Lateritic Soil and Bentonite Mixtures Assessment for Liner Usage Purpose

Magali Kenya Farnezi, Adilson do Lago Leite

Abstract. Among all the constructive elements in waste containment systems, base liner may be the one of greatest concern. In Brazil, the use of compacted lateritic soils for this purpose is widely accepted due to their wide occurrence. While they usually achieve compaction and strength requirements, sometimes they may need some improvement in terms of hydraulic conductivity and reactivity to contaminants. This paper describes a laboratory assessment of two mixtures of a lateritic soil and bentonite for liner usage purpose. Laboratory tests included geotechnical, mineralogical and physicochemical characterization, hydraulic conductivity determination in compacted samples and modified Atterberg Limits and grain-size distribution analysis. These two last tests were intended to evaluate the compatibility between the soil and different chemical solutions. The characterization tests showed that the clay content of the mixtures decreased and the plasticity and activity increased relative to the natural soil sample. The hydraulic conductivity of the compacted mixtures decreased two to four times when compared to the compacted natural soil. The results also have demonstrated that the addition of bentonite in the proportions used in the tests increased the compatibility of the natural soil.

Key words: hydraulic conductivity, compatibility, lateritic soil, liner.

1. Introduction

Liner systems play a major role in waste containment facilities such as sanitary and industrial landfills and waste ponds. They are intended to reduce infiltration and attenuate the effect of contaminant solutions by means of sorption, filtration, redox reactions and other processes. Compacted clayey liners (CCL) are widely accepted as one of the main components of these systems (Daniel & Koerner, 1995; Rowe *et al.*, 1995; Bouazza & Van Impe, 1998; Rowe, 2001), and are routinely applied along with geosynthetics and other materials.

Lateritic soils are widely used for liner construction in Brazil and other tropical countries. However, while they usually achieve compaction and strength requirements, sometimes they may not attend the required low hydraulic conductivity and high sorption parameters of CCL's (cation exchange capacity-CEC, for instance). Their peculiar mineral constitution, mainly quartz, kaolinite and Fe-Al oxides/hydroxides, is the major responsible for that.

In this way, because of its intense swelling and CEC properties, bentonite seems to be a good alternative, at least theoretically, as an addictive material to improve lateritic soils for liner usage purpose. The swelling property of bentonite when hydrated would seal the pore space between grains, reducing the hydraulic conductivity. Additionally, an increase in the CEC of the soil would be also achieved.

As pointed out by Schackelford (1994), besides the mechanical and the contaminant transport requirements, other important issue that must be addressed for choosing suitable CCL materials is the clay/waste interaction, which is called liner compatibility. A good compatibility means that no significant changes will take place when the liner material is exposed to waste solutions (organic or inorganic).

An extensive literature review has revealed that all the issues aforementioned have been individually addressed in many papers. For instance, Boscov *et al.* (1999a,b), Leite & Paraguassú (1999a,b), Leite & Paraguassú (2002) and Leite *et al.* (2003) researched on the contaminant transport through lateritic soils. Other papers such as Matos *et al.* (1999 and 2000), Fontes *et al.* (2000), Sarkar *et al.* (2000), Gomes *et al.* (2001), Goldberg *et al.* (2001), Saha *et al.* (2001 and 2002) and Grubb & Hemsi (2002) deal exclusively with heavy metal mobility in lateritic soils and pure oxides-hydroxides samples. In turn, the research by Bowders (1985), Bowders & Daniel (1987) and Foreman & Daniel (1984) investigate the interactions of different contaminant solutions with distinct soil samples and the geotechnical response to those interactions. Finally, the papers by Anderson & Hee (1995) and Osinubi & Nwaiwu (2002) deal with geotechnical issues of mixtures of lateritic soils and bentonite for liner usage purpose.

The aforementioned review has showed no investigations on hydraulic conductivity and compatibility issues together, and this lack is even accentuated when mixtures of lateritic soils and bentonite are included. Shackelford (1994) stressed the importance of the compatibility investigation for liner construction.

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This paper describes a combined investigation on the hydraulic conductivity and compatibility of a lateritic soil and its mixtures with two different proportions of bentonite (3 and 6%, dry weight basis). The hydraulic conductivity was assessed by percolating water in compacted samples using rigid wall permeameters and the compatibility was investigated using modified Atterbeg limits and grain size analysis, both tests using water and different chemical solutions.

2. Material and Methods

2.1. Sampling and sample preparation

Disturbed samples were extracted from a site near the city of Ouro Preto, state of Minas Gerais, Brazil. In place, it is a very clayey dark red oxisoil, which means that advanced laterization processes took place in its formation.

