Journal of Rock Mechanics and Geotechnical Engineering 8 (2016) 541–550

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full length article

Hydraulic properties of dune sand–bentonite mixtures of insulation barriers for hazardous waste facilities

M.K. Gueddouda [a,*], I. Goual [a], B. Benabed [a], S. Taibi [b], N. Aboubekr [c]

[a] *Laboratoire de Recherche de Génie Civil, Université Amar Teledji, BP 37 G, Laghouat 3000, Algeria*
[b] *Laboratoire Ondes et Milieux Complexes, Université du Havre, 53 Rue de Prony, Le Havre 76000, France*
[c] *Laboratoire Eau et Ouvrages dans Leur Environnement, Université Abou Bekr Belakid, BP 230, Tlemcen 13000, Algeria*

ARTICLE INFO

Article history:
Received 6 October 2015
Received in revised form
15 February 2016
Accepted 5 April 2016
Available online 26 May 2016

Keywords:
Dune sand
Bentonite
Valorization
Landfill liners
Hydraulic conductivity

ABSTRACT

This paper presents a study on the valorization of local materials such as desert dune sand obtained from Laghouat region in the South Algeria and mine bentonite intended for the realization of liner base layers in the conception of insulation barriers for hazardous waste facilities. In practice, an economical mixture satisfying the hydraulic requirements is generally concerned. First, in order to get an adequate dune sand–bentonite mixture compacted to the optimum Proctor condition, an investigation on saturated hydraulic behavior is carried out in this study for different mixtures. Using oedometer test (indirect measurement), the adequate mixture of 85% dune sand and 15% bentonite satisfies the conditions of saturated hydraulic conductivity ($k < 10^{-9}$ m/s). This result of the adequate mixture is also confirmed by direct measurement of saturated hydraulic conductivity using triaxial cell. Second, the unsaturated hydraulic conductivity of the adequate mixture is measured with an original vapor equilibrium technique (VET) used for $S_r < 30\%$ (very high suction $s > 3$ MPa). This technique is conducted based on the exploitation of the water retention curve in order to establish the relationships between hydraulic conductivity, degree of saturation, and suction. It shows that the hydraulic conductivity increases with the degree of saturation and decreases with the suction. However, the hydraulic conductivity has a constant value for suctions larger than 20 MPa. The selected dune sand–bentonite mixture satisfies the regulation requirements and hence constitutes a good local and economical material for the conception of barrier base liners.

1. Introduction

Rapid technological advances and population needs lead to the generation of increasing quantities of hazardous wastes, for which two fundamental issues will be encountered, e.g., waste management and pollution risk control. One of the solutions for handling these contamination problems is enclosing the wastes in a specific location using insulation barriers. Many different barrier materials exist, for example, plastic membranes, sand–bentonite compacted layers, cement stabilized soils (Santucci de Magistris et al., 1998). The permeability of insulation barriers has been studied by many authors for different types of soils, compacted clays, silty soils, clay and sand–bentonite mixtures (Daniel, 1984; Holtz, 1985; Chapuis, 1990; Blatz et al., 2002; Montañez, 2002; Watabe et al., 2003; Chalermyanont and Arrykul, 2005; Cui et al., 2008). The efficiency of these insulated barriers depends largely on their hydraulic behaviors along with their abilities of contaminant retention (Booker et al., 2004; Sangam and Rowe, 2005; McWatters and Rowe, 2010; Scalia et al., 2013).

Compacted sandy soil with small additions of bentonite (bentonite–sand mixture) has been proposed and used as an adequate material for these insulation layers. In order to be efficient for insulation, the insulation barriers should fulfill some specifications:

(1) Permeability at saturated state ranges between 10^{-8} m/s and 10^{-10} m/s (Chapuis, 1990; Parker et al., 1993; Marcoen et al., 2001; Pierson et al., 2004; Souli et al., 2008).
(2) Physical stability of the material in contact with water (Thériault, 2000).

---

* Corresponding author. Tel.: +213 78 02 31 68.
E-mail address: gueddouda_mk@yahoo.fr (M.K. Gueddouda).
Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

http://dx.doi.org/10.1016/j.jrmge.2016.02.003
1674-7755

(3) A swelling potential that ensures good contact with the host rock and permits the replenishment of existing cracks or that will develop in the future (Yilmaz and Marschalko, 2014).
(4) The sand should possess some characteristics of grain size distribution (GSD), in order to prevent bentonite leaching from the skeleton, hence ensuring the hydraulic stability of the mixture (Malusis and Di Emidio, 2014).

In current engineering applications, soil–bentonite mixtures are mainly used to build impervious cores of earth dams and contain groundwater pollution. Many authors reported different percentages of bentonite additions, and they have found the percentages of 5–8% for sodium bentonite and 9–15% for calcium bentonite (Chapuis, 1990; Egloffstein, 2001; Graham et al., 2001; Montañez, 2002).

Algeria generates (12–14) × 10^6 tons of household wastes each year, where 3000 illegal dumping sites are observed. These landfills occupy a total area of about 150,000 ha (MPET, 2013). In the South Algeria, the rapid development of urban areas and the growth of oil industry begin to generate enormous quantities of hazardous wastes. In order to avoid groundwater pollution and environment degradation, an insulation barrier using dune sand is proposed for construction of waste disposal sites. Dune sand is an available and economical local material, which can be enhanced by a small addition of bentonite. Generally, the free swelling ratio between sodium bentonite and calcium bentonite is about 3, and hence results in a low hydraulic conductivity of the mixtures (Ait Saadi, 2003). The ratio of hydraulic conductivity between sodium bentonite and calcium bentonite is about 10^{-2} (Pierson et al., 2004; Rowe et al., 2012; Du et al., 2015). However, the sodium bentonite is less available through the world; instead, the calcium bentonite is used in the study, which is brought from Maghnia region in the west of Algeria. The study presented herein describes laboratory-testing techniques and analyzes the experimental results of dune sand-bentonite mixtures, suggesting an adequate combination in order to satisfy the regulation criteria.

An investigation study is carried out on several mixtures with different percentages of bentonite additions, which vary from 3% to 15%, for an adequate mixture purpose. The adequate amount of bentonite addition, which satisfies the required conditions on the saturated hydraulic conductivity, is obtained using oedometer test. After then, the saturated hydraulic conductivity under different confining stresses is investigated with direct methods using triaxial cell. The measurement of unsaturated hydraulic conductivity is conducted using vapor equilibrium technique (VET). The VET couples the total soil suction control in desiccators with saturated salt solutions, and water mass is measured from a digital balance laboratory (Fredlund and Rahardjo, 1993; Taibi et al., 2009). This paper presents an investigation in order to valorize local materials based on desert dune sand and bentonite for the design of insulation barriers intended to the realization of the hazardous waste facilities.

2. Materials and methods

2.1. Materials

The bentonite used in this study is extracted from Maghnia mine (Hammam Boughrara, 600 km west of the capital Algiers). The GSD curve of the bentonite is shown in Fig. 1. It is very fine clay; more than 60% of particles have a diameter less than 2 μm. Liquid limit (w_L) and plasticity index (PI) of the bentonite are 141% and 93%, respectively. The PI of bentonite indicates that the bentonite is a very highly plastic material and has high swelling potential, which is characterized by a large specific surface area (S_s = 462 m^2/g). According to the Skempton classification based on the activity (Skempton, 1953), the bentonite from Maghnia presents a high percentage of calcite montmorillonite (Ca^{2+}). X-ray diffraction is one of the widely used methods for identification of clay minerals and studying their crystal structure within the soils. Diffraction test with X-ray diffraction method carried out on bentonite shows that the predominant clay mineral is the montmorillonite group. In addition, quartz, calcite, and traces of kaolinite and illite are observed. Results of chemical analysis show that Maghnia bentonite is composed mostly of silicate, with 17% aluminates (Table 1). The ratio of SiO_2 to Al_2O_3 is around 3.77, which is in agreement with the reported values of 2–5.5 (Pierson et al., 2004).

