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The effectiveness of "clay" liners as basal isolation of landfills: a case study

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Abstract A sodium bentonite is being used in the basal-isolation of landfills installed on strongly fractured granitic rocks of N Portugal (NW Iberia). To evaluate the performance of this "clay" as the ultimate impermeable basal barrier, a detailed study was carried out of the granite fracture network; the Na-bentonite was tested to characterize its mechanical and geotechnical behavior; geophysical surveys were made to search for landfill leakage and the waters found around the landfill were geochemically characterized. Fractures in the granite are usually open and devoid of mineral infillings or clay materials and, thus, permeability of the granite is very high. Dispersal of contaminants can be further enhanced by the position of the landfill on a high steep-sided ridge. Geophysical and geochemical data show evidence for seepage and dispersion of pollutants, which means that the clay failed as an ultimate isolation barrier against seepage. This investigation shows that this can be due to fracturing of the clay under load and/or to its non-homogeneous saturation and extreme shrinking character upon drying, which are accompanied by the formation of extensive cracking. Observations and experimental results suggest that the use of synthetic "clays" in the safe building of landfill bottom liners needs further research, and extreme care should be taken in preventing that clay water content suffers large variations after saturation, as this process considerably degrades the mechanical behavior and sealing properties of the studied Na-bentonite.

Keywords Fractured granite basement · Basal isolation · Clay flow and fracturing · High permeability · Pollution risks · NW Portugal · Braga

Introduction

According to officially reported data (INR 2000), municipal solid waste production in Portugal mainland increased by nearly 36% between 1995 and 2000, reaching 4.5 million tons in the latter year, which is equivalent to ca. 450 kg/person/year. Recent data show that solid waste production is still increasing and demanding solutions other than their simple disposal by means of incineration or of landfill dumping. Solid waste management in Portugal is dominated by landfill dumping, and that is the reason why the impervious effectiveness of a bottom liner used in a recently built facility in Northern Portugal was investigated. The relevance of this issue is enhanced by the fact that new landfill sites will have to be located on strongly fractured Paleozoic granite massifs and metasediments, Meso-Cenozoic carbonate sediments, and highly permeable recent sands, which together make up the geological substrate of a very large part of Portugal. Therefore, the impermeable character of bottom liners must be tested, guaranteed and monitored in the landfills.

Economic and safety issues involved in waste disposal, especially the need to avoid groundwater contamination by landfill leachate seepage, have led to the recent technological development of multi-barrier systems of impermeable layers. Geosynthetics and geocomposites are fundamental parts of these sealing systems, and their use is widespread in many countries. In a newly built landfill facility in northern Portugal, the lowermost leachate barrier is composed of geosynthetic clay liner (GCL) strips; adjoining strips are not seamed and, instead, they partially overlap and are ‘‘sealed’’ by a layer of sodium bentonite powder. The performance of this Na-bentonite is the main object of investigation of the present study. In laboratory reconstructions, the mechanical and geotechnical behavior of the Na-bentonite placed in between adjoining GCL strips was evaluated, as it is in this situation that the Na-bentonite is less constrained physically and thus more prone to deformation and rupture. X-ray diffractometry was used to identify the minerals making up the clay powder; several laboratory tests were performed to geotechnically characterize the clay; and cakes with different water contents of sodium bentonite were submitted to pure shear deformation using an automated pure shear rig to evaluate its behavior when subject to load, and wetting and drying cycles. The results prompted the search for field evidence of clay rupture around the landfill facility by means of geophysical and geochemical investigations, together with a detailed structural study of the fracture network of the granite massif.

If water (rain or other) did not reach the clay layer during the landfill construction, and if the overlying high density polyethylene (HDPE) membrane was really impermeable, then the only water that could seep into the clay layer would come from below, from high groundwater levels during rainy season. This can easily happen because the sand-like bottom layer below the GCL is highly permeable and the considerable open fracture frequency in the underlying granite promotes easy groundwater flow. Therefore, the effects of wetting on the properties of the Na-bentonite under variable water contents were investigated and tested.

