Geomechanics and Engineering, Vol. 2, No. 3 (2010) 213-227 213
DOI: http://dx.doi.org/10.12989/gae.2010.2.3.213

Hydraulic behaviour of dune sand-bentonite mixtures under confining stress

M.K. Gueddouda*[1], M. Lamara[1], N. Abou-bekr[2] and S. Taibi[3]

[1]Laboratoire de recherche de Génie Civil, Université Amar Teledji, BP.37, G. Laghouat, Algeria
[2]Laboratoire Eau et Ouvrages dans Leur Environnement, Université A. Belkaid, BP 230 - 13000 Tlemcen, Algeria
[3]Laboratoire Ondes et Milieux Complexes, FRE CNRS 1302, Université du Havre, 53 rue de Prony, 76600 Le Havre, France

(Received March 7, 2010, Accepted September 8, 2010)

Abstract. Compacted layers of sand-bentonite mixtures have been proposed and used in a variety of geotechnical projects as engineered barriers for the enhancement of impervious landfill liners, cores of zoned earth dams and radioactive waste repository systems. This paper presents a study on the valorization of local materiel such as dune sand from Laghouat region and mine bentonite intended for the realization of liner base layers in the conception of insulation barriers for hazardous waste centers. In the practice we try to get an economical mixture that satisfies the hydraulic and mechanical properties specified by regulation rules. The effect of the bentonite additions on the mixture is reflected by its capability of clogging the matrix pores upon swelling. In order to get an adequate dune sand-bentonite mixture, an investigation on hydraulic and mechanical behaviours is carried out in this study for different mixtures. Using oedometer test, the adequate bentonite addition to the mixture, which satisfies the conditions on permeability, is found to be around 12% to 15%. These results are also confirmed by direct measurement using triaxial cell.

Keywords: dune sand; bentonite; saturated permeability; insulation barriers; south of Algeria.

1. Introduction

Rapids technological advances and population needs lead to the generation of increasingly hazardous wastes. The society should face two fundamental issues, waste management and pollution risks control. One of the actual solutions, for handling these contamination problems, is by enclosing the wastes in a specific location using insulated barriers. Many different barrier materials exist, for example, plastic membranes, sand-bentonite compacted layers, cement stabilised soils (Sallfors and Oberg-Hogsta 2002). The permeability of insulation barriers has been studied by many authors for different type of soil, compacted clays (Daniel 1984, Harrop-Williams 1985), silty soils (Holtz 1985), and sand-bentonite mixtures (Chapuis 1990, Montañez 2002, Blatz *et al*. 2002, Chalermyanont et Arrykul 2005, Aina 2006, Cui *et al*. 2008). The efficiency of these insulated barriers depends largely on their hydraulic and mechanical behaviour along with their abilities of

*Corresponding author, E-mail: gueddouda_ mk@yahoo.fr

contaminant retention.

Compacted sandy soils with small additions of bentonite (bentonite-sand mixture) have been proposed and used as an adequate material for these insulation layers. In order to be efficient, theses insulation barriers should fulfil some specifications: (i) the typical thickness for these layers should be between 15 and 30 cm (Thériault 2000); (ii) permeability at saturated state ranges between 10^{-8} and 10^{-10} m/s (Chapuis 1990, Parker 1990, Souli *et al*. 2008); (iii) properties of exchange and adsorption should be able to hold some preferentially polluting substances (Gleason *et al*. 1997); (iv) physical stability of the material in contact with water (Thériault 2000, ADEME 1999); (v) 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; (vi) the sand should possess some characteristics of grain size distribution, in order to prevent bentonite leaching from the skeleton and hence ensuring the hydraulic stability of the mixture.

In current engineering applications, soil-bentonite mixtures are mainly used to build impervious cores of earth dams and contain groundwater pollution. Different percentages of bentonite additions are reported by many authors (Montañez 2002, Lopez *et al*. 1984, Nishigaki *et al*. 1994, D'Appolonia 1980, Schnitter and Zeller 1961, Lauffer and Schober 1961). Gillham (1984), Chapuis (1990) have found this percentage ranges between 5% to 8% for sodium bentonite and from 9% to 15% for calcium bentonite.

In Algeria is generated each year 10 to 12 million tonnes of household waste. It has 3,000 illegal dumps. These landfills occupy a total area of about 150.000 hectars (Ministry of Planning, Environment and Tourism of Algeria. Technicians Guide for the management of municipal waste and assimilated (2009)).

In Algeria, the rapid development of urban areas and the growth of oil industry in the southern regions, 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, enhanced by small addition of bentonite.

Generally, sodium bentonite is used more extensively than calcium bentonite because of its superior swelling potential and hence results in a low hydraulic conductivity of the mixtures (Reschke A.E, Haug M.D 1991), however it is less available through the world, instead the calcium bentonite is used in the study, which brought from the region of Maghnia in the west of Algeria.

In searching for an adequate mixture, an investigation study is carried out on several_mixtures with different percentages of bentonite additions, which varies between 3% to 15%. The adequate bentonite addition to the mixtures, which satisfies the required conditions on the permeability, is obtained using oedometer test. Than after, the hydraulic conductivity under confining stress and different pressures gradient is investigation with direct methods using triaxial cell. The measurements of permeability is conducted under two modes, permanent (constant head) and transitory (pulse test).

2. Materials used for study

In most cases the maximum allowable hydraulic conductivity for a barrier material is specified by the regulations. The base material, sand is usually a local material and can be different from regions to regions. The task to produce a barrier material in place, which meets the given criteria at adequate cost, requires good knowledge of the different factors affecting the final hydraulic conductivity of the mixture.

