A Review of Non-Traditional Dry Covers

MEND Report 2.21.3b

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By

Dr. M.D. Haug, Ph.D.
Gordon Pauls, M.Sc.
Dept. of Civil Engineering,
University of Saskatchewan

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Executive Summary

A review of non-traditional materials used in environmental cover applications was conducted to determine which of these materials have potential for use in acid rock/acid mine drainage cover systems. The following non-traditional materials were reviewed:

- asphalt covers,
- cementitious covers (polypropylene fibre reinforced shotcrete),
- cement-stabilized coal fly ash grout (Cefill, fly ash mixtures, and geopolymers),
- synthetic liners and covers (geomembranes, spray-on membranes barriers, and geosynthetic clay liners),
- bentonite modified soil barriers (soil-bentonite mixtures, polymer modified soil, and polymer surfactants),
- mine waste (tailings and waste rock), and
- wax barriers.

The above non-traditional cover materials can all be characterized as possessing low hydraulic conductivity. A majority of the case studies reviewed were focused primarily on the nature of the hydraulic barrier (i.e. measuring saturated hydraulic conductivity). This may be adequate for landfill and other similar waste applications where water infiltration alone needs to be controlled. In AMD applications, it is also necessary to prevent the transfer (ingress) of oxygen to the underlaying waste. Also, it is desirable that the cover systems provide a medium for the development of sustainable vegetation.

Most of the non-traditional cover materials reviewed have the potential to be used as

hydraulic barriers to control infiltration into acid generating waste piles. Some of the non-traditional cover materials may also be effective oxygen barriers. These materials would include bentonite modified sands, asphalt, spray-on membranes and wax. Other materials such as Cefill, geosynthetic clay liners and soil modified barriers can also be effective oxygen barriers if they are maintained in a saturated state.

The most attractive non-traditional cover materials for AMD applications are mining wastes (tailings and waste rocks) since these materials are already on site. These materials may need to be modified with bentonite or possibly fly ash to make the tailings and waste rock suitable as cover materials. The use of bentonite modified sand appears to provide one of the best combination of low permeability water barrier and oxygen ingress barrier, however, this material may be more costly than using mining wastes. Many of the remaining non-traditional cover materials have potential uses especially as a redundant hydraulic barrier in multi-layer cover systems using capillary barriers and capillary breaks.

Sommaire

Une étude a été réalisée dans le but de déterminer lesquels, parmi les matériaux non traditionnels mentionnés ci-après, peuvent être utilisés comme couvertures sur les haldes de stériles et les parcs à résidus miniers potentiellement acidogènes afin de protéger l'environnement contre le drainage acide.

• Les matériaux bitumineux.

• Les matériaux de cimentation (béton projeté renforcé de fibres de polypropylène).

• Les coulis à base de cendres volantes provenant de la combustion du charbon et

stabilisées au ciment (Cefill, mélanges de cendres volantes et géopolymères).

• Les membranes et barrières synthétiques (géomembranes, membranes à projeter et couches d’argile géosynthétiques).

• Les barrières d’étanchéité bentonitiques (mélanges terre-bentonite, terre modifiée par polymères et surfactifs polymériques).

• Les déchets miniers (résidus et stériles).

• Les membranes de paraffine.

Les matériaux mentionnés précédemment présentent tous une faible conductivité hydraulique. Dans la majorité des cas, l’étude a principalement porté sur la nature et l’efficacité de la barrière hydraulique (par la mesure de la conductivité hydraulique saturée). Ceci peut convenir dans le cas de dépotoirs et autres lieux d’enfouissement ordinaires, où seule l’infiltration d’eau doit être contrôlée. Dans le cas des sites miniers à drainage acide, il importe également d’empêcher la diffusion de l’oxygène dans les déchets miniers. De plus, il est souhaitable que les matériaux de recouvrement utilisés assurent un milieu propice à la croissance d’une végétation renouvelable. La plupart des matériaux non traditionnels étudiés possèdent les caractéristiques nécessaires pour pouvoir être utilisés comme barrières destinées à empêcher les infiltrations d’eau dans les déchets miniers acidogènes. Certains peuvent également constituer des barrières efficaces contre la diffusion de l’oxygène. Il s’agit des sables bentonitiques, des matériaux bitumineux, des membranes à projeter et des membranes de paraffine. Maintenus dans un état saturé, le Cefill, les couches d’argile géosynthétiques et les barrières de terre modifiée peuvent être tout aussi efficaces.

Les matériaux de recouvrement les plus intéressants pour contrer le drainage acide sont les déchets miniers (résidus et stériles), puisque ces matériaux se trouvent déjà sur le site. Pour les rendre plus adéquats, il peut être nécessaire de leur ajouter de la bentonite ou encore des cendres volantes. Le sable bentonitique, toutefois, semble être le matériau le plus efficace, à la fois comme barrière à faible perméabilité à l'eau et comme barrière à l'oxygène, mais plus coûteux par contre que les déchets miniers. Plusieurs des autres matériaux non traditionnels peuvent être utilisés notamment comme barrière redondante dans les couvertures multicouches comportant des barrières capillaires et des bris de capillarité.

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1.0 Introduction

A MEND project (2.21.3a) with the University of Saskatchewan was initiated in 1995 to complete a peer review of dry covers with special emphasis on the Waite Amulet and Heath Steele covers. The scientific peer review of dry covers by an independent expert will determine whether any additional experimentation is required prior to the development of protocols for the design, construction and maintenance of engineered soil covers over sulphide tailings and waste rocks, to be prepared under a subsequent project (MEND Project 2.21.4) The peer review will ensure the quality and completeness of the field data and interpretations which will provide the basis for developing the various protocols and will enhance the scientific credibility of several key project reports. The review will also assist mine operators and regulators by increasing public confidence in the technical feasibility and cost of engineered soil covers. The objectives of the first study (MEND Project 2.21.3a) were to conduct a literature review of processes and methods of soil cover analysis. While the tasks described in that work attempted to thoroughly encompass the “dry cover” area, it did not attempt to address the many non-soil based covers that are gaining some prominence. The objective of this present project (MEND Project 2.21.3b) was to conduct a review and assessment of non-traditional soil and synthetic cover materials for potential use in acid mine drainage cover applications.

This report presents the results of the review into the applications and performance of non-traditional materials used in environmental cover applications. The term non-traditional cover materials is intended to include any cover materials other than compacted earth. Acid mine drainage (AMD) is a major problem facing the mining industry. One of the most common methods of controlling the generation of AMD is to cover exposed waste rock in order to prevent the infiltration of precipitation and ingress of oxygen. Both oxygen and water are required to generate AMD. Considerable research has been conducted into the use of single- and multi-layered compacted earth covers to control AMD. However, suitable soil may not be available in close proximity to the site to construct a compacted earth cover.

This report describes the use and performance of non-traditional cover materials for environmental cover applications. In most cases, these covers were not constructed to control AMD, however, these covers were subjected to the same physical, chemical and biological processes, which degrade all covers. As a result, the review of the performance of these non-traditional cover materials provides an insight on their potential application as materials suitable for AMD covers. Detailed information, where available are documented on the properties, performance and costs relating to these non-traditional material in their particular application environment. The non-traditional cover materials reviewed include: 1) asphalt, 2) cementitious covers (including shotcrete), 3) fly ash mixtures and geopolymers, 4) flexible membrane liners (FML's) which include geomembranes and GCLs (geosynthetic clay liner's), 5) modified soil barriers incorporating bentonite and/or polymers, 6) mine wastes such as tailings and waste rocks, and 7) wax.

The topic of oxygen consuming covers, which incorporate organic materials was not included in this report. A report by Pierce (1992) (MEND Report 2.25.1) already exists on this topic.

2.0 Asphalt Covers

2.1 Introduction

The use of asphaltic covers for mine tailings had been investigated by the U.S. Department of Energy's (DOE) Uranium Mill Tailings Remedial Action Project (UMTRA) office. In 1976, Pacific National Laboratory (PNL) conducted laboratory tests to determine the effectiveness of sprayed-on asphalt emulsion seals (Koehmstedt *et al.*, 1977) in cover applications.

Hydraulic asphalt concrete (HAC), used as liners for hydraulic structures and waste disposal sites, were compacted hot mixtures of asphalt cement and mineral fillers. HAC is similar to hot mix asphaltic concrete used for highway paving, except that it has a higher percentage of mineral fillers, a higher percentage of asphalt cement, and lower air void content (Asphalt Institute, 1976; Matrecon, 1980). Hydraulic asphalt concrete can be compacted to have a hydraulic conductivity less than 1×10^{-9} m/s (Matrecon, 1980 and 1988).

2.2 Asphalt Stability

Hartley *et al.* (1981) identified five factors which may contribute to the degradation of an asphalt barrier on uranium mill tailings: 1) oxidation, 2) microbial attack, 3) aqueous leaching, 4) temperature cycling, and 5) subsidence of waste. These factors would also apply to asphalt barriers used for prevention of AMD on sulphidic wastes.

Oxidative degradation of asphalt may be catalyzed by light or microbial attack (Hartley *et al.*, 1981). Photo-oxidation can be eliminated by covering the asphalt with overburden. Oxygen may be available at the asphalt-overburden interface in either the dissolved or gaseous state. Initial oxidative weathering of asphalt causes an asphaltene skin to form on the surface. The asphaltene skin will slow oxygen transport into the bulk asphalt and retard the rate of oxidation (Hartley *et al.*, 1981).

Poly-condensation reactions cause increases in average molecular size as asphalt ages. These reactions are largely the result of oxidative coupling reactions, in which small molecular weight materials are combined into larger molecules. This increase in molecular size is one of the primary causes in age hardening (brittling) of asphaltic materials (Peterson *et al.*, 1995).

The rate of microbial attack depends on the amount of asphalt surface area exposed to oxygenated water (Zobell and Molecke, 1978). A low volume of interstitial voids results in a low-surface-area/asphalt-weight ratio of the admix seal, which slows the rate of microbial degradation. Microbial activity is concentrated along mineral aggregate surfaces, which are the primary source of nutrients for microorganisms in such an environment (Parker and Benefield, 1991). Traxler *et al.* (1965) noted that microbial utilization of hydrocarbons is selective with low molecular weight constituents being more susceptible.

A process known as stripping, results in the break down of adhesive bonds between asphalt and aggregate in asphaltic concrete. Either a strong physical force or a chemical or biochemical action is necessary to achieve such a result. Microbial activity can produce by-products that strip hydrocarbons from rock. Laboratory tests conducted by Brown *et al.* (1990) showed that certain microorganisms can significantly accelerate stripping. Microbial growth can be prevented with lime, mercury chloride or organofunctional silane to reduce stripping.

Aqueous leaching (surface dissolution) of asphalt occurs when leached water oxidized constituents in asphalt. The potential for aqueous leaching is greatest at the top and bottom interfaces of the asphalt barrier.

Freeze/thaw exposure of asphaltic concrete also results in stripping of the asphalt from the aggregate. Freeze/thaw exposure results in debonding of the asphalt from the aggregate, because of water in the interstitial voids in the mix (Hartley *et al.*, 1981). Extreme temperature cycling also causes destruction in pavements at the macroscopic level. Transverse cracking found in roadways results at low temperatures because of thermal contraction.

Baker *et al.* (1983) studied the effects of oxidation, microbial attack, aqueous leaching, and temperature cycling on asphalt in the laboratory and determined that none of these appeared to be a significant factor in the long-term integrity of the asphalt seal at the uranium mine tailings site near Grand Junction, Colorado site (Section 2.4.1). Asphalt cover degradation can be minimized with proper mix design and construction, as well as by the placement of a protective layer of cover soil above it.

2.3 Asphalt Analogs

Natural asphalt deposits are found in subterranean reservoirs, in seepages such as the asphalt lake in Trinidad (Abraham, 1960), or in tar sands, such as the Athabasca Tar Sands in Alberta. Most natural asphalt sources share common features that made them attractive for use by early cultures including: a high degree of water repellence, lack of volatility, pronounced adhesiveness, low permeability, and longevity (Peterson *et al.*, 1995).

Asphalt had been used throughout the development of civilization. It had been used as a waterproof agent, as a preservative, an adhesive and a mortar. Traces of asphalt had been found in 7000 year old tools (Marschner and Wright, 1978). There is evidence that asphaltic mortar was used in Mesopotamia (3000 B.C.), India (3000 B.C.), ancient Egypt, and in Classical Greece and the Roman Empire (Abraham, 1960).

Marschner and Wright (1978) studied asphalts from Middle East archaeological sites. The asphalt cements generally averaged 60% by weight of mineral matter, which consists of limestone, silicates and/or feldspars. Composition comparisons of the artefact asphalt with the source asphalt indicated an increase in asphaltene fraction in the artefact asphalt. Marschner and Wright (1978) concluded that the end product of exposure to the elements was similar, whether it occurred over geologic ages deep underground or over millennia near the surface. Hartley *et al.*, (1981) suggested that the millennia time scale required for conversion to asphaltene may be acceptably slow to satisfy the requirement for radon seal stability on uranium mill tailings. As noted earlier, asphaltene has very low permeability as compared to asphalt.

Peterson *et al.* (1995) reported on analogue studies at Hanford, which was used to evaluate the long-term performance of asphalt. Archaeological samples of asphalt are used to establish the long-term durability of modern, manufactured asphalt materials. Archaeological artefacts containing asphalt, manufactured by the Chumash Indians in the Santa Barbara, California region, were compared to source asphalt found in natural asphalt seeps. It was necessary to establish that the asphalt in the artefacts originated from the asphalt seeps. Radiocarbon dating of wood, shell or bone artefacts directly associated with the asphalt artefacts established that the asphalt artefacts ranged from 400 to 4160 years in age. By comparing the physical and chemical properties of artefacts with fresh, naturally occurring asphalts, it was possible to characterize the long-term aging process of asphalt in buried environments (Wing and Gee, 1994). Poly-condensation reactions cause increases in average molecular size and an increase in percentage of large molecular size (LMS) materials as asphalts age. Peterson *et al.* (1995) reported a direct correlation between the quantity of LMS and measured duration of asphalt internment. Such correlations may be used to develop an accelerated aging procedure.

The Asphalt Institute (1976) recommended high asphalt content (generally 6 to 9.5%), a well graded aggregate, and air voids not exceeding 4% to ensure low hydraulic conductivity in asphalt admixes. The barrier must also be sufficiently flexible to withstand settlement without cracking. The asphalt surface should be sealed with an asphalt sealer.

2.4 Field Tests

2.4.1 UMTRA Site: Grand Junction Colorado

Asphalt barriers were installed and field-tested at a uranium mill tailings site near Grand Junction, Colorado from 1979 to 1981 (Baker *et al.*, 1984). Asphaltic seals tested included cold mix admixtures applied with a paver, hot rubberized asphalt, in situ mixing of asphalt emulsion with aggregate (mixed with motor grader), sprayed on asphalt emulsion and asphalt chip seal. Admix seals were approximately 60 mm thick. All seals were covered with approximately 0.6 m of overburden to provide protection from ultra-violet exposure, rain, extreme temperatures and sudden temperature changes.

Field tests showed that it was difficult to apply the seal directly over the tailings and that compacted overburden should be used as a base (Baker *et al.*, 1984). The sprayed-on asphalt emulsion (fog seal) was found to have poor mechanical stability. The resulting sprayed-on membrane had a thickness of 1 cm. The membrane fractured when overburden was applied (Hartley *et al.*, 1981). The chip seal system resulted in poor mixing and a layered system. Admixtures placed with the cold mix paver resulted in the greatest mechanical stability (Baker *et al.*, 1984).

The effectiveness of the asphalt covers tested at Grand Junction, Colorado, was evaluated by measuring the radon flux throughout the asphalt cover (Freeman *et al.*, 1984). Asphalt seals were found to reduce radon fluxes by over 99% (Baker *et al.*, 1984). Baker *et al.* (1984) reported radon diffusion coefficients for asphalt seals tested in the laboratory (Table 2.1) and measured in the field (Table 2.2). The cold mix paver seal was found to form the most effective seal against radon diffusion, even though it did not have the lowest diffusion coefficient, because it could be applied the most consistently over the test plot area (Baker *et al.*, 1984).

Table 2.1. Diffusion coefficients for radon of laboratory prepared asphalt seals (Baker et al., 1984).

Type of Seal	Seal Thickness (cm)	Asphalt Content * (wt%)	Diffusion Coefficient, De (cm^2/s)
Sprayed on asphalt seals	~1	100	10^{-6} to 10^{-7}
Admixes made with tailings	8 to 12	17 to 29	10^{-1} to 10^{-4}
Admixes made with concrete sand	4 to 8	22	10^{-4} to 10^{-6}
Soil	——	0	~10^{-2}

* Wt of asphalt/wt of dry aggregate
† Note: several samples did not even form a seal

Table 2.2. Diffusion coefficients for field-test asphalt seals (Baker et al., 1984).

Application Date	Seal Type	Average Thickness (cm)	Average Asphalt Content * (wt%)	Diffusion Coefficient, De† (cm^2/s)
1981	Cold mix paver	6	20	7×10^{-5}
1980	Cold mix paver	8	22	2×10^{-4}
1980	Hot rubberized asphalt	1	100	6×10^{-6}
1980	Soil stabilizer *(in situ)*	13	22	4×10^{-4}
1979	Soil stabilizer *(in situ)*	15	11	8×10^{-4}

* Wt of asphalt/wt of dry aggregate
† Measured on samples taken in June, 1983

Although radon diffusion may not be a concern regarding acid generating mine wastes, it gives an indication of the effectiveness that asphalt seals may have in preventing oxygen transport. Asphalt seals showed good potential in reducing radon diffusion, and should therefore be effective in reducing oxygen diffusion (SRK, 1989).

The costs of complete cold-mix paver asphalt cover systems at Grand Junction were estimated to range from \$23 to \$28 (U.S.) per m^2 (Baker *et al.*, 1984). This cost included 100 mm compacted sub-base, 60 mm cold mix asphalt, and approximately 600 mm of overburden placed on top of the asphalt.

2.4.2 Asphaltic Concrete Cap: Aggie Chemical Property, San Antonio, Texas

Anthony *et al.* (1993) reported on the remediation of the 0.53 hectare former Aggie Chemical pesticide site. The site was contaminated with pesticide compounds including aldrin, lindane, chlordane, DDD, DDT, dieldrin, methoxychlor, PCP, Silvex and toxaphene. Capping of the site commenced in 1989 with a high-bitumen-content hydraulic asphalt concrete (HAC) pavement installed as an infiltration barrier.

The site was stripped of vegetation, graded to facilitate runoff, and covered with a compacted granular base. A 100 mm thick HAC was placed on top of the compacted base material. Once the HAC had cured, it was covered with an asphaltic seal coat.

Three cores were taken at different locations on the site after the asphalt sealant had been allowed to cure for ten months. The cores were tested in the laboratory for saturated hydraulic conductivity. Saturated hydraulic conductivity values ranging from 1×10^{-10} to 2×10^{-12} m/s were obtained. The cost of this site remediation, including all aspects of the design and construction, from the site investigation to closure verification, totalled approximately \$500,000 (U.S.) or approximately \$100 (U.S.) per square metre. This property is currently used as a parking lot.

2.4.3 Asphalt Cap: Western Processing Co. Inc., Superfund Site, Kent, Washington

Repa *et al.* (1987) reported on the effectiveness of an asphalt cap constructed at an old chemical reaction pond superfund site in Kent, Washington. The cap at this site covers approximately 1.6 ha.

Repa *et al.* (1987) evaluated the effectiveness of the cap based on hydraulic conductivities measured from eighteen asphalt concrete cores obtained from nine sampling locations on the cap. Hydraulic conductivities measured for the asphalt cores ranged from 3 x 10^{-5} to 5 x 10^{-3} m/s. These values of hydraulic conductivity were several orders of magnitude higher than those of typical asphaltic barrier materials (x 10^{-9} m/s). It was postulated that high hydraulic conductivity values measured were likely related to high air voids content, which ranged from 12 to 17%, as opposed to the desired less than 4% air voids. The high air voids found in this asphaltic cover may have been caused by insufficient compaction of the cap.

2.4.4 Asphalt in the Hanford Site Permanent Isolation Surface Barrier

The Hanford site permanent isolation surface barrier development program incorporated a low hydraulic conductivity asphalt barrier into a multi-layered surface cap (Wing and Gee, 1994 and Peterson *et al.*, 1995). In 1994, a 2-hectare prototype cap was constructed at Hanford consisting of a fine-soil layer to store moisture, placed above a coarse capillary break layer, underlain with asphalt concrete (Fig. 2.1). The asphalt concrete functions as a hydraulic barrier, providing redundancy to the design, intercepting any water percolating through the capillary barrier, and preventing the upward movement of noxious gases from the waste zone (Wing and Gee, 1994). Testing and monitoring of the prototype cap was planned for a minimum of three years following completion of construction (Peterson *et al.*, 1995).