The landfill site

An important landfill facility was built in a granitic environment and has been operating in N Portugal since 1998. The landfill is located on top of the south-facing slope of a granite ridge (largely dominated by porphyry biotite-granites with homogeneous mineralogical composition), and occupies the upper reaches of an ephemeral creek, whose bed inclines on average 1.5° downhill inside the landfill, although lateral slopes can be as steep as 15°. The ridge, culminating at 451 m above sea level, is structurally controlled by sub-vertical fractures and faults striking between ENE-WSW and E-W (Marques et al. 2001, 2002)(Fig. 1); locally, NNW-SSE and N-S fractures and fault systems are common. Overall, fracture density is extremely variable, but box-counting methods have shown that the mapable fracture pattern is self-similar at all scales observed, which range from 1:100 to 1:25000 (Mateus et al. 2003; Marques et al. 2002). Aerial and Digital Terrain Model interpretation, coupled with field observations around the landfill site, suggest that a major fracture zone controls the water creek notch where the landfill is installed. The site is in a wet temperate climate and high relief. The thickness of the weathering profiles is quite variable, even at local scale, and it varies generally from negligible, where fracture densities are low, to around 4–5 m in densely fractured areas. Geophysical data show that it may reach 20–30 m in the main fault zones that cut the granite beneath the landfill.

The presence of water springs S of the landfill implies that at least part of the groundwater flow must be from the landfill to the fractured and altered granite. This is confirmed by the fact that spring waters that emerge nearer the landfill differ significantly from background waters, as shown by their trace element concentrations, their electrical conductivity (at least twice the background values) and pH values (increased by 1–2 units relative to background—Mateus et al. 2002). This geochemical anomaly is generated by leaks at the bottom liner, as shown by geophysical surveys (Fig. 2; Mota et al. 2004). Resistivity profiling across the landfill facility reveals a superficial resistive layer overlying a generally more conductive horizon, which should be interpreted as unsaturated and saturated granitic rock, respectively. Major fractures show up by a decrease in resistivity, correlative of the enhanced water circulation and resulting granite alteration. An anomalous well-defined domain characterized by very low resistivity can be seen directly below the leachate collecting pipes. This domain extends underground northwards and southwards of the facility, and seems to intersect the surface to the south of the waste piles. This would justify the visible signs of pollution shown by the creek that flows from the south fence wall of the facility. Inverse numerical modeling of the resistivity values using the

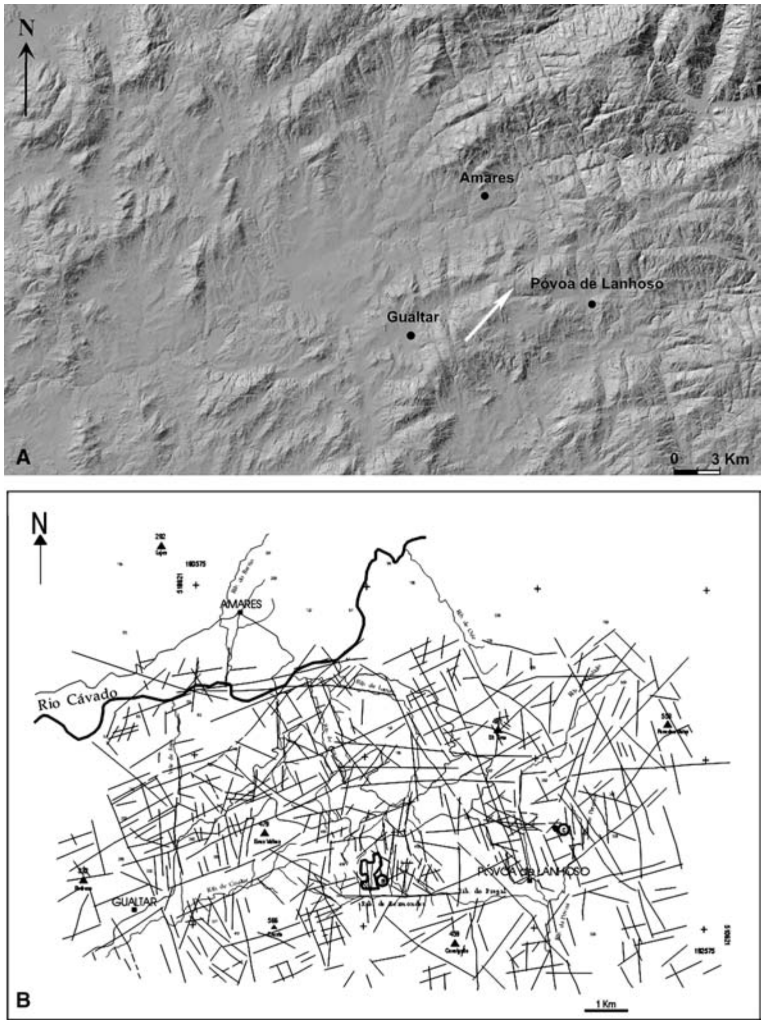
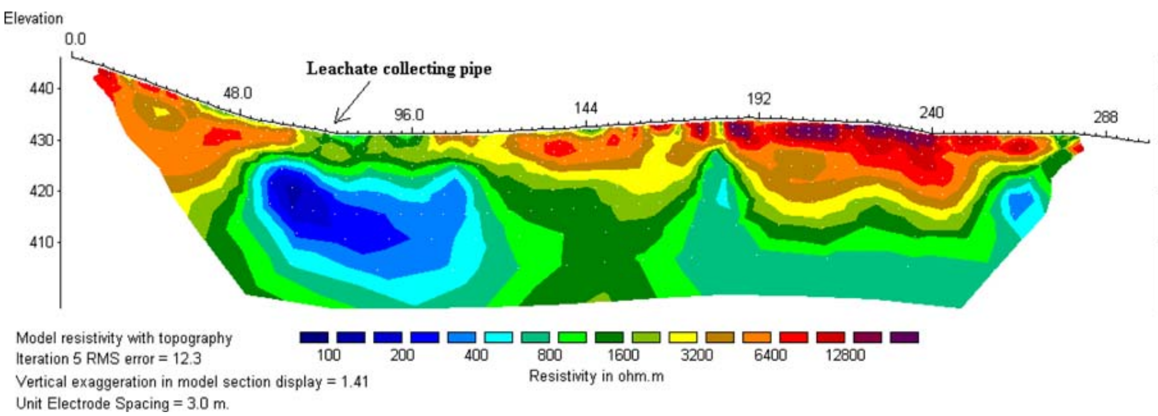


