals is likely to produce high calcium concentrations and increase the pH of the groundwater. The interaction of some reactive materials can also affect the indigenous microbial consortia. Incomplete reactions may produce reaction degradation products that are more toxic than the original contaminants. Such effects and their implications should be considered in the course of developing a PRB design.

Life Span Limitations and Barrier Fate. In sorption barriers, contaminants will continue to migrate through the sorption zone until released at the downgradient side. This process, called the chromatographic effect, will limit the life-span of the barrier. Precipitation barriers have a similar limitation in that some contaminant-containing minerals will redissolve after the reactive material or plume is expended. These limitations can be overcome by emplacing greater amounts of reactive material than required throughout the compliance period. The barrier material could be removed and disposed or the metals reprocessed; or the contaminants could be stabilized in place (e.g., by aging) so that they would no longer be mobile. If the risk management goals for the site can not be achieved by managing the contaminants in place, it may be necessary to excavate them or remove them by *in situ* leaching methods (Morrison and Spangler, 1995). For PRBs designed to degrade contaminants, these issues are less important.

The ultimate fate of sorption/precipitation barriers is a regulatory issue that may be addressed by using risk-based versus cost considerations. In some cases, the contaminants in the barrier may be transferred above ground for recovery or disposal. In other cases, the reactive material may be retrievable and the entire matrix recovered. The PRB may be stabilized so that the contaminants are permanently fixed within the reactive material. When appropriate, the barrier may be left in place strictly on the basis of a risk-based exposure assessment. However, if stabilized contaminants are left in place, certain states (e.g., Texas) may require some form of deed recordation under current rules.

11.2.4 Constructability and Emplacement Methods

As described in the previous section, treatment of contaminated groundwater using permeable reactive barriers involves the capture, treatment, and release of the treated groundwater. The contaminant plume must be captured and hydraulically controlled so that it passes through the permeable reactive zone. Following treatment in the permeable reactive zone, the treated groundwater is returned to the aquifer. Proper construction and reactive zone emplacement methods are critical to achieving these objectives. Among the potential pitfalls of the construction process are: (1) aquifer formation disturbance and/or reactive zone pluggage, resulting in impeded flow through the permeable reactive zone; (2) emplacement limitations resulting in incomplete plume capture and potential flow bypass; and (3) production of large volumes of excavated materials (i.e., soil, debris) which may be contaminated and require treatment or disposal.

There has been relatively little experience with the actual construction of permeable reactive barriers to date. The few existing systems have been shallow (less than 30 feet deep) and have been constructed using standard design and construction approaches. Emplacement below a 30 foot depth is unproven to date, although several potential approaches have been proposed. The cost of emplacement will increase with depth and may limit the extent to which PRB technology is put into practice.

Various methods for constructing permeable reactive barriers are discussed in this section. These are categorized based on whether they would be used for reactive wall emplacement (for example, a continuous wall PRB) or to form an injected treatment zone PRB. Funnel and gate PRB construction is not discussed specifically in this section since much of the construction involves low hydraulic conductivity barrier wall emplacement (discussed elsewhere in this volume). Approaches for continuous wall PRB emplacement include:

- standard excavation methods,
- specialized trenching methods,
- jet grouting technology,
- mandrel-based technology,
- vibrating beam technology, and
- soil mixing.

Approaches for emplacement of injected treatment PRBs include:

- fluid injection through wells, and
- hydraulic fracturing.

The following discussion is intended to reflect the state-of-the-art as currently applied in the field or being considered for near-term application.

a. Continuous Wall PRB Emplacement. Standard Excavation Methods. After excavation of a trench using normal construction methods, the trench is backfilled with the permeable treatment media. A geotextile filter fabric may be placed adjacent to the trench wall prior to backfilling to control invasion of soil fines into the treatment media. Various methods have been devised to maintain an open trench until backfilling is completed. Readers are referred to Section 3 of this volume for a more complete treatment of construction methods for installing barrier walls.

Modifications to the standard excavation approach have been developed to address the need for stabilizing the trench. The *EnviroWall*™ groundwater barrier and pass through system was used recently to construct an interceptor trench system at the U.S. Department of Energy's Savannah River Plant in South Carolina (Phifer, 1995). This method utilizes excavators and movable guide boxes to maintain a consistent open area in the trench until various materials such as barrier geomembrane and permeable backfill can be emplaced. The system allows for up to 120 feet of wall to be constructed at a time to depths up to 40 feet. The guide boxes are such that geomembrane barrier or geotextile filter can be placed alongside permeable backfill. Barrier and "pass through" zones can be positioned to manipulate groundwater flow to discrete openings on the downgradient side of the trench (similar to a funnel and gate approach), while providing for treatment within the reactive media core of the trench. The Savannah River interceptor trench was constructed to a depth of 30 feet using this technique. Inert sand backfill takes the place of the reactive media core in this installation. The interceptor trench is 240 feet long and consists of two 120 foot long wing sections with a central groundwater collection assembly toward which groundwater is funneled by the wings.

The University of Waterloo (Gillham, et al. 1995) constructed a permeable wall by first driving steel sheet piling to the desired depth (about 20 feet) around the intended wall location, and excavating the soil from within the sheet piling. The excavation was backfilled with an iron-sand (22% iron by weight) mixture. The sheet piling was removed after backfilling was completed, leaving a wall about 18 feet long and five feet wide.

A similar reactive wall was constructed by excavating within pre-driven steel sheet piling (Warner, et al., 1995). The final design of the system consisted of a 40-foot long by 20-foot deep by 8-foot wide permeable wall (four feet of granular iron sandwiched between two-foot thick sections of pea gravel), upgradient lateral hydraulic barriers of soil-cement-bentonite slurry walls (about 300 feet long on one side and 225 feet long on the other), and a downgradient steel sheet pile lateral barrier extending about 20 feet beyond one end of the permeable wall. The permeable wall section was constructed by driving sheet piles around the perimeter of the intended excavation, digging out the soil with a backhoe, placing steel sheet piling in the excavation to create three compartments, backfilling the trench compartments with permeable material, and extracting the piles.

Standard excavator-based construction has several advantages. It utilizes readily available equipment with proven ability to dig in a broad spectrum of soils ranging in difficulty up to shallow weathered bedrock. In addition, since excavation permits the operation to be viewed, treatment media can be placed more precisely, allowing segregated placement of different treatment media, treatment and filter media, or treatment media and barrier (sheet piling or geomembrane), within the same trench. Limitations to this approach are that the width of the trench is a function of the desired depth of excavation and the width of the boom, stick, and bucket of the excavator and the large costs of constructing trenches to great depths. The excavated contaminated soil may require disposal or, alternatively, a special allowance may be needed to return soil to the trench (under current U.S. EPA rules). Excavation may create a large soil surface area with higher levels of personnel protection required at sites involving volatile contaminants, resulting in higher costs and reduced productivity.

Specialized Trenching Methods. Specialized excavation methods using trenching machines can be adapted to emplace permeable reactive walls. Trenching machines come in a wide variety of designs and sizes. Designs include "chain saw" and "circular saw" devices with depth capability varying from several feet to over 100 feet and trench widths varying from less than one foot to several feet. There are plans by U.S. EPA and the University of Waterloo to construct an iron-filled permeable wall using specialized trenching methods at a pilot site near Elizabeth City, North Carolina in 1996 (Puls, et al., 1995).

The primary advantage of a trenching machine is that it can productively trench in a wide variety of materials ranging from soils to soft rock. Trenching machines adapted for interceptor well construction in particular can be highly productive since the trench shoring moves along with the cutter head. Production on the order of one lateral foot of trench per minute to an excavated depth of 20 feet can be achieved. High production rates may translate directly into low cost for depths where this method is applicable. Disadvantages are similar to the standard excavation approach.

Jet Grouting Technology. Jet grouting has been used typically for emplacing cement grouts to form low permeability barriers and is not believed to have been used to date for permeable reactive wall emplacement. Application to reactive wall emplacement would require development, particularly for identifying suitable carrier agents and nozzle configurations to convey treatment media to the subsurface. However, the technique can be applied selectively at discrete depths and in a variety of soils, including cobble and debris-bearing soils. Also, it appears practical for avoiding underground utilities by proper positioning of the drill holes. The technique has the potential for rapidly emplacing large amounts of reactive material in a relatively short time. Disadvantages include the relative lack of control during emplacement and the inability to directly monitor construction.

Mandrel-Based Technology. Mandrel-based emplacement using vibratory hammers is commonly used to install prefabricated vertical drains for soil consolidation. These drains are constructed of geotextile fabric wrapped around a plastic spacer to create a thin conduit for groundwater flow. The drains are installed by inserting the drain into a steel emplacement mandrel (a rectangular hollow steel casing). A drive shoe/anchor is attached to the drain and placed over the leading edge of the mandrel. A vibratory hammer then drives the mandrel into the soil to the desired depth. The drive shoe prevents soil from entering the mandrel during emplacement and securely anchors the drain. The process is repeated at the next location.

To form a permeable reactive wall, the drain can be filled with reactive media or eliminated, with the mandrel being used simply as a tremie device. In the latter case, the treatment zone materials are placed through the mandrel as it is extracted thus filling the soil void. Mandrel emplacements can be arranged in panels to form a more or less continuous permeable barrier perpendicular to groundwater flow.

This approach was used to install treatment zones in the first phase of the *Lasagna*™ project conducted by a consortium of Monsanto, DuPont, and General Electric at the U.S. Department of Energy's Paducah, Kentucky plant. The *Lasagna*™ process uses a combination of *in situ* electrodes and *in situ* treatment zones (i.e., permeable treatment panels) to remediate soils of low hydraulic conductivity. In the Paducah pilot, two 15-foot long electrode rows were placed ten feet apart to a depth of 15 feet. Four rows of treatment zones were placed equally-spaced in between the electrodes. An electric field was applied to drive TCE-contaminated water into the treatment zones by electroosmosis. The treatment material in this case was activated carbon contained within 18-inch wide strip drains. Mandrel size was four inches by 21 inches in outer dimension. After driving the mandrel to the desired depth (15 feet), the carbon-filled strip drains were inserted into the mandrel cavity. The mandrel then was pulled out leaving the drive shoe and treatment panel in the ground. The treatment panels were staggered and overlapped a few inches so as to approximate a continuous wall. This approach proved satisfactory within the context of electroosmotic flow.

A Phase II test is being planned at the Paducah site, involving placement to greater depth (about 45 feet). The mandrel approach will be used, but a reactive iron-clay mixture will be substituted for the carbon treatment zone material. The treatment zone mixture will be emplaced directly as a thick slurry using the mandrel as a tremie device. The emplacement is expected to be completed in the first half of 1996.

Advantages of this method are that virtually no waste soil is generated and worker exposure is limited. Since conventional vertical drains have been emplaced to depths up to 190 feet using this technology, there is good potential for emplacing treatment media to similar depths.

Since typical mandrel sizes range in outer dimension from 1 inch x 5 inches up to 4 inches by 21 inches, resulting wall thicknesses are relatively thin. This may limit the applicability of the technique to PRBs requiring little wall retention time or to sites having very slow groundwater seepage velocities. Soil conditions must be conducive to driving the mandrel. Cobbles and debris may cause refusal. In addition, since the soil is compacted during mandrel emplacement, there is potential for formation disturbance. While this is not an issue with respect to electroosmotic flow, it could be a limitation in adapting the technique to a passive flow application.

Vibrating Beam Technology. The vibrating beam technology is used commercially for emplacing interlocking geomembrane panels for vertical containment barriers at hazardous waste sites. The technique appears to be readily adaptable for emplacement of permeable reactive walls. As with the mandrel emplacement technology, virtually no waste soil is created, and worker exposure is minimized. The overlapping drive pattern allows construction of a more or less continuous trench. Wall thicknesses are limited by the thickness of the I-beam.

As with the mandrel emplacement method, a potential disadvantage is that soil conditions must be conducive to driving the vibrating beam. Cobbles, debris, or other impenetrable materials can cause the beam to meet refusal. Furthermore, emplacement accuracy may be of some concern since the drive shoe can encounter hard materials which can push the beam off the intended path, particularly at depths greater than 40 feet.

Soil Mixing. The soil mixing process has been used commercially for several years as a way to carry out solidification and stabilization of soils and sludges *in situ*. This method employs soil augers of various sizes to drill into the soil and inject and mix reagents.

Soil mixing could be used to emplace reactive material into an aquifer in such a way as to create a permeable wall. Equipment is commercially available (e.g., GeoCon, Inc. and others) which can penetrate weak soils up to 40 feet with eight to 12-foot diameter augers, or up to 150 feet with three-foot diameter augers. The soil mixing technology has been used to form soil-cement groundwater cutoff walls by augering in an overlapping, offset pattern (Rumer and Ryan, 1995).

A pilot-scale version of the soil mixing approach was performed recently by the U.S. EPA and University of Waterloo at the Elizabeth City, North Carolina test site. Relatively small diameter hollow stem augers (six-inch inner diameter) were used with a standard geotechnical drill rig to emplaced a series of 21 permeable reactive cylinders into a shallow aquifer in a fence pattern. The fence posts were located on roughly 1.0 to 1.5-foot centers and arranged in three rows of seven each, oriented transverse to groundwater flow direction. The treatment media consisted of reactive iron filings, native aquifer material, and coarse washed silica sand. The estimated diameter of the cylinders was eight inches and they were installed to depths ranging from 10 feet to 26 feet below ground surface. Performance of this system is described in section 11.3.5.

b. Injected Treatment Zone PRB Emplacement. As discussed previously, injected treatment zone systems differ from the reactive wall approach in the method of emplacement and the anticipated final form of the treatment zone. Injected systems involve injection of fluids or fluid/particulate mixtures to distribute a treatment zone through a target strata (i.e., aquifer). The resulting zone may be oriented vertically or horizontally, but generally will be placed perpendicular to the primary groundwater flow direction. There has been less field experience using this approach than with the permeable wall approach. The primary means considered for emplacing injected treatment zones are: (1) injection of liquid solutions or gas through wells with subsequent precipitation of treatment media on the aquifer matrix, and (2) hydraulic fracturing to delivery treatment media either as a liquid solution or as particles entrained in a carrier fluid.

Injection Through Wells. In this method, a dissolved or gaseous reactant is injected through a series of wells and precipitates or sorbs onto the target aquifer matrix. The treatment zone may be distributed throughout the plume or as a discrete zone emplaced along the plume's leading edge. The distributed reactant can perform its treatment function either by reacting immediately with plume constituents or by reacting over time as the plume migrates through the treatment zone.

Several concepts have been tested in the laboratory, some of which were described in the previous section. Rust Geotech of Grand Junction, Colorado has developed a technique for injecting a ferric chloride solution through wells to react with alkaline aquifer materials to form a ferric oxyhydroxide treatment zone distributed on the aquifer matrix (Morrison and Spangler, 1995; Morrison, et al., 1996). Iron hydroxides are excellent adsorbents for dissolved metals. A field demonstration of this approach was designed and wells constructed at an abandoned uranium mill tailings site in the western U.S., but the project was discontinued before injection due to regulatory concerns.

The use of hydrogen sulfide in aqueous solution to treat mine wastes in Wyoming has been reported (Marozas, et al., 1995). Hydrogen sulfide is used to reduce and precipitate dissolved metals as sulfide minerals. A demonstration by Sandia National Laboratory at Kirtland Air Force Base in New Mexico involved injection of gaseous hydrogen sulfide through wells into the vadose zone to precipitate metals present in soil pore water. Other reducing agents may be used in a similar manner. A field test involving sodium dithionite injection is being planned to reduce and precipitate dissolved chromium at the U.S. Department of Energy's Hanford, Washington site.

An injection process has been proposed to deliver a cationic surfactant solution into an aquifer to form a sorptive barrier (Burris and Antworth, 1992). The injected surfactant molecules would become associated with exchange sites on the aquifer material. With appropriate arrangement of injection wells, a continuous sorptive barrier could be created across the plume to remove sorbable dissolved organics. Since the chromatographic effect will limit the life-span of a sorptive barrier, it was further proposed that bioremediation amendments be injected upgradient of the treatment zone to promote degradation of the sorbed constituents within the zone.

Although there have been no definitive field trials of the injected treatment zone approach to date, observations of naturally occurring permeable barriers are useful in assessing the potential effectiveness of this technology. The concept of a chemically-reducing permeable barrier has its origins from observations of ore-forming processes. Ore deposits have formed at locations where oxidizing, metal- and uranium-bearing groundwaters encountered a reducing horizon in the subsurface. These ore-forming systems can be thought of as natural analogs to injected permeable barriers.

The potential advantages of the injected treatment zone approach are: costs should be relatively low since trench construction is eliminated; treatment zones could be emplaced to relatively great depths limited only by the ability to drill; worker exposure and waste generation can be minimized, since an open excavation is not involved; and the method lends itself reasonably well to follow-up emplacement through additional injection to address untreated flow paths.

A principal concern with this approach relates to the questionable reliability of injection to create a homogeneous treatment zone. Emplacing mixtures of reactive materials to form a well distributed treatment zone may prove difficult. Thus, predicting the efficiency of contact between contaminated groundwater and reactive material could be difficult. Furthermore, chemical and microbiological reactions that occur in the subsurface are not well understood and could affect performance. This emplacement method has a greater potential to allow untreated flow through preferential flow paths than the constructed wall approach. Finally, injected solution chemistry may adversely affect aquifer water quality. As an example, injection of ferric chloride to precipitate iron oxyhydroxide will release calcium and chloride ions which may have to be pumped out of the aquifer.

<u>Hydraulic Fracturing</u>. The primary purpose of hydraulic fracturing is to enhance seepage flow in the vicinity of a well. A secondary application is to deliver solid compounds into the subsurface (Murdoch, et al., 1991). In this latter application, the fractures are filled with granular compounds to form an *in situ* treatment zone. Fine-grained reactive iron, encapsulated sodium percarbonate (oxygen source for biodegradation), and various sorption agents (e.g., activated carbon) have been considered.

Since hydraulically-induced fractures generally have a horizontal to gently-dipping orientation, flat-lying treatment zones could be created to address vertically downward migration in the vadose zone, within confining layers beneath contaminated aquifers, or in fractured bedrock systems.

A detailed description of the hydraulic fracturing process using fluid injection as applied in remediation can be found in Murdoch, et al. (1991). Major issues affecting the choice of a carrier agent for treatment zone formation include the ability to transport the granular material into the fracture during propagation and avoidance of agents which could impede flow through the zone after emplacement. Biopolymer gels, such as guar gum and xanthan gum, are biodegradable and have been proposed for remediation applications. Hydraulic fracturing at shallow depth (ten to 40 feet) have created propped fractures generally less than one inch thick and 20 to 40 feet in diameter.

No record of field application of fracturing to form treatment zones could be found to date. However, D. Marcus of EMCON Associates (Burbank, California), at a recent meeting of the Remediation Technologies Development Forum, described a plan for delivery of a reactive zinc-based proppant through fractures in a TCE- contaminated bedrock aquifer. In this case, hydraulic fracturing is being used to open and prop existing fractures in the bedrock. Zinc was selected for its dechlorinating ability and its relatively low density when coated on an inert proppant core. Guar gum is planned as the carrier agent. Field work is expected to begin at a California test site in January 1996.

11.3 FIELD PERFORMANCE OF THE TECHNOLOGY

Relatively few field demonstrations of permeable reactive barriers have been undertaken to date. The earliest known application of this concept used limestone-filled permeable barriers installed by the U.S. Bureau of Mines to control acid mine drainage (Kleinmann, et al., 1983). However, long-term performance of these trenches has not been well documented. In response to the rising interest in the potential application of PRBs in remediation applications, a handful of well-documented, small scale field experiments have been initiated. However, one small commercial scale application (a reactive iron wall for TCE and DCE destruction) for which results were available was undertaken in early 1995 in Sunnyvale, California. The early performance of this PRB has been encouraging. However, there has been insufficient period of record to evaluate long-term effectiveness. A summary of the documented field experience to date is given below.