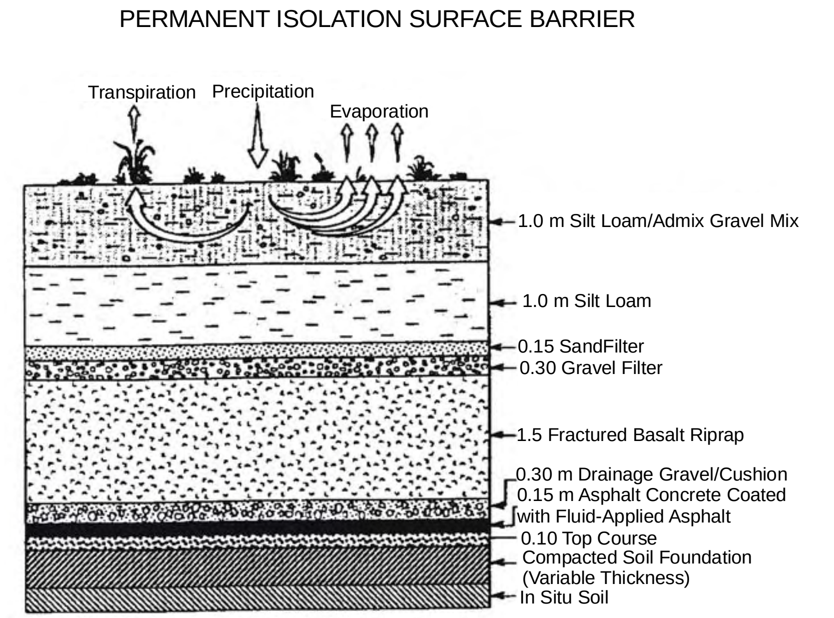

Fig. 2.1. Profile of Hanford permanent isolation surface barrier (Wing and Gee, 1994).

The Hanford prototype was designed to function for 1000 years in a semi-arid to sub-humid climate, limiting recharge of water throughout the cap into the waste to 0.5 mm per year (Peterson *et al.*, 1995). It was also intended to limit exhalation of noxious gases, be resistant to erosion problems and minimize the likelihood of plant, animal, and human intrusion.

The in situ hydraulic conductivity of a hot-mixed asphalt concrete prototype barrier layer, measured with a falling head permeameter, ranged from 10^{-9} to 10^{-11} m/s (Peterson *et al.*, 1995). Likewise, five asphalt concrete core samples taken from the prototype barrier were tested in the laboratory and had average hydraulic conductivities of 4.7×10^{-12} m/s.

The cost of the Hanford prototype was approximately $86 to $96 (U.S.) per square meter (Wing and Gee, 1994). Such a barrier is likely too expensive for large-scale use in prevention of AMD. However, it may be possible to modify some of the elements of the prototype barrier to provide a more economical solution without altering performance significantly. Although asphalt likely would not be used as a stand-alone barrier, it may be useful as a hydraulic barrier in concert with a protective cover material, such as the UMTRA design at Grand Junction reported by Baker *et al.*, (1984).

2.4.5 Asphalt Surface Cap: Federal Pioneer Ltd. Site, Regina

In 1981, remedial measures were undertaken to contain a PCB spill at the Federal Pioneer site in Regina. Remediation included construction of a 10.5 m vertical vibrating beam cutoff wall around the perimeter of the site, as well as a surface cap above the contaminated soil. The surface cap consisted of 300 mm of lime modified clay, overlain with 150 mm of hot mix asphaltic concrete (Haug *et al.*, 1988). This barrier was found to limit infiltration to approximately 0.5% of precipitation. This value was considered near optimal for an industrial site containing buildings.

The goal of the remedial measures at the Federal Pioneer site was to limit or reverse the normal downward hydraulic gradient under the site. Analysis of the performance of this barrier showed that it required periodic maintenance to seal surface cracks. These cracks were sealed with various asphaltic compounds on a regular basis.

2.5 Summary of Advantages and Disadvantages of Asphalt Covers

Asphalt covers have been found to be useful in different situations. Some of the advantages and disadvantages of using asphalt covers are presented below.

2.5.1 Advantages

- Barrier to water
- Barrier to radon flux
- Barrier to oxygen diffusion

2.5.2 Disadvantages

Asphalt covers are susceptible to the following:

- Oxidation - resulting in asphalt brittling
- Microbial attack - resulting in stripping
- Aqueous leaching - resulting in dissolution of the asphalt layer
- Temperature cycling - resulting in the stripping of asphalt from the aggregate
- Cracks formation due to subsidence of the base material supporting the cover.

Asphalt covers have not typically been used for the control of acid mine drainage. However, in select circumstances it could be the option of choice, depending upon the requirements for closure. Long-term data showing the benefits of asphalt for specific applications does exist, however, costs must be taken into consideration for all applications, including acid mine drainage.

3.0 Cementitious Covers

Four different types of cementitious covers are presented in the following section. These include polypropylene fibre reinforced shotcrete, Cefill, fly ash, and geopolymers. Not all of these different cementitious covers have been used for AMD abatement, however the properties are such that there is the potential to use them for the reduction of AMD. Field installations for the different cover types are presented where available.

3.1 Polypropylene Fibre Reinforced Shotcrete

In 1985, CANMET initiated developmental work on concrete mixtures containing high volumes of low calcium fly ash (Class F). CANMET (Malhotra *et al.*, 1990; Malhotra, 1991) and the private sector (Langley and Dibble, 1990; Wong *et al.*, 1993; Northwest Geochem, 1991; Jones and Wong, 1994; Langley, 1991) have studied the use of polypropylene fibres in high volume fly ash concrete mixtures. Polypropylene fibres provide reinforcement, which reduces plastic shrinkage cracking, and distributes cracking of the shotcrete layer to tolerable crack widths and spacing. The use of mine tailings as an aggregate source for shotcrete had been investigated (Northwest Geochem, 1991; Gerencher *et al.*, 1991) in order to decrease transportation costs of raw materials.

The addition of polypropylene fibres does not significantly change the hydraulic conductivity of shotcrete. Allan and Kukacka (1994) found that small amounts of polypropylene fibres (0.1 to 0.2% volume fraction) did not significantly change hydraulic conductivity of cementitous grouts. CANMET polypropylene fibre reinforced shotcrete technology utilizes fibre contents from 3 to 6 kg/m^3 (0.33 to 0.67% by volume).

Polypropylene is attractive as fibre reinforcement because of its resistance to moisture, acids or alkalis. It is also less expensive than steel fibre on a volume basis. Polypropylene, unlike steel, is not elastic under small strains, thus stress relaxation may occur reducing its reinforcing capabilities over time. Polypropylene fibres have the following properties (Batson, 1972):

- Tensile strength (kPa) 550 to 760 x 10^3
- Elastic modulus (kPa) 35 x 10^5
- Ultimate elongation (%) ~ 25
- Specific gravity 0.90

3.1.1 Field Trials

The use of polypropylene fibre reinforced high volume fly ash shotcrete had been field tested as a capping material on acid generating wastes at Myra Falls, Mount Washington and Halifax airport.

3.1.1.1 Westmin Resources Ltd., Myra Falls, B.C.

A small demonstration project using shotcrete to cap reactive pyrite waste rock at Myra Falls was started in 1990. Shotcreting was placed with a hand held nozzle.

A 3500 m^2 test site was capped with shotcrete in August, 1992 (MEND 2.34.1). Approximately 100 m^2 of the trial area was covered with a mix containing mine tailings. Mine tailings were used as aggregate for the shotcrete mix since local aggregate is not available. The test area was resloped (maximum grade 22%) and compacted. A 75 mm thick shotcrete cover was applied with a truck mounted robotic arm, achieving average production rates of 150 m^2/hour. Six panels, 1 m x 1 m x 150 mm, were prepared for laboratory testing. Figure 3.1 shows a photograph of this test cover taken in 1999.

Table 3.1. Cost breakdown of shotcrete application at Myra Falls (Wong *et al.*, 1993)

	Cost (Cdn $/$m^2$)
Cement	1.28
Polypropylene fibres	1.88
Fly ash (Class F)	1.40
Aggregate (includes transportation)	7.10
Labour	3.30
Equipment	3.50
Total	18.46

Table 3.1 provides a breakdown of costs associated with the cover at Myra Falls. Myra Falls is located within a national park and local aggregate is not available. As a result, transportation of the aggregate was a substantial component of the cost. Substantial savings may be realized if a local aggregate source was available, in which case the unit cost per square meter of cover could be less than $12 Cdn (Wong *et al.*, 1993).

The robotic application system appeared to produce good quality application with high rates of productivity and a uniform of placement of shotcrete. Difficulties encountered have led to suggestions for design modifications for the robotic system. The shotcrete material exhibited good compressive strength and moderate ductility as measured by laboratory tests. The cover was intact and functioning well one year after application (Jones and Wong, 1994). Some shrinkage cracks were observed immediately after application; however, the cracks did not appear to have expanded after one year of exposure. Some reduction in compressive strength was observed in the shotcrete after 400 days and this was attributed to oxidation of the sulfide minerals in the mine tailings materials used in the shotcrete. The mine tailings at Myra Falls have high sulphur content. The study did not evaluate the effectiveness of the cover in restricting acid generation in the waste rock. Site inspections carried out in 1997 and 1998 found that this cover was intact, however, cracking appeared to be progressing on the steeper sections of this cover.

Fig. 3.1. View of Myra Falls Shotcrete Cover showing sharp change in slope (Photo by M.D. Haug).

3.1.1.2 Mount Washington Mine Reclamation Project

The Mount Washington copper mine operated for one year following its opening in 1965. Two pits were created, only one of which generates acid (Galbraith, 1993). As one of many different tests to control acid generation, approximately 2300 m^3 of waste was recontoured and encapsulated with a 40 - 50 mm thick layer of polypropylene fibre reinforced shotcrete in 1991 (Seabrook, 1992). As a result of the different capping scenarios, the rate of copper leaching from the site in the spring of 1992 was reduced by an estimated 15% (Galbraith, 1993). The Mount Washington project has been put on hold at present, however, inspection carried out in 1998 and 1999 (Figure 3.2) indicate that the fibre reinforced shotcrete cap is in relatively good shape.

Fig. 3.2. View of Mount Washington Shotcrete Test Cover (Photo by M. D. Haug).

3.1.1.3 Halifax Airport

Acidic drainage associated with the oxidation of pyritic slates in several areas of Nova Scotia has contributed to the degradation of surface and groundwater supplies. Langley and Dibble (1990) have described field trials of polypropylene fibre reinforced shotcrete to cap a compacted rock fill area at the Halifax International Airport.

In 1989 six test panels (15 m x 30 m each) were placed on level ground at the south end of Runway 06. Five of the panels were shotcreted and one panel was placed with an asphalt spreader. The total cementitious material in the shotcrete varied from 375 kg/m^3 to 450 kg/m^3, and the fibre content varied from 3 to 6 kg/m^3.

Table 3.2. In situ hydraulic conductivity tests (m/s) (Langley, 1991).

Panel No	Fibre Content (kg/m^3)	Fall 1990 Surface Conditions Cracked	Fall 1990 Surface Conditions Uncracked	Spring 1991 Surface Conditions Cracked	Spring 1991 Surface Condition Uncracked
1	3	10.2 x 10^{-9}	1.6 x 10^{-9}	28.7 x 10^{-7}	19.2 x 10^{-9}
2	5	2.5 x 10^{-6}	7.0 x 10^{-9}	5.4 x 10^{-7}	1.5 x 10^{-9}
3	3	0.1 x10-6	3.8 x 10^{-9}	8.2 x 10^{-7}	51.7 x 10^{-9}
4	5	4.7 x 10^{-6}	6.3 x 10^{-9}	58.5 x 10^{-7}	1.3 x 10^{-9}
5	5	3.4 x 10^{-6}	10.0 x 10^{-9}	55.7 x 10^{-7}	5.1 x 10^{-9}
6	6	0.3 x 10^{-6}	5.8 x 10^{-9}	55.4 x 10^{-7}	3.1 x 10^{-9}

It was found that there was slightly less cracking with higher fibre content. The performance of the test panels appeared to be more dependent on fibre content than any other single factor, such as cementitious content. Monitoring of the panels indicated that 3 kg/m^3 was likely too low to control crack width to a tolerable level (Table 3.2). However, the maximum limit of fibre content was about 6 kg/m^3, which was dependent on method of placement (Langley, 1991).

Figure 3.3 shows a section of the test covers a few weeks after construction. According to Langley (1991) crack repair may not be necessary unless very low hydraulic conductivity is required. Crack repair would be labour intensive and costly. Elastomeric type sealants were found to offer the greatest potential for crack repair (Table 3.3).

Fig. 3.3 – Halifax Shotcrete Test Covers (Photo by M. D. Haug).

Table 3.3. In situ hydraulic conductivity (m/s) tests on crack repairs, summer 1991 (Langley, 1991).

	Crack Repair Material		
Panel No.	Gemite	Polymeric Sealant	Elastomeric
3	3.9×10^{-7}	—	—
5	2.2×10^{-6}	1.7×10^{-6}	4.2×10^{-8}

In general, the incorporation of supplementary cementing materials such as fly ash, slag, silica fume and natural pozzolans resulted in fine pore structure and changes to the aggregate/paste interface, leading to a change in hydraulic conductivity (Malhotra, *et al.*, 1992). Young (1992) suggested the use of cement-based materials for waste containment, particularly Densified with Small Particles (DSP) cements and carbonated cements. Concretes have low permeability and therefore concretes containing silica fume (up to 10% by weight of cement) are used as overlays on bridge decks and parking garage floors to protect reinforcing steel against corrosion. DSP cements contain about 20% silica fume by weight of cement and result in optimized densification of particle packing. DSP cements are brittle materials with relatively low tensile strengths and low tensile strain capacity (Young, 1992), therefore the addition of fibres to reinforce the matrix and prevent crack growth may be necessary.

3.2 Cefill

A cement-stabilized coal fly ash grout (Cefill) was used to create a sealing layer in a cover on a waste rock pile near Bersbo, Sweden. The waste rock dates from the 19th century, although mining in the area was active from the medieval period to about 1900 (Lundgren and Lindahl, 1991).

Waste from small heaps and shallow deposits were concentrated into two larger piles, which were contoured and covered. One of the waste piles, the Storgruve deposit with a 5 horizontal: 1 vertical slope, was covered with Cefill as a sealing layer and a protective layer of till (Lundgren and Lindahl, 1991).

The cover of the Storgruve deposit was completed in the spring of 1989. The sealing layer in the cover was constructed from a 0.25 m thick layer of crushed rock aggregate and grouted with Cefill. Cefill is a pumpable mixture of coal fly ash (91% by dry weight), cement (8% by dry weight), additives (1% by dry weight) and water (35-40%) (Lundgren, 1990). A 2 m thick protective layer of till was placed in two lifts and compacted on top of the Cefill layer. The final surface was smoothed and vegetated with pine and birch taproots. The cost of the Cefill barrier at the time of placement was approximately US$15.00 /$m^2$ (K100/m^2) (Lundgren, 1996).

The specification of the sealing layer included a saturated hydraulic conductivity of less than 1 x 10^{-9} m/s to minimize oxygen diffusion and infiltration. The Cefill layer met the requirement, except for some minor areas where the hydraulic conductivity was slightly higher. However, it is thought that the pozzolanic properties of the Cefill would cause the layer to become tighter with time (Lundgren, 1990).

Standpipes, lysimeters, permeability cells, oxygen diffusion cells and pore pressure cells were permanently installed in the cover to monitor performance. No water percolation was observed in the five lysimeters installed below the Cefill layer (Lundgren and Lindahl, 1991) during the first year. However, due to pozzolanic reactions, which likely consumed all the water in the Cefill matrix, the Cefill layer may have been unsaturated for more than one year after installation of the cover (Lundgren and Lindahl, 1991).

Oxygen concentrations in the pile decreased from $>$ 20% to $<$ 0.5% after the cover was completed (Lundgren and Lindahl, 1991).

At present, the Cefill sealing layer is functioning well as an infiltration barrier, but poorly as an oxygen diffusion barrier (Lundgren, 1990, 1997). The combination of the geometry of the Storgruve pile (steeply sloping), and the physical properties of the Cefill, resulted in unsaturated conditions in the Cefill sealing layer, particularly at the top of the pile. The unsaturated state of the Cefill layer resulted in reduced hydraulic conductivity, thus providing an excellent barrier against infiltration. However, the negative aspect resulting from the unsaturated condition was that the Cefill in the unsaturated state is a poor oxygen barrier.

3.3 Further Applications of Fly Ash

ASTM classifications divide fly ash into classes C and F. Class C fly ash is produced from subbituminous coal, is rich in calcium and self-cementing. Class F fly ash, produced from bituminous and lignite coal, is low in calcium and not self-cementing. The addition of lime or cement to class F fly ash causes pozzolanic reactions.

The abundance of fly ash in the eastern United States and its proximity to existing acid drainage sites, makes it an attractive candidate for hydraulic barriers, provided the ash can be engineered to have the required properties (Bowders *et al.*, 1994). Harshberger and Bowders (1991) found that fly ash based grouts with various waste products used as admixtures can produce hydraulic conductivities as low as 10^{-8} m/s (Table 3.4).

Table 3.4. Ranking of grout mixes (lowest to the highest hydraulic conductivity), Harshberger and Bowders (1991).

Material	Mix Ratio (by wt.)	Average Hydraulic Conductivity (m/s)
Fly ash: S.S. (No Calcilox)	70:30	$1.31. \times 10^{-8}$
Fly ash: Kaolinite Clay	90:10	3.72×10^{-7}
Fly ash: Kaolinite Clay	80:20	8.25×10^{-7}
Fly ash: Hydrated Lime	90:10	1.45×10^{-6}
Fly ash: Hydrated Lime	80:20	1.49×10^{-6}
Fly ash: S.S. (No Calcilox)	90:10	1.60×10^{-6}
Fly ash: S.S. (No Calcilox)	80:20	1.83×10^{-6}
Fly ash: S.S. (With Calcilox)	80:20	2.87×10^{-6}
Fly ash: S.S. (With Calcilox)	90:10	3.21×10^{-6}
Fort Martin Fly ash	No Mix	5.82×10^{-6}
Fly ash: Bentonite	95:5	6.27×10^{-6}
Fly ash: Fluidized bed ash	80:20	1.97×10^{-4}
S.S. (with no Calcilox)	No Mix	2.70×10^{-4}
Fly ash: Fluidized bed ash	95:5	3.59×10^{-4}
Fly ash Fluidized bed ash	90:10	5.19×10^{-4}

Note: S.S. is Scrubber Sludge; Calcilox is a stabilizer

Grouts containing waste products such as fly ash, AMD treatment sludge and fluidized bed ash, are desirable from an economic and recycling standpoint, as they utilize waste material. However, grout stability and shrinkage characteristics may be of concern and further research is needed to address such issues (Gabr *et al.*, 1996).

Scheetz *et al.* (1995) reported on field applications of cementitious grouts to address the mitigation of AMD in the bituminous coal fields of Central Pennsylvania. Fluidized bed combustors (FBC) are low emission combustion systems. At the McCloskey site in Clearfield County, Pennsylvania, cementitious grouts of FBC ash were used to cap 10 acre plots. Water was metered onto the cementitious formulation and roller compacted in lifts of 15 cm to a total thickness of 90 cm. The hydraulic conductivity of the cap was lower than 10^{-9} m/s. Expansion was found to be less than 1 percent by volume as the cementitious material hydrated. Decreases in flow through the site have been reported, with approximately 50% of the project completed.

At the Fran site in Clinton County, Pennsylvania, piles of black shale and other pyritic containing materials were hydraulically isolated with a cementitious grout cap injected into the interstices of the backfill (Scheetz *et al.*, 1995). Grouting was completed during the fall of 1992 and the summer and fall of 1993. Reductions in acidity, sulphate content and metal loadings of the effluents have been reported. Scheetz *et al.* (1995) suggested that their fly ash based grout, at its worst will continue to function until the grout is dissolved from the emplacement. Studies suggested that this time frame would be in the multiple-of-hundred of years (Zhao, 1995; Fontana, 1993).

Edil *et al.* (1987) investigated the possibility of using pozzolanic fly ash (class C) sand mixtures for low permeability barriers. It was found that hydraulic conductivities of lower than 1 x 10^{-9} m/s could be achieved from properly compacted fly ash and fly ash/sand mixtures. Moreover, exposure to wet/dry and freeze/thaw cycles was not found to substantially (less than half an order of magnitude) affect the hydraulic conductivity and mechanical properties. The fly ash/sand specimens tested were ten times more flexible than concrete. Construction conditions such as water content, compactive effort and time between adding water and compacting the fly ash, are important variables in the properties of the final product. Pozzolanic reactions began as soon as water was added in class C fly ash, and after 20 to 30 minutes the material began to set up and harden (Edil *et al.*, 1987). The hydraulic conductivity of other class C fly ashes, such as the Mae Moh fly ash from Thailand may be as low as 1 x 10^{-9} m/s and lower when well-compacted (Indraratna, *et al.*, 1991).

Stabilization of compacted class F fly ash with lime or cement results in decreased hydraulic conductivity (Bowders *et al.*, 1987). However, pozzolanic activity also decreases the flexibility of the material. Joshi *et al.* (1994) found that, for Alberta fly ash, modification of at least 15% lime was required to decrease hydraulic conductivity below 1 x 10^{-9} m/s. However, the low hydraulic conductivity achieved with the addition of lime also resulted in sample hardening, which invariably increases the compressive strength and rigidity, but decreases flexibility (Joshi *et al.*, 1994). Such a material would be susceptible to cracking in the event of differential settlement that often occurs in waste rock piles. Cracks in lime and fly ash mixes heal with time through a process called autogenous healing. During this process, pozzolanic products are formed which cement the cracked surfaces. Autogenous healing had been known to heal cracks of significant dimensions (Ahlberg and Barenberg, 1965; Callahan *et al.*, 1962).

Bowders *et al.* (1994) conducted laboratory tests to minimize the hydraulic conductivity of fly ash by adding clay and/or sand, while maintaining the fly ash as the principal constituent. A low hydraulic conductivity of 1.5×10^{-9} m/s was obtained for a compacted mixture containing 40% fly ash, 30% clay and 30% sand.