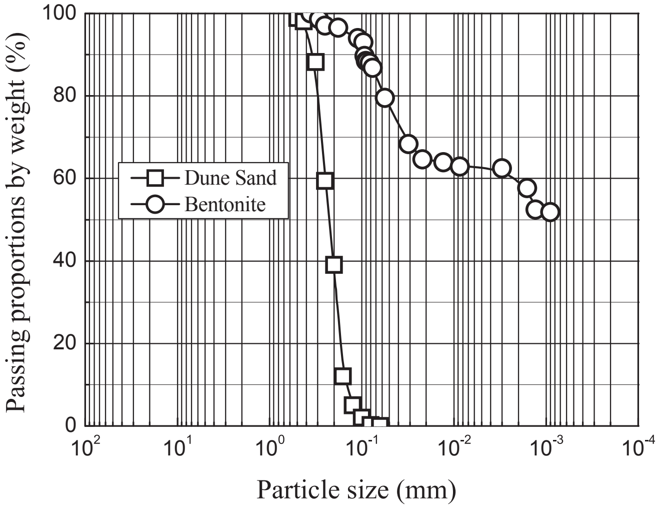

Fig. 1. Grain size distribution of bentonite and dune sand.

Dune sand known as desert sand is a material largely available in the South Algeria. The GSD curve of the dune sand is shown in Fig. 1. The corresponding coefficient of uniformity (C_u) and coefficient of curvature (C_c) are 1.7 and 1.1, respectively. According to the unified soil classification system (USCS), the dune sand is classified as poorly graded fine sand. The chemical analysis (Table 1) shows that the major component of dune sand is the silicate groups SiO_2 (about 95%).

2.2. Test methods

In order to get the required soil combination, several dune sand–bentonite mixtures have been considered in this study: 3% B + 97% S, 5% B + 95% S, 10% B + 90% S, 12% B + 88% S, 15% B + 85% S (B: bentonite, S: dune sand). Determination of the minimum proportion of bentonite to sand (B/S, %) should yield a suitable hydraulic conductivity (i.e. 10^{-8} – 10^{-10} m/s) or smaller (Chapuis, 1990; Parker et al., 1993), thus both the consolidation and compaction tests were used.

2.2.1. Compaction test

Compaction tests were carried out to assess the optimum water contents and the maximum dry unit weights of sand–bentonite mixtures. Sand–bentonite mixtures were prepared by mixing 3 kg of oven-dried dune sand with air-dried bentonite. The bentonite contents used were 0%, 3%, 5%, 10%, 12% and 15% by weight. For each sand–bentonite mixture, the optimum water content and the maximum dry unit weight were determined using standard Proctor compaction method (ASTM D698-91, 1991). The maximum dry unit

Table 1
Chemical composition of bentonite and dune sand.

Material	SiO_2	Al_2O_3	SO_3	$CaCO_3$	Cl^-	Na_2O	CaO	K_2O	MgO	Fe_2O_3
Bentonite	65.2	17.25	–	–	–	3.65	5.1	2.4	3.6	2.8
Dune sand	95.87	–	1.27	2.5	0.36	–	–	–	–	–

weight and corresponding optimum water content were obtained graphically from the peak of compaction curves.

For each compaction curve, the dry unit weight of the sand–bentonite mixture increases with increasing water content. As shown in Fig. 2, when the bentonite content varies from 3% to 15%, the maximum dry unit weight decreases from 19.2 kN/m^3 to 17 kN/m^3, and the corresponding optimum water content of the compacted sand–bentonite mixtures increases from 10% to 15.2%. When fine content (i.e. bentonite) is mixed with sand, more water is required in compaction in order to achieve the maximum dry unit weight (Holtz, 1985). When water is added to the mixture, the water acts like lubricant that allows soil particles to move closer to each other. When air void is minimized, higher unit weight can be achieved. Dry unit weight of compacted dune sand–bentonite mixtures decreases considerably with high contents of bentonites (10%, 12% and 15%).

2.2.2. Atterberg limits and free swelling tests

In soil mechanics, the fine materials are classified on the basis of Atterberg limits, which can provide information for macroscopic behavior (Mitchel, 1993). Mixtures with less than 10% of bentonite (3% B + 97% S, 5% B + 95% S) are nonplastic soils. For percentage of bentonite additions between 10% and 12%, the soils become as the low plastic clayey soils; whilst with percentage of 15%, the soil appears to be plastic. The plastic index ranges from 5.9% to 12.5% for bentonite content variation between 10% and 15%. Evolution of w_L according to percentage of bentonite additions follows a parabolic law (Fig. 3).

Swelling tests are carried out using a classical odometer. Dimensions of samples are 50 mm in diameter and 20 mm in height. The test is realized according to the free swelling method (Chen, 1988; Serratrice and Soyez, 1996). The dune sand–bentonite mixture samples are prepared by a static compaction (at a displacement rate of 1 mm/min) for water contents and dry densities corresponding to the optimum Proctor condition. The total free swelling G (%) is computed using the following relation:

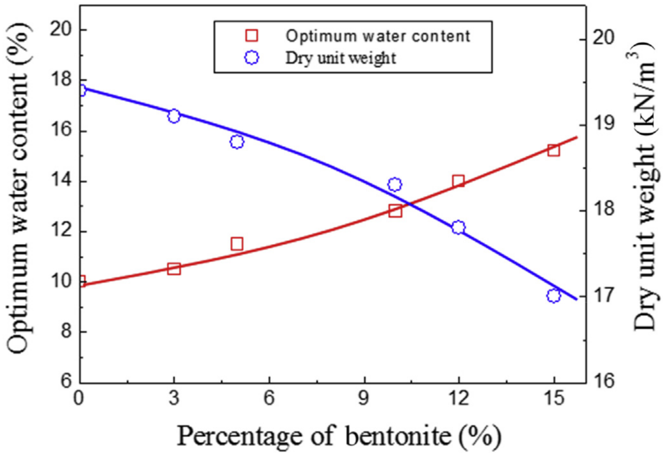

Fig. 2. Variation of various parameters of bentonites used in the context.

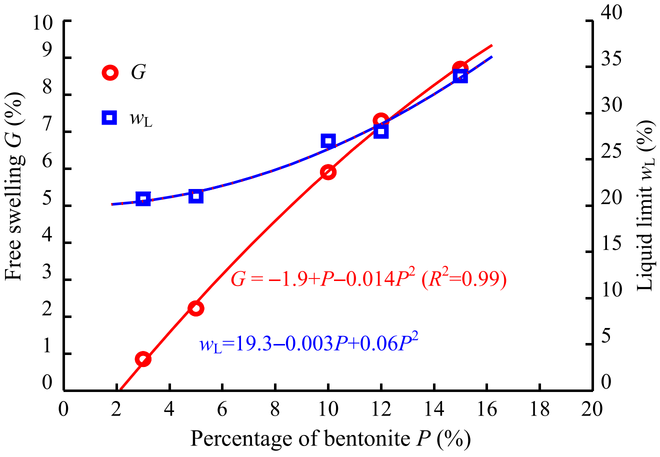

Fig. 3. Free swelling and liquid limit with various percentages of bentonite.

$$\left.\begin{aligned} G &= \frac{\Delta H}{H} \times 100\% \\ \Delta H &= H_f - H_0 \end{aligned}\right\} \qquad (1)$$

where H_0 is the initial height (before swelling), H_f is the final height (after swelling), and H is the height of specimen.

Results of free swelling with various percentages of bentonite are shown in Fig. 3. The free swelling value of the bentonite is approximately 47.5% and the swelling pressure according to ASTM D4546–90 (1990) is about 180 kPa; whilst the free swelling for sand–bentonite mixtures varies between 0.85% and 8.7% for bentonite content of 3–15%. As expected, the free swelling is proportional to the content of bentonite additions.

2.2.3. Indirect and direct measurements of saturated hydraulic conductivity

A compacted mixture of bentonite and sand is often used to form a seepage barrier (Kuroda et al., 1993). Sand–bentonite mixtures are composed of two radically different soils with respect to grain size, permeability, chemical activity, and strength.