11.3.1 Borden, Ontario Reactive Iron Barrier

The University of Waterloo project consists of a permeable reactive iron barrier placed in the path of a chlorocarbon plume (TCE, PCE, and chloroform) at the Canadian Forces Base Borden, Ontario site (Gillham, et al., 1995). The wall, which was constructed in 1991, had been in continuous operation for about four years at the time the workshop was held. The permeable wall was constructed about 15 feet downgradient from the contaminant source zone in an aquifer comprised of fine to medium sand (see Figure 11-2). Average groundwater seepage velocity is about 0.3 feet per day with flow direction varying seasonally by up to 30 degrees.

Laboratory column tests using reactive iron indicated half-lives of about 15 hours could be expected for both TCE and PCE. Since TCE degradation to less than the MCL of 0.005 mg/l would require about 16 half lives (ten days), the minimum wall thickness was set at three feet. The actual constructed wall was five feet thick and 18 feet long (transverse to flow). The reactive media backfill consisted of 22% by weight iron grindings (collected from a local machine shop) and 78% concrete sand. The sand was included to ensure greater hydraulic conductivity in the wall than the adjacent aquifer. In spite of a dense sampling network (a total of 348 separate sampling points), definitive monitoring was hampered to some degree by the narrow plume dimension and the changing nature of the flow direction.

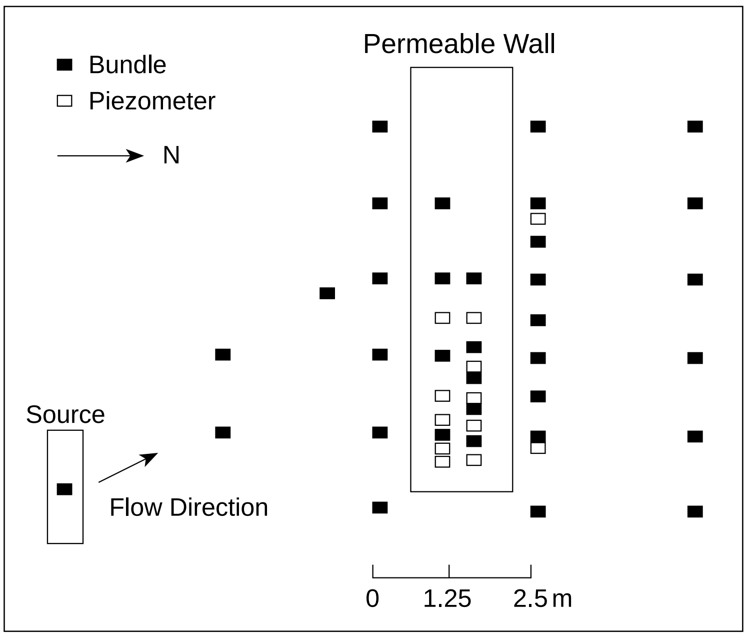

Figure 11-2 Plan view of the Borden test site showing the contaminant source, the reactive wall, and the monitoring network (Gillham, et al., 1993)

Monitoring during the four years since wall construction has shown consistent removal of about 90% for TCE and 86% for PCE. The downgradient concentrations are, however, about three orders of magnitude greater than the MCL. Increasing chloride concentrations (ranging from less than 10 mg/l up to over 100 mg/l) across the wall is evidence that TCE and PCE are being dechlorinated (see Figure 11-3). In addition, DCE isomer formation and disappearance has been tracked across the wall. The principal DCE isomer formed is cis 1,2-DCE at a peak concentration of 2 mg/l at a distance of 1.5 feet into the wall. The DCE concentration declines to about 0.2 mg/l at the downgradient monitor point. No vinyl chloride production has been detected (see Figure 11-4).

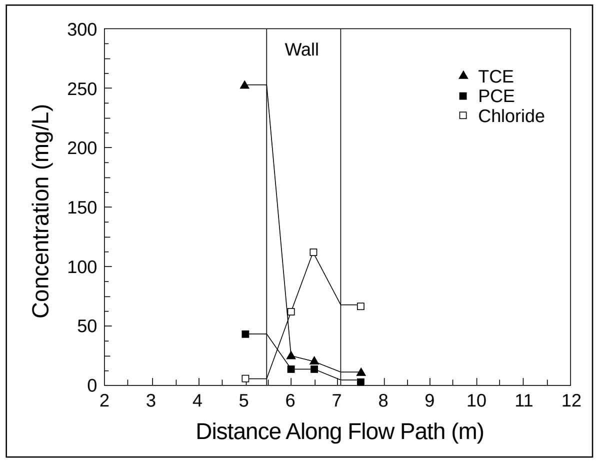

Figure 11-3 Maximum PCE, TCE, and C1⁻ concentrations measured across the reactive wall 199 days after installation of the wall (Gillham, et al., 1993)

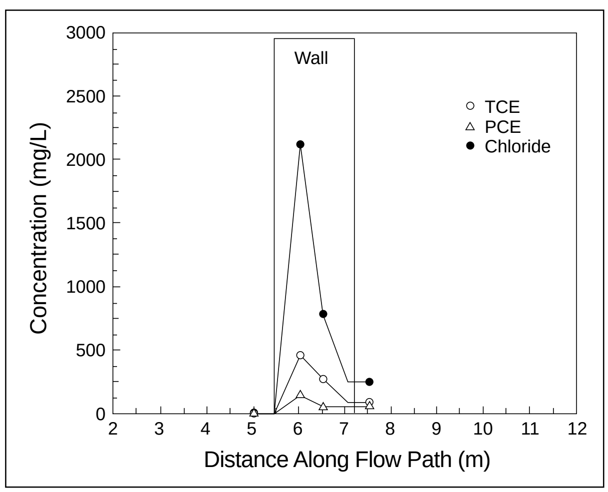

Figure 11-4 Distribution of chlorinated degradation products across the wall, 299 days after installation of the wall (Gillham, et al., 1993)

Core samples collected from the wall after a year of operation for biological study, indicated no evidence of biological growth and no chemical precipitation or alteration of the iron (Matheson, 1994). Precipitates were not detected by x-ray diffraction or scanning electron microscopy. Although losses of 185 mg/l of calcium and 82 mg/l of bicarbonate were noted across the wall, any precipitates that were being formed were not detectable. There was no evidence that performance was being affected. Additional sampling of wall material was planned at the conclusion of the test. The levels of TCE and PCE destruction were below that expected, based on the initial laboratory column tests. Subsequent lab tests, designed to better simulate the conditions in the wall, suggest that a higher proportion of iron to sand might have resulted in more complete removal.

11.3.2 SGL Printed Circuit Site, Wayne, New Jersey

A test of reactive iron technology for chlorocarbon destruction at Wayne, New Jersey (EnviroMetal Technologies, Inc., Guelph, Ontario) is being monitored under the SITE Program of the U.S. EPA (Gillham, et al., 1995). Although the project has focused on on-site treatability testing of iron using an ex situ reactor, the experience gained is relevant to permeable wall technology.

Chlorocarbon contaminants (PCE and TCE) are present in eight to 12 feet of silty clay and the underlying fractured bedrock. Maximum concentrations measured in the groundwater are 50 mg/l and 3 mg/l for PCE and TCE, respectively. Much of the groundwater is believed to flow through a thin permeable zone at the overburden-bedrock interface. It has been proposed to install a drain at the overburden-bedrock interface to intercept the groundwater flow and direct it to a subsurface iron treatment zone. To evaluate the feasibility of treatment, laboratory column tests were conducted and an above-ground reactor demonstration initiated.

Lab column tests were conducted using water collected from the site and the commercial granular iron treatment medium being considered for use at the site. The collected water contained PCE ranging from 4 to 12 mg/l, TCE at 1 mg/l, and cis 1,2-DCE at 0.15 mg/l. No vinyl chloride was detected in the collected water. DCE and vinyl chloride were detected at intermediate points in the iron-filled column, but not in the effluent from the column. Observed half-lives using 100% granular iron were: PCE, 0.4 to 0.6 hours; TCE, 0.5 to 0.7 hours; DCE, 1.5 to 3.7 hours; and VC, 1.2 to 0.9 hours.

Design of the above-ground field test was based on a flow rate of two liters per minute and a maximum PCE concentration of 30 mg/l. Based on lab results, it was calculated that cis 1,2-DCE and VC concentrations as high as 3 mg/l and 0.3 mg/l, respectively, could be encountered. The design residence time of 24 hours for the field reactor was based on a New Jersey standard of 0.01 mg/l for 1,2-DCE in the effluent. Considering the two liters per minute flow rate, the packed bed reactor was sized to hold about 280 cubic feet of granular iron. It consists of an eight-foot diameter fiberglass tank filled with granular iron to a depth of 5.5 feet. Water from the subsurface tile drain flows into a sump from which it is pumped to the top of the reactor. Flow through the reactor is vertically downward. Ponded water is maintained above the iron surface to avoid invasion by atmospheric oxygen.

Concentration profiles for PCE, TCE, cis 1,2-DCE, after 30 days and 60 days of operation showed nondetectable levels in the effluent (Figure 11-5). In fact, disappearance of all three constituents was observed to occur roughly midway through the reactor. As shown in Figure 11-5, PCE and TCE influent concentrations were somewhat lower than those in the collected water used in the lab testing and assumed in sizing the reactor. Nonetheless, results appear favorable and indicate the reactor is performing as designed. Precipitate formation was observed at the top of the reactor but analyses of the precipitates had not been performed at the time of the workshop. Calcium carbonate, siderite, and possibly iron hydroxide formation were anticipated.

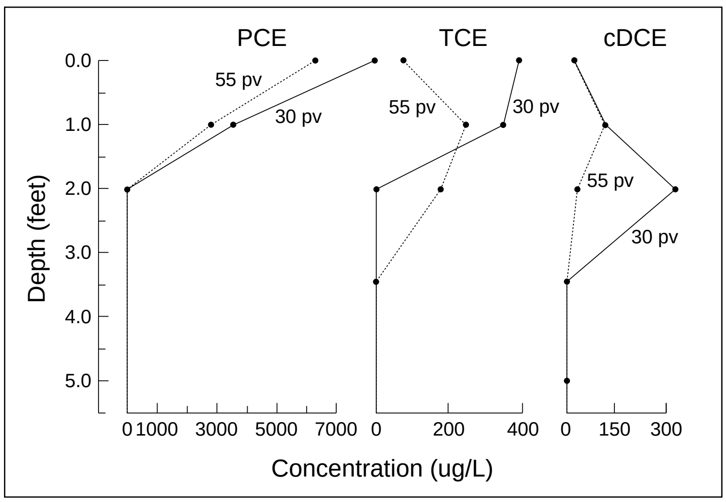

Figure 11-5 Concentration profiles of PCE, TCE, and cDCE after 30 and 60 days of operation of the New Jersey above-ground demonstration test (Gillham, et al., 1995)

11.3.3 Hill Air Force Base, Utah

Laboratory column studies and an above-ground field test using a small canister were completed at Hill Air Force Base in 1994 as part of the evaluation of a reactive iron permeable barrier application (Wray, 1995). The source of contamination under study is a landfill which received municipal waste between 1955 and 1967, and possibly waste solvents from aircraft maintenance operations at the base. The contaminant of concern is TCE. Maximum measured concentration is 18 mg/l with an average of 3 mg/l. The plume occupies about 69 acres and is located within the upper 25 feet of a shallow aquifer, the top of which is 30 feet below ground surface.

Laboratory column studies were initiated by the University of Waterloo in January 1994. Three column tests were completed, one with 100% granular iron, one with 50% iron and 50% silica sand (by weight), and the third with 100% silica sand as a control. Influent to the columns was site groundwater containing 3 mg/l TCE, 0.1 mg/l PCE, 0.05 mg/l cis 1,2-DCE, and 0.002 mg/l vinyl chloride. These experiments showed rapid degradation of TCE with a half-life of about 0.5 hours for the 100% iron column. Both cis 1,2-DCE and vinyl chloride were observed as intermediate reaction products. They were completely degraded in the 100% iron column but persisted in the effluent from the 50% iron column. Retention time in these columns was not provided.

The above-ground canister test was operated from June to October, 1994. The test used groundwater extracted from the site as the influent water. This water contained 2 mg/l TCE. The fiberglass canister (4.5 feet long and 1.0 foot in diameter) was filled with 100% Master Builder iron (Strongsville, Ohio), "Blend B, GX-27." About five inches of pea gravel was placed at each end of the reactor. Flow was upward through the canister and rates varied from 0.01 gallons per minute to 0.1 gallons per minute, corresponding to a retention time between about two to 22 hours.

Influent pH was stable at about 7.5 throughout the test; however, pH increased to above 9 during flow through the canister. Influent dissolved oxygen ranged between 4 to 6 mg/l. The majority of the TCE was degraded by the first sampling port, located about 8.6 inches into the canister. Concentrations of cis 1,2-DCE and vinyl chloride were observed to increase within the canister but were less than 0.001 mg/l in the effluent. Ethene and ethane were produced and accounted for 60% of the initial TCE mass in the effluent.

Gradual plugging of the canister occurred, as evidenced by an increase of pressure drop across the canister from 0.5 psi to 9 psi during the test. X-ray diffraction tests on iron samples taken from the canister indicated that iron and calcium carbonate compounds were precipitated and appeared to cause the plugging. About 5 grams of carbonate per 100 grams of granular iron were deposited, corresponding to a porosity loss of 14%. Remaining porosity loss was attributed to trapped particulates carried in the influent.

Implementation of a field scale treatment system is pending the resolution of a number of issues. First, a determination of whether carbonate precipitation would occur in an *in situ* barrier must be made, and if so, a remedy identified. Second, the best location of the field system is on off-site property for which access is not currently available. Third, the Superfund Record of Decision (ROD) for the site, which has been signed and currently does not include this remedy, would have to be amended.

11.3.4 Sunnyvale, California Semiconductor Site

This full-scale commercial application involved a permeable granular iron reactive barrier for destruction of TCE, cis 1,2-DCE, and vinyl chloride in groundwater. A low hydraulic conductivity soil-cement-bentonite and cement-bentonite slurry wall system was designed to route groundwater through the treatment zone. This system has received regulatory approval as a final remedy by state and local regulators, replacing an interim groundwater pump and treat system. Construction at the site was completed in February, 1995 (Warner, et al., 1995; Gillham, et al., 1995).

Site conditions consist of up to 20 feet of heterogeneous silt, sand, and clay overburden on top of a 65-foot thick clay aquitard. Groundwater in the overburden contains TCE (0.05 to 0.2 mg/l), cis 1,2-DCE (0.45 to 1.0 mg/l), vinyl chloride (0.1 to 0.5 mg/l), and CFC-113 (0.02 to 0.06 mg/l). Concerns over the heterogeneous nature of the overburden and variable seasonal flow direction were addressed in the barrier design and construction.

An above-ground pilot test (9 month duration) was performed using a six foot long by two foot diameter flow-through canister. The treatment medium contained equal amounts of granular iron and sand. Half lives determined from this test were: TCE, less than 1.7 hours; cis 1,2 DCE, 0.9 hours; vinyl chloride, 2.0 to 4.0 hours; and CFC-113, less than 1.6 hours. The water at this site is highly mineralized and, although precipitate formation was evident at the inlet end of the reactor, the rate of degradation remained relatively constant throughout the test.

The permeable component is about 40 feet wide and 20 feet deep. The eight-foot thick treatment zone consists of four feet of granular reactive iron sandwiched between two-foot thick upgradient and downgradient pea gravel sections, designed to distribute flow evenly through the iron. The system included upgradient slurry walls to guide groundwater flow laterally into the wall. The eastern wall also was extended with sheet piling a distance of 20 feet downgradient from the treatment barrier, thereby creating a "chute" which caused flow convergence downgradient from the barrier rather than within the treatment zone. As an added safety factor, the barrier was constructed with 100% granular iron (i.e., no sand mixture), even though the barrier was sized based on the above-ground test half life results.

The site is regulated under an order issued by the California Regional Water Quality Control Board - San Francisco Bay Region that requires quarterly groundwater monitoring. Cleanup standards are developed from California maximum contaminant levels (MCLs) with the lowest MCL being 0.0005 mg/l for vinyl chloride.

Results from two quarterly groundwater sampling events indicate that the concentration of total VOCs in groundwater about ten feet upgradient from the barrier was approximately 1.0 mg/l during the most recent sampling event with principal constituents of cis 1,2-DCE at 0.8 mg/l, vinyl chloride at 0.1 mg/l, and TCE at about 0.03 mg/l. Groundwater samples collected from performance monitoring wells within the barrier showed no VOCs above the detection limit of 0.0005 mg/l. Field water quality measurements in the barrier showed high pH and low Eh, indicating that the barrier is affecting groundwater in the expected manner. An assessment of hydraulic performance indicated the system is performing within design specifications.

The former above-ground treatment system and groundwater extraction wells have been removed from the property, and the formerly vacant site has been leased and is now in use.

11.3.5 Elizabeth City, North Carolina Field Study

This small-scale field test at Elizabeth City, completed in 1995, evaluated the *in situ* treatment of groundwater contaminated with chromate and chlorocarbons. The permeable treatment system was composed of a granular iron/coarse sand/native aquifer mixture installed as "fence posts" in a staggered design using hollow stem augers (Puls, et al., 1995; Blowes, et al., 1995). The project has been a cooperative effort by RSKERL, the University of Waterloo, and the Oregon Graduate Institute, with assistance by the U.S. Bureau of Mines.

The site is located at the U.S. Coast Guard Support Center near Elizabeth City in the coastal plain region of North Carolina. The site geology consists of coastal plain sediments with variable sequences of sand, silt and clay (Puls, et al., 1994). Groundwater flow velocity is variable with depth, with highest flows through a highly conductive layer at roughly 15 to 22 feet below ground surface. This layer coincides with the highest groundwater concentrations of chromate and chlorocarbons (TCE, cis 1,2-DCE, and vinyl chloride). While the test was not specifically designed to remediate the chlorocarbons, their concentration was monitored during the test. The water table ranges from five to seven feet below ground surface.

Sources of iron included low-grade steel waste stock turned on a lathe using diamond bits to produce 200 liters of turnings (source - Ada Iron and Metal, Ada, Oklahoma), with a size range from 1 to 10 mm, and heated cast iron in the form of iron chips (source - Master Builder's Supply, Streetsboro, Ohio), with a size range from 0.1 to 2 mm.

Twenty-one eight-inch diameter "fence posts" were emplaced on 1.0 to 1.5-foot centers in three rows oriented transverse to flow (Figure 11-6). In addition, 24 monitoring wells were installed within the roughly 60 square foot treatment zone. Most of these were 5/8-inch PVC wells with 1.0 to 1.5-foot long screens set between 14 to 20 feet below ground surface. In addition to these permanent points, temporary sampling points were used to increase spatial resolution of the data. Tracer tests using bromide to evaluate groundwater flow velocity through the fence region were performed prior to and following emplacement.