Achari and Joshi (1994) investigated the use of fly ash mixed with bentonite as a liner material. Laboratory testing indicated that the hydraulic conductivity of Alberta fly ash could be reduced to lower than 1×10^{-9} m/s by the inclusion of over 50% bentonite (Achari and Joshi 1994). Fly ash reduces the cracking potential of pure compacted bentonite by reducing the plasticity of the mixture. (Note: The addition of bentonite in such large quantities may be very expensive as compared to the addition of 15 % lime to obtain the same hydraulic conductivity).

Maher *et al.* (1993) investigated the use of lime sludge to amend class F fly ash. The lime sludge was a product of a water treatment process and was difficult to de-water. Hydraulic conductivities in the order of 10^{-9} m/s were obtained for mix ratios of dry fly ash to wet sludge of 2:1 to 3:1 by weight.

Fly ash may contain toxic metals and trace elements. It is essential to determine if toxic leachates will be released when a particular fly ash is used in covers.

3.4 Geopolymers

Geopolymers are synthetic mineral polymers, containing primarily silica, phosphate and oxygen that bond to form a ceramic type product. Geopolymers of amorphous to semi-crystalline three dimensional silico-aluminate structures are of the types poly (sialate) (-SI-O-Al-O-), poly (sialate-siloxo) (-Si-O-Al-O-Si-O-) and poly (sialate-disiloxo) (-Si-O-Al-O-Si-O-Si-O-) (Davidovits, 1994a & 1994b). Geopolymers are obtained from the polycondensation of polymeric alumino-silicates and alkali-silicates, yielding three-dimensional polymeric frameworks (Davidovits, 1994c). Properties of geopolymeric binders include high early strength, high ultimate strength, low shrinkage, freeze/thaw resistance, sulphate resistance, corrosion resistance, and low alkali-aggregate expansion (Davidovits, 1987; D. Comrie Consulting Ltd., 1988).

Ancient cement formulations used by Romans (200 BC - 100 AD) which are similar in structure to synthetic geopolymers, illustrate potential longevity of geopolymeric cements. In contrast, modern portland cement structures have suffered extensive damage in the same localities and under the same conditions (Davidovits, 1987).

In 1987, a laboratory test program was conducted for CANMET by D. Comrie Consulting Ltd. (1988) in which mine tailings were geopolymerized. The mine tailings materials tested were base metal tailings from Kam Kotia (high sulphide content), near Timmins, Ontario; coal (high sulphide content), Hinton, Alberta; potash (high NaCl content), Potash Corporation of Saskatchewan; and uranium (high content of arsenides, nuclides and sulphides) from the Midwest property, Saskatchewan (D. Comrie Consulting, 1988; Davidovits *et al.*, 1990). Hazardous elements were effectively locked into the geopolymeric matrix, thus leachate levels of radium, sodium and chlorine, and heavy metals were dramatically reduced (D. Comrie Consultants, 1988; Davidovits *et al.*, 1990). The costs of this process were high and it was not considered viable for most waste containment applications.

Geopolymeric concrete has potential for use in capping mine tailings and waste rock. Hydraulic conductivity of geopolymeric concretes may be in the order of 10^{-11} m/s (Davidovits, 1994c). D. Comrie Consultants Ltd. (1988) estimated the cost for a 75 mm thick cover, utilizing 15% by weight geopolymer with tailings as aggregate, at approximately $\$6.60/m^2$ (Canadian). However, production costs vary significantly with performance requirements and the type of geopolymer used.

It is possible that geopolymers could be used in a shotcrete application, similar to the high-volume fly ash shotcrete trial applications at the Halifax Airport and the Myra Falls site. The properties such as high early strength, low shrinkage, freeze/thaw resistance, sulphate resistance, corrosion resistance, and low hydraulic conductivity make geopolymeric concrete an attractive option for capping waste.

3.5 Advantages and Disadvantages of Cementitious Covers

Cementitious covers have been used in a variety of settings. The advantages and disadvantages of the four cementitious covers described in this section are outlined below.

3.5.1 Polypropylene Fibre Reinforced Shotcrete

Advantages

- Polypropylene fibres provide reinforcement, reduce shrinkage cracking and crack size
- Polypropylene fibres are resistant to moisture, acid, and alkalis
- Good compressive strength
- Moderate ductility
- Resistant to freeze/thaw cycles

Disadvantages

- Stress relaxation may occur as polypropylene fibre are not elastic

3.5.2 Cefill

Cefill has not been used extensively, however some success has been realized where it has been used. Some of the advantages and disadvantages of Cefill are itemized below:

Advantages

- If unsaturated, the Cefill layer provides a good barrier to water
- If saturated, the Cefill layer inhibits the diffusion of oxygen
- Pozzolanic properties of the material may cause hydraulic conductivity to decrease

Disadvantages

- If unsaturated, the Cefill layer will poorly inhibit the diffusion of oxygen
- If saturated, the Cefill layer does not provide a good barrier to water

3.5.3 Fly Ash

Types C and F fly ashes have been incorporated into cementitious mediums and used for injection within mine waste and also the capping of mine waste. The advantages of using fly ash are given below:

Advantages

- Low hydraulic conductivity
- Little expansion with hydration of the layer
- Resistant to the impact of freeze/thaw cycles
- Resistant to wet/dry cycles
- Reduces acidity, sulphate content and metal content of water flowing out from the site
- Fly ash is a waste product from coal, which could be used in a positive manner to reduce acid mine drainage (AMD)
- Cracks ‘self-heal’ with time when lime is added to type F fly ash
- Fly ash is an alkaline waste product with a high pH and will neutralize acid

Disadvantages

- Injected grout would dissolve with time
- Long-term stability and shrinkage of grout used as a capping layer is unknown
- Flexibility of the grout decreases with the addition of lime (lime is added to stabilize type F fly ash)

3.5.4 Geopolymers

Geopolymers are synthetic mineral polymers that contain silica, phosphate and oxygen and form a ceramic type product. There may be potential for this material to be used for capping mine wastes. The advantages and disadvantages of this material are given:

Advantages

- High early strength
- High ultimate strength
- Low shrinkage
- Resistant to freeze/thaw
- Resistant to sulphate
- Resistant to corrosion
- Low alkali-aggregate expansion
- Low hydraulic conductivity

Disadvantages

- Costs could be prohibitive depending on the performance requirements

The advantages and disadvantages of the different types of cementitious covers have been outlined. The properties of the materials do show a potential for being useful as covers in the area of AMD abatement.

4.0 Synthetic Liners and Covers

Numerous synthetic liners and covers have emerged over the last few decades. The general classifications include geotextiles, geonets, geogrids, and geocomposites. Geotextiles perform one of five functions: separation of layers, reinforcement, filtration, drainage, and moisture barrier (in coated form). Geonets are utilized for drainage or reinforcement. Geogrids are used primarily for reinforcement or support. Geocomposites are composed of a combination of geotextiles, geogrids, geonets and goemembranes. These composites provide a higher system performance than can be attained by an individual liner installed alone. Of these synthetic liners, geomembranes, spray-on membrane barriers, and geosynthetic clay liners are presented in this report.

4.1 Geomembranes

Flexible membrane liners (FMLs) are commonly referred to as geomembranes or geosynthetics. Some types of geomembranes in current use are shown in Table 4.1. FML cover systems should be constructed with a degree of redundancy. Melchior et al. (1993) reported that a FML used in concert with a compacted soil liner had performed well in a German landfill cover. Three categories of polymers can be used to make geomembranes, namely: thermoset elastomers, thermoplastics and bituminous types. Thermoset elastomers are rarely used in waste containment, primarily because of difficulty in making field seams. Examples of thermoset materials are butyl and EPDM. Bituminous geomembranes are rarely used in North America, but are still used in Europe, particularly in France. The majority of geomembranes used in North America today are thermoplastics (Koerner, 1993). Thermoplastic materials by definition are materials that become soft and pliable when heated, without any change in inherent properties

Table 4.1. Major types of geomembranes in current use (after Koerner, 1990).

Thermoplastic Polymers

- Polyvinyl chloride (PVC)
- Polypropylene (PP)
- Polyethylene (VLDPE, LDPE, LLDPE, MDPE, HDPE, referring to very low, linear low, medium, and high density)
- Chlorinated polyethylene (CPE)
- Elasticized polyolefin (3110)
- Ethylene interpolymer alloy (EIA), also know as XR-5
- Polyamide (PA)

Thermoset Polymers

- Isoprene-isobutylene (IIR), or butyl
- Epichlorohydrin rubber
- Ethylene propylene diene monomer (EPDM)
- Polychloroprene (neoprene)
- Ethylene propylene terpolymer (EPT)
- Ethylene vinyl acetate (EVA)

Combinations

- PVC-nitrile rubber
- PE-EDPM
- PVC-ethyl vinyl acetate
- Cross-linked CPE
- Chlorosulfonated polyethylene (CSPE), also known as Hypalon®

4.1.1 Historical Perspective of Polymeric Geomembranes

Development and use of geomembranes is intrinsically tied to the development and growth of the polymer industry (Fig. 4.1). The earliest type of thin prefabricated polymer sheets placed on a prepared soil base as currently practised, was the use of PVC as swimming pool liners in the early 1930's (Staff, 1984). Testing and design of PVC for canal liners by the U. S. Bureau of Reclamation in the 1950s (Koerner, 1993) also led to use of PVC as canal liners in Canada, Russia, Taiwan and in Europe throughout the 1960’s and 1970’s (Fig. 4.1). Polyethylene liners (semicrystalline liners) were developed in West Germany in the 1960’s (Fig. 4.1) and spread throughout the world.

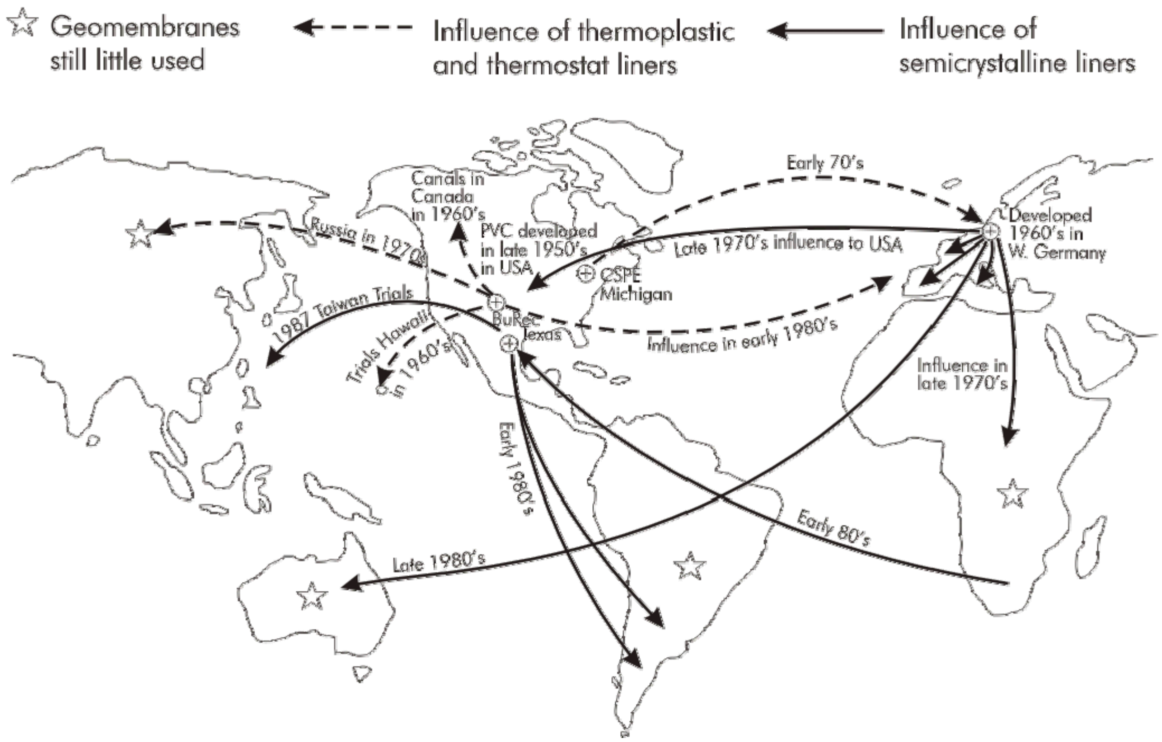

Fig. 4.1. The initial development and spread of geomembranes on a worldwide basis (Koerner, 1990).

An indication of relative usage of different types of geomembranes in North America is given in Fig. 4.2. Polyethylene has been established as the most frequently used polymer for geomembranes, largely because it has good resistance to chemical exposure, low cost and ease of seamability.

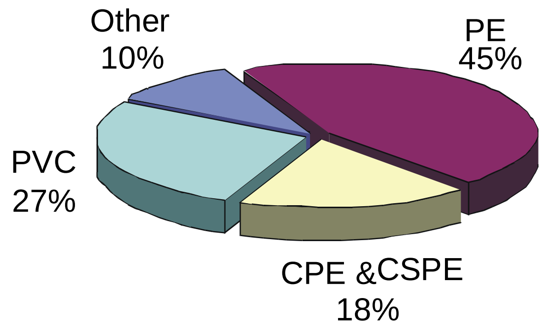

Fig. 4.2. Geomembrane use in North America (Sadlier, 1993).

4.1.2 Composition

Various additives, fillers, and plasticizers are used with the base resin which gives the geomembrane its name (Table 4.2). Three classifications of polymers used for current geosynthetic materials are: thermoset, amorphous thermoplastic, and semicrystalline thermoplastic. Most of the polymers used in geosynthetics are semicrystalline thermoplastics. Crystallinity may vary from as little as 30% in some PVCs to as high as 75% in HDPE (Koerner, 1990).

Table 4.2. Typical range of formulations for thermoplastic geomembranes (Koerner, 1993).

Type of Geomembrane*	Resin (%)	Plasticizer (%)	Carbon Black and Filler (%)	Additives†
HDPE	96-97	0	2-2.5	0.5-1
VLDPE	96-97	0	2-2.5	0.5-1
CPE or CPE-R	60-75	10-15	20-30	3-5
CSPE-R	75-50	2-5	45-50	2-4
EIA-R	50-65	10-20	20-30	3-5
PVC	45-50	35-40	10-15	3-5

* The 'R' refers to fabric reinforcement
† Includes antioxidant, processing aids and lubricants

Plasticizers (oils or waxes) are solvents, which are absorbed into amorphous polymers and amorphous regions of crystalline polymers. Plasticizers are added to some resins to lower the glass transition temperature, thereby making the material softer and more pliable (Moore and Kline, 1984). An important consideration in the use of plasticizers is whether the plasticizer stays mixed with the polymer resin. If the plasticizer stays mixed with the polymer resin, the eventual loss of plasticizer would mean loss of pliability and cracking resistance.

Increasing crystallinity impacts the behaviour of geomembranes in the following ways: increased stiffness or hardness, heat resistance, tensile strength, modulus, and chemical resistance, as well as decreasing permeability, elongation, flexibility, impact strength, and stress crack resistance (Koerner, 1990).

Polyethylene (PE), (Fig. 4.3a) has the least reactive chemical structure of all commercial thermoplastics. The appeal of PEs is its lack of reactivity, low cost and acceptable physical characteristics (Cassidy *et al.*, 1992). Polypropylene (PP) is similar to PE (both PP and PE are polyolefins) in structure (Fig. 4.3b) and has good chemical resistance and better flexural strength. Polyvinyl chloride (PVC) (Fig. 4.3c) requires the addition of plasticizers for use in geomembranes (Table 4.2). PVC is known to absorb organic liquids, which leads to softening (Cassidy *et al.*, 1992). Polyester (PET), in its most common form, is shown in Fig. 4.3d. Chlorosulfonated polyethylene (CSPE) (Fig. 4.3e) is a PE derivative, brought about by reacting PE with a mixture of chlorine and sulphur dioxide under UV irradiation (Cassidy *et al.*, 1992). CSPE is known to have outstanding weather resistance (Schoenbeck, 1988). Chlorinated polyethylene (CPE) (Fig. 4.3f) is formed by PE reacting with chlorine. The structural regularity of CPE (as in CSPE) is reduced by the addition of chlorine, which results in decreased crystallinity. Both CPE and CSPE exhibit good resistance to weathering and chemical attack. Ethylene interpolymer alloy (EIA) (Fig. 4.3g) is a blend of ethylene vinyl acetate copolymer and PVC, resulting in a thermoplastic polymer (Matrecon Inc., 1988).

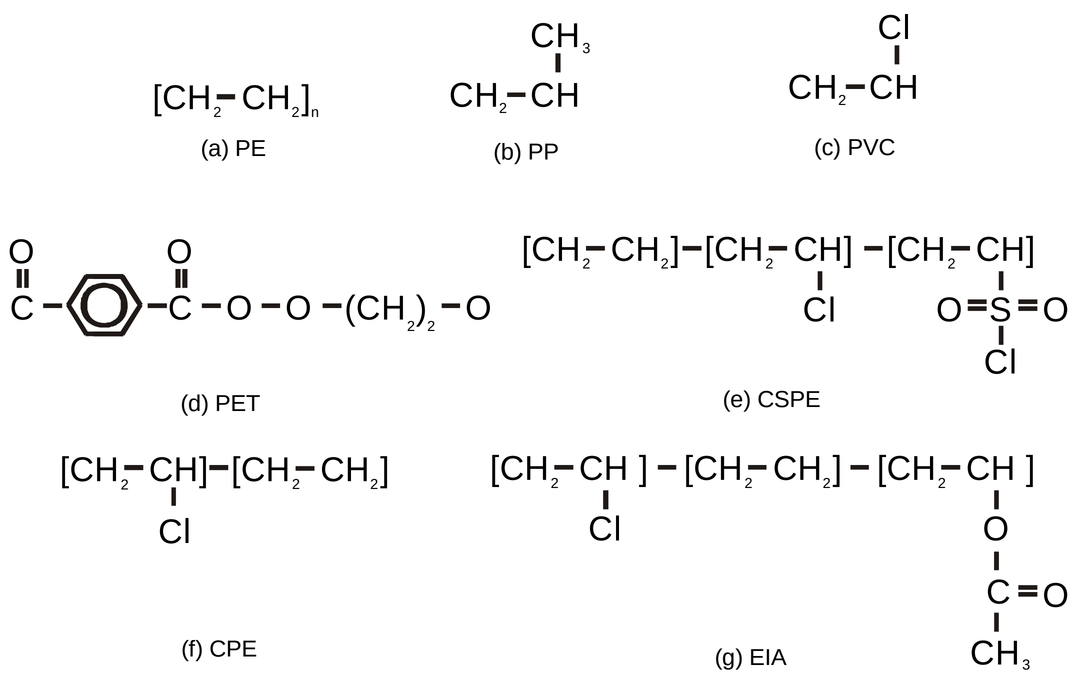

Fig. 4.3. Polymers commonly used in geosynthetic applications: a) polyethylene (PE), b) polypropylene (PP), c) polyvinyl chloride (PVC), d) polyethylene terephthalate (PET), e) chlorosulfonated polyethylene, f) chlorinated polyethylene (CPE), and g) ethylene interpolymer alloy (EIA).

Polymeric materials eventually degrade by a number of different actions, which include ultraviolet light, high-energy radiation, oxidation, hydrolysis and chemical reaction (Koerner *et al.*, 1992). However, in the absence of ultraviolet light, radioactive materials and aggressive chemicals (such as organic solvents), the main mechanisms of degradation in geosynthetic materials are oxidation and hydrolysis (Koerner *et al.*, 1992). The time for measurable oxidation of polyolefins at ambient temperatures is very long. For instance, HDPE is predicted to have a lifetime of 200 to 750 years (Koerner *et al.*, 1990). The major concern of geosynthetics made from polyester is hydrolytic reaction.

For example, polyester (PET) could combine with water and could revert back to acid and glycol (Odian, 1982).

Resistance to degradation of all geosynthetic polymers can be improved by increasing the molecular weight, as well as the degree of crystallinity (Koerner *et al.*, 1992). However, stress cracking becomes a concern in polyolefins of high crystallinity. Anti-oxidants are added to polyolefins to inhibit oxidation. Carbon black is also added to absorb ultraviolet radiation (Moore and Kline, 1984).

4.1.3 Properties

4.1.3.1 Mechanical

Time dependent behaviour of polymeric materials under load is an important consideration in many geosynthetic design situations. Geosynthetic creep (strain with time) and stress relaxation (stress reduction with time) are two issues of mechanical behaviour which must be addressed. Test set-ups and generalized response curves for creep and stress relaxation are given in Fig. 4.4.

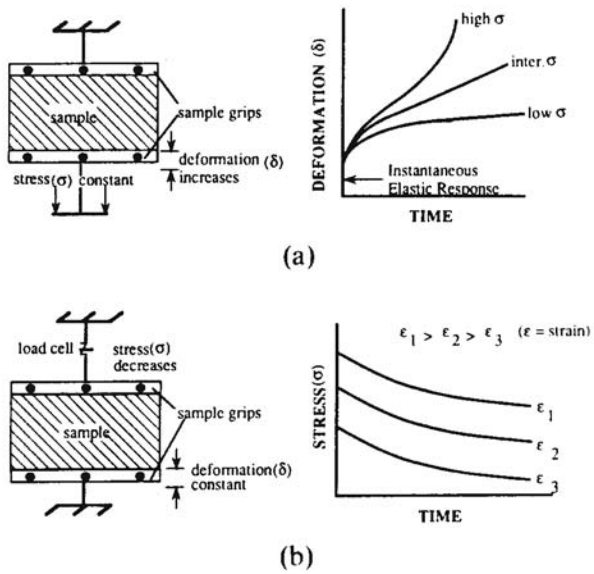

Fig. 4.4. Schematic diagrams and typical response curves of creep and stress relaxation of geosynthetic materials: a) Creep test set up and generalized curves; b) stress relaxation test set up and generalized response curves (Koerner et al., 1992).