2.1 Bentonite of Maghnia

The term 'Bentonite' is now well established, and used to describe a clay material whose major mineralogical components belong to Smectite groups. Hence, bentonite is a very expansive soil. The most important bentonite mines in Algeria are situated in the western regions. The bentonite used in this study is extracted from Maghnia mine (Hammam Boughrara, 600 km west of the capital Algiers).

X-ray diffraction is one of the most widely used methods for clay minerals identification and studying their crystal structure within the soils. Diffraction test carried out on bentonite, showed that the predominant clay minerals are smectic types Fig. 1, beside it reveals also the presence of illite, quartz, and traces of kaolinite.

The grain size distribution of the bentonite is shown in Fig. 2. It is very fine clay; more than 60% of particles have a diameter less than 2 μm. It's Plasticity Indexes (LL=141%, PI=93%); indicate that the bentonite of Maghnia is highly plastic clay, this is also confirmed by a large specific surface (Ss=462 m^2/g). According to the Skempton classification (Skempton 1953), based on the activity (Eq. (1)), the bentonite of Maghnia presents a high percentage of calcite Montmorillonite (Ca^{+2}).

$$A = \frac{PI}{C_2} \tag{1}$$

A : activity

C_2: percentage of particles with a diameter less than 2 μm

2.2 Dune sand

Dune sand known as desert sand is a material largely available within the Algerian south. In this study the dune sand used is a local material from Laghouat region. According to the chemical

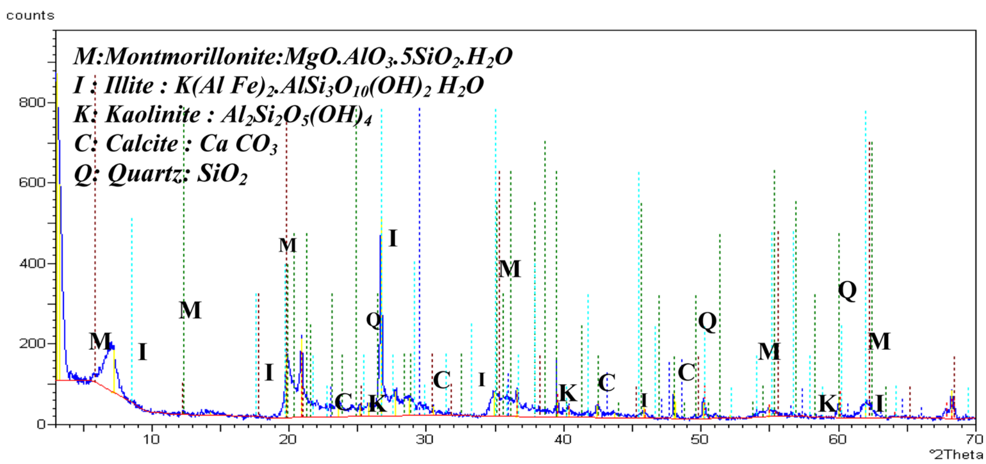

Fig. 1 X-ray diffraction analysis of bentonite of Maghnia

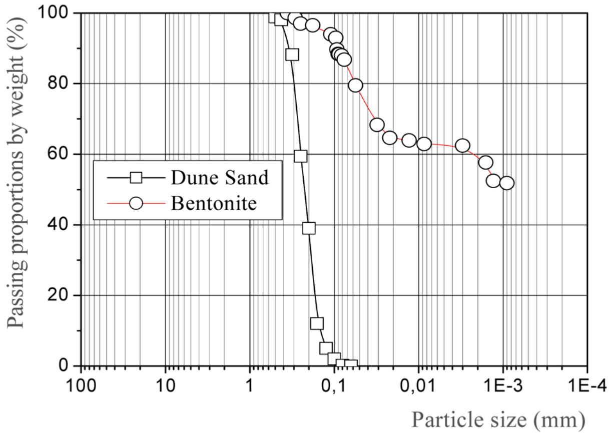

Fig. 2 Grain size distribution of bentonite and dune sand

Table 1 Chemical composition of the dune sand

SiO_2 %	SO_3%	Cl %	$CaCO_3$%	M.O %
95.87	0.91	0.36	2.5	–

analysis (Table 1), the major component of dune sand (around 95%), is the silicate groups SiO_2.

The grain size distribution of dune sand is shown in Fig. 2. Values corresponding to uniformity and curvature coefficients are C_u=1.67 and C_c=1.1, respectively. Thus, according to the Unified Soil Classification System (USCS), the dune sand is classified as poorly graded fine sand.

2.3 Properties of dune sand - bentonite mixtures

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).

2.3.1 Consistence limits

In soil mechanics, the fine materials are classified on the basis of consistency limits which can provide information on macroscopic behaviour (Mitchell 1993) Atterberg limits obtained for different mixtures are presented in Fig. 3. The consistency limits are proportional with percentage of bentonite additions. Mixtures with less than 10% of the bentonite are non plastic soils. For percentage of bentonite additions between 10% and 12%, the soil shows a low plasticity, while with percentage of 15% the soil appears to have a moderate plasticity around 12.5% (Table 2).