Fig. 1 **a** Shaded relief based on a Digital Terrain Model of the area surrounding the landfill facility (marked by *white arrow*). Note the abundance of lineaments, which correspond to fault zones (mostly) and major fractures. **b** Mapped fault network in the region around the landfill (whose perimeter is marked by **b**); **c** indicates an old, presently sealed, dump

measured resistivity of the leachate (0.41–1 Ω m) leads to a calculated bulk resistivity of 20–100 Ω m, assuming a fracture density of 1–2%. A more realistic modeling using a somewhat diluted leachate as the fluid circulating in fractures (resistivity 3 Ω m) gives an optimal agreement (147–287 Ω m) with the anomaly values quoted above (Mota et al. 2004).

Fig. 2 Resistivity model considering the effects of topography obtained from inversion of Dipole-Dipole data. Note the low resistivity area just below the landfill, which is located in the topographic depression



Before landfill installation, all soil and weathered rock were removed till the hard unweathered granite was exposed, which was mostly made up of corestones. A multi-barrier system was laid on top of this highly irregular surface and comprised, from bottom to top: (1) a sandy material of variable thickness to fill in the depressions between corestones and smooth the irregular surface, which still kept corestones. (2) A 5-mm thick geotextile layer that works as a basal protection against deterioration by puncture and friction of the overlying layer. (3) A 5-mm thick GCL laid as strips, with about 20 cm of non-seamed overlap with the neighboring strips. Joints between adjoining strips were ''sealed'' by Na-bentonite powder, i.e., a 5–10 mm thick layer of clay was placed in between the non-seamed overlapping strips. Because the landfill bottom surface was not smooth and cylindrical (mostly due to spherical corestones), the stiff GCL did not lay perfectly adjusted to the bottom surface and spaces had to be filled with Na-bentonite. Note that the GCL and intervening clay layer are supposed to work as the ultimate barriers against seepage, if the overlying geomembrane fails. GCL and intervening clay layer were subjected to wet and dry cycles during landfill construction, which prompted the study of wetting and drying effects on the clay layer. (4) A layer of high-density polyethylene (HDPE), made up of seamed stripes, to work as a first impermeable barrier against seepage. The entire bottom liner was built before the operation of the facility began, and is being covered by waste from N to S, i.e., from high to low topography. This means that the lower lying parts of the liner remained exposed to weather conditions for many months, which may have caused degradation of the HDPE layer by solar radiation (Rowe and Sangam 2002).

