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Technical Aspects Related to the Design and Construction of Engineered Containment Barriers for Environmental Remediation



TECHNICAL ASPECTS RELATED TO THE DESIGN AND CONSTRUCTION OF ENGINEERED CONTAINMENT BARRIERS FOR ENVIRONMENTAL REMEDIATION The following States are Members of the International Atomic Energy Agency:

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INTERNATIONAL ATOMIC ENERGY AGENCY VIENNA, 2023

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FOREWORD

The IAEA attaches great importance to the dissemination of information that can assist Member States with the development, implementation, maintenance and continuous improvement of systems, programmes and activities that support the nuclear fuel cycle and nuclear applications. This includes managing the legacy of past practices and accidents that involves different aspects of environmental remediation. In this regard, the IAEA envisions that Member States will eventually have in place the proper infrastructure and technologies to resolve all related issues in a timely, safe and cost effective manner. Sharing best practices and providing the technical basis for the adoption of appropriate options to be used in the remediation of contaminated sites is therefore of crucial importance.

Barrier systems are among the most widely used technologies for remediating contaminated sites. A broad range of designs have been developed to address specific needs. However, without a proper understanding of the basic principles governing the adoption of each option, the desired results are unlikely to be achieved or the desired performance may not be sustainable over the intended period.

This publication introduces technical aspects related to the design and construction of engineered containment barriers for low level radioactive waste that is generated in the remediation of contaminated sites. The types of barrier described include basal (bottom) liners, final covers, in situ vertical barriers and in situ permeable reactive barriers. Determining which type of barrier is needed depends on the specific site and to a large extent on whether a prescriptive or a performance based approach is required by the regulatory system. Complementing the theoretical discussions, case studies are provided to enhance the understanding of the relevant practical aspects.

The IAEA wishes to thank the contributors to the work presented in this publication. The IAEA officer responsible for this publication was H. Monken-Fernandes of the Division of Nuclear Fuel Cycle and Waste Technology.

EDITORIAL NOTE

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1. INTRODUCTION

1.1. BACKGROUND

Environmental remediation activities may generate significant quantities of waste that require safe handling. In some cases, the waste requires disposal in engineered structures (on- or off-site) that will need to perform according to predefined safety criteria for long periods of time. These facilities are expected to ensure containment of the waste, prevent contact of the waste with the biosphere, minimize or eliminate the release of mobile contaminants to the surrounding environment, and provide acceptable land use in accordance with an agreed end state. The release of aqueous and gaseous contaminants to air and groundwater is often a primary concern.

1.2. OBJECTIVE

The objective of this publication is to introduce the technical aspects related to the design and construction of engineered containment barriers for low level radioactive waste. The types of barriers described include basal (bottom) liners, final covers, in situ vertical barriers and in situ permeable reactive barriers (PRBs). Determining which type of barrier is needed depends on the specific site and to a large extent on whether a prescriptive or a performance based approach is desired or required by the regulatory system.

Guidance and recommendations provided here in relation to identified good practices represent expert opinion but are not made on the basis of a consensus of all Member States.

1.3. SCOPE

This publication addresses the technical aspects that govern the design of engineered barriers for use in environmental remediation projects. The publication provides basic concepts that should be observed when such barriers are considered. It encompasses containment systems that include (i) cover to restrict the infiltration of precipitation and release of gaseous contaminants; (ii) basal liners to contain leachate to preclude groundwater contamination; and (iii) vertical barriers and vertical permeable barriers to impede the lateral movement of water and contaminants, respectively.

1.4. STRUCTURE

This publication comprises six sections, with this introduction being the first section. The second section provides a brief discussion on the adoption of prescriptive criteria versus those based on performance to be used in the design and construction of the engineered barriers covered in this publication. The third section describes the primary considerations that need to be taken into account in the use of bottom waste containment barriers or liners, starting with the functions that a liner will perform. The fourth section describes the use of final covers and addresses three main types of structure: resistive barrier covers, evapotranspiration covers and hybrid covers. The fifth section discusses the application of vertical containment barriers. The sixth section presents the technical aspects related to the use of PRBs and details considerations for the use of continuous PRBs, funnel and gate systems, multiple reactive zones in series, and other PRB configurations. Sections 3–6 also present case studies that aim to provide a deeper insight into the technical aspects discussed.

2. PRESCRIPTIVE VERSUS PERFORMANCE BASED APPROACH

In many countries, the disposal of solid waste is permitted through a regulatory system that is based on a prescriptive approach. This approach prescribes specific types of engineered barriers for various applications. For example, the approach might prescribe the required maximum hydraulic conductivity of a clay liner, or the type of geosynthetic material required in a particular section of the liner. Prescriptive design regulations often allow an application with little modification for the environmental conditions at a specific site.

The technical basis for such 'regulatory engineering' consists of known material properties of essential components (e.g. a layer of low permeability clay alone or in combination with a geomembrane in a final cover), combined with installation methods and quality assurance (QA) procedures to ensure that the incidence of material and construction defects remains within an acceptable limit. With this 'materials and methods' approach, performance is generally assumed and field performance monitoring is typically minimal. The advantage of this approach is the relative ease of implementation and regulation. The approach is also favoured by owners and operators of facilities as responsibility

for poor performance is largely removed if compliance with the prescribed criteria is achieved.

In contrast, designs based on performance emphasize a design process that starts with a clear definition of the required performance criteria. A conceptual design is selected based on the site specific environmental, waste, economic and social factors listed in this publication. The final design is based on detailed site characterization, field studies and numerical simulations to ensure that predicted performance meets the required performance criteria. This emphasis on the design process (rather than on the construction materials and methods) suggests that the process itself may be made prescriptive in that regulators might require consideration of a list of risk factors including (but not limited to) those mentioned in this publication. The design engineer is then obligated to consider each factor. The performance based design process is usually more expensive than the prescriptive design approach, but significant cost savings in construction and long term maintenance can be achieved.

The appropriate choice of barrier design requires careful consideration of the varied metrics for performance, cost, regulatory requirements and stakeholder acceptance. Simple containment systems could suffice for some wastes, whereas other wastes might pose such a significant threat to human health and the environment that sophisticated and/or redundant containment systems are required. The determination of required performance can be challenging; multiple site-specific factors typically need to be considered and no containment design can guarantee acceptable performance over the entire lifetime of the hazardous waste. For this reason, significant performance monitoring is typically required.

Factors important to barrier design include the following:

- Containment philosophy (i.e. minimized release or controlled release);
- Design philosophy (i.e. prescriptive or performance based design);
- Climate (e.g. annual and seasonal precipitation, rain versus snow, freezing temperatures);
- Type and longevity of the waste;
- Waste packaging;
- Depth to groundwater;
- Capacity of underlying geologic layers to attenuate the movement of contaminants that might escape the engineered containment system;
- Proximity to existing or planned uses of groundwater resources;
- Proximity to sensitive environmental niches or other receptors (e.g. critical habitats or endangered species);
- Acceptable release of gaseous contaminants;
- Required lifetime or maintenance period;
- Cost (e.g. of design, construction, maintenance).

3. WASTE CONTAINMENT LINERS

3.1. FUNCTION OF A LINER

A bottom or basal liner is used primarily to prevent the movement of contaminants derived from waste, typically in a liquid form known as leachate, into the surrounding environment (which is usually the subsurface beneath a waste disposal site). Groundwater resources can be protected and an accumulation of contaminants in the subsurface is prevented. As contaminants of concern are conveyed in the liquid, the liner needs to be as impermeable as possible.

The majority of existing tailings storage facilities (TSFs) retaining tailings from operations such as the processing of base metal, mineral sands, diamond and coal are likely to be unlined. This situation is not likely to be an acceptable option for new TSFs, especially those retaining radioactive tailings. Similarly, waste rock dumps that potentially contain radioactive waste, even at very low levels, are likely to require a lining system. In exceptional circumstances, the local hydrogeological conditions may be such that a lining system is deemed not to be required (e.g. in the case where many tens of metres of naturally occurring, very low permeability clay are located immediately beneath the planned TSF and the groundwater table is extremely deep). However, any new unlined TSF or waste rock dump containing radioactive material would need to be justified for each site and an exhaustive risk evaluation should be required before proceeding with such a design.

A decision should be taken at the outset as to whether any seepage loss through the lining system is allowable or not. Some liners are designed based on an 'acceptable seepage' approach, whereby a small seepage rate (perhaps expressed as a percentage of the annual precipitation or a defined flux) is acceptable. This approach has sometimes been justified on account of local hydrogeological conditions that provide additional retardation, dilution or attenuation effects, but it is unlikely to be acceptable for radioactive waste. The likely design criterion will specify zero seepage through the liner, which will affect the choice of lining system.

A lining system usually consists of several components. Some of these components will, by necessity, serve as barriers to seepage that are relatively impermeable, whereas others, such as drainage layers or drainage features, might be relatively permeable.

The main design objective for low permeability layers that are part of a lining system is to minimize or even prevent the hydraulically driven flow of water or contaminated liquid, also commonly referred to as advection. A secondary consideration is the diffusion of contaminants across a low permeability layer.

Although advection is almost always the primary consideration, either of these processes could result in an unacceptable rate of discharge through the lining system to the environment.

3.1.1. Advection

Advection is the movement of a liquid in response to a hydraulic gradient. When considering a lining system, advection typically refers to the flow of water or contaminated liquid (e.g. leachate) through the pores of the materials that comprise the various liner components (e.g. through the pores in a compacted clay liner [CCL] or a geosynthetic clay liner [GCL]). Advection can also occur through defects such as discrete holes or puncture holes in a geomembrane (a polymer sheet that serves as a barrier).

The rate of advection through a liner is governed by the difference in total hydraulic head across the liner and the permeability of the liner as reflected by the hydraulic conductivity of the liner material (e.g. the hydraulic conductivity of a CCL). The flow rate (Q), which is a volumetric flow rate (expressed in units of volume or length cubed per unit time, L^3T^{-1} , e.g. $m^3/s)^1$, is calculated using the empirical equation commonly known as Darcy's law (Eq. 1):

$$Q=KiA$$
 (1)

where

K is the hydraulic conductivity, LT^{-1} , where *L* is length and *T* is time; *i* is the hydraulic gradient, LL^{-1} :

and A is the cross-sectional area of flow (L^2) , which for a liner is usually considered as a unit area (i.e. per 1 m² when considered in plan view).

The hydraulic gradient can be represented by $i = -\Delta h/\Delta l$ where Δh is the head loss between two points (*L*), usually two different elevations such as the top and bottom of a lining system, and Δl is the macroscopic length or distance between the same two points (*L*). Thus, the hydraulic gradient is dimensionless.

In general, hydraulic conductivity decreases as the particle size of a soil decreases. This is why clay is used for compacted liners. For a clayey material, as the type of clay becomes more 'active', the amount of a higher activity clay

¹ In general, units in this publication will be presented as system independent, where L = unit of length, T = unit of time and M = unit of mass. Units from the International System of Units may also be used in specific cases to indicate order of magnitude values.

TABLE 1. VARIABILITY IN HYDRAULIC CONDUCTIVITY (K) BY SOIL TYPE

Soil	<i>K</i> (m/s)	Comment
Gravel	>10 ⁻² -10 ⁻³	Values assume 'clean' gravels (little or no fines)
Sand	$10^{-3} - 10^{-6}$	Variation based on gradation (e.g. coarse sand versus fine sand)
Silt	$10^{-4} - 10^{-8}$	Variation owing to particle size and mineralogy of silt particles
Clay	$10^{-8} - 10^{-12}$	Variation owing to particle size and mineralogy of clay particles

Note: 'Fines' is defined as the percentage by mass of particles less than $75 \,\mu m$.

mineral, such as montmorillonite, will increase. As the soil becomes better graded, it will present a greater distribution of particle sizes. Typical values of hydraulic conductivity for different soil types and the potential variability for a particular soil are given in Table 1.

From Eq. (1), the ratio Q/A is defined as the liquid flux or specific discharge q(=Q/A), with net units of velocity (LT^{-1}) . However, the actual rate of movement of a water molecule would be faster than given by q, with the true flow rate being equal to the seepage velocity, v = q/n, where n is the porosity of the porous medium defined as the ratio of the volume of voids to total volume (L^3L^{-3}) . For example, if the porosity of the clay is 0.4, the true liquid flow rate through the clay would be 2.5 times faster than the specific discharge. This definition for velocity assumes that all pores in the porous medium conduct liquid flow or seepage. In some cases, dead end or non-conductive pores may exist such that only a fraction of the pores conduct seepage. In this case, the pore conducting volume is defined as the effective porosity, n_e (<n), and velocity is defined as q/n_e .

3.1.2. Diffusion

Diffusion is a process whereby random molecular motions result in the movement of a chemical species (e.g. ion, radionuclide) from a region of higher chemical potential to a region of lower chemical potential [1]. Since the chemical potential is related to chemical concentration, diffusion is more commonly described as the process whereby chemical species migrate in response to a

concentration gradient, from higher concentration to lower concentration. Thus, if there is no concentration gradient, diffusion will not occur. Although diffusion is usually much slower than advection, it can still result in the movement of specific chemical species in the absence of advection. Thus, the objective in the design of liners is to minimize the head of the leachate and consequently the advection in such a way that diffusion becomes the dominant process for contaminant transport. However, because diffusion can still result in contaminant fluxes that can be detrimental, minimizing advection, although necessary, may not be a sufficient condition to ensure an acceptable level of containment [1].

For one dimensional diffusion through a porous medium, the governing equation is Fick's first law, which may be written as Eq. (2):

$$J_{\rm d} = nDi_{\rm c} \tag{2}$$

where

- $J_{\rm d}$ is the diffusive mass flux of the species of interest (e.g. a radionuclide) per unit cross-sectional area perpendicular to the direction of diffusion, $ML^{-2}T^{-1}$;
- *n* is the total porosity as previously defined;
- *D* is the effective diffusion coefficient, L^2T^{-1} ;

and i_c is the mass based concentration gradient (ML^{-4}) in the direction of diffusion.

For a more detailed discussion of the diffusion of radionuclides through engineered barriers, see Ref. [2].

3.2. TYPES OF LINER SYSTEM

3.2.1. Single liners

Although desirable, no porous material is completely impermeable. However, an engineered liner system can reduce advection perpendicular to the plane of the liner component (i.e. 'through' a liner) to very low values. Single liners may comprise natural materials such as clay, or synthetic materials such as the variety of polymeric materials that are available today. The simplest lining systems consist of a low hydraulic conductivity layer such as a CCL, a GCL, a geomembrane liner (GML) or a compacted sand–bentonite liner (SBL) overlain by a collection layer (Fig. 1). The liner minimizes the migration of liquid from the tailings into the soil beneath the TSF. The collection layer has two functions:



FIG. 1. Illustration of a single liner system.

(i) it collects the contaminated liquid percolating through the waste (tailings), commonly referred to as leachate, and drains this liquid to a collection sump; (ii) it serves to minimize the buildup of liquid on the underlying liner to minimize the seepage of the leachate through the liner. However, the drainage through the collection layer is usually by gravity and some buildup of liquid on the liner is therefore necessary before flow to the sump begins. Under this static head condition, seepage can potentially occur through the liner (as can diffusion, although diffusion generally occurs at a much lower rate than advection), either through the pores of a CCL, GCL or SBL, or through defects.

3.2.1.1. Compacted clay liner

The use of compacted clay to achieve a liner with low hydraulic conductivity is attractive when suitable clay is available close to the site and at a relatively low cost. When dealing with natural materials with low hydraulic conductivity, the tendency is to use the generic term 'clay'. The word 'clay' refers both to texture, particle sizes smaller than 2 µm, and to material containing one or more clay minerals, usually with traces of other minerals, such as metal oxides, and/or other constituents, such as organic matter. When found naturally, clays typically occur together with less weathered natural (and inert) materials such as silts and sands that usually comprise silica in the form of quartz (SiO₂). The clay minerals of a given clay material have a major influence on the engineering characteristics of that particular clay and consequently on its suitability for use as a barrier to seepage. Aluminosilicate clay minerals common to temperate regions occur as laminated sheets with charged surfaces and associated adsorbed ions. Water attracted to these charged sites results in swelling and closure of the large pores in the soil matrix with the consequent effect of reducing hydraulic conductivity. Soils that are appropriate for use as clay barriers usually classify as either low plasticity clay (with the symbol CL) or high plasticity clay (with the symbol CH)

in the Unified Soil Classification System (USCS) [3] and span the clay range of the plasticity chart defined in that standard.

A CCL is prepared by conditioning a suitable natural clayey material to the desired water content, and then compacting this clay to the desired density to produce the required hydraulic conductivity that may be measured by laboratory testing (e.g. ASTM D5084 [4], ASTM D2434 [5] or equivalent). The material used does not need to be 100% clay sized (i.e. 100% less than the 2 µm particle size) but does need to have sufficient fine grained particles (i.e. particles with sizes less than 75 µm) to achieve a maximum threshold hydraulic conductivity when compacted of typically $\leq 10^{-9}$ m/s. The origin of this threshold hydraulic conductivity value is unclear, but this value first appeared in guidelines in the mid-1970s in the United States of America (USA) and in draft federal regulations in the early 1980s. When adopted in 1980, the value 10^{-9} m/s was assumed to correspond to an essentially impermeable liner but still to be a practical, achievable value.

To achieve this value, suitable clay will usually meet the following criteria [6]:

- A fines content of $\leq 30\%$ (particles of $<75 \mu m$);
- A plasticity index of 7–12% (to ensure adequately low hydraulic conductivity), but between 20% and 50% to achieve good 'workability' during construction;
- A gravel content (particles of 4.76–76.2 mm) between 20% and 50%, and preferably no particles of 25–50 mm (and of less than 25 mm for a layer (lift) placed in direct contact with a CCL).

The plasticity index is a measure of the range of water content over which a material exhibits plasticity. Generally, the higher the plasticity index, the lower the resulting hydraulic conductivity of the material when correctly compacted. The plasticity index is determined by carrying out a series of index tests called Atterberg limits tests, which can be conducted by most commercial geotechnical laboratories (ASTM D4318 [7] or equivalent). A more detailed description of the Atterberg limits tests and how they are conducted is provided in Ref. [8].

The effect of gravel content on the hydraulic conductivity of a material with otherwise low hydraulic conductivity can be addressed by adding gravel to kaolin or mine spoil. For the kaolin (a clay of predominantly kaolinite), hydraulic conductivity begins to increase sharply as gravel is added. Only about 20% additional gravel increases the hydraulic conductivity of the mine spoil. Hence, the guidance value of allowable gravel content (20–50%) cannot be made more specific. However, the results were obtained in the laboratory on carefully prepared specimens of the materials, which are difficult to replicate in

the field. The inability to control the conditions of the materials in the field to the same extent as those in the laboratory as well as field compaction limitations (e.g. the inability of compaction 'feet' or probes to penetrate the full layer (lift) of compacted material) are likely to limit the actual gravel content to less than about 25% [9].

The importance of compaction water content (defined as the ratio of the mass of water to the mass of dry solids) and dry density (defined as the ratio of the mass of solids to the total volume of the soil) in achieving the required hydraulic conductivity is shown in Fig. 2. The determination of the link between dry density and water content is straightforward and can be performed by most commercial geotechnical laboratories. The relationship between dry density and hydraulic conductivity is more difficult to determine and usually requires more specialized testing facilities to measure hydraulic conductivity [8].

As illustrated in Fig. 2, as the compaction water content increases above the optimum value, the hydraulic conductivity decreases rapidly. The decrease in hydraulic conductivity achieved by altering the compaction water content from



Compaction Water Content, w (%)

FIG. 2. The relationship between compaction water content and resulting dry density (top) and hydraulic conductivity (bottom).

the optimum value to a value of about two percentage points wet of optimum can be as much as two to four orders of magnitude [9].

The relationship between dry density and water content is determined using a standardized laboratory test [8]. For liner applications, the relevant test is the standard Proctor compaction test (named after the developer of the test, R.R. Proctor) [10]). The objective in the field is to achieve a particular dry density and water content simultaneously. In applications such as road construction, where strength and stiffness are important, the common practice is to set a dry density specification that is no less than 95% of the maximum value achieved in the laboratory, with the as-placed water content of no more than ± 1 percentage point of the optimum water content value. In liner applications, low hydraulic conductivity is the primary objective, whereas strength and stiffness are secondary considerations.

As previously explained, compaction is usually carried out at about two percentage points above (i.e. wetter) than the optimum value. This means that the compacted clay is relatively wet on completion of the compaction process. If the clay dries out, it will shrink and crack and thus undermine the key objective of compacting the clay, which is a low hydraulic conductivity. A compacted clay layer needs to be covered as soon as possible after placement and compaction to protect it from atmospheric distress (e.g. desiccation or freezing) and/or other detrimental events. In this regard, the drainage layer shown in Fig. 1 could also serve as a protection layer.

Construction specifications for compacted clay barriers are typically given in terms of the water content of the clay (often wet of optimum), barrier thickness (often 450–600 mm), lift thickness (often 150–300 mm, although thinner is preferred) and in situ hydraulic conductivity (often $\leq 1 \times 10^9$ m/s). In general, construction practice for low conductivity layers includes thin lifts of soil placed wet of the line of optimums, a high level of kneading compaction and interlift bonding [11, 12].

Adequate interlift bonding between adjacent compacted lifts is crucial. The interface between layers may otherwise provide a preferential seepage path, negating the benefits of the CCL to some extent. This issue arises particularly when successive layers are not placed continuously (e.g. when a second layer is placed one day after the first layer). In such cases, the first layer needs to be lightly disturbed (e.g. using a disc or harrow), conditioned to the correct water content and compacted again before the new second layer is placed and compacted.

In terms of compaction equipment, a sheep-foot roller is preferred for clayey soils. The feet or protrusions of the compactor penetrate the compacted lift, producing a kneading action. This helps to blend and homogenize the clay during compaction. The clay is remoulded, its structure is removed and interlift bonding between successive lifts is improved, particularly if the feet on the roller penetrate through the new layer that is being placed into the older, previously placed layer. A smooth drum roller — with or without vibration of the roller — is often used to leave a smooth surface, which is necessary when an overlying geomembrane is included in the design.

The purpose of these procedures is to remove all residual soil structure present at the borrow source and to remould the entire clay barrier into a monolithic structure free of large voids. The type of compactor to be used depends on suitability to the material being compacted and availability. Further information on suitable compaction equipment can be found in Ref. [13].

If suitable clayey material is compacted correctly, a relatively low hydraulic conductivity layer of bonded, compacted lifts is created. Thorough on-site supervision of CCL construction is critical because even a relatively small area of poorly compacted material can severely compromise the performance of the CCL. Extensive field experience shows that these procedures are very effective for achieving a compacted clay barrier with a hydraulic conductivity of $\leq 1 \times 10^{-9}$ m/s. For example, 74% of the field tests from a survey of 85 compacted clay barriers indicated that this objective was achieved at the time of construction, whereas an additional 20% had a hydraulic conductivity $\leq 3 \times 10^{-9}$ m/s [14]. The study also concluded that, given appropriate material index properties (e.g. grain size distribution and Atterberg limits), the single most important criterion for achieving a suitably low hydraulic conductivity is that the soil is compacted to the point equivalent to the wet of the line of optimums at the time of placement. Quality control (QC) during the construction of a CCL is also extremely important. Inadequate QC can lead to the design hydraulic conductivity not being achieved and the seepage through the liner being higher than predicted.

The effect of clods on achievable compaction quality is sometimes not fully appreciated. Table 2 shows how the size of clods affects the achievable hydraulic conductivity when highly plastic clay was compacted at different values of water content [15]. For clay compacted on the dry side of optimum, the hydraulic conductivity decreased by a factor of up to six orders of magnitude when the clod size was reduced from 20 mm to about 5 mm. However, of greater interest are the compaction water content values (*w*) of 18% (the optimum value) and 20% when the clay is compacted on the wet side of optimum. For these values, the effect of clod size is less important, with a difference of one order of magnitude in the hydraulic conductivity resulting from different clod sizes. This result emphasizes once again the importance of compacting clay to values on the wet side of optimum; this practice has the added advantage of negating the potential increase in the hydraulic conductivity caused by large clods.

The intrinsic variations of a natural material such as clayey soil, as well as variations that result from differences in compaction energy and water content during compaction, mean that setting a target for compaction as a single value of dry density and a single value of water content is not possible. Rather, when specifying the required compaction parameters, a range of both dry density and water content is used. For liner applications, where water content values wet of optimum are advisable, a target range such as that shown in Fig. 3 is specified.

The shaded area in Fig. 3 is the zone of acceptable combinations of water content and dry density. This means that water content values as low as the optimum water content are acceptable according to this specification. However,

TABLE 2. EFFECT OF CLODS AND THEIR SIZE ON HYDRAULIC CONDUCTIVITY OF A CLAY LINER COMPACTED AT DIFFERENT WATER CONTENT VALUES [15]

Composition water content $w(0/)$ -	Hydraulic conductivity, K (m/s)		
Compaction water content, w (%)	5 mm clod	20 mm clod	
12	2×10^{-10}	$4 imes 10^{-6}$	
16	2×10^{-11}	9×10^{-4}	
18	1×10^{-11}	8×10^{-12}	
20	2×10^{-11}	6×10^{-12}	



Compaction Water Content, w (%)

FIG. 3. Example of a conventional (traditional) acceptable zone of dry density and water content specified for compacted clay liners.

the use of this type of specification has been shown to result often in unacceptably high values of hydraulic conductivity of the compacted clay. An alternative or so called 'modern' specification that has been shown to provide better results is based on the line of optimums. To explain this approach, additional background information is provided next.

Using site-specific maximum dry density and optimum moisture content points from standard and modified proctor tests, a best fit line of optimums can be developed. The best fit line of optimums is one approach for establishing whether soil material properties have changed significantly. It can also be used to define pass/fail criteria for compacted density and compacted moisture content during the construction of recompacted soil liners, separatory liner/leachate collection systems and cap system soil barrier layers [16].

Figure 4 illustrates the compaction curve obtained for the same soil using three different compaction energies. As the compaction energy increases, the optimum water content decreases and the resulting maximum dry density increases. This result is common for almost all clayey soils. A line joining the top point or optimum water content of each curve constitutes the line of optimums.

The best outcome in terms of achieving a low hydraulic conductivity occurs when the clayey soil is compacted above the line of optimums (but below the zero air voids curve). However, the required degree of compaction has to be confirmed in the field, which then ensures that the required value of hydraulic



FIG. 4. Effect of compaction energy on the compaction curve of clayey soil, and the concept of the line of optimums.

conductivity has also been achieved. A sufficient number of QC tests need to be carried out on each compacted lift. A common approach is to measure the dry density and water content and then use a relationship such as that shown in Fig. 4 to infer hydraulic conductivity. The most common QC test is the sand replacement test [14], although measurements using a nuclear device [17] are also common. A key concern with nuclear devices is ensuring the security of the equipment, which usually contains a radioactive source. The equipment also needs to be calibrated frequently to ensure an accurate measurement of the water content and the dry density.

Typically, CCLs are at least 0.6 m thick, with reported thicknesses being as high as 1.2 m. Thicknesses as low as 0.3 m have been reported for a CCL [18], where the liner was used as part of a composite liner, although such thin layers are not advisable. The importance of layer thickness is clearly shown in Fig. 5 (data from Ref. [19]), which demonstrates the effect of layer thickness on saturated hydraulic conductivity. The data are grouped according to whether construction practices were considered 'poor,' 'good' or 'excellent'. The trend lines indicate 'poor' (long dashes), 'good' (short dashes) and 'excellent' (solid line) compaction practices.

Figure 5 shows that even good to excellent compaction practice will struggle to achieve a hydraulic conductivity of less than 10^{-9} m/s unless the liner thickness exceeds 0.6 m. Hydraulic conductivities of this value were only consistently achieved when liner thicknesses of about 1 m or more were used. Thus, the use of a CCL may be an expensive option, unless a sufficient quantity of good quality clay is readily available close to the site in question.



FIG. 5. The variation of saturated hydraulic conductivity with a thickness of liner; results are differentiated according to quality of construction [19].

Care needs to be taken when choosing the material to be used in a CCL. Laboratory tests to determine saturated hydraulic conductivity should be verified by comparison with values actually measured in the field. Considerable evidence indicates that the laboratory scale measurement of hydraulic conductivity on small specimens tends to underestimate the values that are measured in the field on more representative specimens [20]. When planning a QA programme to verify that in situ hydraulic conductivities are acceptable, the inevitable variability of hydraulic conductivity has to be kept in mind. Reference [21] discusses minimum testing frequencies and Refs [22] and [23] provide additional guidance on this topic.

When sufficient high quality clay is not economically available, the lining system may include one (or more) of several synthetic materials (or an SBL). The benefit of using synthetic materials is that QA/QC for the constituent materials and method of manufacture can be performed to a high level in the manufacturing facility. This contrasts to compacted clay, for which QC can be performed only after it has been applied at the site. Although a range of synthetic materials have been advocated as liner materials, the most prevalent materials are GCLs or GMLs.

3.2.1.2. Geosynthetic clay liners

A GCL usually comprises a layer of bentonite (either powdered or granulated) sandwiched between two geosynthetic layers, typically geotextiles, which are then held together by stitch bonding or needle punching through the bounding geotextiles. When hydrated, bentonite swells (sodium bentonite will typically increase in volume by 15–18 times when wetted). However, in a GCL, the connected geotextile layers restrain this swelling, resulting in a dense composite material with very low hydraulic conductivity. A GCL is thin, with a thickness of usually no more than 10 ± 5 mm.

The correct hydration of the bentonite is critical to the successful use of a GCL layer, as discussed in Ref. [24]. As with geomembranes, GCLs are supplied in rolls. On-site, these rolls are not physically bonded together (unlike geomembranes), but rather are simply overlapped by at least 300 mm with powdered bentonite sprinkled between the overlapping GCLs at about 0.4 kg/m. The hydration and swelling of this bentonite, together with the applied overburden stress, ensure the integrity of the overlap and the overall performance of the GCL.

Since their introduction in circa 1990, the use of GCLs in liner applications has become widespread, particularly in landfill applications. An increasing number of GCL products are available on the market, although some may be of poorer quality than others.

Although the word 'bentonite' is used generically to describe the clay used in the manufacture of GCLs, different types of bentonite are available and care is needed to ensure that the type of bentonite chosen is consistent with desired performance objectives:

- Sodium bentonite. Bentonite comprising high swelling sodium montmorillonite as the predominant clay mineral is preferred for most GCLs. When confined by geosynthetic layers, as in GCLs, the very high swelling characteristics of this bentonite (known as sodium bentonite) produce very low hydraulic conductivities. Bentonite used in most GCLs typically contains 60–90% sodium montmorillonite. However, the higher the montmorillonite content of the bentonite, the greater the tendency for the bentonite to be more susceptible to incompatibility upon exposure to chemical solutions, which can result in significant increases in the hydraulic conductivity [25]. Also, multivalent-for-monovalent cation exchange combined with wet-dry cycling can lead to a significant increase in the hydraulic conductivity of the GCL and overall poor performance [26, 27].
- Calcium bentonite. To overcome the issues with sodium bentonite, calcium bentonite (i.e. bentonite dominated by Ca²⁺ as the exchangeable cation) may be considered for use in some GCLs, although this is not common. The key advantage of calcium bentonite is its reduced swelling capacity. Once installed and hydrated, a calcium bentonite GCL will not be as susceptible to changes in the hydraulic conductivity achieved on installation (i.e. little or no increase in hydraulic conductivity over time owing to cation exchange). The problem is that to achieve the same low hydraulic conductivity to water ratio as is possible with sodium bentonite GCLs, much higher dose rates (bentonite g/m²) are required because of the reduced swelling capacity of calcium bentonite relative to sodium bentonite.
- Sodium activated bentonite. Sodium activated bentonite calcium bentonite that has been treated with soda to convert the calcium bentonite to sodium bentonite — is also available. Sodium activated bentonites are more common in Europe than in other locations, such as North America, because naturally occurring calcium bentonites are more prevalent in Europe.