2.3.2 Swelling behaviour

Swelling tests are carried out using a classical oedometer, according to the free swelling method (Serratrice and Soyez 1996). The dune sand – bentonite mixture samples were prepared by a static compaction for water contents and dry densities corresponding to the optimum Proctor conditions. A fixed displacement rate of 1 mm/min was applied by a compression machine. The total free

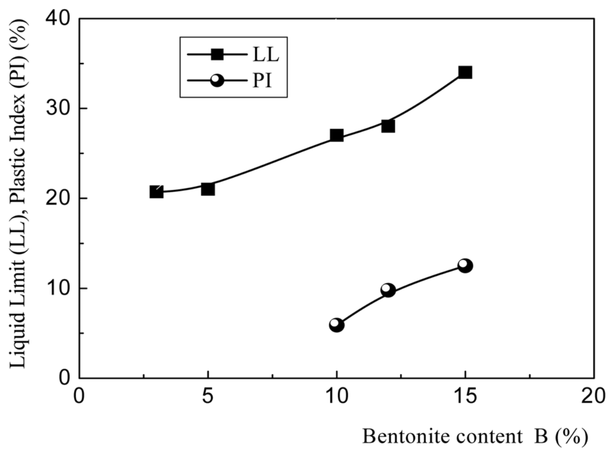

Fig. 3 Relationship between Atterberg limits and bentonite content

Table 2. Physical and mechanical properties of S/B mixtures

%S	%B	γ_d (kN/m^3)	w_{opt} (%)	LL %	PL %	PI %	G %	Pg (kPa)
100	0	19.2	10	–	–	–	–	–
97	3	19.1	10.5	20	–	–	0.85	17
95	5	18.8	11.5	21	–	–	2.22	38
90	10	18.3	12.8	27	21	6	5.90	124
88	12	17.8	14.0	28	18	10	7.30	150
85	15	17	15.2	34	22	12	8.70	178
0	100	12.1	32.0	141	48	93	47.5	852

swelling (G %) is computed using the following relationship

$$G(\%) = \frac{\Delta H}{H} \times 100 \tag{2}$$

$\Delta H = H_f - H_0$
H_0 : initial height (before swelling)
H_f : final height (after swelling)

Evolution of free swelling rates (G %) over time are shown in Fig. 4. The free swell rate of the bentonite is approximately 47.5%, while for S/B mixtures vary between 0.85% and 8.70%. As expected the free swell is proportional with bentonite additions. For swelling pressure test the constant volume method is adopted (Serratrice and Soyez 1996). Results of swell pressure (Pg) of S/B mixtures are indicated in Table 2. The swelling pressures of S/B mixtures increases from 17 to 178 kPa for bentonite content 3 to 15%. When bentonite content addition is more than 10%, swelling pressure becomes greater than 100 kPa.

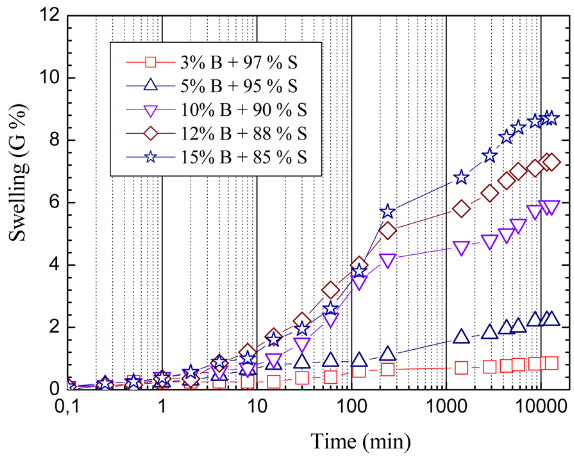

Fig. 4 Swelling evolution of S/B mixtures versus time

3. Measurement of permeability

A compacted layer of sand-bentonite mixture is often used as an insulation barrier (Haug et *al.* 1988, Haug *et al*. 1992, Chapuis *et al.* 1992, Kuroda *et al*. 1993). This two types of soils are clearly dissimilar in their properties such grain size, permeability, chemical activity, and strength, however when they are combined in an adequate ratio, can yield an excellent materiel for insulation barrier to leachate due to its impervious characteristics.

The aim of this study is to highlight the effects of confining stress and the hydraulic gradient on the permeability coefficient with strain_containment. Three measurement techniques have been used to carry out this study: oedometer test, constant hydraulic head test "steady state", and measurement with pulse test "transition state".

3.1 Permeability using odometer test

Indirect methods for evaluating saturated permeability k are based on results of odometer test (Olson and Daniel 1981, Olson 1986). A specimen of 50 mm in diameter and 20 mm height is placed in metal ring and saturated during 24 hours. In this study, the 24 hours incremental loading oedometer test was carried out to investigate the variation of hydraulic conductivity during consolidation, with an incremental loading ratio according to a geometric progression σ_{i+1}'/σ_i' of 2. The pressure applied to the specimen was from 10 kPa to 800 kPa, in total 7 stages.

The hydraulic conductivity k is obtained from both the coefficient of consolidation C_v [m^2/s] and the coefficient of volume compressibility m_v [m^2/kN]. In this method the coefficients C_v and m_v deduced from compressibility curves and consolidation curves respectively (Fig. 5), are used to obtain the permeability.

In this study, k is calculated from both C_v and m_v, but not considering the primary consolidation ratio. The C_v coefficient is evaluated by Taylor's approach.