(1) Indirect measurements of saturated hydraulic conductivity

The indirect methods for evaluating saturated hydraulic conductivity (k, m/s) are based on the results of odometer test (permeameter with rigid walls). The odometer test consists of vertical consolidation. For all mixtures, a specimen with 50 mm in diameter and 20 mm in height is prepared by static compaction at optimum normal Proctor condition. The specimen is placed in a metal ring and saturated for 24 h. In this study, the odometer test of 24 h incremental loading was carried out to investigate the variation of hydraulic conductivity during consolidation. An incremental loading ratio $\sigma'_{v(i+1)}/\sigma'_{v(i)}$ of 2 is used (σ'_v (kPa) is the vertical stress applied) according to geometric progression. The applied vertical stress to the specimen was from 10 kPa to 800 kPa (Holtz and Kovacs, 1996). The hydraulic conductivity (k) is obtained from both the coefficient of consolidation C_V (m^2/s) evaluated by Taylor's approach and the coefficient of volume compressibility m_v (m^2/kN) (see Eq. (2)). In this method, the coefficients C_V and m_v are deduced from compressibility curves and consolidation curves, respectively, in order to obtain the hydraulic conductivity.

In this study, the hydraulic conductivity k is written as

$$k = C_V m_v \gamma_w \tag{2}$$

where γ_w is the unit weight of water (kN/m^3).

(2) Direct measurements of saturated hydraulic conductivity

The direct saturated hydraulic conductivity is measured using a permeameter with flexible walls. The experimental setup is composed of a triaxial cell of the Bishop–Wesley type, controlled with three pressure-volume controllers of "GDS". The triaxial cell permits to apply an isotropic confining stress (up to 1700 kPa). The flow is made vertically from the bottom towards the top. The test is realized on the optimal mixtures (85% S + 15% B) obtained from the results of odometer tests. The specimen is prepared by static compaction with double piston at optimum normal Proctor condition ($w_{opt} = 15\%$, $\gamma_{d\ max} = 17\ kN/m^3$). Specimen dimensions are $d = 35$ mm and $H = 70$ mm. The displacement speed of the press is about 1.14 mm/s. The static compaction is retained because it permits to obtain more homogeneous specimens (Gueddouda et al., 2010).

The saturated hydraulic conductivity is measured under steady state (constant head). As the phase of saturation, once the specimen is placed inside the triaxial cell, a confining stress of 100 kPa is applied first. In order to extrude the existing air bubbles between the membrane and the soil, a low back pressure is applied at the base of the specimen (u_e: back pressure at the base at the sample, $u_e = 20$ kPa; u_s: the pore pressure at the top at the sample, $u_s = 0$ kPa). The progressive increases of the confining stress and the back pressure allow free air bubbles from the connecting tubes (i.e. $u_e = 30$ kPa and 40 kPa), and meanwhile preserve an average constant effective stress. The final confining stress applied on the sample during the saturation phase must be greater than or equal to the swelling pressure of the mixture, which is about 180 kPa. The vertical deformation versus time of the samples was recorded and the samples were considered as saturated when the displacement of the piston became constant (less than 0.01 mm in 24 h) (Souli et al., 2008).

An experimental program is carried out in order to investigate the effect of the average confining effective stress ($\sigma'_3 = 400$–1300 kPa) with a constant hydraulic gradient ($\Delta u = u_e - u_s = 40$ kPa).

The saturated hydraulic conductivity was computed using the Darcy's law, which describes the relation between the quantity of water flow through a cross-section during elapsed time. It is expressed by the following equation:

$$v = \frac{Q}{A} = ki \tag{3}$$

where v is the Darcy flux (m/s); Q is the volumetric flow rate (m^3/s); A is the flow area (m^2); and i is the hydraulic gradient, which can be expressed as

$$i = \frac{\Delta u}{\gamma_w H} \tag{4}$$

where Δu is the difference of pressure (kPa).

2.2.4. *Measurement of unsaturated hydraulic conductivity (VET: desiccators with saturated salt solutions and a digital balance)*

For higher suctions and continuous air interface phase, the transfer of water can occur in the vapor phase. The desiccators with saturated salt solutions are routinely used to determine the soil water retention curve (SWRC) data for total suction values ranging from 3 MPa to 400 MPa (Fleureau et al., 1993; Nishigaki et al., 1994; Delage et al., 1998; Delage and Fry, 2000; Tang and Cui, 2005; Taibi et al., 2005, 2009; Blatz et al., 2008; Goual et al., 2011). The sample is prepared with two initial moisture conditions: dried pastes (dried at 105 °C) and saturated slurry (water content of $1.5w_L$).

The data of the SWRC of the dune sand–bentonite mixtures (85% S + 15% B) were determined independently using different methods:

(1) Tensiometric plates were used to achieve low soil suction values between 1 kPa and 20 kPa. The specimens were placed on sintered glass filters, and a negative pressure was applied to the water. The air pressure is the atmospheric pressure.
(2) Osmosis was used to achieve the intermediate soil suctions between 0.05 MPa and 1.5 MPa. In the osmotic technique, Visking dialysis membranes obtained from Union Carbide, with very small pores (50 nm), were placed between the specimens and a solution of polyethylene glycol 20000 (PEG) (from Merck Laboratories) to prevent the passage of macromolecules. As the macromolecules tend to hydrate and attract water from the soil, the specimen was subjected to negative pore-water pressure, which depends on the PEG concentration in the solution.
(3) To achieve high suctions between 4 MPa and 400 MPa, the relative humidity (RH) in the soil specimens was controlled by salt solutions (Table 2). Details of the experimental techniques are given in the studies (Verbrugge, 1975; Biarez et al., 1987; Verbrugge and Fleureau, 2002).

This relatively low cost laboratory technique consists of placing the soil specimens in totally closed desiccators that contain a given saturated salt solution. The saturated salt solutions are used to control the RH of the atmosphere in the desiccators containing the soil specimens. The imposed total soil suction is related to the RH of the vapor space where the specimen is equilibrated by the following equation, which is derived from the Kelvin's law (Fredlund and Rahardjo, 1993):

$$s = \frac{-RT}{M_w \rho_w^{-1}} \ln RH \tag{5}$$

where R is the universal gas constant, and $R = 8.3143$ J/(mol K); T is the absolute temperature (K); M_w is the molecular weight of water, and $M_w = 18.016$ g/mol; ρ_w is the density of water (Mg/m^3) as a function of T; and RH is defined as u_v/u_{v0}, where u_v is the partial pressure of pore-water vapor in the soil specimen, and u_{v0} is the

Table 2
Relationship between saturated salt solutions and suction (at 24 °C).

Saturated salt solutions	$s = u_a - u_w$ (MPa)
K_2SO_4	4.23
$CuSO_4$	7.13
$ZnSO_4 \cdot 7H_2O$	14.64
KCl	22.59
NH_4Cl	34.54
NaCl	38.15
CH_3COOH	102.02
$CaCl_2 \cdot 6H_2O$	156.22

saturation pressure of pore-water vapor above a flat surface of water at temperature T.

The value of RH depends on the temperature, type and concentration of the salt solution used (Tang and Cui, 2005; Shackelford et al., 2010; Savoye et al., 2012). Therefore, the accuracy of this technique depends on the calibration used to determine the RH value generated by the used salt solution at the measured temperature (about 20 °C ± 2 °C). The laboratory experimental technique used for determining the hydraulic conductivity (k_w) functions of low permeability soils couples the total soil suction control in desiccators with saturated salt solutions with water mass measured from a digital balance (precision 10^{-3} g). The basic experimental setup consists of several desiccators with different saturated salt solutions and a digital laboratory balance (Fig. 4). The suction difference (Δs) between the soil specimen (s) and that imposed by the RH generated by the saturated salt solution in the desiccator (s_0) causes the water to flow in or out of the soil specimen until the potential equilibrium is reached. During the wetting or drying paths, the balance monitors the pore water gain (wetting path) or loss (drying path). The soil specimens are laterally sealed with paraffin for creating one-dimensional (1D) flow condition. During the unsteady state flow in the sample, one measures the variation of the average water content with time by weighing the soil specimen until the specimen's final suction reaches equilibrium. At this time, the flow stops (hydraulic flow rate (v) is equal to zero). The governing equation for 1D flow, neglecting gravity, is

$$\frac{\partial\psi}{\partial t} = D\frac{\partial^2\psi}{\partial z^2} \tag{6}$$

where ψ is the suction head, t is the time, z is the distance, and D is the diffusivity.