However, the specification and the use of sodium bentonite GCLs are preferred as the performance of these products is likely to be superior to those using calcium bentonite or sodium activated bentonite.

Different GCLs use different geotextiles to encase the bentonite and different methods to bond the two geotextile sheets. Geotextiles may be woven or non-woven, and geotextile sheets may be joined by adhesive or by the process known as needle punching or stitch bonding. Some GCLs have a geomembrane

glued to one of the geotextile sheets, the intention being to provide a single product that constitutes a composite liner. Bentonite is typically not required for the overlap of GCLs backed by a geomembrane.

A GCL may be used in isolation (e.g. as a single liner), although this is relatively uncommon, or as a component of a composite liner, as discussed later. A GCL is often used in place of a CCL, and offers the following benefits over a CCL:

- It is typically less expensive (particularly when a CCL requires significant haulage of clay material).
- It is quicker and usually easier to install.
- It performs better (a lower flux rate is usually achieved).
- It takes up less space, which is particularly important in applications such as landfills where the waste storage volume ('airspace') constitutes a revenue source; increasing the airspace by using a thinner liner is therefore advantageous. In applications storing uranium tailings, airspace is likely to be a less important consideration.

Potential issues with GCLs include increases in the hydraulic conductivity because of chemical incompatibility and issues related to their thinness (10 ± 5 mm), which makes GCLs susceptible to defects due to puncture and greater diffusive mass flux relative to thicker CCLs.

3.2.1.3. Geomembranes

A geomembrane is made from a polymer material that has been formed into a sheet. The type of polymer used varies, with typical polymers being high density polyethene (HDPE), linear low density polyethene (LLDPE), very flexible polyethene (VFPE) and polyvinyl chloride (PVC). These materials are thin (with a thickness of 2 mm or less) and are supplied in rolls of material typically with a width of about 6.8 m. This means that successively placed layers of geomembrane need to be joined on-site, usually through welding or heat bonding. Extreme care needs to be taken with the QA/QC aspects of geomembrane installation, as many of the issues associated with the use of these materials have been the result of poor installation practices. Further details on geomembranes are provided in Ref. [28].

3.2.1.4. Sand–bentonite liners

If insufficient natural clay is available near the site of a proposed TSF or waste storage facility, and the cost of a GCL or geomembrane is considered unacceptable, an alternative might be the construction of a compacted SBL. This approach uses locally available sand (which, on its own, has an unacceptably high hydraulic conductivity for use as a liner), mixed with a small percentage of bentonite (usually around 4–5% by dry mass). Bentonite content of greater than 10% by mass is usually not economical, although for uranium tailings disposal, a higher percentage of bentonite of 15–20% may be necessary owing to the presence of radioactive elements in the tailings. The actual percentage of bentonite to be used is typically determined by laboratory hydraulic conductivity tests performed on compacted sand–bentonite specimens with varying bentonite content.

Similar to CCLs, the sand-bentonite mixture is compacted in lifts to form the SBL, and may be hydrated with water before waste is placed on top of the liner. The resulting SBL will have a suitably low hydraulic conductivity if correctly placed, compacted and hydrated. Similar to the situation with GCLs, sodium bentonite is preferred for use in compacted SBLs. Potential problems of compatibility of the bentonite with chemical species in solution need to be evaluated before construction of the SBL.

Bentonite is used, rather than other clays such as kaolin or attapulgite (palygorskite) based clays. This is because the large volume change of the swelling sodium bentonite, when effectively hydrated, will fill the voids between sand particles. As a result, low permeability to the compacted SBL will be imparted. The smaller volumetric strain (expansion) of other clays means that, although they are present in voids between sand particles, they do not fill the voids to the same extent because they do not swell to the same extent. As a result, greater amounts of these other clays are typically required to completely fill the voids between the sand particles. An illustration of the change in the hydraulic conductivity resulting from the addition of different 'clay soils' is shown in Fig. 6.

As previously noted, adding more than 10% bentonite by mass is not likely to be economically justifiable. Figure 6 is reproduced from the data in Ref. [29] and shows that the addition of 10% bentonite resulted in a decrease of hydraulic conductivity of over five orders of magnitude, whereas the addition of 10% attapulgite clay resulted in a decrease in hydraulic conductivity of only approximately one order of magnitude. However, the results in Fig. 6 are based on permeation with tap water. A subsequent study in Ref. [30] indicated significant increases in the hydraulic conductivity of compacted specimens of the same mixtures based on permeation with calcium chloride solutions.

Also as previously noted, the type of bentonite to be used is important. Sodium bentonite (which is widely used in drilling mud) expands more than calcium bentonite, providing a lower hydraulic conductivity when used in an SBL system than can be achieved with calcium bentonite. Experimental results [31] demonstrate this concept as shown in Fig. 7.



FIG. 6. The decrease in the hydraulic conductivity of sand with the addition of attapulgite clay and bentonite.



FIG. 7. Changes in the hydraulic conductivity of sand resulting from the addition of different clays, illustrating the significant difference in the behaviour of sodium bentonite and calcium bentonite blends.

An SBL can be an attractive option as a lining system, as long as good quality bentonite is available at a reasonable cost (including the cost of transportation to the site). As shown in Ref. [32], reductions in hydraulic conductivity resulting

from the addition of 8% powdered bentonite differed by up to three orders of magnitude depending on whether bentonite of high or marginal quality was used.

If implemented correctly, an SBL can be a good alternative to compacted clay or some combination of geosynthetic lining systems, as the SBL retains the strength and low compressibility that are characteristic of sand but has the very low hydraulic conductivity characteristic of bentonite.

Much like a CCL, the lowest hydraulic conductivity of an SBL is often achieved at a water content of about 2% wet of optimum. Figure 8 illustrates the link between compaction water content, dry density and hydraulic conductivity



FIG. 8. Change in dry density (top) and hydraulic conductivity (bottom) of a sand-bentonite liner as a result of compaction at different water content.

based on Ref. [33]. However, SBLs are relatively insensitive to compaction water content (i.e. $\Delta \log K < 1$), primarily because of the high swelling capacity of the sodium bentonite in the SBL.

The optimum bentonite content obtained from laboratory tests will generally provide an optimistic estimate of the actual bentonite content required in the field (i.e. a higher bentonite content will be required to achieve the same hydraulic conductivity in the field as in the laboratory). This difference in bentonite content is primarily a result of the difficulty of ensuring a thorough and homogeneous mixing of bentonite with the sand that is being stabilized. If there are zones of minimal bentonite (or worse, no bentonite at all), then that section of the liner will be leaky and potentially completely negate the benefits of adding bentonite. The proper mixing of bentonite into the sand to be stabilized is therefore critical.

Possibly the best way of mixing the sand and bentonite is by using a pug mill. The sand and the (dry) bentonite are loaded from storage hoppers, via a conveyor, into a rotary mixing device (pug mill), which produces an homogeneous blend if operated correctly. Difficulties in accessing such equipment have sometimes resulted in mixing performed on the ground. Piles of bentonite are placed on the ground at the intervals determined by the desired bentonite content and then mixed using earth-moving equipment such as graders and loaders. Although convenient, this approach does not provide the same quality of blending that can be achieved using a pug mill.

In principle, SBLs do not suffer from the same problem of damage (increased hydraulic conductivity) caused by freeze-thaw effects as CCLs do. However, evidence to the contrary is provided in Ref. [34]. Results from Ref. [34] are summarized in Fig. 9, which shows that the hydraulic conductivity actually decreases as the number of freeze-thaw cycles increases. Other data reported in Ref. [35] show a slight deterioration in performance (slightly increased hydraulic conductivity) resulting from the freeze-thaw cycles, although the change in performance was very small. One reason that freeze-thaw cycles do not significantly affect the hydraulic conductivity of sand-bentonite mixtures is that upon thawing, the bentonite becomes rehydrated and the resulting swelling refills the pores between the adjacent sand particles.

Similar to freeze-thaw cycling, cycles of drying and wetting do not have a significant impact on the performance of SBLs (as long as good quality sodium bentonite is used). Figure 10 summarizes data from Ref. [36] reflecting this. Even after five cycles of drying and wetting, the change in hydraulic conductivity is negligible, unlike for a CCL.

The preceding discussion dealt with components that may be used in a single liner system. The potential flaw with any single liner system is that if the liner is damaged or breached in any way, leakage into the soil underlying the waste storage facility will occur and detection of this leakage will be difficult until contamination is observed at a location outside the facility (e.g. in monitoring wells). A better practice is therefore to use a form of composite liner. A composite liner might be the only liner used, or more likely in the case of uranium tailings, will be the primary liner in a double lining system.



FIG. 9. Variation in hydraulic conductivity with the number of freeze-thaw cycles for Ottawa sand stabilized with sodium bentonite according to bentonite content.



FIG. 10. Demonstration of the relative insensitivity of sand–bentonite liners to cycles of drying and wetting.

3.2.2. Composite liners

A composite liner consists of two different liners placed in close contact with one another. A composite liner usually consists of a geomembrane overlying a clay liner, a GCL or an SBL. A GML–CCL composite liner is illustrated in Fig. 11.

Composite liners are significantly more effective at limiting liquid migration into the subsurface than any single liner system is, and the synergistic effect of a composite liner is better than the sum of the individual effects of the two components (such as an individual GML or GCL). This is because a hole or defect occurring in the GML is not likely to line up with a zone of high hydraulic conductivity in the underlying CCL, GCL or SBL. This effect is shown schematically in Fig. 12.

Without the CCL, leachate would flow through the hole directly into the subsurface. With the CCL, leachate is impeded from flowing into the subsurface because of the low hydraulic conductivity of the CCL. The condition of the interface between the GML and the CCL is crucially important. If the contact between the GML and the CCL is poor, leachate can pass through the hole, travel along the top surface of the CCL and may potentially reach a poorly compacted



FIG. 11. Schematic of the composite liner, showing a geomembrane liner overlying a compacted clay liner (or geosynthetic clay liner).



FIG. 12. Schematic of the effectiveness of the composite liner: the compacted clay liner provides a barrier to flow if a hole occurs in the geomembrane liner.

(high hydraulic conductivity) region of the CCL and then seep through such a zone. To harness the full potential of a composite liner, good contact between the two components of the composite liner is essential.

Under no circumstances should the two components (e.g. the GML and the CCL) be separated by a sand or geotextile (or similar) layer. (This is sometimes done to protect the underlying layer until the GML is placed, for example to prevent the drying out and shrinking of a CCL that would result in cracking.) The installation of a high hydraulic conductivity layer (such as sand or geotextile) would negate the synergistic benefit that is otherwise obtained from a composite liner.

As previously noted, the installation of a GML with no defects is extremely difficult, although the size and number of defects can be minimized. To illustrate this point, Ref. [37] surveyed 28 areas lined with geomembranes, of which 23 were HDPE, 4 were PVC and 1 was a chlorosulfonated polyethylene (CSPE) geomembrane. On average, 26 leaks/ha of installed geomembrane were found. Reference [38] reported values of 3 holes/ha following installation and 12 holes/ha following placement of a drainage layer. Reference [39] suggested that design calculations be based on 2.5–5 holes/ha, provided that strict QA procedures were adhered to. With current, improved installation procedures, the incidence of defects has dropped to 1–2 leaks/ha. Although much reduced, defects still result in unacceptably high leakage rates and demonstrate why a composite (or perhaps a multiple) liner is necessary.

Figure 13 shows seepage rates in terms of litres per hectare-day (L/ha-d) for various types of lining systems, as well as the effect of various levels of QC, an aspect that is discussed later. Results have been modified based on the data from Ref. [40].

Table 3 shows that covering a CCL with a GML, thus forming a composite liner, in this case had relatively little benefit and approximately halved the leakage rate. A far more significant improvement was achieved by using a GML–GCL composite, which reduced flow rates by a factor of almost 30. Results from another study [6] are summarized in Table 3.

A composite liner needs to be overlain by a drainage collection and removal layer, similar to a single liner system, to remove liquid from the top of the lining system as quickly and as efficiently as possible. During the operational phase of a TSF (i.e. while tailings are being deposited), the provision of an adequate drainage collection system is particularly important. If this is not provided, a very high head of leachate can build up on the liner. This head buildup will increase the hydraulic gradient and cause higher seepage rates through a liner. Once tailings cease being deposited in a TSF, the water level in the tailings (usually referred to as the phreatic surface) should gradually decrease, resulting in less demand on the drainage system and, thus, less hydraulic head on the liner. However, if the cover system allows the continued infiltration of precipitation into the tailings, the phreatic surface may remain elevated for many years or even decades. (See Section 4 for more information on cover systems.)

Although very low leakage rates can be achieved with a composite lining system, a risk analysis of the potential for groundwater contamination from a uranium TSF, particularly given the extremely long half-lives of many radionuclides, may indicate that a single liner, albeit a composite liner, is not sufficient.

TABLE 3. COMPARISON OF LEAKAGE RATES THROUGH TWO DIFFERENT COMPOSITE LINERS AND A SINGLE GEOMEMBRANE LINER [6]

Type of liner	No. of cells	Leakage rate (L/ha-d)
Geomembrane liner (GML)	28	200
Geomembrane liner–compacted clay liner (GML–CCL)	11	90
Geomembrane liner–geosynthetic clay liner (GML–GCL)	19	0.7



FIG. 13. Leakage rates through different lining systems, with results shown for both good contact and poor contact between the two components of a composite liner.
3.2.3. Multiple liner systems

A well-constructed composite liner can still leak (Fig. 13). Any seepage through the composite liner would be difficult to detect until contamination is observed in downstream monitoring boreholes or wells, by which time a significant quantity of contamination may already have been released from the TSF. A method to intercept and collect this seepage is desirable. The use of a form of double lining system addresses the problem of undetected seepage by providing a way to collect and remove the seepage occurring through the primary liner, although the seepage through the secondary liner is similarly difficult to detect as in the case of a single composite liner.

The simplest double liner consists of a composite liner, such as those discussed above, underlain by a leakage detection system and another lining system (usually a single liner). Other examples include a single liner, such as a geomembrane, and a secondary, composite liner. A more elaborate composite liner system, consisting of two composite liners, is illustrated in Fig. 14.

A multiple liner system has the following benefits:

- Leakage rates are further reduced when compared with single composite liners.
- The leakage detection and collection system (LDCS) indicates the performance of the upper lining component. The system also reduces the heat on the lower component of the lining system (e.g. the lower GML– CCL composite (2) shown in Fig. 14) to a negligible amount, as the system should only need to cope with very low leakage rates.



FIG. 14. A multiple (double) liner system, comprising two composite liners separated by a leakage detection and collection system.

The decision on the type of lining system to be used for storing tailings or waste rock should be based on an evaluation of the hazard posed by the material to be stored, taking into account factors such as the probability of failure and the consequence of failure (where failure is defined here as 'lack of acceptable performance', such as seepage rates higher than those based on the design). A form of multiple liner system is probably advisable for all facilities that will be storing uranium mill tailings or waste rock, unless the potential risk to the environment and/or surrounding communities can be shown to be negligible. This would be the case, for example, for potential groundwater contamination particularly if the groundwater is extremely deep and is separated from the TSF by a material with low hydraulic conductivity that is also able to attenuate radionuclides to some extent. Such a favourable condition is not common, and the final decision on which liner configuration to choose should be based on a risk evaluation process that accounts for the potential impacts of leachate release.

The key components of a lining system are the low hydraulic conductivity layers, such as CCLs, GCLs, GMLs and/or SBLs. Other important components are the high hydraulic conductivity layers (i.e. the drainage layers that collect and remove leachate, either from the top surface of the primary lining system or from beneath the primary lining system, in which case flow rates within the drainage layer are likely to become low).

3.2.4. Leachate collection and removal systems

In a lining system, the common practice is to include layers that have a relatively high hydraulic conductivity, counterintuitive though this might be. This provides a drainage path for free-flowing liquid. If a layer with a low hydraulic conductivity is functioning correctly, the liquid will accumulate on top of this layer, generating a hydraulic head that may build up to an unacceptably high value. In some municipal solid waste landfills, the hydraulic head buildup eventually compromised the stability of the outer slopes of the landfill. High hydraulic conductivity layers are used to prevent the buildup of a hydraulic head on the primary liner and conduct the liquid to a collection sump, allowing treatment of the liquid, if required. High hydraulic conductivity layers are also used in leakage detection layers, which form a component of composite liners, as discussed earlier.

Drainage layers may consist of either natural materials (gravels or sands) or synthetic products, such as geotextiles or geonets. A geonet is formed by a continuous extrusion of parallel sets of polymeric ribs at acute angles to one another that conduct water through the openings between the ribs. If a geonet is used, the potential for the intrusion of surrounding material, such as clay from a CCL or bentonite from a GCL, needs to be prevented. A geotextile is a

flexible, porous fabric that is similarly able to conduct water in the plane of the fabric. Combinations of geonets and geotextiles are usually used in the leachate detection and drainage layer installed beneath the primary liner in a double or multiple liner system. Key issues of such an application include the plugging of the geonet openings and the impedance of drainage through geotextiles under high normal stresses. A key advantage of these systems is the relative ease of installation, particularly on steep side slopes where the installation of granular layers such as gravel or sand is particularly difficult.

The function of a leachate collection system is to remove leachate that collects on the liner to reduce the head on the liner to an acceptably low level. The preferred option for the leachate collection system is usually a layer of gravel, or very coarse sand, which should have a sufficiently high hydraulic conductivity to transport leachate into the drainage pipes. The key advantage of a layer of gravel or sand compared with a geosynthetic drainage layer is that the former has a much larger pore volume than a typical geosynthetic does, thus providing much improved resistance to clogging or blinding. The gravel or sand layer should typically be between 150 mm and 300 mm thick and have a hydraulic conductivity of no less than 0.01 m/s. The choice of the gravel or sand drainage layer should be based on matching the particle size distribution of the gravel or sand to that of the overlying tailings, using conventional filter criteria [41].

In leachate collection systems in landfills, a geotextile layer is often used above the gravel layer to prevent the biological clogging of the gravel layer. As biological clogging is not usually a concern with uranium mill tailings, the geotextile layer is generally not required. Placing and spreading the gravel or sand layer is a straightforward process on flat or relatively flat ground, but may become difficult on steep side slopes.

Numerous publications describe the methods for selecting the drainage medium, pipe spacing, pipe size and pipe perforations needed to achieve a desired maximum leachate depth. Reference [42] discusses how to compute the depth of leachate for a given leachate collection system design. A detailed discussion of pipe deflections is found in Ref. [43]. References [44] and [45] describe how to select pipe sizes to ensure adequate flow capacity.

3.2.5. Leakage detection systems

A leachate detection system is usually a layer of material with a high hydraulic conductivity sandwiched between two lining systems, as shown in Fig. 14. The function of this layer is to collect any leakage through the primary liner and to transport the leachate to collection pipes for removal from the storage facility. Flow rates in the leachate detection system are likely to be very low (ideally zero); this layer can therefore be relatively thin. This layer could be composed of gravel or coarse sand, or a high transmissivity geotextile and/or geonet, although, under the high overburden stresses of overlying tailings, a geotextile may compress and transmissivity will thus decrease, sometimes to an unacceptable extent. To ensure that the leachate detection system functions properly, the use of gravel material is preferable.

3.3. SITE-SPECIFIC CONDITIONS

3.3.1. Performance criteria

One major performance requirement of a lining system for a facility containing potentially radioactive tailings is minimal — ideally zero — leakage. Any leakage that does occur should be at such a low rate that the receiving environment can assimilate the contaminants without affecting the health of surrounding communities. A key requirement is also that the leachate collection system and the leakage detection system both function satisfactorily for the lifetime of the facility. This is particularly challenging for the waste storage facilities under consideration, as the expected design life is so long.

All uranium TSFs should not be required to implement the same lining system. Rather, a design and selection approach based on a comprehensive risk assessment is preferable. The hidden risk of this approach is its potential for misuse by proponents wishing to minimize the cost of the TSF. When applying a risk assessment approach, an independent, third-party review of the risk assessment should be a requirement.

The reason for suggesting a risk based approach that is non-prescriptive is that both the conditions and consequences of a particular configuration can be very different. A small TSF in a semiarid region, with a phreatic surface within the tailings of only one or two metres, will pose a much smaller threat of leakage to the surrounding environment than a TSF that is more than 50 m high and in a region of very high rainfall. Similarly, contaminated seepage will pose a much greater risk in an area where the groundwater is close to the surface and is used by humans than in an area where the groundwater is both extremely deep and hypersaline (i.e. unusable by humans or animals). A 'one-size-fits-all' approach is not appropriate for the selection of a suitable lining system.

3.3.2. Climate

The impact of climate on the performance of a liner is not as significant as its effect on covers. Nevertheless, neglecting the importance of climate can result in changes to a lining system that will lead to unsatisfactory performance. Examples include the desiccation of CCLs before covering them with a geomembrane in the case of composite liners, or more likely, the desiccation of a single compacted clay layer. Although less common, the freezing and subsequent thawing of a compacted clay layer will also result in a deterioration of the hydraulic conductivity of the layer. Once a CCL or GCL is covered (e.g. with tailings), the effect of climate is likely to be minimal; when these materials are used in liner applications, they remain isolated from the atmosphere.

3.3.3. Availability of materials

Using a CCL in a lining system is only likely to be economical if suitable clay is readily available close to the site in question. Similarly, an SBL is only likely to be viable if suitable sand is readily available, large volumes of suitable clay are absent, and the cost of geosynthetic materials is deemed too high.

If geosynthetics (e.g. geomembranes, GCLs, geotextiles) are to be used in the lining system, the selection of a reputable supplier is important, as the quality of manufacture varies significantly. An appropriate QA/QC programme should be implemented for all geosynthetic installations and should include verification of the quality of manufacture of the product used.

3.4. FAILURE MECHANISMS

3.4.1. Chemical incompatibility

Geosynthetic clay liners that use sodium bentonite are vulnerable to cation exchange, especially when exposed to water containing divalent cations (e.g. calcium, magnesium, many of the heavy metals) which affects the swelling characteristics of the bentonite and may increase the hydraulic conductivity of the GCL.

The hydration of a GCL is an important consideration. The design hydraulic conductivity of a GCL will not be achieved unless the GCL is correctly hydrated. Geosynthetic clay liners should be hydrated with uncontaminated water (i.e. the use of actual site leachate is inadvisable and can negatively impact performance). Even if hydrated with uncontaminated water, subsequent exposure to some chemicals in tailings water has been shown to later increase hydraulic conductivity by up to four orders of magnitude, thus rendering the GCL ineffective [46].

The timing of the hydration is also important. If a GCL is fully hydrated (i.e. fully saturated) before the placement of subsequent layers, difficulties relating to sliding on slopes and the extrusion of bentonite (and thus localized thinning of the GCL) are possible in basal layers.

Aqueous inorganic leachates do not generally have a detrimental impact on the hydraulic conductivity of CCLs unless the electrolyte concentration is very high (>1 M or more) or the pH is extremely low (<2) or high (>12) [47, 48].

Different types of geomembrane (e.g. HDPE versus PVC) are susceptible to degradation when exposed to different types of leachate. The most aggressive type of leachate is usually leachate containing certain organic compounds, such as those found in municipal solid waste leachate. The exposure of some geomembranes to such leachate results in swelling or, in a smaller number of cases, shrinkage and in consequent loss of ductility, durability and strength of the geomembrane. Such loss of physical properties will also render the geomembrane more permeable. Leachate from uranium tailings will invariably be inorganic: such processes are therefore usually not a major concern. However, leachate with a very low or very high pH can also be very detrimental to the long term performance of some geomembranes. Most manufacturers can provide a chart showing the compatibility of their product with various liquids; however, such charts are usually for very specific liquids, perhaps containing only one particular compound of concern. Most leachates (particularly municipal solid waste leachate) are a cocktail of contaminants and the cumulative effect of this leachate cocktail is often the reason for geomembrane degradation. Appropriate chemical compatibility testing may be advisable if the leachate is considered to be particularly unusual, if the risk to water resources is high, or if a new, unproven geomembrane is to be considered for use.

3.4.2. Desiccation (prior to waste filling)

The problem of desiccation of a clay layer is also discussed in Section 3.2. It can be a major concern in cover systems that use a CCL. In a liner application, desiccation of either a CCL or a GCL will severely impair the key function of the layer (i.e. the retention of a liquid). Desiccation occurs when a CCL or a GCL is exposed to the atmosphere and dries out, and is then covered (e.g. by a geomembrane or by a drainage layer). The drying out of the CCL or GCL results in cracking and the cracks thus formed (particularly in a CCL) do not seal upon rehydration. In lining applications, the greatest risk is immediately after placement of the CCL or GCL and prior to covering with either a protection layer or overlying waste. Once buried and under overburden pressure, desiccation should be a minor concern.

Figure 15 illustrates differences in the hydraulic conductivity of a CCL resulting from desiccation cracking, based on data from Ref. [49].

A composite liner comprising a geomembrane, together with a CCL, GCL or an SBL that is properly hydrated prior to being covered, is unlikely to suffer the problem of performance deterioration induced by desiccation. Once a liner is



FIG. 15. Effect of desiccation cracking on the hydraulic conductivity of compacted clay specimens. Results are shown for various confining pressures (equivalent to overburden pressure).

covered and tailings deposition begins, desiccation is unlikely to be a concern as the liner is then within a high humidity environment.

3.4.3. Freeze-thaw effects

Although freeze-thaw cycles are not likely to have major consequences for most liners, as the liner will be buried beneath tailings and thus insulated from freezing, freezing and thawing may still occur immediately after placement, before covering with drainage material and tailings. In extreme circumstances (e.g. very cold climates and a relatively thin layer of tailings), the liner may still be subjected to freezing and subsequent thawing.

The consequences of a liner undergoing freeze-thaw cycles are illustrated in Fig. 16 (based on Ref. [50]), which shows the increase in the hydraulic conductivity of liners at four different sites resulting from freeze-thaw cycles. The most pronounced effect is caused by the first freeze-thaw cycle, but further cycles cause a cumulative increase in the hydraulic conductivity.

Precautions, such as covering, should be taken as soon as possible after compaction to protect a CCL in cold climates. Frozen soil should never be compacted, as required densities will not be achieved.

A CCL that has been desiccated or subjected to freeze-thaw cycles can be considered permanently damaged. If a damaged liner can be rewetted to the required water content and then recompacted to the required density, restoration of the required hydraulic conductivity may be possible. However, the hydraulic



FIG. 16. Illustration of the increase in hydraulic conductivity resulting from freeze-thaw cycles for compacted clay liners at four different sites.

conductivity would need to be checked using an appropriate technique, such as those discussed in Ref. [51].

3.4.4. Installation defects and problems

An appropriate QA/QC programme is essential for the construction or installation of any lining system. Without an appropriate QA/QC programme, even an extremely well designed system is unlikely to be successful. Problems can arise if QA/QC is not implemented properly.

In the case of CCLs, the proper control of water content is critical. Field compaction at a water content below the optimum value results in field hydraulic conductivities that can be up to two orders of magnitude higher than laboratory results. If the water content at compaction is higher than the optimum water content, the hydraulic conductivity achieved in the field may be similar to that achieved using laboratory tests on specimens of the compacted clay.

Aside from desiccation (discussed previously), another potential problem with CCLs includes the non-uniform distribution of water during compaction. This produces clods that result in higher hydraulic conductivity values (also discussed previously).

Geomembranes and GCLs are supplied in rolls. Correctly placing (installing) a geomembrane is critical to ensuring optimal performance. The installation of a geomembrane with zero defects is generally accepted to be impossible; the objective is to minimize the number and size of the defects. The most common types of defects are holes and imperfect seams (joints)

between individual geomembrane panels. Holes can occur for several reasons: manufacturing defects; improper handling of geomembrane rolls; puncturing from underlying stones, vegetation stumps or roots; placement of drainage gravel or stone over the liner; traffic over the liner; and placement of waste over the liner. Over time, stress cracking may occur as the geomembrane ages, further adding to the risk of developing holes or tears. Prior to laying a geomembrane, the underlying surface should be checked for protrusions such as roots and stones. Once two sheets of geomembrane have been placed adjacent to one another, allowing an overlap, the sheets are joined, usually by some form of welding (see Ref. [28]). Joining individual sheets of a geomembrane is a highly skilled activity that only qualified and experienced contractors ought to perform. Such contractors will have a comprehensive QA/QC plan for such work, the aim being to minimize seam defects. However, even with the highest quality of work, achieving zero seam defects is virtually impossible.

As with a geomembrane, proper preparation of the underlying soil layer (subgrade) is essential before placing a GCL layer. The goal is a smooth subgrade, with no stones or other protrusions larger than 25 mm and no ruts of more than 25 mm deep. The compaction of the subgrade should be carried out prior to the installation of a GML or GCL. Conventional compaction criteria (e.g. the Proctor compaction criterion) should be used, ensuring that the level of compaction is sufficient to avoid rutting of the subgrade surface by the subsequent trafficking of installation and construction equipment. Placement of overlying material, such as drainage layers, should be done carefully to ensure that no damage to the liner occurs.

A key reason for placing such emphasis on the preparation of the subgrade and the placement of overlying layers is that, otherwise, the risk of puncturing is extremely high, particularly for GMLs and GCLs. Although a GCL may to some extent be able to 'self-heal', the process being that saturated bentonite will gradually fill up a puncture hole, this should not be relied upon; rather, prevention of the puncture occurring in the first place ought to be the focus of the installation crew.

Another potential problem with a GCL is the thinning of the bentonite, which usually occurs because of stress differences. The most prevalent mechanism is the localized stress concentrations caused by gravel particles adjacent to a GCL, which squeezes the low strength bentonite to areas of lower stress [52].

The problem of bentonite thinning can be mitigated by using earthen material above the GCL that does not contain aggregate of large size. An alternative is to use a geotextile protection layer, although that may be more expensive.

Although potentially time consuming, care taken during the placement and installation of a GML and/or a GCL will go a long way to ensuring that the in-service hydraulic conductivity is satisfactory.

3.4.5. Durability

The component of a lining system that is most likely to be of concern in terms of durability is a geomembrane, as the primary function of a geomembrane (low hydraulic conductivity) is severely compromised by deterioration processes. Other synthetic products (e.g. geotextiles) may also degrade with time.