Results of the eight month long test are summarized in Table 11-4 (from Puls, et al., 1995). Untreated groundwater from the test area is characterized by chromate concentrations in the range of 1.0 to 3.0 mg/l, dissolved iron levels less than 0.05 mg/l, TCE+DCE about 6.5 mg/l, and Eh greater than 400 millivolts. Treated zones (represented by wells located within or downgradient of the fence posts) showed chromate decreased to less than 0.01 mg/l, dissolved iron increased to 1 to 20 mg/l, TCE+DCE decreased to

1.5 mg/l, and Eh decreased to the range of 0 to 200 millivolts. In addition, dissolved oxygen was shown to decrease from about 0.6 mg/l to less than 0.1 mg/l. Evidence of sulfate reduction was present, and a slight pH increase was evident. In all, the data present a picture of chemical reduction and precipitation of the chromate, reductive dechlorination of TCE, and accompanying oxidation of iron metal to form dissolved iron. It should be noted that while TCE was reduced on average by about 75% in the treated zones and vinyl chloride was reduced to less than 0.002 mg/l, the cis 1,2-DCE showed little change. This was attributed to the relatively short residence time in the "fence posts" (10 to 16 hours estimated per post).

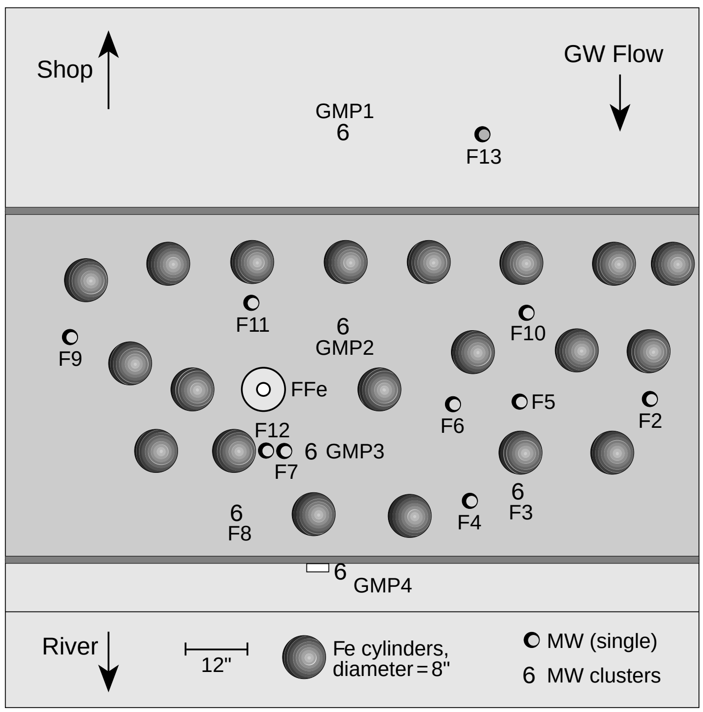

Figure 11-6 Plan view showing site monitoring locations at Elizabeth City, NC and location of reactive "fence posts" (Puls et al., 1995)

While considerable variation in hydraulic conductivity exists at the site, on average the iron-sand mixture was estimated to be about 75 times more permeable than the aquifer. While this increase was considered desirable from a standpoint of promoting flow through the fence posts, increasing plume capture, and mitigating pluggage due to geochemical changes, it also decreased residence time. This decreased residence time was not a limitation for chromate reduction, but affected the degree of chlorocarbon degradation. Tracer test data before and after fence emplacement showed increased groundwater velocity through the fence area. Post-emplacement velocities were approximately 1.0 and 1.7 feet per day at depths of 15.5 to 16.5 feet and 17.0 to 18.0 feet, respectively. Pre-emplacement velocities were 0.3 and 0.7 feet per day at these two locations. Much of the increase was due to temporal variation in recharge during the winter and spring months of the test.

TABLE 11-4 Monitoring Results (from Puls, et al, 1995)

Chemical Property	**Untreated Zones**	**Treated Zones**
Cr (VI)	1 - 3 mg/L	< 0.01 mg/L
Fe (II)	0	1 20 mg/L
TCE/DCE	6.5	1.5
Eh	> 400 mv (Pt)	0 - 200 mv (Pt)
pH	5.6 - 5.9	> 6.1
DO	~ 0.6 mg/L	< 0.1 mg/L
Sulfides/SO_4	absent/stable	present/slight decrease
Alkalinity	no change	increased

In summary, field test results showed nearly instantaneous reduction and removal of the chromate forming an insoluble trivalent chromium-iron hydroxide phase. The reduction of chlorocarbons was variable but followed the expected order of TCE>>vinyl chloride>cis 1,2-DCE. A Phase II study using a more traditional permeable reactive wall was in the planning stage for 1996 as of the workshop date. Improvement in chlorocarbon degradation was to be a focus of the Phase II study. A trenching machine was being considered for construction of the reactive wall.

11.3.6 Sudbury, Ontario Field Trial

A small-scale field test of a reactive wall system was initiated in October 1993 at the Nickel Rim tailings impoundment site near Sudbury, Ontario (Blowes, et al., 1995b). The reactive wall test made use of organic matter to promote sulfate-reducing conditions thereby treating acid mine drainage and associated dissolved metals (e.g., iron, calcium, nickel, and zinc).

The reactive wall test cell was excavated into an unconfined sand aquifer within a shallow alluvial valley about 250 feet downgradient from the Nickel Rim mine tailings impoundment. A contaminant plume of sulfate and dissolved metals formed downgradient from the impoundment in the shallow unconfined sand aquifer. At the location of the test cell, sulfate levels are in the 3000 to 4000 mg/l range while dissolved iron levels vary from 600 to 1000 mg/l. The Eh is moderately oxidized (300 to 500 mV), pH slightly acidic (4.5

to 5.5), and alkalinity is less than 50 mg/l (as $CaCO_3$).

The final dimensions of the excavated cell were about 4.8 ft long by 3.6 ft wide by 4.0 ft deep. The reactive mixture consisted of composted leaf mulch, pine mulch, and pine bark (as carbon sources), creek sediment (as a bacterial source), agricultural limestone (neutralizing agent), and coarse sand and gravel (to increase permeability in the cell). The cell was constructed by first driving steel sheet piling, excavating soil from within the piling and backfilling the void with the reactive mixture, then removing the piling.

Bundle-style piezometers were used to sample groundwater upgradient, within, and downgradient of the cell. Results within the test cell and immediately downgradient over a two year period following construction showed increased pH (to >6), decreased Eh (to <250 mV), and corresponding declines in sulfate (to <250 mg/l) and dissolved iron (10 to 50 mg/l). Nickel levels decreased from roughly 2 mg/l in upgradient groundwater to less than 0.005 mg/l in the test cell. In addition, hydrogen sulfide was measured at levels up to 5 mg/l within the cell.

Based on these positive results, construction of a full-scale test was completed about a week prior to the workshop in August 1995. The full-scale system was placed across a narrow margin (50 ft across) in the alluvial valley downgradient from the impoundment. Construction was completed with techniques similar to those used in the test cell construction, with the PRB keyed into underlying bedrock in both the valley bottom and the side walls. The reactive mix included organic matter obtained from several municipal compost sources mixed with pea gravel to obtain a hydraulic conductivity about an order of magnitude greater than the aquifer. Results from the full scale system were not available at the time of the workshop.

11.3.7 Future Planned Field Tests

The following briefly summarizes the status of some proposed tests.

a. Durango, Colorado Uranium Mill Tailing Site. The U.S. Department of Energy's Uranium Mill Tailing Remedial Action (UMTRA) program is conducting a pilot scale study through Sandia National Laboratory (SNL) at an UMTRA-owned site in Durango, Colorado. The focus of the study is to find effective, cost-efficient methods for remediating sites with uranium-contaminated groundwater (Marozas, et al., 1995; Marozas, 1995).

The use of reactive iron to reduce mobile radioactive metal species, such as uranium, was based on the studies on chromate conducted by RSKERL and the University of Waterloo. In the UMTRA pilot study, uranium and other inorganic contaminants will be treated with metallic iron in a subsurface reactive infiltration bed. Three different reactive iron reagents will be screened for efficiency in removing uranium, selenium, and molybdenum from mill tailing effluent. Since these are common constituents in uranium mill tailings, the results will be used to evaluate whether reactive barriers are an acceptable remediation option for groundwater cleanup at UMTRA sites.

The test site is located in the Bodo Canyon Disposal cell near Durango. Roughly 2.5 million cubic yards of tailings were relocated to the canyon from another location near Durango in 1990. The tailings are underlain by about 50 feet of silty clay, silt, and sand alluvium which overlie sandstone bedrock. Shallow groundwater occurs in the alluvium.

The Bodo Canyon disposal cell is designed to limit infiltration and, with time, the alluvium below the cell is expected to become dewatered. At that point, the unsaturated zone will attenuate seepage from the bottom of the cell. At present, fluids draining from the cell are collected by a gravity drain at the toe of the cell and are treated by active chemical flocculation in a lined retention pond. No contamination has been detected in downgradient groundwater. Tailing effluent contains an average of 6 mg/l of dissolved uranium, compared with a Colorado standard of 2 mg/l.

Like chromium, uranium generally is found in two oxidation states, the oxidized and relatively mobile U(VI) state, and the reduced and relatively immobile U(IV) state. Reactive wall chemistries have focused on chemical reduction and precipitation to stabilize uranium.

An engineered subsurface treatment system will be installed inside the existing lined retention pond. The pond is 60 feet by 80 feet and six feet deep. The treatment system will be placed in the path of seepage from the toe drain within a segmented area of the pond. The installed system will simulate flow and treatment of mill tailings groundwater, but in a controlled setting.

The new treatment system will distribute tailings effluent into one of four treatment zones that include two subsurface drain fields and two flow-through enclosed containers. In both systems, effluent will percolate by gravity through the treatment media into an underdrain. The treatment systems will be installed below the subsurface to prevent oxygen from entering the reaction zone and to prevent freezing during the winter.

Steel wool, steel wool treated with copper, and a metallic iron foam were selected for this study, based on laboratory testing. These reagents were selected for their environmentally benign nature, expected longevity without maintenance, availability, and cost.

b. Shiprock, New Mexico UMTRA Site. A permeable barrier design is being prepared for an UMTRA site located near Shiprock, New Mexico. The design uses naturally occurring microorganisms to achieve reduction and subsequent precipitation in the barrier (Thomson, 1995).

The Shiprock UMTRA site is located on an elevated river terrace approximately 50 feet above the flood plain of the San Juan River. Underlying strata consist of coarse alluvial material underlain by shale bedrock. The flood plain alluvium is highly permeable and consists of 15 to 30 feet of unconsolidated silts, sands, gravels, and cobbles overlying the shale. A steep escarpment separates the elevated terrace and the flood plain. Principal groundwater contaminants include sulfate (mean concentration 13,000 mg/l), nitrate (3300 mg/l), uranium (2.8 mg/l), selenium (0.12 mg/l), and radium-226 (0.1 picocuries/l).

Design of the permeable barrier is proceeding with laboratory studies and groundwater modeling. A recent study at the University of New Mexico developed a numerical model of groundwater flow in the flood plain. The USGS MODFLOW code and PATH3D were used to simulate head distribution and develop streamlines and groundwater travel times. The model was calibrated using water level data from a system of 30 monitor wells constructed in the flood plain. The calibrated model was used to consider a variety of gradient manipulation techniques using permeable or impermeable barrier locations to facilitate either flushing of the aquifer or remediation. The optimum design has not yet been determined but will include a trench-based permeable barrier keyed into the shale.

The permeable barrier will use an anaerobic bacterial consortium contained in a cellulosic substrate to produce sulfate-reducing conditions in the barrier. Substrates being considered include various agricultural waste products, e.g., straw, wheat chaff, corn husks, or sawdust. Substrate will need to remain in place for a period of at least several years to make the technology viable. Column experiments are planned to determine rate of substrate consumption.

The geochemical form of the reduced metals is important in determining their ultimate fate in the barrier. Well-ordered crystallized mineral phases are much less soluble than amorphous precipitates and are believed to oxidize more slowly in aerobic environments. A major objective of current studies is to characterize the solid phases formed during sulfate reduction. The two basic options available for management of the spent reactor wall are: (1) leave the barrier and the associated contaminants in place, or (2) remove the barrier and dispose of the contaminants in an appropriate facility. If the first option is selected, the ultimate fate of the metals will depend on the long-term geochemistry in the barrier. It is reasonable to expect that metals will remain in place as long as reducing conditions are maintained. Field sampling in uncontaminated regions of the Shiprock site indicate reducing conditions presently exit. The alternative is to remove the spent reactive wall once remediation is completed and the substrate has been consumed. This option has been proposed for the Shiprock site to remove all possibility of future release.

c. Gasworks Sites - Southern Germany. Two former gasworks sites in southern Germany are under investigation for application of sorptive barriers. Both of the sites are quite large (several centers of contamination of roughly 80 feet by 80 feet at each site spread over a large area), and broad groundwater plumes have formed with widths of 150 to 500 feet. Contaminants of concern include three to six ring polynuclear aromatic compounds (Teutsch and Grathwohl, 1995). The proposed reactive wall design utilizes the funnel and gate approach. Because of the expense in building a replenishable *in situ* reactor, the researchers wish to minimize the size of the reactors.

The two sites under consideration are located in valley settings above unconsolidated, heterogeneous sand and gravel aquifers. Concern about the effect of subsurface heterogeneity on the overall performance of the systems is being addressed by conducting small scale tracer tests to establish local flow conditions and velocities. Data from the tracer tests will be used in system design to set reactor sizes and configurations and to determine the feasibility of using a hanging gate design.

The reactor system under study consists of a number of high organic carbon materials including activated carbon, bituminous coal, and bituminous shale. The reactor construction employs a steel frame with individual cells (length of 7.9 feet and width in direction of flow of 1.7 to 2.6 feet). Two to three rows of the cells will be arranged perpendicular to flow direction. The system is intended to have flexibility in filling, monitoring, and replenishment of the reactor material. The steel frame is sized to allow individual cells to be emptied entirely. Future studies will focus on extension of the reactor system to combine a bioreactor with sorption.

11.4 EVALUATION OF THE TECHNOLOGY

The permeable reactive barrier technology is still largely in the developmental/ demonstration stage. However, as indicated by the previous sections, a great amount of work is being performed and the technology has been applied or proposed for a wide range of common groundwater contaminants including metals, chlorocarbons, and petroleum hydrocarbons. The technology has undergone an exponential increase in research and development activity over the past four years.

The major conceptual advantage of the permeable reactive barrier relates to the potential for remediating contaminated groundwater more cost-effectively than with pump and treat systems. This appears to be particularly true for sites which involve persistent sources of groundwater contamination for which remediation methods have not been demonstrated or are cost-prohibitive (e.g., DNAPL sites). At these sites, the PRB may actually outlast the plume life and eliminate the need for institutional controls over groundwater use at the site. On the other hand, PRBs are not yet considered a proven technology and a number of potential disadvantages and technical hurdles have been identified that will require further effort before the technology is applied routinely.

11.4.1 Cost Evaluation

The incentive to develop PRBs as a viable technology relates directly to the increasingly recognized limitations of pump and treat. A principal expected advantage of the permeable barrier technology is the greatly reduced operation and maintenance (O&M) costs. For example, consider the generic case for the reactive iron PRB applied to chlorocarbon plume containment presented by Schultz (1995). Conventional methods for treating chlorinated solvents in groundwater include liquid phase activated carbon adsorption and air stripping, often combined with vapor phase carbon treatment. A recent review within DuPont produced the following rules of thumb for the cost of such systems:

- installation costs between \$10,000 and \$30,000 per gallon per minute (gpm) of treatment capacity, and
- O&M costs ranging between \$5 to \$20 per 1000 gallons of treated water.

Costs for O&M of permeable reactive barrier systems will not be available until they have been operating for many years, but it is generally believed they will be lower than those for pump and treat. On the other hand, barrier emplacement can be estimated today based on costs to construct barrier containment systems, such as sheet piles and slurry walls. Assuming a relatively shallow plume depth of 20 feet, a corresponding emplacement cost of \$5 to \$10 per square foot of wall face (from slurry wall construction experience), and delivered cost for granular iron of \$400 per ton (as quoted by Peerless Iron of Detroit, Michigan), the construction cost of a reactive iron permeable wall was estimated to range from about \$19,000 to \$23,000 per installed gpm (somewhere in the middle of the range for pump and treat systems). If this estimate proves correct and if the expectation of much lower O&M costs prove true, this technology would be more cost-effective than pump and treat for chlorocarbon containment.

This cost comparison is supported by the Sunnyvale, California commercial PRB application. For this site, a detailed feasibility and cost study were completed prior to selecting the reactive iron PRB for TCE plume containment. It was estimated that the cost of removing an existing pump and treat system from the site and installing the permeable barrier would result in a savings of nearly \$5 million on a present value basis over a 30 year period. The cost advantage was attributed to reduced O&M, elimination of treatment system, discharge monitoring and reporting, and revenue gains from the ability to lease the property for commercial use once pump and treat surface facilities were removed. It should be noted that significant safety factors were incorporated into the Sunnyvale design. For example, the reactive iron zone thickness was doubled from the required two feet predicted by modeling, and the proportion of iron in the wall was increased from 50% by weight to 100%. Thus, four times as much iron was used in the wall than the least cost design optimum. Still, the PRB alternative had substantial cost advantage over the existing pump and treat system.

The cost to implement the PRB technology is directly related to the cost of the reactive materials; thus, it is highly dependent on the volume of groundwater requiring treatment. A decision must be made whether the PRB should be used as a final remedy for all affected groundwater at a site, or if it may be more cost effective as a source control with another treatment method (such as intrinsic biodegradation) used to remediate lower concentration fringe areas. An assessment of the cost effectiveness of the final remedy must also consider the costs associated with achieving a specified factor of safety for the final design. Other remedial alternatives may be more cost-effective than pursuing additional site characterization or employing ultraconservative design assumptions. Finally, the practicality of PRBs should be assessed considering site-specific factors and then compared to the feasibility of implementing other traditional and/or innovative technologies.

11.4.2 Other Conceptual Advantages

It should be possible to design and construct a "robust" PRB system with little or no operating equipment. A major problem with pump and treat systems is the tendency for equipment breakdown and system failure; e.g., recovery wells plug up and require redevelopment; treatment systems, especially air strippers, require frequent maintenance; and pumps fail. Hydraulic control over the plume is lost for some period whenever a pump and treat system goes off line for repair or maintenance. Depending on the aquifer characteristics, the length of time to re-establish the zone of capture can be a few hours to a few months. As a permanent physical structure, the PRB (if designed properly) should have the advantage of full-time operation.

The inherently slow nature of groundwater movement works to the advantage of the technology by allowing use of slow destruction technologies that are not practical in an above-ground treatment setting. The reactive granular iron technology is an excellent example. The rate of destruction is such that reactor residence time of several hours to a few days may be needed to achieve destruction to the parts per billion levels currently required for chlorocarbons. It is generally not practical to size an above-ground system to achieve such long residence times, whereas a below-ground PRB of the required size is quite feasible.

Other factors to consider are: (1) the inherent energy efficiency of the process, (2) the fact that little or no surface facilities are required, (3) the process is conservative of groundwater resources, and (4) the need for surface water discharge or groundwater recharge is eliminated.

PRBs offer other conceptual advantages when compared with low hydraulic conductivity physical barriers. For example, PRBs are not designed to exclude groundwater flow but instead work within the natural flow regime. This is particularly true of the continuous wall PRB design. It is less true of the funnel and gate concept. In any event, flaws and imperfections in the construction are not nearly as critical as in the construction of an impermeable barrier, where much higher pressure gradients may exist across the barrier. Also, in contrast to an impermeable barrier, there may not be a need to control upgradient surface water infiltration (e.g., by capping) to reduce flux.

