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A Review on the Hydraulic Conductivity of Geosynthetic Clay Liner

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ABSTRACT

In landfill design, the containment of solid and liquid contaminant is essential. Leachate is produced from the liquid squeezed out of the waste itself (primary leachate) and by water that infiltrates into the landfill and that percolates through the waste (secondary leachate). This liquid can become a health hazard if it enters into the groundwater system. Liners are placed beneath leachate collection systems to avoid leachate from seeping into the soil underneath the landfill. The liner system is the single most important element of alandfill.Geosynthetic clay liners (GCLs), usually containing bentonite clay, are widely used as bottom liners and final covers for landfills. However currently used GCLs suffer problems of slope stability due to low shear strength. A shearing failure involving a Geosynthetic Clay Liner can occur at three possible locations: (1) The external interface between the top of the GCL and the overlying material (Soil or geosynthetic); (2) internally within the GCL;and (3) the external interface between the bottom of the GCL and the underlying material. MSW Landfills undergo settlement during and after their active life. This settlement is caused by the gradual decomposition and consolidation of the waste mass. It is essential that the landfill cover system be capable of sustaining the anticipated differential settlement of the waste mass.

Keywords: Landfill, Geosynthetic Clay Liners (GCL), Slope stability, Differential Settlement, Municipal Solid Waste (MSW).

I. INTRODUCTION

Waste is commonly referred as material with no consumer value after abandonment. According to World Health Organization, (WHO) the term ‘solid waste’ is applied to unwanted and discarded materials from houses, street sweepings, commercial and agricultural operations arising out of mass activities. Waste maybehandled or disposed in three ways: burial, incineration, or reusing/recycling (Qian et al. 2002). An Engineered landfill is a controlled method ofwaste disposal to reduce the environmental and health impacts of waste. The main function of a landfill design is to contain the liquid and gas pollutants from contaminating the surrounding areas and their proper disposal. (Qian, X., Gray, D. H, and Koerner, R. M.,2001).

Leachate is the contaminated liquid generated in landfills, which consists of solutes from the solid waste dissolved in liquid either squeezed from the waste itself or water that infiltrates into the landfill and percolates through the waste (Qian et al. 2002, Yesillerand Shackelford 2010). Escape of leachate from the landfill can cause environmental hazard. Hence, it is of primary importance that the landfills have an effective hydraulic barrier to prevent movement of leachate into the environment and ensure their proper collection and disposal.

Most municipal solid waste landfills, are composed of seven systems (R M. Koerner., 2001)

a. Bottom and lateral side linerssystem
b. Leachate collection and removalsystem
c. Gas collection and controlsystem
d. Final coversystem
e. Storm water managementsystem
f. Groundwater monitoringsystem
g. Gas monitoringsystem

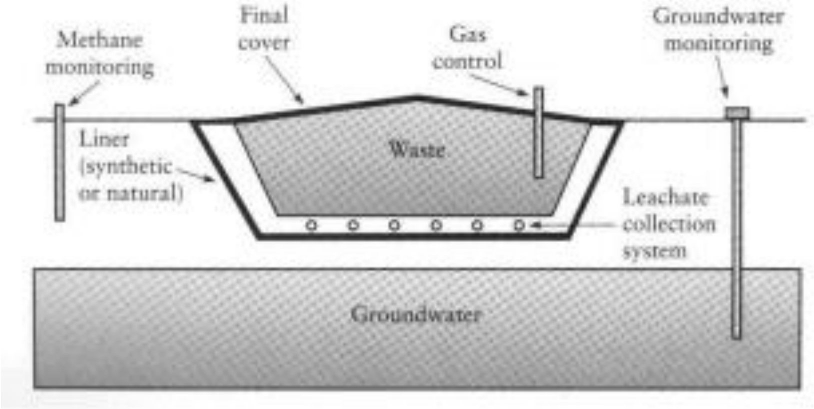

Fig.1:Schematic Diagram of a MSW Landfill Containment System

The above systems are used to ensure that pollutants generated in the landfill waste are efficiently contained and disposed. The waste in landfills may be placed in a variety of configurations. Some common geometrical configurations for waste placement are presented in Fig. 1

Area fills involve placing waste on areas with little to no excavation. This type of landfill is used in areas with high groundwater tables or where the terrain does not allow for excavation (Qian et al. 2002). Trench fills are series of deep and narrow trenches. Because of their geometry, trench fills are generally only used tocontain small quantitiesof waste compared to other types of landfills. Above and below ground fill is like a combination of the Area Fill and Trench Fill. The depthof the excavation generally depends on the depth of the groundwater table and natural clay layer. Canyon or valley hills consist of waste placed between hills or rolling terrain (Qian, X., Gray D. H. and Koerner R. M.2001).

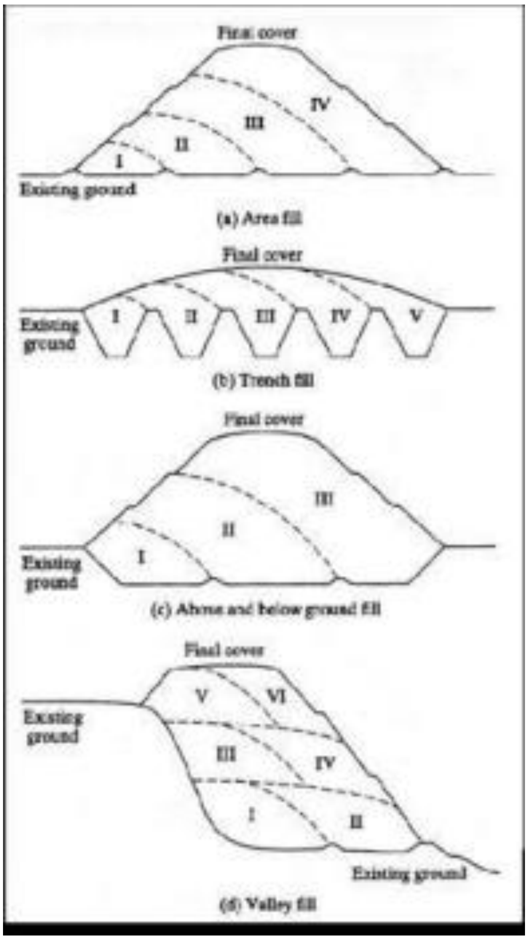

Fig. 2: Common Landfill Configurations (Qian et al. 2002)

Landfills are lined with systems that consist of alternating layers of low and highly permeable materials that perform barrier functions and drainage or filtration functions, respectively. The two basic types of liner systems are single and double liner systems. Single liner systems include a hydraulic barrier layer overlain by a high permeability layer called the leachate collection system. Double liner systems include two hydraulic barrier layers that are separated by a drainage layer, known as the leak detection system, and overlain by the leachate collectionsystem.

The barrier component of the liner system isthe component that performs containment functions and has two types: Single and Composite Liners. Single liners consist of a Geosynthetic clay liner (GCL), single compacted clay liner (CCL), or a Geomembrane (GM) (Environmental Protection Agency, USA). Single liners are sometimes used in landfills designed to hold construction and demolition debris (C&DD). Examples of single liners are presented in Fig. 3. (Kerry L. Hughes, Ann D. Christy, and Joe E. H.2005)

Composite liners consist of a paired GCL and GM or a paired CCL and GM. Composite -liners are required in municipal solid waste (MSW) landfills. Examples of composite liners are presented in Fig.4. Composite liners combine the advantages and eliminate the disadvantages of single liners. The GCL (or CCL) beneath thegeomembraneminimizes the impact of holes in the geomembrane byserving as a low permeability material that is relatively resistant to formation of holes or macro voids when hydrated (Joe et al.2005).

A doubleliner consists of either two single liners, two composite liners, or a single and a composite liner as shown in Fig.5. The upper (primary) liner usually functions to collect the leachate, while the lower (secondary) liner acts as a leak- detection system and backup to the primary liner. Double-liner systems are used in some municipal solid waste landfills and in all hazardous wastelandfills.