In general, Eq. (6) is nonlinear and difficult to be solved analytically. However, if the increments of s are sufficiently small, it is possible to assume that D is constant and that the s–θ relationship is linear between two steps, which make it possible to linearize Eq. (6) and obtain its analytical solution, considering the initial and boundary conditions as

$$\psi(z,0) = \psi \text{ (soil specimen initial suction)} \tag{7}$$

$$\psi(0,t) = \psi(\mathrm{l},t) = \psi_0 \text{ (imposed suction in the desiccator)} \tag{8}$$

where l is the length of soil specimen.

Using the method suggested by Gardner (1956), the solution to Eq. (6) is

$$\psi_w(z,t) = \psi_0 + \frac{4\Delta\psi}{\pi}\sum_{n=1}^{\infty}\frac{1}{n}\sin\frac{n\pi}{l}z\exp\left[-\left(\frac{n\pi}{l}\right)^2 Dt\right] \tag{9}$$

$$V_{w(t)} = V_{wT}\left[1-\frac{8}{\pi^2}\exp\left(-\frac{\pi^2}{l^2}Dt\right)\right] \tag{10}$$

$$\ln\left[V_{wT}-V_{w(t)}\right] = \ln\left(\frac{8V_{wT}}{\pi^2}\right) - \left(\frac{\pi^2}{l^2}D\right)t \tag{11}$$

$$D = \frac{l^2\mu}{\pi^2} \tag{12}$$

where $V_{w(t)}$ is the change in water volume at time t, V_{wT} is the total change in water volume, μ is the slope of the straight line in t – $\ln[V_{wT}-V_{w(t)}]$ space, and ψ_0 is the initial suction of soil specimen.

The coefficient of diffusivity D is related to the hydraulic conductivity k_w (Klute and Heermann, 1974; Taibi et al., 2009): $D = k_w(\partial\psi/\partial\theta)$, where θ is the volumetric water content, defined as the ratio of pore water volume V_w to the total volume of sample V_T, i.e. $\theta = V_w/V_T$.

When the sample is placed in the desiccator, its capillary potential varies from ψ_1 (initial capillary potential) to ψ_2 (imposed capillary potential in the desiccator), and then $\Delta\psi = \psi_2 - \psi_1$. The volumetric water content varies from θ_1 to θ_2, then we have $\Delta\theta = \theta_2 - \theta_1$.

The chosen $\Delta\psi$ must be small enough to meet the assumption ($\partial\psi/\partial\theta \approx \Delta\psi/\Delta\theta$), but large enough to provide a measurable volume of flow. The mean diffusivity is then written as $D = k_w\,(\Delta\psi/\Delta\theta)$, or $k_w = D(\Delta\psi/\Delta\theta)$. The volume of pore water V_W is related to its mass W_w by $V_w = W_w/\rho_w$. The variation of the volumetric water content $\Delta\theta$ can be related to the variation of the total change in water mass of the sample W_{wT} by $\Delta\theta = W_{wT}/(\rho_w V_T)$ and the k_w–D relationship is defined by (Lorenzetti et al., 2005):

$$k_w = \frac{D}{\partial\psi/\partial\theta} = \frac{D}{\Delta\psi}\frac{W_{wT}}{\rho_w V_T} \tag{13}$$

This method assumes that k_w is constant during the flow process. The soil mass measurements throughout the test, and the dry mass and total volume of the soil specimen measured at the end of the test are used to back-calculate the mean specimen volumetric water content at each step.

The external total volume of the specimens (between 1 cm^3 and 3 cm^3) was determined by immersing the specimens for 2 h in a

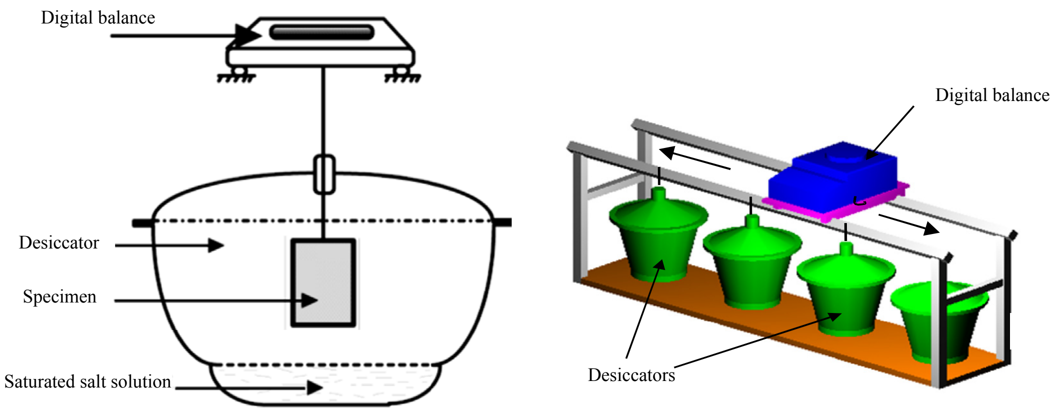

Fig. 4. Schematic diagram of the experimental VET equipment: (a) Sample during vapor equilibrium in desiccators, and (b) Several desiccators with different salt solutions.

non-wetting oil (commercial Kerdane, BP) to fill the largest pores without swelling and weighing them before and after imbibition (Fleureau et al., 1993; Shackelford et al., 2010).

The k_w is determined for each increment of suction, and the experiment is repeated for different magnitudes of suction or water content to yield a water hydraulic conductivity-suction (k_w–s) relationship. On wetting path, the k_w–s relationship is obtained by proceeding through a series of steady states with smaller and smaller suctions.

The method is also used for obtaining the drying k_w–s relationship (i.e. a series of steady states with increasing suction). The same soil specimen was continuously moistened and dried so that there is no hysteresis associated with alternately wetting and drying soil.

The time required to reach equilibrium for the soil specimen is obtained by taking the mass measurements throughout the test. The frequency of weighing varies from 1 per day at the beginning of the test, to 3 per day at the end. The tests are carried out on very small soil specimens (10 mm in height and 35 mm in diameter).

The samples used in the VET for the measurement of k_w functions were first statically compacted in a mold at the standard Proctor optimum water content (or saturation S_{rp}) and corresponding density. To evaluate whether hysteresis exists in the k_w–s relationships for the sand-mixtures, the procedures were used to prepare the soil specimens:

(1) For the wetting path starting from $S_{ro} = 0\%$, some of the compacted specimens were allowed to dry in the oven at 105 °C ± 5 °C for 24 h, and
(2) For the drying path starting from $S_{ro} = 100\%$, some of the specimens were saturated by applying vacuum and flushing de-aired water into the specimen for a period of 120 min.

3. Results and discussions

3.1. Saturated hydraulic conductivity

Evolution of saturated hydraulic conductivity of soil–bentonite mixture as a function of loading pressure is shown in Fig. 5. According to the results obtained, it shows that:

(1) For all mixtures, the hydraulic conductivity varies inversely with the loading pressure. For example, when the loading pressure varies from 25 kPa to 800 kPa, the hydraulic conductivity for mixtures, with higher bentonite additions, decreases approximately two orders of magnitude.
(2) The saturated hydraulic conductivity for dune sand varies between 1.1×10^{-5} m/s and 1.9×10^{-6} m/s; whereas for mixtures with 15% bentonite addition, the value ranges from 7.41×10^{-9} m/s to 4.58×10^{-11} m/s.
(3) The effect of loading pressures applied on hydraulic conductivity is less significant, once the vertical pressure is more than 200 kPa. Other researchers found these limiting values were around 100 kPa (Wu and Khera, 1990) and 200 kPa (Alston et al., 1997).
(4) Hydraulic conductivity of the dune sand–bentonite mixtures decreases with increasing bentonite content. For bentonite content more than 12%, the saturated hydraulic conductivity is less than 10^{-8} m/s.
(5) The target values relative to the saturated hydraulic conductivity for contaminant liners, which should be between 10^{-8} m/s and 10^{-10} m/s, can be achieved for bentonite content greater than 12%, under an applied vertical pressure larger than 100 kPa. While for the bentonite addition of around 15%, this target value is obtained under a low vertical pressure (around 25 kPa).
(6) According to these results, the adequate sand–bentonite mixture used for the following studies is 85% S + 15% B ($S_{85}B_{15}$). This mixture satisfies the principal condition of permeability (10^{-8} m/s $< k <$ 10^{-10} m/s). Hence, this combination referred as $S_{85}B_{15}$ will be used in the design of engineered barriers for the achievement of the insulation barriers of storage facilities for hazardous wastes.