For the soil properties determination and compatibility tests, the samples were prepared according to the standard ABNT-NBR6457 (1986) and Nogueira (1995), which basically consists of air drying at room temperature, homogenization, sample reduction and sieving. These samples are called here as SN.

The bentonite sample (BK), in turn, is commercially available. According to the vendor, it came from the city of Boa Vista, Paraíba, Brazil. Santos (1989) and Rodrigues *et al.* (2004) mentioned that most of the bentonite from this city is treated with concentrated sodium chloride solutions in order to improve their swelling characteristics. More properties of this sample can be found in the item Results and Discussion - Sample Properties.

The proportions of soil and bentonite used in the mixtures (M3 and M6 samples) were chosen based on the values from Anderson & Hee (1995). They are as follows (dry weight basis):

• M3 Sample = 97% soil (SN sample) + 3% bentonite (BK sample)

• M6 Sample = 94% soil (SN sample) + 6% bentonite (BK sample)

The samples used in the hydraulic conductivity tests were dynamically compacted under Normal Proctor energy, at moisture contents from 2 to 4% wet of optimum. After compaction, cylindrical samples were molded with dimensions of 5 cm in diameter and 12 cm height.

2.2. Sample properties determination

The references used to determine the sample properties are presented in Table 1. Most of these properties were determined by conventional and standardized tests usually applied in geotechnical engineering. Some of them, however, are not routinely found in geotechnical laboratories or are more related to geology and pedology. These last tests deserve more details, as given in the paragraphs below.

Free swell tests, as given by Acar & Olivieri (1989), consists of pouring 10 cm^3 of dry soil into a 100 cm^3 graduated cylinder, which is after filled with destilled-deionized water. After a 24-hour period of rest, the free swelling index (S) can be obtained according to Eq. (1).

$$S(\%) = \frac{V_f}{V_i} \times 100 \quad (1)$$

where V_i is the initial volume of the sediment column inside the graduated cylinder and V_f is its final volume after swelling.

The pH and electrical conductivity (EC) were determined based on Camargo *et al.* (1986). The pH was directly measured in a suspension of 1:2.5 (soil:solution ratio). EC was measured in the water extract obtained from a 1:1 (soil:solution) suspension.

The cation exchange capacity (CEC) and the specific surface (SS) can be estimated from the blue dye adsorption test (filter paper spot test). According to Pejon (1992), the methylene blue ($C_{16}H_{18}N_3SCl\text{-}3H_2O$) is a strong dye which adsorbs on the mineral surface via cation exchange. The amount of adsorbed blue dye is directly proportional to the CEC and SS of the soil sample when the adsorption saturation is spotted on a filter paper. This test was executed in the soil sample (2 to 4 g, dry weight) that passed through the 10 mm sieve.

The mineral composition of the fine fraction (passing in sieve 0,075 mm) of SN and BK samples was estimated by X-ray diffraction techniques. For that, oriented thin glass sections were elaborated by sedimentation. In turn, two of these thin sections were saturated in ethylene-glycol gas and in KCl solution. The X-ray conditions were: cupper

Table 1 - References used in sample characterization.

Test	Reference
Physical characterization	
- Grain-size analysis	ABNT-NBR7181(1984)
- Liquid limit - ω_L	ABNT-NBR6459 (1984)
- Plastic limit - ω_P	ABNT-NBR7180 (1984)
- Activity - A	Skempton (1953), cited in Michell (1993)
- Specific gravity of solids - G_s	ABNT-NBR6508 (1984)
- Compaction (normal proctor)	ABNT-NBR7182 (1986)
- Optimum moisture content - w_{ot}	
- Maximum dry unit weight - ρ_{dmax}	
- Free swelling test - S	Acar & Olivieri (1989)
Physicochemical characterization	
- pH determination (H_2O)	Camargo *et al.* (1986)
- pH determination (KCl)	Idem
- Electrical Conductivity	Idem
- Blue metilene adsorption test	Pejon (1992)
- Cation exchange capacity	
- Specific Surface	
Mineralogical evaluation	
- X-ray diffraction	Brown & Brindley (1984)

tube, velocity of 1 to 2° per second and angles from 2 to 70°.

2.3. Hydraulic conductivity tests

The hydraulic conductivities (k) of SN, M3 and M6 samples were estimated according to the standard ABNT - NBR14545 (2000). The BK sample was not tested because of operational reasons, since bentonite is well known to be difficult to percolate. Anyway, k values of less than 10^{-12} ms^{-1} were expected for this sample when compacted.