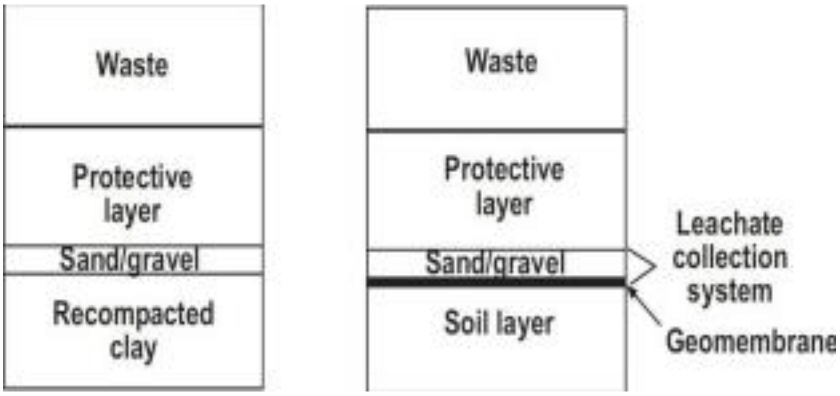

Fig. 3: Single Liner Systems (Kerry L. Hughes, Ann D. Christy, and Joe E. H. 2005)

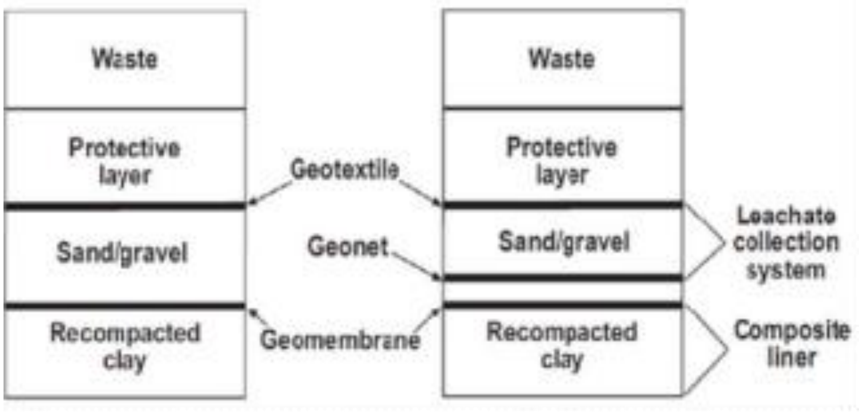

Fig. 4:Composite Liner Systems

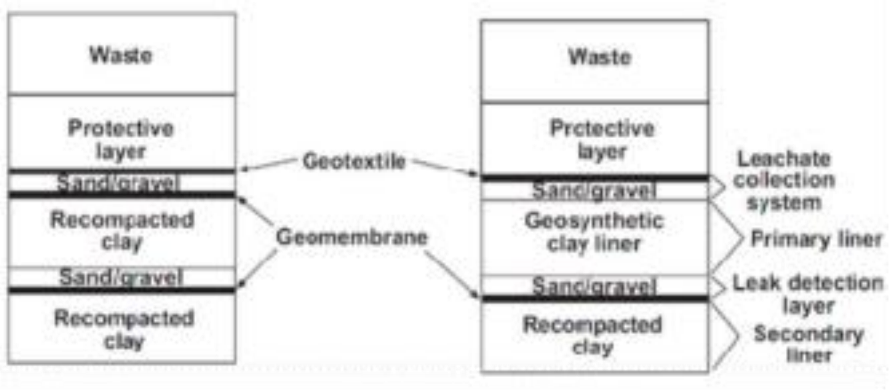

Fig. 5:Double Liner Systems

II. GEOSYNTHETICS USED IN A LANDFILL CONTAINMENT SYSTEM (OAC 3745-27-08,USEPA)

Geomembrane:

Geomembrane are also called flexible membrane liners. These liners are constructed from various plastic materials, including polyvinyl chloride (PVC) and high-density polyethylene (HDPE). The preferred material for use in MSW is HDPE. This material is strong, chemical resistant and is considered to be impervious. Therefore, HDPEminimizes the transfer ofleachate from the landfill totheenvironment.Thethicknessof

Geomembranes range from 0.75 to 3mm. HDPE Geomembrane must have a minimum thickness of 1.5 mm for use in MSWlandfills.

Geotextiles:

In landfill liners, Geotextiles are used to prevent the movement of small soil particles and refuse particles into the leachate collection system and to protect Geomembrane from punctures. These materials allow themovement of water but trap particles to reduce clogging in the leachate collectionsystem.

Geonet:

A Geonet is a plastic net-like drainage blanket which may be used in landfill liners in place of sand or gravel for the leachate collection system. Sand andgravel are usually used due to cost considerations, and because Geonets are more susceptible to clogging by small particles. This clogging would impair the performance of the leachate collection system. Geonets convey liquid more rapidly than sand andgravel.

Geogrid:

Geogrids are used in a landfill containment system to reinforce slopes beneath the waste as well as for veneer reinforcement of the cover soils above geomembranes (Zornberg et al.2001).

Geopipe:

A geopipe systemis used in the leachate collection layer to facilitate collection and rapid drainage of the leachate to a sump and removal system. Geopipes are also used in sidewall risers and manholes for removingleachate.

III.GEOSYNTHETIC CLAY LINERS (GCL)

GCLs were introduced to the waste containment area in the mid -1980s, represent a distinct difference and challenge to compacted clay liners (CCLs) (Koerner et al., 2002). ASTM D4439 - 11 defines a Geosynthetic Clay Liner as “a manufactured hydraulic barrier consisting of clay bonded to a layer or layers of Geosynthetic materials”. Essentially, most GCLs consist of a bentonite clay either calcium or sodium core held between two Geotextiles. The Geotextiles ma y be woven or nonwoven, depending on the desired strength characteristics. The GCLs currently available vary by type of geotextile, the inclusion of a geomembrane, the presence and type of reinforcement, and thegradation and type of the bentonite (Koerner1997).

3.1 Types and current uses of GCLs

The Geotextiles are held together by adhesive bonding (Fig. 6a, Fig.7a); needle punching (Fig. 6c, Fig.7b, c), or stitch bonding (Fig.6c). Some GCLs also contain a plastic film or geomembranecomponent,eitheradheredtoageotextile encased GCL (Fig.7) or bonded directly to bentonite clay by an adhesive (Fig.6b). The unreinforced GCLs include geotextile- encased, adhesive bonded GCLs and geomembrane-supported, adhesive-bonded GCLs. Unreinforced GCLsare held together by chemical adhesives and, once hydrated, provide relatively low resistance to shear. The reinforced GCLs include both geotextile-encased, stitch- bonded GCLs and needle-punched GCLs. The integrated matrix of bentonite and needle-punched fibers provides high internal shear strength. The increase in internal strength achieved by stitch bonding the encapsulating Geotextiles together is veryconsiderable.

Peak shear strengths for the unreinforced GCL products were found to be similar to those for bentonite (i.e. verylow shear strength), which makes them prone to instability. This is the reason they are not usually recommended forslopes steeper than 10H: 1V(Frobel, 1996; Richardson, 1997). The reinforced GCLs shows greater internal shear strength due to the presence of the fibers. However, itwas also shown that their behavior was governed by the fibers resistance against pull-out and tearing of the reinforcing fibers and the shear strength of the bentonite. It isworth noting that despite the fact that it was shown, in the laboratory, that internal failure could occur in reinforced GCLs, there are no known cases of slope failures, which can be attributed to internal shear failure of reinforced GCLs. Laboratory interface shear tests are normally conducted to evaluate interface friction between GCLs and soils or geosynthetics under operating conditions. As a result, a more extensive database is now available (Garcin et al., 1995; `Bressi et al., 1995; Feki et al., 1997; Gilbert et al., 1996; Von Maubeuge and Eberle, 1998; Eid et al., 1999; Triplett and Fox,2001).