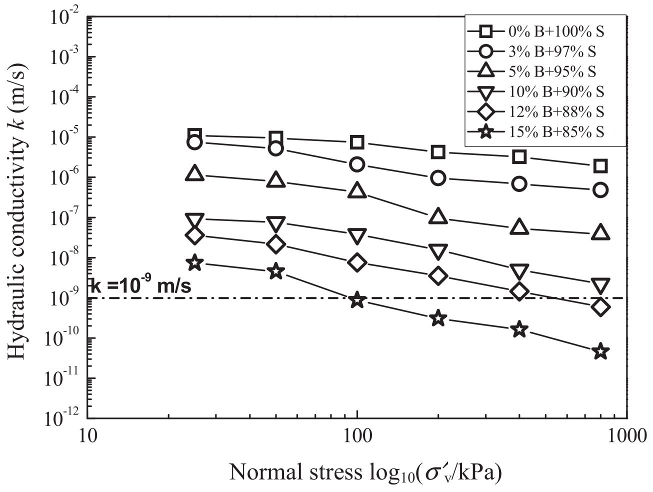

Fig. 5. Hydraulic conductivity of sand–bentonite mixtures with applied vertical stress.

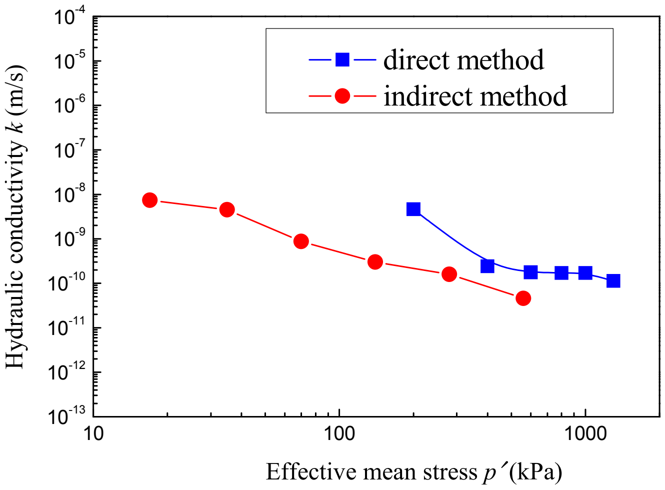

Fig. 6. Hydraulic conductivity variation versus average confining effective stress ($S_{85}B_{15}$).

Fig. 6 shows the saturated hydraulic conductivity variation versus the average effective stresses when hydraulic gradient is constant, i.e. $\Delta u = 40$ kPa and $i = 57.14$. The odometer test consists of vertical consolidation and the triaxial test in isotropic consolidation. The mean stress of consolidation $p\prime$ is determined for the both test methods. For the odometer test (Fleureau et al., 2002), we have

$$p\prime = \frac{(1 + 2k_0)\sigma'_v}{3} \tag{14}$$

$$\sigma'_{\rm H} = k_0\sigma'_{\rm v} \tag{15}$$

where $k_0 = 1 - \sin\varphi$ is the pressure coefficient of the soil and φ is the friction angle determined by triaxial test ($\varphi = 30°$).

Substituting Eq. (15) to Eq. (14) yields

$$p' = \frac{\sigma'_{\rm v}+2\sigma'_{\rm H}}{3} \tag{16}$$

According to Eq. (14), when $k_0 = 0.5$, we have

$$p' = \frac{2\sigma'_{\rm v}}{3} \tag{17}$$

For triaxial test in isotropic consolidation, we have $\sigma'_1 = \sigma'_2 = \sigma'_3$, thus it yields $p' = \sigma'_3$. Fig. 6 shows the comparison of the results obtained by the two methods. It can be observed that the effect of the average confining effective stress on the saturated hydraulic conductivity is more significant for values less than 200 kPa. Beyond this stress value, the saturated hydraulic conductivity seems to be constant, which agrees with odometer test results. The results of the saturated hydraulic conductivity obtained by the odometer tests are lower than those obtained by the triaxial cell. This can be attributed to the lack of saturation of the specimen under odometer conditions and some air bulbs may remain trapped within the soil (Gueddouda et al., 2010).

Table 3 summarizes the positioning of mixture $S_{85}B_{15}$ compared to the regulatory recommendations. The studied mixture satisfies the requirements of the realization of liner base layers in the conception of insulation barriers for hazardous waste facilities.

3.2. *Unsaturated hydraulic conductivity*

The unsaturated hydraulic conductivity of sand–bentonite mixture ($S_{85}B_{15}$) is determined using the VET desiccators with saturated salt solutions and a digital balance. However, problems were met with these devices and eventually suctions were derived from the SWRC using the relationship between water contents and suctions. The data of the SWRC of the dune sand–bentonite mixtures ($S_{85}B_{15}$) were determined using different methods independently, i.e. tensiometric plates (1–20 kPa), osmotic technique (0.05–1.5 MPa) and VET with saturated salt solutions (4–300 MPa) (see Table 2) to control the soil suction (Fig. 7).

Compacted specimens are partially saturated and their pore-water pressure is negative. In most cases, a calibrated filter paper (Whatman 42) is used to obtain the initial negative pore-water pressure of the soil at the end of the rest period. This value is derived from the water content of the filter paper, which is measured with a very precise force transducer (10^{-6} N) by reference to a calibration curve (ASTM D5298–94, 1994). The calibrated filter papers were placed in the specimens during compaction, protected on both sides by ordinary filter papers, between two soil layers. It must be noted that the filter papers were directly in contact with the soil, thus only the matrix negative pore-water pressure was measured (Fleureau et al., 2002). The SWRC data for

Table 3
Positioning of mixture $S_{85}B_{15}$ from the regulatory recommendations.

Regulatory recommendations	Mixture $S_{85}B_{15}$
$w_{\rm L} \geq$ 25–30%	$w_{\rm L} = 34\%$
$PI \geq$ 10–20%	$PI = 12.5\%$
<10% of gravel diameter larger than 5 mm	0%
Organic carbon content< 5%	0%
Swelling properties G <10%	$G = 8.7\%$
Very low permeability $k < 10^{-9}$ m/s	$k \approx 2.1 \times 10^{-10}$ m/s

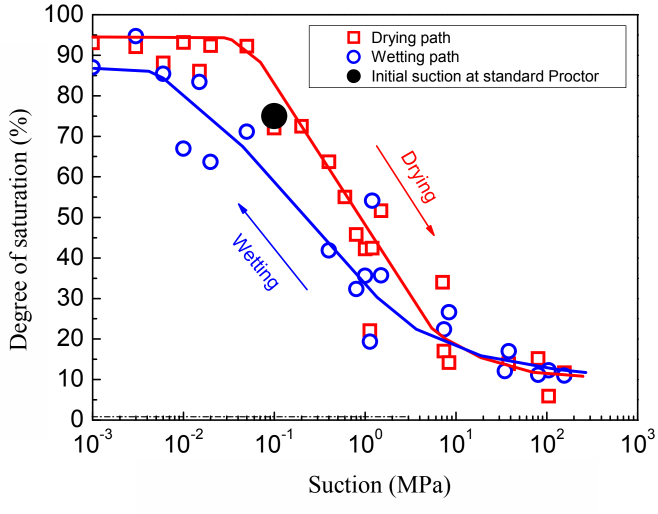

Fig. 7. SWRC data for the compacted dune sand–bentonite mixture (VET method) specimens.