11.4.3 Potential Disadvantages

The potential disadvantages associated with the PRB technology are primarily related to its early stage of development and the relative lack of field experience. First and foremost, the viability of this technology relates directly to its ability to provide long-term, relatively trouble-free service. Since there is no extended-term performance record beyond four years (e.g., the Borden reactive iron wall demo), the remediation community may be reluctant to put trust in the technology until a longer-term track record is established. With respect to long-term performance, are the following factors are of concern:

- possible plugging and reductions in hydraulic conductivity over time as a result of particle invasion, chemical incompatibility between reactant and host formation, or microbial activity, and
- possible gradual loss of reactivity as the reactant is either depleted or coated by reaction by-products (i.e., "rind" formation).

Theoretically, it should be possible to address each of these issues through laboratory evaluation, modeling, and proper design. However, the tools to complete such evaluations have not been completely developed and verified at this stage. There is a lack of tested and proven design procedures and protocols developed to this point. In addition, a protocol for effective site characterization in support of PRB design is still evolving. As discussed in Section 11.2, a different focus is needed in the site characterization from the standard investigations done at hazardous waste sites. Greater attention must be given to aquifer characteristics, local and seasonal groundwater flow patterns, and groundwater geochemistry.

Long-term analogs exist in nature for many of the processes being proposed for PRB application. For example, the use of redox barriers to precipitate trace metals has a direct corollary in the creation of uranium deposits in reducing zones in relative shallow sand aquifers of the western U.S. Study of such corollaries may lead to a greater level of assurance that PRBs can operate and remain geochemically stable for very long periods.

Another disadvantage relates to the potential depth limitations for installation of PRBs. With current technology, the cost of PRB emplacement is strongly related to depth of emplacement. The current rule of thumb for emplacement depth limits using excavation techniques is about 100 feet. At greater depths, alternative emplacement methods such as jet grouting, mandrel emplacement, or injection well emplacement must be considered. None of these methods have been demonstrated for this purpose and may require additional development.

An obvious disadvantage of PRBs, compared with certain other technologies, is that PRBs are passive; the contaminant must come to the PRB. This is not necessarily a disadvantage where the cleanup objectives are based on risk management, but may be a disadvantage when site restoration is the goal. However, both pump and treat done for hydraulic control purposes and impermeable physical barrier technologies possess this same limitation. Nevertheless, the PRB approach would appear to have limited flexibility once installed, compared to a pump and treat system where individual wells may be added, removed, or replaced relatively easily.

Finally, emplacement of PRBs may face greater logistical constraints than siting a pump and treat system. Underground utilities, buildings, pavement, and overhead structures all must be identified, avoided (if possible), or moved during PRB construction. In addition, excavation may result in the generation of contaminated soil, requiring management and added costs.

11.5 DEVELOPMENT NEEDS

It is apparent that the technology is still in the development stage and that there are hurdles to be overcome before the remediation community will accept and apply the technology. While a number of technical needs have been identified, the one basic need is to increase the comfort level of the remediation community and validate the technology through successful field demonstrations. The success of demonstration projects in turn hinges on the development of effective and readily available design tools and protocols.

Other identified needs include:

- development of effective/proven modeling and design tools,
- increased experience with field applications, and wider dissemination of "know-how",
- cheaper, more effective emplacement methods, especially for deeper (i.e., greater than 30 feet) PRBs,
- enhanced mechanistic research on treatment processes leading to development of optimized media,
- engineering evaluation of alternative PRB configurations, with guidance for optimizing hydraulic controls.

Each of these needs in discussed in more detail below.

11.5.1 Modeling and Design Tools

A basic need exists to develop the "know-how" for modeling and design of PRB systems. Of greatest priority is the development of recognized and accepted protocols for the following:

- geochemical and hydrologic site characterization supportive of PRB design;
- performance of treatability studies, including guidance concerning

what levels of treatability study are sufficient in a given setting;
- design of the hydraulic control and reactor systems;
- safety factor "rules of thumb", and/or methods to mitigate pluggage and ensure long-term performance; and
- development of proven methods for "working over" PRBs which have lost hydraulic efficiency and/or reactivity.

Development of a well-recognized basis for designing PRBs for long-term performance will accomplish several objectives. First and most obvious, the successful performance of future PRBs will be better assured. Second, the technology will become better accepted by the remediation community as greater comfort is acquired with the design and operation of these systems. Third, a basis will be established for regulatory review and approval.

11.5.2 Accelerated Field Experience and Communications

Coupled with the need for recognized modeling and design tools is the need for increased field experience, and communication of this experience to the remediation community. The widespread successful application of any technology generally follows only after a sufficient level of know-how is developed. The know-how in turn can only come from the extensive level of experience derived and shared from many applications. Both successful and unsuccessful applications can contribute to the experience base.

As previously discussed, there has been little field experience with PRBs, with one well-documented commercial scale project and a handful of smaller scale field demonstrations having been undertaken. Thus far, the track record looks promising, but it is too early to generalize from these results. Confidence in the use of permeable reactive barriers will increase as the number of successful field demonstrations increases. Several field demonstrations are currently in the planning stage and there appears to be a high level of interest in PRB technology.

Several PRB technologies are ready for field testing. In a few cases, principally the reactive iron technologies for chlorocarbons and reducible metals, the field effort has started and valuable lessons are being gained. However, other types of permeable barriers useful for non-reducible organics and metals have lagged in development. In particular, the sorption barrier technology could benefit from performance of a few well-planned field experiments with a variety of contaminants, treatment media, and geologic settings.

Timely communication of results and findings associated with laboratory and field projects is important and forms the basis for effective technology transfer. This includes not only communication and collaboration among the technology developers, but also with the remediation community at large. Formation of a central clearinghouse for demonstration plans and reports, application costs, and sharing of basic know-how would facilitate a more rapid pace of development.

11.5.3 Development of Better Emplacement Methods

A potential limiting factor to the use of permeable reactive barrier technology appears to be the inability to cheaply emplace the barriers, particularly at depths greater than 30 feet. PRB emplacement to a depth of 30 feet is possible with current technology. Emplacement at greater depths (from 30 to 100 feet) is more difficult and increases costs. Emplacement below depths of 100 feet is essentially unproven at this stage with current technology. The existence of surface obstructions (i.e., buildings, roads, utilities, etc.) presents other challenges to the emplacement of a PRB.

Emplacement methods that require excavation may generate substantial volumes of contaminated soil. Under current RCRA rules, these soils may need to be handled as waste, thereby increasing project costs. Worker exposure may also be an issue. Excavators and trenching equipment tend to produce a fixed trench width which may or may not fit the particular PRB design requirement (for example, retention time considerations may indicate a PRB could be built with a width of one foot whereas trenching machines are often two feet wide). This difference could result in over-designed systems with associated added costs, particularly where expensive treatment media are involved.

The jet grouting method and mandrel emplacement methods have potential for producing less waste and emplacing deeper PRB systems than the excavation-based approach. However, they are relatively unproven and may have certain drawbacks of their own. For example, adequate control over emplacement may be an issue with the jet grouting approach, whereas mandrel emplacement technology has been questioned with respect to formation disturbance and plugging around the reactor. Nonetheless, improvement of these methods could help to reduce the cost of deeper emplacements. The development of improved emplacement methods, possibly involving new equipment and techniques to achieve greater depths, could benefit the development of PRB technology. In addition, the development of "no waste" emplacement, or alternatively, regulatory policies which provide greater flexibility in handling excavated soils, should also contribute to the development of PRB technology.

11.5.4 Research on Treatment Processes

Considerable laboratory-based research has been conducted to date on a variety of treatment processes. For example, sorption of organics appears to be fairly well understood considering its widespread use in above-ground flow-through systems.

Additional mechanistic research on certain key PRB reactive chemistries is needed. This is particularly true with respect to the use of reactive iron for chlorocarbon destruction. At this stage, much remains to be learned about the reactive iron dechlorination process. For example, Burris (1995) points out that two basic reaction pathways for PCE/TCE dechlorination are evident (one involving sequential dechlorination to ethene and the other involving acetylene as the end-product). Since the acetylene route appears to produce less intermediates (e.g., DCE and vinyl chloride), it is regarded as the more desirable pathway for dechlorination. Research is currently underway at the Armstrong Lab at Tyndall Air Force Base to identify and influence the factors involved in this process.

Extensive work is ongoing concerning aspects of the reactive iron technology. This work has been quite fruitful in understanding the mechanisms, suggesting improvements, and translating the knowledge gained into better field designs. For example, the role of dissolved oxygen in causing premature reactor plugging has been identified as a result of mechanistic work in the laboratory. Work at U.S. EPA's Athens, Georgia laboratory has yielded additional suggestions for preventing pluggage and for maintaining reaction rates by mixing iron sulfide with the granular iron to stabilize pH (Holser, et al., 1995). Other improvements can be expected based on the knowledge gained from laboratory research.

11.5.5 Evaluation of Configuration Alternatives

The continuous wall approach involves the construction of the reactor portion of the PRB across the entire front of the plume, whereas the funnel-and-gate option reduces the reactor size in favor of low hydraulic conductivity barrier funnels to guide the groundwater flow. In the continuous wall approach, the reactor is sized to match the plume life, or at the least to provide for a very substantial life of treatment (e.g., 30 years). Thus, a substantial up-front cost is realized associated with the purchase of the reactor media for the given lifetime. Although the funnel-and-gate design reduces reactor volume, it may require periodic replenishment of the reactant and periodic O&M.

All things being equal, the choice of a PRB configuration alternative would be based on economic considerations. The need then, is for a set of economic evaluations to be performed which would identify the important cost variables and aid decision-making. A cost model for the reactive iron PRB technology is nearing completion under a multi-year project administered by the Air Force through the Strategic Environmental Research and Development Program (SERDP) (Smith, 1995). This model will function as a module in the Air Force's ENVEST remediation costing program. The new ENVEST module could serve as a tool for carrying out these economic evaluations.

11.5.6 Recommendations for Addressing Needs

First and foremost, it is recommended that a suite of definitive field demonstrations be conducted in a variety of hydrogeologic settings and with a range of groundwater contaminants.

Second, there needs to be a collaborative development of recognized procedures and protocols for: (a) site investigation, (b) treatability studies, and (c) hydraulic control and reactor design. These tools should be developed as consensus documents by leaders in the R&D community and shared widely with the remediation and regulatory community. U.S. EPA's Remediation Technologies Development Forum, which features cooperative technology development between public and private organizations, is viewed as a model framework for establishing this collaborative effort and in sharing the findings.

Third, a concerted effort is recommended to educate the remediation community, including regulators, to help pave the way for future applications. A clearinghouse for information exchange should be developed to effect timely technology transfer. The clearinghouse would serve as a mechanism for sharing status of research programs, demonstration project plans and reports, and notice of meetings. A newsletter format (either as a stand-alone letter or added as a section to an existing newsletter) would be preferred. Also, publishing the newsletter on the Internet may be a good mechanism for broad dissemination of information related to PRB developments.

11.5.7 Intellectual Property and PRB Development

One issue not addressed at the workshop concerns the rights to intellectual property and the evolving relationships between technology developers, technology users, and intellectual property owners. While cooperative development was seen by the panel as the optimum path for moving this technology into full-scale application, it may not be realistic to expect that sharing of patents and know-how necessary for true cooperative development will take place. On the other hand, technology development and field testing, particularly by the public sector, could result in increased business for technology owners if performance is verified independently. Thus, there are incentives which could outweigh the impediments.

This issue certainly is not unique to PRB technology. However, potential users should be cognizant that several PRB processes have been patented (e.g., Gillham, 1993; Blowes and Ptacek, 1994) which could limit the potential for cooperative development of certain aspects of the technology.

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SECTION 12

PERFORMANCE MONITORING AND EVALUATION

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12.1 BACKGROUND

12.1.1 Containment System Performance Monitoring Objectives

Waste containment system performance data are needed to conduct assessments that can be grouped into three primary categories: exposure risk assessment; source term estimation of contaminant concentrations for contaminant transport models; and facility permitting and future maintenance needs assessment. Long term performance of containment systems is a concern because of the long design service lives of most facilities. Typically, design service lives range from as little as 10 years for slurry walls to more than 1,000 years for radioactive waste storage structures. The longer the service life of a containment system, the greater the probability of system failure (i.e. greater opportunities exist for excessive damage accumulation and contaminant transport). Considering that most components of waste containment systems exist underground, constraint is placed on direct visual inspection as a monitoring method. Thus, several traditional and evolving techniques of indirect observation need to be employed to obtain performance data for the following categories of assessments.

a. Exposure Risk Assessment. Exposure risk assessment is usually conducted within the framework of environmental and human health risk assessment. Many agencies and facility owners have adopted risk-based approaches to selecting and formulating contaminated site remediation plans. The use of innovative, cost-effective monitoring methods can provide accurate estimates of parameters that are included in exposure assessment equations, especially, when the objective is to quantify the reduction of risk that may attend the implementation of a containment system.

Many computational methods and recommendations (Marnicio at al., 1991; Schanz and Salhotra, 1990; Imam et al., 1990; Tonn and Wagner, 1990; Kostecki et al., 1989; Taylor et al., 1987; Preuss et al., 1986; U.S. EPA, 1990a; U.S. EPA 1990b; and U.S. EPA, 1990c) have been developed for use in risk assessment. Equation 12-1 is an example of a general relationship for exposure assessment. The use of innovative monitoring techniques for contaminants around facilities can improve numerical estimates of parameters C, EF, and ED of Equation 12-1.

$$IN = \frac{(C)(IR)(EF)(ED)}{(BW)(AT)} \tag{12-1}$$

IN = intake = the amount of a specific chemical in a contaminated medium taken by a receptor organism (mg/kg of body weight/ day).
C = concentration = average chemical concentration contacted over the exposure period (mg/l, or mg/mg).
IR = intake rate (or contact rate = amount of contaminated medium contacted per unit time or event) (mg/day or l/day).
EF = exposure frequency (upper bound value), (days/ year).
ED = exposure duration (upper bound value), (years).
BW = body weight = average body weight over the exposure period (kg).
AT = averaging time = time period over which exposure is averaged = exposure duration for non-carcinogens, and 70 years for carcinogens, (years).

b. Source Term Estimation for Transport Models. Uncertainties characterize numerical estimates of source term concentrations of contaminants that may travel through barrier systems. Monitoring systems may be installed to measure source strengths for verification of estimates generated by numerical models. Monitoring data can be incorporated into contaminant transport models to estimate future concentrations at locations away from the source. Figure 12-1 shows a surface impoundment of basal width d, which has released contaminants into the vadose zone. The source term concentration, C_a , as well as the fluid migration rate q, can be monitored. Leachate plume concentration, C_o, can be monitored and/or numerically estimated at locations around the containment system, using several relevant parameters among which are plume velocity V_h, and thickness b. In the case illustrated, the contaminant release rate is partly proportional to the rate at which the containment system deteriorates. Within the context of source term estimation, the structural state of the barriers as well as contaminant concentration parameters, C_a and C_o, can be monitored.

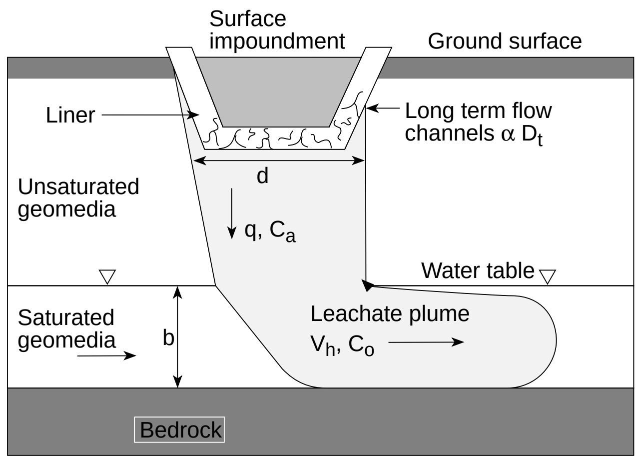

Figure 12-1 Illustration of source term parameters for a simplified assessment of contaminant migration from a leaking surface impoundment (Inyang, 1994b)

c. Facility Permitting and Maintenance. Post-construction performance monitoring by waste management facility owners is necessary in order to refine maintenance plans. Also, information on performance is useful to regulatory personnel who are involved in permitting of initial facility construction as well as expansions. Some containment system rating and performance prediction methods have been developed for these purposes (Inyang, 1994a; and Koerner and Daniel, 1992). Figure 12-2 illustrates the conceptual long term deterioration pattern and maintenance scheme proposed by Inyang (1994a) for waste containment systems. It should be noted that this is a conceptual scheme and does not simulate the exact deterioration pattern of a specific containment system. Following curve 1, the effectiveness of a containment system is believed to decrease under service conditions, from an initial value of E_{to} to a value E_{tr}, at which regulations require the implementation of repair activities. If maintenance is implemented at time t_m, the effectiveness of the containment system may increase instantaneously to a value, E_{tm}. A damaging natural event such as an earthquake or flood may cause a sharp decrease in effectiveness as illustrated in Figure 12-2 by a drop from E_1 to E_{tg} at time t_g. In order to verify and improve the accuracy of this type of modeling approach, monitoring data are required. Due to the large volume of data required, the use of non-invasive geophysical and electrochemical sensing systems is desirable.

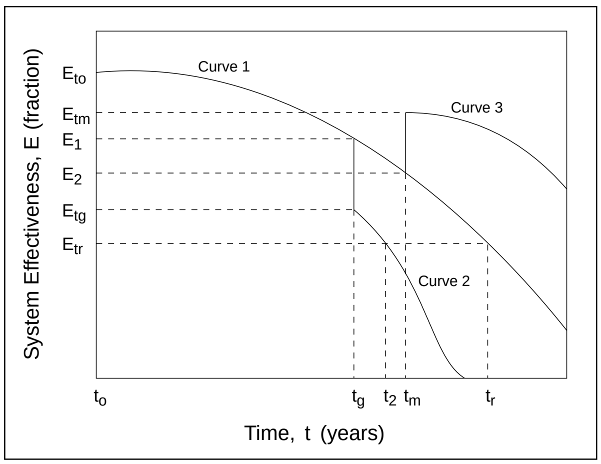

Figure 12-2 A conceptual long term deterioration pattern and maintenance scheme for waste containment systems (Inyang, 1994b)

12.1.2 Containment System Performance Factors

Containment systems are constructed facilities. All constructed facilities contain flaws and are susceptible to deterioration processes. Flaws may not always occur at sufficient scale and frequency to cause functional failure of a containment system. An appreciation of probable deterioration processes and damage-producing transient events is essential for optimal design of containment system monitoring schemes. For example, excessive settlement of a landfill cover can result from a high rate of waste decay within a landfill. Settlement estimates can be used to optimize the distribution of monitoring points on the landfill cover.

a. Deterioration Mechanisms. Containment system deterioration mechanisms fall into two categories: physico-chemical processes; and transient events. Physico-chemical processes often occur continuously but slowly. They may wane or intensify with time, depending on the chemical environment, temperature, and pressure conditions. It is difficult to monitor physico-chemical processes directly for most practical assessments. Typically, the effects of physico-chemical processes on the containment system are measured, rather than the processes themselves. Examples of such processes are barrier flocculation, dissolution, desiccation and slurry settlement. Some transient events occur dramatically, e.g., ground movements due to earthquakes. The primary determinant of the probability that a well-constructed containment system will be affected by a transient event during the postconstruction period is its location. Zoning systems have been developed for some transient events such as earthquakes and freeze-thaw action (Inyang, 1991; Wright et al., 1993).

b. Deterioration-Time Pattern. Engineered waste containment systems have existed for a short time in comparison with other types of structures. Numerical estimates of the design lives and potential deterioration pattern of containment systems have been made without the benefit of field data on past performance of similar systems over reasonably long time intervals. Most of the data that are available on the damage of components are not time-distributed. Thus, it is difficult to assess the rate at which the damages accumulated and the relative contributions of various deteriorations processes to the observed damages. Table 12-1 provides a summary of field experiences with some waste containment systems compiled by Bass et al. (1985). Laine and Miklas (1989) also documented leaks in impoundments as shown in Table 12-2.