Differences in composition and structure will cause different geosynthetics to respond differently to various loading conditions (including exposure to leachates). The intrinsic durability of a geosynthetic material depends on the polymer used in manufacture, the added compounds (such as plasticizer to make a geomembrane more flexible) and the method of manufacture. The deterioration of a geosynthetic material manifests in several ways, including softening and loss of physical properties (such as strength) owing to polymer degradation; embrittlement owing to loss of plasticizer; increase in hydraulic conductivity caused by swelling of a geomembrane; or failure of geomembrane seams owing to interaction with the contained leachate.

No single test exists that can quantify the potential degradation of a geosynthetic material, because of the range of factors that may contribute to such degradation. Specific tests may be used for specific concerns (e.g. ASTM D5747 [53] for evaluating chemical compatibility (and thus durability), which is discussed in Section 3.5.2.3). Achieving the most durable product in a particular application is possible only by ensuring that the best available geosynthetic material is used. Cost savings achieved by selecting a low quality, unproven geosynthetic material are much more likely to result in poor in-service performance than if a product is chosen that has a record of successful field applications.

3.4.6. Stability on side slopes

The placement of a liner only on the base of the TSF is usually not sufficient. Invariably the liner will also need to be placed on the internal side slopes to prevent any seepage of contaminated water into the environment. The placement and compaction of clay on side slopes is difficult, although an experienced contractor would be able to achieve the required degree of compaction.

Ensuring the stability of geosynthetic lining components after placement can be more challenging. Components such as GMLs and/or GCLs need to be anchored at the top of the slope to prevent them from sliding down the slope. This anchoring, in turn, generates tensile stresses in the liner, which it has to be able to withstand. Additionally, the friction between various liner components, as well as friction between the liner and underlying subgrade and between the liner and overlying waste need to be accounted for. With GCLs the internal shear strength also has to be considered. The hydrated strength of bentonite is extremely low; the internal strength of a GCL therefore depends on the method used to bond the geosynthetics used to manufacture it. The bonding method used has to withstand the internal shear stress that results from the placement of the lining system on a slope.

Additionally, if hydrated bentonite is squeezed out of the GCL owing to overburden stresses, this bentonite may result in a lubricating effect between the GCL and adjacent materials, providing a very low shear strength layer which needs to be accounted for in the design process.

3.4.7. Clogging of drainage layers

The integrity of a drainage layer is critical. The high hydraulic conductivity needs to be retained for the entire lifetime of the TSF, meaning that the porosity of the drainage material has to be ensured. This is achieved first by selecting the correct material. An otherwise suitable sandy material may prove to be unsuitable as a drainage layer even if mixed with only a relatively small amount of fine grained material such as clay or silt. Once installed, the pore size of a drainage layer can be reduced by the accumulation of biofilms, as can occur in municipal solid waste landfills, and by the accumulation of chemical precipitates in some TSFs. The importance of the long term performance of drainage layers cannot be overemphasized; deterioration of the drainage function might not only compromise the stability of a TSF, but will also increase the potential for seepage through the lining system as the ensuing hydraulic head increases, as in Eq. (1).

Clogging of leachate collection and removal systems (LCRSs) can have a number of causes, including calcium carbonate encrustation, iron deposition, the formation of biological slimes, and physical mechanisms.

The formation of biological slimes is unlikely to be a problem in uranium TSFs. Clogging from this formation is generally associated with municipal solid waste landfills. However, clogging caused by chemical precipitation may be a valid concern with some tailings water. Water containing calcium bicarbonate or iron bicarbonate may convert to carbonate forms, which then precipitate as calcium carbonate encrustations that can eventually block the pores in a drainage layer. Similarly, ferrous solutes that are present in oxygen depleted water will, when exposed to oxygen, oxidize to the ferric form of iron and precipitate as hydrated oxides. Exposure to oxygen may occur, for example, when leachate reaches the base of the primary lining system and enters the leakage detection system; as the latter is connected to a drainage sump, oxygen is likely to be present in the pores of the leakage detection system. The process of iron hydroxide precipitation also results in the loss of drainage capacity of an LCRS.

Physical mechanisms that may contribute to the clogging of a drainage system are primarily related to failure (collapse) of the drainage pipes, either as a result of damage during installation or the overburden stress being too large. Failure of the drainage pipes will prevent collected seepage water from being removed from the TSF, causing the head of retained liquid to build up and either emerge on side slopes, which can destabilize the facility, or accelerate seepage through the liner (because of the higher induced hydraulic gradient).

3.5. DESIGN OF LINERS

3.5.1. Conceptual design, performance prediction and iteration

Preceding sections discussed the various layers that may constitute an engineered lining system and the materials, both natural and synthetic, that make up components of these lining systems. This subsection deals with the materials in more detail. However, some key points that drive the selection of a particular liner need to be addressed first.

As a starting point, precedent is always useful (i.e. what was used in similar or equivalent applications). Sometimes the chosen liner may not have performed satisfactorily, which is in itself a useful indicator of what should and should not be used. If natural materials are to be used, the availability, cost and quality need to be considered. For example, if compacted clay is considered for the lining system, the source of clay should be available reasonably close to the site otherwise transportation costs may become prohibitive. Another potential problem is the choice of aggregate (e.g. gravel or sand for a drainage layer). Some aggregates may deteriorate rapidly, resulting in a low hydraulic conductivity layer rather than a high hydraulic conductivity layer. An illustration of how subtle differences in the mineralogy of a basalt aggregate lead to vastly different durability characteristics is provided by Ref. [54].

Consideration of only the properties and likely performance of each component of a lining system in isolation is not sufficient; interactions between different components of a lining system are also crucially important. As an example, the friction between a geomembrane and a GCL may be extremely low, particularly if the geomembrane is smooth; this would result in a plane of weakness should this combination of liner components be used on the side slope of a TSF, an effect that would be exacerbated should saturated bentonite be squeezed into the space between the geomembrane and the GCL. The potential interaction between various liner components (e.g. the impact of carbonate stone drainage layers, which contain calcium, directly over a sodium bentonite GCL) also needs

to be considered because of the potentially damaging impact of cation exchange on the hydraulic conductivity of a GCL, as discussed earlier.

The key performance criterion of a lining system is the prevention of seepage of a contaminant into the environment or, at the very least, reduction of the seepage rate to a value that is not harmful to the environment (i.e. the contaminant can be assimilated by the receiving environment). However, to fulfil the primary objective of seepage limitation, the integrity and stability of the lining system also have to be ensured.

Stability on side slopes is a major consideration, as the weight of the lining system itself, together with the weight of overlying waste material, may cause the entire lining system (or parts thereof) to slide down the slope if the resistance to sliding is not adequate. Stability evaluations may be conducted using conventional analytical techniques, utilizing large scale (probably at least 300 mm \times 300 mm) direct shear machines to provide the relevant shear strength parameters. Although many relevant interface strength data are provided in the literature, these data are to be used with caution as site specific conditions may render them inappropriate. Wherever possible, laboratory tests are to be carried out, using the materials that have been specified for use on-site.

Further to the issue of sliding, sufficient attention has to be paid to anchoring adequately the lining system at the top of the slope. This anchoring effect induces tensile stresses within geosynthetic components of the lining system. A calculation of the induced tensile stresses and the strength of the lining elements to withstand these stresses may be necessary. Anchoring is usually achieved by securing the geosynthetic(s) in an anchor trench, as shown schematically in Fig. 17. As an indication of size, anchor trenches are usually 0.3–0.5 m wide and 0.5–1 m deep. The geosynthetic component(s) are laid down the front face and the base of the trench, but not the back face (Fig. 17). The trench should be carefully backfilled, ensuring that the geosynthetics are not damaged in the process. Compaction of the backfill may be required to ensure an adequate anchoring of the liner.



FIG. 17. Illustration of the method used to anchor a geomembrane liner at the top of a slope.

The primary function of a liner is to minimize seepage into the surrounding environment. A CCL may have the secondary benefit of retaining some contaminants, including radionuclides [55]. The retention capacity will depend on the type of clay minerals present in the CCL, and it can be quantified through tests such as determination of the cation exchange capacity in the case where cation exchange is a dominant mechanism. Owing to the limited thickness and, therefore, the limited mass of bentonite in GCLs, the cation attenuation capacity of GCLs will typically be insignificant compared with that of a thicker CCL, even though the cation exchange capacity of the CCL is lower.

The correct functioning of drainage layers is crucial to achieving a satisfactory performance of any lining system. The drainage layers need to have a sufficiently high hydraulic conductivity and need to retain this hydraulic conductivity over the long term (for as long as the storage facility is required). Changes in hydraulic conductivity due to the clogging of voids by chemical precipitate have to be considered. If geotextiles are to be used as part of the drainage system, they should not be wrapped around drainage pipes, as this configuration has proven to be the most susceptible to clogging problems [56, 57]. Rather, the geotextile can be laid flat, on top of the primary (granular) layer of the leakage detection and collection system. The satisfactory functioning of a drainage layer also depends on the material remaining in place on the side slopes (i.e. not sliding down because of inadequate frictional resistance between the drainage material and underlying components such as geomembranes or GCLs).

3.5.2. Detailed design

3.5.2.1. Choice and installation of materials for lining systems

This subsection discusses design issues relating to the materials (natural and synthetic) that are available for use in both low hydraulic conductivity layers and high hydraulic conductivity layers.

The choice of whether to use natural or synthetic materials (and which type of synthetic material) for low hydraulic conductivity layers depends on many factors, such as cost, availability, ease of installation and the availability of skilled personnel to supervise construction and implement adequate QA/QC programmes.

As an example, a potential advantage of using a CCL (assuming suitable material is locally available) is that local contractors may be familiar with techniques for placing and compacting the clay to desired specifications. The installation and testing of a GML, however, requires many more specific skills and much more experience, which may not be available locally. The required skills and experience need to be adequately demonstrated, as poor installation

practices can render the installation of a geomembrane (in particular) virtually worthless. When designing an engineered lining system for a particular site, the risk evaluation process should take such factors into account, in addition to the more technical aspects of material selection, which are discussed in this subsection.

Although potentially more difficult to install correctly, a GML offers the advantage that the integrity of the product is (or should be) ensured during the manufacturing process. As long as the quality of the polymer used is suitable and appropriate manufacturing processes are implemented, properties such as the hydraulic conductivity, strength and durability of the geomembrane should be consistent with the manufacturer's specifications. Problems have arisen in the past when an alternative product has been supplied in place of the product originally specified, with the alternative product supposedly having properties equivalent to the original, but which, in fact, were inferior.

3.5.2.2. Types of geomembrane

The most commonly used type of geomembrane for lining waste disposal facilities is high density polyethene (HDPE). The advantages of HDPE include the generally excellent chemical resistance. In that regard, manufacturer product information should list those chemical compounds that are not compatible with HDPE. Ultraviolet and ozone resistance will not be an issue provided that high quality HDPE is specified. The geomembrane is usually delivered in large rolls with a width of about 6.8 m. Once placed in the required position, overlapping rolls need to be joined. This is usually performed using a heat welding process that requires specialist skills and equipment. As is the case for all geomembranes, HDPE geomembranes are thin, with a typical maximum thickness of 2 mm; many geomembranes are even thinner, with some installations using membranes with a thickness of just 1 mm.

Although an HDPE geomembrane has a high puncture resistance, care needs to be taken during installation as the membrane is only 1–2 mm thick. The surface on which the geomembrane is to be placed has to be free of sharp protrusions such as rocks and tree roots; the number of people walking on an exposed geomembrane should be minimized; and the placement of material (such as a drainage layer) over the geomembrane needs to be implemented carefully. The performance of a GML is only as good as the installation process followed. Advantages and disadvantages of various geomembranes are summarized in Table 4.

If a geomembrane is to be used in a liner installation, specialist advice needs to be obtained, particularly if a geomembrane comprising an atypical polymer is being considered. Persuasive arguments can be made for using other materials by

Geomembrane	Advantages	Disadvantages
HDPE	 Chemical resistance Weld strength Tolerates low temperatures Relatively inexpensive 	 Potential for environmental stress cracking High thermal expansion Relatively poor puncture resistance Poor multiaxial strain properties
LLDPE	 More flexible than HDPE Easier to install than HDPE Good multiaxial strain properties Can be factory fabricated and folded 	 Inferior ultraviolet resistance compared with HDPE Inferior chemical resistance compared with HDPE
FPP	 Can be factory fabricated and folded Excellent multiaxial strain properties Good conformability Broad seaming temperature window 	 Limited resistance to hydrocarbons and chlorinated water
PVC	 Good workability and conformability Easy to join Can be folded, resulting in fewer field fabricated seams 	 Poor resistance to ultraviolet and ozone unless specially formulated Poor resistance to weathering Poor performance at high and low temperatures

TABLE 4. ADVANTAGES AND DISADVANTAGES OF SOME COMMONLY USED GEOMEMBRANES

Note: FPP — flexible polypropylene.

citing specific advantages of a particular material, but optimal field performance is ensured by a product that simultaneously has many good properties. Some of the key properties to consider when selecting a geomembrane for inclusion in a lining system beneath a uranium tailings deposit are discussed in the following subsections.

3.5.2.3. Chemical durability

Some geomembranes degrade when exposed to particular liquids, such as concentrated organic compounds. To evaluate the compatibility of a particular geomembrane when in contact with a particular fluid, the most widely used test is the ASTM D5747 test [53]², in which samples (often referred to as coupons) are immersed in the liquid of interest for a period of time (usually up to 90 days). Afterwards, the coupons are tested to detect any changes in various mechanical properties (such as strength) as a result of the immersion. The rejection of a particular geomembrane would require selecting another type of geomembrane, which — considering the time needed for the test — could negatively impact the progress of a particular project.

Such tests should be run on more than one candidate geomembrane if any uncertainly exists about the type of membrane to be used. Very few laboratories are set up to conduct tests according to Ref. [53] because of the safety issues associated with the test procedure. Rather than send geomembranes for testing, a particular product is often specified on account of its reported success in a similar application.

3.5.2.4. Tensile strength

In a tensile strength test, a coupon of material is clamped between two testing jaws and a tensile load is gradually increased until the specimen snaps or breaks. During the application of load, the relation between applied force (*F*) and the resulting axial strain (ε_a) is monitored. A typical result is shown in Fig. 18.

A tensile test is not only used to determine the breaking (or ultimate) strength. The yield strength, which is the force above which the geomembrane continues to elongate with relatively little (if any) further application of force, is usually also reported.

In Fig. 18, the geomembrane initially exhibited a linear force–strain response, the relationship between which is referred to as the modulus of elasticity (sometimes known as Young's modulus). Once the yield stress was exceeded, large strains occurred with little further increase in force until the ultimate (breaking) force was reached. Behaviour such as that shown in Fig. 18 is referred to as ductile behaviour. The opposite would be brittle behaviour, where a geomembrane might snap or break shortly after reaching the yield force. In atypical cases, the yield and break forces may coincide. In a lining application,

² Note that reference is sometimes made to the EPA 9090 test method developed by the United States Environmental Protection Agency [58]. However, this test has largely been superseded by the ASTM D5747 test [53].

some ductility is desirable so that the geomembrane can stretch if required without breaking (e.g. due to differential settlement of the underlying soils).

Although a geomembrane would never be used in a reinforcing application, tensile strength is important for geomembranes placed on the side slopes of a TSF. The geomembrane would invariably be anchored at the top of the slope (Fig. 17) and as the weight from overlying material such as tailings is applied, tensile stress may develop in the geomembrane. Thus, tensile strength is a consideration.

Typical values for tensile strength are shown in Table 5. These values should not be used for design purposes, but rather are provided solely as an indication of typical values. Most manufacturers will provide certified laboratory test results for their particular geomembrane and, as long as the supplier is reputable, this value is usually reliable.



FIG. 18. Illustration of a result from a tensile test on a geomembrane specimen.

Test property	HDPE	LLDPE	PVC	CSPE
Maximum stress (kPa)	15 900	7 600	13 800	31 000
Corresponding strain (%)	15	400+	210	23
Modulus (MPa)	450	69	20	300
Yield stress (kPa)	11 000	7 600	13 800	2 800
Corresponding strain (%)	400+	400+	210	79

TABLE 5. TYPICAL PROPERTIES OF GEOMEMBRANES FROM WIDE-WIDTH TENSILE TESTS

Table 5 shows the very large strains that develop at both yield and breaking (maximum) stress in LLDPE tests, compared with the corresponding strains generated in tests on HDPE and CSPE geomembranes.

3.5.2.5. Interface strength

Although the primary function of a GML is to prevent the seepage of contaminated fluid into the environment, considerations of the geotechnical stability of the geomembrane are crucially important to ensure that the geomembrane can fulfil the seepage prevention function. The problem of stability is most apparent for GMLs on steep side slopes (see Fig. 17). Slippage is possible between the GML and underlying soil, or between the GML and any overlying layer (geosynthetic or otherwise). Resistance to slipping is determined by measuring the interface strength between two materials and is usually carried out in a large direct shear box, with typical dimensions of $0.3 \text{ m} \times 0.3 \text{ m}$ in plan view.

Smooth geomembranes have a very low interface strength. One way to compensate for this is to increase interface strength by texturing the surface of the geomembrane during the manufacturing process. This 'texturing' provides additional roughness to the geomembrane surface, which generates higher frictional strength in contact with adjacent materials. Typical values of interface friction strengths are given in Table 6 (for illustrative purposes only).

The benefits of texturing geomembranes are evident. Also apparent is that very different interface strengths develop, depending on the material with which the geomembrane is in contact. The interface strength between a smooth GML and a CCL would be lower than the values shown in Table 6 because the angle of internal friction³ of the clay is lower than that of sand.

When used in contact with other geosynthetics, the interface strength between the geomembrane and the adjacent geosynthetic material is also extremely important. Table 7 shows values for smooth and textured geomembranes in contact with various geotextiles.

3.5.2.6. Water and solvent vapour transmission

The water vapour transmission test provides a direct measurement of the diffusion of water through a geomembrane, as water molecules pass through

³ An important parameter of granular soils is the ability of the rock or soil mass to withstand shear stress. This can be measured by the angle of internal friction (also known as the angle of shearing resistance, or friction angle). The angle of internal friction (ϕ) is the angle that is measured between the normal force (σ) and resultant force (R) when failure occurs in response to a shearing stress (T).

the plane of the geomembrane. A common standard test for this property is the ASTM E-96 test [59], in which the geomembrane is sealed over an aluminium container containing water. The container is placed in a controlled relative humidity environment and the change in mass is recorded with time and converted to a vapour transmission rate. The experimental set-up is illustrated in Fig. 19.

OF SAND AND A RANGE OF GEOMEMBRANES					
Geomembrane	Concrete sand $(\phi' = 30^{\circ})^{a}$	Ottawa sand $(\phi' = 28^\circ)$			

25°

25°

18°

35°

___b

21°

18°

2.5°

TABLE 6. ANGLE OF FRICTION BETWEEN TWO DIFFERENT GRAINS OF SAND AND A RANGE OF GEOMEMBRANES

^a ϕ' refers to the effective stress angle of internal friction (the 'friction angle') of the sand.

^b —: not tested.

Smooth HDPE

Textured HDPE

PVC

CSPE

TABLE 7. INTERFACE STRENGTHS BETWEEN VARIOUS GEOTEXTILES AND GEOMEMBRANES

	Geomembrane					
Geotextile	HDPE		PVC		CSPE	
	Textured	Smooth	Rough	Smooth	Undulating	
Non-woven needle punched	32°	8°	23°	21°	15°	
Non-woven heat bonded	28°	11°	20°	18°	21°	
Woven filament	19°	6°	11°	10°	9°	
Woven slit-film	32°	10°	28°	24°	13°	

The change in weight per unit time is divided by the area of the test specimen, giving a value of water vapour transmission rate in g/m^2 -d. Typical values are summarized in Table 8. The table shows that for similar thicknesses, the HDPE geomembrane has a much lower water vapour transmission rate than the other geomembranes, with the PVC having a value some two orders of magnitude higher than the HDPE geomembrane. The benefit of a thicker geomembrane is also evident from the results of the two different thicknesses of geomembrane.

TABLE 8. TYPICAL WATER VAPOUR TRANSMISSION RATES THROUGH VARIOUS TYPES OF GEOMEMBRANE

Geomembrane	Thickness (mm)	Water vapour transmission rate (g/m^2-d)
PVC	0.76	1.80
CPE	0.97	0.56
CSPE	0.89	0.44
HDPE-1	0.79	0.017
HDPE-2	2.40	0.006



FIG. 19. Illustration of the water vapour transmission test and trend in data obtained from such a test.

For a particular geomembrane, the vapour transmission rate will be different for water relative to other chemicals (e.g. the rate for acetone may be one hundred times higher than for water). However, when retaining uranium mill tailings, the tailings process water will usually be sufficiently similar to water so that the value for the water vapour transmission rate can be used.

3.5.2.7. Environmental stress cracking

Environmental stress cracking is a visually brittle failure that occurs with constant applied stress that is lower than the yield stress of the geomembrane. It is particularly prevalent at locations where a geomembrane is deformed or bent (e.g. at the intersection of a steep slope and the natural ground surface). All HDPE geomembranes are susceptible to environmental stress cracking and installing them in ways to minimize environmental stresses (such as bends or distortions) is important. Standard test methods exist, such as those in Ref. [60]. Factors that influence environmental stress cracking include overheating during seaming and over-grinding parallel to seams, protruding stones, wrinkles and differential settlement.

3.5.2.8. Ozone and ultraviolet light resistance

When exposed to sunlight, photochemical degradation and thermal oxidation of a geomembrane may occur. This process results in degradation of the geomembrane, usually resulting in a weaker and more brittle material. Manufacturers have various ways of improving the ozone and ultraviolet resistance of their products (e.g. by adding compounds such as carbon black). This is another reason why sourcing a geomembrane from a reputable supplier is important in any project of a sensitive or critical nature, such as for uranium tailings.

3.5.2.9. Puncture resistance

A hole may be punctured through a GML from the impact of dropped tools, falling rocks or by sharp, angular objects present in the subgrade that apply large, concentrated loads on the geomembrane when the liner is covered by overburden, such as deposited tailings. The resistance to puncturing depends on the type of polymer used (a more flexible geomembrane is less likely to puncture than a brittle geomembrane), the thickness of the GML and the rate of loading. Reference [61] is recommended for evaluating the puncture resistance of a particular geomembrane.

Puncture holes can also occur in GCLs for the same reasons, but are usually less of a concern as GCLs are often able to 'self-heal', a process whereby bentonite extrudes and/or swells into the hole and seals it. A punctured geomembrane has no similar self-healing capability.

3.5.2.10. Installation

The placement of the geomembranes requires proper welding and testing of seams to prevent punctures. Therefore, field QA activities are stringent, ensuring that construction practices attain the desired material parameters (i.e. low hydraulic conductivity). A detailed description of construction methods and QA requirements for geomembrane barrier layers used in conventional covers is provided in Ref. [13].

The quality of a GML system can be severely compromised by poor installation. Adequate protection of a geomembrane starts when the material is first delivered on-site. The geomembrane rolls need to be protected from sunlight during storage (before installation). This protection can be achieved simply by covering them with adequate sheeting. Before installation, the surface on which the geomembrane will be laid has to be free of any sharp protrusions that might otherwise puncture the geomembrane. During installation, the geomembrane needs to be protected from vehicles and, as much as possible, from workers on foot. After placement and the joining of adjacent sheets, the GML will usually be covered by a form of drainage layer.

The drainage layer has to be placed carefully, so as not to damage the GML. Drainage material should not contain sharp aggregate and the equipment used to place the layer should be as light as possible. Once placed, and before coverage with a drainage layer, the geomembrane sheets have to be joined. This is a specialist activity to be carried out by experienced and specialist contractors. All seams have to be inspected and tested, and any defects are patched and verified to be leak-proof before the geomembrane is covered. These aspects of QA/QC are critical. As the GML is an expensive component of a lining system, incorrect installation or poor site supervision of the installation process can severely compromise the effectiveness of a geomembrane and potentially waste money.

Another problem that can occur with geomembranes after installation is wrinkles. Wrinkles occur as a result of the high coefficient of thermal expansion of (particularly black) geomembranes, causing them to expand when heated. Wrinkles may develop in a variety of shapes, as shown schematically in Fig. 20.

As is apparent from Fig. 20, severe damage to a GML can result when the geomembrane is subsequently covered by drainage material, or any other material, including tailings. Wrinkles do not disappear once buried; they remain



Under Low Normal Stress

One Sided Fold-Over



After Final Loading

cally Flattened Flohzontally Flattened

FIG. 20. Illustration of various wrinkles in a geomembrane.

in place, although are perhaps distorted. Wrinkles are undesirable for the following reasons:

- They might block the flow of water through the drainage layer, thus impeding collection in drainage sumps.
- They might compromise contact between the geomembrane and the underlying CCL or GCL (or equivalent), thus negating the benefit of a composite liner.
- They might threaten the integrity of the geomembrane. Strains of 5% caused by wrinkling have been measured. Such values can exacerbate environmental stress cracking.

Given the problems associated with them, wrinkles should be prevented as much as possible, and controlling their formation during installation should be a priority. Ways to achieve this include minimizing unnecessary 'slack' (i.e. by installing a GML in a taut condition) and covering the GML early in the morning before the heat of the sun causes it to expand.

Another potential problem encountered during installation is the lifting of the geomembrane by relatively high winds. Wind speeds of around 18–20 km/h are usually sufficient to lift a 1.5 mm or 2 mm thick geomembrane. Such winds may move the geomembrane some distance, risking severe injury to workers and at the very least damaging the geomembrane, which would require rejection of the affected sections and the purchase and installation of a new geomembrane. Suitable precautions include anchoring or ballasting geomembranes during installation until they are covered (e.g. by a drainage layer).

Geomembrane thickness (mm)	No. of cells	Av. leakage rate (L/ha-d)
1.5	11	55
2.0	11	55
2.5	1	325

TABLE 9. LEAKAGE RATES THROUGH SINGLE GEOMEMBRANES [6]

Note: The table shows only the leakage rates through the upper, primary, single geomembrane liner (not through the entire double liner system).

3.5.2.11. Leakage in geomembrane liners

Even if the procedures discussed previously are closely followed, no geomembrane can be installed without defects (no matter how small) and leakage occurring. In one study reported in Ref. [6], where single geomembranes were installed and underlain by a leakage detection system and a secondary (composite) liner, reported leakage rates during active disposal were 55–325 L/ha-d, as shown in Table 9 [6].

The field data in Table 9 imply that the thickness of the geomembrane has little impact on leakage rates. This is because leakage occurs through defects, not through the plane of the geomembrane itself. Installation variables such as joining and covering procedures affect the leakage rate far more. Money is therefore better spent on good quality installation and thorough QA/QC practices than on the purchase of a thicker geomembrane. While it is true that a thicker geomembrane is less easily punctured or damaged during installation, the thickness itself has little impact on leakage rates, as already mentioned.

3.5.2.12. Selection of a suitable geomembrane

No particular geomembrane is suitable for all installations, and the following factors need to be taken into account when selecting the most suitable geomembrane:

— To ensure the best service life and chemical resistance, HDPE is usually the best choice. Problems with the use of HDPE geomembranes include wrinkles, stress cracking and the rigidity of the material (that can make installation around sumps, for example, more difficult).

- Where flexibility is highly desirable, very flexible polyethene (VFPE), PVC and polypropylene (PP) are most suitable. Problems with these materials include a lower service life (compared with HDPE), ultraviolet degradation and chemical resistance.
- Where interface strength is a concern (such as on steep side slopes), textured polyethene (PE) or PVC is advisable.

Useful references for specifying a geomembrane for a particular application include Ref. [62] for HDPE geomembranes and Ref. [63] for PVC geomembranes.

3.5.2.13. Geosynthetic clay liners

Different GCLs may have different dosage rates of bentonite per square metre. Higher dosage rates will result in lower hydraulic conductivity values and thus better performance. When specifying a GCL, care should be taken to check the dosage rate used in a particular product. Most GCLs used for the lining of waste disposal sites such as landfills use a dosage rate of 4–5 kg of sodium bentonite per square metre.

A GCL only develops resistance to advection (i.e. develops a low hydraulic conductivity) when properly hydrated. Hydration should use uncontaminated water (i.e. not process water or landfill leachate, but water such as tap water or locally available groundwater — assuming this is not hypersaline, for example). There is sometimes a preference for allowing the bentonite in a GCL to hydrate by adsorbing water from the subgrade on which the GCL is placed. This is acceptable, although again, only if the water present in the subgrade is not hypersaline.

Once placed and hydrated, a GCL used in a lining application is subjected to increasing overburden pressure as the overlying tailings are deposited into the TSF. This increasing overburden pressure can be beneficial. Figure 21 is drawn based on the data from Ref. [51] and the results show the change in hydraulic conductivity with increasing compressive stress (equivalent to overburden pressure) for a range of different types of GCLs. The clear tendency is for the hydraulic conductivity to decrease as the overburden pressure increases.

3.5.2.14. Leachate collection and removal systems

The LCRS collects and removes leachate that builds up on the primary liner by draining the leachate to collection sumps. The typical thickness of an LCRS is between 0.3 m and 0.6 m, and the required hydraulic conductivity should be as high as 10^{-4} m/s, preferably even higher. To facilitate drainage, the base of a TSF is usually not flat but instead slopes towards drainage collection pipes as shown



FIG. 21. Decrease in hydraulic conductivity with increasing confining stress for several different types of geosynthetic clay liner.



FIG. 22. Sample base of a tailings storage facility, which is sloped towards drainage collection pipes embedded in shallow trenches (note: the vertical scale is exaggerated).

in Fig. 22. The inclination of this sloped base is usually 2–5% (in general, the steeper the better), with pipe spacing between 15 m and 70 m. The typical pipe diameters are ≥ 0.1 m.

If natural materials (such as gravel) are to be used in drainage layers, the material should be clean and free of fines particles ($<75 \,\mu$ m) to ensure satisfactory long term drainage. The material should also be durable, and stone such as quartzite is preferred because of its superior durability. Aggregate derived from limestone or other carbonate rock is highly undesirable, as carbonate materials will potentially slake and/or gradually dissolve when exposed to slightly acidic leachates, such as those that may be encountered in a uranium TSF.

During the installation of a drainage layer, handling should be minimized; each movement of the material will lead to particle breakdown, producing more fine material which reduces the hydraulic conductivity of the drainage layer. This



FIG. 23. Illustration of a geomembrane on a steep side slope, overlain by a leachate collection and removal system.

problem is particularly acute with crushed stone, as many of the sharp and jagged edges and points are chipped off during handling and placing. To further guard against the degradation of drainage media during installation, traffic directly on top of the drainage layer should also be minimized.

Slippage of the drainage layer down the slope has to be prevented; otherwise, the integrity of the entire lining system is compromised. Considering Fig. 23, which illustrates an LCRS overlying a geomembrane on a slope, slippage can occur within the drainage material itself, or at the interface between the drainage layer and the geomembrane (remembering that slippage between the geomembrane and the underlying subgrade is prevented by anchoring the geomembrane at the top of the slope). For a slope angle β , slippage will not occur if the relevant friction angle is greater than β . The relevant friction angles are φ' , which is the angle of internal friction of the drainage material, and δ , which is the friction angle between the drainage material and the geomembrane interface. However, if seepage occurs within the drainage layer, towards the base of the slope, the available strength due to friction is decreased. For this reason, the slope angle is typically limited to no more than 3:1 (i.e. approximately 18.4° to the horizontal).