$$k = C_v \times m_v \times \gamma_w \tag{3}$$

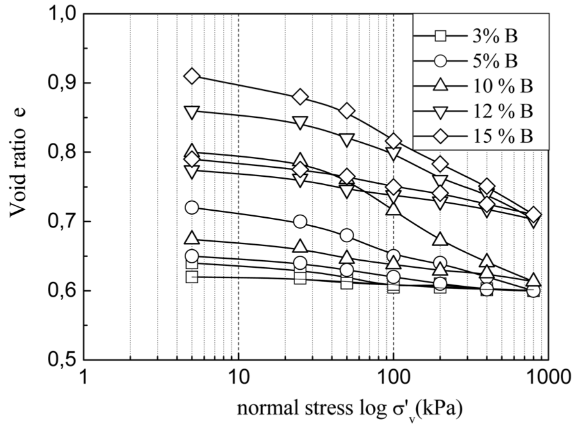

Fig. 5 Compressibility curves of mixtures S/B *e*-logσ' relationships

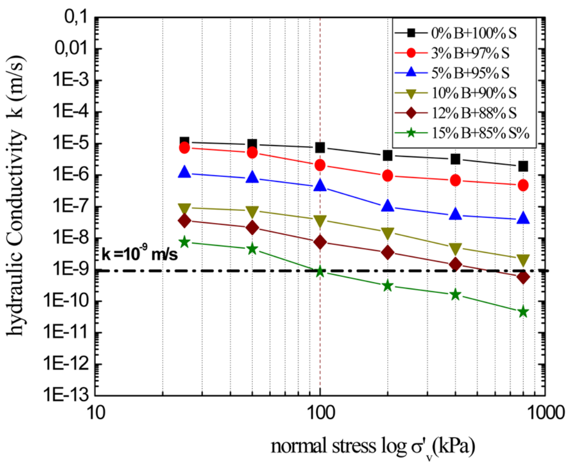

Fig. 6 Saturated hydraulic conductivity vs normal stress of S/B mixtures

Evolution of saturated hydraulic conductivity of S/B mixture as function of loading pressures is shown in Fig. 6.

According to the results obtained it can be noted that:

– For all soils, the permeability varies inversely with the loading pressures. For examples, when the loading pressure varies from 25 to 800 kPa, the permeability for mixtures, with higher bentonite additions, decreases approximately two orders of magnitude.
– The saturated permeability for dune sand varies between 1.1×10^{-5} to 1.9×10^{-6} m/s; whereas for mixtures with 15% bentonite addition the values range from 7.41×10^{-9} to 4.58×10^{-11} m/s.
– The effect of applied loading pressures on permeability is less significant, once the vertical pressure becomes more than 200 kPa. Other researchers found these limiting values around 100 kPa (Wu and Khera 1990) and 200 kPa (Alston *et al*. 1997).
– Permeability of the dune sand-bentonite mixtures decreases with increasing bentonite content. For high bentonite content more than 12%, the saturated permeability is less than 10^{-8} m/s.
– The target values relative to saturated permeability for contaminant liners, which should lay between 10^{-8} and 10^{-10} m/s, can be achieved for percentages of bentonite content greater than 12%, under an applied vertical pressure over than 100 kPa. While for percentage of bentonite addition around 15%, this target values is obtained under a low vertical pressure (around 25 kPa).

According to these results the adequate sand bentonite mixture which will be used for the following studies is 85% S+15% B.

3.2 Measure of permeability using triaxial cell

This section presents the effect of confining stresses and hydraulic gradients on the permeability of compacted mixture (85%S+15%B). The measurement of the saturated permeability is carried out with two different modes. The first is the permanent mode (constant head) and the second is the transitory mode (pulse method).

Fig. 7 Triaxial cell of type Bishop-Wesley revolution controlled with three pressure-volume controllers

3.2.1 Materials and specimens

The saturated permeability is measured using a Perméamétre with flexible walls. The experimental set up is composed of a triaxial cell (Bishop-Wesley revolution) equipped with three pressure-volume controllers (Fig. 7), The set up allows testing specimens of 35 or 50 mm in diameter and variable height to diameter ratio H/D. The triaxial cell permits to apply an isotropic confining stress (up to 1700 kPa). The flow is directed vertically from the bottom towards the top.

Specimens are prepared by static compaction with double piston at Optimum Normal Proctor conditions (w_{opt}=15.2%, $\gamma_{d\ max}$=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 was retained because it permits to obtain more homogeneous specimen (Gueddouda *et al.* 2007).

3.2.2 Phase of saturation

Once the specimen is placed inside the triaxial cell, a confining stress of 100 kPa is applied in the first time. 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 (*ue*: back pressure at the base at the sample, *ue* =20 kPa, *us*: the pore pressure at the top at the sample, *us*=0 kPa). The progressive increase of the confining stress and the backpressure allows to free air bubbles from the connecting tubes (*ue*=30, *ue*=40 kPa) meanwhile preserving an average constant effective stress. The final confining stress applied on the sample during the saturation phase must be greater or equal to the swelling pressure of the mixture, which is around 180 kPa. The vertical deformation of the samples was recorded versus time and the samples were considered as saturated when the displacement of the piston became constant (less than 0.01 mm in 24 h) (Saouli *et al.*2008).

3.2.3 Measurement of permeability in permanent mode (constant head)

The experimental program is carried out using two series-tests. The first one consists to investigate the effect of the average confining effective stress with a constant hydraulic gradient (Δu =constant), whereas the second consists to analyse the effect the hydraulic gradient with a constant average confining effective stress (σ_3'=constant) (Table 3).

Table 3. Loading sequences for permeability measurement in permanent mode

	σ_3 (kPa)	*ue* (kPa)	*us* (kPa)	Δu (kPa) ($ue-us$)	*i*	σ_3' (kPa)
Constant hydraulic gradient	200	40	0	40	57.14	180
	420	240	200	40	57.14	200
	620	240	200	40	57.14	400
	820	240	200	40	57.14	600
	1020	240	200	40	57.14	800
	1220	240	200	40	57.14	1000
	1520	240	200	40	57.14	1300
Constant effective stress	620	240	200	40	57.14	400
	640	280	200	80	114.28	400
	660	320	200	120	171.42	400
	675	350	200	150	214.28	400

With:

$$\sigma_3' = \frac{(\sigma_3 - ue) + (\sigma_3 - us)}{2} \tag{4}$$

The saturated permeability of soils is based on the Darcy law; which describes the relation between water quantity, which flows through a cross section during an elapsed time, and hydraulic gradient. It is expressed by the following equation

$$v = \frac{Q}{A} = -ki \tag{5}$$

v : Darcy flux [m/s]
Q : volumetric flow rate [m^3/s]
A : flow area [m^2]
k : permeability [m/s]
i : hydraulic gradient

$$i = \frac{\Delta u}{\gamma_w H} \tag{6}$$

γ_w : unit weight of water [kN/m^3]
Δu : pressure gradient variation ($u_e - u_s$) [kPa]
H : height of specimen [m]

3.2.4 Measurement of permeability in transitory mode (method of pulsates)

Principle of the method

Because of the short duration of the test compared with the other techniques, this method is very useful to obtain a first estimation of the coefficient of permeability (Brace *et al*. 1968, Gaidi-Sayad 2003).