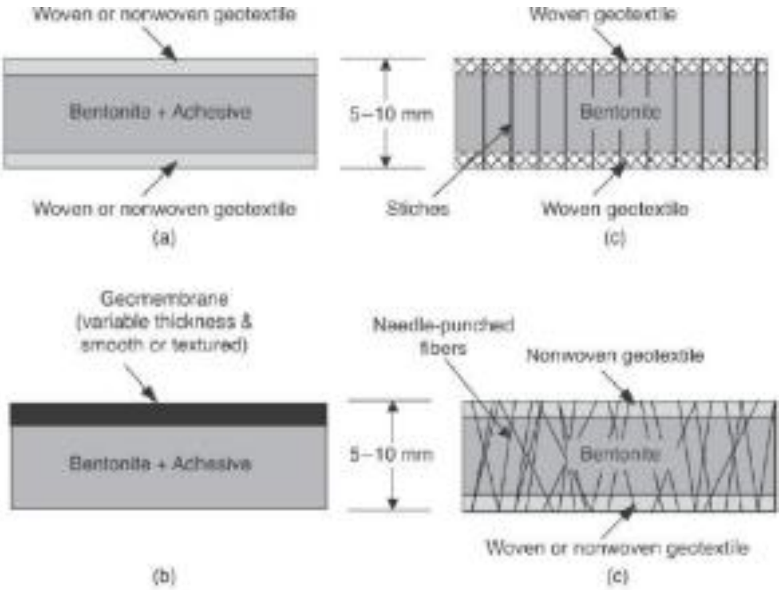

Fig. 6:Conventional GCLs available (Yesiller and Shackelford 2010)

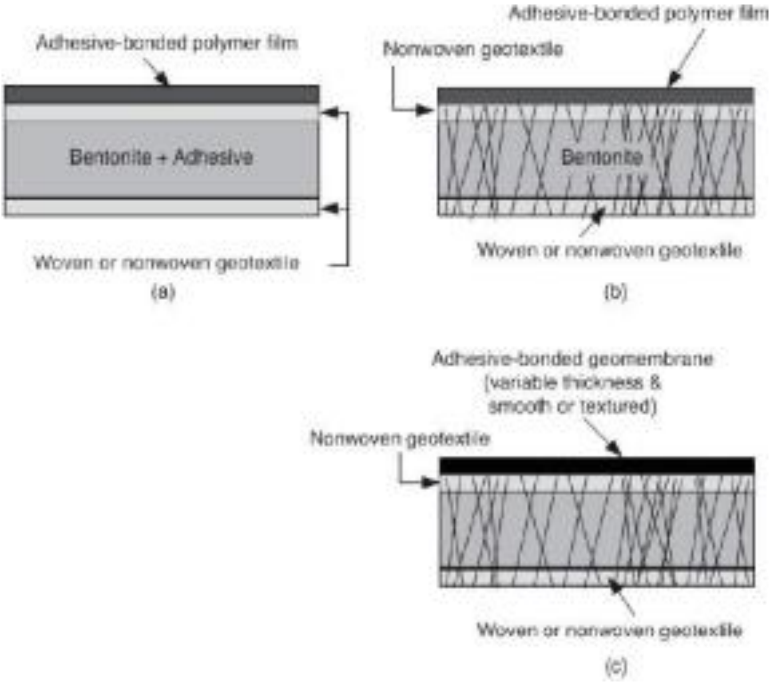

Fig.7:GM-Backed, GT Encased GCLs (Yesiller and Shackelford 2010)

3.2 Advantages and Disadvantages of GCL

The main advantages of GCLs are their low thickness and ease of installation. A GCLof approximately 5-10 mm thickness has comparable advective flux to a CCL of about 1 m thickness. The reduced thickness allows for greater economy, as the difference in thickness can be used as volume for waste placement in a landfill with a thinner barrier. GCLs are easy to install, as they are flexible and are manufactured in large rolls. GCLs can be installed by a small crew of workers unrolling the product from a large spool. This compares to transporting loads of clay to a landfill site and then compacting the clay in lifts with a sheep foot compactor and smoothing the surface of the clay with a heavy smooth drum roller, as would be required for a CCL (Koerner 2005, Wagner and Schnatmeyer 2002). Quality assurance is simpler for GCLthan for a CCL. GCLs are not vulnerable to desiccation damage during construction and may be less vulnerable after construction (USEPA, 1993). Because GCLs are lighter than CCLs, GCLs tend to cause less settlement of underlying waste in a final cover system. In addition, the GCL themselves can tolerate appreciably more differential settlementthan CCLs. Hydrated GCL is an effective gas barrier (A. Bouazza,2001).

Major disadvantages of GCLs are high levels of diffusive flux dueto the low thickness and sensitivity to temperature cycling, wet-dry cycling, and cationexchange.The sensitivity to temperature cycling, wet-dry cycling, and cation exchange factors has caused problems with GCL panels shrinking resulting to increasing hydraulic conductivity.(Thiel and Richardson 2005, Koerner 2005). GCLs with sodium bentonite are prone to ion exchange. Bentonite has low shear strength when hydrated and is susceptible to lateral squeezing under rapid changes in loading. A potential strength problem arises at interface with other materials (A.Bouazza, 2001).

IV. CATION EXCHANGE IN GCL

Permeants with lower ionic charge(NaCl solution) cause greater swelling because morewatermoleculesorcationsarerequiredtobesorbedtobalancethenegative charge of the clay particle. Exchange ofCa2+ for Na+ is thermodynamically favorable inclays (Benson et al. 2007). If a sodium bentonite is exposed to permeants with Ca2+ inthe pore water of the GCL, cation exchange will occur as one divalent cation will replace two sorbed monovalent cations, unless a far greater abundance of monovalent ions exist in the permeant fluid to offset the influence of the divalent cations (Benson et al. 2007). This process in turn will reduce the amount of swelling, which will increase the hydraulic conductivity of the GCL in a relativelyshort duration of time.

Multivalent cations(Ca+2, Mg+2) can be supplied from various sources. Most test data simulate the multivalent cations being supplied byleachate moving downward into the GCL. Multivalent cations can come from the soil underneath the GCL as well (Touze- Foltz et al. 2006). This is certainly a concern when placing GCLs above soils rich in calcium carbonate. Care needs to be takento ensure that sources of multivalent cations, both above and below the liner system, are accounted for whenever designing a GCL component.

4.1 Prehydration

Prehydrationof GCLs is a proposed technique to ensure that GCLs maintain low hydraulic conductivity when permeated with fluids containing multivalent cations. Prehydrationis the hydration of GCLs with water prior to the GCL being subjected tochemicals (Jo et al. 2004). Prehydration ensures that GCLs have undergone osmotic swelling before cation exchange can occur, which allows the permeability of the GCL to remainlow.

Some GCLs manufactured in Europe are prehydratedin the factory to improve hydraulic performance (Mazzieri 2011, Mazzieriand Pasqualini 2011). Prehydrationmaybe ineffective when wet dry c yclesare intense, as the benefits of osmotic swelling are lost if desiccation is severe (Mazzieri2011).

V. CHARACTERISTICS OF BENTONITE

The clay used in GCLs typically consists of bentonite. Bentonite is anaturally occurring clay mineral derived from chemical variation of volcanic ash (Mitchell 1993). Bentonite refers to any material that is composed mainly of montmorillonite (smectite) minerals and has properties and behavior similar to those ofmontmorillonite. Bentonites used in GCLs typically have montmorillonite contents ranging from 65 to 90% (Shackelford et al. 2000). Montmorillonite is a clay mineral characterized by a high cation exchange capacity, large specific surface area, high swelling potential and low hydraulic conductivity to water (Gleason et al. 1997). Montmorillonite isan alumino-silicate mineral with a 2:1 unit layer structure. Individual layers are about 1nm thick, but up to several orders of magnitude larger in the other directions (Grim, 1968; Mitchell,1993).

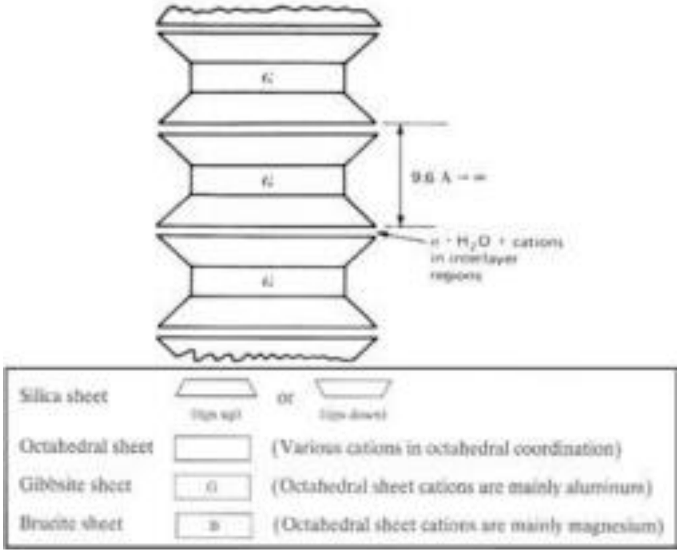

Fig. 8: Schematic of Montmorillonite Mineral (Mitchell and Soga 2005)

There are two major typesofbentonite: Sodium (Na) and Calcium (Ca). Bentonite iscategorized by the type of external cation that is adsorbed to the surface of the clay particle during bentonite processing or particle formation, thus sodium is adsorbed to sodium bentonite clay particles and calcium is adsorbed to calcium bentonite. Sodium Bentonite is used widely than Calcium Bentonite because of its superior swelling potential and its very low hydraulic conductivity (Alther 1982, 1987; Reschke and Haug 1991). Na bentonitecan swell to 8 to10 times their original volume. In contrast, Ca bentonite only swells to twice their original volume (Egloffstein 1995). Some researchers have suggested that calcium bentonitemay be more stable when exposed to chemical permeants. The properties of bentonite are greatly affected by interactions between its particles and the 13 surrounding pore fluid. The negative electrical charge on bentonite particles (Mitchell 1993) causes dissolved cations in thesurrounding pore water to be attracted to the surfaces of the clay. The layer of water and adsorbed ions that surrounds a clay particle is referred to as the electrical double layer or diffuse double layer (DDL) (Mitchell 1993; Shackelford1994).