If a smooth geomembrane is used, the slope angle may need to be even shallower because the interface strength between the geomembrane and the drainage layer might be very low. An alternative would be to use a textured geomembrane.

3.5.2.15. Leachate detection systems

The flow rate in a leachate detection system (if any) should be minimal, as any flow represents liquid permeating through the primary liner. The leachate detection system commonly consists of a geonet, sometimes in combination with a geotextile, which prevents the intrusion of soil from a CCL or of bentonite from a GCL, should these be placed either above or below the leachate detection system. A geonet will usually be able to provide adequate flow capacity because, as mentioned previously, the flow rates are likely to become low. A key consideration is the protection of the flow capacity during installation and upon loading from overlying tailings as these accumulate, as the added overburden load will tend to compress the geonet and reduce the flow capacity. The geonet needs to retain its flow capacity.

3.6. PERFORMANCE MONITORING

Once a liner is buried under tailings, visual monitoring or inspection using instrumentation is effectively impossible. Reliance is therefore placed on monitoring the lack of performance. In a double liner system, for example, the effectiveness of the primary liner can be quantified by measuring the flow quantity (and perhaps quality) emerging from the leachate detection system, and, if necessary, actions can be taken to address deficient performance. The performance of the secondary liner (i.e. the liner immediately beneath the leachate detection system) is more difficult to quantify, however. Leakage through the secondary liner may enter the subsurface and only be detected in more distant monitoring boreholes. If contaminated seepage is detected in monitoring wells, a large quantity of liquid is likely to have already escaped from the TSF. In such a case, the installation of a form of barrier or treatment zone such as those discussed in Sections 4 and 5 of this publication may be necessary.

Emphasis should be placed on monitoring the quality of the installation and construction of a lining system, rather than only on in-service performance. A poorly managed GML installation can negate any potential benefits that might otherwise accrue. Far less money will be spent on a QA/QC programme with sufficient resources than on trying to locate and repair leaks in the liner once the TSF is operational.

An issue that is sometimes overlooked is that tailings at the base of a TSF (and therefore adjacent to the LCRS) are subjected to a substantial overburden pressure. Depending on their characteristics (such as particle size distribution and mineralogy), the hydraulic conductivity of the tailings may decrease by more than an order of magnitude as a result of this pressure. Seepage rates from a lined facility may thus actually decrease with time, even though the imposed head on the liner increases with time.

Another aspect to be aware of when monitoring flow from a leachate detection system is that water may squeeze out of a CCL (should this be part of the lining system) as the load of deposited tailings is applied. This is known as 'consolidation water' and is as much as 1300 L/ha-d [64]. This water does not represent percolation

through the primary liner and is thus not a measure of performance of the lining system. Differentiation between consolidation water and true seepage from the tailings is possible by comparing the chemistry of the tailings with that of the water used to condition the clay before compaction in a CCL.

The literature indicates that the use of a well-constructed double lining system, with the primary liner being either a GML–CCL composite or a GML–GCL composite, can provide adequate protection of the underlying subsurface. For example, Ref. [6] found that flow rates through primary liners varied from zero to about 50 L/ha-d, with most values being less than 2 L/ha-d. As this is leakage through the primary liner that should be collected and removed by the leachate detection system, seepage rates through the secondary liner can be expected to be minimal. Most of the data on seepage rates published in the literature have been obtained from measurements made at municipal solid waste landfills. These facilities rarely have a hydraulic head (phreatic surface) in the landfill body, meaning that the head on the lining system is likely to be small. This is unlikely to be the case in a TSF, as water is continually being deposited together with the tailings, and a phreatic surface invariably develops, even with an underdrainage system (e.g. LCRS). Leakage rates through primary liners could thus be higher than the values obtained from measurements of landfill performance.

Despite the above caveat, published data show that it is possible to construct an engineered lining system that provides excellent protection for the adjacent subsurface. The following two questions still need to be considered:

- (1) Is the calculated seepage rate acceptable? For example, even a very low seepage rate may be considered unacceptable if a high quality aquifer, which provides drinking water for local communities, is located immediately below the TSF.
- (2) Will the components of the barrier system retain their required engineering characteristics for an acceptable time, remembering that uranium tailings will retain their pollution potential for decades or even centuries, depending on the contaminant of concern? Once tailings deposition ceases, and the tailings are covered and rehabilitation takes place, the hydraulic head on the lining system should decrease with time, potentially becoming zero eventually. At this time, the risk posed by the stored tailings would be negligible, as effectively no mechanism to transport contamination exists except for diffusion through partially saturated tailings. The required performance life of the components of an engineered liner may thus potentially be less than the design life of the TSF. A reduction in a head on a liner can only occur if the cover system allows less water to enter the covered tailings than is extracted through the LCRS. Section 4 deals with cover systems in detail.

3.7. CASE STUDIES

An abundance of literature describes various case studies of uranium mill tailings and methods that were adopted for their impoundment. Two case studies are discussed in this subsection to provide an indication of past problems and to illustrate how standards of design, construction and management of uranium tailings are continuously improving.

3.7.1. The Boršt mill tailings site, Zirovski Vrh uranium mine, Slovenia

The Boršt mill tailings site is a legacy site from the period of intense uranium mining that took place in former Soviet States prior to 1989. Ore production started in 1982, with yellow cake production beginning in 1984. Production only lasted for six years, terminating in 1990, shortly before the dissolution of the former USSR. Only 610 000 t of ore with 0.7 kgU/t were processed during this time, resulting in a tailings volume of about 375 000 m³. The TSF had an area of 4.11 ha.

The groundwater level at the site was relatively close to the surface and, prior to the placement of a layer of clayey material over the basin of the TSF, drainage pipes were installed (i.e. below the clay layer). Shortly after deposition of the tailings began, groundwater from below the tailings (from the installed drainage pipes) was found to be chemically and radioactively contaminated. A gradual increase in the concentrations of contaminants was observed. The compacted clay layer had only slightly restricted the flow of contaminants from the tailings into the subsurface.

The problems of the Boršt tailings were exacerbated by the existence of karstic sediments below the TSF, meaning that any contaminated seepage flowing into these geological sequences could easily enter the groundwater system. Furthermore, the geotechnical stability of the TSF was questionable. Shortly after heavy rains in November 1990, a crack having vertical movements of 0.2–0.3 m appeared in the road on top of the TSF. Investigations found that the TSF had been placed on an old landslide.

Subsequent activities appear to have stabilized the landslide, although the risk posed by the pre-existing landslide surface is still a concern. Remedial work to address the contamination potential of the tailings included the installation of additional drainage and a reduction of the angles of the outer slopes. At the end of 2009, the tailings were covered with around 2 m of soil (compared with the original cover of 0.25 m of topsoil) and vegetation was established. Seepage flow rates have decreased to around 52 m³ per day. The cost of the TSF remediation (in 2001) was estimated at €37 million, although the final cost may have been higher.

This case study illustrates the inadequacy of measures initially adopted to manage uranium tailings at the Boršt facility. The cover was inadequate, as was the lining system. Furthermore, existing geological problems, such as karstic sediments, were either undetected or ignored. An important lesson from this case study is that, aside from the need for much better containment strategies, a uranium TSF cannot be designed in isolation; prevailing geological, geotechnical, hydrological and groundwater conditions have to be taken into account. There is no substitute for engaging appropriately qualified experts when planning a new uranium TSF or the remediation of an existing facility. For further information about the Boršt tailings facility, see Ref. [65].

3.7.2. In-pit tailings disposal at the Langer Heinrich mine, Namibia

In 2013, Namibia was ranked as the fifth largest producer of uranium worldwide, and its position in this ranking is likely to rise further. The Langer Heinrich mine is located in a very sensitive area, being in the Namib-Naukluft National Park, a unique ecosystem with an extremely arid climate (with annual average rainfall of 67 mm and potential evaporation of 2300 mm). Despite the very low rainfall, and thus very low risk of percolation through the tailings from precipitation, precautions have been taken to protect the groundwater from tailings water that will occur principally as a consequence of tailings consolidation (and the associated expulsion of water).

The dominant strategy for tailings disposal at Langer Heinrich is storage within worked-out areas of the mine pit. The base of the pit is lined with 0.25 m of compacted clay, overlain by a 2 mm thick HDPE liner. The HDPE liner is overlain by a 0.25 m drainage layer, which collects and transports water to collection sumps. The lining system adopted is thus an illustration of a composite liner. Although the compacted clay layer component of this system is relatively thin, the rationale for choosing a thickness of 0.25 m is likely to relate to the post-closure absence of precipitation resulting in seepage through the tailings. The primary function of the liner is thus prevention of seepage during the operational life of the facility. The usual requirement of functionality for many decades (or even centuries) is less of a concern at this arid site than would be in more temperate and tropical climates.

This example illustrates the high standards of uranium tailings management that are being adopted worldwide, even in places where the adoption of inferior standards may have been the norm. Additionally, the case study shows that not all uranium TSFs necessarily need to follow the same approach. Particularly arid climates, such as Namibia, where precipitation and thus seepage are likely to be very low, may not need as extensive (and expensive) a lining system as would a uranium TSF in a location such as eastern Europe, which has much higher rainfall and much lower evaporation rates. For further information, see Ref. [66].

4. FINAL COVERS FOR WASTE CONTAINMENT

4.1. PURPOSE OF FINAL COVERS

Final covers serve various purposes according to the requirements of each site. Generally, covers are required to (i) provide physical containment of the waste, (ii) control vector intrusion into the waste, (iii) minimize or eliminate the release of mobile contaminants to the surrounding environment (via aqueous and gaseous pathways), (iv) persist for the design life of the containment facility (or the hazardous life of the waste), and (v) provide an acceptable end land use for the site of the containment facility. Hydrologic processes provide one mechanism for the release of contaminants from a waste containment facility. When water from precipitation passes through waste, mobile contaminants may be transported as leachate off site and/or to the groundwater system. Another release mechanism for contaminants is through gaseous diffusion from the waste; an example is the generation of radon gas from uranium mill tailings. To control the release of gaseous contaminants, covers may employ (i) resistive barriers to restrict gas movement (i.e. geomembranes or fine grained soil layers designed to maintain a high level of water filled pore space) or (ii) thicker soil layers designed to slow the migration of radioactive gases (i.e. radon) to allow decay within the cover profile.

Covers might or might not be combined with a basal liner. Liners usually emphasize the use of resistive barriers (e.g. compacted clay and/or geomembranes) to contain leachate. Traditionally, covers followed a similar evolution in design but in recent years they have included store-and-release mechanisms as an alternative at sites with suitable climates.

4.2. TYPES OF COVER

There are two common conceptual designs for hydraulic control. Covers may rely either on (i) a resistive barrier comprised of a layer (or layers) with low hydraulic conductivity or (ii) on a dual mechanism that combines water (precipitation) storage in the cover soils followed by removal of the water by evapotranspiration. This mechanism is sometimes called 'store-and-release'. Both conceptual designs typically include additional components (i.e. erosion control features, drainage layers and radon barriers) designed to meet the overall performance criteria and to be consistent with the site conditions. The different concepts may be combined for a hybrid cover design. The following subsections provide brief descriptions of resistive and store-and-release cover designs.

4.2.1. Resistive barrier covers

Traditional cover designs employ low conductivity materials to impede the downward movement of water (and/or the egress of gas). These covers are known as 'resistive barrier covers' and typically use geomembranes and/or compacted clays as the primary impediment to water movement. The resistive nature of these materials is described as the saturated hydraulic conductivity, which describes the ability of the material to transmit water when the entire pore space is water filled. The features of resistive barrier covers are similar in concept and design to common designs for bottom liners, and reflect the co-evolution in design. The similarities originate from concern for the so-called 'bathtub effect' that might result if percolation through a cover exceeds leakage through the liner.

Covers that rely solely on a compacted soil barrier are often referred to as 'compacted clay covers'. A geosynthetic clay layer (a thin layer of sodium bentonite sandwiched between geosynthetic layers, referred to as a GCL) is sometimes substituted for the soil layer. Covers in which the barrier is a geomembrane overlying a low conductivity soil layer (either compacted clay or a GCL) are referred to as composite covers.

Resistive barrier covers often require a drainage layer overlying the barrier layer, and a vegetated or armoured surface layer. The drainage layer allows the lateral diversion of water retained at the barrier. This prevents increased pore water pressures that can lead to slope instability and failure. A vegetated surface layer serves aesthetic purposes and provides resistance to erosion and damage to the barrier layer from the effects of freeze–thaw cycling. Examples of resistive covers are shown in Fig. 24.

4.2.2. Evapotranspiration covers

Cover systems that rely on a combination of temporary storage of precipitation in surface soil followed by later removal of the stored water by evaporation and transpiration may provide excellent performance, particularly in drier climates. Covers that function on this principle are described by a variety of names, including evapotranspiration covers, store-and-release covers and water balance covers.



FIG. 24. Schematic of resistive covers characterized by a low conductivity barrier of soil (compacted clay or a geosynthetic clay layer) or a composite of low conductivity soil and a geomembrane.

Figure 25 shows the two common configurations of evapotranspiration covers: monolithic and capillary barrier designs. The monolithic design consists of a thick layer of fine textured soil and the capillary barrier design consists of a similar layer with an overlying thinner layer of clean coarse grained soil. Both are vegetated and roots from the vegetation extend throughout the fine textured layer in both configurations.

4.2.3. Hybrid covers

Combinations of resistive barrier and evapotranspiration cover components might serve the requirements of some sites and might provide superior performance. Indeed, even in conventional final covers that rely on resistive



FIG. 25. Basic configurations of common monolithic and capillary barrier evapotranspiration covers.

barriers, much of the water balance can be managed by the vegetated surface layer using the store-and-release principles of evapotranspiration covers. Research shows that the percolation through composite covers occurs primarily during periods of lateral drainage on the geomembrane [67]. The frequency and quantity of lateral drainage may be controlled by improvements in the store-and-release mechanisms in the vegetated surface soil layer.

4.3. TECHNICAL BASIS AND DESIGN

4.3.1. Compacted clay covers

The functional basis for compacted clay covers is that a layer of fine grained soil can be placed with sufficiently low saturated hydraulic conductivity to resist the downward movement of water and that the low conductivity character of that layer will persist for the design life of the containment facility. An overlying vegetated soil layer is typically intended to protect the resistive barrier from erosion and the harmful effects of freeze–thaw and wet–dry cycles (Fig. 24). A drainage layer overlying the clay layer might be required to prevent excess
pore water pressure at the interface between the clay and the topsoil, as well as the possibility of slope failure.

Recent research casts considerable doubt on the ability of clay barriers to maintain their low conductivity properties when exposed to environmental effects near the surface. Wet–dry and freeze–thaw cycles and intrusions by plant roots, burrowing animals and insects invariably result in detrimental changes to material properties. These processes are of concern when compacted clay layers are used in cover applications; when used in basal liners, the clay layers are not subject to the same environmental stresses. Liner applications usually maintain the barrier layers at a constant temperature, there are little or no changes in water content (often liners are subject to positive hydraulic pressure from standing water) and there are no biological intrusions.

The saturated hydraulic conductivity (K_s) is the key design parameter of a compacted clay layer. A common requirement is $K_s \leq 1 \times 10^{-9}$ m/s; this property may be tested in the laboratory using ASTM D5084 [4], D2434 [5] or equivalent. Material characteristics correlated with low hydraulic conductivity include the percentage of fines, clay particles and gravel, maximum particle size and Atterberg limits. The borrow source investigation has to identify a suitable quantity of soil to meet the areal extent and thickness of the clay layer.

Construction specifications for compacted clay barriers are typically given in terms of barrier thickness (typically at least 450 mm), lift thickness (often 150 mm) and as-built saturated hydraulic conductivity. In general, construction practices for low conductivity layers include thin lifts of soil placed wet of the line of optimums, a high level of kneading compaction, and interlift bonding [11]. The purpose of these procedures is to remove all residual soil structure present at the borrow source and to remould the entire clay barrier into a monolithic structure free of large voids. A common specification for the compacted clay barrier in a cover is a 450 mm thickness comprised of three lifts, each 150 mm thick. Lifts are thin to ensure that the entire barrier is remoulded during compaction. For details on compacted clay layers, refer to Sections 3.2.1.1 and 3.2.2.

A vegetated layer of topsoil completes the design of a compacted clay cover. This layer should be thick enough to prevent damage to the clay layer by erosion and freezing and should provide a suitable habitat for the selected plant community.

4.3.2. Composite covers

The functional basis for composite covers is that a thick geomembrane overlying a low conductivity soil layer (either clay or a GCL) will resist the downward movement of water, and the service life of this composite barrier will equal or exceed the design life of the containment facility. An overlying vegetated soil layer protects the geomembrane from physical damage and the harmful effect of solar radiation while it also protects the underlying soil barrier from freeze-thaw cycles (Fig. 24). A drainage layer between the vegetated and composite layers is required to prevent excess pore water pressure at the interface between the barrier and the topsoil and the possibility of slope failure.

The resistive layers in composite covers are similar to those developed for bottom liners and reflect the co-evolution in design. Design considerations for composite covers include specifications of the material and the construction methods for the low conductivity soil layer and the geomembrane. As a manufactured product, geomembranes are much less variable than the soil component, thus there is more detail for material and construction specifications of the soil layer than for the geomembrane.

Specifications for the low conductivity soil layer are the same as for compacted clay barrier covers described in Section 4.3.1. The compacted clay layer in a composite cover has to be rolled smoothly before placement of the geomembrane to ensure intimate contact between the soil and synthetic layers.

Geomembranes are typically specified according to polymer type (e.g. HDPE, LLDPE), thickness and surface texture. Methods for the placement of geomembranes emphasize welding and testing of seams and preparation of the subgrade to prevent punctures. Field QA activities are critical to ensure that the construction practices achieve the desired material parameters (i.e. low hydraulic conductivity). A detailed description of construction methods and QA requirements for geomembrane barrier layers used in conventional covers can be found in Ref. [68].

4.3.3. Evapotranspiration covers

Evapotranspiration covers provide two essential functions in controlling the percolation of water: (i) the stored water is removed by evapotranspiration during periods of lesser precipitation and higher evaporation, and (ii) water is stored in the fine grained soil layer when precipitation exceeds evapotranspiration. The water balance for an evapotranspiration cover is illustrated in Fig. 26, with data from the field test section located on the central coast of California, USA. The graph shows the amount of water stored within the cover as well as precipitation and percolation. Soil water storage increases as water accumulates within the cover during the wetter (winter) season, and decreases as water is removed from the cover by evapotranspiration during the drier (summer) season. The total storage capacity of the cover is the water content at which percolation begins.

A common assumption is that the driest condition for the cover soil is the water content at which transpiration ceases due to the wilting of the plant community (the 'wilting point' of the soil). The quantity of stored water fluctuates



FIG. 26. The water balance of an evapotranspiration cover test section.

between maximum and minimum values on an annual basis and this describes the available storage capacity of the cover. The available water storage depends on the cover thickness and the hydraulic properties of the soil. Removal of the stored water depends on plant properties and the evaporative demand imposed by the climatic conditions of the site.

Differently from evapotranspiration, percolation occurs when the total storage capacity of the cover soil is exceeded. Percolation may also occur because of preferential flow through macropores in the soil.

The design of evapotranspiration covers differs significantly from conventional designs. Since there are no layers of the specified material property, there is no opportunity to assume that performance will be acceptable by basing the design on specific materials and construction methods. The design should begin with a description of the required performance, often in terms of a maximum (or mean) annual percolation rate. Determining the required performance can be challenging; multiple site specific factors typically need to be considered and no cover design will guarantee leak-free performance for the hazardous lifetime of the waste. A conceptual design (monolithic or capillary barrier) should be validated based on regional experience, modelling, availability of soils and climate information.

A borrow source investigation is then conducted to determine the availability of suitable cover soils and to characterize their hydraulic and agronomic properties. Data from the borrow source investigation are used to select a preliminary cover profile (soil types and layer thickness). A revegetation plan is developed for a plant community that will provide adequate transpiration, erosion control and ecological resilience. Ecological reference areas can suggest potential plant species for the borrow soil and act as a basis for developing a revegetation plan. Performance cannot be assumed based on material properties or cover thickness. Field test sections or computer simulations are therefore often used to predict the cover's performance, to compare different designs and to understand mechanisms important to performance. Informed by prediction, the design is refined to meet the performance requirements as well as other constraints (e.g. cost, physical stability and land use after the site's closure).

Even this very general description suggests that no single design for an evapotranspiration cover can meet the needs of all sites with variable performance criteria, different climate, soil and plant characteristics, and economic constraints. Evapotranspiration covers are not appropriate for all sites.

Evapotranspiration covers can be designed in two distinct phases: (i) a preliminary design to test the viability of an evapotranspiration cover for the site and to estimate the required thickness of the cover, and (ii) a refined design using computer models to assess various 'what if' scenarios. The preliminary design process uses climate factors to address how much water has to be stored (required storage) and soil data to address how much water can be stored (available storage) in the proposed cover profile.

4.3.3.1. Available storage

The total storage capacity (S_c) of a cover profile represents the soil water present when percolation is just about to occur and indicates the storage status of the soil. This signifies that any additional drop of water at the top of the cover will result in the percolation of a drop from the bottom. The volumetric soil water content corresponding to this situation is known as the field capacity water content (θ_c) and is determined by integrating the field capacity water content over the cover thickness (Fig. 27).

Not all of the total storage capacity is available for use because plants wilt before all water is removed from the soil and the soil never becomes completely dry. When transpiration stops, the soil water content is described as the 'wilting point'. The available storage capacity of a soil layer is the total storage capacity (when the entire cover profile is at field capacity) less the storage when the soil is at wilting point water content. Soil water content is typically expressed in volumetric units (i.e. the volume of water per unit volume of soil). However, precipitation is described in units of length (i.e. millimetres) and, for purposes of comparison, volumetric soil water quantities are expressed as per unit of the cover surface area resulting in units of length. The equation for this is shown as part of Fig. 27.



FIG. 27. Total soil water storage capacity (S_o) of a soil profile (indicated on the left) of thickness (L) is determined by integrating the water content profile at field capacity (θ_o) over the thickness of the layer. Available soil water storage (S_a) is S_c minus the water content at wilting point (θ_v) (i.e. the minimum soil water content).

Field capacity and wilting point are normally assumed to be the water content corresponding to suctions of 33 kPa and 1500 kPa, respectively. The relationship between water content and suction is described by the soil water characteristic curve (SWCC). The SWCC is determined with a laboratory procedure [69, 70]. Laboratory data are fitted to a curve using the van Genuchten equation, Eq. (3).

$$\frac{\theta - \theta_{\rm r}}{\theta_{\rm s} - \theta_{\rm r}} = \left\{ \frac{1}{1 + (\alpha \psi)^{\rm n}} \right\}^{\rm m}$$
(3)

where

- θ is the water content;
- ψ is the soil water suction;
- $\theta_{\rm s}$ is the saturated soil water content;
- $\theta_{\rm r}$ is the residual soil water content;
- α is a scaling factor;

and m and n are fit parameters of the model related to the shape of the curve.



FIG. 28. Soil water characteristic curves for a sandy soil (top graph) and a silt loam soil (lower graph).

Examples of SWCC curves fitted from laboratory data using the van Genuchten equation are shown in Fig. 28 along with calculated values for field capacity and wilting point.

4.3.3.2. Required storage

The soil water content of evapotranspiration covers ranges between annual high and low values (Fig. 26), and the difference represents the available storage. Percolation occurs when the annual required storage exceeds the available storage. The fine textured soil of the cover acts as a sponge to store precipitation for later release to the atmosphere by evaporation and transpiration. Percolation occurs when soil water storage exceeds total storage capacity (indicated by the

dotted line in Fig. 26). At the site in Fig. 26, actual storage exceeded storage capacity each year during the wet winter seasons. The result was significant percolation each year. Percolation can be minimized by increasing total storage capacity to account for the indicated required storage capacity.

Various methods for estimating required storage are employed, most expressed as a defined fraction of precipitation on an annual or seasonal basis. For example, a conservative estimate for required storage might be the quantity of precipitation that occurs during a winter season when plants are dormant and transpiration is low or non-existent.

Computer models can be used to estimate the annual range in soil water content of a cover. Another method to estimate required storage was developed from field studies at 12 sites in different climates [70]. The primary advantage of that method is that it is entirely derived from field data. However, all of the field sites were in the USA and differences in soil, plant and climate factors at other locations might limit the value of this method for sites elsewhere. The application of the method to sites in eastern Australia did not accurately predict performance. This could be because of much higher surface runoff rates caused by intense precipitation events common to the region. The final determination of required storage is often a result of negotiation between site owners and regulators with appropriate consideration of the risk to human health and the environment.

4.3.3.3. Calculation of cover thickness

With estimated values for required storage and soil hydraulic properties, the minimum required cover thickness (L) is computed by equating the available and required storage water content as shown in Eq. (4):

$$L \ge \frac{S_{\rm r}}{\left(\theta_{\rm c} - \theta_{\rm m}\right)} \tag{4}$$

where

- *L* is the minimum required cover thickness;
- $S_{\rm r}$ is the required storage water content;
- $\theta_{\rm c}$ is the field capacity water content;

and $\theta_{\rm m}$ is the minimum soil water content.

4.3.3.4. Available storage and thickness for capillary barrier covers

Capillary barrier designs comprising a fine textured layer over a coarse textured layer (Fig. 29) increase water storage by increasing the water content at the bottom of the layer at which percolation first occurs [71].

Figure 30 shows the SWCCs for a fine textured and a coarse grained soil. Owing to a capillary break, the SWCCs for a finer and coarser soil show an increase in the storage capacity. The finer textured soil without the underlying coarse layer would drain at field capacity water content (θ_c). With a capillary break, percolation does not occur until the suction at the interface of the finer and coarser layers reaches ψ_B . This results in higher water content (θ_{BF}) and increased storage in the finer layer at breakthrough.

At 33 kPa (suction corresponding to field capacity), the water content of the coarser layer is very low. This low water content corresponds to a low hydraulic conductivity, which prevents percolation from the finer layer. As a result, water content increases at the bottom of the finer soil layer where percolation occurs. An appreciable flow of water into the coarser layer will not occur until the suction at the interface between the layers drops to the breakthrough suction $\psi_{\rm B}$ for the coarser soil (see Fig. 30). For the coarser soil, the breakthrough suction $\psi_{\rm B}$ corresponds to the point near the 'elbow' in the SWCC where the water content begins to increase as the suction diminishes.

The addition of a coarse layer beneath the fine textured layer can increase the water storage of the finer layer by the difference in water content corresponding to ψ_c and ψ_{BF} before appreciable percolation from the finer layer will occur [72]. Low breakthrough suction in the coarser layer requires relatively large pores (low air entry suction) and narrow distribution of pore sizes. Accordingly, clean and uniformly graded coarse grained soils (sands and gravels) are used to create capillary breaks.



FIG. 29. Schematic of a capillary break illustrating the layering of fine soil over coarse soil. The capillary break is created by the contrast in hydraulic properties at the interface.



FIG. 30. Soil water characteristic curves for a finer and coarser soil show the increase in storage capacity created by the capillary break.

4.4. PERFORMANCE

4.4.1. Compacted clay covers

Compacted clay covers are sometimes recommended for the closure of sites that are unlined or lined with clay. Common experience shows that these fine grained soils resist the passage of water and extensive research indicates that low conductivity layers can be constructed in the field given proper material selection and construction practice (see Section 4.3.1). In the as-built condition, compacted clay covers may limit percolation to acceptable levels. However, cover soils are exposed to environmental conditions (e.g. wet-dry and freeze-thaw cycling, biological intrusion by plant roots and burrowing animals) that can damage soil barrier layers and affect performance. Field studies indicate that percolation rates from clay barrier covers can increase significantly within periods of time (several months to a few years) that are considerably shorter than the expected design life of modern waste containment facilities [73]. Most of the sites listed in Table 10 tested clay barriers compacted to achieve $K_s \leq 1 \times 10^{-9}$ m/s. A soil layer with this property, continuously saturated (unit gradient condition) will allow percolation of around 32 mm/a. The field results shown in Table 10 indicate significant damage to some of these layers within just a few years. Data from the field-scale test section (Fig. 31) show the effect of preferential flow through a clay barrier in less than one year following construction. Cracks that developed in the

Location and reference	No. of test years	Annual precipitation (mm)	Annual drainage (mm)			
Cool and humid sites						
Cedar Rapids, IA, USA [74]	2	784–1182	94–171			
Milwaukee, WI, USA [75]	4	578-896	1.5-60			
Hamburg, Germany [76]	8	714–1032	1.9–201			
Kalamazoo, MI, USA [77, 78]	7	795–1109	16–70			
Reedsburg, WI, USA [79, 80]	3	780–929	17–386			
Warm and humid sites						
Albany, GA, USA [74]	3	298–996	49–292			
Atlanta, GA, USA [81]	3	3 1188–1721 30–150				
Arid and semiarid sites						
Apple Valley, CA, USA [74]	2	86–351	0–22			
Albuquerque, NM, USA [82]	5	144–300	0–3.56			
Wenatchee, WA, USA [81]	3	140-260	2–22			

TABLE 10. DRAINAGE RATES FROM LYSIMETER BASED FIELD STUDIES ON COMPACTED CLAY BARRIER FINAL COVERS

barrier from the first significant drying event did not heal and resulted in greatly increased percolation even with increased water content in the clay barrier.

Data such as those shown in Table 10 and Fig. 31 cast considerable doubt on the long term effectiveness of compacted clay covers for waste containment. Figure 31 shows that a drought occurred just six months following construction and resulted in the first significant desiccation of the clay barrier. When precipitation resumed, the percolation rate increased and followed a 'stair step' pattern indicating the development of preferential flow paths. When combined with an overlying geomembrane in a composite cover or basal liner, the clay barrier layer is isolated from desiccation damage and biological intrusion and may adequately serve the intended purpose.



FIG. 31. Data from a field test section of a compacted clay cover.

4.4.2. Composite covers

Leakage through composite barriers is well documented despite the extremely low conductivity of the geomembrane materials. Post-construction surveys have noted the occurrence of holes, at 1–12 holes/ha [38, 83]. Reference [84] suggests that proper installation with strict construction QA can limit holes to 5 holes/ha. Holes can result from manufacturing defects, improper handling during transportation, placement, seaming, wrinkles, and placement of overlying material. Geomembranes are typically placed over a low conductivity soil barrier. The contact between the materials is important. Imperfect contact provides a transmissive layer; flow through a geomembrane defect can move laterally before percolating through the soil barrier.

As manufactured materials, geomembranes have been available for only a few decades, thus the long term lifespan of these materials could not be directly observed. Rowe [83] summarized studies to date of various factors relating to the degradation of geomembranes and estimated service life exceeding 500 years in environmental conditions typical for covers.

The geomembrane that overlies the soil barrier in composite designs eliminates most biotic intrusion and protects the soil layer from fluctuations in water content caused by evapotranspiration [76]. However, geomembranes do not protect the clay component from damage caused by freeze–thaw cycling [85] or from water content changes caused by variations in thermodynamic conditions beneath the cover.