Findley (1987) evaluated 26-year duration experimental creep data on thick sections of PVC and polyethylene and found that the creep strain response behaved according to the following equation:

$\varepsilon(t) = E_0 + e^a t^n$

where:

$\varepsilon(t)$ = creep strain as a function of time

t = time

E_0, a and n = constants

Crissman (1991) determined the creep response of polyethylene at various stress levels and temperatures, suggesting that the best criterion to predict lifetime is the time when necking (cross-sectional area at a location reduces and material elongates rapidly) occurs.

Stress relaxation in geomembranes follows the following mathematical relationship (Koerner et al, 1992):

$\sigma(t) \quad = ct^{-b}$

where:

$\sigma(t)$ = stress level at time t

c,b = constants

t = time

Unsupported creep gradually gives way to stress relaxation. Geomembranes would initially go into tension and creep due to subsidence. However, after subsidence had ceased, stress relaxation would be the dominant mechanism (Koerner *et al.*, 1992).

A polymer may exhibit failure in a ductile or brittle manner, depending on stress level and time duration (Bright, 1993). Sufficiently high stresses can cause crazes or cracks, which may propagate when exposed to various agents, leading to the deterioration of mechanical properties of the polymer. This propagation of cracks and loss of mechanical properties is referred to as environmental stress cracking (ESC). ESC is dependent upon time duration, stress level, environmental agents (chemicals, ultra-violet radiation, etc.) and temperature (Fig. 4.5). ASTM D1693 provides a test method for testing ESC in polyethylene materials. The higher the density (hence crystallinity), the more relevant the ASTM D1693 test (Koerner, 1990).

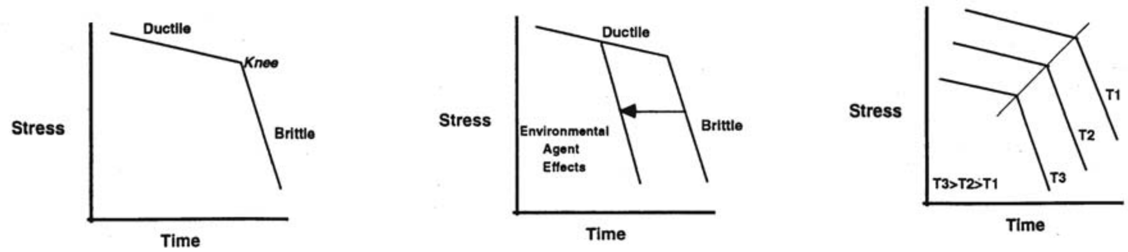

Fig. 4.5. Environmental stress cracking (ESC) a) behaviour in air, b) effect of environmental agents, and c) effect of temperature (Bright, 1993).

Peggs and Carlson (1989 and1990) noted that the majority of stress cracking failure in PE geomembranes is related to improper design and installation procedures. Cracking was generally found to occur along seams and points of high local stress (e.g. stone impinging the underside of a geomembrane). Cracking is related to the nature and distribution of flaws in the liner system, as well as the stress concentrations induced by the applied load on these flaws (Lustiger and Rosenberg, 1989). Stress concentrations are set up in the boundary between the melted and unmelted material in welds, rather than in the interface itself (Lustiger and Rosenberg, 1989).

Berg and Bonaparte (1993) proposed a rational approach for evaluating long-term allowable stresses, which takes into account time, temperature, ductile and brittle fracture, chemical environment, welded seams, installation damage, and an over-all factor of safety. An allowable long-term tensile stress may be computed by using:

$$\sigma_a = \frac{\sigma_r \times FC \times FW \times FI}{FS} \qquad \text{(Berg and Bonaparte, 1993)}$$

where: σ_a = allowable tensile stress at the given in-service temperature and design life

σ_r = rupture stress at the given in-service temperature and design life

FC = reduction factor to account for chemical (or radiation) degradation over the design life

FW = reduction factor (i.e. weld factor) to account for long-term seam strength

FS = overall factor of safety

FI = reduction factor for installation damage (unitless)

Rollin *et al.* (1991) tested samples of a seven-year-old HDPE geomembrane used in a landfill containing contaminated soil in Ville LaSalle, near Montreal. The contaminated soil contained cyanides, phenolic compounds, and heavy hydrocarbon fractions. The containment system consisted of a 2 mm thick HDPE geomembrane liner sandwiched between compacted clay layers and a 1.6 mm thick HDPE geomembrane cap, which is exposed to the atmosphere. The analysis of the tested samples indicated that aging had occurred. It was more severe for samples at the base of the liner (a more severe chemical environment) than along the sides and the surface cap. Aging was detected as an increase in yield strength, a decrease in the tensile resistance at rupture and a reduction of the elongation at break. Microanalysis indicated stress cracking was present along a seam.

4.1.3.2 Chemical Resistance

ASTM D543 outlines testing for chemical degradation under the title “Resistance of plastics to chemical reagents”. A list of 50 standard reagents is given, which attempts to provide standardization, which may be tested for various exposure times and exposure at elevated temperatures.

Chemical resistance of some commonly used geomembranes to various chemicals at 38° C are given in Table 4.3. Geomembranes generally have good chemical resistance to acids, bases, heavy metals, and salts.

Table 4.3. General chemical resistance (at 38ºC) guidelines of commonly used geomembranes (after Vandervoort).

Geomembrane Type

Chemical	Butyl Rubber	Elasticized Polyolefin	Neoprene	EPDM	PE	PVC	CSPE	CPE
Acids:								
Organic	x	x	x	x	x	x	x	x
Inorganic	x	x	x	x	x	x	x	x
Bases:								
Organic	x	x	x	x	x	x	x	x
Inorganic	x	x	x	x	x	x	x	x
Heavy Metals	x	x	x	x	x	x	x	x
Salts	x	x	x	x	x	x	x	x
General:								
Aliphatic Hydrocarbons		x	x		x			x
Aromatic Hydrocarbons		x	x		x			x
Chlorinated Solvents	x	x	x	x	x			
Oxygenated Solvents	x	x	x	x	X			
Crude Petroleum Solvents		x	x		x			x
Alcohols	x	x	x	x	x	x		x

x = generally good resistance

Polymeric geomembranes have been used successfully in hostile chemical environments. At the Gilt Edge Mine, a VLDPE geomembrane was installed in 1989 over an asphaltic concrete liner system (Smith, 1993a). The VLDPE geomembrane was required to be resistant to solvents contained in the asphalt products, ultra violet light exposure, sodium cyanide solution at pH 11, heavy loading, and be suitable for installation and operation in cold weather. Performance of the VLDPE geomembrane at Gilt Edge has been favourable, and the measured leakage rates are low (Smith, 1993a). Mills (1993) reported on testing of geomembranes for containment of sulphur, which was to be placed molten on liners at 125° C. Based on testing, a PVC alloy material was selected and the material had worked well.

HDPE geomembrane liners used for containment of pulp mill hot black liquors (pH 12, T=80° C) have been found to be susceptible to environmental stress cracking (Peggs *et al.*, 1993). However, geomembranes based on polypropylene appear to offer a viable alternative for the safe, durable containment of black liquor (Peggs *et al.*, 1993).

4.1.3.3 Biological Resistance

Geomembrane resistance to fungi can be evaluated according to ASTM G21 and resistance to bacteria according to ASTM method G22. There is little concern for biological degradation of the resins in geomembranes because of their high molecular weight (Koerner, 1990 and 1993).

Ionescu *et al.* (1982) evaluated six fabrics (four of polypropylene, one of polyester, and a composite) in eight media (distilled water, iron bacteria culture, desulfovibrios and levansynthesizing bacteria, liquid mineral medium, sea water, compost and soil) for 5 to 17 months. None of the geotextile types tested showed any signs of biodegradation during the test.

4.1.4 Issues of Concern

4.1.4.1 Lifetime Prediction Tests

Several different methods of lifetime prediction may be applied to polymeric geomembranes (Table 4.4). The Rate process method is based on the ductile-to-brittle transition of polyethylene under constant stress conditions (Halse *et al.*, 1990). Koerner *et al.* (1990) described the use of the Rate process method for HDPE geomembrane lifetime prediction.

The Hoechst multiparameter approach, initially developed for testing HDPE pipe, has been applied to predict long-term behaviour of HDPE geomembranes (Kork *et al.*, 1987). The procedure is worth considering, however, the amount of data required is large and difficult to obtain because the required field strain values are essentially non-existent (Koerner *et al.*, 1993).

Table 4.4. Lifetime prediction methods used for polymeric materials (Koerner et al., 1993).

Method	Principle	Advantages	Disadvantages
Rate Process Method	Capped pipe sections (either as-received or notched) tested at various pressures and at elevated temperatures until failure occurs.	•Can be adapted to geosynthetics •Results in ductile-brittle transition curves at each temperature •Desired lifetime is prescribed and allowable stress (with FS) is obtained	•Relates to polyofins only •Requires curve fitting (extrapolation) to obtain ambient temperature data •Accelerating wetting liquid is sometimes used
Hoechst Multiparameter Approach	Creep and stress relaxation data are superimposed with field strain values to estimate lifetime.	•Applicable to geosynthetics •Challenges polymer in numerous stress states •Includes field data •Can possibly incorporate seams	•Applied to HDPE only •Requires considerable laboratory data •Monitoring field seams is extremely difficult
Arrhenius Modelling	High temperature incubation under site specific conditions, then use of curve fitting to obtain equivalent lifetime.	•Applicable to geosynthetics •Allows for complete simulation of field conditions •All polymers can be evaluated •Can possibly incorporate seams	•Difficult test setup •Assumes linear activation energy •Long test time required •Data must be extrapolated

Arrhenius modelling is the most widely accepted method for lifetime prediction of polymers (Koerner *et al.* 1990). The key hypothesis of Arrhenius modelling is that elevated temperature incubation is directly related to time degradation of a particular property of the material and that the time-temperature superposition principle is applicable (Koerner *et al.*, 1992). Temperatures elevated above their glass transition temperature cause thermoplastic polymers to soften. Thus, high temperature modelling may not be representative of normal end use if test temperatures exceed the glass transition temperature (Daniel, 1990).

Polymers such as polyethylene have been successful in outdoor applications in cable sheathing and buried pipe applications for more than 40 years (Tisinger and Giroud, 1993). The rationale behind any barrier system is to ensure adequate protection for as long as there exists a potential threat to the surrounding environment (Rowe and Brachman, 1994). However, the length of time a geosynthetic will perform at its design level cannot be known for sure. Fundamental assumptions made for accelerated testing may be difficult to defend for a real life waste containment facility.

4.1.4.2 Installation Survivability

A geomembrane may be rendered ineffective from the start of its service life if it is poorly installed. Koerner and Koerner (1990) studied installation survivability of 75 geotextiles and geogrids from 48 different construction sites in the northeastern USA. Some important findings from the study were:

- Coarse, irregular, and frozen subgrades were very damaging
- Heavy construction equipment created major damage to thin soil cover lift thickness
- Large cover soil particle sizes and poorly graded mixes were troublesome
- Low mass per unit area geotextiles suffered the greatest strength reductions and

number of holes

The use of a co-extruded HDPE/VLDPE multi-layered geomembrane as described by Kolbasuk (1991) may be useful. Such a geomembrane combines the increased puncture resistance and flexibility of VLDPE with the inertness and strength characteristics of HDPE.

4.1.4.3 Permeability and Leakage

Polymeric geomembranes are generally regarded as being impermeable. Hydraulic conductivities of thermoplastic and thermoset geomembranes are typically in the order of 10^{-13} to 10^{-15} m/s (Koerner, 1990). Seepage through geomembranes is primarily due to defects which may have resulted from tears, punctures or improper welding of seams. An experimental and theoretical study of leakage through flaws in geomembranes was conducted by Jayawickrama *et al.* (1988). The results of the study show that the type and thickness of the geomembrane have a relatively small influence on leakage rates. Also, a low hydraulic conductivity sub-base is important for restricting flow through a flawed liner.

Bonaparte *et al.* (1989) described methods for evaluating rates of leakage through landfill liners constructed with geomembranes and proposed an equation for evaluating flow through defects in geomembranes:

$$Q = 3\, a^{0.75}\, h^{0.75}\, k_d^{0.5}$$

where: Q = flow through defects (m^3/s)

a = hole (defect) area (m^2)

h = water head on geomembrane (m)

k_d = hydraulic conductivity of the drainage material overlying the geomembrane m/s)

Bonaparte *et al.* (1989) evaluated composite liners (soil + geomembrane), as well as geomembrane liners alone, and found that leakage rates through holes are significantly reduced by placing low hydraulic conductivity material beneath the geomembrane.

4.1.4.4 Gas Diffusion

An important characteristic of geomembranes is their intrinsic low permeability to a broad range of gases, vapours and liquids, both as single-component fluids and as complex mixtures of many constituents (Haxo, 1990). Chemical species and gases pass through geomembranes according to Fick's Law. Haxo (1990) presented various methods to assess the permeability of geomembranes to individual gases, vapours, and liquids as well as for measurement of permeation through geomembranes of species in complex mixtures.

4.1.4.5 Interface shear strength

Low interface shear strength in geotextiles may present stability problems in cover and liner systems. Koerner *et al.* (1986) found the efficiency of the frictional component (E_f) to range from 50% to 100% for geomembrane-soil interfaces of various polymers tested in direct shear with several different fine-grained soils. The efficiency of the frictional component is defined as:

$$E_f = \delta/\phi$$

where: δ = the friction angle of geomembrane to soil

ϕ = the friction angle of soil to itself

In general, Koerner *et al.* (1986) found E_f is higher for soft geomembranes, such as CPE and EPDM, than for harder geomembranes such as HDPE and PVC. Fishman and Pal (1994) studied geomembrane/cohesive soil interface behaviour with consolidated drained and consolidated undrained direct shear tests and found the shear strength of clay-smooth HDPE interfaces to be less that that of the clay alone for all cases studied. Perhaps one of the most extensively studied and reported stability failures involving a geomembrane liner, is the Kettleman Hills landfill failure in California on March 19, 1988 (Seed *et al.*, 1988; Mitchell *et al.*, 1990a, 1990b, 1993; Seed *et al.*, 1990; and Byrne *et al.*, 1992.).

Slope failures have also occurred at mining and solid waste facilities along geosynthetic interfaces. Poulter (1994) discussed two case histories of slope failures which occurred along geosynthetic liner interfaces at two heap leach facilities. At one of the heap leach facilities, slope failure occurred along an HDPE compacted clay interface, whereas at the other, failure took place along a geosynthetic liner interface.

Koutsourais *et al.* (1991) found that surface roughness, flexibility and elasticity of the geosynthetics have a large impact on the interface friction angle. Interface friction between layers of geosynthetics and/or soils varies with normal stress, resulting in a curved Mohr-Coulomb failure envelope. Paruvakat *et al.* (1992) showed that either flexibility or roughness in a geomembrane could be used to ensure stability of composite caps along side slopes. Paruvakat *et al.* (1992) reported on the use of a VLDPE geomembrane and a textured HDPE geomembrane used in tests plots constructed along 4H:1V slopes of the Anoka Regional Sanitary Landfill, Minnesota. Both systems were found to have excellent stability against slope failure. Michelangeli *et al.* (1991) reported on the use of a rough, highly flexible PE geomembrane with sand interfaces to cover slopes up to 35º at the Nuova Samim, Porto Vesme landfill in Italy.

Stark *et al.* (1996) used torsional ring shear tests to determine interface shear resistance between HDPE geomembranes/geotextiles and a drainage geocomposite. Textured HDPE were found to provide a substantial increase (200 to 300%) in interface shear strength over smooth HDPE geomembranes. However, shearing under high normal stress was found to damage or remove texturing on the geomembrane. The difference between peak and residual friction angles was more pronounced for textured than for smooth geomembranes.

Negussey *et al.* (1988) reported an increase in the differences between peak and residual interface angles with increasing angularity of sand at the sliding surface. Peak interface resistances appeared to have developed more from resistance to scouring than as a result of dilatancy.

Stamatopoulos and Kotzias (1996) analysed a failure of the inside slope of an empty water reservoir in Greece after a prolonged period of heavy rainfall in 1994. Seepage forces resulted in destabilizing the side-slope of the reservoir. Failure occurred along an interface between silty sand and a smooth HDPE geomembrane. Stamatopoulos and Kotzias (1996) found that the sliding of water-saturated materials over the geomembrane was caused by the superposition of seepage on gravity forces.

Giroud *et al.* (1990) used laboratory and full-scale field tests to successfully design a geosynthetic landfill cap on a 3H:1V slope. Stability of the cap was achieved with a combination of two measures: using a rough geomembrane in contact with a needle-punched geotextile heat-bonded to a geonet.

Vallejo and Zhou (1995) proposed the use of fractal theory to measure roughness of textured geomembranes, rather than simply classifying geomembranes as smooth or rough. The fractal approach allows for quantification of surface roughness with the fractal dimension (D).

Evaluation of tensile forces that develop in geosynthetics used in slopes is an important consideration. Load-deformation compatibility at interfaces and within components is used to predict stresses for analysing stability of geosynthetic composite liner and cover systems (Gilbert *et al.*, 1993). Long *et al.* (1994) used two methods which considered both force equilibrium and displacement compatibility to analyse stability of soil/geosynthetic cover systems (Fig. 4.6). It was found that axial loads within the components of a cover system are very small at slope angles less than the minimum interface friction angle. However, at greater slope angles, development of axial loads in geosynthetics may be significant, even if the slope was predicted to be stable.

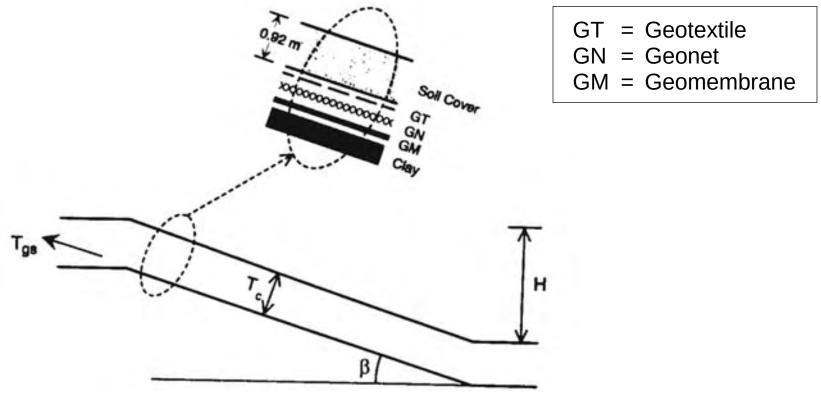

Fig. 4.6. Configuration used to evaluate stability of a soil/geosynthetic cover system (Long et al.,1994).

It is recommended that an interface shear testing program be implemented for designs of geosynthetics used for slopes. Geomembrane-soil interface shear strength is dependent on the properties of both the soil and the geomembrane. Swan *et al.* (1991) found that various aspects of compaction such as compactive effort and water content had an impact on interface shear strength.

4.1.4.6 FML Selection

Some advantages and disadvantages are listed for a variety of geomembranes, on the basis of basic polymers in Table 4.5. In the U.S.A., installed costs of geomembranes average approximately $7.50 per square metre for geomembranes of 1 mm thickness (Daniel and Koerner, 1993). Szymanski and MacPhie (1994) estimated the cost of a geomembrane cover installation (including bedding, protective soil cover, anchoring, etc.) for preventing AMD, at approximately $20 (Cdn) per square metre.

Table 4.5. Advantages and disadvantages for a variety of geomembranes. Modified by Koerner (1990) from Woodley (1978).

Advantages	Disadvantages
Polyvinyl Chloride (PVC) Thermoplastic	
Low Cost	Plasticized for flexibility
Tough without reinforcement	Poor weathering, backfill required
Lightweight as single ply	Plasticizer leaches over time
Good seams - dielectric, solvent and heat	Poor cold crack
Large variation in thickness	Poor high-temperature preformance
	Blocking* possible
Chlorinated Polyethylene (CPE) Thermoplastic	
Good weathering	Moderate cost
Easy seams - dielectric and solvent	Plasticized with PVC
Cold crack resistance is good	Seam Reliability
Chemical resistance is good	Delamination possible
Chlorosulfonated Polyethylene (CSPE) Thermoplastic Rubber	
Excellent weathering	Moderate cost
Cold crack resistance is good	Fair in high temperatures
Chemical resistance is good	Blocking possible
Good seams - heat and adhesive	
Elasticized Polyolefin (3110), Thermoplastic EPDM - Cured Rubbers	
Good weathering	Unsupported only
Lightweight as single ply	Poor high-temperature performance
Cold crack resistance is good	Special seaming equipment required
Chemical resistance is good	Field repairs are difficult
EPDM (4060) Thermoplastic Rubber	
Good weathering	Moderate cost
Cold crack resistance below 60° F	Fair in high temperatures
Good seams - heat-bonded	Blocking possible
No adhesives required	Fair chemical resistance
Butyl, Butyl/EPDM, EPDM - Cured Rubbers	
Fair to good weathering	Moderate to high cost
Low permeability to gases	Poor field seams
High temperature resistance is good	Small panels
Nonblocking	Fair chemical resistance
Chloroprene (Neoprene) Cured Rubber	
Good weathering	High cost
Good high temperature	Fair field seams - solvent and tape
Good chemical resistance	Fair seams to foreign surfaces
High-Density Polyethylene (HDPE) Semicrystalline Thermoplastic	
Chemical resistance is excellent	Low-friction surfaces
Good seams - thermal and extrusion	Stress crack sensitive
Large variation in thickness	Seam workmanship critical
Low cost	High thermal expansion/contraction
Medium-, Low-, Very-Low-Density Polyethylene (MDPE, LDPE, VLDPE) Semicrystalline Thermoplastic	
Chemical resistance is good	Moderate thermal expansion/contraction
Good seams - thermal and extrusion	LDPE and VLDPE rarely used
Large variation in thickness	MDPE often mistaken for HDPE
Low cost	
No stress crack	
Linear-Low-Density Polyethylene (LLDPE) Semicrystalline Thermoplastic	
Chemical resistance is very good	Moderate cost
Good seams - thermal and extrusion	LLDPE newly introduced
Large variation in thickness	
High-friction surface	

4.1.5 Case Studies

4.1.5.1 Kjøli Mine, Norway

SRK (1991) and MEND 2.33.1 reported on the placement of a geosynthetic cover over an acid generating mine rock pile in order to control infiltration at the Kjøli mine in Norway. In the summer of 1989, mine rock littered throughout the Kjøli mine site was relocated and consolidated into one main rock pile. The rock pile was recontoured to provide a maximum gradient of 3.5H:1V and covered. A total surface area of 27 000 m^2 was covered.