Figure 1 depicts the permeability cell used for the tests. The main details of this cell are as follows:

• Course grained sand and filter paper (high drainage) were used at the top and base of the soil sample as protection filters;

• Use of wet bentonite (BK sample) to seal the void between the compacted soil sample and the permeability cell wall so as to reduce the "wall effect";

• Use of paraffin at the top and base of the bentonite seal, surrounding the soil sample, in order to prevent the contact between the bentonite and the sand layer.

Test procedures basically involve soil sample saturation and percolation at variable head conditions.

The saturation process was performed by connecting the water tube to the base water port of the permeability cell (see Fig. 1). In this way water would percolate from the bottom to the top of soil sample, making it easier to eliminate the air bubbles trapped in the soil. The air vent (see Fig. 1) was used throughout the saturation time. Full saturation was considered when water popped up at the top water port. This process usually lasted between a month and half.

It is believed that no "wall effect" or other preferential flow way took place at the permeability cell because of the sealing capacity provided by the swelled bentonite and paraffin together (see Fig. 1). Anyway, the use of flexible wall permeameter is suggested in the future. The referred preferential ways are not expected in this type of test. Anderson & Hee (1995) performed several tests on a flexible wall permeameter under confining pressures of 10, 50 and 100 kPa and they found reasonable results.

After saturation, percolation and water level measures (falling head condittions) usually lasted for a week. The hydraulic gradients used in the tests are presented in Table 2, where it can be seen that the maximum gradient did not reach more than 1.84. These low gradient values were intended to prevent any sample disturbance and consequent leaks. A total of 7 hydraulic conductivity tests were performed, 3 for SN sample, 2 for M6 sample and 2 for M3 sample.

2.4. Compatibility tests

Schakelford (1994) mentioned two types of laboratory tests for estimating the compatibility between soil samples and different chemical solutions: (1) modified index properties and (2) percolation tests.

Type-1 test gives only a qualitative indication of possible changes on soil behavior when it is exposed to different chemical solutions. The compatibility is evaluated by comparing the results from conventional tests using water with the results from the tests in which other fluids are applied. This type of tests were applied by Bowders (1985), Bowders & Daniel (1987), Acar & Olivieri (1989) and others.

It is introduced here the so called Plastic Incompatibility Index (IC_p), defined in Eq. (2). This parameter directly indicates the incompatibility of the soil to different

Table 2 - Hydraulic gradients used in tests.

Sample	Minimum value	Maximum value	Average
SN	0.42	1.84	1.14
M3	0.20	0.63	0.39
M6	0.01	0.64	0.16

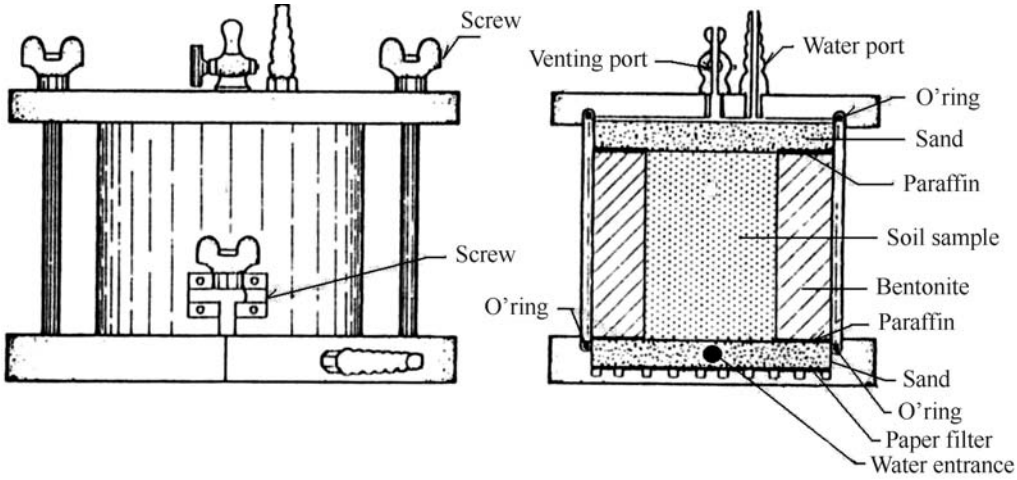

Figure 1 - Schematic of the permeability cell used in the hydraulic conductivity tests.

fluids in terms of plasticity. From Eq. (2) it can be inferred that the higher the IC_p value, the higher the incompatibility of the soil with the percolating fluid compared with water. Additionally, positive values of IC_p indicate a reduction in the plasticity of sample with the fluid in relation to water and vice-versa.

$$IC_p(\%) = \frac{I_{pw} - I_{pf}}{I_{pw}} \times 100 \quad (2)$$

where I_{pw} means the plasticity index with water and I_{pf} the plasticity index with the analyzed fluid.