The paucity of time-distributed monitoring data for a wide variety of waste containment system configurations has necessitated the development of general deterioration-time models based on observed deterioration pattern of most constructed facilities. Inyang and Tomassoni (1992) have discussed this problem in detail. Furthermore, Inyang (1994a) proposed the use of Equation 12-2 for time-scaling of the effectiveness or reliability of waste containment systems.

$$R_t = \exp -\{ [t - t_o]/n\}b \tag{12-2}$$

- R_t = reliability of the containment system at the future time of reference, t
- t = future time of reference [T]
- t_o = time considered to be the start of deterioration for analysis purposes (location parameter) [T]
- b = shape parameter [dimensionless]
- n = scale parameter [T]

The shape parameter is directly proportional to the design conservatism of the waste containment system that needs to be monitored. For barrier containment systems, the shape parameter ranges from 2 to 5, approximately. This is the range for which the deterioration versus time curve exhibits the usual negative exponential decay geometry. The scale parameter is a

TABLE 12-1 Summary of some experiences with containment system performance in the field (Bass et al., 1985)

Site ID	Single (S) or Double (D) Liner	Primary Liner Material	Primary Liner (mil)	Total Surface Area (ac)	Exposed (E) or Buried (B)	Moni-toring System	Layers in Liner System* (bottom to top)	Air Vents	Problems with Liner
V1-1	S	OR-CPE (R)	36	10	B	No	Gr/GeoTex/S&G/Geo-Tex/FML/Soil cement	Yes	-
V1-2	S	CSPE (R)	36	22	E	Yes	Comp Clay/S/FML	Yes	Yes
V1-3	S	PVC (U)	30	2	B	No	LimeRk/S/FML/S/ Lime/Rk	No	-
V1-4	S	PVC (U)	30	10	B	No	Comp Soil/FML/Soil	No	-
V1-5	D	PVC (U), CSPE (?)	20, 36	1	E	Yes	Comp Clay/S/FML/ S/FML	Yes	-
V1-6	S	PVC (U)	30	2	B	No	Old Fill/Clean Fill/FML/Clay	Yes	-
V2-1	S	CSPE (R)	30	120	E	No	Comp Clay and Limestone/FML	Yes	Yes
V2-2	S	CSPE (R)	30	8	E	No	Comp Soil/S&G/FML	No	Yes
V2-3	S	CSPE (R)	30	2.3	E	No	Comp Sub-base/FML	No	Yes
V2-4	S	CSPE (R)	30	4.3	B	Yes	Comp Fill/FML/S/G	No	-
V3-1	S	PO (R)	30	42	B	Yes	Prepared Limestone/ FML/Stone	No	Yes
V3-2	S	PVC (U)	20	75	B	Yes	Comp Clay/FML/S	No	Yes
V3-3	S	PVC (U)	20	8	B	No	Comp Soil/FML/S	No	-
V3-4	S	Soil Sealant	4 in	25	B	Yes	Comp Sand/Liner/S	No	Yes
V3-5	S	Asphalt-concrete	5 in	2	E	Yes	Comp Soil/Asphalt (2 lifts)	No	-
V4-1	S	HDPE (U)	100	18	E	Yes	Comp Sand/FML	No	-
V4-2	D	HDPE (U)	100	18.5	E	Yes	Comp Sand/S/FML	Yes	-
V4-3	S	HDPE (U)	80	88	E	No	Comp Subgrade/FML	Yes	-
V4-4	D	HDPE (U)	80	6	E (sides)	Yes	Clay/S/Comp Soil/ FML/Comp Soil	Yes	-
V4-5	D	HDPE (U)	80	3.2	B	Yes	Comp Clay/FML/ Comp Clay	No	-
V4-6	S	HDPE (U)	100	0.3	E	Yes	Comp Soil/FML	No	-
V4-7	S	HDPE (U)	80	66	E (sides)	No	Subgrade/FML/S (bottom only)	No	-
V5-1	3D, 1S	CPE (U), CPE (U)	20, 30	1.5	E (CIM only)	Yes	Subgrade/CPE/Soil/ Concrete/CIM	No (?)	Yes
V5-2	S	CPE (U)/ PVC (U)	20, 10	13	B	Yes	Nat Soil/FML/Nat Soil	No	Yes
V5-3	S	CPU (U)	30	0.7	B	Yes	Nat Soil/FML/Nat Soil/Soil Cement	No	-
V5-4	D	PVC (U)	20	1.4	B	Yes	Comp Soil/Clay/ S/FML/Nat Soil	No (?)	Yes
V5-5	Triple	2xCPE(R), PVC (U)	30, 20	0.75	E	Yes	Comp Fill/CPE/G/ PVC/CPE/?	No (?)	-

*Comp = compacted; FML = flexible membrane liner; G = gravel; GeoTex = geotextile; Gr = ground; Nat = natural; Rk = rock; S = sand

TABLE 12-2 Leak detection and survey data for impoundments where the bottom floor areas were surveyed (Laine and Miklas, 1989)

Survey No.	Size in ft^2	Total Leaks	Leaks Located in			Leaks per 10,000 ft^2
			Bottom	Seam	Sheet	
1	958	2	2	2	0	20.9
2	958	3	3	3	0	31.3
3	958	3	3	3	0	31.3
4	1,000	4	4	3	1	40.0
5	1,798	0	0	0	0	0.0
6	2,625	6	6	6	0	22.9
7	3,000	21	21	21	0	70.0
8	3,000	4	4	4	0	13.3
9	3,200	0	0	0	0	0.0
10	4,951	0	0	0	0	0.0
11	4,951	17	17	17	0	34.3
12	4,951	2	2	2	0	4.0
13	5,175	2	2	1	1	3.9
14	7,007	4	4	4	0	5.7
15	12,600	7	7	7	0	5.6
16	18,346	50	50	35	15	27.3
17	26,016	7	7	7	0	2.7
18	26,016	4	4	4	0	1.5
19	27,297	8	8	6	2	2.9
20	32,292	25	25	25	0	7.7
21	43,560	2	2	2	0	0.5
22	45,345	4	4	4	0	0.9
23	50,000	6	6	6	0	1.2
24	50,400	193	193	188	5	38.3
25	54,500	29	29	18	11	5.3
26	55,025	12	12	12	0	2.2
27	58,900	8	8	6	2	1.4
28	62,500	21	21	19	2	3.4
29	64,583	29	29	21	8	4.5
30	65,340	56	56	55	1	8.6
31	65,369	6	6	6	0	0.9
32	65,369	7	7	5	2	1.1
33	65,369	5	5	4	2	0.8
34	65,500	7	7	5	2	1.1
35	65,500	5	5	3	2	0.8
36	74,088	20	20	19	1	2.7
37	82,500	18	18	15	3	2.2
38	87,120	8	8	7	1	0.9
39	87,120	17	17	17	0	2.0
40	99,050	18	18	14	4	1.8
41	135,036	17	17	16	1	1.3
42	150,781	64	64	46	18	4.2
43	152,460	2	2	2	0	0.1
44	152,460	7	7	7	0	0.5
45	157,584	12	12	10	2	0.8
46	164,085	18	18	16	2	1.1
47	362,690	51	51	37	14	1.4
TOTALS	2,769,336	811	811	709	102	2.9

normalization factor for time. It is the time duration that corresponds to a containment system failure probability of 0.632. The necessity for implementing automatic monitoring systems increases as $(t-t_o)/n$ increases and the shape parameter, b decreases. This temporal reliability concept has not yet been applied to indexing of long term performance of waste containment systems. A requirement for the practical application of Equation 12-2 is the formulation of a scheme for relating the shape parameter to initial design configuration, barrier material properties, and the probable intensities of possible damaging events and processes.

12.1.3 Monitoring Approaches and System Failure Determination

In terms of containment system effectiveness, two types of failure categories can be identified:

- structural failure: one or more components of the containment system fails structurally (excessive deformation and/or introduction of flow channels).
- functional failure: the containment system can no longer perform the function for which it was designed (prevention of excessive contamination of the protected environment).

Structural failure can occur without functional failure although structural failure may eventually lead to functional failure. Furthermore, for multi-component systems, the structural or functional failure of a component does not necessarily imply that the entire system has failed. It depends on the criticality of the failed component. With respect to the use of monitoring data to define containment system functional failure, the following two approaches can be identified:

- arrival of contaminants in excessive concentrations at a critical point within or just outside the containment system, and
- excessive contamination of a protected natural resource by contaminants that have traveled away from the containment system (examples of natural resources are surface waterbodies and groundwater aquifers).

Three principal approaches can be adopted to monitor structural and functional failures of waste containment systems. In most cases (as described in section 12.5.3), these approaches are complementary. Various types of monitoring methods that fit within these three principal approaches are summarized in Table 12-3 and Table 12-4.

a. Barrier Integrity Monitoring. This approach involves monitoring deformation and flaw development within barriers. Such flaws may compromise the structural integrity of the containment system and serve as flow channels for moisture and/or contaminants. Figure 12-3 illustrates the increase in infiltration through cracked portions of a concrete cover, relative to intact portions of the cover.

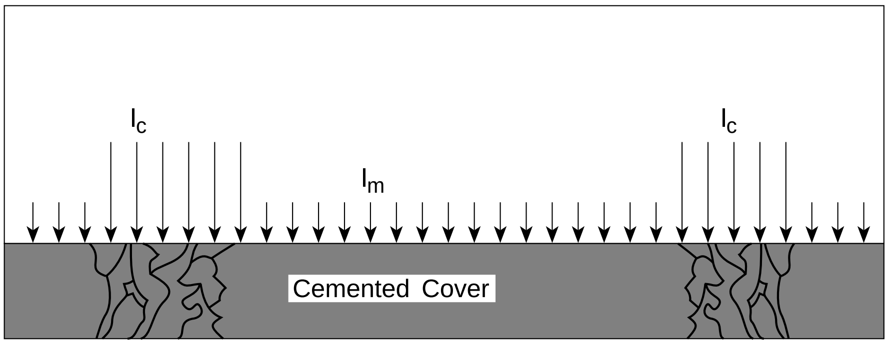

Figure 12-3 Concrete cover with illustration of infiltration through cracks (I_C) relative to infiltration through matrix (I_M) (Inyang and Myers, 1993)

b. Barrier Permeation Monitoring. This approach involves sensing of moisture and contaminant concentrations and flow rates within components of a containment system. As examples, the infiltration rate of moisture through a cover system and/or the concentration of specific contaminants in the leachate within a drainage layer can be monitored.

TABLE 12-3 General application of monitoring approaches

Barrier Monitoring Approach	**Monitoring Method**				
	Well Network	Geophysical Methods	Electro-chemical Methods	Mechanical and Electro-chemical Methods	Electrical Methods
1 Barrier Integrity Monitoring	U	G	R	C	G
2 Barrier Permeation Monitoring	U	R	G	R	C
3 External Monitoring	C	G	G	R	G
Key: C = conventional; G = growing application; R = rare; U = unfeasible					
Note: These methods comprise several specific techniques, some of which may not necessarily fit into these three categories.					

TABLE 12-4 Examples of traditional and innovative monitoring techniques for specific waste containment problems

	WASTE CONTAINMENT PROBLEM	MONITORING TECHNIQUES
1.	Cover slope deformation and failure	Inclinometers, electrical resistance shear strips, and vibrating wire extensiometers
2.	Lateral deformation of vertical walls	Electrical inclinometers, extensiometers, and tiltmeters
3.	Excessive settlement and cracking of soil and concrete covers	Sealed Borros anchor, inclinometers, settlement platforms, electrical crack guages, convergence guages, microwave sensors, dye-tracing
4.	Excessive infiltration of cover systems	Tensiometers, gypsum blocks, psychrometers, neutron guages and piezometers in covers, thermo-electric flow velocity guages in drainage layers, and fiber-optic moisture sensor in covers
5.	Voids and high permeability zones in emplaced and natural barriers	Crosshole seismic surveys, ground penetrating radar surveys, seismic tomography, surface-based seismic refraction surveys, soil gas surveys, temperature logging, cone penetration testing, and dye tracing
6.	Leachate plume migration from waste containment systems; and plume capture by permeable reactive walls	Electro-chemical sensing cables and fiber-optic chemical sensors around containment structure, DC electrical and electromagnetic resistance surveys above ground, ground penetrating radar surveys, dye tracing, and groundwater monitoring
7.	Contaminant flow through bottom barriers and low hydraulic conductivity walls	Electro-chemical sensing cables and fiber-optic chemical sensors within, upgradient, and downgradient from barriers, and pumping tests
8.	Leaks in geomembranes and sheet pile joints	Electrical leak detection systems and fiber-optic monitoring systems for joints
9.	Flow of leachate through reactive permeable walls	Thermal flow sensors upstream, within, and downstream from wall, electro-chemical sensors, pH and Eh sensors

c. External Monitoring. External monitoring involves the measurement of contaminant concentrations outside a containment system. It is geared towards the identification of functional failure. Monitoring locations proximal to and/or distant from the containment system can be selected. Hydrogeological factors and containment system configuration are important factors in external monitoring point selection. Inyang and Tumay (1995) have analyzed various containment system configurations. External monitoring may involve one or more of the following techniques, depending on the purpose of the monitoring effort:

- successive measurements at a point to determine the arrival time of moisture and/or contaminants following the construction of a containment system,
- successive measurements at various points to estimate the transport rates of moisture and contaminants through the media around the containment system, and
- one-time measurement of contaminant concentrations at various locations around the containment system to delineate the boundaries of a plume.

12.2 CURRENT AND EVOLVING MONITORING TECHNIQUES AND TECHNOLOGIES

Four primary categories of techniques and technologies are currently used to monitor waste containment system performance within the three approaches outlined in the preceding section. As summarized in Table 12-3, despite the potential cost-effectiveness of some of the techniques, they have not been used widely to monitor performance. Current and evolving monitoring systems and the principles of their operation are briefly discussed below. For details, the reader is referred to Inyang (1994b), Betsill and Gruebel (1995) and U.S. EPA (1993a,b).

12.2.1 Groundwater and Leachate Monitoring Well Network

Traditionally, monitoring of containment systems has been primarily performed through groundwater and leachate collection and testing. Leachates are usually extracted from sumps within the containment area, or from water wells sited around the containment system as illustrated in Figure 12-4. Minimum requirements generally specify the placement of one upgradient and three downgradient wells around a containment system. Water is extracted periodically from these wells and monitored for specific contaminants. Leachate collection sumps are part of the Leachate Detection System (LDS) of landfill systems. A survey of 28 landfills (U.S. EPA, 1992) indicated that at most of the facilities, sumps are inspected daily. A recent study (GAO, 1995) revealed that 1209 hazardous waste land disposal facilities in the United States are required to implement groundwater monitoring programs. Seventy-seven percent of the facilities use one well to track potential releases; whereas, the rest require from 2 to 17 wells. An increase in the number of wells can improve monitoring effectiveness. However, it is emphasized that properly designed and installed monitoring wells are still an indispensable tool for monitoring containment performance. Several technical guidance documents (e.g., U.S. EPA, 1991c and U.S. EPA, 1989) have been developed for use in the implementation of effective groundwater and leachate monitoring systems.

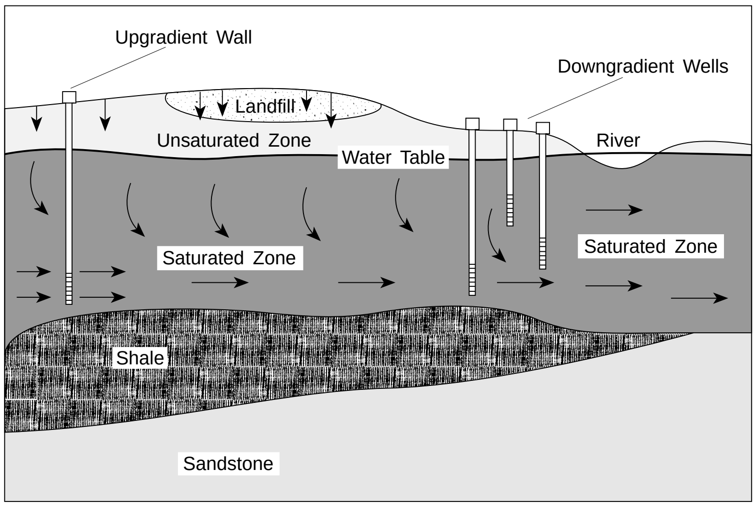

Figure 12-4 Traditional groundwater monitoring system using wells (GAO, 1995)

12.2.2 Strain Gauging and Mechanical Detection of Flaws

Gauges capable of measuring deformation of subsurface and exposed components of waste containment systems are in use today as some facilities. These gauges have previously been used for many decades in geotechnical structures (e.g., embankments, diaphragm walls, building foundations, and dams) to monitor deformations exemplified by settlement, lateral movement, inclination, and slope failures. Strain gauges can provide information on the onset and location of potential structural failures within a containment system. The deformation of a strain gauge that is compliant with a containment system in which it is installed can be read from a scale or converted to electrical impulses for motion indication. Strain causing actions such as shear, extensions, compression, and deflection can be detected with these gauges.

Dunnicliff (1982), King and Kusenberger (1973), and Dietrich and Salley (1975) have provided details on various strain sensing principles. Inyang (1994b) has described the principle of operation of an electrical resistance shear strip, e.g., the one illustrated in Figure 12-5. The strip can sense deformation occurring transverse to its orientation. Typically, an electric circuit runs through two parallel strips that are embedded in the monitored structure. Since distortions in the atomic lattice of a metal rod during straining influence its electrical resistivity and straining reduces the cross-sectional area through which current flows, it follows that the change in electrical resistance of a strain gauge is directly proportional to its change in length. Equation 12-3 represents this proportionality and is used in practice to scale linear deformation to the electrical resistance of an embedded gauge.

$$S = \frac{(\Delta R)}{R_o} = \frac{\Delta L}{L_o} \tag{12-3}$$

S = strain sensitivity factor
R_o = initial resistance of the gauge wire
L_o = initial length of the wire
ΔR = change in resistance due to straining of the wire
ΔL = change in length of the wire

The theoretical value of S ranges from -12 to 6 for common metals (King and Kusenberger, 1973). Other devices for measurement of deformation of containment system components include:

- interface slip measurement using electrical resistance shear strips,
- settlement gauges for covers, liners, waste piles and confined disposal facilities,
- inclinometers for slurry walls, and
- extensometers for covers, liners and confined disposal facilities.

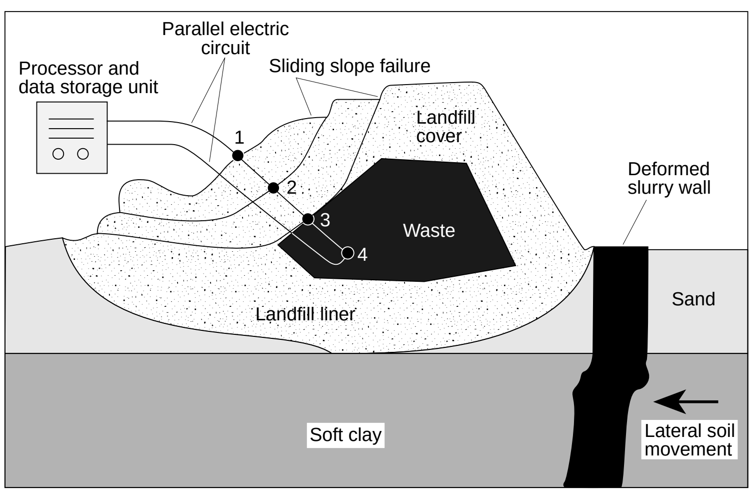

Figure 12-5 A schematic illustration of the use of an electrical shear strip to sense the flow failure of a landfill cover (Inyang, 1994b)

Unfortunately, despite several deformations and failures of containment systems reported by Mitchell et al., 1990; Byrne et al., 1992; Suter et al., 1993; Kahle and Rowlands, 1982; Yen and Scanlon, 1975; and Dodt et al., 1987, very few containment systems have been instrumented with strain and structural failure sensors. The focus has been on the detection of barrier permeation.