The composition of the cover (Fig. 4.7) consisted of a geotextile geofabric placed directly on the mine rock, followed by a 2 mm HDPE geomembrane. A geonet layer was placed on top of the geomembrane to provide a drainage layer, and covered with a geotextile to serve as a filter. This drainage layer was considered necessary to ensure the stability of the steeper parts of the slopes. Two lifts of till (0.5 m each) were placed on top of the geotextile to provide a physical protective cover and vegetative layer. Riprap was placed on areas with steep slopes to minimize erosion. A seal around the perimeter of the rock pile was constructed by extending the geomembrane to the base of the ditch constructed around the pile.

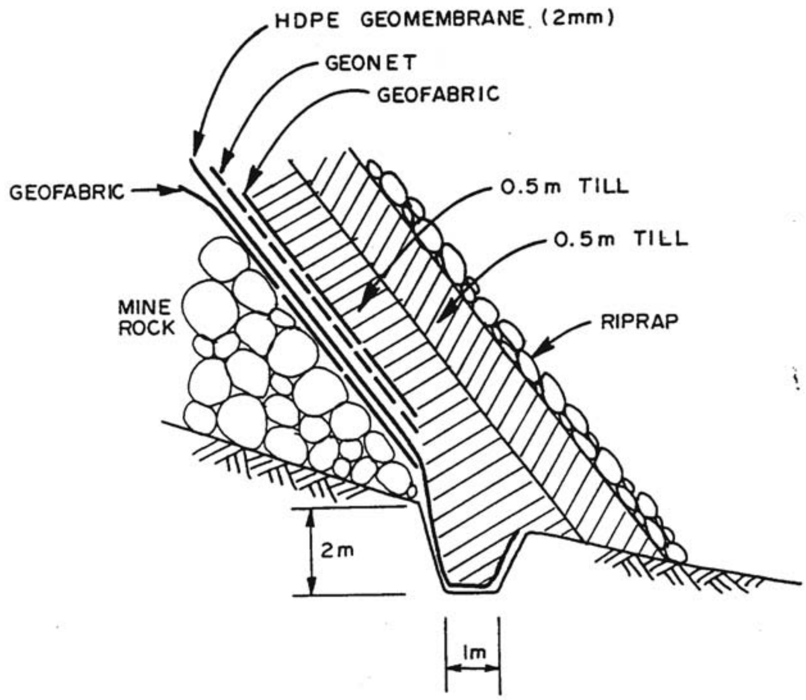

Fig. 4.7. Cover composition and perimeter seal at the Kjøli Mine, Norway (SRK, 1991).

Some damages to the HDPE membrane occurred during placement of the till lifts, resulting from heavy equipment movement. Damage was repaired by conventional patching techniques.

4.1.5.1.1 Costs

The cost of cover placement was about \$5.13 Cdn/m^2 for contouring and ditching, \$21.85 for the supply and installation of all the geosynthetics and \$13.44 for the placement of the till and riprap, resulting in a total cost of \$40.42 Cdn per square meter. The cost per tonne for mine rock consolidation and placing the cover was \$10.00 Cdn.

4.1.5.1.2 Performance

Rapid oxygen depletion in the rock pile was observed, following completion of the cover. Oxygen levels decreased from initial concentrations of about 20% down to concentrations of 5 to 10%. Although oxygen concentrations declined substantially, sufficient concentrations of oxygen are still entering the pile to maintain acid generation.

The cover has effectively reduced acidic drainage from the Kjøli site. The total copper transported from the site was reduced from 8.3 tonnes in 1989 to 2.3 tonnes in 1990, and down to an estimate of less than 1 tonne in 1991.

The placement of riprap in layers and berms has shown to be effective as an erosion control measure. Little evidence of erosion was found on the cover or at adjacent ditching surrounding the cover.

4.1.5.2 University of Saskatchewan Chemical Landfill Cover, Saskatoon

Kozicki (1992) reported on the remediation of a chemical landfill operated by the University of Saskatchewan in Saskatoon. The clean up involved removal of liquid and solid chemicals from the site, and placement of contaminated soil and low-grade solid waste into a containment facility constructed nearby.

Contaminated soil and shredded waste were placed over a till liner and compacted with a vibratory roller. The top surface of the compacted waste was covered with a drainage system consisting of slotted drainage pipes in a layer of coarse filter material, which was covered with a filter blanket. The filter blanket was covered with a top seal, consisting of a 40 mil sheet of HDPE sandwiched between two layers of compacted till (300 mm each) (Figure 4.8). Finally, the upper compacted till layer was covered with 500 mm of topsoil and seeded to grass. Surface slopes of the cover were 50H:1V on the top of the cap and 3H:1V along the sides of the cap.

Fig. 4.8 – Construction of U of S Chemical Landfill Cover (Photo by M. D. Haug).

The 40 mil thick HDPE liner consisted of ten individual rolls, welded together on site to form a continuous barrier. Stringent quality control was enforced to ensure integrity of the welded seams. Initial testing of a weld consisted of performing a peel test on small sections of the weld which had been removed. If areas selected for testing failed, the entire weld was pulled apart and rewelded. After passing the peel test, the weld was subjected to 25 psi of air pressure, which had to be maintained for 30 minutes. Any leaks detected were repaired by bonding a patch with an extrusion weld. Placement of the HDPE geomembrane and compaction of the till on top of the geomembrane were restricted to the mornings, in order to prevent thermal expansion of the geomembrane in the warmer afternoon air.

4.1.5.3 HDPE Geomembrane Test Panel; Halifax International Airport

A 15 m x 30 m test area was covered with a HDPE geomembrane in 1989 at the Halifax airport (Langley, 1991). The 60 mil (1.5 mm) thick geomembrane was placed on a sloping, levelled area and covered with 200 mm of gravel. Originally, the geomembrane was to be covered with till, however, environmental constraints prohibited the use of till.

Results of physical tests on the HDPE found that aging appears to have decreased the breaking strength and elongation properties, while the tear strength has increased.

4.1.5.4 HDPE Geomembrane, Waite Amulet Covers Project

The Waite Amulet covers project was initiated under the MEND program in 1990 (MEND 2.21.2). Two multi-layered sand/compacted-clay/sand test covers, as well as a sand/HDPE geomembrane/sand composite cover were constructed. They were installed on partially oxidized sulfidic tailings at the decommissioned Waite Amulet site near Rouyn-Noranda, Quebec (Yanful *et al.*, 1994). The designs allowed for a direct comparison between the performances of the clay and HDPE layers (Yanful and St-Arnaud, 1991).

Yanful *et al.* (1994) reported on construction of the 20 m x 20 m composite test plot, which consisted of a gravelly sand (30 cm thick) below and a fine to medium sand layer (30 cm thick) above a 2 mm thick HDPE geomembrane. The side slopes (3H:1V) and perimeter drainage ditches were designed to conduct surface runoff away from the cover. A 10 cm thick gravel crust was placed on the upper sand layer to reduce erosion. Construction of the field test plots was completed in September 1990.

Volumetric water contents in the sand layer below the HDPE geomembrane and compacted clay of the different covers, and in the tailings immediately below the covers, were virtually the same. Measurements of volumetric water contents in 1993 in the sand and the tailings were approximately 9% and 50%, respectively. This information indicated that the clay and geomembrane barriers were equally effective in preventing infiltration.

Oxygen measurements during 1992 were consistently at 20.9 % (atmospheric) above and 6% below the geomembrane. It was felt that horizontal inflow of oxygen from the edge of the cover was responsible for the oxygen observed in the sand base (Yanful *et al.*, 1993a and b).

4.1.5.4 Iron Mountain Mine Superfund Site, California

Iron Mountain, near Redding, California, has been exploited since the 1860's for iron ore, silver, gold, copper, zinc and pyrite. AMD is produced as water percolates through the extensive underground mine workings and is discharged into Slickrock Creek and Boulder Creek (Fig. 4.9). Biggs (1991) reported on remedial actions taken at Iron Mountain which included, amongst other measures, the construction of a partial cap over the Brick Flat Pit (Fig. 4.10).

The Brick Flat Pit, formed from mining at the top of the mountain, acted as a collection basin for surface water runoff, and greatly contributed to the volume of water flowing down and throughout the mineralized zone. The design of the cap over the Brick Flat Pit required a large volume of fill in order to raise the elevation of the pit to allow water to flow off and around the pit area. Tailings from the nearby Minnesota Flats area were used to fill the pit.

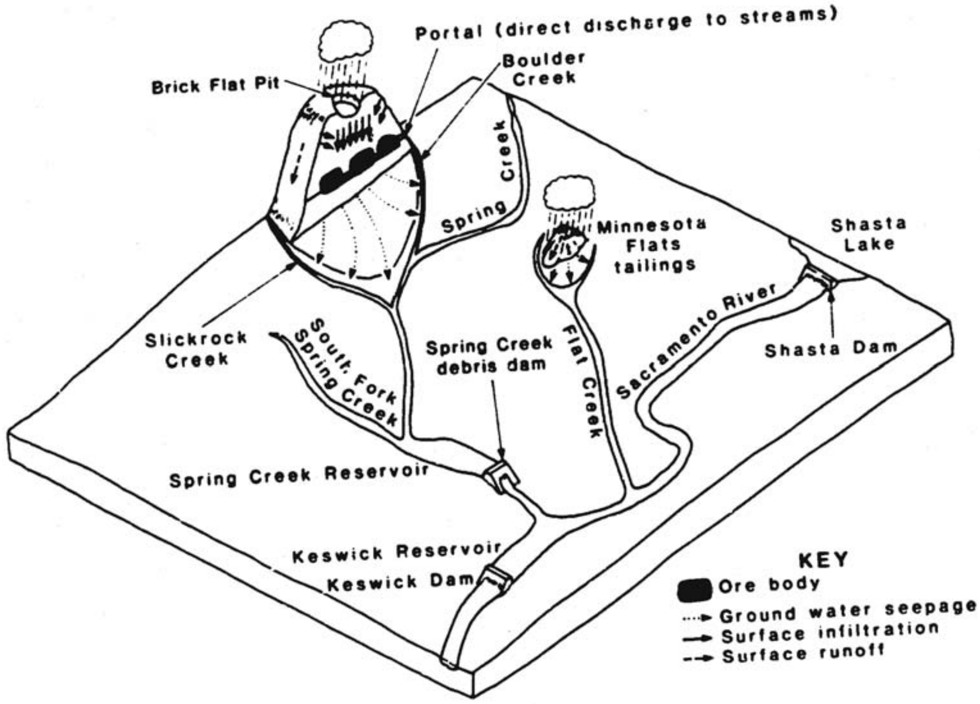

Fig. 4.9. Iron Mountain Mine drainage area (Biggs, 1991).

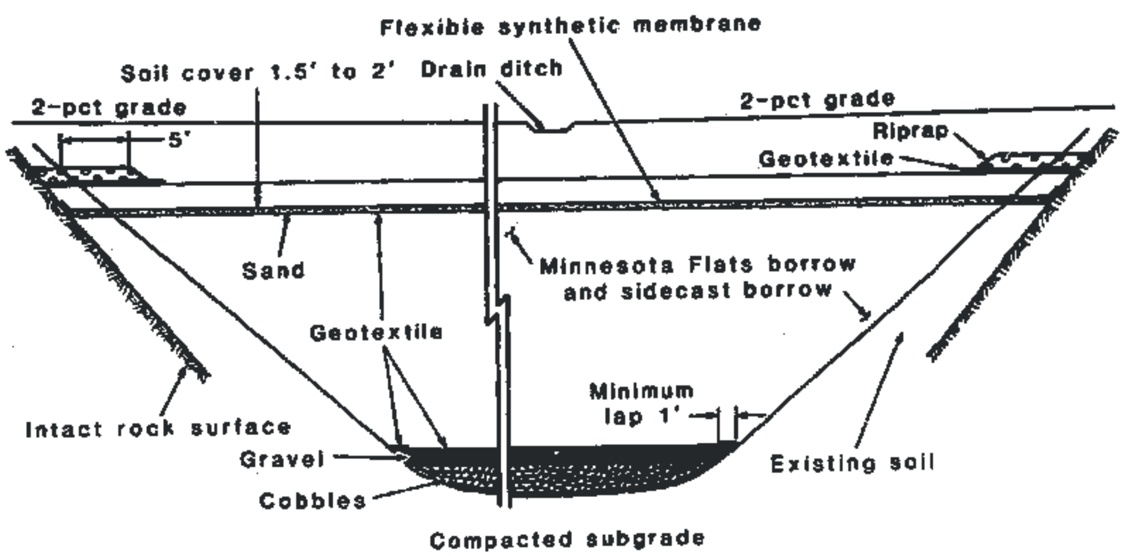

Fig. 4.10 Typical section of the Brick Flat Pit cap (Biggs, 1991).

The design of the cap over the Brick Flat Pit was based on an “inverse filter concept”, where larger cobbles and rocks are placed in the bottom of the pit and smaller material is placed on top (Fig. 4.10). This design was used to inhibit channelling effects in case of water inflows (Biggs, 1991).

The pit floor was graded and compacted, and a layer of geotextile was placed over the floor. Two feet of cobbles were placed over the geotextile, followed by a one-foot thick (0.3m) layer of gravel and another geotextile (Fig. 4.10). The pit was then filled with the Minnesota Flats borrow and sidecast borrow to a depth of about 5.5 m (18 ft) (involving a volume of 25 000m^3 or 30 000 yd^3). The borrow was topped off with another layer of geotextile, 150 mm (6 inches) of sand and a geomembrane (Fig. 4.10). Finally, the geomembrane was covered with 300 to 450 mm (1.5 to 2 ft) of soil at a 3% slope toward the centre of the cap, feeding runoff into a drainage ditch bisecting the cap. The cost of this cap along with filling of caved and cracked ground, and interception ditches was $3 million (US).

4.1.5.5 P.T. Kelian Equatorial Mining Gold mine in East Kalimantan, Indonesia

HDPE was used to provide cover for low-grade ore and waste rock at the P. T. Kelian Equatorial Mining gold mine in East Kalimantan, Indonesia (Firth and van der Linden, 1997). The total area covered with HDPE was 30.9 ha. The main function of the HDPE was to prevent water infiltration and hence flushing of the dumps. The HDPE was also meant to limit air flow and to reduce total suspended solids in the runoff. Monitoring has shown that the HDPE cover was effective in reducing acid leachate by as much as 88%. The life of the HDPE was estimated to be only 50 years and hence it was not considered a suitable long-term option. It was also found at the P. T. Kelian gold mine site that the

HDPE requires high maintenance due to high wind, flood, theft and vandalism.

4.1.5.5 HDPE Geomembrane, SPPA Potash Tailings Pile Cover Research Project

Haug and Wong (1991) reported on the construction of a small, 1.5 mm thick HDPE geomembrane test cover on a potash tailings pile in 1991. The cover was placed directly over the tailings and anchored with a salt berm around each side. This test cover was constructed to determine the effects of chemical and physical exposure on membrane performance, as well as to study the management of runoff from this type of surface. Haug and Wong (1993) observed dissolution erosion of the salt tailings berms holding the HDPE geomembrane in place, such that the geomembrane “rolled up” around the edges. As a result, it was necessary to repair the cover in 1993 by placing large boulders on its surface (Haug *et al.*, 1994). The HDPE geomembrane does not appear to have been affected by exposure to the salt in the tailings pile or to the outside climate conditions (Haug and Pavier, 1995).

4.1.5.6 HDPE Geomembrane, Ste-Gertrude Landfill

A geosynthetic cover system was installed in 1992 on the Ste-Gertrude, Quebec sanitary landfill, as described by Firlotte *et al.* (1993). The purpose of the cover system was to limit leachate production. The landfill was contoured to have 3.5% top slopes and 4H:1V side slopes. A 1 mm thick HDPE geomembrane was used as an infiltration barrier, which was placed on 600 mm of sand bedding (Fig. 4.11). A geonet was placed on the side slopes to ensure drainage and prevent porewater pressure build-up above the geomembrane, thus preventing stability problems. A 500 mm cover of sand was placed above the geotextiles, as well as 100 mm of topsoil at the surface.

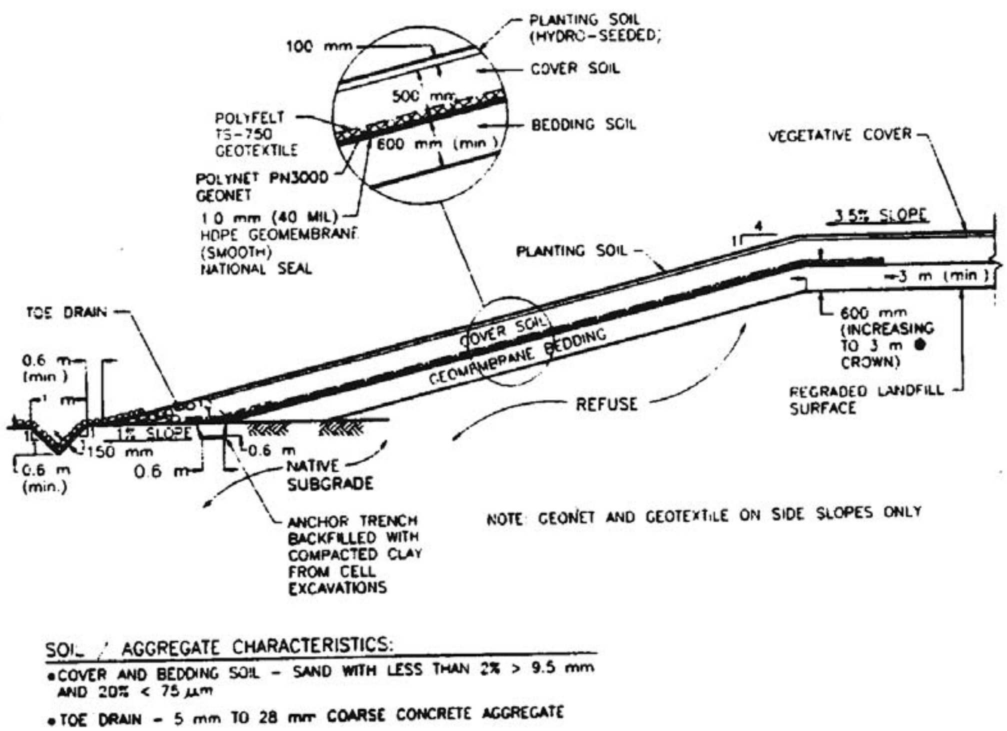

Fig. 4.11. Slope geometry and cover components of Ste-Gertrude sanitary landfill (Firlotte et. al, 1993).

Firlotte *et al.* (1993) emphasized that quality control was extremely important to minimize damage to the geomembrane during placement. Placement of sand cover material should be done during the coolest part of the day to minimize development of wrinkles in the geomembrane. As well, a minimum thickness of 300 mm is needed to protect the geomembrane from smaller construction equipment (e.g. Caterpillar D-4). A 900 mm minimum thickness is recommended for other heavy equipment.

4.1.5.7 HDPE Geomembrane, Trail Road Landfill, Nepean, Ontario

A cover on a municipal landfill cell was constructed in 1988 at the Trail Road Landfill in Nepean, Ontario. The cover system incorporates a HDPE geomembrane as a hydraulic barrier to limit infiltration. Warith *et al.* (1995) studied the integrity of this cover in light of observed settlement experienced by the cover.

Warith *et al.* (1995) demonstrated the use of state-of-the-art evaluation methods for addressing topics of infiltration through the geomembrane, and analysis of tensile stresses associated with differential settlements. Infiltration prediction was done in light of reduction in cover slope, which led to a predicted increase in head above the geomembrane. The analysis of strains in the geomembrane correspond to depressions developing since 1988 indicating that a factor of safety of about 2 exists between the maximum allowable strain at yield and the effective strain in the HDPE geomembrane cover.

4.1.5.8 PVC Geomembrane: Dyer Boulevard Landfill, Palm Beach County, Florida, USA

Levin and Hammond (1990) reported on the status of a 0.5 mm thick PVC geomembrane, installed as a cover over the Dyer Boulevard Landfill, after about five years of service. In 1989, the two feet of soil backfill on top of the PVC geomembrane was removed in several locations to expose the liner material. Nine samples of the PVC liner (one seamed, eight unseamed) were cut from the geomembrane for testing.

Material testing of the PVC specimens indicated a 13% loss of plasticizer, which resulted in an increase in stiffness and a decrease in elongation. Whether or not the PVC will continue to exhibit sufficient elongation and ductility to sustain extensions in the geomembrane resulting from differential settlement is of concern. Differential settlement is a concern for many types of waste rock piles.