Type-2 test directly measure the changes in hydraulic conductivity induced by percolating the chemical solutions through the soil. By comparing the k value obtained with percolation of water (k_w) and other fluids (k_f), the compatibility is directly evaluated using the relative hydraulic conductivity index (k_R), as can be seen in Eq. (3). This type of test was used by Fernandez & Quigley (1985), Foreman & Daniel (1986), Bowders & Daniel (1987), Fernandes & Quigley (1988), Acar & Olivieri (1989), Budhu *et al.* (1991) and others.

$$K_R = \frac{K_f}{K_w} \quad (3)$$

In the present study, type-1 tests were performed through modified Atterberg limits and grain-size distribution analysis.

The modified Atterberg limits, plastic limit (ω_p) and liquid limit (ω_L), were determined with water and six different solutions: Sodium Chloride-NaCl (1000 mgL^{-1}); Sodium Hydroxide-NaOH (pH = 9 and pH = 11); Nitric Acid-HNO_3 (pH = 3 and pH = 5) and pure Toluene. Exception made to Toluene, the same solutions were applied to modified grain-size distribution analysis.

These different chemical solutions have been chosen considering four specific and extreme environments: acidic, alkaline, salty and pure organic with low dielectric constant. These conditions would represent some of the most "aggressive" environments that liners could be exposed in waste containment facilities, exception made to radioactive and high temperature conditions.

Procedures for Atterberg Limits determination followed the standards ABNT-NBR6459 (1984) and ABNT-NBR7180 (1984). In modified grain-size distribution analysis the hydrometer test was performed in three ways: (1) using a solution of water+defloculant (Sodium Hexametaphosfate), a conventional procedure that is indicated in the standard ABNT-NBR 7181 (1984); (2) using only water and (3) using the chemical solutions aforementioned, except the Toluene. Procedure (2) was intended to better evaluate the effect of the defloculant solution, sodium hexametafosfate-45,7 gL^{-1} + sodium carbonate-pH 8 to 9 (ABNT-NBR7181, 1984). Procedure (3), in turn, intended to assess the effect of the chemical solutions in the grain-size fractions.

3. Results and Discussions

3.1. Sample properties

Table 3 presents the geotechnical properties of the samples. It must be emphasized that the grain-size distribution presented in this table was determined using method (1) aforementioned.

It can be seen from Table 3 that the clay fractions in M3 and M6 samples are smaller than in SN sample. This result was not expected since bentonite (BK samples) was mixed with M3 and M6 samples, which theoretically would lead to an increase in clay fraction of the sample. On the other hand, the silt and fine sand fractions in M3 and M6 samples are bigger than in the SN sample. These facts conduct to the conclusion that the clay provided by the bentonite addition aggregates the SN grains, leading to bigger silt and sand fractions.

As it was expected, ω_p, ω_L and activity of M3 and M6 samples considerably increased compared to the SN sample. This is due to the higher specific surface of bentonite, which hold more water and increased the plasticity of the SN sample. In terms of compaction, w_{ot} slightly increased for the M3 and M6 samples compared to the SN sample. On the other hand, the maximum dry unit weight decreased with bentonite addition.

It is interesting to observe that M3 and M6 samples are classified as CH (high plasticity inorganic clays), according to Unified Classification System. In turn, SN sample is CL (low plasticity inorganic clays), which also shows the effect of bentonite addition in SN sample.

Table 4 depicts the grain-size fractions obtained using procedure-(2) test, as described in Materials and Methods item.

From Tables 3 and 4 it can be seen that all the fractions but clay and silt (M6 sample) have increased and that the coarse sand fraction has experimented the biggest increase. This increase was expected since there was no deflocculant solution. Therefore it is concluded that this solution was effective in dispersing grains in hydrometer test.

Table 5 presents some of the physicochemical properties of the samples. It is observed that:(1) the SN sample was acidic in water with positive ΔpH. This was expected because of the high laterization of this sample; (2) the BK sample was quite alkaline in water and KCl and this fact increased the pH of M3 and M6 samples; (3) ΔpH was negative for M3, M6 and BK samples as was expected since bentonite have predominantly permanent and negative surface charge; (4) the EC of BK sample was quite high, which demonstrates the high salinity of this sample; (5) the differences between CEC and SS values between SN and M3-M6 samples are significant, which shows the expressive contribution of the BK sample in these mixtures.

Table 3 - Sample properties.