Bentonite swells when it is hydrated with water and certain other permeants. The larger swell of Na-bentonite is reduced when higher valence cations such as Ca+2 replace Na+, effectively converting Na bentonite into lower swelling capacity and higher hydraulic conductivity bentonite such as Ca bentonite. Two typesof swelling may occur in bentonite: the crystalline phase and the osmoticphase. Crystalline swelling occurs first as water molecules move into theinterlayer region of the diffuse double layer to hydrate the mineral surface and associated adsorbed cations.

Crystalline swelling causes the interlayer to separate by several water molecules and thus causes minor swelling (Scalia and Benson 2011). Osmotic swelling occurs when water molecules flow into the interlayer region because ofa cation concentration gradient between the interlayer portion and the free pore water in the bentonite clay (Scalia and Benson 2011). Osmotic swelling produces much superior swelling because itallows additional water to flow into the interlayer portion between particles, which has implications for the hydraulic conductivity of GCLs as well as dimensional stability. Osmotic swelling only occurs when the cations in the interlayer space of the diffuse double layer are predominantly monovalent such as Na+. When multivalent cations such as Ca+2, Mg +2 and Ba+2 are predominant, only crystalline swelling can occur (Scalia and Benson 2011). The reported values of specific gravity of the bentonite used in the literature ranges from 2.74 to 2.85 (Kenney et al. 1992, Mollins et al. 1996, Gleason et al. 1998, Sivapullaiah et al. 1999, Dixon et al. 1999). The highervalue of the specific gravity of the bentonite can be attributed to the higher percentage ofFe2O3. The characteristics of the bentonite such as mineralogical composition and the exchangeable cations affect the swelling and hydraulic conductivity of the bentonite (Alther,1986).

5.1 Index Properties

Gleason et al. (1997) conducted Atterberg tests on bentonite. Inorganic salt solutions with various concentrations of NaCl and CaCl2 were used as test liquids for the Atterberg tests. The relationship between the Liquid limit of both sodium and calcium bentonite and 0.25M CaCl2 was investigated by Gleason(1997).

The 0.25 M CaCl2 solution greatly reduced the swelling capacity ofsodium and calcium bentonite. However, the sodium bentonite underwent a much larger reduction in Liquid Limit compared to the calcium bentonite. Liquid limit tests have been used byseveral investigators as surrogate compatibility tests (Bowders et al. 1986; Sridharan et al. 1986; Bowdersand Daniel 1987; Daniel et al. 1988; Acarand Olivieri1989; Edil et al. 1991; Shackelford 1994; Gleason et al. 1997; Petrov and Rowe 1997; Lin and Benson 2000; Sridharan and Prakash 2000). Results ofthese studies generally have shown that an increase in cation valence and electrolyte concentration (inorganic chemical solutions) causes the LL to decrease and the hydraulic conductivity to increase, with greatereffects occurring for soils containing higher activity clays, such as sodium bentonite. Robert J.Petrov and R. Kerry Rowe (1997) showed that the Liquid Limit of a sodium bentonite from a GCL decreased from 530 to 96 as the sodium chloride concentration of the testing solution increased from 0 (water) to 2.0 M, and the hydraulic conductivity of the GCL increased from 10–9 to 10–6 cm/s for the same increase in NaCl concentration.

5.2 Swelling and Compressibility

Several researchers indicated that the increase in the chemical solution concentration shrinks the Diffuse Double Layer (DDL),resultinginaflocculationofthe clay particles and reduces the swelling of the clays (A lawaji, 1999; Kolstad et al., 2004a; Lee et al., 2005). Di Maio (1996) investigated volume changes of bentonite exposed to NaCl or CaCl2 solutions and towater.

The results of swell index tests on sodium bentonite from a GCL conducted by Shackelford et al. (2000) andJo et al. (2001) also showed that Free swell indexof the bentonite was responsive to the cation valence and electrolyte concentration in a manner that was consistent with changes in the thickness of theadsorbedlayerof cations.It is also noticed that an increase in liquid concentration reduces the coefficient ofcompressibility of thebentonite.

5.3 Shear Strength

Sodium bentonite has the lowest shear strength when hydrated and many of the interfaces with GCLs also exhibit low strengths. Hydration of bentonite may affect the properties ofreinforced GCLs by stretching the reinforcement as the bentonite swells. The internal and interface strengths for a GCL will tend to reduce with increased swelling. Most of the studieshave demonstrated that bentonite from the GCLs may extrude into the GCL/GM interface resulting in a significant decrease in 15 interface shear strength. The bentonite extrusion from GCLs is dependent on many factors, including the type ofgeotextiles (woven and nonwoven), roughness of Geomembrane surface, type of hydration fluid, hydration condition of GCL, stress level applied, sequence of hydration and loading, loading rate, and shearing rate. Hewitt etal. (1997) and Koerner et al. (1998, 1998) found that extrusion occurred when the bentonite was hydrated and mobile. Bouazza (2002) and Triplett & Fox (2001) reported that bentonite was easilyextruded through a woven geotextiles/textured GM interface. The extrusion was less for nonwoven geotextiles interfaces and smooth GM interfaces.

Stark (1997) found that the amount of bentonite extrusion was related to the swell pressure of the bentonite in the GCL. Mark H. Gleason, David E. Daniel andGerald R. Eykholt(1997) performed consolidated drained direct shear tests on thin layers ofbentonite as per ASTM D3080. The samples were consolidated to the desired normal stress, the time and displacement to failure were estimated, and the samples were sheared at the calculated rate. The times to failure varied from days to weeks. Further details are provided by Gleason(1993).

The Mohr-Coulomb failure envelopes forthe consolidated-drained direct shear tests are shown in Fig. 9. For the range of normal stress (35-150 kPa) employed in this testing program, the shear strength of the calcium bentonite is approximately twice the shear strength of the Sodiumbentonite.

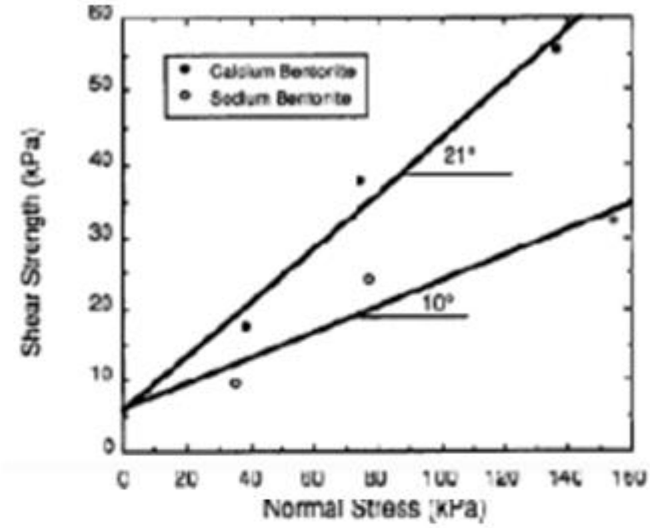

Fig.9:Results of Consolidated-Drained Direct Shear Tests on Bentonite (Gleason 1997) Ana Vukelic(2007) concluded that for tests carried out on prehydrated samples at lower shear rates, lower contact shear strengths were obtained and more extensive bentonite extrusion to the 16 contact area was observed. High-quality test data are available in the open literature for GCL internal shear strengths and geomembrane/GCL interface shear

strengths. Much less information is available on shear strength behavior forother common GCL interfaces (e.g. soil/GCL, drainage geocomposite/GCL), especially at high normal stress conditions (Chiu and Fox2004)