The permeability values were measured using the pulse method (Brace *et al*. 1968) in a flexible wall permeameter linked to 3 pressure-volume controllers (from GDS Ltd. (Fig. 7)). The principle of the impulse shock test also called "pulse test" consists to subject a soil specimen to an instantaneous differential pressures applied on both ends ($\Delta u_0 = ue - us$). The method consisted in imposing a pressure increase (example: from u_0=200 to u_0=250 kPa) in the lower chamber connected to the base of the sample for 3 min while maintaining the pressure in the upper chamber (u_1=200 kPa) and the confining pressure constant. Then the volume of the solution in the lower chamber was kept constant and its pressure began to decrease, due to the flow entering the specimen. The pulse value (50 kPa) was chosen as the best compromise between the duration of the test and the disturbance of the specimen (Saouli *et al*. 2008). The permeability value was derived from the volume of solution infiltrated under the effect of the hydraulic gradient between the times t and $t+\Delta t$, which is related to the change in pressure during the same period

$$\Delta V_w = C_w \cdot V_w \cdot \Delta u_0 \tag{7}$$

where ΔV_w is the volume of water flowing into the specimen under the pressure gradient, V_w, the initial volume of the lower chamber (m^3), du_0, the pressure change in the lower chamber between t and $t+\Delta t$ (kPa), C_w, the compressibility coefficient of the lower chamber (kPa^{-1}).

In the case of saturated specimen, the coefficient of permeability k can be computed constantly using Darcy law, considering the flow is running out under the average hydraulic head during the pressure dissipation within an elapsed time.

The permeability is obtained using the first part of the dissipation curve (start of the drop) which corresponds to the time interval between 500 to1000 s. The coefficient of permeability is given by the following equation (Saouli *et al.* 2008)

$$k = \frac{\Delta V_w \cdot H}{\Delta t \cdot S\left(\frac{ue - us}{\gamma w} + H\right)} \tag{8}$$

ΔV_W: volume of infiltrated water [m^3]
S : cross section of specimen [m²]
Δt : time interval [s]
H : height of specimen [m]

Table 4. Loading sequences for permeability measurement in transitory mode (Method of pulse test)

	σ_3 (kPa)	ue (kPa)	us (kPa)	Δu_0 (kPa)	σ_3' (kPa)
Initial pressure variation (Δu_0) constant	600	200	200	50	400
	800	200	200	50	600
	1000	200	200	50	800
	1200	200	200	50	1000
	1500	200	200	50	1300
Constant effective stress	600	200	200	30	400
	600	200	200	50	400
	600	200	200	100	400
	600	200	200	150	400

As in the permanent mode, the experimental program of the transitory mode (Table 4) is carried out on two series of tests. The first consists to investigate the effect of the average confining effective stress and the second consists to analyse the effect of the pressure gradient variation Δu on the permeability coefficient (Table 4).

3.3 Experimental results and discussions

3.3.1 Permanent mode (constant head)

• *Average confining effective stress effect*

Fig. 8 show the variations of void ratio and the coefficient of permeability as a function of the average confining effective stress, respectively. It can be noted that the effect of the average confining effective stress on the permeability is more significant for value less than 200 kPa, beyond this stress the permeability seems to be almost constant, which in agreement with oedometer results. The measurement of the change in the volume of water throughout the test, allow deducing the final volume of voids. From the same figure, it can be seen that the void ratio decreases when the average confining effective stress increases and consequently result in a reduction of permeability.

• *Average hydraulic gradient effect*

In the second case of the measurement of the saturated permeability, the average confining effective stress is kipped constant (σ_3'=400 kPa) while varying the hydraulic gradient.

Fig. 9 presents the evolution of velocity of water through the specimen as a function of the hydraulic gradient. The experimental results can be approximated by the following linear relation.

$$v = 6.72 \times 10^{-10}.i \tag{9}$$

Such linear relationships agree with the Darcy law in the range of hydraulic gradient considered.

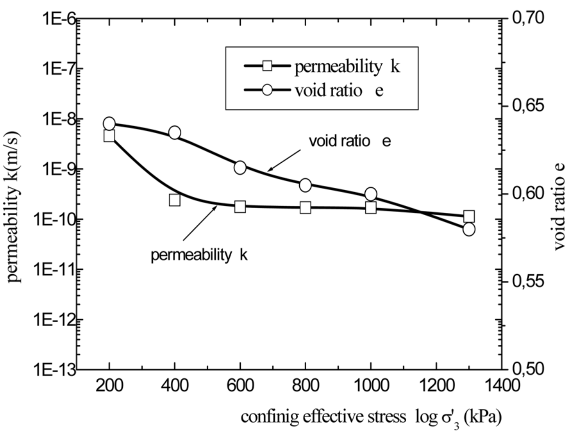

Fig. 8 Permeability and void ratio variations versus the average confining effective stresses (i= 57.14)

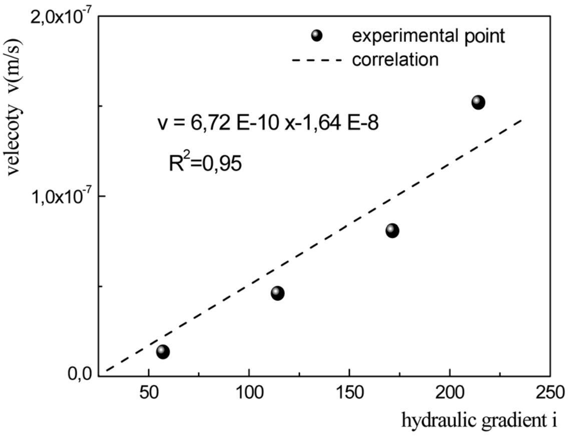

Fig. 9 Velcoty evolution of flow through specimen as a function of the hydraulic gradient, (σ_3'= 400 kPa)