The data from test sections using drainage lysimeters were collected and analysed [67, 76, 82]. The field scale studies of composite covers at several sites

were conducted in large drainage lysimeters that provide the only method for direct measurement of percolation. The combined results revealed that for annual precipitation of less than around 400 mm/a, percolation is negligible; for higher annual precipitation rates, the percolation rate increased approximately linearly with higher annual precipitation. At one site, the geomembrane was damaged during construction and percolation was as high as 45 mm/a. When constructed using methods that minimize the potential for damage to the geomembrane, the average annual percolation was limited to approximately 3 mm/a. Much higher percolation rates are realized when the cover is constructed with methods that promote the puncture of the geomembrane. An example of improper geomembrane installation in a test section is the damage caused during installation by the surface soil layer being placed directly onto the membrane (i.e. without a layer of geotextile or a drainage layer). In such cases, the studies mentioned above demonstrated that over the first few years of monitoring, the investigated cover allowed 20-50 mm/a of percolation. This finding demonstrates the importance of construction methods and QA for covers that include a geomembrane barrier.

4.4.3. Evapotranspiration cover

The variation of annual percolation versus annual precipitation recorded in lysimeter-based test sections from several field studies [86] and data from Refs [67, 76, 82] were analysed. It has been found that percolation from the test sections varied widely depending on annual precipitation, design variables (available storage and potential evapotranspiration) and other factors such as preferential flow.

At sites with low annual precipitation (<250 mm/a), percolation was very low (<5 mm/a) because of the low requirement for water storage in the cover profiles and high potential evapotranspiration. At sites with high annual precipitation (>750 mm/a), percolation rates typically exceed 100 mm/a because of the larger amount of water to be managed relative to the energy available for evaporation and transpiration. At sites with moderate annual precipitation (250–750 mm/a), percolation rates ranged from negligible (<1 mm/a) to exceeding 100 mm/a, and were sensitive to design conditions. An important note is that these data are from test sections used in research on evapotranspiration covers.

4.4.4. Processes for change

The physical properties of earthen and geosynthetic materials used in final covers change over time in response to interactions with the surrounding environment. Alterations in cover materials are to be expected. Some materials experience small changes in engineered properties (e.g. geomembranes); other materials undergo more significant changes (e.g. densely compacted clay barriers). During the construction, final covers are not in equilibrium with the continuously fluctuating surrounding environment. This requires alterations to the cover materials until an equilibrium condition is attained or the temporal changes in the cover become consistent with the temporal changes of the surrounding landscape. These aspects need to be considered by the designer.

Benson et al. [73] reported on exhumations of final cover test sections that had been in service for 4–8 years. Both soils and geosynthetic materials were evaluated for changes in material properties. The in-service age of the materials was short compared with the typical expected design life for final covers, but the results can be extrapolated to guide those who design and evaluate final covers. Points to consider can be summarized as follows:

- Owing to processes such as wet–dry and freeze–thaw cycling, the saturated hydraulic conductivity (K_s) of earthen barrier and storage layers will increase with time and the layers having lower as-built K_s experience larger increases. Increases will occur until K_s is in the range of approximately 8×10^{-8} to 6×10^{-6} m/s. The changes occur regardless of climate, cover profile, or placement condition.
- Soils with a greater fraction of coarse particles will be more resistant to changes in soil structure and hydraulic properties. For practical purposes, earthen storage and barrier layers should be constructed using soils containing a broad range of coarse and fine particles along with a modest amount of clay sized particles.
- Equilibrium porosity and dry unit weight can be defined using the data from local analogue sites. Compaction near optimum water content is recommended and compaction wet of optimum water content is to be avoided.
- The barrier layers containing GCLs in the final covers are altered owing to the replacement of the native sodium in the bentonite by the divalent cations present in the environment.
- When ion exchange is combined with wet–dry cycles, GCLs have the potential to become much more permeable, with K_s of 10^{-8} to 10^{-6} m/s. Geosynthetic clay liners should be protected with a geomembrane and positioned on a subgrade with an initial water content exceeding 10%. If the overlying geomembrane remains intact, the bentonite will undergo osmotic swell and retain low K_s (<5 × 10^{-11} m/s) even with ion exchange implying that the life of the GCL is controlled by the service life of the geomembrane.
- A detailed study on the behaviour of the material properties of geosynthetics (geomembranes and geosynthetic drainage layers) over a long period shows a minor change compared with earthen materials or GCLs. This finding is

consistent with other reports. Antioxidant depletion rates observed in this study and by others can be used to compute the minimum service life of geomembranes. The antioxidant depletion rates show that the minimum lifespan of geomembranes is 55–125 years depending on the type of polymer employed. A similar minimum lifespan based on long term prototype tests has been reported by Rowe et al. [87]. This computation predicts that the service life of the geomembrane ends when the antioxidants are depleted. However, the actual lifetime should be longer, and methods to estimate the actual lifespan are found in Koerner [88] and Rowe et al. [89]. A periodic inspection and the replacement of geosynthetics is recommended.

4.5. MONITORING FINAL COVERS

Monitoring is often required to ensure that the cover is functioning as predicted. This is best conducted by continuously monitoring the primary performance variable (percolation) using in situ devices (direct monitoring) and monitoring secondary variables related to the primary performance variable that can be used to understand and interpret the percolation data (interpretive monitoring).

Pan lysimeters (or drainage lysimeters) provide the only method for direct monitoring of percolation. A pan lysimeter consists of a large collection surface, often made of geomembrane, placed beneath a portion of the cover (or as a separate test section) to collect percolation. Water collected in the pan is piped to a monitoring station where the flow is measured. Lysimeters are advantageous because they provide a large scale passive measurement of the percolation rate with minimal maintenance. The disadvantages of lysimeters include higher installation costs compared with other monitoring methods and the impact of the artificial boundary imposed by the lysimeter pan.

A capillary break formed by the drainage layer at the base of the lysimeter is considered the most significant technical issue associated with lysimetry. This feature enhances storage within the cover profile and may reduce the percolation rate. However, this issue may be addressed by including a geosynthetic root barrier and a layer of interim cover soil between the lysimeter membrane and the cover soils.

The lysimeter should be sufficiently large to provide a reliable spatially averaged measurement. The minimum dimension of the lysimeter therefore needs to be at least three times the spatial correlation length of the hydraulic properties of the cover soils [90]. Engineered fill soils have a spatial correlation length of 1-3 m [90], thus the suggested minimum dimension of a lysimeter is

no less than 9 m. The lysimeter walls should also be designed so that they do not induce focusing or divergence of flow.

Percolation measurements from a lysimeter provide a highly accurate indication of the overall performance of the cover but alone are often not sufficient to answer the question of why the cover performs as observed.

Additional information regarding soil water content in the cover profile, the climate and the plant community on the cover is needed for a more complete understanding. The final cover monitoring using a lysimeter is especially important when direct monitoring is unsatisfactory (e.g. when the percolation rate is higher than a design standard) [91].

Without data from secondary variables, the root cause of unsatisfactory performance can be difficult or impossible to determine, which hampers the selection of an appropriate remedy. Water content probes at multiple depths in the cover profile provide knowledge of the status of the soil water. A meteorological station should include, as a minimum, instruments to measure precipitation, temperature and humidity. Periodic vegetation and reconnaissance surveys should also be conducted, particularly for evapotranspiration covers that rely strongly on vegetation to manage the water balance.

5. VERTICAL CONTAINMENT BARRIERS

5.1. INTRODUCTION

5.1.1. Definition

Vertical containment barriers are media with low permeability that are either inserted into the subsurface or constructed in situ generally to control regional or local groundwater migration or seepage through porous media (e.g. earthen dams). These low permeability barriers are in contrast to permeable reactive barriers, referred to in this publication as PRBs, which are zones of high permeability typically constructed in situ that contain reactive media to treat a contaminated waste stream, such as a contaminated groundwater plume, as it passes through the reactive zone. Since the primary objective of vertical barriers is hydraulic control and environmental (chemical) containment, whereas the objective for PRBs is treatment, the design considerations for these two types of barriers are different. As a result, descriptions of each of these two types of barriers are considered separately, with vertical barriers the focus of this section and PRBs the focus of Section 6.

5.1.2. Applications of vertical containment barriers

Vertical containment barriers are used primarily for the subsurface hydraulic control of seepage or groundwater and have been used extensively for both traditional (>50 years) civil engineering applications and more recently (the past 30–40 years) for environmental applications. Examples of traditional civil engineering applications include controlling the localized seepage of uncontaminated groundwater into engineered subsurface excavations for construction activity and controlling seepage and pore water pressures in and beneath earthen dams for water storage [92, 93]. Environmental applications inevitably involve the containment of a liquid waste stream (e.g. leachate derived from disposed waste, chemical solution stored in a lined holding pond) to prevent subsurface contamination, or containment and/or control of an existing contaminated liquid (e.g. an existing contaminated groundwater plume).

In the case of traditional civil engineering applications, the primary emphasis is on the hydraulic control of water, whereas in the case of environmental applications, the primary emphasis is on the control of pollutants dissolved in water. Although the terms 'contaminant' and 'pollutant' are typically used interchangeably, there is a slight distinction between the two terms [94]. The term 'contaminant' may be considered as a chemical that represents a potential environmental or health concern but has not yet exceeded a specified or stipulated environmental regulatory threshold value, such as a regulated drinking water standard (known as DWS) or a maximum contaminant level (known as MCL). The term 'pollutant' may be considered to be a contaminant that has exceeded such a threshold value. Despite this distinction, the two terms are commonly used interchangeably.

The emphasis in environmental applications on the control of contaminants requires that the design and evaluation of vertical barriers be based on contaminant migration considerations, which include not only the transport processes such as advection (hydraulic driven transport) and diffusion (chemically driven transport), but also the chemical processes (e.g. dissolution/precipitation, acid/base, oxidation/reduction) and biological processes (e.g. biodegradation, biotransformation) that may affect the 'fate' of the contaminants during transport through the porous media.

These contaminant migration considerations are often referred to collectively as 'fate and transport'. Thus, when designing and evaluating vertical barriers for environmental applications, it is necessary to consider not only the application of Darcy's law and the hydraulic conductivity of the barrier material to describe or predict the rate of liquid flow or seepage through the vertical barrier, but typically also contaminant migration via Fick's laws for diffusion as well as the potential effects of a wide variety of chemical and/or biological processes.

5.1.3. Vertical containment scenarios

Three containment scenarios for vertical barriers associated with mine tailings impoundments are illustrated in Figs 32 and 33 with respect to circumferential, upgradient and downgradient aspects. In each of these three scenarios, extraction wells (or drainage systems) can be employed to remove contaminated groundwater and to control the direction of the local hydraulic gradient to enhance the overall containment efficiency of the system.

In the circumferential containment scenario (Figs 32 (a) and 33 (a)), the entire area representing the potential source of subsurface contamination is encircled by a vertical barrier, such that the regional groundwater flow is diverted around the contained area rather than through it to minimize the escape of contaminated groundwater. The placement of extraction wells between the tailings impoundment and the vertical barrier at specified locations provides for enhanced containment via control of the local hydraulic gradient within the contained area, and the removal of any contaminated groundwater.

The upgradient scenario (Figs 32 (b) and 33 (b)) is similar to the circumferential scenario, except that the barrier does not entirely encircle the contaminated area. Instead, the barrier is placed only upgradient of the contaminated area such that the regional groundwater flow is still diverted around the source of contamination. In this case, the placement of the vertical barrier requires groundwater modelling to ensure that the diverted groundwater does not intercept any contaminated groundwater coming from the source of contamination. Vertical wells are again located between the vertical barrier and the source of contamination both for enhanced containment via hydraulic gradient control and for the removal of any contaminated groundwater. However, the pumping rates from these wells would likely need to be higher than those for the circumferential scenario to ensure that the local groundwater flow beneath the entire footprint of the source of contamination is directed towards the wells.

The downgradient scenario, whereby the vertical barrier is placed only downgradient of the potential source of contamination (Figs 32 (c) and 33 (c)), is fundamentally different from the circumferential and upgradient scenarios in that the primary purpose of the downgradient scenario is the collection and the removal of any contaminated groundwater — as opposed to the diversion of regional groundwater flow — to minimize the migration of contaminants downgradient scenario, the vertical barrier has to be sufficiently long to encompass any potential lateral spreading of groundwater contamination. Furthermore, vertical wells or drainage systems between the barrier and the source of contamination may be required to ensure that contaminated groundwater does not accumulate behind the barrier such that the containment is compromised via overtopping or lateral divergence.



FIG. 32. Plan (aerial) views of vertical containment scenarios with tailings impoundments: (a) circumferential; (b) upgradient; (c) downgradient.



FIG. 33. Cross-sectional views of vertical containment scenarios with tailings impoundments: (a) circumferential; (b) upgradient; (c) downgradient.

5.2. TYPES OF VERTICAL CONTAINMENT BARRIER

Vertical barriers may be conveniently separated into two types: those based with slurry and those that are not. Slurry based barriers are constructed in place via trenching and involve a slurry based on bentonite (i.e. a suspension of bentonite in water) that assists primarily in maintaining trench stability during construction but also imparts some of the desired rheological and engineering properties to the barrier materials. Any vertical barrier that does not involve a slurry based on bentonite can include barriers constructed by inserting structural units into the subsurface, such as driving sheet-piles (sheet-piling), or by injecting low permeability substances into the subsurface, such as cement grouts. For environmental applications, the use of barriers based on slurry predominates. The emphasis in this subsection is therefore placed on these types of barriers, although some aspects of non-slurry based barriers are also described for completeness.

5.2.1. Slurry cut-off walls

There are three general types of vertical containment barriers based on slurry, also known as slurry cut-off walls (SCOWs): (i) soil-bentonite (SB) cut-off walls; (ii) cement-bentonite (CB) cut-off walls; and (iii) soil-cement-bentonite (SCB) or plastic concrete cut-off walls [95–97]. All three types generally involve excavating a trench and placing a bentonite-based slurry within the trench to maintain trench stability. Differences among these three types of SCOWs include whether the stabilized trench is backfilled with trench spoils (SB and SCB) or not (CB), and whether cement is used in the wall construction (CB and SCB) or not (SB). Both SB and CB walls have been used more extensively than SCB walls, which are a relatively recent development. Soil-bentonite walls have traditionally been more popular in North America [98, 99], whereas CB walls have traditionally been more popular in Europe [100].

All three types of SCOWs are constructed using trenching technology, whereby a trench is first excavated to the desired depth, typically using a backhoe for shallower depths or a clamshell for deeper depths. Depths of 50 m are possible in some cases [101], although much shallower depths (<30 m) are more common, especially for CB and SCB SCOWs. The width of SCOWs is generally the same as the width of the trenching equipment used in the construction, which is typically 1 ± 0.5 m, although other widths are possible.

The types of trench excavation equipment typically used and the associated widths and depths of the trenches are summarized in Table 11 (adapted from Ref. [102]).

The bottom of the trench is usually excavated or 'keyed' 1 m into an underlying low permeability stratum, such as clay (e.g. aquitard, aquiclude)

or bedrock, to minimize the possibility of contaminants migrating beneath the SCOW. When the depth to such a low permeability stratum is much deeper than the depth of the SCOW required to control or contain the contamination, such keying may be problematic and not required, resulting in what is commonly known as a 'hanging' SCOW (Fig. 34).



FIG. 34. Cross-sections of slurry cut-off walls: (a) a hanging slurry cut-off wall; (b) a keyed slurry cut-off wall.

Type of equipment	Common trench dimensions		Commonto	
	Width (m)	Depth (m)	- Comments	
Standard backhoe	0.3–1.5	≤15	Most rapid and least expensive method	
Modified backhoe	0.6–1.5	≤24	Modified with an extended dipper rod, modified engine and a counterweighted frame; also rapid and relatively inexpensive	
Clamshell	0.3–1.5	>46	Attached to a kelly drive or appropriate MG crane; can be mechanical or hydraulic	
Dragline	1.2–3.0	>36	Primarily used for wide and deep SB trenches	

TABLE 11. EXCAVATION EQUIPMENT COMMONLY USED FOR SLURRY TRENCH CONSTRUCTION^a

^a Adapted from table 5-4 of Ref. [102].

In the case of SB and SCB walls, the stability of the opened trench is maintained by filling the trench with bentonite–water slurry to a level within the trench that is higher than the surrounding groundwater level, whereas, in the case of CB walls, the trench stability is maintained by filling the trench with a bentonite–cement–water slurry. In both cases, the elevated slurry level within the trench drives liquid migration towards the outer sidewalls of the trench, such that a thin layer of low permeability bentonite or cement–bentonite, known as a 'filter cake', develops within the soil adjacent to the trench sidewalls. The thickness of this filter cake varies from a few to several millimetres, depending on the texture of the surrounding soil, with thinner filter cakes for finer textured soils, such as clays and silts, and thicker filter cakes for coarser textured soils, such as sand and gravels. The low permeability filter cake offers hydraulic resistance to continued lateral liquid migration, such that the pressure of the slurry within the trench against the filter cake serves to stabilize the trench and prevent it from collapse.

The bentonite–water slurry used to construct SB and SCB walls generally consists of 2–6% by dry weight of an approved sodium or sodium activated bentonite (calcium bentonite that has been processed to replace exchangeable calcium cations with sodium cations) suspended within potable water of relatively low ionic strength to prevent flocculation and settling of the bentonite particles. In some cases where only non-potable or poor-quality water is available, the addition of a dispersing or deflocculating agent (e.g. sodium hexametaphosphate $(NaPO_3)_6$) may be required to assist in minimizing the flocculation, although this additive increases the cost of the technology. Alternatively, a less active clay than sodium bentonite, such as attapulgite clay (palygorskite), may be considered for use, especially when the SCOW is to be exposed after construction to water with high ionic strength or a concentrated waste stream [103, 104].

The actual amount of bentonite to be used for the slurry depends on the desired rheological properties, such as the viscosity and the density of the slurry. The viscosity and the density are typically measured following Recommended Practice 13B-1 of the American Petroleum Institute (API) [105] using a mud balance and a Marsh funnel viscometer or 'Marsh cone', respectively.

The bentonite and water are blended via high colloidal vortex mixing and left to hydrate for a minimum period prior to testing, usually at least 24 hours and preferably 48 hours. Typical desired values for density and viscosity are 1.05 mg/m³ and 40 s Marsh, respectively. The density of the slurry after placement in the trench is likely to be higher than that of the freshly mixed slurry prior to placement, because additional particles of excavated materials typically fall into and become suspended in the slurry during trench excavation and simultaneous placement of the slurry [13, 102]. The density of the placed slurry may be 30% higher (or more) than that of a freshly mixed slurry [102].

Two other important properties of bentonite slurries include the filtrate loss and the pH of the slurry. The filtrate loss test illustrated in Fig. 35 is conducted to determine if the slurry has sufficient bentonite to produce a filter cake and the associated amount of slurry loss in forming a filter cake, and is typically measured following API Recommended Practice 13B-1 [105] using a filter press. The pH is measured using a pH meter and electrode; in general, the lower the pH, the greater the tendency for the bentonite particles within the suspension to flocculate and settle out of suspension, which is an undesirable characteristic, although such flocculation is usually prevalent at relatively low pH values (e.g. pH <4).

Finally, another property of bentonite slurry is the gel strength, which is related to the thixotropy of the slurry. When bentonite slurry is allowed to stand undisturbed for a few minutes, the slurry changes from a viscous solution to a substance that behaves like a gel. Upon agitation or mixing, the gel-like substance reverts to a viscous solution. Each time agitation of the slurry stops, the gel-like behaviour resumes — a process referred to as thixotropy. Thixotropy is important because it is the gel structure of the slurry that keeps the particles of the trench spoils suspended within the slurry [102].

The gel strength represents the minimum shear stress required to cause the flow of slurry that acts as a Bingham fluid (Fig. 36), and a measure of gel strength over time is a measure of the thixotropic behaviour of the slurry. The degree of thixotropy of slurry is considered to increase as the rate of gel formation and the magnitude of gel strength increase. For this reason, the gel strength of a slurry is typically measured using a Fann rotational viscometer at 10 s and 10 min. The 10 min gel strength should be only slightly greater than the 10 s gel strength when high quality bentonite is used [102]. Typical values of gel strength are around 8 Pa [102, 106].

Examples of bentonite slurry properties as a function of the amount of conventional sodium bentonite (CSB) or unconventional, polymerized bentonite (known as PB) can be seen in Ref. [107]. The authors propose that less polymerized bentonite is required to achieve the necessary Marsh viscosity of 40 s relative to the amount of CSB required (i.e. 2% polymerized bentonite versus 5% CSB). Thus, the amount of polymerized bentonite required would be 2.5 times lower than the amount of CSB, thereby providing cost savings, although the unit cost of polymerized bentonite would undoubtedly be higher than that of CSB.

Additionally, the density of the two investigated bentonite slurries increases as the bentonite content increases, and there is virtually no difference in density between the two different bentonites. The polymerized bentonite slurries result in substantially lower filtrate losses relative to those for the CSB. Finally, it could be observed that the pH is essentially independent of the bentonite content,





FIG. 35. Filtrate loss test: schematic of testing apparatus and procedure (top); photograph of the apparatus (bottom).



FIG. 36. Flow curve for a Bingham fluid and associated parameter values.

although the pH of the polymerized bentonite slurries is two units lower than that of the CSB slurries.

Unlike SB and SCB SCOWs, where the bentonite–water slurry within the trench is displaced by backfilling during construction, the bentonite–cement–water slurry for CB SCOWs is left to cure or harden (set) in place without adding backfill. Thus, in the case of CB SCOWs, the trench spoils resulting from excavation are to be discarded, possibly within a controlled waste containment facility if the trench spoils are contaminated, and the properties of the hardened bentonite–cement–water slurry will represent the properties of the CB SCOW in service. Generally, the procedure involves first preparing a bentonite–water slurry in the same manner as previously described for SB and SCB SCOWs, and then slowly adding the cement or other pozzolanic material such as blast furnace slag and pulverized fuel ash (e.g. bottom and fly ash from coal combustion) under continuous agitation to avoid agglomeration or lumping.

The addition of fly ash has been shown to maintain better water tightness and to resist disintegration [106], and the use of pozzolanic materials such as fly ash and blast furnace slag may assist in reducing the amount of cement required, thereby reducing cost (i.e. assuming a source of such materials is conveniently located near the construction site). Also, a curing retarder is typically added during construction as a small percentage of the overall mixture (e.g. 0.1%) to delay the initial setting of the mixture so that the placed slurry does not set up (harden) too quickly to allow for efficient and effective construction [106].

Different amounts of cement and/or pozzolanic materials may be added to achieve different values of the desired engineering properties of the CB SCOW, such as strength and hydraulic conductivity. However, the final compositions of the bentonite–cement–water slurries, exclusive of the addition of pozzolanic or other materials (e.g. fly ash or aggregates entering the trench during construction as a by-product of the excavation process), typically include 2–4% bentonite and 15–25% cement [95, 107], although other compositions are possible. Higher cement compositions generally result in higher strengths upon setting, but will also result in more rigid (brittle) CB SCOWs with potentially greater susceptibility to cracking and ultimately poorer performance from a liquid containment perspective.

Because the curing process associated with bentonite-cement-water slurries is time-dependent, the engineering properties of the CB SCOW are also time-dependent. In general, the compressive strength of the bentonite-cement-water slurry increases with time, whereas the hydraulic conductivity of the bentonite-cement-water slurry decreases with time, as illustrated in Fig. 37. Based primarily on the concrete industry, the maximum compressive strength and the minimum value of hydraulic conductivity are generally achieved within 28 days of curing, with values after 7, 14 and 21 days of curing typically representing approximately $60\% \pm 10, 80\%$ \pm 10, and \geq 90% of these limiting values, respectively. For this reason, the actual amounts of cement and other materials to be used in the CB SCOW are based primarily on testing for compressive strength and/or hydraulic conductivity of cylinders comprised of different compositions that have been cured for 7, 14, 21 and/or 28 days, with the final composition to be used in the construction of the CB SCOW based largely on the composition that provides the desired values of maximum compressive strength and/or minimum hydraulic conductivity. However, other considerations such as the chemical resistance (compatibility) of the CB mixture relative to the actual liquids to be contained may require the composition to be modified.



FIG. 37. Effect of curing time on properties of bentonite–cement–water slurries: (a) compressive strength; (b) hydraulic conductivity.

As previously noted, and in contrast to CB SCOWs, the slurry within the stabilized trenches for SB and SCB SCOWs is displaced by backfilling, such that the final properties and performance of SB and SCB SCOWs are largely a function of the backfill component of the SCOW.

The procedures for placing an SB or an SCB backfill in a stabilized trench are identical, except that the inherent characteristic of the hardening, owing to the presence of the cement in the SCB backfill, usually requires placement of the SCB backfill within a few hours of mixing, whereas the placement of an SB backfill is essentially independent of time [108].

The backfill placement procedure is initiated by lowering the backfill to the bottom of the trench (e.g. with a clamshell bucket) or by placing the backfill below the slurry surface with a tremie pipe (similar to a very long funnel) until the backfill rises above the surface of the slurry trench at the starting point of the trench [109]. Afterwards, additional backfill is typically pushed into the trench with a bulldozer, such that the viscous backfill sloughs downward and displaces the slurry in the trench.

The backfill typically comprises the trench spoils that have been mixed with the bentonite–water slurry to achieve a desired slump for the backfill before placement into the slurry fills the trench. If the trench spoils contain little or no fines (i.e. silt and/or clay soil particles less than the No. 200 or 75 mm sieve), dry sodium bentonite may be added to the trench spoils to decrease the hydraulic conductivity of the backfill and, in the case of SCB SCOWs, dry cement is added to the trench spoils to increase the strength of the backfill.

Other amendments to the backfill, such as chemically reactive media, may also be considered to improve the attenuation properties of the backfill for specified contaminants, provided these other materials do not adversely impact the other desired properties of the backfill such as the slump, strength, compressibility and/or hydraulic conductivity [110–112]. In cases where the SCOW is constructed within a contaminated subsurface, the excavated trench spoils are not likely to be suitable for use as backfill and have to be discarded or disposed of as previously noted for CB SCOWs. In this case, the base soil for the backfill has to be imported, typically at an additional cost.

Although the cement component of SCB backfills increases the strength of these backfills relative to that for SB backfills, the mixing, handling and overall construction and construction QC of SCB backfills are more complicated than those for SB backfills for at least two reasons [108]. First, the time-dependent effects of the curing process for the cement component of SCB backfills (Fig. 37) mean that these backfills will set up and become more difficult to handle with time, such that SCB backfills typically have to be placed within the slurry filled excavated trenches sooner than SB backfills are. Second, the conflicting actions of the various components of SCB backfills complicate the design of

these structures, leading to the fact that the properties of SCB backfills are more variable than those of SB backfills. As a result, the design of SCB backfills is more complicated than SB backfills. One method for achieving better quality during mixing is to add the cement in the form of a premixed grout, rather than adding dry cement, because liquid grout is much easier to mix thoroughly with soil than dry cement is, and may have other technical advantages) [108]. However, careful proportioning and mixing needs to be practised because Portland cement can represent 50% of the installation cost. Also, the use of a cement grout for mixing with the backfill will require a grout plant in the field.

As previously mentioned, the trench spoils and any additives are mixed (sluiced) with slurry (grout) to achieve a desired slump for the backfill to facilitate placement into the slurry filled trench. Similar to the case of standard concrete mixing, slump testing is required and is typically performed using a standard slump cone in accordance with a standardized procedure, although a miniaturized slump cone has recently been proposed to minimize the amount of material required to perform the test [113]. The standardized cone is filled with the mixed backfill, the cone is removed, and the unconfined backfill slumps because of a lack of confinement. The distance from the top of the slumped backfill to the top of the cone is measured as the slump, or $-\Delta H$ (>0).

In the case of SB backfills and SCB backfills that include dry cement, a bentonite slurry is mixed (sluiced) with the backfill to give it the desired workability (slump). However, as previously noted, in the case of SCB backfills where adding dry cement results in mixing difficulties, both the cement content of the backfill and the desired slump may be achieved by mixing the backfill with a bentonite–cement grout. In general, the greater the amount of slurry or grout mixed with the trench spoils and additives, the higher the gravimetric water content of the backfill and the greater the slump.

Thus, the slump $(-\Delta H)$ increases with increasing water content, and the relationship between slump and backfill water content (w_B) , referred to as a 'slump curve', tends to be linear [114]. The desired slumps are generally in the range of 50–150 mm for SB backfills and 100–200 mm for SCB backfills [97, 108]. These ranges of slump have been found to provide sufficient stiffness for the backfill to be able to displace the slurry within the trench, while maintaining sufficient pliability to allow the backfill to flow without the likelihood of developing 'windows' of higher permeability zones within the wall that are not filled with backfill.

The water content required to achieve this range of slumps typically increases with the increase in the number of fines contained within a sand–clay backfill (or the amount of dry bentonite contained within a sand–bentonite backfill, as clays in general and bentonite in particular are hydrophilic materials. Since the water content for the backfill is achieved by mixing the backfill with slurry, increasing the water content also increases the bentonite content and, in the case of SCB backfills mixed with a cement–bentonite grout, the cement content beyond the amounts that would be added as dry constituents to the backfill before mixing. For example, based on the slump test results, the bentonite contents corresponding to a slump of 100 mm for the sand–clay backfills with fines content ranging from 20% to 89% fines ranged from 1.2% to 2.1%, respectively, whereas those for the sand–bentonite backfills amended with 2%, 3%, 4% or 5% of dry bentonite were 3.6%, 3.8%, 6.0% and 7.2% bentonite, respectively.

Because of the time dependent properties of the SCB backfills (owing to the pozzolanic activity of the cement), the properties of design compositions are measured at different durations, typically at 7, 14, 21 and/or 28 days, after curing of backfill specimens like that previously noted for CB compositions (Fig. 37). Specimens of design compositions are typically prepared and tested as small cylinders (50 mm \times 100 mm) and, because the specimens are tested after different curing durations and because of the inherent variability of SCB backfills, the preparation of numerous cylinders (>20) may be required for each possible design composition [108].

Backfill is placed in the trench after the excavation has been completed by forming a slope of the mixed material that slumps down and displaces the liquid slurry forward. Because the SCB backfill hardens, the SCB backfill slope typically ranges from 3:1 to 6:1 (horizontal:vertical) [108], which is much steeper than SB backfill slopes that usually range from 6:1 to 12:1 [97].

The excavation and the placement of the slurry within the trench generally proceed at the same rate as backfilling. In this way, the procedure is continuous and the distance between the excavator and the backfill placement point remains relatively constant [108, 109]. However, the steeper backfill slopes associated with SCB correlate with a reduced length of an open trench for SCB backfills compared with SB backfills. This will result in greater trench stability [108].