5.4 Hydraulic Conductivity

Lutz and Kemper (1958) investigated how salt solution concentration affected the hydraulic conductivity of sodium and calcium bentonites from Utah. The bentonites were permeated with Distilled water (DI), NaCl solutions (Na bentonite only), or CaCl2 solutions (Ca bentonite only). When permeated with 0.005 N solutions, the Na bentonite and Ca bentonite were approximately two times more permeable than to DI water. For 0.5 N solutions, the Na bentonite was 11 times more permeable than to DI water and the Ca bentonite was four times more permeable than to DI water. McNeal and Coleman (1966) investigated how the hydraulic conductivity of six soils with montmorillonite content ranging from 25 to 48% varied as the concentration of NaCl and CaCl2 in the permeant liquid increased. At low concentrations (<0.02 N), the hydraulic conductivity to NaCl was typically 20 times lower than to CaCl2. When the concentration exceeded 0.6 N, the soils had similar hydraulic conductivity regardless of whether they were permeated with a NaCl solution or a CaCl2 solution.

Alther et al. (1985) studied how valence and concentration of inorganic salt solutions affect the hydraulic conductivity of bentonite. Filter press tests were conducted on contaminant-resistant (polymerized) bentonite and untreated bentonite using 16 inorganic aqueous solutions. The hydraulic conductivity of the bentonite increased with increasing electrolyte concentration.

Gleason et al. (1997) investigated the properties of Na-bentonite and Ca-bentonite by conducting hydraulic conductivity tests on a thin bentonite layer simulating a GCL. The bentonite was permeated with tap water and 0.25 M CaCl2 solution at an effective stress of 35 kPa. Nabentonite permeated with 0.25 M CaCl2 solution had higher hydraulic conductivity (9x 10-7 cm/s) compared to Na-bentonite permeated with tap water (6x10-10 cm/s).

Petrov et al. (1997) investigated how increasing concentrations of NaCl solutions affected the hydraulic conductivity of a needle-punched GCL containing Sodium Bentonite. All tests were conducted in a fixed-ring permeameter with a vertical stress of 35 kPa. An increase in NaCl concentration resulted in an increase in the hydraulic conductivity, which is consistent with a reduction in the volume of bound water.

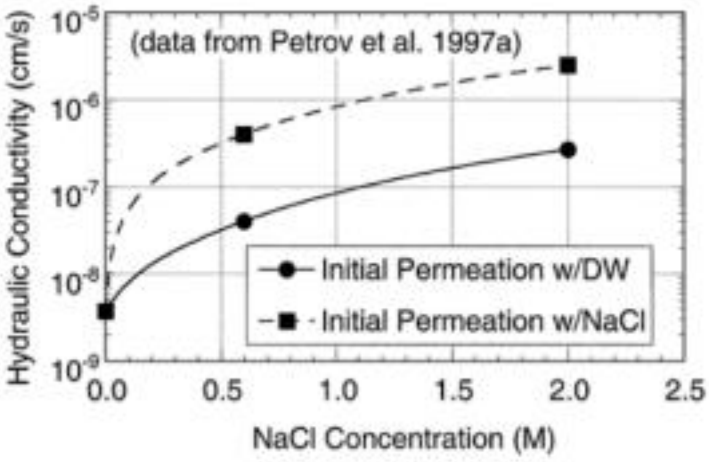

Fig. 10:Hydraulic conductivity of a Sodium Bentonite versus NaCl concentration for specimens initially permeated with distilled water (DW) and specimens permeated directly with NaCl solutions (Petrov 1997)

VI. CHARACTERISTICS OF SAND

Sand is a pervious material in nature. Hydraulic conductivity of sand ranges from 1 to 1×10 - 5 cm/s (Freeze and Cherry, 1979). The hydraulic conductivity of sand can be reduced if sand is mixed with a very impervious material such as bentonite. The table below gives a detailed description of different sands used by several researchers.

Table 1:Detailed description of different sands used

Researchers	Properties of Sand Used
1) Chalermyanont and Arrykul (2005)	Songkhla local sand The corresponding coefficient of uniformity (Cu) and coefficient of curvature (Cc) are 11.7 and 1.0, respectively. The sand is classified as SW-SM according to the Unified Soil Classification System (USCS)
2) Tay and Stewart (2000)	Sherburn Yellow Building Sand (Quartz Sand) Effective size (D10) = 212µm Specific Gravity = 2.68 Coefficient of uniformity, (Cu) = 2 Max. and Min. Void Ratio = 0.767 and 0.375
3) Mollins et al. (1995)	Knapton Quarry sand is a slightly silty fine angular quartz sand (containing some iron oxides and silicate minerals). Percentage fines = 7% Effective size (D10) = 0.07 mm Specific gravity = 2.67

6.1 Sand Bentonite Mixtures

Chapuis (1990) presented numerous results of laboratory permeability tests for soil-bentonite mixtures used as hydraulic containment liners. Sand-bentonite mixtures are comprised of two different soils with regard to grain size, chemical activity, permeability and strength, which when combined in optimum proportion, can form an outstanding seepage barrier that possess a low hydraulic conductivity and is dimensionally stable.

Sand-Bentonite mixtures are widely used as hydraulic barriers to prevent the movement of leachate from waste disposal facilities because sand-bentonite can combine relatively high strength and low compressibility with very low hydraulic conductivity. This is achieved by using a mixture that contains sufficient sand to ensure the stability of the mixture and enough bentonite to seal the voids between the sand particles. The mixture containing modest amounts of bentonite is fairly resistant to the effects of desiccation (Tay et al. 2001), and the bentonite in mixture has a high chemical buffering capacity (Yong, 1999). As the sand component in sand-bentonite mixtures decreases the shrinkage on drying, mixtures can meet the hydraulic conductivity criteria without suffering from shrinkage cracking (Dixon et al., 1985).

The hydraulic conductivity of a mixture is dependent on the fabric of bentonite in the sand-bentonite mixture which in turn is governed by conditions at the time of mixing of sand and bentonite. In mixtures containing up to 20% bentonite in dry weight, sand forms the loadsupporting framework and provides dimensional stability at the macro level in the sand bentonite mixtures. The table below gives a detailed description of work carried out by researchers on sand-bentonite mixtures.

Table 2:Detailed description of work on sand-bentonite mixtures

Researchers	Materials Used	Properties of material	Properties studied
1.Mollins and Cousens (1996)	Fine silica sand Na-bentonite powder Sand-Bentonite mixtures- 5, 10 & 20% Bentonite by dry weight.	Silica Sand - SP according to USCS. Percentage fines = 7% Effective size , D10 = 0.07 mm Specific gravity = 2.67 Moisture content ~4.5%	1.Swelling behavior and hydraulic conductivity of Sodium-bentonite powder and mixtures. 2. The study concluded that hydraulic conductivity of a Sand-
		Na-montmorillonite Bentonite Powder Liquid limit = 407% Plastic limit = 48% Specific gravity = 2.76 Moisture content ~ 13.5%	Bentonite mixture is dependent on the bentonite content.
2.Agus SetyoMuntohar (2004)	Sodium Bentonite Powder, Kaolin and Fine Sand. The percentage of Sodium Bentonite by weight is varied between 5% and 100%.	Sand: Effective Size, D50 = 0.82 Percentage Sand = 71.5% Percentage Silt = 4.0% Percentage Clay = 0.0% Bentonite: Liquid Limit = 307.3% Plastic Limit = 45.4% Activity = 3.6 Kaolin: Liquid Limit = 72.3% Plastic Limit = 39.8%	1. Study of swelling and compressibility behavior of mixtures. 2. Study of deformation behavior of Bentonite mixed with different amount and types of non swelling soil such as (kaolin) and finer fractions (silt).
3.Arrykul and Chalermyanont (2006)	Local Songkhla Sand. Powdered Sodium Bentonite	Bentonite: Liquid limit = 487% Plasticity index = 450% Specific Gravity = 2.55	1. Compaction tests were carried out to find out the optimum water contents and maximum dry unit weight of sand-bentonite mixtures. 2. Based on the standard test results, a suitable sand-bentonite mixture that yields low hydraulic conductivity maintaining relatively high shear strength was suggested as liner in hydraulic containment applications.