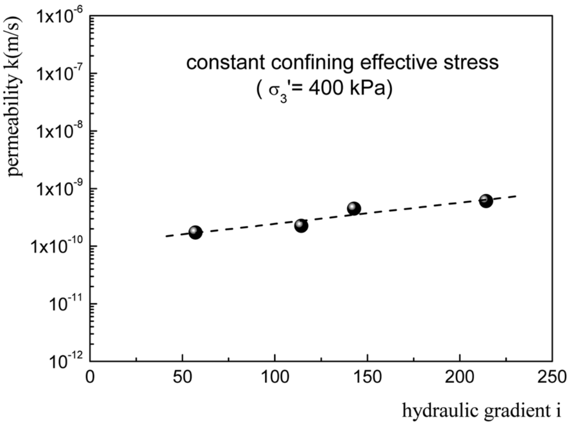

Fig. 10 The permeability results as a function of the hydraulic gradient (σ_3'=400 kPa)

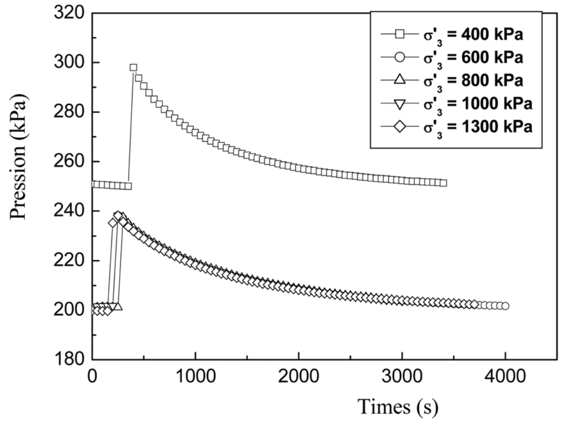

Fig. 11 Pressure dissipation versus time for each average confining effective stress

Similar results have been obtained by other authors (Kenney *et al.* 1992, Gaidi-Sayad 2003). From Fig. 10, it can be noted that there is minor effect of the hydraulic gradient on the permeability results.

3.3.2 Transitory mode (method of pulse test)

• Average Confining stress effect (constant hydraulic gradient)

Fig. 11 shows the pressure dissipation versus time for each average confining effective stress applied (400-1300 kPa). The time of pressure stabilization (*ue*=*us*) where the permeability can be estimated, is situated between 30 and 50 min.

Fig. 12 presents the permeability variation versus average confining effective stress for Δu_0=50 kPa. It can be seen that the permeability which is about 2×10^{-09} m/s seems to be constant with applied average confining effective stresses ($\sigma_3' \geq 400$ kPa). Which are in accord with results

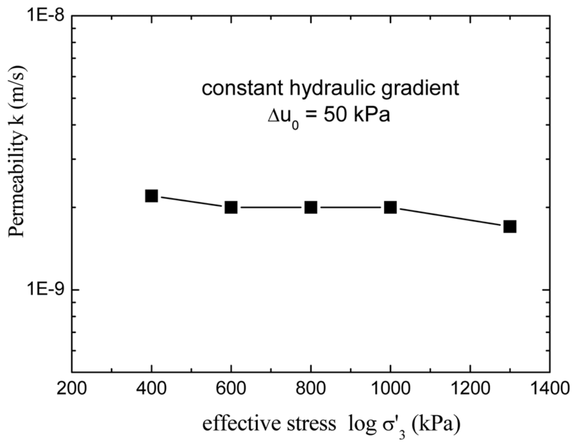

Fig. 12 Permeability variation versus average confining effective stress (Δu_0=50 kPa)

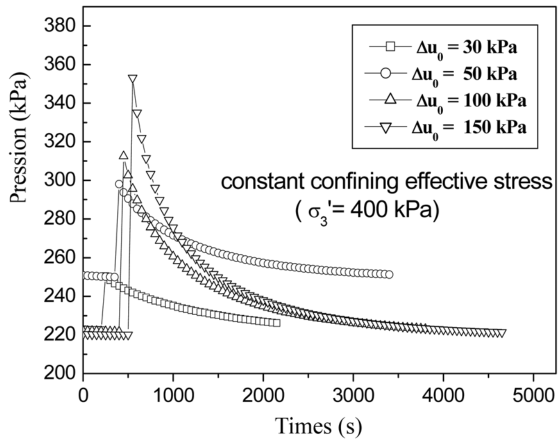

Fig. 13 Pressure dissipation versus time for each variation of initial pressure (Δu_0), σ_3' =400 kPa

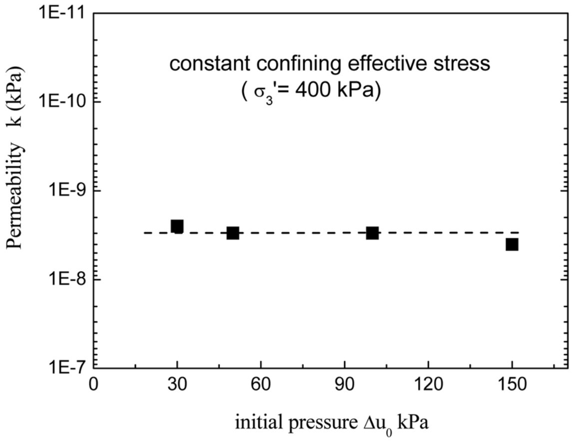

Fig. 14 The permeability variation as a function of initial pressure variation (σ_3'=400 kPa)