4.1.5.9 Bituminous Geomembranes, Unnamed Spoil Heap in Northern France

Bienaimé and Herment (1990) reported on the use of a bituminous geomembrane for infiltration control on an ore spoil heap in northern France. The 10 m high ore spoil heap contained soluble salts, with a surface area of about four hectares. A COLETANCHE bituminous geomembrane was used for the cover, which consisted (from bottom up) of:

- a puncture-proof film
- a non-woven polyester reinforcement impregnated and coated with 100/40 fillerized bitumen containing 20% of fines of size smaller than 80 microns
- a sanded surface
- an anti-adhesion film

The bituminous geomembrane had a thickness of 3.9 mm and hydraulic conductivity of 10^{-14} m/s. It was selected for its ability to withstand chemicals such as alkaline salts, and normal ageing agents such as sun, water, soil, and microorganisms.

The waste heap is shown in Fig. 4.12. The geomembrane was installed in 1986 and has successfully withstood storms with wind gusts up to speeds of 133 km/hour.

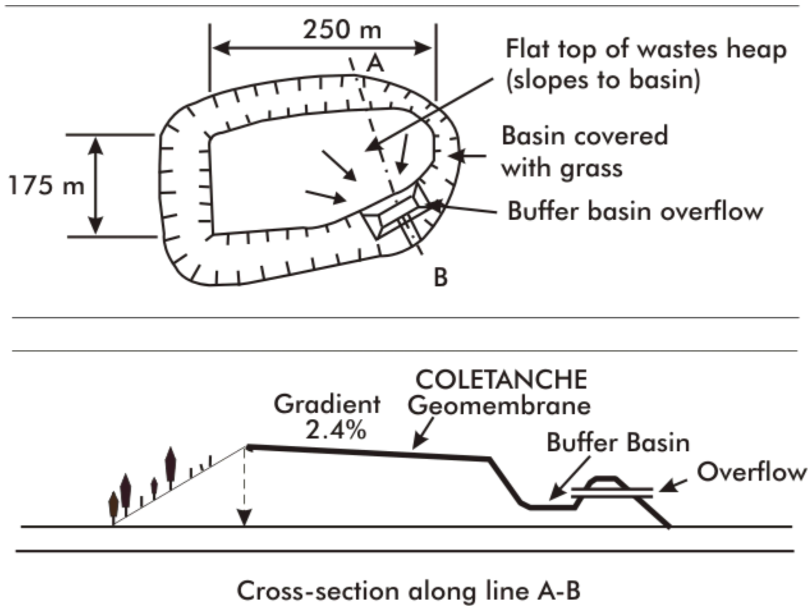

Fig. 4.12. Bituminous geomembrane waterproofing system for waste heap in Northern France (Bienaimé and Herment, 1990).

4.1.5.10 Bituminous Geomembrane over Waste Rock Pile at Mount Washington

An asphalt emulsion/geotextile waterproof membrane was placed in a 1991 pilot program on the surface of acid generating waste at Mount Washington, to inhibit infiltration (Galbraith *et al.*, 1992). Figure 4.13 shows a view of the cover taken in 1999. Material costs of the asphalt emulsion/geotextile membrane were about $5 Cdn per square metre. Galbraith (1993) found that although the use of the asphalt emulsion/geotextile membrane held promise for providing the strength, low cost and flexibility essential for a waste cover, it was difficult to construct. Galbraith (1993) recommended that because asphalt is not a material amenable to hand work, further use of this type of cover should await development of a mechanical application method. Transportation of the material to this particular site is also a challenge.

Fig. 4.13 – Asphalt emulsion/ geotextile waterproof membrane (Photo by M. D. Haug).

4.1.5.11 HDPE Liner, Poirier Mine Site, Joutel, Quebec, Canada

Lewis *et al* (2000) reported on the reclamation program for the Poirier Site. Poirier was an underground copper and zinc mine located approximately 200 kilometres north of Rouyn-Noranda, near the community of Joutel. The mine operated from the mid-1960s to 1975, when it closed due to low copper prices and high costs of production.

There are approximately 4 million tonnes of acid generating tailings which has contributed acidity and metals to the local watershed. Two streams have been affected by effluent from the site. To reduce the loadings from the site to the environment, five reclamation options were considered: a soil cover; a geomembrane liner with a protection layer; to collect and treat; a clay cover with frost protection; and burying the tailings.

A common feature is the clean up of acid generating materials (i.e., spilled tailings) and placing them in the main tailings deposit consolidating all of the acid generating materials at one location. An important feature of the site is the presence of a low permeability clay layer, underlying the main tailings deposit, which acts as a barrier to limit groundwater contamination.

One of the important objectives of the reclamation plan was to minimize or eliminate the need for long-term operations at the site, such as water treatment and sludge disposal. The final closure option involved a cover consisting of a geomembrane liner over the main tailings deposit and the relocated mine waste materials together with 1 m thick soil layer to protect the geomembrane liner. The liner will substantially reduce the infiltration through the cover, act as an oxygen barrier to limit further oxidation of tailings, and protect tailings from wind and water erosion. The environmental effect of the residual seepage was assessed from an ecological risk perspective and the liner was found to have the lowest loadings to the surrounding environment.

Work commenced in 1998. A smooth 60 mil HDPE liner was used on the graded and compacted beach and central waste piles. The liner installation has been difficult due to inclement weather throughout fall. The consistency of the fine tailings material is such, that it takes at least two days for the sub-surface to drain sufficiently to allow the installation and welding of the liner. On the tailings basin perimeter slope, a textured geomembrane liner 80 mil thick was used on the slope with a geonet drainage layer above the textured liner. A 0.5 m protective layer of clay was placed over all of the liner. Over the clay, 0.5 to 1.5 m of till was placed to both protect the clay and provide a growing medium.

A high level of quality control and workmanship is required to obtain the necessary bonding between the sheets of liner. A rigorous quality control program was implemented at the site. The contractor, who performed the placement and sealing work, tested each seam for air leaks. At random times, samples of the seams were removed and tested for durability and measured for the area of contact.

On routine intervals, sample test strips were sent to independent laboratories for third party verification.

The site remediation was completed with the planting of vegetation in the year 2000. The efficiency of the liner will be closely monitored with moisture, oxygen and temperature probes. Sections of the geomembrane liner will be removed and evaluated in order to develop longevity data. Predicted improvements to the local environment will also be monitored.

4.1.5.12 HDPE Liner, Weedon Mine Site, Quebec, Canada

ARD mitigation at the Weedon Mine in Quebec, involving the use of a high density polyethylene membrane, was reported by Tremblay and Bedard.

The Weedon Mine is located about 100 km east of Sherbrooke. This is an old copper mine that operated between 1952 and 1959. Approximately 425,000 tonnes of tailings were deposited in two ponds. Crushed waste rock and pyretic material were also found on site.

Over the years the barrier surrounding Pond A had broken, allowing the tailings to spread over an area of about 1 square km and contaminate a local river. A study was commissioned for reclamation work. Capping, using a 1.5 mm (60 mils) high density polyethylene membrane was selected as the method of containment. A one-meter layer of protective soil was applied on top of the membrane and seeded.

A new impermeable dyke was constructed for Pond A. All of the loose tailings and pyretic material were placed in Pond A (54,000 sq. meters) which was then capped. Due to its size, Pond A was sub-divided into four cells using the acid generating waste rock that was to be placed in the pond. Pond B (12,000 sq. meters), the smaller of the two ponds, was capped in placed and seeded. Seepage from the two ponds are collected and treated using a natural wetland system.

The work of the two ponds was completed in 1993 at a cost of $3.8 million Cdn. Monitoring of the ponds has been on-going since.

4.2 Spray-on Membrane Barriers

A polyurethane based spray-on lining material for mine-wall support in underground hard-rock mines called Mineguard™, was developed by the Mining Industry Research Organization of Canada (Archibald *et al.*, 1995). The polyurethane membrane is formed by combining sprays of two liquid chemicals into a solid layer, which cures within seconds of application and is capable of developing adhesion to intact and broken rock surfaces (Archibald *et al.*, 1992).

Archibald and Lausch (1996) proposed the use of polyurethane sprays as surface tailings covers for AMD prevention. A summary of physical properties of three spray on materials (Table 4.6) indicates desirable behaviour of spray-on liners, such as low oxygen transmission, low hydraulic conductivity, good flexibility and strength, and good chemical resistance.

Table 4.6. Summary of polyurethane and polyurea material physical properties (Archibald and Lausch, 1996).

Material	Oxygen Gas Transmission ($cm^3/m^2/day$)	Hydraulic Conductivity (m/s)	Tensile Strength (MPa)	Elongation Capacity (%)	Abrasion Resistance	Acid Resistance
Type 877FR (polyurethane) (@ 0.96 mm)	29.0	2×10^{-12}	19-24	50-157	Excellent	Excellent
Urea-Tuff 3000 (polyurea) (@ 0.58 mm)	397.5	6×10^{-13}	18.3	312.5	Very Good	N/A
Type Soft-2 (polyurea)	N/A	N/A	9.7	304.5	N/A	Very Good

Archibald and Lausch (1996) ruled out the Type Soft-2 polyurea (Table 4.7) based on its lower strength and high cost. Costs of polyurethane and polyurea installation are $165,000 and $175,000 Cdn per hectare, respectively (Table 4.7).

Table 4.7. Summary of anticipated polyurethane and polyurea material spray installation costs in $ Cdn (Archibald and Lausch, 1996).

	Cost per Square Metre Covered @ 1.5 mm thickness		
Material	Raw Material	Application	Total
Type 877FR Polyurethane	15.39	1.12	16.51
Urea-Tuff 3000 Polyurea	16.41	1.12	17.53

The development of polyurethane and polyurea sprayed membranes for infiltration and oxygen barriers is at its infancy. Further research required includes field testing and dealing with additional concerns such as covering with soil and long-term effectiveness. The membrane, following placement on the tailings surface, will likely be covered with soil to facilitate vegetation. The 1.5 mm membrane thickness, as suggested by Archibald and Lausch (1996), and uniformity of spray-on membranes may be difficult to regulate during field application. Fabrication of field constructed membranes cannot expect the same level of quality control as geomembranes manufactured in a factory under controlled conditions. One major advantage of spray-on membranes over conventional geomembranes is the absence of seams.

Sengupta (1993) lists several other spray-on surface sealants which may be applied to the surface of waste to form a barrier to infiltration and oxygen diffusion. These include:

- alkyd
- asphalt
- epoxy
- polyester
- polysulfide
- silicone
- synthetic rubber
- thermoplastic molten sulphur
- vinyl

These materials have been developed for applications such as caulking sealants, soil stabilizers, waterproof barriers and corrosion protective coatings. Application to mine waste covers is limited (Sengupta, 1993).

4.3 Geosynthetic Clay Liners (GCLs)

Geosynthetic clay liners (GCLs) have been commercially available since the early 1980's (Lowry, 1992). The number of GCL producers has decreased significantly over the past decade as the smaller operations have been taken over by larger producers. GCLs consist of thin layers of processed clay (typically bentonite) placed between geotextiles, or bonded onto a geomembrane. GCLs have sometimes been used by themselves as a barrier layer in liners and covers. However, the primary use of GCLs is as the clay component of a primary composite liner for double liner systems and as the clay component of a composite liner in final cover systems for landfills or site remediation projects (Daniel, 1993a). Secondary containment structures have also been lined with GCLs (Bruton, 1991).

GCLs are manufactured in two configurations, bentonite sandwiched between two geotextiles (Figs. 4.14a & b), and bentonite mixed with an adhesive and glued to a geomembrane (Fig. 4.14c). With the case of bentonite sandwiched between two geotextiles, the bentonite may be held in place either by needle-punched fibres (Fig. 4.14a) or by mixing the bentonite with an adhesive (Fig. 4.14b). Needle punching is done to contain the bentonite, as well as to enhance the in-plane shear strength. GCLs typically contain between 3 and 6% bentonite per square metre, and are manufactured in widths of 4 to 5 m. GCLs are typically 7 to 10 mm thick in their hydrated state.

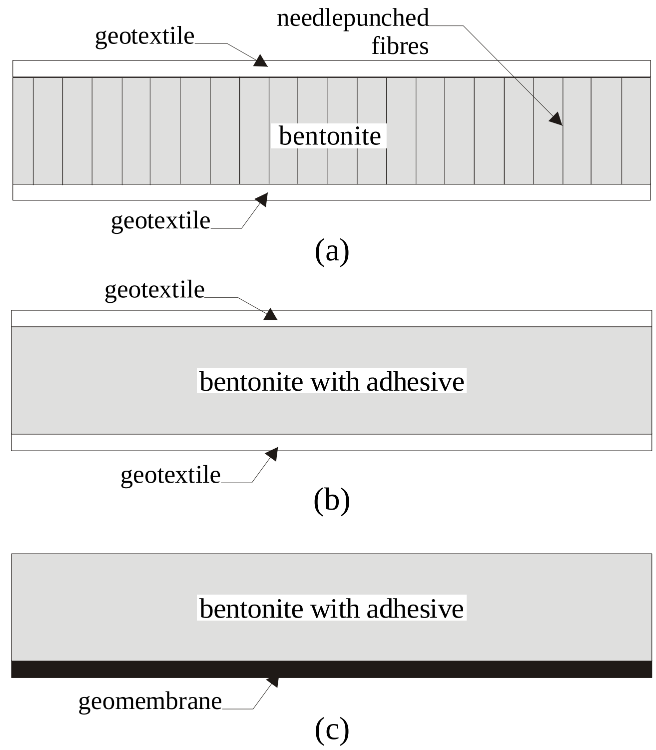

Fig. 4.14. Schematic diagram of three geosynthetic clay liner configurations.

Installation of GCLs itself is relatively quick provided the surface has been properly prepared and the correct installation equipment is available. No mechanical seaming is required between joints. Required minimum overlaps range from 150 mm to 300 mm, depending on the specific material product (Eith *et al.*, 1991). In most cases, additional bentonite is placed between overlaps to enhance sealing at overlaps.

GCLs need to be covered immediately so that the bentonite hydrates in the presence of a confining pressure. Otherwise the bentonite may not swell evenly and the overlaps may not self-seam.

Extrusion of bentonite through the support fabrics may present a problem when a GCL is placed next to a geonet. In this case, it may be necessary to place a geosynthetic layer between the GCL and the geonet (Simpson, 1991) to protect the GCL and prevent migration of clay particles.

GCLs have been used in composite barrier systems. Geomembranes may be placed over GCLs (Rowe and Fraser, 1994) to increase redundancy of the barrier system.

4.3.1 Engineering Properties

4.3.1.1 Shear Strength

Shan and Daniel (1991) performed drained direct shear tests on three early types of GCLs (Table 4.8). Bentomat® has higher shear strength parameters because it is needle punched, which increases in-plane shear strength. The Claymax® and Gundseal GCLs consist of bentonite mixed with adhesive sandwiched between two geotextiles and bentonite mixed with adhesive attached to a geomembrane, respectively.

Table 4.8. Drained internal shear strength parameters of three GCLs (Shan and Daniel, 1991).

GCL	Effective Cohesion (kPa)	Effective Friction Angle
Bentomax®	30	26°
Claymax®	4	9°
Gundseal	8	8°

Bentonite is a montmorillonite-rich clay of volcanic origin. Montmorillonite clays are known to exhibit low shear strength. Mesri and Olson (1970) found that the slope of the failure envelope of sodium montmorillonite decreased from 4º to 0º as effective stresses varied from 70 to 56 kPa. The residual effective friction angle of natural clays rich in montmorillonite, such as Cretaceous shale of the Western Interior Basin, is known to be low, with reported values of 6.5° (Kelly *et al.*, 1995), 6.2º (Eckel *et al.*, 1987) and 6.7º (Sauer and Christiansen, 1987).

Gilbert *et al.* (1996) conducted direct shear tests to evaluate the internal strength of a reinforced GCL. Although the fibre reinforcement increased peak shear strength, (compared to GCLs without reinforcement) large displacement (43 mm) had resulted in pullout of reinforcing fibres from the geotextile. Thus for large displacements, the internal shear strength of the reinforced GCL was approximately equal to that of the unreinforced GCL.

Gilbert *et al.* (1996) also observed extrusion of bentonite into interfaces with the GCL in all of the interface tests, which likely lowered interface shearing resistance. The GCL/smooth geomembrane interface shear resistance was lower than the internal shear resistance of an unreinforced GCL.

The use of pure bentonite in GCLs creates potential stability concerns. GCL joints, unlike welded geomembrane seams, cannot sustain appreciable tension or shearing forces. Covers utilizing GCLs must be designed to ensure that tension does not develop between joints. Composite covers with a GCL placed in contact with a geomembrane may suffer serious stability problems. Localized shear displacement in reinforced GCLs, causing pullout of reinforcing fibres may lead to progressive instability.

4.3.1.2 Bearing Capacity

Bearing capacity of GCLs is a concern because of their low strength. Adequate backfill must be placed before the GCL hydrates, in order to avoid lateral squeezing of bentonite under concentrated vertical applied loads. Koerner and Narejo (1995) conducted laboratory tests to determine the suitable thickness of soil cover required over GCLs. Koerner and Narejo (1995) determined that a height-over-breadth ratio (H/B) equal to 1.0 was adequate for GCLs with a sand cover. Bearing failure must be contained within the cover soil above the GCL.

4.3.1.3 Hydraulic Conductivity

Daniel (1993a) presented a summary on the hydraulic conductivity data of GCLs (Fig. 4.15). The hydraulic conductivity of GCLs was found to vary between 10^{-12} and 10^{-10} m/s, depending on the compressive stress. Estornell and Daniel (1992) conducted bench scale (2.4 m x 1.2 m) hydraulic conductivity tests on individual and seamed sheets of GCLs. Except for the case where low (< 7 kPa) vertical effective stress was employed, the hydraulic conductivities of overlapped materials were almost identical to values measured on control samples with no overlap.

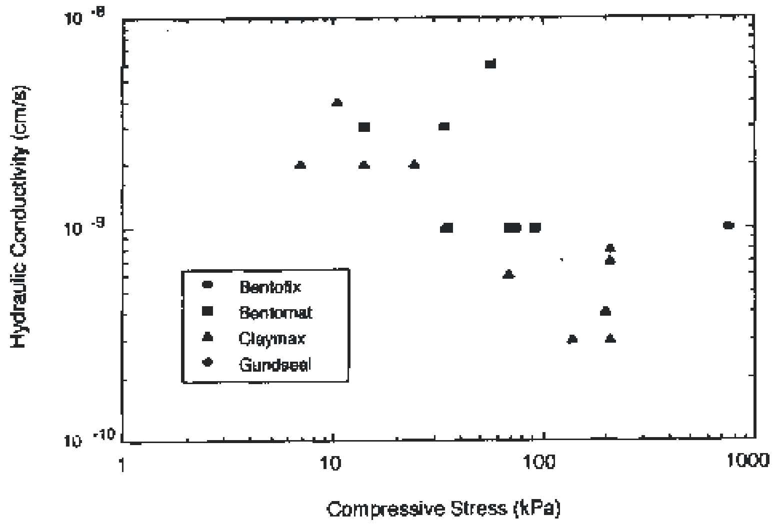

Fig. 4.15. Hydraulic conductivity of geosynthetic clay liners as a function of compressive stress (Daniel, 1993a).

Boardman and Daniel (1996) performed bench scale (2.4 m x 1.2 m) hydraulic conductivity tests to determine the effect of one cycle of wetting and drying on several GCLs. Samples were hydrated and hydraulic conductivity tests were conducted. After the initial test, the intact samples were desiccated (water content equalled 12% in an excavated sample) and became severely cracked. When the desiccated samples were rehydrated, water initially flowed rapidly through the cracks. However, the bentonite quickly expanded, sealing up the cracks. The long-term steady state value of hydraulic conductivity was essentially the same before and after the desiccation cycle. However, care must be taken in design, to ensure that openings in the geotextile component of the GCL are small enough to prevent migration of the overlying soil into cracks that may develop in the bentonite. Further research is required to determine GCLs response to numerous wet/dry cycles in the field.

4.3.2 Case Studies

Heerten and List (1990) reported on the use of a fibre-reinforced GCL in a double liner system installed during rehabilitation of the Lechkanal. The GCL, Bentofix® lining type A was used as a secondary liner underlying the primary sealing layer which consisted of a 14 cm thick asphalt liner with a drainage layer. In total, 60,000 m^2 Bentofix® was used in rehabilitation of the canal which took place in 1989.

4.3.2.2 Munich Airport

Scheu *et al.* (1990) reported on the use of a GCL in construction of a maintenance free purification network for collecting and biologically purifying de-icing fluid in situ at the Munich II airport. Construction of the network took place in 1989 at a total cost of DM 20 million where the cost of the GCL - Bentofix® (supplied and installed) was DM $8.5 million (~DM $13/$m^2$).

Approximately 650,000 m^2 of Bentofix® was used to isolate the 28 km long x 22 m wide maintenance free purification network for collecting and purifying de-icing fluid, from the groundwater below. The GCL was installed at rates of up to 7000 m^2 per day (Scheu *et al.* 1990).

4.3.2.3 Base Seal in a Swedish Sludge Basin

The GCL, Bentofix® type B, was installed to seal the foundation pit base and the side slopes of a sludge basin in Himmerfjärdsverket, Sweden. Approximately 35,000 m^2 of the GCL was used to line the base and side slopes of the basin and designed to hold 150,000 m^2 of dewatered digested sludge. Crushed rock aggregate was placed on the GCL to protect against mechanical damage and to equalize swelling in the GCL when the bentonite hydrates (Naue-Fasertechnik, 1991).