4.Karunaratne et al. (2001)	Sodium bentonite and Kaolinite	Sodium Bentonite: Liquid limit = 465% Plasticity index = 424% Specific Gravity = 2.77 Kaolinite: Liquid limit = 74% Plasticity index = 40% Specific Gravity = 2.64	1. Study the geotechnical properties of bentonite: kaolinite mixtures when subjected to a variety of permeants.

VII. INDEX PROPERTIES

The consistency limits are useful indicatorsof clay behavior (Jefferson and Rogers, 1998). It should be pointed out that there has not been a general consensus regarding the effect of chemicals on the consistency limits of clays. Bowdersand Daniel (1987) advocated that many chemicals tend to reduce the thickness of the DDL, causing the soil skeleton to shrink and decrease in repulsive forces, thus promoting flocculation of clay particles, and to dehydrate interlayer zones of expandable clays, resulting in a reduction in liquidlimit.

G.P. Karunaratne, S.H. Chew, S.L. Lee, and A.N. Sinha (2001) carried out swell index tests and liquid limit tests with different permeants for bentonite, B:K = 50:50, and kaolinite. Fig.11 shows the variation of the liquid limit with bentonite percentage.

Fig.12 shows the variation of the swell index with bentonite percentage. The 0.25M calcium chloride permeant reduces the swell index significantly compared to distilledwater.

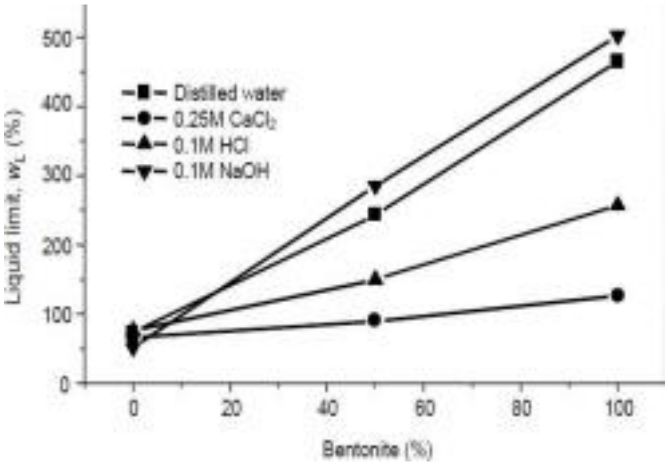

Fig. 11:Variation of the liquid limit, wL , with percentage of bentonite content (Lee 2001)

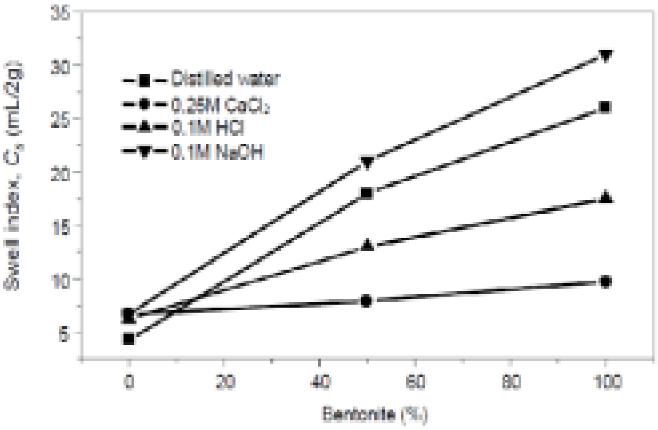

Fig. 12:Variation of the swell index with percentage of bentonite content (Chew 2001) A swelling test was developed to investigate the behavior of sodium bentonite powder and sand bentonite mixtures (5, 10 and 20% bentonite by dry weight) by Mollins, Stewart and Cousens. Dry samples of sodium bentonite and mixtures were placed in an oedometer ring, subjected to a vertical stress while the sample height was monitored until swelling ceased. Tests were performed at various vertical effective stresses, ranging from 1 to 450kPa.

Fig. 13 shows a linear relationship between the final void ratio of the clay, ec, and the logarithm of vertical effective stress, σv'. By maintaining homogeneity in compaction by tamping, vibration and static loading, it is found that the final void ratio is independent to sample preparation.

Fig. 14 shows plot of sand-bentonite mixtures with their final clay void ratio against the logarithm of vertical effective stress. At low effective stress levels the mixtures behave as clay and the externally applied stress is supported by the clay soil. When the externally applied stress exceeds a threshold value, the difference between the externally applied stress and the threshold value is supported largely by the sand, whose particles must now be in contact. The threshold value which causes a change in the mixture's behavior is found to be a function of the clay content. Graham et al. (1986) proposed a similar explanation of the mechanical behavior of sand-bentonitemixtures.

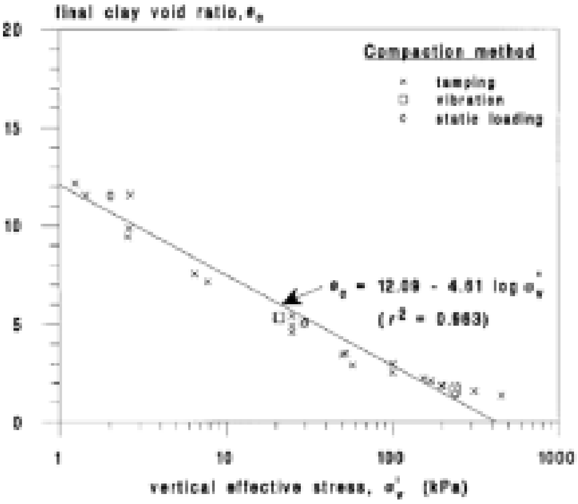

Fig. 13:Swelling of Bentonite powder with distilled water

Fig. 14:Final clay void ratioof sand-bentonite mixtures after swelling with distilled water The swelling mechanism of compacted bentonite clay can be illustrated by the model asgiven by

AgusSetyoMuntohar, 2004 in Fig. 15. The volume of swelling clay particles such as montmorillonite increaseby absorbing water into the interlayer portion of montmorillonite, andthe void in the compacted bentonite clay are filled by this volume. After water uptake, the compacted bentonite clay can swell at a constant vertical pressure as in the swelling deformation test. The volume of compacted clay increases as the volume of swelling clay particles increases until the swelling pressure of the clay particles equals the verticalpressure.

Fig. 15:The model of the swelling deformation of compacted bentonite clay 8. CompactionCharacteristics
Compaction tests were carried out by Chalermyanont and Arrykul (2006) to assess the optimum water contents and maximum dry unit weights of sand-bentonite mixtures which were determined using Standard Proctor compaction method (ASTM D698). The bentonite contents used varied between 0%, 3%, 5%, 7%, and 9% by dryweight.

The maximum dry unit weights and corresponding optimum water contents were obtained from the peak of compaction curves as shown in Fig. 16 As bentonite content increased, optimum water content (OMC) increased and maximum dry unit weight (MDD) decreased. As shown in Fig. 16, when the bentonite content varies from 0 to 9%, the maximum dry unit weight decreases from 19.47 to 18.56 kN/m3and the corresponding optimum water content of the compacted sand-bentonite mixtures increases from 9 to 12%. When bentonite is mixed with sand, more water is required in compaction in order to reach maximum dry unit weight (Holtz and Kovac1981).

Shariatmadari (2011) concluded that inorganic salt solution increases the maximum dry density and decrease the optimum water content of mixtures. Higher cation valance leads to higher increase in the maximum dry density and higher decrease in optimum water content. The decrease of the diffuse double layer's thickness is the source of thistrend.

Fig. 16:Compaction curves of sand-bentonite mixtures.

8.1 One-DimensionalConsolidation

One-dimensional consolidation tests were carried out by Karunaratne et al. (2001) on clay mixtures at the liquid limit with distilled water to determine the change of consolidation properties with the addition of increased amounts of bentonite. Specimens were consolidated in stainless steel circular containers (70 mm in diameter and 19 mm high). The loading procedure comprised the following stages: (i) a seating pressure of 12.5 kPa was applied; (ii) stage loading up to 100 kPa with a load increment ratio of one was applied; (iii) unloaded to 25 kPa; and (iv) reloaded up to 800 kPa with a load increment ratio of one. As the depth of most landfills are in the range of 30 to 80 m, 800 kPa is a reasonably high enough stress for the examination of consolidationproperties.