The Na-bentonite

Mineralogical characterization

To characterize the mineralogical nature of the Na-bentonite, six samples were studied by X-ray diffraction and results show a relatively homogeneous mineralogical composition. It consists of a complex mixture of different smectites, those of sodium nature clearly prevailing, with trace amounts of quartz, calcite and muscovite. This is determined by the fact that all the heated specimens collapse their main 001 spacing to 10 Å and expand that same spacing to 16.5–16.8 Å upon treatment with glycerol. Superposition of peaks and structural variability of the smectites prevent the accurate identification of the minerals constituting this mixture, although it seems that Na-montmorillonite is the most abundant species.

Geotechnical characterization

Due to the lack of standards for testing this type of material, standard geotechnical tests were used to assess the properties of the Na-bentonite, relevant for its behavior evaluation under the conditions found when it is used as part of a sealing material between adjacent strips of GCL. The tests used included grain size analysis, liquid limit with Casagrande apparatus and cone penetration methods, free swelling and direct shear. Plastic and shrinkage limits tests were also carried out but the results were very inconsistent mainly due to the extreme swelling behavior of the clay. In fact, it was not possible to produce the 3 mm diameter clay rolls required for the plastic limit test (ASTM D4318-00 2000), and the extreme volumetric decrease in the shrinkage tests (ASTM D427-98 1998) led to a systematic disintegration of the specimens during drying, which prevented the measurement of their final volume.

Grain size analysis was performed with a LASER particle size analyzer. Data obtained showed fairly small variations between specimens and the material is mainly composed of more than 70% silt (2–62.5 μm) and approximately 25% of clay size (< 2 μm) particles considering European standards (DIN, AFNOR). Fine sand particles content is lower than 5%, with the studied material corresponding to clayey silt.

Liquid limit (w_L) test sets with the Casagrande apparatus (ASTM D4318-00 2000) were performed providing w_L values between 381 and 408. The regression of all data points in the $n=15$ to $n=40$ range provides a mean $w_L =$ 394. The data scatter is small and within the ±3% range of expectable error (Bowles 1984) due to the natural limitations of this test. The liquid limit cone penetration method (BS 1377-2. 1990) provided a lower mean value, $w_L=322$, (326; 319) for tests made 24 h after the specimen preparation, while tests made immediately after specimen preparation provided a higher mean value, around 420 (457; 407; 400), but with a slightly higher scatter of data points (Fig. 3). The latter data are comparable with the liquid limit found with the Casagrande apparatus, which is a dynamic test, although some difference arises. The former datum tends to express a static behavior of the studied clay, which is highly dependent on the nature of the applied loads. This datum suggests that a large amount of further work is needed to establish standard test procedures adequate to the characterization of these highly swelling materials for the purposes of GCL application.

The free swell tests were performed with a LNEC apparatus (LNEC 1967), which measures height variation of 1-cm thick cylindrical and laterally confined specimen of the soil, caused by capillarity-induced saturation with distilled water, under negligible confinement (vertical) pressure. This test was selected mainly