The hardening of the SCB backfill after placement within the trench can result in the existence of joints between successively placed layers. Although the existence of such joints may seem to be detrimental to SCOW performance, Ryan and Day [108] do not recommend taking measures to treat or otherwise eliminate these joints for several reasons: First, all of the placed backfills are of relatively low strength such that the presence of an angled joint with lower strength should have little, if any, impact on the overall strong performance of the SCOW in most situations. Second, measures to destroy the joints by scraping or re-excavating a portion of the set slope can cause more significant problems resulting from trench sidewall disturbance and slowing production, which creates more joints. Finally, treatment of the joints typically requires additional equipment adjacent to the trench, which can be problematic. The mixing of the backfill for both SB and SCB SCOWS is usually performed using mobile equipment (e.g. hydraulic excavators, loaders, bulldozers, skid-mounted mix plants). SCB backfill usually requires additional, more complex equipment than SB backfill [97, 108]. For example, a typical SB wall will have backfill that is mixed on the ground next to the trench alignment and then placed back into the trench, all with the same machine. In contrast, SCB backfill commonly requires two separate machines (usually hydraulic excavators) that perform the proportioning, mixing and placing. Additional efforts such as making use of mixing boxes and/or mixing pits to control the proportions may be necessary [108].

5.2.1.1. Engineering properties of slurry cut-off walls

The primary engineering properties of concern for SCOWs are the strength or compressibility and the hydraulic conductivity. Strength generally correlates indirectly with compressibility (i.e. the higher the strength, the lower the compressibility, and vice versa). In general, the typical relationship among the three types of SCOWs in terms of either strength or hydraulic conductivity is in the order $CB \ge SCB \ge SB$, with the strength of SB backfills being lower than that of many types of natural clays [115]. This relative relationship in strength and hydraulic conductivity results directly from the contrasting effects of the cement and bentonite components of the various SCOWs (i.e. in general, the higher the bentonite content, the lower the strength and hydraulic conductivity). However, depending on the relative compositions of individual types of backfill, these properties can overlap significantly.

Both cement and bentonite are hydrophilic, meaning that both materials have an affinity for water. However, this affinity for water results from different mechanisms. In the case of bentonite, water is absorbed within the mineralogical composition of the bentonite particles largely in the form of hydrated cations undergoing cation exchange, which results in swelling of the bentonite and the formation of a 'tight' porous matrix with small pores and low hydraulic conductivity. This swelling of bentonite and the associated tendency to form a matrix with low permeability, often referred to as 'gelation', is the primary reason that bentonite is used as a constituent in hydraulic containment barriers [116]. However, sodium bentonite is also an inherently soft and weak material that provides relatively little resistance to compression and shear [116, 117].

In the case of cement, water reacts chemically with the cement in a process referred to as hydration, which results in an exothermic release of heat (i.e. referred to as the 'heat of hydration'), which leads to curing or hardening, resulting in an increase in strength. In general, the strength of cement based SCOWs increases with increasing cement content. However, a higher cement content also leads to higher cost and more brittle behaviour, which makes these SCOWs more susceptible to cracking. Also, the relatively high water to cement ratios, which typically range from 3:1 to 11:1 for CB mixtures, result in a relatively porous matrix and high permeability [102]. Finally, a higher cement content also inhibits the ability of the bentonite to swell and form a matrix with low permeability.

Thus, when cement is the dominant constituent, as in the case of CB SCOWs, the strength is high but the hydraulic conductivity is also high, and when bentonite is the dominant constituent, as in the case of SB SCOWs, both the strength and the hydraulic conductivity are low. When neither constituent is dominant, as in the case of SCB SCOWs, both constituents compete more or less equally for the available water. Values for both the strength and the hydraulic conductivity of the resulting SCB SCOW lie between those for CB and SB SCOWs. The SCB SCOWs were first considered as a way to provide a higher strength relative to SB SCOWs and a lower hydraulic conductivity relative to CB SCOWs. Of course, the actual magnitudes of strength and hydraulic conductivity will be a function of factors such as the amount and quality of cement and/or bentonite; the nature of the trench spoils in the case of SB and SCB backfills; the water content; the curing period for CB and SCB SCOWs; the amount, if any, of coarse grained soils or other amendments such as blast furnace slag; and, in the case of SCB backfills, whether the cement is added as dry cement to the backfill before mixing with the slurry or is added in the form of cement grout during sluicing. However, amendments and other additives, such as lignosulfonate thinners used as deflocculating agents [116] to prepare bentonite slurries, may assist in reducing hydraulic conductivity, but may also complicate the construction process and add to the cost. Furthermore, simply increasing the quantity of bentonite in an SCB backfill will not necessarily have the same beneficial effect as it does for the SB backfill, owing to the interference of the cement on the bentonite swelling and the need to increase the amount of water to wet the backfill for placement [108].

As a result of the aforementioned considerations, SB SCOWs are generally preferred to CB SCOWs for containment applications where low permeability is desired. SB SCOWs can achieve hydraulic conductivity values lower than the generally regulated maximum value of 10^{-9} m/s, whereas the lower limit on hydraulic conductivity values for CB SCOWs is generally about 10^{-8} m/s (although hydraulic conductivity values of $\leq 10^{-9}$ m/s have been achieved by supplementing the CB mixture with ground granulated blast furnace slag) [118–121]. For example, the hydraulic conductivity of CB SCOWS with 2.5–7.5% of blast furnace slag additive can be around 10^{-9} m/s after seven days of hardening, whereas the hydraulic conductivity of conventional CB SCOWs

or CB SCOWS with fly ash additive after seven days of hardening is typically $1-5 \times 10^{-8}$ m/s [100].

Finally, the hydraulic conductivity of SCB SCOWs is generally between that of SB and CB SCOWs, with typical minimum hydraulic conductivity values of $1-5 \times 10^{-9}$ m/s being achievable, depending on several factors, including the number of fines inherent in the backfill, the amounts of cement and dry bentonite added to the backfill and whether the cement is added as dry cement before sluicing or as a cement grout during sluicing [108]. For example, Ryan and Day [108] refer to the use of an SCB SCOW to seal the foundation soil supporting a long earthen dike with a specified maximum hydraulic conductivity of 5×10^{-9} m/s. It was found that this low hydraulic conductivity value could only readily be achieved when (i) at least 2% dry bentonite was added to the backfill, (ii) the backfill had a 'high' but unspecified fines content, and (iii) the cement was added to the backfill via sluicing with a grout.

In terms of strength, CB SCOWs generally attain maximum or 28-day strengths of 700–2800 kPa [106], whereas 28-day strengths for SCB SCOWs are generally 105–2100 kPa [108]. The compressive strengths of SB backfills are likely to be at the lower range of values for SCB SCOWs, although evidence suggests that SB backfills will have a similar strength and compressibility to that of SCB backfills when tested under confined conditions (i.e. as opposed to unconfined conditions) [104].

For environmental applications where the SCOW is going to be exposed to waste streams of contaminants, such as leachates laden with chemicals derived from waste disposal practices, the chemical constituents within the waste stream can react unfavourably with the material constituents within the SCOW (principally bentonite and/or cement). This can result in increases in the hydraulic conductivity values of the SCOW and decreases in containment efficiency and performance. For example, Ryan [104] describes a case study where a conventional SB SCOW was being considered to contain hazardous waste leachate that had emanated from a landfill and was polluting the surrounding groundwater. The measured flow rates of the leachate through CSB, commercially available bentonite treated to resist chemical attack, and attapulgite (palygorskite) clay relative to those with tap water are shown as a function of the pore volumes of flow in Fig. 38 (based on data from Ref. [104]). The results indicated that (i) the conventional bentonite was not chemically resistant to the leachate (i.e. the bentonite was incompatible with the leachate); (ii) the treated bentonite performed worse than the conventional bentonite; and (iii) the attapulgite clay was essentially unaffected upon permeation with the leachate. As a result, the SCOW was constructed using attapulgite clay instead of bentonite both for the slurry and for the mixed backfill, but at an added cost owing to the need for a larger amount of more expensive attapulgite clay, and the



FIG. 38. Chemical compatibility testing of three possible backfills for use in a slurry cut-off wall to contain hazardous waste leachate [104].

need for special equipment and procedures. The results also serve to indicate that specially treated or 'chemically resistant' bentonites might not be more effective in terms of chemical resistance, as the treatment process for the bentonite, which is commonly a trade secret, might not be appropriate for the actual chemical solution to be contained. As a result, the chemical resistance or compatibility of the actual materials to be used in the construction of the SCOW with the actual chemical solution or leachate to be contained should be evaluated for each case.

The lack of chemical resistance or incompatibility of the cement in CB walls can reportedly have a more significant effect on the hydraulic conductivity of CB SCOWs (i.e. relative to the bentonite, because of the support function of the cement in the CB wall after hardening of the mixture) [100, 122]. For this reason, CB SCOWs are not normally used where exposure to detrimental chemicals is expected [102]. In this regard, fly ash amendment can typically increase the resistance of the CB SCOW to chemical attack, whereas blast furnace slag cannot, a result that contrasts to the relative effects of fly ash and blast furnace slag amendments on the hydraulic conductivity of CB SCOWs to tap water mentioned previously [100, 102]. However, only very strong acids and high concentrations of sulphites have reportedly had a significant adverse impact on the hydraulic conductivity of CB SCOWs when the groundwater contamination is associated with low pH environments, such as in the disposal of sulphidic tailings.

For SB SCOWs, the soil composition and bentonite content are important factors affecting the resistance of SB backfills to chemical attack [123, 124].

In particular, backfills prepared from base soils graded for use in wells with appreciable native fines require less bentonite and are generally less susceptible to increases in hydraulic conductivity relative to backfills with low native fines content. For example, ratios of hydraulic conductivity based on chemical permeation (K_c) relative to hydraulic conductivity based on water permeation (K_w), or K_c/K_w , ranging from one to five, were reported by D'Appolonia [99] for backfills containing 30–40% native fines and only 1% bentonite for permeation with solutions containing 1000–10 000 mg/L of Ca²⁺ or Mg²⁺. In contrast, a K_c/K_w value of ten was reported by Evans et al. [125] for a sand–bentonite backfill containing 5% native fines and permeated with a 4 mg/L chromium (Cr³⁺) solution, which indicates that backfills most vulnerable to chemical attack are those in which hydraulic conductivity is controlled by the bentonite fraction [123].

However, Malusis and McKeehan [123] also report the results of a study aimed at evaluating the effects on hydraulic conductivity of SB backfills comprised of three different bentonites, with bentonite content ranging from 4.5% to 5.7% (by dry weight), owing to permeation with chemical solutions containing 10–1000 mM CaCl₂ (K_c) relative to hydraulic conductivity based on permeation with tap water (K_w). They reported values of $K_c/K_w \leq 5$ regardless of the CaCl₂ concentration or the type of bentonite. The available data suggest that the hydraulic conductivity of SB backfills, even with a higher bentonite content, can be expected to be up to ten times higher when permeated with chemical solutions that present similar characteristics of waste streams commonly encountered in practice, relative to the hydraulic conductivity that is based on permeation with water. However, prudence dictates that the design value of hydraulic conductivity be based on the permeation of site-specific materials with site-specific liquids in all cases.

In the case where hydraulic conductivity testing of backfill with site-specific liquids indicates a compatibility issue resulting primarily from the bentonite component of the backfill, alternative materials can be considered as substitutes for the bentonite. For example, as previously noted, Ryan [104] substituted non-reactive attapulgite clay for bentonite in the construction of a SCOW because permeation of the bentonite based backfill with the site-specific liquids had indicated a compatibility issue. More recently, the use of chemically modified or polymerized bentonites for improved chemical resistance has been considered [107, 123, 126].

5.2.1.2. Construction quality assurance/control for SCOWs

Quality assurance and control for construction are critical to ensure that the SCOW is constructed following the design specifications. The overall competency with which construction QC and QA are performed is likely to have a major influence on the ability of the SCOW to achieve the required performance criteria, whether these criteria are based on specific regulations or simply on considerations for the protection of human health and the environment.

In construction, the terms QC and QA are often used interchangeably. However, both terms have slightly different meanings [13, 109]. The QC process monitors and controls the quality of a construction project, and refers to the action taken by the contractor regarding material compliance and execution of the plan in accordance with project specifications. In contrast, the QA process assures that the facility is constructed in accordance with the specified design, and includes measures to assess the contractor's compliance with the plans and project specifications. Because the actual inspections (QC) or activities (QA) are typically based on the same criteria, the terms are often combined as QA/QC. Entities representing owners typically perform construction QC, whereas entities representing regulators typically perform construction QA.

The detailed requirements for QA/QC for the construction of SCOWs are beyond the scope of this publication. Examples of typical QA/QC requirements for properties and test methods for bentonite and CB slurries are given in Tables 12 and 13, and a QC programme for SB SCOWs is given in Table 14. Further details regarding QA/QC requirements for construction can be found in Refs [97, 102, 108, 109, 129, 131, 132].

Property	Definition (units)	Test method
Bentonite concentration	Mass of bentonite per volume of slurry (kg/L)	Methylene blue test [127] or electrical conductivity test [128]
Density	Total mass (solids plus water) per volume of slurry (Mg/m ³)	Mud balance
Marsh viscosity	The time required for a specified amount of slurry to drain from a standard funnel cone (s)	Marsh funnel (cone) viscometer
Filtration of fluid loss	The volume of slurry lost in a given time from a fixed volume of slurry when filtered at a standard pressure through a standard filter (mL)	Filter press apparatus (e.g. Fig. 35)

TABLE 12. COMMON SLURRY PROPERTIES AND TESTING METHODS^a

TABLE 12. COMMON SLURRY PROPERTIES AND TESTING METHODS^a (cont.)

Property	Definition (units)	Test method
Filter cake	Thickness (mm)	Estimated from filtrate loss test
	Hydraulic conductivity (m/s)	Determined by permeating water or containment liquid through filter cake infiltrate loss test (e.g. Fig. 35)
Gel strength	Initial (10 s) shear strength after stirring slurry (kPa)	Rotational (e.g. Fann) viscometer
	Final (10 min) shear strength obtained after allowing 10 min to elapse between stirring and measurement (kPa)	Rotational (e.g. Fann) viscometer
рН	$pH = -log \{H+\}, where \{H+\} = hydrogen ion concentration$	pH electrode and meter
Sand content	Solid particles with equivalent diameters between 4.75 mm and 0.075 mm (%)	Sieve analysis

^a Adapted from table 5-3 of Ref. [102]. Refer to API 13B-1 [105].

TABLE 13. SPECIFIED PROPERTIES OF BENTONITE AND CEMENT–BENTONITE SLURRIES^a

Property (units)	Bentonite slurry		Cement-bentonite slurry	
	Fresh hydrated	During excavation	Fresh hydrated	During excavation
Density (Mg/m ³)	1.01-1.04	1.10-1.24	1.03–1.4	n.a. ^b
Apparent Marsh viscosity (s)	38–45	38–68	40-45	38–80
Plastic viscosity (Pa × s)	<20 ^c	n.a.	9	30–50
	Bentonite slurry		Cement-bentonite slurry	
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Property (units)	Fresh hydrated	During excavation	Fresh hydrated	During excavation
Filtrate loss (mL)	<30 Range: 15–30	Range: 15–70 Typical av.: 40–60	100–300	n.a.
pH	7.7–12	10.5–12	12–13	n.a.
Gravimetric water content (% by weight)	~93–97	~78-82	76	55–70
Bentonite content (% by weight)	4–7	~6	6	6
Other ingredients (% by weight)				
Sand	<1 ^c	<5 ^c	n.a.	n.a.
Solids	~2	3–16	15–30	30–45
Cement	n.a.	n.a.	18	n.a.
Gel strength (Pa)				
@ 10 s	d	_	15	10
@ 10 min	7–30	20–40	18	22
Strain @ failure (%)		_	≥15	_

TABLE 13. SPECIFIED PROPERTIES OF BENTONITE AND CEMENT– BENTONITE SLURRIES^a (cont.)

^a Adapted from table 2-2 of Ref. [102, 129].

^b n.a.: not applicable.

^c Specification for construction of tremie concrete diaphragm walls.

^d -: data not available.

TABLE 14. QUALITY CONTROL PROGRAMME FOR SOIL– BENTONITE SLURRY CUT-OFF WALLS^a

Component	Subject	Standard	Type of test: Specified values	Frequency
Materials	Water	b	 pH and total hardness as required to properly hydrate bentonite with the approved additives Determined by slurry viscosity and gel strength tests 	Per water source or as changes occur
	Additives	_	Manufacturer certificate of compliance for stated characteristics as approved by engineer	_
	Bentonite	API 13B-1 [105]	 Manufacturer certificate of compliance Premium grade sodium bentonite 	_
	Backfill soils	_	 Gradation of selected soils obtained from a borrow area approved by the engineer: 65–100% passing 3/8 in sieve (9.5 mm) 35–85% passing No. 20 sieve (0.85 mm) 15–35% passing No. 200 sieve (0.075 mm) 	
Slurry	Freshly prepared	API 13B-1 [105]	 Density: 1.03 Mg/m³ Viscosity: 40 s Marsh @ 20°C (or 15 mPa·s) Filtrate loss: 15–25 mL in 30 min @ 689 kPa 	1 set per shift or per batch (pond)
	In trench	API 13B-1 [105]	pH: 8 Density: 1.03–1.36 Mg/m ³	1 set per shift at point of trenching

Component	Subject	Standard	Type of test: Specified values	Frequency
Backfill	At trench	ASTM C143 (slump cone test) [130]	Slump: 50–150 mm Gradation: — 65–100% passing 3/8 in sieve (9.5 mm) — 35–85% passing No. 20 sieve (0.85 mm) — 15–35% passing No. 200 sieve (0.075 mm)	1 set per 150 m ³

TABLE 14. QUALITY CONTROL PROGRAMME FOR SOIL– BENTONITE SLURRY CUT-OFF WALLS^a (cont.)

^a Adapted from table 5-1 of Ref. [102].

^b —: data not available.

5.2.1.3. Potential failure mechanisms for slurry cut-off walls

Failure mechanisms for SCOWs may be classified into the two categories [95] of (i) construction defects and (ii) post-construction property changes. In the cases of SB and SCB SCOWs that require backfills, construction defects include (a) improperly mixed backfill (e.g. backfill with lumps of unmixed granular soils and/or pockets of slurry); (b) backfill heterogeneity resulting from slurry entrapment during placement (e.g. because of too stiff (low slump) backfill or improper placement of backfill) or from the settling of the coarser fraction of the backfill during periods of inactivity (e.g. overnight, weekends, holidays); and (c) collapse of portions of the trench prior to or during backfill placement, resulting in the possibility of 'windows' of higher hydraulic conductivity within the SCOW [133, 134]. Figure 39 [109] illustrates these issues. Also, improper or lack of keying into an underlying low permeability stratum (e.g. Fig. 34 (a)) can result in excessive under seepage and, ultimately, poor containment performance [133, 134].

In the case of CB SCOWs, construction defects resulting from improperly mixed backfill and backfill heterogeneity are not relevant since this type of SCOW does not have backfill. However, trench collapse during slurry placement is still an issue. Construction defects that are unique to CB SCOWs include inaccurate batching of the slurry components; 'first batch' effects, which refers to the preparation of the first batch of materials in a clean mixer that results in a thicker slurry than subsequent batches prepared in an unclean mixer; filtrate loss (loss of too much slurry to surrounding soil during placement); and 'cold joints',



FIG. 39. Other considerations for slurry cut-off walls: (a) issues related to improper trenching, backfilling and protection; (b) capping SCOWs for protection and improved load-bearing capacity.

which refers to an unplanned joint that results when the CB slurry hardens prior to placement of the next batch of slurry [97]. The potential for cold joints also exists with SCB backfills.

Post-construction property changes increase hydraulic conductivity from external (atmospheric) distress near the ground surface, which can be exacerbated through localized fluctuations in the water table (Fig. 39 (a)). Such distress can result from cycles of wetting and drying, cycles of freezing and thawing, the development of ice lenses within the backfill (SB and SCB SCOWs) and desiccation [135]. These potential issues can be mitigated by placing a cap (cover) at the ground surface for protection (Fig. 39 (b)).

Capping the SCOW also serves other purposes, such as preventing the application of concentrated loads on top of the SCOW, reducing surface infiltration, and controlling runoff and erosion around a wall, especially when the SCOW is located near a steep slope and/or in an area where precipitation is high [102]. Control of erosion can also be facilitated by seeding vegetation at the surface. Other potential failure mechanisms include chemical incompatibility, biotic intrusion, hydraulic fracturing and seismic activity from earthquakes.

Biotic intrusion refers to the potential impact of biota (vegetation) that may penetrate the barrier such that the integrity of the barrier is compromised. In this regard, the stiffer CB and SCB SCOWs provide more resistance to biotic intrusion.

Hydraulic fracturing of the barrier (e.g. because of hydraulic gradients applied across the cut-off wall) can occur when the drawdown of the groundwater level within the containment area is significant (Figs 32 and 33). For example, McCandles and Bodocsi [136] report evidence of hydraulic fracturing in physical model tests of SB SCOWs that they conducted in the laboratory, presumably a result of applying high horizontal hydraulic gradients. However, hydraulic fracturing is also less of an issue with CB SCOWs than with SB SCOWs [97].

Seismic activity from earthquakes is generally not considered to cause problems for slurry wall stability because the compressibility of a slurry wall backfill is generally designed to allow for deformations without cracking [102]. Furthermore, a significant portion of most SCOWs is located beneath the water table such that SCOWs tend to retain sufficient water to remain somewhat plastic. For this reason, stresses and strains resulting from seismic activity are likely to cause the wall material to flow and fill any cracks that may result from such activity. Of course, for the stiffer cement based CB and SCB SCOWs, which are inherently more brittle than SB SCOWs, cracking resulting from seismic activity may be more problematic. However, in such cases, the cracking of the SCOW can be repaired by injecting grout [102].

5.2.1.4. Contaminant transport considerations for slurry cut-off walls

Unlike the more traditional use of SCOWs for purely hydraulic containment, where seepage of uncontaminated groundwater through the SCOW is the primary concern, the use of SCOWs for in situ containment of contaminated groundwater also needs to consider the migration or transport of contaminants through the SCOW. This consideration includes both physical processes and chemical/biological processes [137, 138]. Physical processes result in the advancement of the contaminants through the barrier and include advection, diffusion and mechanical dispersion. Chemical and biological processes affect the concentration of a chemical or otherwise transform the chemical into a different form during migration through the barrier, collectively referred to as 'fate'. A summary of the prevailing chemical and biological processes that affect aqueous miscible contaminant transport through porous media is provided in Table 15 [139]. Physical processes are described briefly below.

TABLE 15. CHEMICAL AND BIOLOGICAL PROCESSES AFFECTING AQUEOUS MISCIBLE CONTAMINANT TRANSPORT^a

Process	Definition	Significance
Sorption ^b	Partitioning of a contaminant between pore water and porous medium	Adsorption reduces the rate of contaminant transport
Radioactive decay	An irreversible decline in the concentration (activity) of a radionuclide	Important attenuation mechanism when the half-life for decay is \leq the residence time in the flow system; results in by-products
Dissolution/ precipitation	Reactions resulting in the release of contaminants in the solid phase (dissolution) or removal of contaminants via transformation to a solid phase (precipitation)	Dissolution results in a source of contamination; precipitation is an important attenuation mechanism, especially in high pH systems (e.g. pH >7)
Acid/base	Reactions involving a transfer of protons (H+)	Important in controlling other reactions (e.g. dissolution/ precipitation)
Complexation	Combinations of anions and cations into a more complex form	Affects chemical speciation that can affect sorption, solubility and other processes
Hydrolysis/ substitution	The reaction of a halogenated organic compound with water (H_2O) or a component ion of water hydrolysis or with another anion (substitution)	Typically transforms an organic compound into a more soluble form or a form more susceptible to biodegradation
Oxidation/ reduction (redox)	Reactions involve a loss (oxidation) or a gain (reduction) of electrons (e–)	Important attenuation mechanism in terms of controlling precipitation of metals
Biodegradation	Reduction in concentration owing to microbial processes	Important attenuation mechanism for organic compounds; may result in undesirable by-products

^a Adapted from table 2.1 of Ref. [139].

^b See Section 6.2.1 for more on sorption and adsorption.

Advection refers to the fundamental process by which dissolved or aqueous miscible chemicals (solutes) are transported along with the flowing fluid or solvent, typically water, by Darcy's law. Thus, advection is a function of the hydraulic conductivity of the barrier and the hydraulic gradient across the barrier.

Diffusion refers to the fundamental process by which solutes migrate within the solvent (water) following Fick's law (i.e. from a location of higher concentration (chemical potential) to a location of lower concentration (chemical potential)). Therefore, diffusion occurs in the absence of a hydraulic gradient.

Mechanical dispersion is the process by which the solute disperses or spreads out during advection as a result of pore-scale variations (e.g. sizes of the pores between adjacent particles and the tortuosity related to the interconnectivity of the pores) and macroscopic variations (macro-scale heterogeneities) in hydraulic conductivity. Mechanical dispersion is affected by the magnitude of the flow rate and the distance of solute migration, such that mechanical dispersion diminishes as the flow rate and the distance of solute migration decrease. Thus, for relatively thin, low permeability barriers, such as SCOWs, mechanical dispersion can generally be neglected without significant loss in accuracy [140].

Concerning advection and diffusion, three cases are relevant, as illustrated in Fig. 40. The case of pure diffusion (Fig. 40 (a)) occurs when the water levels on each side of the barrier are the same, resulting in no hydraulic gradient across the barrier to drive advection. As a result, only diffusion is prevalent, such that the diffusive contaminant mass flux is directed from the containment side of the barrier with the higher contaminant concentration to the outside of the barrier with the lower contaminant concentration. The case of diffusion plus advection (Fig. 40 (b)) occurs when placement of the low permeability barrier results in a rise in the groundwater level on the upgradient or contaminant side of the barrier, such that both the diffusive and advective contaminant mass fluxes are directed outwards (i.e. positive *x*-direction). Finally, the case of diffusion minus advection (Fig. 40 (c)) occurs when the groundwater level on the contaminant side of the barrier is drawn down with wells and/or drains such that advection tends to counteract and, therefore, diminish or prevent outward diffusion.

In general, diffusion becomes more significant as advection decreases, such that the pure diffusion scenario represents a limiting case. Although advection will decrease with a decrease in the hydraulic conductivity of the barrier and/or a decrease in the hydraulic gradient across the barrier (i.e. decrease in the difference in the water levels on either side of the barrier), the typical inability to achieve hydraulic conductivity values for SCOWs lower than about 10^{-10} m/s suggests that the significance of diffusion will be governed more by the magnitude of a hydraulic gradient than by the magnitude of hydraulic conductivity [1]. Further details pertaining to the evaluation of advection and diffusion through vertical SCOWs are provided by Refs [111, 121, 122, 140–144].



FIG. 40. Limiting contaminant transport scenarios for vertical barriers: (a) pure diffusion; (b) diffusion plus advection; (c) diffusion minus advection. C — concentration.

The concept of enhancing the containment function of SCOWs by adding constituent materials within the backfill to increase the attenuation capacity of the backfill for specific contaminants of concern has also been proposed [125]. For example, Malusis et al. [111] showed that sand–bentonite backfills amended with 2–10% granular or powdered activated carbon could be effective in increasing the containment duration of a 1 m thick SCOW by as much as several orders of magnitude. Also, Hong et al. [112] considered the same sand–bentonite backfills amended with 2–10% high cation exchange capacity zeolites (182–259 cmol_c/kg) to increase the sorption capacity of the backfills concerning heavy metals. (See Section 6.2.1.5 for more on sorption.) Although the concept of SCOWs with enhanced attenuation capacities shows promise, its practical implementation appears to be lacking.

5.2.1.5. Composite and combination slurry cut-off walls

Composite vertical cut-off walls typically refer to the placement of a geomembrane vertically along the centreline of SB or CB SCOWs, although other materials such as steel sheeting can be inserted within the SCOW. This type of wall originated in Europe where CB walls have traditionally been more commonly installed than SB walls, primarily because of the aforementioned potential for incompatibility of CB SCOWs when exposed to groundwater contaminated with aggressive chemicals (e.g. groundwater contaminated with mixtures of high concentrations of multiple contaminants and/or having very low pH). The geomembrane may be installed either in a slurry filled trench or directly into the ground using a special insertion plate. These composite SCOWs will generally provide significant improvement in performance relative to the alternative single SCOWs, but are more expensive to install. Further design

considerations for composite walls, including an assessment of the groundwater flow through the joints connecting adjacent geomembrane panels, are provided in Refs [141, 144].

Combination SCOWs refer simply to the use of more than one SCOW, such as incorporating CB or SCB panels in an SB SCOW to traverse a steep slope [145]. However, the terminology has also been used to describe the use of drains in conjunction with a SCOW to control the hydraulic conditions at the site.

5.2.2. Non-slurry cut-off walls

Although the use of SCOWs as vertical containment barriers has been the focus of discussion above, a wide variety of other materials and/or technologies can be used for vertical containment. These materials include sheet-pile walls, geomembranes, grout curtains, deep soil mixing, vertical drains and frozen ground. Brief descriptions of each of these materials and technologies follow; further details can be found in Refs [97, 121, 132, 146]. Other methods, such as the diaphragm and vibrating bean cut-off walls, are not discussed, as these types of walls are generally too expensive for most chemical containment applications [97].

Sheet-pile walls typically refer to interlocking sections of thin, 5–25 mm, steel sheeting panels, 0.4–1.0 m long. The walls are installed by driving or vibrating interlocking steel sheet piles into the ground (hence sheet-pile walls are often referred to as sheet-piling). The interlocks or joints between the panels represent a primary pathway for groundwater flow through the walls, but a cord of material that expands when hydrated and attains a very low permeability may be inserted in the interlock to improve performance [97, 121, 146]. Although sheet-pile walls have been used extensively for hydraulic containment applications, concerns for leakage through the interlocks or joints of sheet-pile walls have limited the use of sheet-pile walls as barriers for chemical containment applications [121].

Geomembranes have also been used individually as vertical cut-off walls, primarily for hydraulic containment applications [121, 146]. However, similar to sheet-piling, concern associated with the ability to effectively seal the geomembrane panels has limited the sole use of individual geomembranes as vertical barriers for chemical containment applications.

Grout curtains represent barriers that are formed when a grout, typically a mixture of cement and water, is introduced into the native soil such that, upon hardening, the grout solidifies to form a barrier to liquid migration. The grout typically is introduced by injection under high pressure (~34.5–41.5 MPa) through one or more holes in a narrow rod that is inserted into the ground — a process known as jet grouting [132]. The cement grout mixes with the soil to form a

cement mixture column, known as soil-crete, in the ground. The grout is injected at locations such that the soil-crete columns overlap, thereby forming a barrier. Although the technology has been used extensively for hydraulic containment applications, such as seepage cut-offs beneath dams, the use of the technology for chemical containment applications has been limited because of concerns with the continuity or integrity of the barrier, and the lack of a reliable method for evaluating the continuity of the barrier [121]. There also is a potential for chemical incompatibility between the contaminated groundwater and the grout.

Deep soil mixing is similar to grouting, in that a chemical grout (cement plus water), or bentonite or cement-bentonite slurry is introduced into the subsurface in a pattern, such that the end result is a series of overlapping mixed zones that form a continuous, low permeability barrier [93]. However, unlike jet grouting, deep soil mixing is facilitated by using a hollow stem auger that contains injection ports in the auger blades, such that the grout or slurry is injected and simultaneously mixed with the soil. The issues with using deep soil mixing for chemical containment are the same as those for jet grouting (i.e. the inability to control or verify the continuity and integrity of the resulting wall).