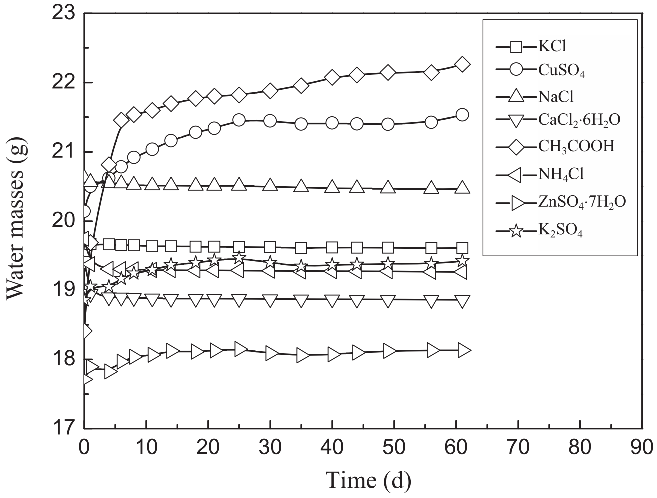

Fig. 8. Variation of water masses with time using different imposed suctions.

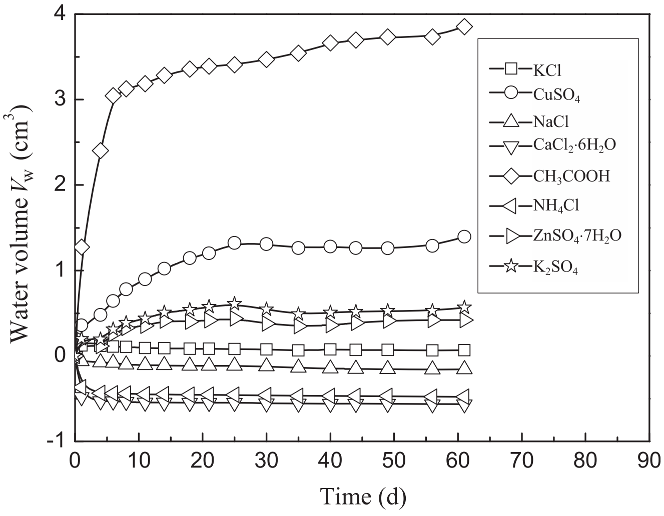

Fig. 9. Variation of samples' water volume with time using different imposed suctions.

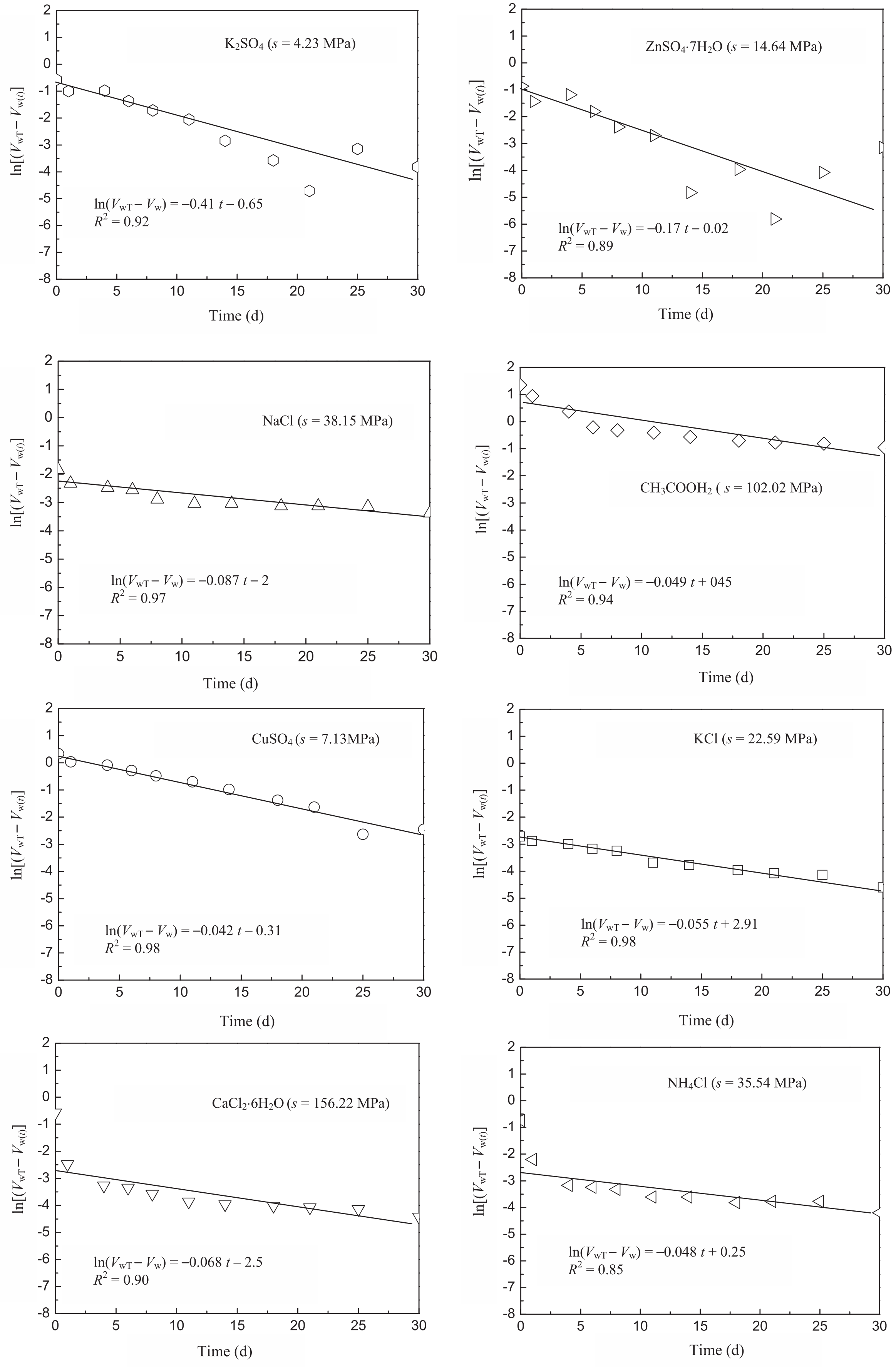

Fig. 10. Variation of samples water volume [$\ln(V_{wT} - V_{w(t)}$] with time using different imposed suctions.

the dune sand–bentonite mixtures obtained from the VET tests are presented in Fig. 7.

The variations of the water mass, water volume and the resulting $\ln[V_{WT} - V_{w(t)}]$ against time for the tested dune sand–bentonite mixtures are presented in Figs. 8–10, respectively.

The values of suction are determined by the VET, and the corresponding saturation values are obtained using a non-wetting oil (commercial Kerdane, BP) to determine the external total volume of the specimens (Fleureau et al., 1993).

Equilibrium time was reached after about 60 d for the small tested dune sand–bentonite mixtures specimens (10 mm in height and 35 mm in diameter). The hydraulic conductivity is analytically determined from the graph of $\ln[V_{WT} - V_{W(t)}]$ versus time. Examples of the resulting plot for the drying path of the saturated dune sand–bentonite mixture are presented in Fig. 9. The experimental results presented in Fig. 10 confirm the assumed linearity of the $\ln[V_{wT} - V_{W(0)}]$–time relationship for the materials tested. Equilibrium is reached when the variation of the weight of the water mass versus time (Fig. 8) does not exceed 0.01% per day. Taibi et al. (2009) mentioned that the equilibrium time depends on the soil properties, the soil specimen sizes and the suction differences.