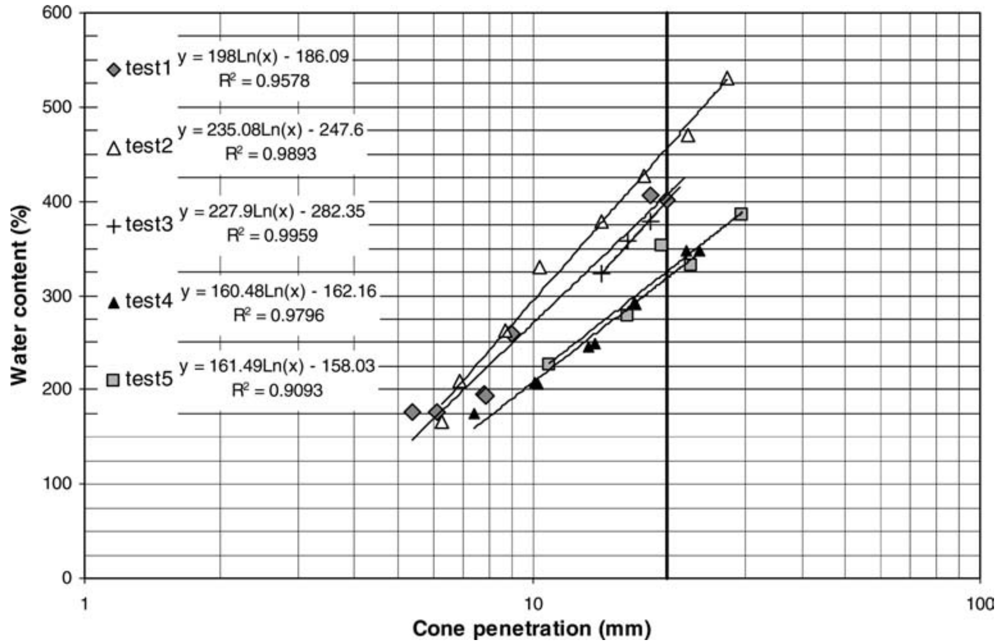


Fig. 3 Results of liquid limit tests by the cone penetration method. Tests 1 to 3 performed one day after preparation and tests 4 and 5 performed immediately after preparation

because the first saturation of the landfill bottom liner occurred probably with no waste placed on it, and thus under very low confining pressure. In this case, and because of the extreme swelling behavior of the clay, which exceeds the range of the measuring device, a specimen thickness of 5 mm was used, which is approximately the thickness of the clay layer included in the composite liner. Data obtained lies within a relatively narrow range, with the clay swelling up to 270–300% with complete saturation (Fig. 4). However, the saturation is highly irregular, with the clay forming separated flakes and is always accompanied by extensive cracking that does not disappear with complete saturation, in spite of the fact that the entire base of the specimen is in contact with a highly permeable saturated porous stone. After saturation, drying of the specimens was also monitored to assess the shrinking behavior. Cracks formed during saturation persist and at the final dry state, the specimens still indicate a 100% swell mainly due to the presence of open cracks. This cracking behavior is an expected feature of these materials (Eglоffstein 2001). However, the extent of observed cracking raises the following points: (1) The swelling behavior of the clay was not tested under a variety of overburden