Property		Sample			
		SN	BK	M3	M6
Grain-size distribution[1]					
Clay (ϕ < 0,002 mm)	(%)	44	85	30	29
Silt (0,002 < ϕ < 0,06 mm)	(%)	26	11	34	39
Fine sand (0,06 < ϕ < 0,2 mm)	(%)	16	3	22	19
Median sand (0,2 < ϕ < 0,6 mm)	(%)	12	1	11	9
Coarse sand (0,6 < ϕ < 2 mm)	(%)	2	0	2	3
Gravel (ϕ > 2 mm)	(%)	0	0	1	1
Liquid limit - ω_L	(%)	32.30	494.12	55.20	58.80
Plastic limit - ω_p	(%)	20.05	93.84	26.15	26.69
Plastic index - I_p	(%)	12.25	400.28	29.05	32.11
Activity - A		0.28	4.71	0.97	1.11
Specific gravity of solids (G_s)		3.166	3.075	3.047	2.834
Optimum moisture content - w_{ot}	(%)	20.5	NR[2]	23.7	22.6
Maximum dry unit weight - ρ_{dmax}	(g/cm^3)	1.85	NR	1.68	1.62
Free swelling index - S	(%)	NR	162	NR	NR
Unified classification (SUCS)		CL	CH	CH	CH

[1]Grain-size scale of the Massachusetts Institute of Technology - MIT. [2]No results.

Table 4 - Grain-size fractions of the samples without using a deflocculant solution.

Sample	Fraction (%)					
	Clay	Silt	Fine sand	Medium sand	Coarse sand	Gravel
SN	10	38	26	18	6	2
M3	8	40	27	17	6	2
M6	11	35	25	19	8	2
BK	61	32	7	0	0	0

Concluding, X-ray diffraction results have showed the following composition for the SN sample: kaolinite (dominant), gibbsite, goetite and hematite. This composition is typical for lateritic soils. In turn, the BK sample showed predominantly smectite, some kaolinite, quartz and mica.

3.2. Hydraulic conductivity tests

The hydraulic conductivity tests for SN, M3 and M6 samples (Table 6) shows that k values ranged from the order of 10^{-9} to 10^{-10} ms^{-1}.

The effect of bentonite addition is evidenced by the reduction in the average k-value of M3 and M6 samples in respect to the SN sample. This reduction was more expressive for M6 sample in comparison to M3 sample. In fact, the reduction in the average k value of M3 sample compared to SN sample is not so significant, considering all the possible experimental variability.

3.3. Compatibility tests

The results of modified Atterberg limits are presented in Tables 7 and 8, including statistical parameters and the incompatibility index (IC_p) values (see Eq. (2)).

From Tables 7 and 8 it can be seen that the plasticity index coefficient of variation followed this order:

Table 5 - Physicochemical properties of the samples.

Property	Sample			
	SN	BK	M3	M6
pH (H_20) (1: 2.5 soil:solution ratio)	5.09	10.43	6.94	8.11
pH (KCl) (1: 2.5 soil:solution ratio)	5.39	9.40	5.66	3.23
ΔpH (pH KCl - pH H_2O)	+0.30	-1.03	-1.28	-4.88
Electrical conductivity - EC (mScm^{-1})	0.08	2.37	0.85	1.42
Cation exchance capacity - CEC (cmol kg^{-1})	0.85	53.13	3.3	7.06
Specific surface - SS (m^{2}g^{-1})	6.64	414.88	25.73	55.08

SN ≈ M3 > M6 > BK, which indicates greater homogeneity for M6 and BK samples. It is considered that more variability of data indicates more incompatibility of the samples, thus in this case M6 and BK samples are more compatible than SN and M3 in terms of plasticity.

Table 6 - Hydraulic conductivity results.

Test	k (ms^{-1})		
	SN sample	M3 sample	M6 sample
1	5.06×10^{-9}	1.48×10^{-9}	4.23×10^{-10}
2	1.30×10^{-9}	1.30×10^{-9}	7.49×10^{-10}
3	3.20×10^{-9}	—	—
Average	3.18×10^{-9}	1.39×10^{-9}	5.86×10^{-10}

The following observations can be deduced from the IC_p values exhibited on Tables 7 and 8: (1) the plasticity of all samples decreased with fluids other than water, exception made to the tests with M3 sample (HNO_3, pH5) and BK sample (NaCl, HNO_3, pH3; NaOH, pH9); (2) the biggest IC_p values were registered for samples SN (NaCl) and M3 (NaOH, pH9) and the smallest for samples BK (HNO_3, pH5) and M3 (NaOH, pH11); (3) an overall observation clearly indicates smaller IC_p values for M6 and BK samples, which leads to the conclusion that these samples are more compatible with the fluids being tested than SN and M3 samples.

It is worth to point out that when toluene was used samples did not offer any plasticity. Similar results where reported by Acar & Olivieri (1989), who observed a significant decrease in the plasticity of montmorilonitic clays

Table 7 - Modified Atterberg Limits results for SN and BK samples.