12.2.3 Geophysical Systems

The geomaterials and fabricated materials that form the components of waste containment systems, as well as the natural and subsurface around them, exhibit physical and chemical characteristics that may change with chemical contamination and/or internal void development. These changes can be detected through the use of geophysical methods.

Geophysical techniques are based on the response of the geomedia and pore fluids to various segments of the electromagnetic (EM) spectrum, or seismic and/or acoustic energy, or other potential fields, e.g., a magnetic field or gravitational field. The radiation type employed by a given electromagnetic technique within the spectrum is described in terms of its range of frequency or wavelength. Electrical currents in the ground can be induced remotely using electromagnetic methods, eliminating the need to physically disturb the specific subsurface location being evaluated.

Electromagnetic measurements can be used to investigate materials either in the frequency domain or time domain. Frequency domain evaluations track the material's response to electromagnetic fields at one or a number of frequencies. This is often termed electromagnetic induction (EMI). In time domain electromagnetic measurements (TDEM) or transient electromagnetic (TEM) soundings, the change in response is measured as a function of time following the switch-off of the transmitters. Application of TDEM methods in contaminant migration studies is relatively recent.

Seismic methods are based on the speed of travel of compression-generated disturbances (or waves) in the medium evaluated (Terzaghi, 1943). These waves may be reflected or refracted at boundaries between media of different physical and/or chemical properties. Usually sensors are positioned at known distances from a seismic source to detect reflected or refracted waves. The speed of travel of the waves depends on the layering and properties of the media through which they travel.

Gravitational methods are based on the principle that the differences between measured gravitational acceleration or potential of media or objects reflect their mass differences. Thus, a buried drum accelerates differently from its surrounding soil.

Geophysical methods have been used for many years to verify the continuity of grout curtains and bulbs emplaced in the subsurface. Similar techniques are currently being used to monitor emplaced barriers. However, it should be noted that obtained data may be of limited use for quantitative assessment of contaminant migration. The resolution of currently available geophysical technologies is too large to reveal the details of flaw sizes and directions in constructed barriers. With respect to containment system performance monitoring, the application of geophysical techniques is growing in two areas.

- Assessment of the continuity of natural and emplaced barriers
  - Cross-hole seismic imaging
  - Surface seismic refraction
  - Ground penetrating radar (GPR)
  - Microwaves, ultrasonics, and radio waves

- Tracking of contaminant plume extent around containment systems after release from an existing containment system
  - Electromagnetic (EM) resistivity
  - Ground penetrating radar (GPR)

These geophysical methods are briefly described below.

a. Crosshole Seismic Imaging. This new technique is illustrated in Figure 12-6. Shear and compression waves, generated from an array of transmitters in a borehole, travel through the medium of interest and are detected by an array of receivers in another borehole. The travel times of seismic waves between the transmitters and receivers are inverted into a two-dimensional velocity structure map using the principles of tomography. This method is semi-invasive because of the use of boreholes. However, by placing the boreholes outside the boundaries of the contaminated zone or the grouted media, the risk of drilling through the barrier or contaminants can be minimized.

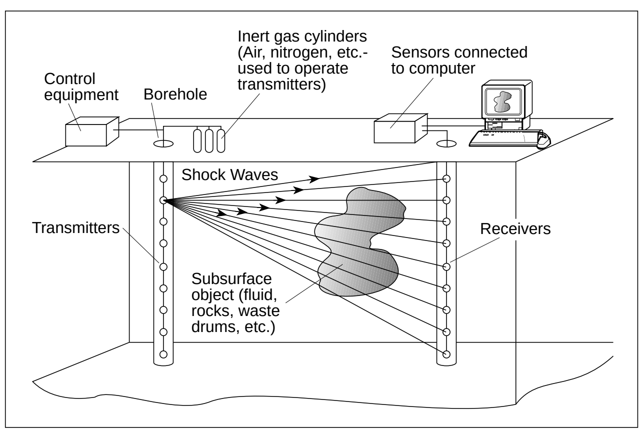

Figure 12-6 Schematic of a crosshole seismic tomography imaging system (U.S. DOE, 1994b)

Compression waves generated by seismic sources are in general, faster in dense, tightly cemented, and wet materials than in loose and dry materials. Thus, seismic wave velocity is directly proportional to the density of the transport media. Grouting of a loose medium generally increases its density. If the density increases significantly, wave travel velocity through the grouted or improved medium will increase. Hence, before and after (grouting) transit time measurements can be used to evaluate the effectiveness of barriers that are large enough to exhibit significant changes in seismic wave velocities.

b. Surface Seismic Refraction. This method is effective mainly for use in delineating relatively horizontal boundaries among strata that exhibit different physical characteristics and moisture saturation levels. It is very effective within the upper 30m of the subsurface, which applies to the depth range of most waste containment and site improvement projects. However, the difficulties associated with interpreting refraction data from irregularly shaped structures within the ground hampers the application of this method for delineating media boundaries in the subsurface. Nevertheless, measurements taken before and after ground improvement can be used qualitatively in indexing the effectiveness of projects.

The operating principle is that primary (compressional) waves are refracted at the interface between media of significantly different physical characteristics; e.g., density, modulus of elasticity, porosity, and moisture content. For shallow hazardous waste site investigations, seismic waves can be generated by a hammer hit or drop weight on the ground. Geophones placed at distances away from the seismic source detect incoming refracted waves and are processed using electrical circuits. The data acquired from geophones are typically processed and plotted as points on time-distance graphs. Straight lines are used to connect points based on the analyzer's interpretation of the data. Changes in the slope of lines indicate differences in wave velocities and hence material characteristics of different layers of the subsurface. A critical assumption in the use of seismic refraction is that within each stratum, density increases with depth. Also, interpretation is easier in cases where denser strata underlie less dense strata.

c. Ground Penetrating Radar (GPR). GPR uses electromagnetic waves to penetrate the ground and delineate differences in pore water content and quality, soil texture, and soil density. Wave frequencies ranging from 80 to 1000 megahertz (broad band) are normally used. Upon contact with materials of different properties, a fraction of the wave energy is reflected back to a receiving antenna located on the ground surface while the remaining fraction propagates deeper into the ground. Reflected electromagnetic pulses can be converted to plots of amplitude versus wave travel time.

Soils with high conductivity of electromagnetic waves (e.g., clays and other fine-grained saturated soils) rapidly dissipate radar energy, thereby limiting depth penetration. While penetration depths of up to 10 meters are common, 5-10% (by weight) of montmorillonitic clay can reduce the penetration depth to about 1m (Walther et al., 1986). Conductivity is also directly proportional to the concentration of dissolved solids in the pore water. In field applications, the surface receiving antenna can be towed at speeds up to 8 km/h. In Port Washington, New York, GPR was used at the Roslyn/Beacon Hill Landfill site to optimize the locations of monitoring wells (Kardos and Ennis, 1993).

d. Electromagnetic (EM) Resistivity. This method is also based on changes of the electrical conductivity that usually occurs between subsurface zones of different physical and chemical properties. Since electromagnetic phenomena are generated in the media investigated, it is not necessary to use electrodes embedded in the ground to transmit and receive current. For this reason, EM resistivity is an effective means for delineating contaminated zones beneath covered areas (e.g., structural foundations, streets, and parking lots).

The principle of inducement of electromagnetic field in geomedia for resistivity measurements is illustrated in Figure 12-7. An alternating current is applied to the terminals of a transmitter coil, fixed on or close to the ground surface, to induce the flow of a current, thereby creating a magnetic field which, in turn, causes electric currents to pass through the earth. Within the earth, a secondary magnetic field is induced. The secondary and primary magnetic fields are detected by a receiving coil placed near the transmitting coil. For a fixed spacing between coils and operating frequency, the magnitude

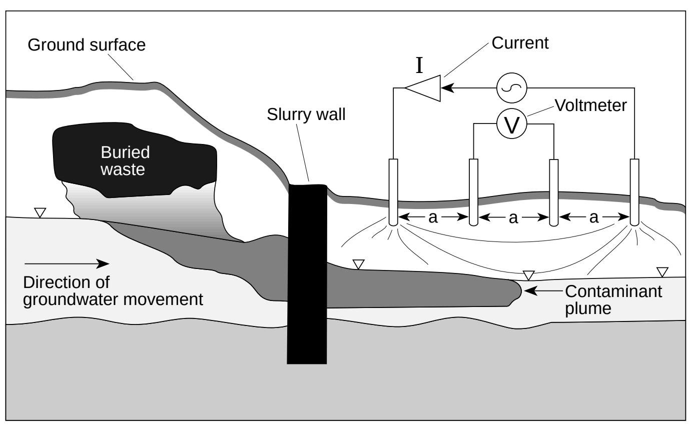

Figure 12-7 Sketch of Wenner resistivity array for delineating contaminant plumes behind a subsurface barrier (Inyang, 1994b)

of the secondary magnetic field (or ratio of the secondary to primary magnetic fields) is directly proportional to the conductivity of the ground. Resistivity can be computed from conductivity data. In the subsurface, the solid matrix acts as an insulator while soil moisture acts largely as the conductor of the electric current. Furthermore, the conductivity of the pore fluid is proportional to the concentration of dissolved ions. Thus, these measurements can be used to delineate contaminated zones around waste containment barriers. If buried metallic objects are present near the surveyed site, the readings may be negatively affected. The effective depth of penetration can be as high as 60m. This method is often called the Frequency Domain Electromagnetic (FDEM) method.

There are two other variations of the electromagnetic induction method. In one variation, the rate at which the magnetic field dissipates is measured after the transmitter is turned off. An eddy current flows through the ground at successively greater depths. The data obtained are interpreted to obtain resistivity variation with depth. This is called Time Domain Electromagnetic (TDEM) survey. It has been applied by Hoekstra et al. (1992) to trace the migration of brines from an oil field brine pit in southwestern Texas. Penetration depths of up to 500m are estimated by Hoekstra et al. (1992). The other variation of (EM) measurement involves the use of very low frequencies (15 to 25 kHz). In this method, ground contact is required for the potential electrodes. It is very suitable for investigating relatively shallow contaminant plumes (20-50m) around waste disposal sites or containment systems.

Techniques based on microwaves, ultrasonics, and radio waves can also be used to detect voids in various types of waste containment barriers, especially in cemented components. Laine et al., (1980) and Koerner et al., (1982) have described methods that are based on radiowaves and microwaves, respectively. As will be discussed in section 12.3, the full potential of geophysical techniques has not been realized in containment system monitoring. Geophysical techniques, which are being adapted from site characterization technology, need further refinement to achieve the spatial resolutions necessary for containment system monitoring programs.

12.2.4 Electrochemical Systems

Electrochemical sensing systems operate on the principle of changes in the physico-chemical characteristics of the sensor due to contact with a solid or fluid. These changes may be directly converted into electrical or optical impulses which are conveyed to processing equipment for interpretation. Interpretation of impulses involves the location of points of contact between the field of concern and the sensor network. Processing units with digital recorders have been developed to track several sensors concurrently. The essential difference between these detection systems and geophysical techniques is that the detection sensor must be in contact with the pore fluid whereas, using geophysical methods, the presence of the fluid is remotely sensed, albeit less precisely.

Sensing systems can be divided into three categories (illustrated in Figure 12-8) on the basis of the spatial coverage of the sensors within a waste containment structure or a potentially contaminated zone. Point systems typically comprise a stem which terminates in a sensor. The sensor may also be located at a point on the embedded instrument. Linear systems are typically cables. Areal systems comprise point and/or linear systems linked together to form a network. For example, a cable can be looped or netted to monitor a wide area beneath a landfill. This is especially effective, considering that it is often difficult to determine a priori, where leakage will occur from a containment system.

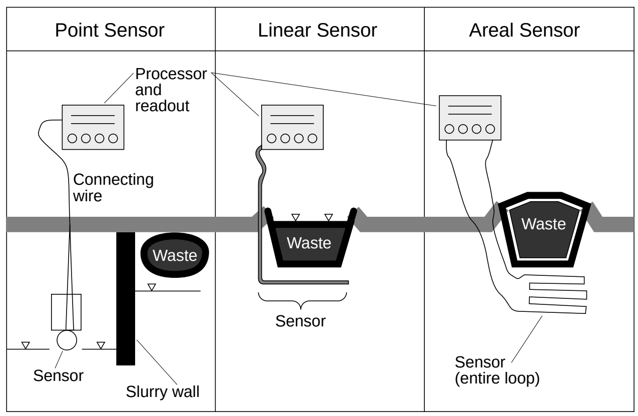

Figure 12-8 Illustration of external monitoring of waste containment structures using point, linear, and areal sensors (Inyang, 1994b)

The Detex Systems and Raychem Corporation sensing cables are depicted in Figures 12-9 and 12-10, respectively. The operation of both cables is based on electrical conductivity measurements, but the mechanism by which voltage drop occurs along the encased wire differs for each sensor. Voltage drop along the Detex Systems conducting cable is caused by the degradation of the coating upon contact with the target substance (hydrocarbons). In contrast, the polymer jacket of the Rachem cable swells upon contacting hydrocarbons, causing contact with an internal conducting wire, resulting in a voltage drop in the circuit. Other physico-chemical mechanisms specific to targeted classes of contaminants or water can be employed. Sensing cables and heads can also be made permeable, exclusively, to water.

Fiber-optic sensors are based on the transmission of probe signals within the visible light regime by optical fibers through very long distances to embedded sensors. Wavelength-dependent optical attenuation of the probe beam or production of fluorescent emissions by the contaminant can be measured and scaled as an index of the presence of the contaminant at the sensor location. Often, an organic dye is added to the sensor to promote fluorescence upon reaction with low concentrations of target compounds. Most of the relevant reactions are reversible. Thus, it is possible for changing contaminant concentration levels to be recorded by a single sensor. The details of a reversible fiber-optic sensor designed to measure carbon tetrachloride (see Figure 12-11) is being developed by Lawrence Livermore National Laboratory (U.S. DOE, 1994b). Currently, fiber-optic sensors are designed for specific classes of contaminants. Figure 12-12 depicts a processing circuit developed for detecting hydrocarbons (DeFilippi and Cody, 1994).

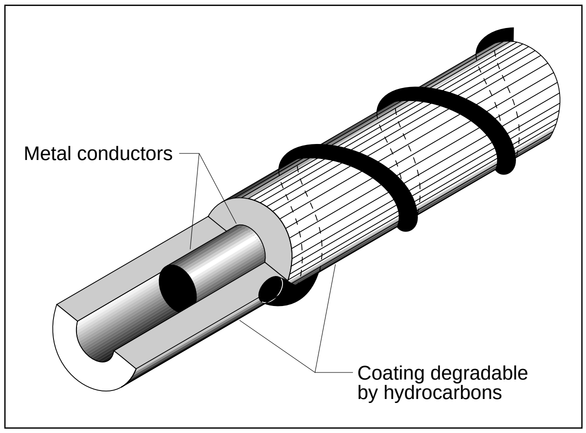

Figure 12-9 Cross section of a Detex Systems electrical conductivity sensor based on coating degradation by hydrocarbons

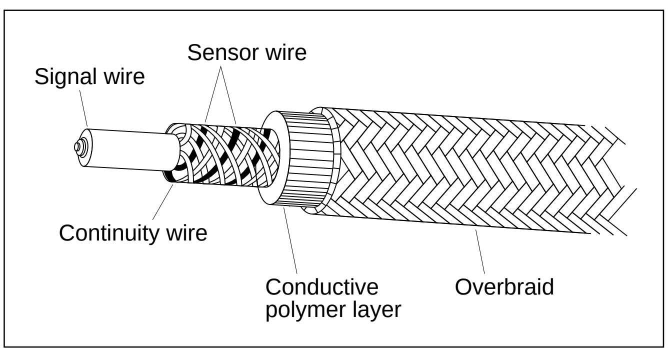

Figure 12-10 Cross section of Raychem Corporation electrical conductivity sensor based on polymer swelling on contact with hydrocarbons

Traditionally, electrochemical sensors have not been installed within subsurface barrier systems. However, within the past five years, advances have been made in sensor development for underground storage tanks. Assessments are being made for installation of sensors in barrier systems. Betsill and Gruebel (1995) cite literature surveys that identified about 25 chemical sensor companies in the United States.

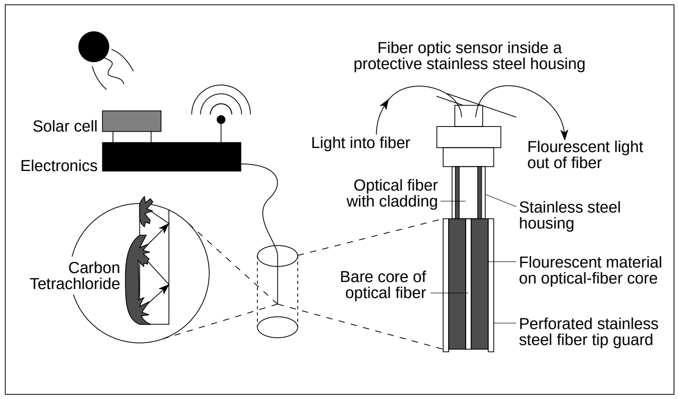

Figure 12-11 Schematic of a reversible fiber-optic carbon tetrachloride sensor (U.S. DOE, 1994b)

12.2.5 Electrical Systems

The term "electrical systems" are herein used generically to cover techniques that involve the passage of electric current through subsurface media or physical interactions between an embedded device and surrounding media that may lead to the transmission of electrical impulse to direct-read systems. These systems include direct-current (DC) resistivity monitors, piezometers, infiltrometers, and flow velocity sensors.

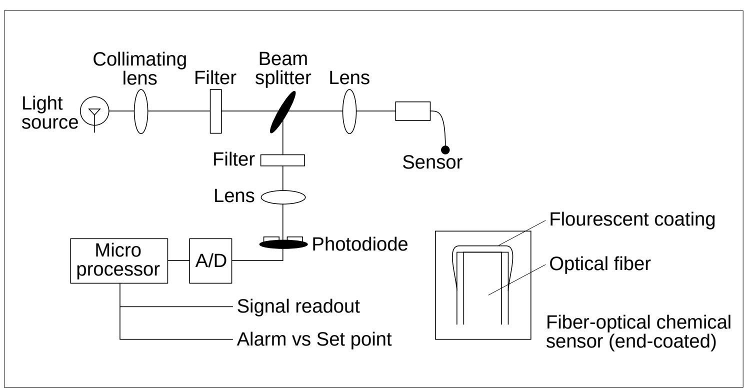

Figure 12-12 Block diagram of a fiber-optic sensor fluorimeter (DeFilippi and Cody, 1994)

a. Direct-Current (DC) Resistivity. This method can be used to detect flaws in geomembranes and the boundaries of a contaminated zone around a containment system. Typically, the frequencies used are so low (under 10

Hz) that no significant electromagnetic current is induced in the ground. The currents are generated by buried electrodes. As in the case of EM resistivity, the properties of the transport medium affect its electrical conductivity or resistivity.