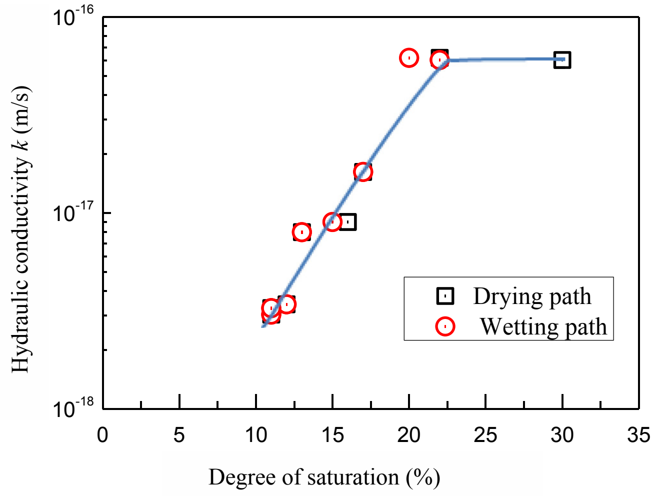

Fig. 11. Evolution of hydraulic conductivity with degree of saturation.

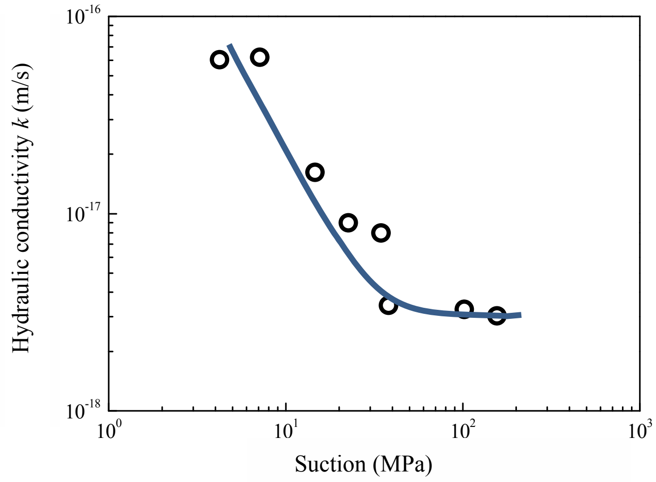

Fig. 12. Evolution of hydraulic conductivity with imposed suction.

In order to decrease the equilibrium time, the measured k_w values from different initial degrees of saturation S_{ro} and suction, for the compacted dune sand–bentonite mixture, are plotted in Figs. 11 and 12, respectively. The experimental results suggest that the hysteretic effect on the k_w–S_r and k_w–s relationships cannot be neglected in this saturation range ($20\% < S_r < 30\%$), and that the k_w-s relationship appears to exhibit less hysteresis than that at relatively high suctions ($4 \text{ MPa} < s < 156 \text{ MPa}$). The results obtained were in good agreement with the data from the literature (Fleureau et al., 2002; Taibi et al., 2009). The unsaturated hydraulic conductivity decreases with increasing suction (see Fig. 12). The unsaturated hydraulic conductivity has a constant value for suctions larger than 20 MPa, which is approximately 7×10^{-18} m/s.

4. Conclusions

In this study, it shows the possibility to obtain an adequate mixture intended for insulation barriers, using desert dune sand and a small amount of bentonite addition. According to the results obtained from this study, the following conclusions can be drawn:

(1) The maximum dry unit weight and optimum water content decrease with increasing bentonite content of the compacted sand–bentonite mixtures.
(2) The common requirement on hydraulic conductivity (less than $10^{-8} - 10^{-10}$ m/s) is met for compacted soil with a minimum bentonite content of 12%. The saturated hydraulic conductivity of the sand–bentonite mixtures decreases with increasing bentonite content.
(3) The saturated hydraulic conductivity obtained by the odometer tests is lower than that obtained by the triaxial cell. This can be attributed to the unsaturation of the specimen under odometer conditions and possibly some air bulbs trapped within the soil.
(4) The unsaturated hydraulic conductivity increases proportionally with the increasing degree of saturation. The unsaturated hydraulic conductivity has a constant value for suctions larger than 20 MPa. The VET can be used to measure very small k_w values (10^{-16} m/s $< k_w < 10^{-18}$ m/s).

Finally, it can be noted that the use of dune sand, which is a locally available material in the South Algeria, with small quantities of bentonite additions can provide technical, economic and environmental insulation barriers for waste disposal management.

Conflict of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

References

Ait Saadi L. Control methodology of homogeneity and permeability of clayey barriers. PhD Thesis. Lyon, France: INSA; 2003.
Alston C, Daniel DE, Devroy DJ. Design and construction of sand-bentonite liner for effluent treatment lagoon, Marathon, Ontario. Canadian Geotechnical Journal 1997;34(6):841–52.
ASTM D4546–90. Standard test methods for one-dimensional swell or collapse of soils. Philadelphia, PA: ASTM Standards; 1990.
ASTM D698–91. Test method for laboratory compaction characteristics of soil using standard effort (600 kN·m/m^3). Philadelphia, PA: ASTM Standards; 1991.
ASTM D5298–94. Standard test method for measurement of soil potential (suction) using filter paper. Philadelphia, PA: ASTM Standards; 1994.
Biarez J, Fleureau JM, Zerhouni MI, Soepandji BS. Variations de volume des sols argileux lors de cycles de draînage-humidification. Revue Française de Géotechnique 1987;41:63–71 (in French).

Blatz JA, Graham J, Chandler NA. Influence of suction on the strength and stiffness of compacted sand–bentonite. Canadian Geotechnical Journal 2002;39(5):1005–15.

Blatz JA, Cui YJ, Oldecop L. Vapour equilibrium and osmotic technique for suction control. Geotechnical and Geological Engineering 2008;26:661–73.

Booker JR, Brachman R, Quigley RM, Rowe KR. Barrier systems for waste disposal facilities. 2nd ed. London: Taylor & Francis; 2004. p. 587.

Chalermyanont T, Arrykul S. Compacted sand-bentonite mixtures for hydraulic containment liners. Songklanakarin Journal of Science and Technology 2005;27(2):313–23.

Chapuis RP. Sand-bentonite liners: predicting permeability from laboratory tests. Canadian Geotechnical Journal 1990;27(1):47–57.

Chen FH. Foundations on expansive soils. Amsterdam: Elsevier; 1988.

Cui YJ, Tang AM, Loiseau C, Delage P. Determining water permeability of compacted bentonite-sand mixture under confined and free-swell condition. Physics and Chemistry of the Earth 2008;33:462–71.

Daniel DE. Predicting hydraulic conductivity of clay liners. Journal of Geotechnical Engineering 1984;110(2):285–300.

Delage P, Howat MD, Cui YJ. The relationship between suction and swelling properties in a heavily compacted unsaturated clay. Engineering Geology 1998;50(1–2):31–48.

Delage P, Fry JJ. Comportement des sols compactés: apport de la mécanique des sols non saturés. Revue Française de Géotechnique 2000;22:17–29 (in French).

Du YJ, Fan RD, Reddy KR, Liu SY, Yang YL. Impacts of presence of lead contamination in clayey soil–calcium bentonite cutoff wall backfills. Applied Clay Science 2015;108:111–22.

Egloffstein TA. Natural bentonites-influence of the ion exchange and partial desiccation on permeability and self-heating capacity of bentonites used in GCLs. Geotextiles and Geomembranes 2001;19(7):427–44.

Fleureau JM, Kheirbek-Saoud S, Soemitro R, Taibi S. Behavior of clayey soils on drying–wetting paths. Canadian Geotechnical Journal 1993;30(2):287–96.

Fleureau JM, Verbrugge JC, Huergo PJ, Correia AG, Kheirbek-Saoud S. Aspects of the behaviour of compacted clayey soils on drying and wetting paths. Canadian Geotechnical Journal 2002;39(6):1341–57.

Fredlund DG, Rahardjo H. Soil mechanics for unsaturated soils. New York: John Wiley & Sons, Inc.; 1993.

Gardner WR. Calculation of capillary conductivity from pressure plate outflow data. Soil Science Society of American Proceedings 1956;20(3):317–20.

Goual I, Goual MS, Taibi S, Abou-Bekr N. Behaviour of unsaturated tuff-calcareous sand mixture on drying-wetting and triaxial paths. Geomechanics and Engineering 2011;3(4):267–84.