Fluid	Atterberg limits							
	SN sample				BK sample			
	ω_L (%)	ω_p (%)	I_p (%)	ICp (%)	ω_L (%)	ω_p (%)	I_p (%)	ICp (%)
Water	32.3	20.1	12.3	-	264.1	71.7	192.4	-
NaCl	30.5	25.8	4.7	61.6	270.2	65.0	205.2	-6.7
HNO_3 - pH 3	34.2	24.7	9.5	22.4	266.7	50.7	216.0	-12.3
HNO_3 - pH 5	34.2	26.6	7.5	38.5	247.2	56.0	191.2	0.6
NaOH - pH 9	33.9	27.1	6.8	44.5	273.7	64.1	209.6	-8.9
NaOH - pH 11	32.3	25.5	6.8	44.5	250.6	62.5	188.1	2.2
Toluene	NP	NP	NP	-	NP	NP	NP	-
Average	32.9	25.0	7.9	-	262.1	61.7	200.4	-
Standard deviation	1.34	2.3	2.4	-	9.8	6.7	10.4	-
Variation coefficient (%)	4.08	9.34	30.14	-	3.75	10.88	5.19	-

NP - Non plastic.

Table 8 - Modified Atterberg Limits results for M3 and M6 samples.

Fluid	Atterberg limits							
	M3 sample				M6 sample			
	ω_L (%)	ω_p (%)	I_p (%)	ICp (%)	ω_L (%)	ω_p (%)	I_p (%)	ICp (%)
Water	54.2	26.6	27.6	-	60.6	29.1	31.5	-
NaCl	46.1	30.1	16.0	42.0	59.7	35.7	24.0	23.8
HNO_3 - pH 3	48.8	24.3	24.5	11.3	51.2	23.6	27.6	12.3
HNO3 - pH 5	57.5	25.9	31.6	-14.6	50.0	28.0	22.0	30.1
NaOH - pH 9	40.3	29.0	11.3	59.1	55.9	29.6	26.3	16.5
NaOH - pH 11	56.7	29.6	27.1	1.8	51.4	22.5	28.9	8.3
Toluene	NP	NP	NP	-	NP	NP	NP	-
Average	50.6	27.6	23.0	-	54.8	28.1	26.7	-
Standard deviation	6.2	2.1	7.1	-	4.2	4.3	3.1	-
Variation coefficient (%)	12.17	7.67	30.74	-	7.68	15.45	11.65	-

NP - Non plastic.

when in contact with organic fluids of low dieletric constant relative to water. Toluene, in turn, presents a dieletric constant of 2.4.

Bar diagrams were made using the data from Tables 7 and 8 in order to identify possible trends, as shown in Fig. 2. It is observed that I_p decreases for high pH values in the case of SN sample (Fig. 2a). This trend was not observed for M3 and M6 samples, where ω_L, ω_p and I_p oscillate with the pH increasing (Figs. 2b and 2c). Figure 2d clearly shows that BK sample offers the smallest variation in Atterberg limits relative to the other samples. At the same time, a very small increase in ω_p with increasing pH is observed for this last sample.

It is expected that in the field scale the variation in the plasticity indicated on Fig. 2 can lead to modifications on the hydraulic and strength performance of the base liner under the influence of the disposed fluid. This variation is probably associated with flocculation and/or dispersion of the soil grains and peds.

Figures 3 to 6 present bar diagrams for the grain-size fractions (%) obtained using different solutions in the modified hydrometer tests.

It is clear from all these figures that the clay fraction significantly decreases in the absence of the deflocculant solution for all samples. At the same time, the silt and sand fractions increase, which indicates that the flocculated soil (peds) are included in these specific fractions.

The clay fraction was absent in SN, M3 and M6 samples when the NaCl salt was used. When comparing this re-

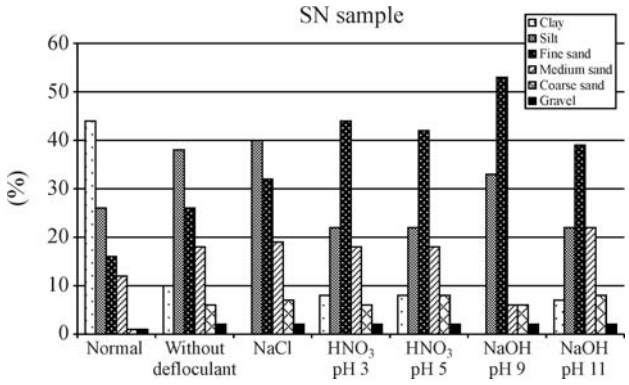

Figure 3 - Grain-size fractions of SN sample exposed to different fluids.