Vertical drains or vertical trenches filled with a coarse-grained porous material, such as gravel, can be used to control groundwater flow and can therefore act as a containment barrier [97, 121]. Vertical wells can also be used for containment via hydraulic gradient control (Fig. 33). In the case of gravel-filled vertical trenches, a liquid collection and removal system at the bottom of the trench is required. The primary performance concerns with gravel-filled trenches are clogging of the collection system and mechanical breakdown of the extraction system. Clogging can occur because of the accumulation of sediment, biological growth and/or mineral precipitation [121]. Also, vertical trenches and vertical wells require continuous maintenance and monitoring, which increases costs but facilitates the identification and remediation of clogging and mechanical breakdown.

Finally, ground freezing is a technique by which coolants (refrigerants) are pumped through conduits (e.g. pipes, shafts) located within the ground to freeze the groundwater adjacent to the conduits, thereby forming a barrier to liquid flow. This method, although extensively used for construction purposes, can be prohibitively expensive for use as a chemical barrier, as the temperature of the groundwater needs to be maintained below freezing throughout the active period of the barrier, which can require extensive energy and material inputs into the system.

6. PERMEABLE REACTIVE BARRIERS

6.1. INTRODUCTION

The concept of an in situ permeable reactive barrier, referred to in this publication as PRB, was first proposed by McMurtry and Elton [147]. This concept, as illustrated in Fig. 41, involves constructing a zone (e.g. trench or excavation) filled with chemical and/or biological reagents or catalysts (e.g. biostimulants) in advance of contaminated groundwater (a plume). The contaminants will degrade or otherwise attenuate (e.g. sorb) during passage through the reactive or treatment zone, resulting in treated or 'cleansed' groundwater emanating from the downgradient side of the zone. Unlike low permeability vertical barriers (e.g. SCOWs), the hydraulic conductivity of PRBs needs to be sufficiently high to allow efficient treatment of the contaminated groundwater, without any increase in the hydraulic head (i.e. resistance to flow through the reactive media). Thus, in reality, the two terms 'permeable' and 'barrier' are incongruent in that 'permeable' implies low resistance to flow, whereas 'barrier' implies high resistance to flow. Furthermore, the attenuation or treatment of the contaminants during interaction with reactive media should occur passively (i.e. without any energy input into the system). For these reasons, PRBs have also been referred to by numerous other terms such as vertical treatment zones, permeable reactive walls or zones, passive treatment systems and in situ treatment curtains [148].



FIG. 41. Schematic of the in situ permeable reactive barrier concept for the treatment of contaminated groundwater.

6.2. ATTENUATION MECHANISMS AND REACTIVE MEDIA

The design and operational management of PRBs requires knowledge of not only the physical properties (e.g. hydraulic conductivity) but also the chemical and/or biological properties of the reactive barrier materials affecting the migration of the contaminants in the groundwater. Thus, an understanding of the potential attenuation mechanisms for the principal chemical species of interest or contaminants of concern is required.

6.2.1. Attenuation mechanisms

A summary of the attenuation mechanisms for several contaminants of concern and the associated reactive media that can be used to facilitate the appropriate reactions for attenuation is provided in Table 16 [149]. Several attenuation mechanisms are noted, including cation exchange, precipitation and co-precipitation, reduction, sorption, and surface complexation. The mechanism of cation exchange is a subset of ion exchange, which also includes anion exchange. Cation exchange is emphasized in Table 16 because heavy metal chemical species, including radionuclides, commonly exist in a cationic form (i.e. as positively charged ions).

6.2.1.1. Cation exchange

The process of cation exchange involves the exchange of one or more cationic species initially in the pore water of a porous media, with one or more cationic species initially held electrostatically to the surfaces of the solid particles comprising the porous media. This exchange occurs in accordance with the requirement for electroneutrality or charge balance, such that the same number of equivalents of ionic charge has to be exchanged. For example, one mole of lead cations (Pb²⁺) initially in the pore water would need to exchange with two moles of sodium cations (Na⁺) initially on the solid phase (i.e. particles) to maintain electroneutrality both in the liquid phase (pore water) and on the particles. Thus, attenuation is achieved by the removal of cations from the liquid phase.

However, a potentially negative aspect of cation exchange as an attenuation mechanism is that the process is reversible; the cations that are initially removed from a solution by cation exchange may be reintroduced back into the solution by subsequent exchange with other cations in the solution. Also, the capacity of a material to hold exchangeable cations (its cation exchange capacity) is finite: once the material is filled with exchangeable contaminants of concern, the material has to be replaced or otherwise replenished.

Reactive medium	Contaminant	Mechanism	
Zero valent iron (ZVI or Fe ⁰) — granular to nanoscale in particle size	Chlorinated solvents	Abiotic reductive dechlorination	
	Energetics (TNT, RDX)	Reductive degradation	
	Redox sensitive metals (Cr ⁶⁺)	Reductive precipitation	
	Redox sensitive oxyanions (U, Se)	Reduction, sorption, precipitation	
	Arsenic	Sorption and co-precipitation	
	Divalent metals (Cu, Zn)	Reduction, sorption, precipitation	
Apatites (phosphates)	Radionuclides (U, Sr)	Precipitation, surface complexation	
	Lead (Pb)	Precipitation	
Zeolites	Radionuclides (Sr)	Cation exchange	
	Ammonium and perchlorate	Cation exchange	

TABLE 16. REACTIVE MEDIA, TARGET CONTAMINANTS AND REACTION MECHANISMS^a

^a From Ref. [149].

6.2.1.2. Precipitation and co-precipitation

Precipitation is a chemical process in which a chemical reaction results in the conversion of a soluble chemical species or compound, which was initially dissolved in aqueous solution, into a solid phase substance (i.e. precipitate). The substance then generally settles out of the solution because it has a higher density than water, thereby becoming immobile. Thus, precipitation immobilizes contaminants of concern by converting them into precipitates, although some precipitates may be suspended as colloids in the liquid phase and be carried along with the flowing liquid. Precipitation is a function of the solubility of the compound, pH, temperature and the rate at which the reactants mix to form the precipitate.

Co-precipitation is a process in which a typically soluble compound is removed from solution by attachment to or association with a precipitate. Co-precipitation may also be defined as the simultaneous precipitation of more than one compound from solution. Co-precipitation is distinguished from precipitation in that co-precipitation involves compounds that tend to be soluble, whereas precipitation involves compounds that tend to be insoluble. Thus, soluble contaminants of concern may be attenuated via co-precipitation.

Like cation exchange, precipitation and co-precipitation are reversible processes. A contaminant of concern that is immobilized via precipitation or co-precipitation can be remobilized via dissolution of the precipitate, for example, by subsequent exposure to a low pH solution. Also, in terms of PRBs, the precipitated contaminant of concern may clog the porous matrix formed by the reactive media within the PRB, requiring removal and replenishment of the reactive media.

6.2.1.3. Reduction

Reduction is a chemical process by which the charge of a chemical species or compound (e.g. metal) is reduced as the result of a gain of electrons (e.g. $Pb^{2+} + 2^{e-} \rightarrow Pb$). The chemical process of reduction generally occurs simultaneously with the chemical process of oxidation pertaining to a loss of electrons (e.g. Mg – $2^{e-} \rightarrow Mg^{2+}$), such that the combined processes are known as oxidation-reduction, or redox, reactions (e.g. $Pb^{2+} + Mg \rightarrow Pb + Mg^{2+}$). The reduction may be used as an attenuation mechanism by converting a contaminant of concern into a less toxic or less mobile form. For example, U^{6+} or U(VI) in the form of the soluble uranyl cation, UO_2^{2+} , can be immobilized by the reduction in the presence of elemental iron (Fe⁰) to the highly insoluble U^{4+} or U(IV) in the form of the amorphous or crystalline uranium oxide precipitate, $UO_2(s)$ [150]. However, strongly reducing conditions have to be maintained, as the reduced U(IV) may become re-oxidized if oxidizing conditions re-occur, resulting in the mobilization of U(VI). Another example where reduction results in the transformation of a more toxic species into a less toxic species is the reduction of hexavalent chromium to trivalent chromium ($Cr^{6+} \rightarrow Cr^{3+}$). Here, Cr^{3+} may be precipitated out of a solution as a metal (oxy)hydroxide [151].

6.2.1.4. Reductive degradation

Reductive degradation generally refers to chemical processes that transform a contaminant of concern into a less toxic and/or a less mobile form. For example,

the removal of chlorine from the industrial solvents such as perchloroethylene (tetrachloroethylene) (PCE, C_2Cl_4) and trichloroethylene (TCE, C_2HCl_4) to form the less toxic ethylene (ET, C_2H_4) via reaction with zero valent iron (ZVI or Fe⁰) is a well recognized attenuation mechanism known as reductive dechlorination. However, the reductive dechlorination of perchloroethylene (tetrachloroethylene) and trichloroethylene proceeds via the formation of the intermediate compounds dichloroethylene (DCE, $C_2H_2Cl_2$) and vinyl chloride (VC, C_2H_3Cl), which can be even more detrimental to the environment (i.e. PCE \rightarrow TCE \rightarrow DCE \rightarrow VC \rightarrow ET).

6.2.1.5. Sorption

Sorption is a general term that refers to a chemical and/or physical process by which one substance (e.g. chemical) is attracted to or within another substance (e.g. soil particles). Sorption includes the processes of adsorption, desorption and absorption:

- Adsorption refers to the attachment of one substance to the surface of another substance.
- Desorption refers to the detachment of a substance from the surface of another substance.
- Absorption refers to the movement of a substance to a location within the domain of another substance (e.g. a sponge absorbs water).

In these examples, the substance being sorbed (e.g. chemical) is referred to as the sorbate and the solid substance that serves to attract the sorbed substance is referred to as the sorbent (e.g. reactive media). In terms of an attenuation process, adsorption is the primary process of interest, although absorption may also be relevant.

Several mechanisms can be associated with adsorption. For example, cation exchange is a sorption mechanism in that the cations that are initially in solution may be attracted electrostatically to the negatively charged surfaces of a solid substance, thereby displacing other cations that were initially held electrostatically to the same surface. Hydrophobic partitioning is another sorption mechanism, whereby hydrophobic organic compounds with little to no water solubility are known to partition from the liquid phase into the organic carbon content of a solid substance (sorbent). Therefore, hydrophobic partitioning can be enhanced as an attenuation mechanism by increasing the organic carbon content of the sorbent.

6.2.1.6. Surface complexation

Surface complexation is another sorption process that is similar to ion exchange in that the sorbate is attracted electrostatically to the surface of a sorbent. However, unlike cation exchange, which involves a charge-neutral reaction between sorbate and sorbent, surface complexation reactions include charges in the reactions. Also, surface complexation reactions are generally affiliated with sorbents with pH-dependent surface charges, such that the magnitude and type of charge (i.e. negative or positive) of the solid phase sorbent is a function of the pH of the surrounding solution. Thus, the extent and magnitude of surface complexation as an attenuation mechanism is a function of the pH and composition of the surrounding solution.

Materials that participate in surface complexation reactions include metal oxides and oxyhydroxides, such as a ferrous oxide (FeO(s)), ferric oxide (Fe $_2O_3(s)$) and hydrous ferric oxide (FeOOH(s)).

6.2.1.7. Biological activity

Aside from the aforementioned attenuation mechanisms, which are largely abiotic chemical reactions, biological activity can also play an important role as an attenuation mechanism. For example, the reduction of inorganic cations can be promoted in the presence of biological activity. In this regard, both direct and indirect reductions are possible. Such reactions include direct reduction of the cation and precipitation, and indirect precipitation resulting from oxidation or reduction of an inorganic anion [150]. An example of direct reduction is the reduction of U(VI) to U(IV) in the presence of acetate (CH₃COO⁻) as a catalyst using the microbe *G. metallireducens* [150]. An example of indirect reduction as an attenuation mechanism is the reduction using sulphate reducing bacteria of the sulphur in sulphate (SO₄²⁻) to sulphide (S²⁻), which then combines with a divalent metal ion (Me²⁺) to form a metal sulphide precipitate: Me²⁺ + S²⁻ \rightarrow MeS(s) [152, 153].

Another example of where biological activity plays a role in attenuation is the use of denitrifying bacteria such as *Paracoccus denitrificans* in the presence of organic substrates to reduce toxic nitrate (NO_3^-) to elemental nitrogen [154]. However, the production of intermediate products, such as ammonium (NH_4^+) and nitrite (NO_2^-), may be problematic, and the production of gases such as a nitrous oxide (N_2O), methane (CH₄) and carbon dioxide (CO₂) may adversely affect the hydraulic performance of PRBs by reducing porosity [154].

6.2.2. Reactive media

The choice of the reactive media to be placed within the PRB is dictated largely by the behaviour of the contaminant of concern and the desired attenuation mechanisms. A summary of a wide range of reactive media used to treat a variety of contaminants of concern can be found in Ref. [154]. Three of the more common reactive media used to attenuate radionuclides via PRBs include ZVI (Fe⁰), apatites and zeolites (Table 16).

Zero valent iron has been used extensively in PRBs for the reduction of contaminants of concern. The effectiveness of ZVI as a reactive media depends on the grain size and specific surface area of the ZVI and the prevailing geochemistry of the pore water. Typical grain sizes of ZVI particles used in PRBs are 0.25–2 mm, with surface areas of 0.5–1.5 m²/g [154]. Nanoscale ZVI particles (known as nZVI) have also been considered for use in PRBs because of the significantly larger surface areas (~10–90 m²/g) and associated greater reactivity [155]. However, a potential issue with the use of nanoscale ZVI particles is the ability to sustain reactivity for a long time.

Apatites are minerals that contain phosphate (PO_4^{3-}), calcium (Ca^{2+}) and one of hydroxide (OH⁻), chloride (Cl⁻) or fluoride (F⁻), with hydroxyapatite ($Ca_5(PO_4)_3OH$) being commonly considered for use as a reactive media. Apatites have very low solubility in water and are characterized by a net negative charge and neutral pH.

Apatites remove contaminants of concern via several mechanisms, including direct sorption, cation exchange, surface complexation and precipitation as phosphates, carbonates, oxides and hydroxides [154]. For example, hydroxyapatite has been shown to immobilize U(VI) via the formation of autunite $(Ca(UO_2)_2(PO_4)_2 \cdot 10H_2O)$, which has a low water solubility [154].

Zeolites are hydrated aluminosilicates with a cage-like structure and a high cation exchange capacity in the range of 200–400 cmol_c/kg (meq/100 g) [154]. Zeolites are commonly used to purify water contaminated with radioactive ions (e.g. caesium and 90 Sr), purify water in fish tanks, remove ammonium from wastewater at treatment plants, adsorb metal ions from industrial wastes and control odour from cat litter [156–158]. As a result of the cage-like structure, zeolites act as sieves in the sense that metal ions contained in solutions that pass through the structure are trapped by ion exchange reactions. For example, zeolites have been used relatively recently as reactive media in two PRBs to attenuate radionuclides (specifically 90 Sr) from contaminated groundwater at the sites of two nuclear facilities in North America [158].

Limestone and other lime related compounds, such as calcium oxide (CaO) and calcium hydroxide (Ca(OH)₂), as well as iron related compounds, have also been considered for use as reactive media in PRBs. For example, the

treatment of groundwater contaminated with uranium and molybdenum has been evaluated using hydrated lime (Ca(OH)₂(s)) as a precipitating agent and ferrous sulphate (FeSO₄) and ferric oxyhydroxide (Fe₂O₃ × nH₂O) as agents of redox, complexation and precipitation [159, 160], and for control of acid (low pH) drainage using anoxic limestone drains to increase the alkalinity of acid water, thereby facilitating precipitation of heavy metals [161–163].

Permeable reactive barriers may also be filled with materials that stimulate or enhance bioactivity (e.g. catalysts or organic substrates) to attenuate contaminants of concern aerobically or anaerobically. Such PRBs are often referred to as biobarriers or in situ bioreactors. A common application involves the use of low cost organic materials, including alfalfa, compost, leaves, manure, peat, sawdust, sewage sludge and wood waste to anaerobically facilitate this reduction of NO₃ to elemental nitrogen via denitrifying bacteria [154].

For example, Blowes et al. [148] describe the use of in-line bioreactors for facilitating anaerobic denitrification of NO_3^- from agriculture. Oxygen reduction and denitrification were enhanced by providing a source of organic carbon (e.g. tree bark, wood chips and leaf compost) for the facultative heterotrophic bacteria in the bioreactors. The bioreactors were successful in reducing nitrogen derived from nitrate (NO_3^-N) concentrations of 3–6 mg/L to less than 0.02 mg/L at rates of 10–60 L/day over a one year period. Blowes et al. [164] concluded that similar bioreactors amended with organic carbon should be effective in treating other redox sensitive contaminants such as arsenic, selenium and chromium.

However, in the case of biobarriers, precautions have to be taken to ensure an adequate supply of organic substrates, which degrade, and to control the environmental conditions (e.g. pH) required to ensure that the microorganisms can survive during the operational period of the PRB. Furthermore, the use of these organic substrates can clog the biobarrier (e.g. either physically or through the generation of gases (CH₄)) such that long term maintenance of biobarriers may be required.

6.3. HYDRAULIC AND CONTAMINANT TRANSPORT CONSIDERATIONS

The major difference between more traditional SCOW containment barriers and PRBs is that the hydraulic conductivity of PRBs needs to be sufficiently high to allow the timely and efficient processing of the contaminated groundwater and to minimize the inducement of additional hydraulic resistance across the wall. Under ideal conditions, the hydraulic conductivity of the PRB will be equal to the hydraulic conductivity of the surrounding aquifer, such that the contaminants will migrate through the reactive zone in response to the aquifer's hydraulic gradient, as illustrated in Fig. 42. Thus, several important considerations for hydraulics have to be considered in the design of PRBs. Of these considerations, the most important is to provide for an adequate residence or retention time.

The retention time refers to the duration of the contaminant of concern within the reactive PRB, which needs to be sufficient to allow the reactive media to attenuate the contaminant to an acceptable level, such as the maximum contaminant level or drinking water standard, prior to exiting the PRB. The magnitude of retention time is a function of the hydraulic and transport properties of the reactive media comprising the PRB and the time required to attenuate the contaminant of concern via chemical and/or biological reactions. For PRBs with high permeability, the physical processes governing chemical transport include purely hydraulic transport or advection and mechanical or hydraulic dispersion, which is the spreading of the solute front owing to variations in the average linear or seepage velocity occurring during solute transport in and through porous media [137]. The seepage velocity is a function of Darcy's law for liquid flow through porous media as shown in Eq. (5):

$$v = \frac{q}{n} = \frac{K \times i}{n} \tag{5}$$

where

- *v* is the seepage velocity;
- q is the liquid flux, also known as the specific discharge or Darcy velocity;
- *K* is the hydraulic conductivity;
- *i* is the hydraulic gradient;

and *n* is the porosity of the porous media (0 < n < 1).

Note that because porosity is less than one (n < 1), seepage velocity is greater than the liquid flux (v > q). The magnitude of seepage velocity represents a measure of the actual rate of migration of the liquid through only the pores of a porous medium, whereas the magnitude of liquid flux represents the rate of liquid flowing through a given cross-section of the porous medium, including the area attributed to the solid particles. Thus, seepage velocity represents a true measure of velocity in the sense that seepage velocity represents the distance of migration over a specified time.

Mechanical dispersion is governed by a mechanical dispersion coefficient (D) which is a product of the seepage velocity and a parameter known as the longitudinal dispersivity (α) (i.e. $D = \alpha v$), which is a property of the porous media with units of length. In general, the greater the magnitude of mechanical



FIG. 42. Schematic cross-section through a permeable reactive barrier. K_{PRB} — hydraulic conductivity of the PRB; $K_{Annifer}$ — hydraulic conductivity of the surrounding aquifer.

dispersion coefficient, the greater the extent of chemical spreading (dispersion) during transport through a porous medium will be. Mechanical dispersion is also known to be scale-dependent, with greater values of dispersivity and, therefore, the mechanical dispersion coefficient being associated with greater scales of measurement. For example, values of longitudinal dispersivity measured on a laboratory column scale are typically on the order of 10^{-4} m to 10^{-2} m, whereas field measured values of longitudinal dispersivity can be 100 m or more. A commonly accepted method is to assume that longitudinal dispersivity is 10% of the distance of transport. For example, for transport through a PRB with a width of 1 m, the magnitude of longitudinal dispersivity would be only 0.1 m. Thus, for transport through short distances, the effect of mechanical dispersion is often ignored without serious consequences.

Two of the more common attenuation mechanisms relied upon for the treatment of contaminants of concern via PRBs are sorption and degradation by first order linear decay. Sorption can be considered by the inclusion of a retardation factor (R_d) representing the ratio of the total mass of a chemical species in both the liquid and solid (sorbed) phases relative to the mass of the same chemical only in the liquid phase. Thus, sorption of a chemical species to the solid phase of the reactive media results in a retardation factor greater than unity ($R_d > 1$), whereas the retardation factor equals unity ($R_d = 1$) for a non-sorbing (non-attenuating) chemical species is assumed to be proportional to the negative product of decay constant (λ) and the concentration of the chemical species, C (i.e. $dC/dt = -\lambda C$). In this case, the decay constant is inversely proportional to the time required to

decrease the initial concentration of the chemical species by half, which is known as the half-life, $t_{0.5}$ (i.e. $\lambda = 0.693/t_{0.5}$).

First order decay is generally relevant when the contaminant of concern is radioactive; an organic compound that is subject to degradation by hydrolysis reactions; biodegradable; or subject to reductive dechlorination [165]. However, the use of first order decay for biodegradation represents an approximation at best, and may over-predict biodegradation because of the lack of a proper electron acceptor restriction [165].

In terms of sorption, based on the assumption of steady-state, purely advective transport without mechanical dispersion (i.e. $\alpha = 0$) but with linear and reversible equilibrium sorption, retention time is related to seepage velocity, the width (thickness) of the PRB and the sorption properties of the reactive barrier materials (i.e. via the retardation factor) for a contaminant of concern as in Eq. (6):

$$t_{\rm r} = \frac{W}{v_{\rm r}} = \frac{R_{\rm d}W}{v} = \frac{R_{\rm d}Wn_{\rm PRB}}{K_{\rm PRB}i_{\rm PRB}} = \frac{R_{\rm d}W^2n_{\rm PRB}}{K_{\rm PRB}\Delta h_{\rm w}}$$
(6)

where

 $t_{\rm r}$ is the retention time;

W is the width (thickness);

 $R_{\rm d}$ is the retardation factor;

v is the seepage velocity;

 $v_{\rm r}$ is the velocity of the reactive (sorbing) contaminant of concern — $v/R_{\rm d}$;

 i_{PRB} is the hydraulic gradient across the PRB;

 n_{PRB} is the porosity (effective) of the PRB;

 K_{PRB} is the hydraulic conductivity of the PRB;

and $\Delta h_{\rm W}$ (<0) is the head loss over the width W.

Thus, in Eq. (6), retention time can be increased by increasing the sorption properties of the wall materials (i.e. increasing the retardation factor) and/or increasing the width of the PRB. Conversely, Eq. (6) may be rearranged to determine the required width of the PRB. If the PRB has the same hydraulic gradient and hydraulic conductivity as the aquifer, then presumably the porosity of the reactive media will also be the same as, or similar to, that of the aquifer material. In this case, the width will be a function primarily of the sorption capacity of the reactive media via its retardation factor and the design life of the PRB (i.e. its retention time), with decreasing widths for decreasing retention times and/or increasing retardation factors.

For example, if the seepage velocity for an aquifer is 100 m/a, and the PRB is assumed to have the same hydraulic properties as the aquifer, then the width of a PRB constructed using reactive media with a sorption capacity (retardation factor) of 100 and an expected operational period of one year would be 1 m. However, if the sorption capacity of the reactive media was only 10, then a 10 m wide PRB would be required, which is excessive. In this case, a 1 m wide PRB could be used, but the reactive media would have to be replaced with fresh reactive media a total of ten times over the one year period. By increasing the width to 2 m, the reactive media would only need to be replaced five times over the same period. As shown in this example, reliance on sorption as the attenuation mechanism may impact the width of the PRB. To prevent excessively wide PRBs, either reactive media with high sorption capacities are required or the reactive media will need to be replaced periodically to replenish the sorption capacity.

Another important assumption implicit in Eq. (6) is that all of the contaminants are assumed to be adsorbed to the reactive media, such that the width of the contaminated area is zero (C(W) in Fig. 42). This assumption is reasonable provided that the sorption capacity of the reactive media is sufficiently greater than the maximum concentration of the contaminant in the pore water of the reactive media. However, as the concentration of the contaminant in the pore water approaches the sorption capacity of the reactive media, the sorption process becomes non-linear, such that use of Eq. (6) may overestimate the amount of contaminant sorbed, resulting in a non-zero contaminant concentration at the downgradient side of the PRB (i.e. C(W) > 0).

In terms of first order decay, the retention time (t_r) is related to the extent of the decrease in concentration, which may be posed in terms of a reduction factor (RF). In terms of the PRB scenario shown in Fig. 42, the reduction factor is the ratio of the source concentration, $C_o (= C(0))$, relative to the desired concentration at the distance, C(W) (i.e. $RF = C_o/C(W)$). In this case, $t_r = \ln(RF)/\lambda$. The resulting retention times for reduction factors of 2, 10, 100 and 1000 are shown as a function of half-life in Fig. 43.

Based on the assumption of purely advective, steady-state transport without mechanical dispersion (i.e. $\alpha = 0$), the PRB width can be estimated based on Eq. (7) [165]:

$$W = 1.44vt_{0.5} \ln(RF)$$
 (7)

where

W is the width (thickness);

v is the seepage velocity;

 $t_{0.5}$ is the half-life;

and RF is the reduction factor.

The relationship given by Eq. (7) is plotted in Fig. 44 for a seepage velocity of 0.001 m/h. As shown in Fig. 44, the width increases with an increase in treatment level (reduction factor) for a given half-life, or with an increased half-life for a given reduction factor. For the case where mechanical dispersion is included in the analysis (i.e. $\alpha = 0.1 W$), and assuming a constant source concentration (i.e. $C(0) = C_o$) and a semi-infinite domain, the relationship between width, seepage velocity, half-life and reduction factor is given in Eq. (8) [165]:

$$W = 3.61vt_{0.5}\{[1+0.2 \ln(RF)]^2 - 1\}$$
(8)

where

W is the width (thickness);

v is the seepage velocity;

 $t_{0.5}$ is the half-life;

and RF is the reduction factor.



FIG. 43. Retention time as a function of contaminant half-life and the reduction factor based on first order decay.



FIG. 44. Permeable reactive barrier width as a function of contaminant half-life and the reduction factor both with and without mechanical dispersion. α — longitudinal dispersivity.

The relationship given by Eq. (8) is also shown in Fig. 44. All other factors being equal, the effect of mechanical dispersion is to increase the wall thickness, albeit slightly. Thus, as previously noted, the overall effect of mechanical dispersion is relatively minor, because the width of transport is relatively short.

The cross-hatched area in Fig. 44 corresponds to PRB widths of 1–3 m, which should be typical for most applications. For situations where greater widths are required, multiple PRBs in sequence and/or alternative treatment options may have to be considered because of excessive costs.

The analyses mentioned so far illustrate some of the hydraulic and contaminant transport considerations affecting the design of PRBs. However, the analyses are simplistic to the extent that caution should be exercised in adopting the analyses directly without consideration for additional factors that can affect the performance of a PRB. For example, as previously noted, the attenuation of contaminants of concern based on degradation may result in the formation of intermediate compounds that can be even more problematic in terms of environmental impact. In this case, more sophisticated, multispecies chemical analyses may be required [166–168].

6.4. TYPICAL CONFIGURATIONS

The number of possible configurations for PRBs is virtually unlimited. However, the two most common configurations are continuous PRBs and funnel and gate PRBs, with the possibility of multiple reactive zones.

6.4.1. Continuous permeable reactive barriers

Continuous PRBs are reactive zones or trenches that are placed downgradient from a migrating contaminant plume to intercept the entirety of the plume. A linear PRB is the simplest case and is illustrated in Fig. 45. However, the term 'continuous' refers to the continuity of the reactive materials, such that continuous PRBs do not need to be linear.

Concerning the scenario depicted in Fig. 45, under ideal conditions where the magnitude of the hydraulic conductivity of the aquifer ($K_{Aquifer}$) is homogeneous (i.e. independent of the spatial location within the flow domain) and there is no dispersion of solute during migration, the length of the PRB (L) needs to be only as long as the width of the migrating plume (B) (i.e. L = B). However, in reality, lateral solute dispersion and/or heterogeneity of the hydraulic conductivity of the aquifer will likely require a length greater than that of the migrating plume (L > B) to prevent the plume from circumventing the PRB (e.g. Ref. [169]). Furthermore, the PRB needs to be sufficiently deep to prevent the contaminant plume from migrating beneath it. When possible, the preference is to key the PRB into an underlying, low permeability aquitard or aquiclude (Fig. 42).

The standard approach of filling the reactive zone or trench in a continuous PRB with reactive media can be modified to allow for contaminant treatment by other approaches. For example, Pankow et al. [170] proposed using remediation technology known as air sparging via the concept of the PRB, as illustrated in Fig. 46. In this case, the air is injected into the PRB via a perforated pipe at the bottom of the PRB, and as the air bubbles migrate upwards through the reactive trench filled with high permeability media, typically gravel, volatile organic compounds partition from the dissolved, aqueous phase into the air phase, and are subsequently removed from the system along with the air bubbles. This concept has also been implemented for the removal of dissolved ferrous iron (Fe²⁺) and manganese (Mn²⁺), where the primary focus is on oxidizing these metals to higher valence, immobile forms such as Fe³⁺ and Mn⁴⁺, respectively [171, 172].

However, there are at least three issues with this approach. First, in the case of volatile organic compounds, the air bubbles containing the compounds have to be collected at the ground surface to avoid their uncontrolled release into the ambient environment. Second, as previously noted, air bubbles existing within porous media can clog the pores of the media, thereby reducing the overall



FIG. 45. Concept of a continuous permeable reactive barrier: (a) plan view; (b) isometric view with dimensions.

hydraulic conductivity, which can result in localized flow channels that migrate around air-clogged pores, such that the contaminants are not effectively exposed to the sparged air. Third, the bubbles themselves may channel during upward migration, such that the dissolved contaminant of concern is not uniformly exposed to the sparged air, reducing the efficiency of the treatment.



FIG. 46. Air sparging via a perforated pipe for treatment in a permeable reactive barrier.

6.4.2. Funnel and gate system

As shown in Fig. 47 (a), a PRB may be combined with a low permeability containment barrier (e.g. a SCOW) to direct or funnel the contaminant plume through the PRB. This concept is referred to as a 'funnel and gate' system (e.g. Ref. [148]). In the case of the funnel and gate system, the design has to consider the factors affecting the performance of not only the PRB (e.g. choice of treatment media, length, width and depth of the treatment zone) but also the low permeability containment barrier (e.g. hydraulic conductivity, compatibility, thickness). Integration of the reactive gate with the low permeability barrier can be an important design issue that has led to the use of in situ reaction chambers contained within a SCOW or slurry trench [173].