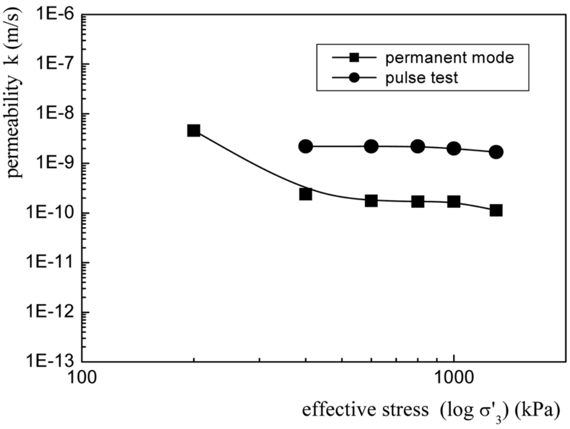

Fig. 15 Permeability variation versus average confining effective stress

obtained by permanent mode.

• *Average effect of initial pressure variation Δu_0 (pulse test)*

Fig. 13 shows the pressure dissipation versus time for each variation of initial pressure (Δu_0) under an average confining effective stress around 400 kPa. It is noted that the time of stabilization of pressure (*ue*=*us*) where the permeability can be estimated in less than one hour.

Fig. 14 presents the permeability variation as a function of initial pressure variation. This coefficient is slightly affected by the value of applied initial pressure. The approximated value is equal to 6×10^{-9} m/s.

The pulse test allows us to get a good estimate of permeability for clayey soil, within an hour compared to the permanent mode which can last for several days.

In Fig. 15 the results of permeability tests using triaxial cell with two modes (permanent mode and transitory mode), are grouped. First we can note that the permeability values obtained in permanent mode are lower than those given by transitory mode. The difference between the two results is around one order of magnitude. This results is according by Youns (2010). The effect of confining effective stress is more clearly for permanent regime.

4. Conclusions

In this research study we have shown that it is possible to obtain an adequate mixture intended for insulation barriers, using dune sand and a small amount of bentonite addition. According to the results obtained from this experimental study, we can advance the following conclusions:

– The common requirement on hydraulic conductivity (should be less than (10^{-8}-10^{-10}) m/s) is met for compacted soil with a minimum of 12% of bentonite addition.
– The relationship between the hydraulic conductivity and the swell of S/B mixture is well illustrated. As expected these two properties are inversely proportional.
– The results of the permeability obtained by the oedometric tests are lower than those obtained

by the triaxial cell, this can be attributed to the lack of saturation of the specimen under oedometric conditions and some air bulbs may remain trapped within the soil.

– Because of the short times of the pulse test compared with the other processes, this test is very useful to obtain an initial estimate of the permeability coefficient.

The permeability decreases with increase of confining stress which due to the reduction of void ratio,

– The results of permeability using two modes (permanent mode and transitory mode) under confining stress conditions are almost similar the difference is around one order.
– According to the results of permeability obtained by different methods, the adequate mixture proposed for the design of the worked barriers in the arid region (southern of Algeria) is 85% S+15% B. In addition such mixture presents a moderate swelling and shrinkage potentials and hence it will be less subjected to cracks under drying conditions.
– Finally, it can be stated that the use of dune sand, which is a local largely available material in the south of Algeria, with small quantities of bentonite additions can provide an economical insulation barrier for waste disposal management.

The procedure on site for mixing and placing of sand-bentonite mixture along with strict control of compaction procedures play an extremely important role for the final quality of the barrier (Haug and Wong 1992).

References

ADEME (1999), "Les installations de stockage de déchets ménagers et assimilés", Techniques et Recommandations, ADEME Editions, Paris.

Aina, M.P. (2006), "Expertises des centres d'enfouissement techniques de déchets urbains dans les PED: contributions a l'élaboration d'un guide méthodologique et a sa validation expérimentale sur sites", Thèse de Doctorat, Université de Limoges, France.

Alston, C., Daniel, D.E. and Devroy, D.J. (1997), "Design and construction of sand Bentonite liner for effluent treatment lagoon, Marathon, Ontario", *Can. Geotech. J.*, **34**, 841-852.

Blatz, J.A., Graham, J. and Chandler, N.A. (2002), "Influence of suction on the strength and stiffness of compacted sand-bentonite", *Can. Geotech. J.*, **39**, 1005-1015.

Brace, W.F., Walsh, J.B. and Frangos, W.T. (1968), "Permeability of granite under high pressure", *J. Geol. Res.*, **73**, 2225-2236.

Chalermyanont, T. and Arrykul, S. (2005), "Compacted sand-bentonite mixtures for hydraulic containment liners", *Songklanakarin J. Sci. Technol.*, **27**(2), 313-323.

Chapuis, R.P. (1990), "Sand-bentonite liners: predicting permeability from laboratory tests", *Can. Geotech. J.*, **27**, 47-57.

Chapuis, R.P., Lavoie, J. and Girard, D. (1992), "Design, construction, and repair of the soil-bentonite liners of two lagoons", *Can. Geotec. J.*, **29**, 638-649.

Cui, Y.J., Tang, A.M., Loiseau, C. and Delage, P. (2008), "Determining water permeability of compacted bentonite-sand mixture under confined and free-swell conditions", *Phys. Chem. Earth,* **33**, S462-S471.

D'Appolonia, D.J. (1980), "Soil-bentonite slurry trench cutoffs", *J. Geotech. Eng. Div. - ASCE*, **106**(4), 399-418.

Daniel, D.E. (1984), "Predicting hydraulic conductivity of clay liners", *J. Geotech. Eng.* - *ASCE*, **110**, 285-300.