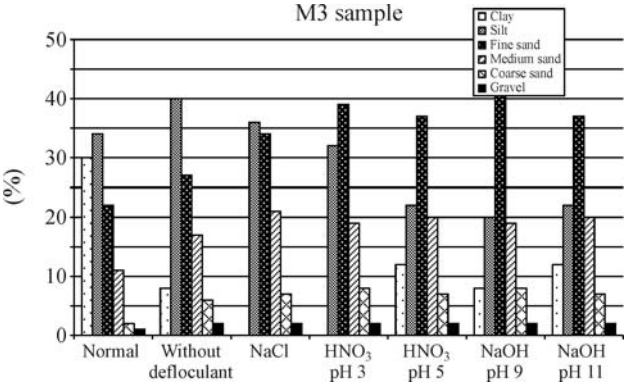

Figure 4 - Grain-size fractions of M3 sample exposed to different fluids.

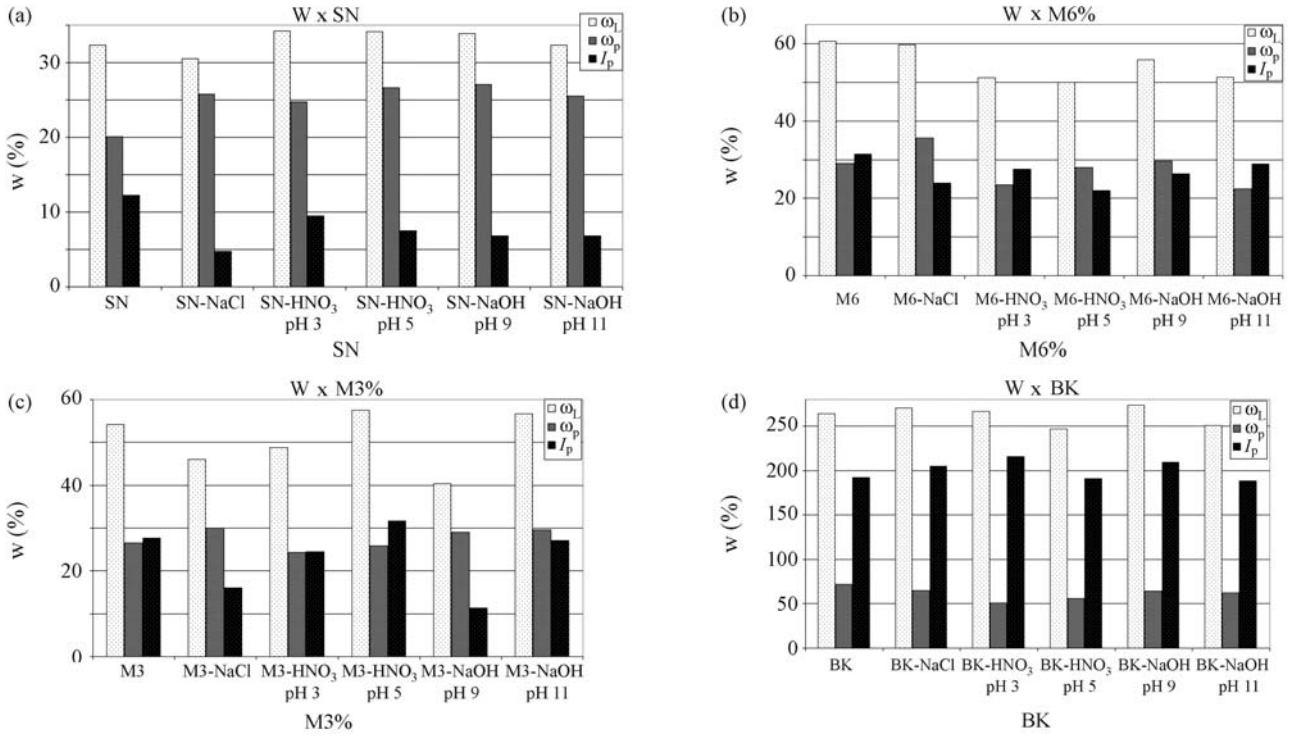

Figure 2 - Atterberg limits using different fluids: (a) SN sample; (b) M3 sample; (c) M6 sample and (d) BK sample.

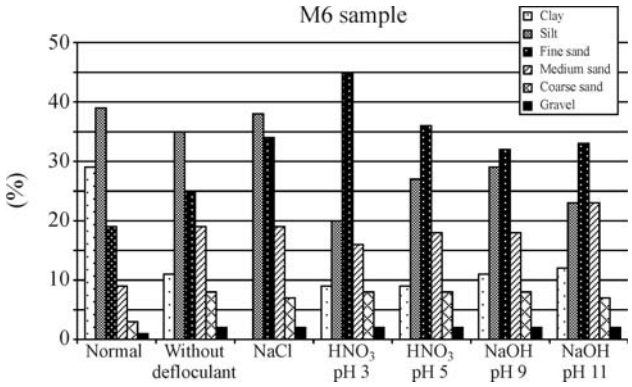

Figure 5 - Grain-size fractions of M6 sample exposed to different fluids.