Fig. 17:The compression index and swell index, Cs , with percentage of bentonite content.

Fig. 18:Variation of the coefficient of consolidation, Cv, with pressure for Bentonite-kaolinite (B:K) mixtures using distilled water as the permeant.

Fig. 17 shows the increasing trend of Cc and Cs with increasing bentonite percentages. Fig. 18 shows the variation of Cv with pressure obtained from oedometers for different B:K ratios using distilled water as the permeant. The coefficient of consolidation, Cv increases with the increasing pressure for pure kaolinite and decreases for pure bentonite. For B:K = 50:50, the swell index, Cs , and compression index, Cc , were approximately 50% of those for pure bentonite. The results shows that the 50:50 B:Kmixture results in coefficient of consolidation, Cvand hydraulic conductivity, k, values approximately the same as that of pure bentonite. Therefore, B:K= 50:50 is the thresh-old mixture ratio. G.P. Karunaratne et al. (2001) concluded that, for the 50:50 B:Kmixture all the hydraulic conductivity values were within the range of limits. This demonstrates that a liner packed with a 50:50 bentonite:kaolinite mixture could be a cost- effective lineroption.

VIII. HYDRAULIC CONDUCTIVITY

Factors affecting the hydraulic conductivity of sand-bentonite mixture

1) Percentage of Bentonite

The amount of bentonite in a sand-bentonite mixture ranges from 5 to 20% of the dry weight of Greater amounts of bentonite tend to form around the sand grains, and the mixture becomes plastic and consequently difficult to compact. Further, decrease in hydraulic conductivity becomes marginal, and will not be cost-effective liner option (Chapius, 1990).

Fig. 19:Hydraulic conductivity (m/s) of a sand-bentonite mixture as a function of percentage of bentonite versus dry density (Goran Sallfors 2002)

Fig. 20:Hydraulic conductivity and swell versus bentonite content of sand-bentonite mixtures. Relationship between hydraulic conductivity of sand-bentonite mixtures and percent bentonite content is shown in Fig.19andFig. 20 The hydraulic conductivity of sand-bentonite mixtures decreases with increasing bentonite content. For bentonite contents of 3 and 5%, the hydraulic conductivity of the mixtures decreases significantly to 5.13×10-8and5.15×10-9 cm/s, respectively.

2) Mixture andCompaction

A thorough mixture of the bentonite and the sand is important for reducing the scatter in the permeability. The most effective compaction is reached when the sand-bentonite mixture has a water content close to optimum or just above (about 2%) (Haug and Wong, 1992).

3) Type ofBentonite

Depending on the pore fluid, different types of bentonite behave differently, and it is therefore important to consider the chemical state of both the bentonite and the fluid when, determining the components of the mixture (Kenney et al., 1992).

4) Time ofPermeation

Time plays a very important role when it comes to the resulting hydraulic conductivity of a sand- bentonite mixture. The sand and bentonite are often mixed dry or at least in an unsaturated phase and then compacted at an optimum water content, resulting in a sample that is not fully saturated. The bentonite swells during uptake of water and, consequently, it will take a long time before the degree of saturation in the sample is homogeneous and low hydraulic conductivity is obtained due, to the low hydraulic conductivity of thebentonite.

Fig. 21:Ratio of inflow to outflow and hydraulic conductivity vs. time of sand with 3% bentonite mixture.

Hydraulic conductivity test results for mixture with 3% bentonite content as a function oftime are demonstrated in Fig. 22. Within the first two weeks, the hydraulic conductivity and rate ofinflow to rate of outflow ratio were fluctuated. After two weeks, these two values were reasonably constant which indicated that the constant flow was reached and reliable hydraulic conductivity had been achieved. For higher bentonite contents, the time to reach constant flow was longer. For a mixture with 9% bentonite, time required to reach constant flow was about 12 weeks

9.1 Measurement of Hydraulic Conductivity

Hydraulic conductivity tests have been performed at vertical effective stresses up to 450 kPa on Na bentonite powder and compacted sand-bentonite mixtures (5, 10 and 20% bentonite by weight) by Mollins and Cousens (1996). Three methods that were used to determine the hydraulic conductivity are asfollow:

Rowe cell constant head tests: Bentonite specimens were tamped into the Rowe cell as dry powder, whereas mixtures (with one exception) were compacted into thecell at optimum moisture content. All samples were confined by a vertical stress, vacuum de-aired, and allowed access to water. A back pressure of 300 kPa was used in all the Rowe cell tests to improve saturation. Two alternative methods were also used for measuring hydraulic conductivity. Standard compaction permeameter falling head tests and an indirect method where the hydraulic conductivity was calculated from consolidation data (Terzaghi, 1943), using the logarithmic method (Casagrande &Fadum,1940).

Falling head compaction permeameter was suitable for measuring the hydraulic conductivity of the sand and 5% bentonite mixture, whereas the method based on consolidation rate required significant, time dependent volume change and was, appropriate for specimens with higher claycontent.

Fig. 23 shows the hydraulic conductivity data measured bythree different methods on such a plot. The data for 100, 20 and 10% bentonite mixtures fall on a single straight line indicating that these mixtures are behaving as the same soil type (the pore-size term in the Kozeny-Carmen equation is the same). The Kozeny-Carmen equation (Bear, 1972) indicates that the relationship tends to linearity at large void ratios. Terzaghi & Peck (1967) suggest a linear relationship between the logarithm of hydraulic conductivity and the logarithm of void ratio for a particular soiltype.

Fig. 23:Hydraulic conductivity of sand-bentonite mixtures.

Chalermyanont and Arrykul (2006) performed hydraulic conductivity tests on compacted sand bentonite mixtures in rigid wall permeameter as shown schematically in Fig. 24. A rigid wall permeameter is a compaction-mold used in compaction test, made of stainless steel tube and mounted on top and bottom with stainless steel plates. A collar was mounted between the top plate and the permeameter in order to obtain 1- D flow through the specimen. The specimens were allowed to swell vertically at the upper end into the collar that was filled with distilled water which was boiled prior to use distilled water was permeated through the specimens from the top of permeameter via a Teflon tube that connected the top plate to the burette. The hydraulic gradient used was 10 to 15. The graduated cylinders were used to collect the effluents at the bottom of thespecimens.

Fig. 24:Rigid wall permeameter

G.P. Karunaratne et al (2001) measured the hydraulic conductivity after each consolidation stage using a falling head permeameter attached to the oedometer cells. A consolidation pressure of 35 kPa was used to facilitate comparison the studies of Ruhl and Daniel (1997), Gleason et al. (1997),andLentzetal.(1984),whoconsidered35kPaasasuitablevalue.

The measured hydraulic conductivity, k, versus pressure for different B:K ratios is shown in Fig. 25 The variation of k for B:K = 50:50 is nearly identical to that of pure bentonite.

Fig. 25:Variation of the measured hydraulic conductivity, k, with pressure for different bentonite:kaolinite (B:K) mixtures using distilled water as the permeant.

Fig. 26:Variation of hydraulic conductivity, k, using the falling head method (distilled water, 0.25M $CaCl_2$, 0.1M HCl, and 0.1M NaOH; consolidation pressure = 35 kPa).

The hydraulic conductivity values for kaolinite and B:K= 50:50 permeated with distilled water, an acid, and an alkaline environment are shown in Fig. 26. The calcium chloride permeant increased the hydraulic conductivity approximately 4.5 times for pure kaolinite, 16 times for Bentonite:Kaolinte = 50:50, and 34 times for pure bentonite. The hydraulic conductivity values measured using Rowe cells under constant head conditions and oedometers under falling head conditions are approximately comparable for Bentonite:Kaolinite = 50:50 andusingdistilled water as the permeant. Hence, for other permeants, which are likely to corrode the metal lining, the falling head method was used at the end of each consolidation pressure increment.

9.2 Hydraulic conductivity of Geosynthetic Clay Liners

Hydraulic conductivity to the actual permeant liquid is usually assessed by a "compatibility test" where the GCL is permeated with the liquid to be contained or a liquid simulating the anticipated liquid. There have been numerous studies that looked into the effect of pore liquid on the engineering properties of GCLs. Most of these investigations focused on the variation of hydraulic conductivity. For example, Schubert (1987), Shan and Daniel (1991), Daniel et al. (1993), and Ruhl and Daniel (1997) have permeated GCLs with various chemical solutions including real or syntheticleachate.