Another consideration for funnel and gate systems is the requirement for continuity of groundwater flow under steady-state conditions. As illustrated in Fig. 47 (b), this requirement means that the volumetric flow rate of the groundwater (Q) is conserved, such that its magnitude entering the funnel has to be equal to its magnitude exiting the gate. Since the cross-sectional area perpendicular to the direction of groundwater flow at the funnel entrance (A_1 in Fig. 47 (b)) is likely to be greater than that associated with the gate (A_2 in Fig. 47 (b)), the liquid flux through the gate (q_2 in Fig. 47 (b)) is likely to be greater than that entering the funnel (q_1 in Fig. 47 (b)). As a result, the seepage velocity through the reactive gate is likely to be greater than that through the aquifer, resulting in shorter retention times for a given width of reactive media relative to that resulting from the use of a continuous PRB (see Eq. (6)), or the need for wider reactive zones in a funnel and gate system to achieve the desired retention time.



FIG. 47. Conceptual schematic of a funnel and gate system of a permeable reactive barrier: (a) plan view; (b) concept for the continuity of groundwater flow for a funnel and gate system.

6.4.3. Multiple reactive zones in series

The funnel and gate system of a PRB depicted in Fig. 47 may also contain several treatment cells to progressively treat a plume containing one or more contaminants of concern [148]. For example, if the plume comprises groundwater laden with metals with a low pH (e.g. <3), such as acid mine drainage, and the attenuation mechanism relied upon for treatment of the metals is precipitation



FIG. 48. Plan and elevation views of a funnel and gate system with a multiple gate system for treating complex contaminants or mixtures of contaminants [148].

via the use of sulphate reducing bacteria, then a pretreatment zone (e.g. zone I in Fig. 48) may be required to buffer the pH to a level that will not be harmful to the bacteria, which then could be located in a subsequent zone (e.g. zone II in Fig. 48). In this case, the pretreatment zone could comprise coarse-grained limestone or a lime-based material (e.g. CaO, CaCO₃), and the zone containing the bacteria would have to include an organic substrate to facilitate the biologically mediated reduction of the metals. Although the length of the various reactive zones can be maintained constant, the widths of the zones are likely to vary depending on the retention times required to facilitate the attenuation process in each zone.

6.4.4. Other configurations of permeable reactive barriers

Although continuous and funnel and gate PRBs are by far the most common configurations of PRBs that have been employed for field applications, a wide variety of other configurations are possible. Some of these other configurations are described briefly in this subsection to illustrate the range of possibilities.

6.4.4.1. Injection wells

Injection wells can also be used to create a PRB system. As illustrated in Fig. 49, this configuration consists of placing several wells downgradient from a contaminant plume, such that the injection of reactive media into the subsurface results in a continuous, overlapping zone of reactive media. In the case where the density of the reactive media is denser than water, such as micro- or nano-size particles of ZVI, the reactive media are likely to have to be suspended in a viscoelastic gel, such as a solution containing biopolymer guar gum and/or xanthan gum [174, 175]. Guar gum is produced by *Cyamopsis tetragonoloba*, whereas xanthan gum is an extracellular polysaccharide excreted by the bacterium *Xanthomonas campestris* [176]. These two biopolymers are non-toxic, inexpensive, hydrophilic, stable and biodegradable. The biodegradable property of these gels is important, as, after injection, the gels biodegrade relatively rapidly, leaving behind only the reactive media.

The primary drawback to using the injection well system is the uncertainty associated with the spatial distribution of the reactive media after injection (e.g. owing to heterogeneities in the aquifer). This uncertainty can be lessened to some extent by using more than one row of injection wells (Fig. 49), but the cost of the system increases as the number of injection wells increases, and there is still some uncertainty relating to the vertical distribution of the reactive media.

6.4.4.2. Vertical reactive zones

Jefferis et al. [173] describe a case study about an industrial site in Belfast, Ireland, contaminated with chlorinated solvents. Owing to a variety of site-specific considerations, including minimal space between the source of contamination and the site boundaries, a more traditional funnel and gate system could not be emplaced. As a result, a vertical reactive zone configuration was employed as illustrated in Fig. 50. The reactor consisted of a cylindrical steel shell 12 m high by 1.2 m in diameter, filled with ZVI filings placed within an enlarged CB cut-off wall that was used to funnel the groundwater flow to the reactor. The CB wall was keyed into an underlying aquiclude at a depth of 10 m, and the enlargement section was taken to a depth of 12 m to accommodate the reactor shell. The vertical configuration of the reactor zone was required to provide for the necessary retention time where a horizontal gate system 5 m wide would not fit. Because of the relatively low permeability and heterogeneity of the surrounding soils, an upgradient collector and a downgradient distributor filled



FIG. 49. Concept for an injection well system for treating contaminated groundwater: (a) single row of injection wells; (b) double row of injection wells.

with gravel were also constructed to collect the contaminated groundwater and distribute the treated groundwater, respectively. Preliminary results after four and seven months of operation revealed the rapid degradation of the chlorinated solvents with elevated concentrations of chloride due to dechlorination, although there were some issues related to sampling and monitoring.



FIG. 50. Elevation (top) and plan (bottom) views of the vertical reactor system.

6.4.4.3. Refractive zones

Dick and Edwards [177] describe a novel alternative to the funnel and gate system that involves employing the concept of refractive groundwater flow at the interface of two porous media with different hydraulic conductivities, as illustrated in Fig. 51. The concept involves the use of the tangent law, which relates the ratio of the tangent of the incipient angle of flow (θ_1 in Fig. 51 (a)) relative to the tangent of the refractive angle of flow (θ_2 in Fig. 51 (a)) to the ratio of the incipient medium hydraulic conductivity (K_1 in Fig. 51 (a)) to the refractive medium hydraulic conductivity (K_2 in Fig. 51 (a)). If the hydraulic conductivity of the refractive medium is greater than that of the incipient medium (i.e. $K_2 > K_1$), then a trench filled with a high permeability medium can be used to refract the incipient flow to a treatment zone. The difference between this refractive zone approach and the funnel and gate approach is illustrated in Fig. 51 (b). Thus, the refractive zone approach avoids the need to construct a low permeability cut-off wall to funnel the contaminated groundwater zone towards a treatment zone. Although the implementation of the refractive flow and treatment zone approach appears to be lacking, the concept illustrates the virtually limitless possibilities in terms of configurations associated with the use of PRBs for the treatment of contaminated groundwater.

6.5. METHODS OF INSTALLATION

Several methods have been used to install PRB systems. The most common (conventional) excavation methods include excavating trenches using conventional equipment (e.g. backhoes, excavators, cranes) and supporting the excavated trenches using braced sheet-piling or biopolymer slurries, or more recently, using one-pass trencher equipment. Other, less common methods include involving a tremie tube and mandrel technique, deep soil mixing, high pressure jetting and vertical hydraulic fracturing [150]. Brief descriptions of primary methods are provided in the following subsections.

6.5.1. Driving and pulling

The procedure for the installation method needs to brace excavation to maintain the stability of the trench and involves sheeting (i.e. thin sheets of metal). The sheeting is driven into the ground in a pattern that outlines the reactive zone. In the case of a trench, two parallel sheets are driven such that the separation distance represents the width, W, of the reactive zone. This is done by using suitable sheet-pile driving equipment; excavating the native soil using



FIG. 51. Concept of refractive flow and treatment: (a) the tangent law for flow between two materials with different hydraulic conductivity; (b) the difference between refractive flow and treatment and a funnel and gate system.

conventional equipment; bracing the sheeting laterally by placing cross supports (struts); filling the braced, excavated trench with the reactive media while simultaneously removing the bracing during filling; and pulling the sheet-piling to leave the reactive media in place. The walls of the excavated trench may be lined with a filter medium such as a polymeric geotextile before filling to encapsulate the reactive media and to prevent it from migrating outwards, away from the trench. However, there may be concerns related to the clogging of the geotextile, as it could interfere with the effective passage of contaminated groundwater through the reactive zone. Maximum depths of excavation are 10 m for standard excavators, 20 m for modified excavators, and >20 m for cranes fitted with clamshells [150]. This approach is common because most components of the procedure are well established from standard geotechnical practices, but

the method can be expensive, with the expense generally increasing with the size of the PRB system.

6.5.2. Biopolymer slurry

A slurry of water containing biopolymer guar gum and/or xanthan gum (see Section 6.4.4.1), or biodegradable slurry, can be used to support or stabilize excavated trenches in place of braced sheeting. In this case, the biopolymer slurry stabilizes the trench and prevents its collapse by increasing the viscosity of the liquid within the trench relative to the surrounding groundwater. This excludes water from entering the trench, reduces the flow of slurry to the surrounding porous media and facilitates an increased outward pressure against the sidewalls of the trench that increases with depth. The general procedure includes excavating the trench to the desired depth (e.g. to reach bedrock) using a biopolymer slurry to stabilize the trench, backfilling the slurry filled trench with the reactive media (e.g. using a tremie), injecting enzymes to break up the biopolymer within the slurry and capping the trench to protect the top of the reactive zone.

Although using biopolymer slurries is generally easier and less costly than using sheet-piling, possible issues include the biopolymer coating the surfaces of the reactive media, thereby reducing or inhibiting the reactivity of the media; the loss of stabilization during installation resulting from the biopolymer degrading or 'braking' before the reactive media are placed; and the ineffective biodegradation of the biopolymer after the reactive media are placed, resulting in the possibility of zones of high viscosity slurry that can affect the localized groundwater flow through the reactive media.

6.5.3. One-pass trencher

One-pass trenchers are large pieces of equipment that cut or excavate and place reactive media in one continuous process without needing trench stabilization as they do not create an open excavation (Fig. 52). The trench is kept open during installation by a trench box attached to a large chain-saw cutting belt mechanism, and the reactive material is loaded into the trench via a hopper attached to the top of the trench box [150]. Depths typically of up to 14 m are possible, as are trenching rates of 60–120 m/d. The use of one-pass trenchers can lead to significant cost savings if a qualified contractor is nearby.



FIG. 52. Trencher for construction of a permeable reactive barrier (with permission of D. Ruffing, Geo-Solutions, Inc., Pittsburgh, PA, USA).

6.6. SITE CHARACTERIZATION

Site characterization is a critical component of implementing a PRB system for remediation of contaminants of concern. According to the United States Environmental Protection Agency [150], site characterization for PRBs includes four components: hydrogeological characterization, contaminant characterization, geochemical characterization and microbial characterization. Brief descriptions of each of these characterizations are provided in the following subsections; further details are provided in Ref. [150].

6.6.1. Hydrogeological characterization

Hydrogeological characterization pertains to determining (i) the direction and magnitude of groundwater flow to locate and size the PRB, and (ii) the stratigraphy (the order and relative position of the geologic strata) and lithology (general physical characteristics of rocks) at the site to determine what type of PRB design to choose. Factors pertinent to groundwater include determining the magnitudes of the relevant parameters affecting groundwater flow (i.e. hydraulic
conductivity, porosity and hydraulic gradient), evaluating the potential seasonal variations in groundwater flow, and ascertaining any other factors that could potentially affect the direction and magnitude of groundwater flow, such as heterogeneity (spatial variation) and/or anisotropic (directional) variation in the hydraulic conductivity. Factors relevant to stratigraphy and lithology include whether or not the PRB can be keyed into an underlying low permeability aquitard or aquiclude, the potential for subsurface strata to interfere with the installation of a PRB (e.g. buried rock interfering with the driving of sheet-piling) and any potential stratigraphic influence on the direction and magnitude of groundwater flow.

For example, the flow through a PRB cannot be higher than the flow that can be delivered by the upstream geology and dissipated by the downstream geology. Furthermore, the permeability of a PRB is likely to decrease over time owing to biofouling or chemical precipitation, and such a reduction of permeability may cause the PRB to act as an unwanted barrier. As a result, the initial permeability of the PRB should be much higher than that of the surrounding soils so that clogging will not have any significant impact on the design life of the PRB. Therefore, based on these factors, the design should aim to provide a sufficiently high permeability for the wall with the groundwater flow controlled by the external geology. This knowledge allows a much more confident estimation of flow rates and their seasonal variation.

6.6.2. Contaminant characterization

Contaminant characterization pertains to determining the dimensions of the contaminant plume, the composition of the contaminants of concern within the plume and their concentrations. In terms of effective treatment of the contaminant of concern by the PRB, consideration needs to be given to the maximum possible concentrations to which the PRB may be exposed.

6.6.3. Geochemical characterization

Aside from the composition and concentrations of the contaminants of concern within the plume, knowledge of other geochemical parameters that can potentially affect the performance of the PRB is required. Such factors can include, but are not necessarily limited to, pH, redox potential, dissolved oxygen concentration, dissolved suspended solids and total suspended solids (e.g. organic matter), and the concentrations of other dissolved chemical species within the groundwater and groundwater plume.

For example, groundwater with high concentrations of dissolved carbonate $(CO_3^{2^-})$ can result in precipitation of calcite $(CaCO_3(s))$ that can clog the PRB and/or reduce the reactivity of the reactive media contained within the PRB.

6.6.4. Microbial characterization

Knowledge of the natural microbial communities is important in order to understand their potential effects on the contaminants of concern, and PRB performance, which can be either beneficial or detrimental. Potential beneficial effects include natural attenuation of contaminants of concern within the groundwater plume, and enhanced contaminant degradation within or downgradient from the PRB (e.g. a reduction in redox potential owing to the presence of sulphate-reducing bacteria may increase the rates of contaminant reduction). A detrimental effect is a potential decrease in the hydraulic conductivity of the PRB owing to microbial buildup within the PRB (i.e. biofouling).

6.7. TREATABILITY STUDIES

Treatability studies are studies that are performed before implementing a full scale PRB and are necessary to determine whether or not the contaminants of concern can be treated that way. More specifically, treatability studies are required to match the physical, chemical and/or biological processes associated with the treatment of specific types of contaminants of concern (i.e. match the attenuation mechanism to the contaminant of concern). In this regard, treatability studies can be related to the scale of the evaluation (i.e. laboratory versus field).

6.7.1. Laboratory scale studies

Two general types of laboratory scale tests are conducted to evaluate treatability: batch tests and column tests. These provide the information necessary for the initial design of the PRB system to be installed in the field.

(a) Batch tests: these tests are conducted primarily to evaluate the ability of specific reactive media to attenuate the contaminant of concern under no-flow (static) conditions. These tests are usually conducted in small reaction vessels (e.g. in sealed glass tubes, vials or bottles) containing a constant aqueous volume, contaminant concentration and reactive media mass [150]. The reaction vessel is then shaken for a specified time, typically 24 or 48 hours, and the concentration of the contaminant is measured to determine the extent of degradation over the time frame for the reactive media. Batch tests are typically faster, cheaper and simpler than column tests, such that several tests can be set up to evaluate different candidate reactive media and/or the degradation rate (e.g. by shaking reaction vessels with the same contents for the different durations such as 24, 48 or 96 hours). However, the results of batch tests should be considered with caution because of the lack of liquid flow.

Column tests: these tests are conducted using the same materials as those (b) in batch tests, but instead of mixing the materials under no-flow conditions, the reactive media are placed within a column, typically made of glass or plastic, and the aqueous solution containing known concentrations of the contaminant of concern is permeated through the column to mimic the situation expected in the field. Contaminant concentrations in the influent and effluent are measured as a function of time to establish the temporal effect of the reactive media in attenuating the contaminants. Sampling ports can also be installed along the length of the column to provide a measurement of the spatial distribution of the contaminants within the column, but the recovery of samples from these ports as well as the potential disturbance to flow through the column resulting from sampling can be problematic. Measurements of other geochemical parameters (e.g. pH, redox potential, dissolved oxygen) and concentrations of chemical species other than the contaminants should be considered to evaluate the potential influence of these geochemical parameters on the attenuation of the contaminants.

The primary advantage of column tests versus batch tests is that column tests allow for an evaluation of the effect of the rate of groundwater flow on the attenuation of the contaminants, thereby providing a more realistic evaluation of the expected performance of the PRB in the field. However, because column tests are typically conducted for longer periods than batch tests and require more maintenance than batch tests, cost or other factors (e.g. lack of equipment or materials) may limit the number of column tests that can be conducted. As a result, column tests are typically conducted based on the results of the batch tests, to limit the number of column test to be evaluated via column testing.

6.7.2. Field scale studies

Because the scale of batch and column testing in the laboratory is typically limited and/or the conditions in the laboratory are not the same as those in the field, an evaluation in the field of a smaller version or prototype of the full scale PRB is desirable. This type of testing is often referred to as pilot scale testing. Although the pilot scale PRB does not have to be full scale, its dimensions should be sufficient for evaluation. For example, for a continuous PRB, the width of the pilot scale PRB should probably match the full scale PRB to provide a realistic assessment of the estimated retention time required for attenuation of the contaminant of concern, but the length need not match the full scale PRB. The pilot scale PRB should be constructed at the field site using the same materials and construction methods expected for the full scale PRB. Finally, an adequate monitoring programme is required to evaluate the performance of the pilot scale PRB. This programme usually includes installing sufficient monitoring wells both downgradient and upgradient from the pilot scale PRB, and possibly within the pilot scale PRB, as well as adequate sampling and chemical measurement protocols to provide the requisite information for evaluating performance.

When the evaluation of a pilot scale PRB indicates inadequate performance, adjustments in the design and/or construction of the full scale PRB are likely to be required. As a result, a morphological assessment of the failed pilot scale PRB should be conducted to ascertain the potential causes for failure, to avoid wasting the time and cost invested in constructing and evaluating the pilot scale PRB, and ideally to avoid similar problems with the full scale PRB.

6.8. POTENTIAL PERFORMANCE ISSUES

Several issues can result in the poor performance of PRBs. Some of these issues are described in Table 17 [149].

Issue	Description	
Construction-related	 Aquifer sediments mixing with reactive media Smearing at the trench interface Short term effects (e.g. reduction from biopolymer guar before it is broken with injected enzymes) Issues during construction (e.g. sloughing of the trench sidewalls) 	
Biofouling	 Direct clogging of the porous media via microbial accumulation and/or indirect clogging of the porous media via biologically facilitated reactions, such as precipitation of metals or compounds 	

TABLE 17. POTENTIAL ISSUES AFFECTING THE PERFORMANCE OF PERMEABLE REACTIVE BARRIERS^a

TABLE 17. POTENTIAL ISSUES AFFECTING THE PERFORMANCE OF PERMEABLE REACTIVE BARRIERS^a (cont.)

Issue	Description	
Mineral fouling	 Precipitates or reaction by-products clog the pores resulting in a reduction of hydraulic conductivity and preferential flow paths that can reduce retention (reaction) times (does not have to be facilitated by microbes) 	
Passivation	 Reaction by-products coating or insulating the surfaces of reactive media, thereby reducing reactivity 	
Duration of reactivity	 Exhaustion of reactive capacity (e.g. cation exchange capacity) requiring rejuvenation or replenishment of reactive media 	
Aquifer heterogeneity	 Inadequate width or length of the permeable reactive barrier owing to aquifer heterogeneity 	

^a From Ref. [149].

In particular, aquifer heterogeneity can have a major impact on PRB performance [169]. For example, consider the scenario depicted in Fig. 53, where a continuous PRB is to be located within a higher hydraulic conductivity aquifer interspersed with lenses of a lower hydraulic conductivity medium (e.g. silt lenses). In this case, the predominant groundwater flow will occur between the lower hydraulic conductivity lenses, such that some areas of the PRB will be overexposed to flow (areas A in Fig. 53), while other areas of the PRB will be underexposed to flow (areas B in Fig. 53). Certainly, homogeneous placement of the reactive media within the PRB would tend to distribute the flow more uniformly once the groundwater enters the PRB, but not necessarily to the extent that the PRB is sufficiently wide to provide an adequate retention time for effective attenuation of the contaminant of concern. Also, the flow may completely bypass the PRB (area C in Fig. 53) such that any contamination associated with this flow would go unmitigated by the PRB.



FIG. 53. Effect of heterogeneity on the performance of a permeable reactive barrier. K — hydraulic conductivity; A — areas overexposed to flow; B — areas underexposed to flow; C — area bypassed by flow.

Hemsi and Shackelford [169] provide factors of safety related to increasing the length and width of PRBs to account for aquifer heterogeneity. However, the values of factors of safety are a function of statistical distributions of measured hydraulic conductivity values, and the cost of those measurements may be prohibitive. One possible alternative approach to deal with heterogeneity is to place a trench filled with non-reactive media with high hydraulic conductivity (e.g. pea gravel) along the upgradient length of the PRB to collect and redistribute the flow, such that contamination is distributed evenly across the face of the PRB before entering it. The length issue can be addressed by employing a funnel and gate system, with a funnel that is sufficiently large to ensure that the complete plume is captured.

6.9. QUALITY ASSURANCE AND MONITORING

A QA plan should be developed before PRB installation to ensure that the PRB functions as required. This plan should specify all procedures and techniques used for site characterization, groundwater monitoring, well installation and development, sample collection, preservation and shipment, sample analysis and

chain-of-custody control [178]. Both compliance and performance monitoring are integral components of such a plan:

- Compliance monitoring: monitoring that is conducted to ensure compliance with any pertinent environmental regulations, such as maximum contaminant levels for the contaminant of concern. Sampling to measure applicable water quality parameters (e.g. concentrations of contaminant of concern and potential by-products or intermediate chemical species resulting from the reactions between the reactive media and the contaminant of concern, as well as any other relevant water quality parameters, such as pH, redox potential) is generally conducted at one or more designated points of compliance located downgradient from the PRB.
- Performance monitoring: monitoring that is related to the actual performance of the PRB, and that differs from compliance monitoring primarily in that sampling for performance monitoring is conducted within or close to the PRB.

Both compliance and performance monitoring include details pertaining to each of the following four topics (e.g. Ref. [178]):

- (1) Monitoring system design: the number and location of monitoring wells required for compliance and for performance assessment. In addition to upgradient and downgradient wells, wells should be located to ensure that contaminated water is not flowing around or under the PRB ([178]). The number and location of monitoring wells will also be a function of the type of PRB system installed (e.g. continuous versus funnel and gate system).
- (2) Sampling methods: the methods used for the recovery and protection of samples. For example, a common practice is to purge monitoring wells by bailing several well-volumes of groundwater before recovering a sample for measurement of the key water quality parameters, to ensure that the recovered samples provide the best estimates of the actual groundwater immediately outside but adjacent to the well. Furthermore, any samples recovered should be properly handled to avoid inaccurate measurements and, therefore, a wrong interpretation of the water quality parameters. For example, samples of chemicals that are volatile or semi-volatile need to be sealed immediately and properly after sampling.
- (3) Sampling frequency: the frequency at which samples are recovered, which should be sufficient to provide accurate information concerning compliance and performance. To a large extent, the frequency of sampling is a function of the groundwater flow rate, with more frequent sampling associated with faster groundwater flow rates.

(4) Sample measurements: the specific water quality parameters that are to be measured to ensure a proper evaluation of compliance and performance. The water quality parameters should include parameters associated with the characterization of the groundwater plume as well as parameters associated with the specific reaction mechanisms for attenuating the contaminant of concern within the PRB.

6.10. OTHER CONSIDERATIONS

Other considerations associated with the installation of a PRB for site remediation include contingency planning and institutional controls. Contingency planning refers to planning for a scenario in which the PRB is not functioning as desired or required. This includes, for example, the remedies to be taken if the reactive capacity of the reactive media within the PRB diminishes or becomes exhausted (Table 17). Depending on the time frame of operation and compliance, contingency planning may be a component of institutional controls.

Institutional controls are the actions taken by an institution for long term site management and control for the protection of human health and the environment [179]. There are two types of institutional controls: (i) active institutional controls, which include controlling site access, performing maintenance or remedial actions, controlling or remediating releases, and system monitoring; (ii) passive institutional controls, which include land ownership or use requirements, placing markers to warn the public, and public records, archives or other methods of preserving the knowledge of the hazards of a site.

6.11. CASE STUDIES

There are numerous case studies associated with the use of PRBs for remediating a wide range of contaminants of concern (e.g. see Refs [149, 150, 180]). However, two case studies from the USA that are particularly relevant to this publication are the funnel and gate PRB installed at the uranium mill tailings site in Monticello, Utah, for the removal of uranium via reduction by ZVI, and the continuous PRB system installed at West Valley, New York, for the removal of radionuclides via adsorption by zeolite.

6.11.1. Mill tailings site in Monticello, Utah

A funnel and gate PRB was installed in the alluvial aquifer at a tailings site near Monticello, Utah, USA, in June 1999 to capture and treat the primary



FIG. 54. Funnel and gate permeable reactive barrier at Monticello, Utah, USA, for the removal of uranium, vanadium, selenium, arsenic, molybdenum and nitrate (adapted from Ref. [171]).

contaminants of concern including arsenic, molybdenum, nitrate, selenium, uranium, and vanadium. The tailings were derived from a uranium and vanadium mill that was in operation from 1942 to 1960.

The funnel and gate system consisting of low permeability SB SCOWs comprised a funnel system and a reactive gate (Fig. 54). The gate comprising the three reactive zones shown in Fig. 54 was 31 m long and 2.4 m wide. The top of the gate was 0.91 m below the ground surface, the bottom was keyed 0.30–0.61 m into low permeability claystone that was approximately 4 m below the ground surface, and the PRB was approximately 3 m high.

The gate was constructed by driving sheet-piling using a 127 MG hydraulic vibratory hammer attached to a 115 MG crane until refusal in bedrock, and then withdrawing the sheet-piling after replacing alluvial materials with reactive materials. The three zones of the gate included (i) a 0.61 m wide pretreatment zone of pea gravel mixed with 12% ZVI by volume; (ii) a 1.2 m wide zone of 100% ZVI; and (iii) a 0.61 m wide zone of 100% pea gravel with an air sparging pipe. The diameter of the ZVI particles ranged from 0.83 mm to 2.63 mm, with a density of 1.84 mg/m³. The ZVI was used to chemically reduce the concentrations of contaminants of concern for subsequent precipitation (e.g. uranium is reduced and subsequently precipitated as uraninite (UO₂) or an amorphous precursor), whereas the third (air sparging) zone was installed to control dissolved iron and manganese concentrations in the groundwater at the downgradient side of

the gate [171]. Immediately following the installation of the funnel and gate system, an extensive monitoring well network was installed to evaluate the performance of the PRB.

Within a year of installation, the PRB was effective in reducing the concentrations of the contaminants of concern. Concentrations of arsenic, selenium, uranium and vanadium exiting the PRB were below reporting limits. Also, concentrations of molybdenum were reduced to nearly non-detectable levels within the PRB.

The hydraulic conductivity of the PRB was measured several times in the 2000–2005 period using slug tests [181]. Its mean value decreased during this period from 2.0×10^{-4} m/s to 4.6×10^{-7} m/s. This decrease in the hydraulic conductivity of the PRB was attributed to precipitation of CaCO₃(s) and other ZVI corrosion products. The significant decrease caused groundwater to mound the upgradient of the PRB, resulting in some contaminated groundwater flowing around the outside of the funnel without treatment [181]. As a result, several alternatives for repairing the existing PRB were evaluated, including (i) extracting and treating groundwater at the PRB; (ii) placing ZVI in borings; (iii) placing ZVI in borings using removable canisters; (iv) jet grouting ZVI; (v) injecting ferric iron; (vi) injecting ferrous iron; and (vii) injecting dithionite [182]. The first alternative (i.e. extracting and treating groundwater at the PRB) was implemented and evaluation of performance is ongoing.

6.11.2. Spent radioactive fuel processing plant, West Valley, New York

A continuous PRB, referred to as a permeable treatment wall (PTW), was installed in November 2010 at a former spent radioactive fuel processing plant located at West Valley, New York, USA, to attenuate 90 Sr in groundwater at a portion of the 68 ha site [178]. The PTW was filled with granular zeolite (clinoptilolite) to attenuate the 90 Sr via cation exchange [158, 183, 184]. The installation of the PTW was based on experience gained from monitoring the performance of a ~9 m long by 3 m wide pilot scale PTW that was installed at the site in 1999. The full scale PTW installed in 2010 was keyed into an underlying, low permeability unweathered till and was 259 m long, 1 m wide, and had a depth of 6–9 m. The trench was excavated and filled with ~2600 t of zeolite using a one-pass trencher (Fig. 52).

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ABBREVIATIONS

CB	cement-bentonite
CCL	compacted clay liner
CSB	conventional sodium bentonite
CSPE	chlorosulfonated polyethylene
GCL	geosynthetic clay liner
GML	geomembrane liner
HDPE	high density polyethene
LCRS	leachate collection and removal system
LLDPE	linear low density polyethene
PRB	permeable reactive barrier
PTW	permeable treatment wall
PVC	polyvinyl chloride
QA/QC	quality assurance/quality control
SB	soil-bentonite
SBL	sand-bentonite liner
SCB	soil-cement-bentonite
SCOW	slurry cut-off wall
SWCC	soil water characteristic curve
TSF	tailings storage facility
ZVI	zero valent iron

GLOSSARY

- **Geomembrane.** A geomembrane is very low permeability synthetic membrane liner or barrier used with any geotechnical engineering related material so as to control fluid (liquid or gas) migration in a human-made project, structure or system. Geomembranes are made from relatively thin continuous polymeric sheets, but they can also be made from the impregnation of geotextiles with asphalt, elastomer or polymer sprays, or as multilayered bitumen geocomposites. Continuous polymer sheet geomembranes are by far the most common.
- **Geotextile.** A synthetic fabric used to stabilize soils, retain soils, prevent the mixing of dissimilar soils, provide a filtering function, pavement support, subgrade reinforcement, drainage, erosion control and silt containment. (http://www.geotechnicalinfo.com/geotechnical_glossary.html#g)
- Leaching. The removal of salts and soluble substances as water flows through a soil medium. Leachate is the leached liquid from the soil, rich in minerals. (http://www.geotechnicalinfo.com/geotechnical_glossary.html#g)
- **Liner.** A relatively impermeable barrier (e.g. plastic or dense clay) designed to prevent leachate from leaking from a landfill. Natural and synthetic liners may be utilized as both a collection device and as a means for isolating leachate within the fill to protect the soil and groundwater below. A protective layer is either an unaltered layer of geologic material or a combination of layers. Constructed liners are systems comprised of compacted soil or synthetic material. The layer or liner needs to provide equal or greater protection than the regulations require for specified manure collection and storage areas. To protect the ground water from landfill contaminants, clay liners are constructed as a simple liner that is 0.6–1.5 m thick. In composite and double liners, the compacted clay layers are usually between 0.6 m and 1.5 m thick, depending on the characteristics of the underlying geology and the type of liner to be installed.
- **Lining.** Lining is the process of reducing seepage loss of liquids by adding an impermeable layer to a given structure (e.g. the edges of the trench).
- Seepage. The flow of water through soil.

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Containment barrier systems are among the most widely used technologies for remediating contaminated sites. Various structures have been engineered to address site-specific needs, while barrier selection depends largely on whether regulatory requirements are prescriptive or performance based. This publication provides an introduction to the design and construction of different containment barriers for low-level radioactive waste generated from remediation activities: basal (bottom) liners, final covers, in situ vertical barriers and in situ permeable reactive barriers. Practical aspects of each structure are discussed in theoretical case studies, which allow remediation project designers, implementers and regulators to make more informed decisions about the use of these barriers.