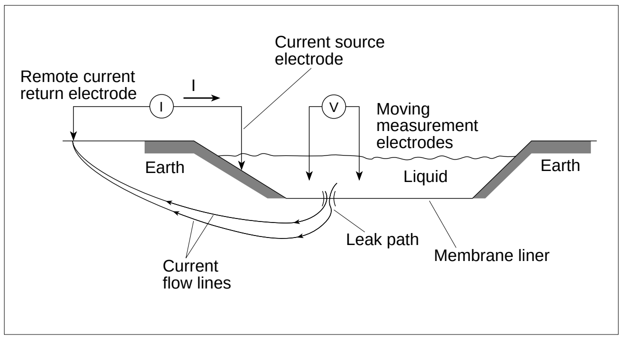

Figure 12-13 Sketch of the electrical leak location method for geomembrane liners (Darilek and Parra, 1988)

For containment systems with barriers that are mostly horizontal and near the ground surface (landfill covers and impoundments), measurements of electrical self potential can be used to locate leaks. As illustrated in Figure 12-13 (Darilek and Parra, 1988), two electrodes are used at a membrane liner of a surface impoundment. One of the electrodes is fixed at a location remote from the impoundment while the other electrode is operated in a scanning mode over the area to be investigated. In the absence of leaks, the voltage

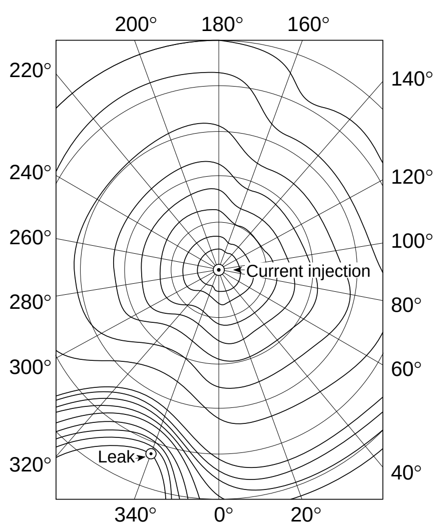

Figure 12-14 Equipotential lines on a liner indicating a leak on the 340° radial (Peters et al., 1982)

imposed on the liner produces a uniform distribution of potential across the surface of the liner. At a leak, conductive fluids permeate the underlying soil to close the electric circuit by producing a path for current flow through the soil to the remote electrode. Figure 12-14 (Peters et al., 1982) illustrates the recording of an anomaly that can result in the equipotential lines around a leak. Due to the high electrical resistivities of geomembranes (data by Peters et al., 1982, suggest values of 10^8-10^{13} ohms), they have been the primary components investigated using this method.

This technique has been successfully used to locate leaks in a one-acre geomembrane-lined impoundment (Schultz et al., 1984). This impoundment facility had design dimensions of 65.8m by 65.8m. Leaks in the 100-ml high density polyethylene liner were identified by the distortion of equipotential voltage lines plotted from DC-resistivity data. Darilek and Parra (1988) conducted similar tests on geomembranes placed between soil covers (see Figure 12-15). Abrupt decreases in recorded voltage during radial traverses from an electrical current injection point indicate the locations of leaks. Thus, the electrical resistivity method can be used for leak location in geomembranes, regardless of whether it is covered by a soil or a liquid. However, the mass of soil over a geomembrane may reduce signal strength and, therefore, require a reduction of the separation distance between the scanner and the geomembrane.

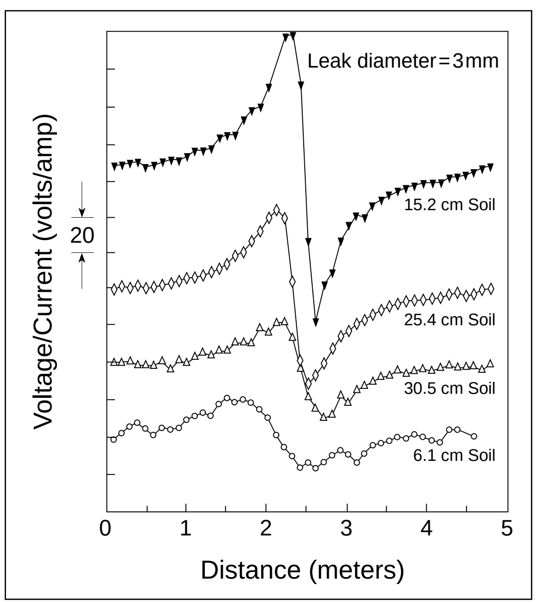

Figure 12-15 Anomalies in voltage/current ratio recorded during electrical resistivity scans of various thicknesses of cover above a geomembrane (Darilek and Parra, 1988)

b. Electrical Piezometers and Infiltrometers. The moisture contents of barriers can affect infiltration rates and barrier structural integrity. Thus, the moisture content of earthen barrier components can directly influence barrier performance. Electrical piezometers and infiltrometers are frequently used to measure pore water pressure and infiltration rates of moisture in earthen materials, respectively. Sensors can be buried in barrier system components and data from these instruments can be converted to electrical impulses for automatic and continuous recording. The most common piezometers operate through the displacement of a fluid inside a tube due to the build-up of a pore water pressure outside the tube. In the arrangement illustrated in Figure 12-16, a U-shaped wire is immersed in a mercury tube and connected to a Wheatstone bridge circuit. The electrical resistance of the wire is proportional to the displacement of the mercury level. An increase in local pore pressure depletes the mercury in the left column and unbalances the Wheatstone bridge of the circuit. As the pore pressure increases in the left tube, the resistance of the left wire increases while that of the right wire decreases. Thus, the voltage output is directly proportional to the measured difference in resistance, ΔR. Equation 12-4 relates the parameters involved.

$$V_o = V[\Delta R/R] = bV[\Delta P] \qquad (12\text{-}4)$$

V_o = voltage output
R = resistance of the wire
P = pressure input
b = scaling factor

Some methods of barrier infiltration analysis, exemplified by the water budget methods, involve the use of mathematical models for computation of the rate of migration of moisture through barriers located in the unsaturated zone. This is commonly done for horizontal barriers such as landfill covers and concrete covers. Tensiometers embedded at various depths in a barrier can be used to track the advance of the wetting front through the barrier during and after hydrological events. Soil suction, which is measurable through the use of tensiometers, reduces to zero under saturated conditions. By monitoring the time distribution of these suction decreases with respect to the depth locations of the tensiometer sensors within the barrier, infiltration rates can be determined.

Several additional physico-chemical principles can be applied to measure moisture content in partially saturated barrier materials so long as water movement by gravitational forces is not significant. Appropriate instrumentation can be installed inside and outside barriers in the subsurface (U.S. EPA, 1993b). The principles of a few of these techniques are briefly outlined below.

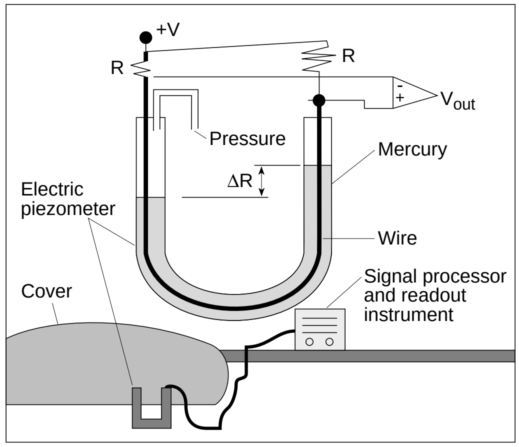

Figure 12-16 Schematic of an electrical piezometer embedded in a landfill cover (not drawn to scale) (Inyang, 1994b)

- Gamma ray attenuation: the degree of attenuation of gamma rays in soil depends on the bulk density and moisture content of the soil. Thus, variations in moisture content can be scaled using gamma ray attenuation data.
- Electrical resistance of embedded gypsum blocks: the electrical properties of an embedded gypsum block changes in proportion to the volume of moisture it absorbs from the surrounding soil.
- Thermocouple psychrometry: the relative humidity of voids within soils is directly related to soil water potential. The latter is the sum of the osmotic and matrix potential. Thus, curves can be developed to relate relative humidity, water potential, and osmotic potential for unsaturated conditions.
- Heat dissipation: the rate of heat dissipation into the surrounding soil from an imbedded heat source is affected by the moisture content of the soil. Using calibration curves developed for the soil/heat source combination in the laboratory, *in situ* moisture content measurements can be made.

12.3 ASSESSMENT OF VARIOUS CATEGORIES OF MONITORING TECHNOLOGIES

12.3.1 System Cost-Effectiveness Factors

The cost-effectiveness of the monitoring systems discussed in the preceding sections depends on the following factors:

- spatial coverage around the containment source,
- resolution and tributary region of the technique and sensor,
- release-to-detection time interval,
- feasibility of automatic operation, and
- invasive versus non-invasive operation.

12.3.2 Cost-Effectiveness of Groundwater Well Systems

Currently, groundwater monitoring wells and leachate collection sumps are the most common systems for monitoring containment system performance. In the United States, waste containment units placed into inactive status prior to November 19, 1980 are generally not required to comply with groundwater monitoring requirements. The common monitoring well configuration (one well upgradient and three wells downgradient around the containment system) is inadequate for cost-effective monitoring of containment system performance since the release-to-detection time interval may be too long, allowing contaminants to short-circuit the monitoring system because of inadequate spatial coverage provided by only a few. Leachate collection and testing is labor-intensive because it involves a sequence of tasks, including: collection, storage, and analytical testing of fluids. Furthermore, the concentrations of contaminants in samples collected from observation wells represent composite values and do not provide information on concentrations at specific exit points from the containment system. In a recent study of 890 leaking waste containment facilities (GAO, 1995), 107 facilities had groundwater monitoring systems incapable of detecting the releases. An additional 153 facilities had monitoring systems that did not provide adequate information for assessing the extent of contaminant releases.

12.3.3 Cost-Effectiveness of Geophysical Techniques

Geophysical methods are cost-effective when used to assess the continuity and dimensions of emplaced and natural barriers. Large areas can be monitored and some methods are non-invasive. However, the resolution of some geophysical techniques such as GPR, seismic refraction, and resistivity profiling may be insufficient for detecting flaws which may serve as moisture and contaminant flow channels through barriers.

12.3.4 Cost-Effectiveness of Electrochemical Sensing Techniques

Electrochemical sensing systems can provide higher spatial coverage at lower cost. Unlike the case of groundwater monitoring wells, the unit cost of increasing monitoring points is relatively modest. Data can be acquired from sensors automatically, and the release to detection time interval can be minimized by placing a dense network of sensors around the containment system. Electrochemical monitoring systems do have one drawback in that sensors are usually fabricated to detect a narrow range of contaminants, implying that the contaminants (or class of contaminants) have to be identified a priori. Furthermore, sensor repair and reinstallation techniques have not yet been developed fully.

12.3.5 Relative Cost-Effectiveness of Various Techniques

The cost-effectiveness of the various monitoring systems depends on the specific performance aspects to be monitored, the desired accuracy, and the frequency of measurement. Each technique (for example, geophysical systems) involves several methods that may have widely different levels of cost-effectiveness when used to monitor a specific aspect of containment system performance. Thus, the patterns of cost-effectiveness illustrated in Figure 12-17 (Inyang, 1994b) are intended to represent general trends only. Geophysical techniques may be the most cost-effective for external contaminant concentration monitoring over large areas, and less cost effective for small monitoring areas, due to the large capital cost investment in equipment. Electrochemical sensor systems and groundwater well monitoring systems exhibit a decrease in cost-effectiveness as the monitoring zone increases in size. This is attributable to the high number of wells or sensors that need to be installed and operated to maintain an acceptable level of probability that contaminant leaks will be detected.

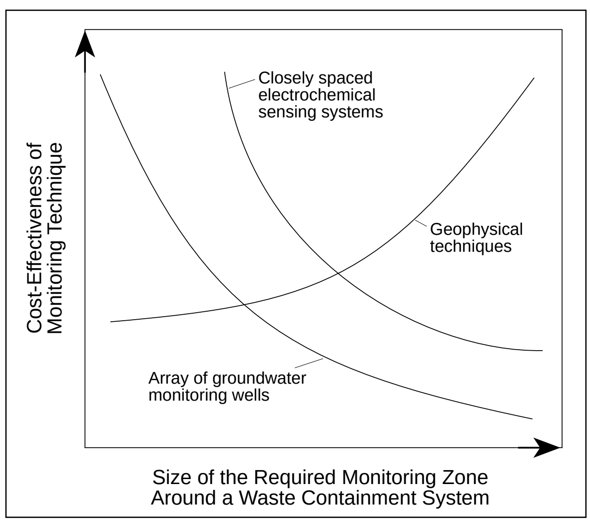

Figure 12-17 Conceptual relationship between the size of monitored area and the cost-effectiveness of various monitoring methods (Inyang, 1994b)

To illustrate the relative cost-effectiveness of the techniques shown in Figure 12-17, a hypothetical landfill monitoring case is used as an example. The cost estimates used are drawn from Inyang (1994b). The hypothetical landfill contains oily wastes, requiring a 20-year post-construction monitoring period. For this analysis, the monitoring system design objectives involve:

- detection of any volatile organic compounds or metals released into the groundwater, and
- location of the contaminated groundwater plume if a release occurs.

The selected arrangement of monitoring points for each of the techniques complies with regulatory guidelines (U.S. EPA, 1991d). The monitoring networks selected are approximately of the same effectiveness, in order to facilitate comparison of cost-effectiveness. However, the size of the area to be monitored has been varied in order to assess the extent to which cost is affected by changes in areal coverage.

Cost estimates for off-site equipment are not included in this example since, in most monitoring programs, analytical testing is usually done off-site by a certified laboratory. Such off-site equipment usage is included in the service charges. Thus, only on-site equipment costs are included in capital cost estimates. Inflation has been factored into the operation and maintenance costs; thus, annual average costs given have been computed for the specified 20-year post-construction monitoring period.

The example site conditions include an aquifer ten feet below the bottom landfill liner; thus, the primary concern is leachate contamination of this aquifer. A semicircular area of 400 ft radius, bordering the landfill on the down-gradient side, is to be monitored. The number of monitoring wells or sensor locations required can be estimated using the following relationships:

$$A_m = \pi R^2/2 \tag{12-5}$$

$$n_m = A_m/a_t = A_m/S^2 \tag{12-6}$$

A_m	= semicircular area to be monitored [L^2]
R	= radius of A_m [L]
n_m	= number of monitoring wells or sensor locations
a_t	= area to be serviced by each well or sensor [L^2]
S	= linear spacing of monitoring wells or sensors [L]

For this example, $S = 80$ ft, approximating the average of values reported in the literature (McNeil, 1982 and Jensen et al., 1992), and $R = 400$ ft, giving A_m @ 251, 327 ft^2, and n_m @ 39. The required number of wells or sensors for different values of A_m are presented in Table 12-5.

TABLE 12-5 The relationship between the size of the monitoring area and the required number of monitoring points for the hypothetical case

MONITORING AREA (ft^2)	LINEAR SPACING OF MONITORING POINTS (ft)	REQUIRED NUMBER OF MONITORING POINTS (approximated to the next whole number)
20,000	80	3
50,000	80	16
150,000	80	24
200,000	80	32
250,000	80	40

The following assumptions have been made in computing and comparing the 20-year costs of installing and operating the three monitoring systems:

- Groundwater Monitoring Wells
  - installation cost: $5,000 per well (Weber et al., 1991)
  - chemical analyses: Samples collected semiannually and tested in accordance with protocols described in SW-846 (U.S. EPA, 1991d). Cost estimates are as follows (Brown, 1993): EPA Method 8260 for VOCs at $600 for two tests per well per year, amounting to $12,000 per well for 20 years; and EPA Method 6010 for metals at $300 for two tests per monitoring well per year, amounting to $6,000 per well for 20 years.
  - operational and management costs: Labor and overhead costs associated with sample collection, system maintenance, and record keeping estimated at $5,000 per year, amounting to $100,000 for 20 years (not dependent on number of monitoring points).

- Electrochemical Sensing
  - data processor: $5,000 (quotation by Raychem Corp.)
  - below ground sensor cables: 100 ft length @ $12/ft = $1,200 for each sensor (Raychem Corp.)
  - above ground connecting cables: $300 per sensor
  - installation costs: $400 per sensor
  - operational and management costs: estimated at $6,000 per year, or $120,000 for 20 years

- Geophysical System
  - electromagnetic resistivity: measurements at pre-selected locations semi-annually
  - outfitted vehicle: $20,000 (for acquiring measurements)
  - data processor/display: $15,000
  - electromagnetic wave generator and receivers: $10,000
  - data management/interpretation software: $1,500

- operational and maintenance costs: \$9,000/yr, or \$180,000 for 20 years (vehicle and system maintenance, labor costs, transportation costs, etc.)

The cost-estimates for the three monitoring methods are summarized in Table 12-6. In this hypothetical example, geophysical sensing has the highest cost for the smallest monitoring area (20,000 ft^2). However, for a monitoring area of 250,000 ft^2, the cost of geophysical resistivity sensing if only about 23% of that required for groundwater monitoring using wells. Electrochemical sensing is the most cost-effective system for the example given here. It is conceivable that the geophysical sensing system could become the most cost-effective for areas greater than 250,000 ft^2.

TABLE 12-6 Cost comparisons of the containment detection techniques for three monitored area sizes using the assumptions discussed in the text

MONITORING TECHNIQUE AND COST ITEM	20-YEAR COST FOR AREA OF SIZE, A_m, TO BE MONITORED						
	UNIT COST (\$)	Am = 20,000 ft^2		Am = 150,000 ft^2		Am = 250,000 ft^2	
		Number Required	Total Item Cost (\$)	Number Required	Total Item Cost (\$)	Number Required	Total Item Cost (\$)
GROUNDWATER MONITORING WELLS							
•Well installation	5,000	3	15,000	24	120,000	40	160,000
•Chemical analyses	18,000 (a)	3	54,000	24	432,000	40	720,000
•Operation and management	100,000	-	100,000	-	100,000	-	100,000
[(a) per well for 20 years]							
20-YEAR TOTALS			169,000		652,000		980,000
ELECTRO-CHEMICAL SENSING							
•Central electronic unit	5,000	1	5,000	1	5,000	1	5,000
•Sensing cables	1,200	3	3,600	24	28,800	40	48,000
•Connecting cables	300	3	900	24	7,200	40	12,000
•Sensor installation	400	3	1,200	24	9,600	40	16,000
•Operation and management	120,000	-	120,000	-	120,000	-	120,000
20-YEAR TOTALS			130,700		170,600		201,000
GEOPHYSICAL SENSING							
•Field vehicle							
•Wave generator and		1	20,000	1	20,000	1	20,000
receivers	20,000	1	10,000	1	10,000	1	10,000
•Data interpretation software	10,000 (b)	1	1,500	1	1,500	1	1,500
•Data processor and	1,500	1	15,000	1	15,000	1	15,000
displayer	15,000	-	180,000	-	180,000	-	180,000
•Operation and management	180,000						
[(b) per set]							
20-YEAR TOTALS			226,500		226,500		226,500

12.4 CONTAINMENT SYSTEM MONITORING NEEDS

12.4.1 Sensor Performance Needs

The performance of sensors plays a key role in the quality of monitoring data obtained for use in the analysis of waste containment system effectiveness. The most critical sensor performance needs are summarized as follows:

- sensor must perform in aggressive physico-chemical environments,
- sensors need high bandwidths so that a wide array of contaminants can be detected,
- sensors must be immune to electromagnetic interference but maintain high sensitivity to contaminants (in the range of ppb to ppm),
- sensors must perform dynamically (record contaminant concentrations regardless of sequence: high before low, or low before high), and
- *in situ* sensors that can track contaminant and/or moisture flow rates.