Graham J, Yuen K, Goh TB, Janzen P, Sivakumar V. Hydraulic conductivity and pore fluid chemistry in artificially weathered plastic clay. Engineering Geology 2001;60(1–4):69–81.

Gueddouda MK, Lamara M, Abou-Bekr N, Taibi S. Hydraulic behaviour of dune sand–bentonite mixtures under confining stress. Geomechanics and Engineering 2010;2(3):213–27.

Holtz WG. Discussion of “Predicting hydraulic conductivity of clay liners” by David E. Daniel. Journal of Geotechnical Engineering 1985;111(12):1457–9.

Holtz R, Kovacs W. An introduction to geotechnical engineering. 2nd ed. Montréal, Canada: Ecole Polytechnique de Montréal; 1996. p. 808.

Klute A, Heermann DF. Soil water profile development under a periodic boundary conditions. Soil Science 1974;117(5):265–71.

Kuroda T, Katahira F, Sasaki T, Imamura S. Outline of Rokkasho low-level radioactive waste disposal center and characteristic of bentonite/sand mixtures. In: Civil Engineering in Japan. Tokyo, Japan: Japan Society of Civil Engineering; 1993. p. 39–93.

Lorenzetti RJ, Bartelt-Hunt SL, Burns SE, Smith JA. Hydraulic conductivities and effective diffusion coefficients of geosynthetic clay liners with organobentonite amendments. Geotextiles and Geomembranes 2005;23(5):385–400.

Malusis M, Di Emidio G. Hydraulic conductivity of sand-bentonite backfills containing HYPER clay. In: Geo-congress 2014. Atlanta, Georgia, USA: American Society of Civil Engineers; 2014. p. 1870–9.

Marcoen JM, Tessier D, Thorez J, Monjoie A, Schroeder CH. Manuel relatif aux matières naturelles pour barrières argileuses ouvragées pour C.E.T et réhabilitation de dépotoirs en Région wallonne. Wallonne: Ministère de la Région Wallonne; 2001 (in French).

McWatters R, Rowe RK. Diffusive transport of VOCs through LLDPE and two coextruded geomembranes. ASCE Journal of Geotechnical and Geoenvironmental Engineering 2010;136(9):1167–77.

Ministry of Planning. Environment and Tourism (MPET) of Algeria. Technicians Guide for the management of municipal waste and assimilated. Algeria: Ministry of Planning, Environment and Tourism of Algeria; 2013.

Mitchel JK. Fundamentals of soil behaviour. New York: John Wiley & Sons, Inc.; 1993.

Montañez JEC. Suction and volume changes of compacted sand-bentonite mixtures. PhD Thesis. London: University of London; 2002.

Nishigaki M, Sun Y, Kono I. A study of deformation and failure of compacted sand/bentonite mixture soil foundation by model test and numerical analysis. In: Proceedings of the International Symposium on Pre-failure Deformation of Deomaterials, vol. 1. IS-Hokkaido, Sapporo, Japan: A.A. Balkema; 1994. p. 523–9.

Parker RJ, Bateman S, Williams D. Design and management of landfills, eotechnical management of waste and contamination. Rotterdam: A.A. Balkema; 1993. p. 209–52.

Pierson P, Cazaux D, Guyonnet D. Fonds d’installations de stockage de déchets, 5eme Rencontres Géosynthétiques Francophones. Lille, France: CFG; 2004. p. 85–96 (in French).

Rowe KR, Chappel MJ, Brachman RWI, Take WA. Field study of wrinkles in a geomembrane at a composite liner test site. Canadian Geotechnical Journal 2012;49(10):1196–211.

Sangam HP, Rowe RK. Effect of surface fluorination on diffusion through a high density polyethylene geomembrane. Journal of Geotechnical and Geoenvironmental Engineering 2005;131(6):694–704.

Santucci de Magistris F, Silvestri F, Vinale F. Physical and mechanical properties of a compacted silty sand with low bentonite fraction. Canadian Geotechnical Journal 1998;35(6):909–25.

Savoye S, Michelot JL, Matray JM, Wittebroodt C, Mifsud A. A laboratory experiment for determining both the hydraulic and diffusive properties and the initial pore-water composition of an argillaceous rock sample: a test with the Opalinus clay (Mont Terri, Switzerland). Journal of Contaminant Hydrology 2012;128(1–4):47–57.

Scalia J, Benson C, Bohnhoff G, Edil T, Shackelford C. Long-term hydraulic conductivity of a bentonite-polymer composite permeated with aggressive inorganic solutions. Journal of Geotechnical and Geoenvironmental Engineering 2013;140(3). 04013025–1-13.

Serratrice JF, Soyez B. Les essais de gonflements. Revue Bulletin des laboratoires des ponts et chaussées. France: French Institute of Science and Technology for Transport, Development and Networks [Ifsttar]; 1996. p. 65–85 (in French).

Shackelford C, Sevick G, Eykhol G. Hydraulic conductivity of geosynthetic clay liners to tailings impoundment solutions. Geotextiles and Geomembranes 2010;28(2):149–62.

Skempton AW. The collaidal activity of clays. In: Proceedings of the Third International Conference on Soil Mechanics and Foundations Engineering. vol. 1. Zurich, Swiss, Selected Papers on Soil Mechanics by Skempton AW. London: Thomas Telford Limited; 1953. p. 57–61.

Souli H, Fleureau JM, Trabelsi Ayadi M, Besnard M. Physicochemical analysis of permeability changes in the presence of zinc. Geoderma 2008;145(1–2):1–7.

Taibi S, Ghembaza MS, Fleureau JM. On the suction control techniques in studying the THM behavior of unsaturated soils. In: Proceedings of International Symposium on Advanced Experimental Unsaturated Soil Mechanics. Rotterdam: A.A. Balkema; 2005. p. 69–75.

Taibi S, Bicalho KV, Sayad-Gaidi C, Fleureau JM. Measurements of unsaturated hydraulic conductivity functions of two fine-grained materials. Soils and Foundations 2009;49(2):181–91.

Tang AM, Cui YJ. Controlling suction by the vapour equilibrium technique at different temperatures, application in determining the water retention properties of MX80 clay. Canadian Geotechnical Journal 2005;42(1):287–96.

Thériault P. Etude de l’influence des métaux lourds sur la conductivité hydraulique de couches sable/bentonite. MSc Thesis. Ville de Québec, Canada: Unnersité Laval; 2000 (in French).

Verbrugge JC. Validité des résultats fournis par les appareils usuels de mesure de la succion et par les psychromètres à effet Peltier: Considérations Théoriques. Revue Générale des Routes et Aérodromes 1975;515:56–65 (in French).

Verbrugge JC, Fleureau JM. Bases expérimentales du comportement des sols non saturés. In Mécanique Des sols non saturés. In: Coussy O, Fleureau JM, editors. Collection MIM. Paris, France: Lavoisier Hermès Science; 2002. p. 69–112 (in French).

Watabe Y, Tsuchida T, Kang M. Influence of sand fraction on the impermeability of seepage control clayey layer. In: Tremblay H, Locat J, Galvez-Cloutier R, editors. Proceedings of the 2nd International Symposium on Contaminated Sediments. Quebec, Canada; 2003. p. 163–9.

Wu JY, Khera RP. Properties of a treated-bentonite/sand mix in contaminant environment. In: Hoddinott KB, Lamb RO, editors. Physic-chemical Aspects of Soil and Related Materials. West Conshohocken, PA, USA: American Society for Testing and Materials; 1990. p. 47–59.

Yilmaz I, Marschalko M. The effect of different types of water on the swelling behaviour of expansive clays. Bulletin of Engineering Geology and the Environment 2014;73(4):1049–62.

Mohammed Kamel Gueddouda obtained PhD in geotechnical engineering from University of Tlemcen – Algeria. Dr. Gueddouda is now research teacher and his main research interests cover swelling soils, behavior of saturated and unsaturated soils, road engineering, valorization of local materials in civil engineering.

Address: Research Laboratory of Civil Engineering (LRGC), University of Laghouat, Algeria.