4.3.2.4 Somex Mine Site, Quebec

ARD mitigation work at the Somex Mine Site in Quebec, involving the use of a “Bentofix” type membrane, was reported by Bienvenu and Dufour (1996).

The Somex mine is located about 50 km north-east of La Tuque, Quebec, in an area well known for fishing and hunting. It is an abandoned site that was restored by the provincial government. Approximately 100,000 tonnes of ore (nickel and copper) was mined at the site. A portable mill was used during operation. Only the tailings area remained.

The main objective of the reclamation work at the small site (1.5 ha) was to confined the tailings and control the production of acid mine drainage. The closure option selected consisted of a “Bentofix” type geomembrane covered with one metre of sand. A topsoil layer was added and seeded. Total cost of remediation was about $250,000 Cdn, a little less than $200,000 Cdn/ha.

4.3.2.5 Memorial Gardens Fresh Water Pond, Yorkton, Saskatchewan, Canada

A 300 m^2 GCL field test project was conducted on a fresh water pond at Memorial Gardens in Yorkton, Saskatchewan. A double lined system, consisting of a GCL over a sand drainage layer over a PVC geomembrane, was installed. A collection system was established at the base of the pond, so that flow through the GCL could be carried horizontally outside the limits of the pond to a vertical riser pipe. The difference in head and flow through the GCL was monitored and compared with the results of seepage modelling to confirm that the system had a bulk hydraulic conductivity of $<10^{-7}$ m/s.

Field tests of GCLs are crucial in establishing a service record. The pond at Memorial Gardens will provide information into the effects of freeze/thaw on the behaviour of GCLs.

4.3.2.6 Summitville Mine, Colorado, USA

The Summitville Mine is an open-pit gold mine that was abandoned in late 1992, when the owners of the mine declared bankruptcy. In the following years, options for controlling ARD were being investigated. The current reclamation activities started in May 1999 include the installation of a geocomposite clay liner to cap the heap leach pad and prevent saturation of the pile to minimize ARD generation. A compacted bentonite clay liner was placed in both the South and North pits. Waste from the Cropsy pile was then backfilled into the pits. A geocomposite clay liner was used to cap the South Pit and a compacted clay cap was placed over the North pit to minimize infiltration into the fill. A 4-foot thick soil cover was placed over all caps for frost protection, topsoil was then placed, and the areas revegetated. Monitoring of the site will continue for up to 10 years, following the completion of the current reclamation activities.

4.3.3 Costs

In the US, installed costs of GCLs vary from $5.40 to $10.80 per square metre, depending on site conditions (Daniel and Koerner, 1993).

4.3.4 Comparison of GCLs with Compacted Clay Liners

Geosynthetic clay liners (GCLs) sometimes are used as an alternative to compacted clay liners (CCLs), and the two are often directly compared (Table 4.9). The main advantages of GCLs are quicker and for the most part easy construction. There is also some evidence that GCLs are capable of resisting the effects of desiccation and freeze/thaw cycles. Some disadvantages regarding GCL use are the lack of data from independent research and field experience, vulnerability to puncture, and low inherent shear strength (Daniel and Estornell, 1991).

Table 4.9. Comparison of geosynthetic clay liners with conventional compacted clay liners (Daniel, 1993a).

Compacted Clay Liner (CCL)	Geosynthetic Clay Liner (GCL)
• Thick (0.6 to 1.5 m)	• Thin ($\leq$10 mm)
• Field constructed	• Manufactured
• Require knowledge to build	• Relatively easy to build (unroll and place)
• Impossible to puncture	• Possible to damage and puncture
• Constructed with heavy equipment	• Light construction equipment can be used
• Often requires test pad at each site	• Repeated field testing not needed
• Site-specific data on soil needed	• Manufactured product – data available
• Large leachate-attenuation capacity	• Small leachate-attenuation time
• Relatively long containment time	• Shorter leachate-attenuation time
• Large thickness takes up space	• Little space is taken
• Cost is highly variable	• More predictable cost
• Soil has low tensile strength	• Higher tensile strength
• Can desiccate and crack	• Cannot crack until wetted
• Difficult to repair	• Easy to repair
• Vulnerable to freeze/thaw damage	• Less vulnerable to freeze/thaw damage
• Performance is highly dependent upon quality of construction	• Hydraulic properties are less sensitive to construction variable
• Slow construction	• Much faster construction

GCLs should not be used in cover designs where large differences in head occur across the barrier layer. In such a case, a large hydraulic gradient may develop across the GCL because of its small thickness

Differential settlement is a concern with GCLs. Bentonite can shift within the carrier layers. EPA sponsored research (LaGatta, 1992) shows thinning of GCLs under a settlement-simulation testing program. A GCL failed (hydraulic conductivity increased to over 10^{-7} m/s) after recording a settlement of less than 25 mm.

5.0 Bentonite Modified Soil Barriers

5.1 Soil-Bentonite Mixtures

Bentonite is a naturally occurring commercial product consisting primarily of the clay mineral montmorillonite. Montmorillonite is one of the smectite group of clay minerals. Sodium montmorillonite has sodium as the primary adsorbed cations. As a natural product, the properties of bentonite vary widely, depending on the circumstances surrounding its formation. Hence, not all bentonites are of equal quality; therefore, in most cases testing is required before a specific bentonite is used in a particular application.

Many soils do not have low hydraulic conductivity. In these cases bentonite may be used as an additive to lower the hydraulic conductivity. This resulting low permeability material may then be suitable for construction of water and waste containment facilities.

Bentonite is a very fine material having a specific surface ranging from 700 to 840 m^2/g (Mitchell, 1976). It also has an expanding crystalline lattice structure which allows it to adsorb dipolar water molecules and swell. This chemically attracted water form an "ice-like" substance around the clay particles. Thus when a bentonite modified soil adsorbs water its effective void ratio (available for water flow) decreases.

Bentonite has been widely used to modify sand in cover and liner applications. Bentonite has also been used with other materials, such as till for barriers. Pusch and Alstermark (1985) reported on the construction of a test cover in Sweden in 1982, which consisted of 10% sodium bentonite and 90% till. In this case the till used was a coarse till, consisting of 65% sand and gravel and 35% silt.

5.1.1 Properties

5.1.1.1 Freeze/Thaw Resistance

The impact of freeze/thaw cycling on the hydraulic conductivity of soil-bentonite liners and cover materials had been investigated by a number of researchers (Wallace, 1987; Haug and Wong, 1993b; Shan and Daniel, 1991; Claire et al, 1993; and Kraus et al, 1997). Considerable resistance to freeze/thaw was observed and in many cases only slight increases in hydraulic conductivity were observed.

5.1.1.2 Compatibility with Acidic Drainage

Cover materials may not suffer the same level of contact with acidic drainage as liners or subsurface barriers would at an acid generating site. However, acid from waste impoundments may migrate into barriers within covers due to capillary rise, as well as from discharge along the side slopes of the impoundment. If the cover design allows for migration of acid into the barrier layer in the cover, the barrier layer must adequately resist degradation caused by the acid.

Yanful and Shikatani (1995) used X-ray diffraction and geochemical analysis to observe a transformation of montmorillonite and illite to kaolinite, when a sand-bentonite mixture was permeated with acidic drainage. It was found that the hydraulic conductivity of the sand-bentonite mixture changed only marginally as a result of the transition.

Evidence of direct transformation of montmorillonite to kaolinite during weathering of Florida soils has been reported by Altschuler *et al.* (1963). In Florida, the transformation of iron-rich montmorillonite to kaolinite takes place in an acidic environment (pH < 7). Liberated SiO_2, Fe_2O_3, MgO and K_2O were leached away and formed secondary chert, geothite and possibly dolomite. Diagenesis of smectite was also known to occur at high temperatures, where a transformation to illite occurred (Inoue *et al.*, 1992).

The hydraulic conductivity may increase if a montmorillonite is chemically reduced in situ after consolidation (Shen *et al.*, 1992). Unoxidized montmorillonite was known to exhibit lower swelling pressure than in its oxidized state (Foster, 1953). Such a situation may arise if a geomembrane was placed directly over a montmorillonite layer, thus limiting oxygen which may cause a reduction of iron (Breu *et al.*, 1993).

5.1.1.3 Subsidence Effects

Subsidence and differential settlement in waste rock and tailings facilities necessitates a degree of flexibility in cover systems. Jessberger and Stone (1991) used a centrifuge model tests to study the response of barriers subjected to differential deformations and the effect of confinement pressure in this event. Model liners fabricated from a sand (64%), silica flour (22%), and bentonite (14%) mixture proved to be resistant to deformation, whereas pure kaolin liners were not.

Jessberger and Stone (1991) found that vertical confinement (Fig. 5.1) could reduce tension cracking in pure kaolinite. In the case of the sand/silica flour/bentonite mixture, only slight surface cracking was observed for deformations of 16°. Moreover, the hydraulic conductivity of the unconfined sand/silica flour/bentonite mixture remained low throughout deformation, increasing from a hydraulic conductivity of 1.9 x 10^{-10} m/s at 9.5° of deformation, to 2.9 x 10^{-10} m/s at 16° of deformation.

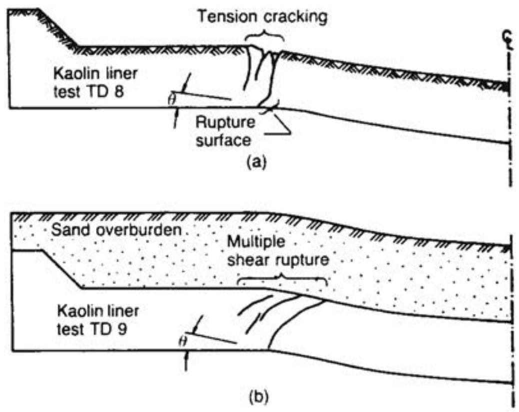

Fig. 5.1. Traces of crack and rupture patterns observed a) without overburden and b) with overburden (Jessberger and Stone, 1991).

Haug *et al* (2000) investigated the impact of compression induced cracking on compacted soil-bentonite specimens. The results of this study found that the hydraulic conductivity of 8% Wyoming bentonite and 92% Ottawa sand samples were not affected by up to 25% strain. Volume change measurements taken during hydraulic conductivity testing suggested that the application of axial strain under undrained conditions, results in the creation of open fractures. These fractures, however, took in water during permeation and saturation was re-established.

5.1.2 Case Studies

5.1.2.1 UMTRA Cover, Maybell, Colorado

Claire *et al.* (1993) reported on the design of a sand-bentonite barrier for a UMTRA site at Maybell, Colorado. Uranium tailings at UMTRA sites are covered to minimize radon diffusion and infiltration. Typical covers include (from tailings surface up): 1) a radon/seepage barrier, 2) a frost protective layer, 3) a drain/bedding layer which drains water laterally off the pile and prevents migration of fine particles, and 4) a riprap cover for erosion protection. The upper edges of the cover slope away from the centre at 2%, whereas the side slopes are 5H:1V. Barrier materials constructed at UMTRA sites are compacted clays and soil-bentonite mixtures (Claire *et al.*, 1993). The standard means of preventing deleterious effects of freeze/thaw on the barrier layer is to place a protective layer above the barrier. The protective layer may range from 0.6 m to 2.3 m. Protective frost layers are costly (from $300,000 to $1,700,000 U.S. at UMTRA sites), therefore the use of radon/infiltration barrier layers which are not susceptible to freeze/thaw deterioration could provide a substantial cost savings.

Claire *et al.* (1993) conducted laboratory testing to determine effects of freeze/thaw cycling on various sand-bentonite mixtures. The degree of freeze/thaw resistance was found to be dependent on bentonite quality. Admixes with high quality Wyoming bentonite performed the best, with hydraulic conductivities in the 1.5 to 3 x 10^{-10} m/s range, after being subjected to five unsaturated closed system freeze/thaw cycles.

Claire *et al.* (1993) suggested that a protective frost barrier at the Maybell, Colorado UMTRA site might not be necessary, based on favourable freeze/thaw testing in the laboratory. However, additional work is required to determine minimum bentonite quantity and quality required to perform adequately under freeze/thaw conditions.

The testing procedure used by Claire *et al.* (1993) may be partially responsible for lack of deleterious effects of freezing and thawing, as samples were tested under effective stress levels up to 21 kPa in flexible walled permeameters. Wallace (1987) tested sand-bentonite specimens, and found that for a high effective confining pressure (70 kPa), no increase in hydraulic conductivity occurred with increasing numbers of freeze/thaw cycles. Kim and Daniel (1992) determined that increasing the confining stress (14 to 35 kPa), lessened increases in hydraulic conductivity for compacted clay samples subjected to freeze/thaw cycles. Wong and Haug (1991) conducted permeability tests involving freeze/thaw cycles at 17.2 kPa of vertical effective confining pressure and obtained test results, which indicated a slight decrease in hydraulic conductivity with freezing and thawing.

5.1.2.2 Sand-Bentonite Cover, SPPA Tailings Cover Project

The Saskatchewan Potash Producers Association (SPPA) had conducted research on covers for potash tailings. Pufahl and Haug (1991) tested potential cover materials for use as potash tailings cover, including three polymer-modified bentonites. These bentonites included Enviroseal™, non-hydrated Volclay™ (Saline Seal 100), and hydrated Volclay™. Samples were mixed with Ottawa sand and insoluble potash tailings. Results indicated that the Enviroseal™ and non-hydrated Volclay™ admixtures both had improved resistance to freeze/thaw.

In 1990, a 400 m^2 sand-bentonite field test cover was constructed on a potash tailings pile at the Potash Company of America, Patience Lake Mine, Saskatchewan (Haug and Wong, 1991). The cover consisted of two 25 cm thick lifts of sand-bentonite placed directly on potash tailings, and covered with a 5 cm thick protective granular layer of screened gravel. A lysimeter was installed beneath the cover.

The sand-bentonite consisting of 8% Enviroseal™ bentonite and 92% sand was mixed at an average water content of 7.5% (optimum water content = 11% at standard Proctor compaction effort). A layer of sand 25 cm thick was first placed on the tailings. Bentonite was then spread dry on the sand. A Bomag pulvi-mixer was used to dry mix the sand and bentonite. Two passes were made in each direction to thoroughly mix the sand and bentonite. Water was then applied and the sand-bentonite was further wet mixed with the pulvi-mixer in two passes in each direction. Care was taken to ensure that the salt tailings were not mixed with the sand and bentonite. The second lift was placed in the same manner as the first.

Monitoring of the test cover had been on-going since 1990. No water had been collected in the lysimeter beneath the cover. Field hydraulic conductivity measurements were taken with a seal single-ring infiltrometer (SSRI) (Table 5.1). Hydraulic conductivity decreased from the time the cover was constructed until 1993, likely because the bentonite required considerable water and time to hydrate (Haug *et al.*, 1994). The initial hydraulic conductivity measurement at the time of construction was relatively high (2 x 10^{-8} m/s) as the field cover was compacted dry of optimum water content. From 1993 to 1994, the hydraulic conductivity had increased by an order of magnitude, which may have been the result of a slight decrease in dry density from the previous year (Haug and Pavier, 1995). Minor surface cracking occurred in the sand-bentonite cover but these are confined to the surface (Haug and Pavier, 1995). However, dissolution of the salt tailings by fresh water runoff at the edge of the cover had made it necessary to periodically repair the downslope portion of the cover (Figure 5.2).

Table 5.1. Field density, water content and hydraulic conductivity measurements for the sand bentonite test cover at the Patience Lake Potash Mine, Saskatchewan (Haug et al., 1994 and 1995).

Date	Density (%)†	Water Content (%)	Hydraulic Conductivity†† (m/s)
Nov./1990	100 to 103	7.5	2×10^{-8}
Aug./1991			3×10^{-10}
Aug./1992	95 to 104	8.5	4×10^{-11}
Sept./1993	96 to 103	9.4	6×10^{-10}
Sept./1994	88 to 99	7.4	7×10^{-9}
1995			

† Density as a percentage of the standard Proctor value of 1.93 Mg/m^3

†† Laboratory established hydraulic conductivity was 4.5×10^{-11} m/s

Fig. 5.2 – Repair Work on the Soil Bentonite Test Cover at the Patience Lake Potash Mine in Saskatchewan (Photo by M. D. Haug).

The most significant problem associated with the sand-bentonite test cover at Patience Lake was erosion along the outside perimeter. Erosion was largely the result of dissolution of tailings, resulting in large holes (up to 10 m deep) which had eroded back and undercut the test cover (Haug *et al.*, 1994). Repairs were necessary and the holes were filled with tailings. Repairs were also made to the perimeter of the cover.

5.2 Polymer Modified Soil

Polymers may be used to modify natural soils to enhance their hydraulic characteristics. Polymers may also be used as grouts and injected into soils to form low hydraulic conductivity barriers. Polymers have been used in the drilling industry to enhance the characteristics of drilling muds.

In MEND Report 6.2, “Polymer-modified clay as impermeable barriers for acid mining tailings”, Zhou *et al.* (1993) described the use of a super absorbent polymer (SAP) for potential use in the abatement of acid mine drainage. The SAP used by Zhou *et al.* (1993) was a polyacrylamide-montmorillonite composite, which hydrates to absorb 680 times its dry weight in fresh water. Pure Wyoming bentonite on the other hand only absorbs 10 times its dry weight in water Zhou *et al.* (1993). Based on its high water-absorption capacity, strong moisture retention capability, and large swelling pressure, SAP could be used to create a seal to reduce infiltration of water and oxygen diffusion.

Zhou *et al.* (1993) tested silica sand and tailings based mixtures mixed with SAP and Wyoming bentonite. The tailings, which were acid generating, came from the Waite Amulet site in Québec. It was found that similar hydraulic (Table 5.2) performance and water retention (Fig. 5.3) properties could be achieved by mixing either bentonite or

SAP with sand.

Table 5.2. Porosity and hydraulic conductivity of sand and tailings based mixtures (Zhou *et al.*, 1993).

Sample No.	Composition (wt%) Sand	Tailings	Bentonite	SAP	Silt	Porosity (%)	Hydraulic Conductivity (m/s)
A	100	—	—	—	—	36.5	5.3 x 10^{-4}
B	99	—	—	1	—	36.3	1.9 x 10^{-8}
C	90	—	10	—	—	33.1	3.9 x 10^{-8}
D	—	100	—	—	—	44.0	1.3 x 10^{-6}
E	—	99	—	1	—	43.9	1.8 x 10^{-6}
F	—	90	10	—	—	44.8	2.8 x 10^{-7}
G	89	—	—	1	10	30.1	<1.0 x 10^{-9}

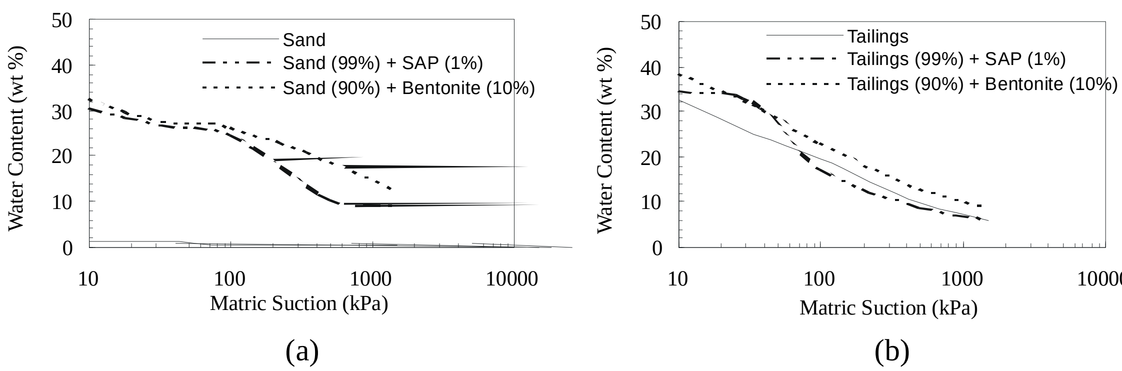

Fig. 5.3. Soil water characteristic curves for a) sand and b) tailings based mixtures (Zhou et al., 1993).

The super absorbent polymer did not perform well when mixed with acidic tailings. Zhou *et al.* (1993) attributed this to the poisoning of SAP by divalent cations (in this case, Ca^{2+} supplied by gypsum) which leached out of the tailings. Divalent cations can cross link the polymer branches, making the polymer more "tight", thus reducing the hydration of the polymer. Even for cover applications over acidic tailings where SAP is mixed with sand rather than acidic tailings, the cover design must ensure that cation diffusion from the tailings into the SAP cover layer does not occur.

5.2.1 Costs

The cost of SAP ($1.17 Cdn/kg) is higher than Wyoming bentonite ($0.085 Cdn/kg) (Zhou *et al.*, 1993). However, for equal performance in water retention and as a hydraulic barrier, Zhou *et al.* (1993) estimated approximately one tenth as much SAP is required compared to Wyoming bentonite. This translates into a substantial saving in transportation cost. Assuming a 30 cm thick barrier layer, 1.0 wt % SAP, and 1.5 Mg/m^3 bulk density, the cost of SAP will be $5.26 Cdn/m^2 of cover.

5.3 Polyacrylamide Enhanced Marginal Quality Bentonite-Sand Mixture

Haug and Boldt-Leppin (1994) investigated the use of polymers to enhance performance of marginal quality bentonite-sand mixture. The polymer used was an anionic polyacrylamide, ALCOMER 228, which was described by the manufacturer as being especially effective when inorganic contaminants were encountered (Allied Colloids, 1992). Haug and Boldt-Leppin (1994) conducted a rheological and triaxial permeability laboratory study, to determine if the low hydraulic conductivity characteristic of a marginal quality bentonite could be improved with the addition of a small quantity of polymer. The addition of polymer concentrations to marginal quality bentonite-sand mixtures in excess of 0.05% by air-dried weight of bentonite, reduced the hydraulic conductivity to near 1 x 10^{-11} m/s. Mixtures of high quality (Wyoming) bentonite and sand also gave resulting hydraulic conductivity measurements equal to 1 x 10^{-11} m/s. Aging of the polymer for one year was not found to result in any changes.