12.4.2 Barrier-type Specific Needs

A summary of techniques for monitoring each type of containment component is presented in Table 12-3. The most important monitoring needs for specific barrier types are summarized as follows:

a. Indigenous and Artificial Floors, and Chemically Based Grouts

- monitoring systems that can delineate the perimeter and continuity of the barrier,
- sensors that can detect leaks with spatial resolution on the order of a few centimeters,
- high resolution imaging systems, and
- tracers that can effectively delineate possible pathways for moisture and contaminant transport through the barrier.

b. Caps (Soil-based and Concrete)

- monitoring systems that can detect and quantify settlement and side-slope deformation,
- monitoring systems that can detect the creation of cracks and fissures,
- monitoring systems that quantify moisture content and flow rates at critical points within the cover,
- monitoring systems that can detect excessive deterioration in concrete quality, and
- monitoring systems that can accurately detect relative movements between geomembranes and cover soils.

c. Sheet Piles and Geomembranes

- interstitial monitoring system for double-wall configurations,

- monitoring systems that can measure the tightness of interlocks, seams, and keys into geomedia,
- sensors that can detect flaws through which contaminants can flow,
- *in situ* sensors that can measure the rates of barrier deterioration processes, and
- geotechnical monitoring methods for barrier deformation.

d. Permeable Reactive Barriers

- monitoring system for *in situ* determination of contaminant concentration and flow velocity upgradient and downgradient,
- sensors for tracking the concentrations of reactants and products at points within reactive barriers, and
- monitoring systems for verification of plume capture.

e. Soil and Cement-based Vertical Barriers

- systems for monitoring hydraulic heads on either side of the vertical barrier,
- devices for *in situ* measurement of hydraulic conductivity,
- methods for non-intrusive determination of wall embedment,
- automatic moisture sensing systems for wall portions above the water table, and
- monitoring systems for measuring contaminant concentrations up-gradient and downgradient from vertical walls.

12.4.3 Installation and Retrieval Needs

Emplacement techniques need to be made more cost-effective, since sensors embedded in containment barriers during construction may need to be replaced during the design lifetime of the system. There is a need for minimally intrusive retrieval and reinstallation methods for sensors. The use of remote data acquisition techniques could eliminate the need for direct connection of sensors to above ground data processing systems. A method for pinpoint location of isolated sensors in subsurface barriers is needed, since ground movements may displace sensors from their original positions during the design lifetime of the containment system.

12.4.4 Monitoring Network Needs

Betsill and Gruebel (1995) have presented a detailed discussion of monitoring system network needs. The most critical needs are as follows:

- automated data and sample collection analysis system that can record and evaluate changes in contaminant concentrations and barrier characteristics (a sensor network is depicted in Figure 12-18),

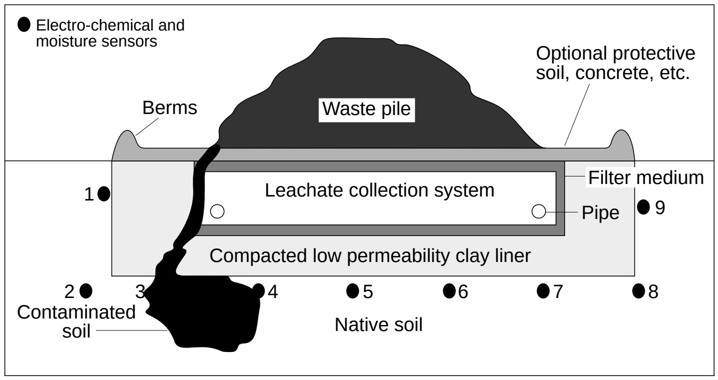

Figure 12-18 Schematic showing installation points of electrochemical and moisture sensors for detection of contaminants beneath a waste pile (Inyang, 1994b)

- passive monitoring systems and alarms (smart sensors) that can alert personnel to failures and malfunctions,
- data management and archiving systems that can eliminate the need for excessive duplicate data sets and simplify future data retrieval and analysis, and
- spatial referencing systems for embedded sensors that can indicate barrier conditions at specific locations in the subsurface (these systems can be integrated with tomography as illustrated in Figure 12-19).

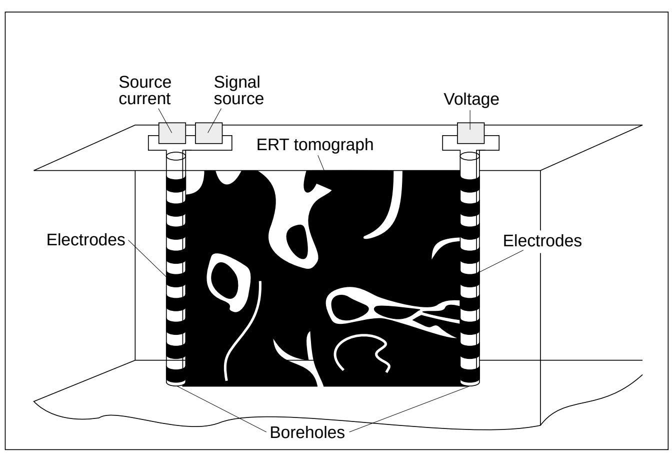

Figure 12-19 A circuit for an electrical resistance tomography system (U.S. DOE, 1994a)

12.4.5 User Interface Needs

- operational computer software for acquisition, interpretation, and visualization of spatial and temporal monitoring data,
- facilities for testing of sensor performance in real-time "dirty" environments that simulate field conditions,
- development of query systems for sensor networks and systems for integrating and reconciling monitoring data obtained using different techniques,
- data analysis systems that can identify out-of-compliance situations,
- a central data bank for storage of data that can be analyzed to show relationships among containment system failure, site characteristics, waste types and system design, and
- user-friendly systems for manipulating collected data to verify and refine both damage accumulation and contaminant transport models.

12.5 SUMMARY AND RECOMMENDATIONS

12.5.1 Barrier Component-Specific Monitoring Methods

There are several monitoring techniques which can be used to track the post-construction performance of waste containment systems. In most cases, it is necessary to combine a number of monitoring techniques in order to acquire the variety of data required to perform comprehensive analyses. Everett et al. (1984a and 1984b) discuss the applications of various vadose zone monitoring techniques. The techniques discussed here focus on flow monitoring.

Table 12-4 provides a general summary of the applications of various monitoring techniques to different types of flaws and deterioration modes in barriers, moisture and contaminant flow rates through barriers, and the distribution of contaminant concentrations around barriers. Table 12-3 offers information on the general utility of the categories of monitoring techniques to the three main monitoring objectives: integrity monitoring, permeation monitoring, and external concentration monitoring. Table 12-4 can be used in conjunction with Table 12-3, with consideration of the following additional points.

- In all cases, groundwater wells with installed piezometers constitute the most common monitoring method currently used.
- A double-walled internally monitored system is a potentially effective monitoring method for vertical walls, but it is not being implemented.
- For indigenous floors, vertical borings/wells can be driven into the top of the floor, and horizontal wells can be driven below the floor. The horizontal well can also be used for contaminant recovery, if necessary.

- For all barrier types, especially, newly developed systems exemplified by permeable reactive walls, monitoring systems should be more conservative for field demonstrations than for commercial applications.
- Liquid and gas tracers are particularly useful for verifying the integrity of cryogenic barriers.
- Piezocone soundings and other penetrometer-borne sensors have been effectively used to measure *in situ* chemical concentrations and hydraulic conductivities in slurry walls. It may be necessary to use grouts to prevent cross-bed contamination.
- Moisture content sensors have been used effectively in cap system monitoring. However, the following limitations and necessary improvements apply:
  - gypsum blocks saturate easily and are suitable for short-term use;
  - neutron moisture sensors need calibration standards;
  - pressure-driven vacuum lysimeters need minimization of gradient to reduce changes in pore-fluid chemistry, and bubbling pressure needs to be sufficient to operate over the full range of capillary pressures;
  - frequency and time-domain reflectance methods are susceptible to soil moisture salinity.

12.5.2 Adoption of Some Available Innovative Technologies

The merger of material science, structural mechanics, sensor and actuator technologies, and advanced data processing techniques has accelerated the development of "smart structures" or "intelligent structures." This development has provided opportunities for incorporating effective neural and responsive systems into waste containment monitoring systems. The intelligent structure concept and technology are being refined for automobiles, robots, high performance aircraft, bridges, and highways. Within the next few years, it should be possible to use these systems to track environmental and physical changes within and around containment systems, interpret such changes, and devise appropriate responses to these changes. The evolving generation of fiber optic sensors promises to be low-cost, lightweight, immune to electromagnetic interference, and rugged enough to be incorporated directly into monitored barrier components. They will have high bandwidths and will perform under temperature extremes. For example, silica-based fibers have an upper operating temperature range of 700°C.

Sensor research conducted by private industry is another potential resource for monitor development. For example, at the Microswitch Division of Honeywell, researchers have developed the concept of smart sensors for heavy manufacturing applications. These sensors use on-board intelligence to link themselves to programmable logic controllers (PLC's) and transmit information regarding their working status (e.g., calibration data) to evaluate the system's ability to function within designated parameters (Murray, 1994). Other companies, such as Motorola, Philips Components, and Texas Instruments are working on smart sensor technology (Ormund, 1993). Although smart sensors are currently utilized primarily for heavy manufacturing, the technology may provide insight for developing "smart" monitoring sensors for environmental applications.

Some of the additional specific sensors that have been developed or are being developed include:

- sensors for use with the cone penetrometer for site characterization activities, including soil moisture and chemical concentration measurements;
- down-hole x-ray fluorescence metals analysis;
- reversible, fiber optic sensors for detection of specific VOC contaminants (e.g., methylene chloride) in the parts per billion (ppb) range;
- real-time detection of radionuclides and other contaminants during drilling;
- robust organic vapor monitors for harsh field environments and conditions;
- on-site analysis of metals using adsorptive stripping analysis;
- portable apochromatic/fiber optic detector for VOCs in air and water—a potential sensor with alarm for initial detection in field applications; and
- borehole liners with real-time VOC monitoring systems.

There is the possibility of developing barrier materials that can be more easily monitored after placement. The physical characteristics of such barriers should contrast sharply with those of the host media. Also, ultra-conductive grouts would render geophysical techniques more effective as monitoring techniques. A number of innovative technologies, developed as part of the U.S. Department of Energy (U.S. DOE) Environmental Management Programs, are described below.

a. Heavy-Weight Cone Penetrometer. This truck-mounted ground penetration device consists of a rod with a 1.75 inch diameter conical tip. The cone penetrometer can be equipped with instruments to monitor for contaminants and geophysical data as the penetrometer is pushed into the ground or to leave monitoring devices in place as the penetrometer is withdrawn. Specialty penetrometer probes are capable of measuring electrical resistivity, hydraulic conductivity, gamma radiation, temperature, soil density, fluorescence (laser induced), pH, etc.

b. Seamist. This is a liner that is deployed inside an open borehole or a well and instrumented with sensors and fluid samplers to assist with *in situ* characterization, monitoring, and remediation. Applications include soil vapor and groundwater sample collection.

c. Halsnif. This is a portable detection device that interacts helium with radio frequency signals in an excitation chamber to measure chlorinated compound concentrations in real time. The device can be used with the cone penetrometer.

d. Cross Borehole Electromagnetic Imaging. This is a source and plume detection technique that uses continuous wave and pulsed radar systems to provide a means of imaging the subsurface with plumes of nonaqueous phase liquids, drums, tanks and cylinders. It provides a mapping of electrical permittivity between boreholes.

e. Rapid Geophysical Surveyor (RGS). The RGS characterizes a buried waste site by collecting high quality, dense sets of magnetic data. It is a manually-maneuvered vehicle that carries a cesium total field magnetometer, a data logger, and data storage hardware and software.

f. Surface Acoustic Wave Sensors (SAWS). This sensor is being investigated by the DOE/EM for contaminant detection. However, the Penn State University has also developed a version of SAWS for implantation in the roadbed of highways or bridges to detect ice formation, flooding, dangerously heavy traffic, and cracks forming in support structures. Each sensor contains a miniature microwave antenna and a piezoelectric transducer that converts microwaves into sound waves. As the sound waves travel through the structures, any strain or physical abnormalities on the structures (bridges or roads) alters the speed and strength of the sound waves which can be detected by other SAWS. The sensors are small and can be emplaced throughout the structure. Signals are fed back to a control center for analysis.

g. Colloidal Borescope. The borescope is used as an *in situ* instrument to directly observe the motion of colloidal size particles, thereby determining groundwater flow direction and rate. The instrument consists of a charge coupled device (CCD) camera, an optical magnification lens, an illumination source, a downhole compass to assess direction of natural flow, and a watertight stainless steel housing. It is approximately 60cm long, with a diameter of 4.4cm. The electronic image is transmitted to the surface by a fiber optic cable and the image is viewed on a high resolution monitor. The instrument can be used in a well as small as 5cm in diameter and measurements can be obtained in about 30 min. The colloidal borescope is capable of determining the vertical and spatial distribution of local groundwater velocity, both magnitude and direction. It is capable of these measurements in low and high hydraulic conductivity materials.

Chemical manufacturing companies are currently installing automatic materials inventory data acquisition systems. This technology is adaptable to contaminant leak detection. The Eco Instrumentation Corporation (EIC) Remote Access Inventory Monitoring System is illustrated in Figure 12-20. Liquid level is monitored using fiber-optic or other sensors. The sensors transmit data to a processing unit at the site. Using ratio frequency signals, the interpreted information is broadcast to computer systems at offices or control centers off-site. The operator can query the system after the occurrence of a natural hazard such as an earthquake or flood to determine the extent of facility damage and loss of stored fluids.

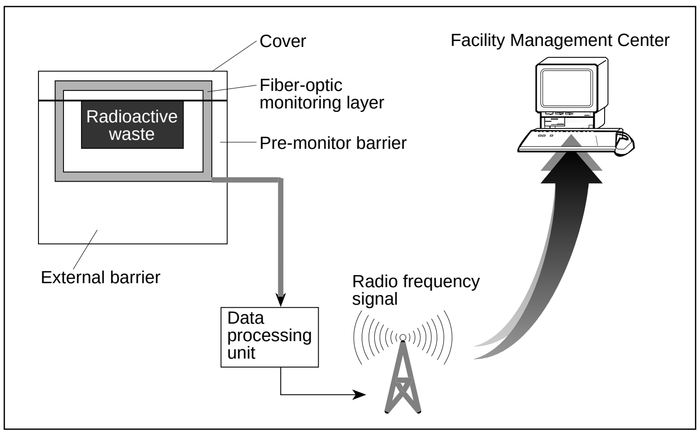

Figure 12-20 Conceptual design of EIC remote access inventory monitoring system

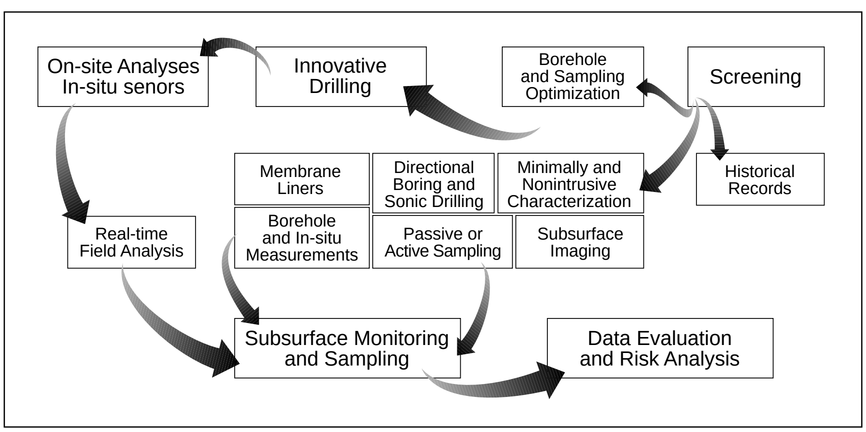

Figure 12-21 The Landfill Assessment and Monitoring System (LAMS) (U.S. DOE, 1995)

The U.S. Department of Energy has developed the Landfill Assessment and Monitoring System (LAMS). This system (illustrated in Figure 12-21) is based on the application of a systems approach to monitoring, with resulting maximizations of data gathering, and minimization of costs, worker exposure, and sampling time. The Expedited Site Characterization (ESC) program focuses on the use of a variety of non-intrusive and minimally intrusive technologies (e.g., surface geophysics, cone penetrometers, hydropunch sampling, vegetation sampling) to optimize sampling locations and minimize monitoring well installation (U.S. DOE, 1995).

12.5.3 Use of Multi-Phased Monitoring Approach

A systematic approach to the selection, implementation, and operation of a containment system monitoring strategy should be adopted. Considering that there are several alternative technologies, monitoring objectives, and barrier configurations, a single technology may not be cost-effective for all applications. Inyang (1994b) has proposed the approach illustrated in Figure 12-22 as the sequence of steps that can be used to select and implement a containment system monitoring strategy. Each step involves analyses which provide input into subsequent steps and monitoring activities.

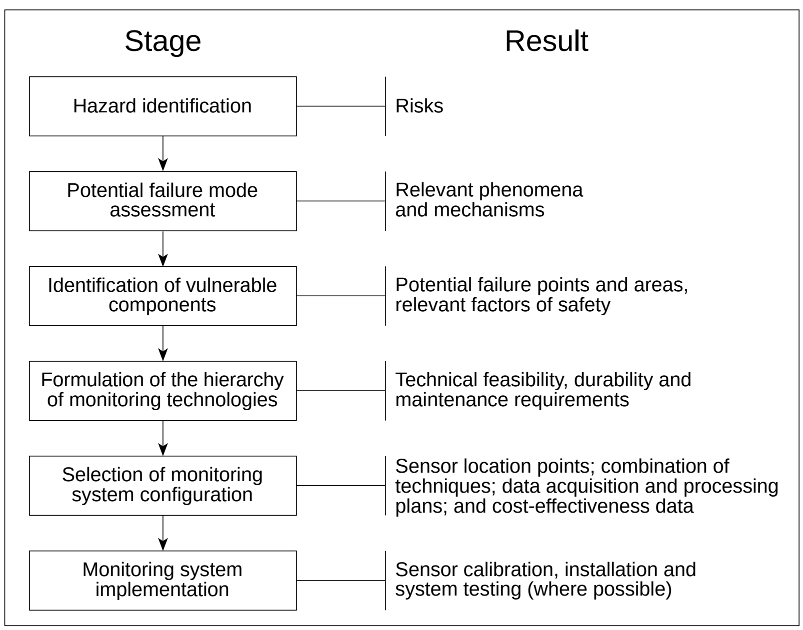

Figure 12-22 Flow chart of the evaluation and selection process for innovative monitoring technologies for subsurface waste containment barriers (Inyang, 1994b)

12.5.4 Specification of System-Specific Failure Rate

Natural and constructed facilities exposed to environmental and human-induced stresses deteriorate as time progresses. This is especially true of waste containment systems because they may be exposed to aggressive environments for time periods that range from decades to centuries. Inyang (1994a), Kargbo et al. (1993), Flemming and Inyang (1995), Kim and Kim (1991) and Peterson (1990) have discussed containment system deterioration processes. However, structural deterioration of some components of a multicomponent system may not always lead to total functional failure of the system. A uniform approach needs to be developed for specifying the "failure condition" of containment systems. Monitoring data can be combined with numerical models to forecast future performance levels, risks, maintenance requirements, and clean-up logistics for constructed containment systems.

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