Gaidi-Sayad Chahira (2003), "Ecoulement dans les milieux poreux peu perméables saturés et non saturés", Thèse de Docteur, France.

Gillham, R.W., Robin, M.J.L., Dytynyshyn, D.J. and Johnston, H.M. (1984), "Diffusion of non reactive and reactive solutes trough fine-grained barrier materials", *Can. Geotech. J.*, **21**, 541-550.

Gleason, M.H., Daniel, D.E. and Eykholt, G.R. (1997), "Calcium and sodium bentonite for hydraulic containment applications", *J. Geotech. Geoenviron. Eng. - ASCE,* **123**(5), 438-445.

Gueddouda, M.K., Lamara, M. and Goual, I. (2007), "Caractérisation et stabilisation des sols expansifs effet de l'association (sel + sable de dune sur les paramètres de gonflement", *Sols et Matériaux à Problèmes*, Coll. Int. Tunis, 11 & 14 février 2007, 27-43.

Guide des techniciens communaux pour la gestion des déchets ménagers et assimiles, Ministère de l'Aménagement du Territoire, de l'Environnement et du Tourisme MATET Algérien, 2009.

Harrop-Williams, K. (1985), "Clay liner permeability: evaluation and variation", *J. Geotech. Eng. - ASCE*, **111**, 1211-1225.

Haug, M.D., Barbour, S.L. and Longval, P. (1988), "Design and construction of a prehydrated sand-bentonite liner to contain brine", *Can. J. Civil Eng.*, **15**, 955-963.

Haug, M.D. and Wong, L.C. (1992), "Impact of molding water content on hydraulic conductivity of compacted sand-bentonite", *Can. Geotech. J.*, **29**, 253-262.

Holtz, W.G. (1985), "Predicting hydraulic conductivity of clay liners: Discussion", *J. Geotech. Eng. - ASCE*, **111**, 1457-1459.

Kenney, T.C., Veen, W.A. van, Sallow, M.A. and Sungaila, M.A. (1992), "Hydraulic conductivity of compacted bentonite-sand mixtures", *Can. Geotech. J.*, **29**, 364-374.

Kuroda, T., Katahira, F., Sasaki, T. and Imamura, S. (1993), "Outline of Rokkasho low-level radioactive waste disposal center and characteristic of bentonite/sand mixtures", *Civil Engineering in Japan '93*, Japan Society of Civil Engineering, 39-93.

Lauffer, H. and Schober, W. (1961), "Investigation for the earth core of the Gapatsch rockfill dam with a height of 150 m", *Proceedings of the 7th International Conference on Large Dams*, Rome, **4**, Q27 R92.

Lopez, R.S., Cheung, C.H. and Dixon, D.A. (1984), "The Canadian program for sealing underground nuclear fuel waste vaults", *Can. Geotech. J.*, **21**, 593-596.

Mitchell, J.K. (1993), *Fundamentals of soil behaviour*, 2^{ed}, John Wiley & Sons, New York.

Montañez, J.E.C. (2002), "Suction and volume changes of compacted sand-bentonite mixtures", Thèse de Doctorat, University of London.

Olson, R.E. and Daniel, D.E. (1981), "Measurement of the hydraulic conductivity of fine grained soils", Permeability and Groundwater Contaminant Transport - ASTM, STP **746**, 18-64.

Nishigaki, M., Sun, Y. and Kono, I. (1994), "A study of deformation and failure of compacted sand/bentonite mixture soil foundation by model test and numerical analysis", *Proceedings of the International Symposium on Pre-failure Deformation of Geomaterials*, IS-Hokkaido, Japan, **1**, 523-529.

Olson, R.E. (1986), "State of art: Consolidation theory, consolidation of soils: testing and evaluation" by R. N. Yang and F.C. Townsend (Eds), ASTM, STP **892**, 7-70.

Parker, R.J., Bateman, S. and Williams, D. (1993), "Design and management of landfills", *Geotechnical management of waste and contamination*, Fell, Phillips & Gerard (eds), Balkema, Rotterdam, 209-252.

Reschke, A.E. and Haug, M.D. (1991), "Physico-chemical properties of bentonites and the performance of sand-bentonite mixtures", *Proc. 44th Canadian Geotechnical Conf.*, 62r1-62r10.

Sallfors, G. and Oberg-Hogsta, A.L. (2002), "Determination of hydraulic conductivity of sand-bentonite mixtures for engineering purposes", *Geotech. Geo. Eng.,* **20**, 65-80.

Schnitter, G. and Zeller, J. (1961), "Geotechnical investigations of mixtures of bitumen, clay or bentonite with sandy gravel", *Proceedings of the 7th International Conference on Large Dams*, Rome, **4**, Q27 R38.

Serratrice, J.F. and Soyez, B. (1996), "Les essais de gonflements", *Bulletin des laboratoires des ponts et chaussées*, Juillet-Août 1996, 204., réf .4082, 65-85.

Skempton, A.W. (1953), "The collaidal activity of clays", *Proceedings of the Third International Conferences on Soil Mechanics and Foundations Engineering*, **1**, 57-61.

Saouli, H., Fleureau, J.M. and Trabelsi, A.M.B. (2008), "Hysicochemical analysis of permeability changes in the presence of zinc", *Elsevier, Geoderma*, 1-7.

Thériault Pascal (2000), "*Etude de l'influence des métaux lourds sur la conductivité hydraulique de couches sable/bentonite*", Mémoire pour l'obtention du grade de maître ès sciences (M.Sc.), Canada, Mai 2000, 119.

Wu, J.Y. and Khera, R.P. (1990), "Properties of a treated-bentonite/sand mix in contaminant environment. physic-chemical aspects of soil and related materials", *ASTM*, STP **1095**, K. B. Hoddinott and R.O. Lamb, Eds., American Society for Testing and Materials, 47-59.

CC