Fig. 27:Results for Hydraulic Conductivity of GCL versus Effective Confining Stress ResultsforHydraulicConductivityofGCLversusEffectiveConfiningStressareshownbyR S Thiel and K Criley in Fig. 27. The hydraulic conductivity of GCL is function of effective confining stress. As the confining pressure increased, the hydraulic conductivity of GCL increased.

The results of the tests conducted by Petrov and Rowe (1997) showed that for GCL specimensprehydrated with water, the hydraulic conductivity of the GCL increased by approximately1.5to2ordersofmagnitudeastheNaClconcentrationincreasedfrom0.01Mto2.0 M, with the increase in hydraulic conductivity being slightly greater for GCLs with higher void ratios. In contrast, for the GCL specimens prehydrated with NaCl solution, the hydraulic conductivity of the GCL increased approximately 2.5 to 3 orders of magnitude as the NaCl concentration increased from 0.1 M to 2.0M. Shackelford et al. (2000) discussed the factors and testing considerations affecting the hydraulic conductivity of GCLs permeated with inorganic liquids. Their test results showed thatinorganic liquids containing both high concentrations of monovalent cations and low concentrationsof divalent cations can cause significant increases in hydraulic conductivity provided the test is performed satisfactorily long to allow for exchange of adsorbed cations. Shan and Daniel (1991) and Daniel et al. (1993) found that the hydraulic conductivityof water- hydrated GCLs did not show a severe increase when permeated with a range of chemical solutions and organicliquids.

Fig. 28:Effect of Needle-Punching Fibers on Hydraulic Conductivity of GCLs Permeated with $CaCl_2$ Solutions

Hydraulic conductivities of the thin specimens of granular bentonite and GCL specimens obtained by Ho Young Jo (2001) are shown in Fig. 28. The thin specimens of bentonite had essentially the same hydraulic conductivity as the GCL specimens when permeated with $CaCl_2$ solutions. Thus, preferential flow along the needle-punching fibers was not responsible for the high hydraulic conductivity of the GCLs observed at higher concentrations (>0.1M).

Ruhl and Daniel (1997) concluded from their experiments that the most important factor that governs 34 GCLs' ability to slow down permeation of chemical solutions is the hydrating liquid. When a GCL specimen had been hydrated with water prior to permeation of other liquids, the hydraulic conductivity of the GCL remained quite low (in the range of 10-11- 10-10 m/s). In addition, permeation of GCLs with real leachate resulted in lower hydraulic conductivity (10-12- 10-10m/s).

IX. DIFFERENTIAL SETTLEMENT

MSW Landfills undergo settlement during and after their active life. This settlementis caused by the gradual decomposition and consolidation of the waste mass. Differential settlement maybecharacterizedbythedistortionΔ/L,whichisdefinedasthesettlementΔ,overthe horizontal distance L. The average tensile strain caused by distortion can be computed. If tensile strains are large enough, the barrier layer may crack and lose its hydraulic conductivity. For this reason, it is essential that the landfill cover system be capable of sustaining the anticipated differential settlement of the waste mass.

Fig. 29:Definition of Distortion, Δ/L

Murphy and Gilbert (1987) concluded that settlement over short distances in landfill cover systems is more threatening to the performance of barrier than relatively uniform settlement over longer distances. The average tensile strain caused by distortion at failure increases with an increasing plasticity index and increasing clay content (Lozano and Aughenbaugh 1995). The tensile strain at failure is between 0.1 to 4% (Koerner1994).

Stone and Jessberger (1991) conducted centrifuge test to look into the responseof compacted kaolin and sand bentonite mixture to different settlement. Angular Distortionof up to 16 ° (Δ/L =0.287) were generated, the barriers were permeated as distortion was induced. Thecompacted Kaolin barrier without any surcharge failed at an angular distortion of 6°(Δ/L=0.105). For sand bentonite mixture without surcharge, slight surface cracking occurred at angular distortion of 7.5°.

Mark D.Lagatta and David E. Daniel investigated to quantify relationship between differential settlement and hydraulic conductivity of GCLs by performing test in steel tank and a water filled bladder was placed beneath the GCL to produce differential settlement. They concluded that GCLs are between CCLs and Geomembranes in terms of ability towithstand tensile strains associated with differential settlement. Based on current knowledge, the general range of tensile strains that various can withstand appears to be asfollows:

i. Compacted Clay Liners : 0.1-4%
ii. Geosynthetic Clay Liners : 1-10%
iii. Geomembranes : 20-100%

10.1 Bentonite Migration inGCLs

Results of field observations and laboratory tests show that the thickness of bentonite in GCLs can decrease under a normal stress, especially around the zone of stress concentration, such as sump or wrinkles in an overlying geomembrane. In a liner system, this decrease in GCL thickness will result in an increase in hydraulic conductivity, regulatory non-compliance and a weak leachate attenuation capacity and decrease in containmenttime.

In a cover system, reduced thicknesses of bentonite cause an increase in infiltration or gas migration through the GCL. Another means for stress concentration development is local differential settlement caused by natural variations in foundation compressibility and shear strength. Suggestion to protect hydrated bentonite from stress concentrations are presented by (T M Stark, 1998), modifying the existing GCLs to include an internalstructure.

Migration of bentonite particles may be due to two mechanisms (J.P. Giroud and K.L. Soderman, 2000) · When bentonite is hydrated, it has very low shear strength and it is conceivable that the overburden stress (i.e. load applied on the GCL due to the weight of the overlying layers) may cause a certain amount of hydrated bentonite to be extruded through geotextile openings. Evidence of this mechanism is provided by Fox et al.(1998).

· If there is a hole in the geomembrane overlying the GCL, the resulting flow of leachate through the hole in the geomembrane, then through the GCL and toward the Geonet, may dislocate some bentonite particles from the GCL and may carry some of these particles through geotextileopenings.

10.2 Comparison of Hydraulic Conductivity Behavior of GCL andCCL

The most commonly used landfill barriers are Compacted Clay Liners (CCL) and Geosynthetic Clay Liners (GCL) offering individual advantages and disadvantages. When used under the right circumstances all systems can achieve good results, however all sealing systems have certainlimitations.

Fig. 30:Hydraulic Conductivity of CCL versus Dielectric Constant (Fernandez & Quigley, 1985)

Fig. 30 indicates that the dielectric constant ofpermeants can have a important effect on hydraulic conductivity caused by the changes in the double layer thickness. Increases in hydraulic conductivity have been observed depending on the type of chemical solutions. Comparable comments were made by a number of investigators (Mesri& Olson, 1971; Fernandez & Quigley,1986).

Fig. 31. Plot of dielectric constant versus coefficient of Permeability ofGCL

Fig. 31 shows a very strong relationship between the coefficient of permeability and dielectric constant of the fluid. It is observed that the permeabilityof GCLtolow/non-polar fluidsisveryhighcomparedwiththepermeabilitytowaterorformamide,ahigh-polarityfluid.

The performance of a GCL, should be either equivalent to or exceed that of a CCL. The considerations of solute flux and breakthrough time, compatibility, and attenuation capacity favourCCLs. The hydraulic conductivities of GCLs and CCLs vary depending on the compressive stress. The measured hydraulic conductivities of hydrated GCLs in laboratory tests designed to simulate the compressive stress conditions in a landfill are typically in the range of 5 × 10-12to5 × 10-11 m/s. These hydraulic conductivities are roughly 10 to 100 times lower than the hydraulic conductivities of CCLs which are typically between 1 × 10-10and 1 × 10-9m/s.

The hydraulic conductivity values given above are for the case when the liquid permeating the CCL or the GCL is water or a low-concentration leachate that does not affect the hydraulic conductivity of the CCL and the GCL. Examples of such leachates are given by Ruhl and Daniel (1997). GCLs are more capable to withstand freeze-thaw and wet-dry cycles, are not vulnerable to desiccation during construction, and can offer equivalent or lower rates of release of fluids and chemicals (Daniel,1994).

CCLs are more effective diffusion barriers than GCLs because diffusion through a CCL is only slightly faster than through a hydrated GCL and the thickness of a CCL is typically on the order of 100 times the thickness of a GCL. The attenuation in CCLs is more effective than inGCLs because the larger thickness of CCLs more than compensates for the greater absorption capability of bentonite in GCLs (Daniel1996).

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