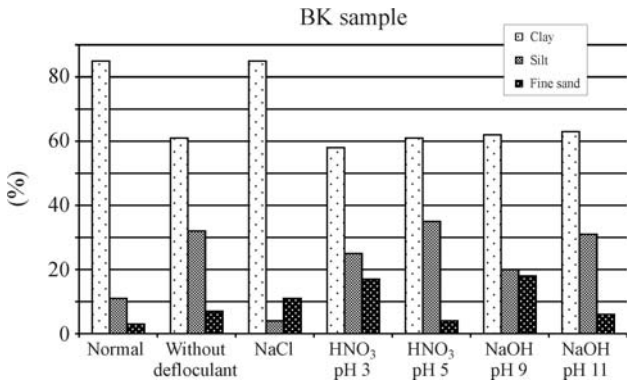

Figure 6 - Grain-size fractions of BK sample exposed to different fluids.

sult with the clay fraction obtained using a solution without deflocculant, it can be concluded that NaCl acted as a flocculant agent. In turn, the clay fraction of BK sample did not change with the use of NaCl, but the silt and sand fractions experimented some variation.

The evaluation of the grain-size distribution for different pH conditions reveals that no specific trends exist for SN, M3 and BK samples. On the other hand, for M6 sample the fine sand fraction decreases as the pH increases. It is worth to point out that for this sample the fine sand fraction achieved its maximum (45%) at pH = 3 compared to all samples and solutions.

Finally, no medium/coarse sand nor gravel were observed in BK sample for all the solutions being tested.

4. Conclusions

In a general way the following conclusions can be stated with respect to the changes in the characteristics of the natural soil sample (SN) when mixed with bentonite (M3 and M6 samples):

(1) The clay fraction of the mixtures decreased relative to the natural soil and this is probably due to the "cement" effect of the bentonite among the SN grains;

(2) As was expected, the presence of bentonite has led to a significant increase in the plasticity of the natural soil;

(3) As was expected, this increase in plasticity has led to a decrease in the maximum dry unit weight obtained under Proctor energy of compaction;

(4) The physical-chemical parameters such as pH, electrical conductivity, cation exchange capacity and specific surface experimented significant changes;

The hydraulic conductivity of the compacted mixtures decreased two to four times when compared to the compacted natural soil. A greater reduction was expected for these mixtures since the bentonite used in the tests is quite swelling.

In terms of compatibility, the results have demonstrated that the greatest changes in the plasticity and grain-size distribution due to the contact with different chemical solutions occurred for SN and M3 samples. These changes may lead to structural modifications in the compacted soil, which may affect its overall performance as liner. On the other hand, M6 and BK samples have shown to be more compatible than SN and M3 samples, which leads to the conclusion that the addition of bentonite in the proportions used in the tests improves the compatibility of the natural soil.

From a perspective of using these mixtures for base liner purposes, it is concluded that the addition of bentonite to lateritic soils brings more activity and compatibility, which may be good in terms of contaminant retention and interaction. Additionally, the influence of the bentonite on the reduction of the hydraulic conductivity was non-linear.

It must be pointed out that the results and conclusions presented here are preliminary and more research on this subject is needed. It is suggested more laboratory tests using different lateritic soils and their direct permeation with different chemical solutions and waste liquids using flexible wall permeameters. Likewise, the structural changes due to bentonite addition deserve more investigation.

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Symbols

A: activity
BK: bentonite sample
CCL: compacted clay liner
EC: electrical conductivity
CEC: cation exchange capacity
G_s: specific gravity of solids
k: hydraulic conductivity
k_R relative hydraulic conductivity
k_f : hydraulic conductivity with a different fluid
k_w: hydraulic conductivity with water
ICp: incompatibility index
ω_L: liquid limit
I_P: plasticity index
I_{pw} : plasticity index using water
I_{pf}: plasticity index using a different fluid
ω_P: plastic limit
M3: SN sample with 3% of bentonite (dry weight basis)
M6: SN sample with 6% of bentonite (dry weight basis)
pH: potential hydrogen
S: free swelling index
SN: natural soil sample
SS: specific surface
w_{ot}: optimum moisture content
ρ_{dmax}: maximum dry unit weight
V_f : final volume of sediment column
V_i: initial volume of sediment column