5.4 Polymer Surfactants

Moskalyk (1995) investigated the use of surfactants to minimize the generation of AMD. Surfactants could be mixed with mine waste to provide a thin protective coating. Polymeric surfactants (Enviroseal™ and IPMC polyurethane sealer) showed the most promise among various surfactants Moskalyk (1995) tested, which included cementitious mixtures. Enviroseal ™ is a polymerized bentonite, which was applied as an emulsion to the waste rock. Both the Enviroseal™ and IPMC polyurethane sealer reduced AMD in leaching column tests, as well as providing acceptable freeze/thaw resistance throughout one winter of exposure in the Sudbury region. Long-term performance of these surfactants is not known, as the leaching tests ran for only 22 weeks.

The use of surfactants premixed into tailings and mine waste rock would only be applicable to new operations, as the surfactants would likely have to be applied in an on-going process. Further tests are required to establish the application method and optimum quantity of sealant to ensure fixation of AMD to suit regulatory agencies (Moskalyk, 1995). The quantity of surfactant required would have a large bearing on cost and may render this AMD prevention scheme non-feasible.

The use of surfactants, which are bactericides to prevent or delay acidic drainage from waste rocks, has also been reported (MEND 2.37.2). Bacteria have been known to enhance sulphide oxidation at low pH (below 5). The surfactants probably work by altering the protective greasy coating that protects the bacteria from the acidic environment, and/or disrupting the contact between the bacteria and the mineral surface. Limited laboratory and field studies conducted to-date have shown that bactericides can inhibit the onset of ARD in coal wastes at reasonably low cost. However, the use of this technique on metal mine waste has not been positively proven.

6.0 Mine Wastes as Potential Cover Material

The use of mine waste, such as tailings or waste rock, as cover materials for AMD applications could translate into a substantial cost savings as well as contribute to recycling of the waste products. Tailings had been used for cover material (Gerencher *et al.*, 1991; MEND 2.34.1) and as an aggregate for cementitious covers (see section 3). As well, tailings had been geopolymerized to render them suitable for certain applications (see section 4) (D. Comrie Consulting Ltd., 1988). Tailings could also be used in conjunction with waste rock as part of a store and release cover system. The use of waste rock as a cover material had been reported by Price and Tremblay (1993), O'Kane *et al*.(2000) and Durham *et al* (2000).

6.1 Tailings

The use of multi-layered cover systems incorporating capillary barriers and capillary breaks for covering acid generating wastes have been reported by Rasmussen and Eriksson (1987), Nicholson *et al.* (1989, 1990), Barbour (1990), Aubertin and Chapuis (1991), Yanful *et al.* (1993c, 1994), Vanapalli *et al* (1997), Aubertin *et al* (1997), Bussiere *et al* (1997) and Ricard *et al* (1997). Fig. 6.1 illustrates the configuration of a multi-layered cover system. Capillary barriers would consist of fine-grained material with a low saturated hydraulic conductivity, whereas the capillary breaks would consist of coarse material with a low air entry value. At the P. T. Kelian Equatorial mining gold mine in East Kalimantan, the dry cover concept was used without an upper capillary break layer to reduce evaporation since precipitation exceeds evaporation in the equatorial climate (Firth and Linden, 1997).

The purpose of the fine-grained layer in 'dry' cover systems was to restrict oxygen diffusion (saturation must be maintained) as well as to restrict infiltration. Clean tailings have been recognized as a material that deserves further investigation for use as the fine-grained water retention layer in multi-layer 'dry' cover systems. Nicholson *et al.* (1989), Ricard *et al* (1997), Bussiere *et al* (1997) and Hanton-Fong *et al* (1997) have suggested the use of processed tailings with low sulphide content and milled sulphide free waste rock as potential cover materials. The non-capillary layers could be constructed from the coarse fraction of tailings, separated from hydrocyclones (Aubertin *et al.*, 1993, 1994) or from prepared waste rock.

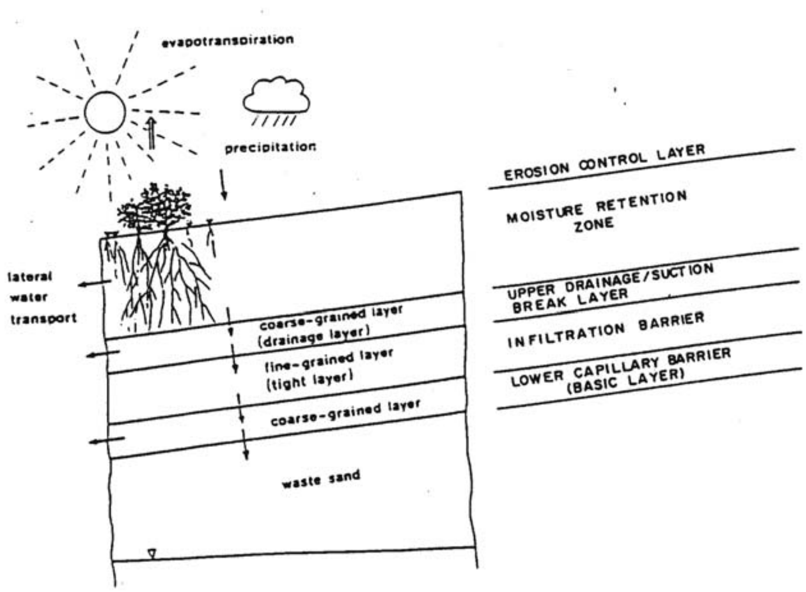

Fig. 6.1. Multi-layered soil cover for reactive tailings (after Rasmusson and Eriksson, 1987).

Aubertin *et al.* (1995), also MEND 2.22.2a and 2.22.2b, tested tailings, classified as low plastic silt (ML), from three sites. These tailings were reconstituted and compacted to various void ratios (e ranging from 0.50 to 0.90). Measured hydraulic conductivities ranged from 5×10^{-8} to 2×10^{-6} m/s, which was high for an infiltration barrier. However, as Aubertin *et al.* (1994) stated, the hydraulic conductivity value was not the overriding issue, rather it is the ability to limit the flow of oxygen that is generally recognized as the measure of effectiveness of covers to control AMD production. Aubertin *et al.* (1995) determined soil-water characteristic curves for the above tailings using the axis-translation technique. The air entry values (AEV) were approximately 20 kPa (for e ranging from 0.67 to 0.93). An important consideration would be to ensure that the AEV is significantly lower in the capillary break of the cover material. Machibroda *et al.* (1993) tested tailings, processed with the thickened discharge method (Ritcey, 1989), which had an AEV of 50 kPa suction. The water retention properties of tailings used for capillary barriers may be improved with the addition of bentonite.

Aubertin *et al* (1997) and MEND 2.22.2c described the construction and instrumentation of experimental cells to evaluate multi-layer cover systems built with clean tailings on a site near Val d'Or, Quebec. Six test cells were installed using five different cover systems and the sixth was used as a control. Each cover system has three layers consisting of a sand layer on top and on the bottom sandwiching a fine-grained layer. The fine-grained layer was constructed of clean tailings with different thicknesses (three covers), of a natural silty soil (one cover), and of a mixture of bentonite and clean tailings (one cover). Underlying each cover system were sulphidic tailings from the adjacent Manitou-Barvue site. At the time of reporting, the multi-layer cover systems were found to behave in accordance with the capillary barrier concept; that is the layered systems create capillary barrier effects that reduce production of AMD mainly by reducing oxygen flow through the cover.

A full scale test involving a dry cover system constructed using old acid consuming gold tailings was reported by Ricard *et al* (1997), MEND 2.22.4a and MEND 2.22.4b. An engineered multi-layer cover system was one of three closure alternatives considered, and was the recommended option for the closure plan at the Les Terrain Auriferes site near Malartic and Val d'Or in Quebec.

Tailings from a previous mining operation at the old Malartic Goldfields were from a non-sulphide ore. The non-sulphidic tailings were acid consuming and contain more than 10% calcite. The old tailings are about 5 m thick and cover 100 ha. Laboratory tests conducted on the old tailings showed that it has an air-entry value of 2.0 to 2.5 m (or 2.0 to 2.5 kPa) and has an oxygen diffusion coefficient of less than 1 x 10^{-8} m^2/s at degrees of saturation higher than 85%. The full scale test cover covers an area of about 60 ha and is consisted of 0.3 m of sand and gravel, over 0.8 m of old tailings underlain by a 0.5 m sand layer acting as a capillary break between the cover and the newer highly sulphidic tailings. Monitoring during the period 1996 to 1998 showed that the cover remained practically saturated on the slopes as well as on top. The average tailings oxidation flux was reduced by 95 % over the three year observation period. Some areas showed local water contents that were lower than the design objective, particularly on the outer slopes. The execution of the project once again demonstrated the feasibility of the capillary barrier concept for cover systems. It was reported that the construction of the cover on high water content tailings was difficult and was achievable only by construction during the winter months.

At the INCO Ltd. Copper Cliff mine, a research program was conducted to evaluate the effectiveness of using low sulphur tailings in the prevention of acid mine drainage (Hanton-Fong *et al*, 1997; and MEND 2.45.2). The field study involved the construction of three 10 m x 15 m field lysimeters containing 1.) low sulphur tailings (0.35 wt. % S), 2.) main tailings (0.98 wt. % S), and 3.) total tailings (2.3 wt. % S), respectively. Semi-annual monitoring was performed to determine porewater composition, pore gas composition and bulk physical properties of the tailings. Geochemical modeling was also performed. It was found that pore-gas oxygen levels were depleted within the upper 20 cm of the main tailings and total tailings lysimeters. Complete gas-phase oxygen depletion was not observed in the low sulphur tailings lysimeters. Oxidation in the main and total tailings lysimeters resulted in the development of acidic conditions. Porewater pH in the low sulphur tailings, however, remained near neutral. Also, substantial increases in the concentrations of dissolved constituents was found in the porewater of the main and total tailings lysimeters within the first year, but has yet to be observed in the low sulphur tailings lysimeter. The research found that low sulphur tailings is relatively inert to oxidation, but does occur.

Bussiere et al (1997) conducted column tests to assess the performance of cover materials derived from desulphurized tailings. Three-layer cover systems were simulated in the column tests. Three desulphurized tailings containing different residual concentrations were investigated. Seven column tests were conducted: one containing a single layer of high sulphide tailings only, three containing a single layer of each of the three desulphurized tailings, and three containing a three-layer cover over a layer of high sulphide tailings. In the three column tests containing a three-layer cover, each of the three desulphurized tailings containing different residual sulphide concentrations were used as the fine-grained, water retaining layer.

The column tests by Bussiere et al (1997) showed that oxidation rates in the high sulphide tailings increased to maximum values three months after the tests were commenced. The oxidation rates for the desulphurized tailings cover correlated with the residual sulphide content and oxygen consumption rates were lower than the high sulphide tailings by as much as a factor of 300. The tailings that were covered with the lowest sulphide content cover exhibited rates that were one-twentieth of the uncovered tailings.

McGregor *et al* (1997) reported on the development of a self-sealing/self-healing cover system. The self-sealing/self-healing barrier is based on micro-scale properties of diffusion and chemical reactions. The process begins with the reactions of dissolved compounds to form precipitate at the interface between two parent materials. Precipitation reactions at the interface result in a decrease in the aqueous concentrations of the reactive components, creating a concentration gradient. This gradient leads to diffusion of additional reagents to the interface, resulting in further precipitation until the pores between the parent materials are completely filled. If the barrier is damaged the remaining parent materials will once again react at the new interface and restart the self-healing process, leading to a new barrier between the parent materials. A variety of different barrier formulations were developed, including using mining wastes, such as tailings. The mining waste barrier is formed when a solution containing iron, sulphate and other dissolved constituents comes into contact with a mixture of materials.

A field trial of the self-sealing/self-healing barrier was conducted at the Falconbridge Ltd’s East Mine tailings impoundment. The tailings were deposited in 1985 and range in particle size from silty sand to sandy silt with a small portion of clay (<10 wt. %). The tailings are underlain by glacial outwash consisting of sand and gravel as well as a thin layer of peat on the east side. The test barrier was installed in July 1985 and involved the following four stages:

1.) Excavation of the overlying oxidized tailings;
2.) Application of lower parent material;
3.) Application of high Fe-S mill washing materials (upper parent material);
4.) Application of overlying oxidized tailings as cover material.

The lower parent material contains 10 wt. % reactive materials. The upper parent material was consisted of mill-washings and has high Fe-S content. The lower and upper parent materials reacted at the interface to form the self-sealing/self-healing barrier. First year preliminary results indicate that the barrier was performing well as a hydraulic and diffusive barrier.

A self-healing test was conducted in the laboratory using the same parent materials used in the field trial. The barrier was mechanically fractured and then allowed to heal. Prior to and after the fracturing event the hydraulic conductivity of the barrier sample was measured continuously under a constant confining pressure. The results show that the barrier had a hydraulic conductivity of 8.8×10^{-13} m/s prior to fracturing. Immediately after fracturing, the hydraulic conductivity increased to 8.9×10^{-7} m/s which is the same as the hydraulic conductivity of the parent material. Within 0.05 pore volumes of flow the hydraulic conductivity of the barrier had decreased to 1.4 x 10^{-9} m/s. After 0.63 pore volumes of flow the hydraulic conductivity had decreased to its original value prior to fracturing.

The installation of the self-sealing/self-healing barrier was estimated to cost between $30,000 to $50,000 Cdn per ha.

6.2 Waste Rock

The use of waste rock as the sole constituent or as part of a cover system had started to be investigated in the past 10 years. This relatively new option for a cover has obvious advantage.

Acid generating tailings at Minnova's Millenbach mine in Québec were covered in 1990 and 1991. A 1- to 2-m thick layer of waste rock was placed directly on the 1-hectare tailings surface (Price and Tremblay, 1993). The layer of waste rock was covered with a 30 cm layer of coarse sand, a 60 cm layer of clay, a 30 cm layer of fine sand, and topped with 20 cm of revegetated organic topsoil. The purpose of the mine waste rock and sand layers were to form capillary breaks above and below the clay barrier. During the first year of operation, the oxygen content beneath the cover had decreased to between 7 to 8% and water quality had improved by approximately 50%.

BHP Iron Ore initiated a test program at their Mt. Whaleback operation in Newman, Western Australia to develop a decommissioning plan for the waste rock material (O'Kane et al, 2000). The field performance of cover systems constructed using run-of-mine waste material is currently being investigated. Test plots have been established on both horizontal and sloped waste rock surfaces. The store and release cover system acted as a buffer for the rainfall entering the underlying waste material and helped reduce net percolation. Two horizontal test plots (Test Plot No. 1 and 2), and one sloped test plot (Test Plot number 3) were installed at the site. A 2 m depth of run-of mine material was placed at Test Plot number 1 and at Test Plot number3. A 4 m depth of run-of-mine material was placed at Test Plot number 2. Evidence of infiltration to a depth of 2 m below the horizontal surface at Test Plots No.1 and 2 was obtained during the August 1997 through June 1999 monitoring period. The infiltration did not appear to have advanced to a depth of 2 m on the sloped test plot (Test Plot No. 3). The volume of water retained within the horizontal cover systems at Test Plots No. 1 and 2 had increased, in large part due to significant rainfall experienced (2.5 times the normal) in the 1998-1999 monitoring period. The data collected from the monitoring to date indicated there is potential for success in using the “store and release “ type cover system at the BHP Iron Ore Mount Whaleback site.

Another site in Australia had also involved waste rock in the cover design. At Kidston Gold Mine in Queensland, oxidized waste rock was used for two different types of low water flux test covers. The covers were installed on the south dump at the mine in December 1995. One test cover consisted of 0.5 m of compacted, oxidized waste rock material overlain with uncompacted oxidized waste rock material dumped to produce a hummocky topography. The depth from the top of the hummock to the trough was 1.5 m. The second cover consisted of a single 2.5 m layer of uncompacted oxidized waste rock similar to that used in the first cover design. Both covers were vegetated with native shrubs in early 1996. Field monitoring at both covers for the 1996 through 1999 time period shows that both covers are performing satisfactorily. The vegetation planted in early 1996 has thinned, and in some areas the waste rock is completely bare, resulting in lower transpiration rates. The vegetation was fertilized only once. The native woody plants and shrubs are becoming more dominant. It is believed that once this vegetation matures further, the wetting trend currently observed in the cover system, would reverse. Both cover systems have shown that the infiltration rates have been reduced significantly and in fact are near zero after 3 years of monitoring. However, computer modelling had indicated that the cover system with the compacted layer would perform in a more suitable manner during high precipitation years.

7.0 Wax Barriers

Wax has been suggested as a barrier material for isolating waste (Anon., 1993; Senes Consultants, 1994; Wilson, 1995). Waxes have varying chemical composition, some occur naturally, while others are synthesized.

Montan wax consists of a mixture of free acids, ranging in molecular weight, esters and alcohols (Abraham, 1960). Montan wax is found naturally occurring in brown and lignite coals. It is derived from the wax formed on plants to protect them from dryness. Since the wax does not decompose over time, it becomes concentrated in coal as other components decay (Wilson, 1995). Montan wax is hard, non-toxic, and has a high melting temperature.

Montan wax may be applied as an emulsified grout for barrier applications, in which a barrier is formed with a hydraulic conductivity as low as 1×10^{-10} m/s (Wilson, 1995). Montan wax may also be applied for surface capping of landfills (Anon., 1993). Montan wax is considered to have good chemical resistance, as well as provide a flexible barrier.

The estimated cost of a thin wax cover is in the neighbourhood of \$6 to \$8/m^2 (Senes Consultants, 1994).

8.0 Conclusions

The non-traditional cover materials case studies reported in this report focused primarily on the hydraulic conductivity of the cover material itself. This focus on low hydraulic conductivity is based on the concept that for most applications covers are essentially liners placed over waste to minimize infiltration and the resulting leachate. This may be adequate for landfill and similar waste applications; however, AMD covers require a higher level of sophistication. AMD covers are required to eliminate oxygen transfer (ingress) to the underlying waste rock as well as minimize infiltration. Furthermore, due to the long-term nature of AMD it is desirable that these cover systems be resistant to acid degradation and provide a medium for the development of sustainable vegetation.

Since the ingredients for acid generation are water and oxygen, any material that inhibits infiltration of water and/or diffusion of oxygen is a potential candidate for cover materials in AMD mitigation. Since most of the non-traditional cover materials reviewed had low hydraulic conductivity, these materials would have potential to be as used cover materials at least to function as a barrier to water infiltration. Some of the reviewed materials may be effective as barrier to oxygen transfer as well. Asphalt, wax and spray-on membrane may be effective to some extent in limiting oxygen ingress into the wastes. Cefill, geosynthetic clay liners and soil modified soil barriers can be effective barriers to oxygen transfer if they are maintained in a saturated state. Degradation could be an important factor with some materials such as asphalts and waxes if they are exposed for extended period of time to the environment. Slope stability could be an issue with materials such as geosynthetic clay liners and geomembranes.

The use of multi-layer cover systems involving a combination of capillary barrier/s and capillary break/s acting as a ‘store and release’ system have been found to be effective for AMD applications. In the multi-layered systems, the capillary barrier limits the ingress of oxygen and the capillary break limits infiltration. Capillary barriers are usually constructed out of fine-grained materials and capillary breaks of coarse materials. Tailings are usually fine-grained and waste rocks are usually coarse-grained. Although tailings seldom contain clay minerals capable of absorbing water and providing on their own a barrier to oxygen ingress, it may be possible to modify the tailings and waste rock with bentonite, fly ash, or other suitable material. Thus the waste materials themselves, i.e., tailings and waste rocks, may be used as components in the construction of the cover systems. The incorporation of mine waste in cover systems could potentially result in savings as well as contribute to the recycling of the waste rock products. However, the use of waste rock involves a number of challenging construction issues.

The use of bentonite modified soil/sand appears to provide an excellent barrier to water seepage and good barrier to oxygen ingress. This is because the bentonite swells in the presence of water to restrict flow. The high affinity of bentonite for water means that it also resists drying, providing a good barrier to oxygen ingress.

Most of the non-traditional cover materials reviewed have the potential to be included in a multi-layered store and release cover systems to provide some redundancy. For instance, a low permeability membrane of asphalt or geomembrane or cementitous material could be placed immediately over the waste materials. This could then be overlay by a dry cover system consisting of a fine textured soil layer sandwiched between two coarse textured soil layers.

9.0 Recommendations

The utilization of mine waste products as materials for the construction of covers for AMD applications is an attractive option which should be further researched. The use of bentonite modified sand/waste barriers also appears to have significant potential. Mine wastes have already been used in a few AMD mitigation projects. The track record of using mine wastes, however, is not well established at this time and research should continue. Bentonite modified sand has more of a track record, however, additional research is required on these materials as well. Further investigation should be conducted on using tailings and waste rocks that are modified with small amount of other non-traditional cover materials (i.e., bentonite, fly ash, etc.) to enhance their behavior so that they would be suitable for use as components of “store and release” cover systems. The effects of wet-dry cycles and freeze/thaw cycles on the long-term behavior of these material used as components of cover systems should be studied. Tests should also be conducted on the potential of leaching of the additives (i.e., other non traditional cover materials, such as fly ash, bentonite, etc.) that were used to enhance the behavior of the mine waste products.

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