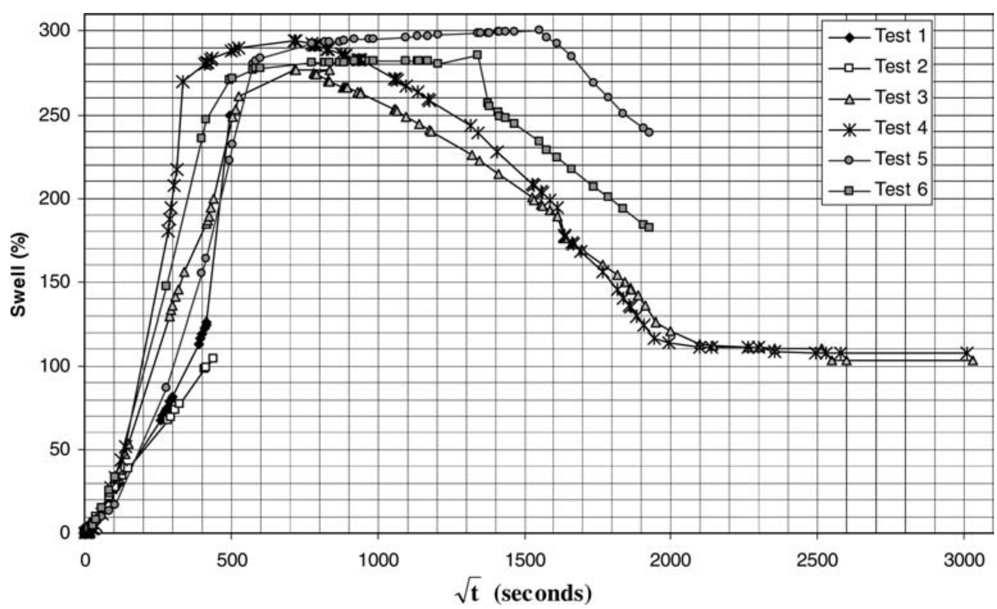


Fig. 4 Results of the free swell tests performed with a LNEC apparatus

pressures, to assess its self-healing properties, i.e., its capacity of self-repairing the desiccation cracks by plastic flow. (2) As suggested by Egloffstein (2001), a complete drying of the GCL used as bottom liner is rather unlikely under natural conditions, suggesting that less severe conditions are likely to occur in landfills. (3) The swelling behavior of the Na bentonite is highly dependent on the type of encapsulation between the cover and the carrier geotextiles and on the chemical composition of the fluids involved. (4) The extensive cracking observed during saturation indicates that the material cannot be confidently used to seal adjoining strips of GCL without any sort of permanent and strong seam. As there are still limited data available on this fundamental topic, the accurate prevision of the GCL behavior in the required service conditions requires further research. For a preliminary evaluation of the shear strength of the clay, consolidated direct shear tests were performed, with shear velocity of 0.02 mm/min, to allow the dissipation of pore pressure generated during shear. However, due to the low permeability of the clay, the tests cannot be considered as fully drained. Specimens were prepared with approximately 250% initial water content. Data obtained indicates, as expected, very low strength properties, with $\phi = 9°$ for $c = 0$ (Fig. 5), which can be taken as close estimates of effective strength parameters, much lower than the typical values for various common liner interface shear strength (Bagchi 1994). These low strength values are likely to play an important role in the overall stability of the landfill, in the steepest outer slopes, where the clay was used to seal the overlapping strips of the GCL without any kind of seams. In fact, the saturation of the clay before the waste disposal would make easier the sliding apart of the overlapping GCL strips, causing liner ruptures and allowing the leakage of contaminated fluids when the HDPE membrane is also damaged.

Mechanical behavior

The gentle slope found in most of the landfill area has determined the simplification of the mechanical tests to pure shear, as these can be more accurately characterized (mostly stress) and are a good approximation to the conditions found at the bottom liner, far from its lateral very steep parts. Here, a combination of simple shear (steep slope) and pure shear (load) must dominate, and more significant effects of high stresses can be expected in the clay liner. The GCL was not tested due to experimental constraints, but the tests performed on the isolated bentonite powder retain their validity because they mimic the conditions in which the Na-bentonite is used to seal together the laterally overlapping strips of GCL. The Na-bentonite was tested under considerable water contents because of wetting (and drying) from rain during landfill construction and also possible seasonal wetting (and drying) from the fractured granite below. After each experiment, the Na-bentonite was left to dry to simulate the drying cycle.

The pure shear rig used to perform the experiments is the one described in Mancktelow (1988) and used by Marques (2001). The walls that are pushed closer are the top walls, and the ones that are pulled apart are the side walls (parallel to the direction of maximum compression$-\sigma_1$—and where constant side stress is monitored) (Fig. 6a). The experiments were performed under the following experimental conditions: temperature equal to 21°C; strain rate constant and equal to 6.00E$-$5 s^{-1} ; constant side stress of about 30 kPa; maximum shortening percentage of about 40%; and initial size of the model 290×70×60 mm. Marker lines were stamped or carved on top of the models to better analyze and quantify the deformation. Thorough lubrication of all confining walls with Vaseline guarantees pure shear.

Experimental results

In the early stages of shortening (less than 10%), the clay cake showed very small fractures like open gashes, subparallel to the shortening direction. Some of these tension fractures continued developing throughout the experiment and gave rise to gashes with considerable opening (Fig. 6a, b). En échelon fracturing and their coalescence commonly preceded the later formation of faults. Other faults were born as small fractures that progressively increased in length by propagation at the ends. At the early stages of fracturing, two sets of conjugate faults formed. The intersection between conjugate

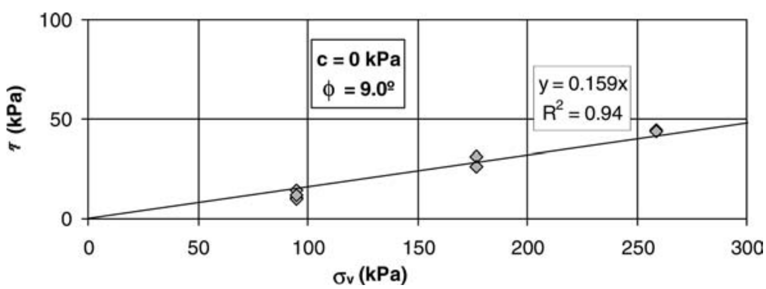


Fig. 5 Results of the direct shear tests

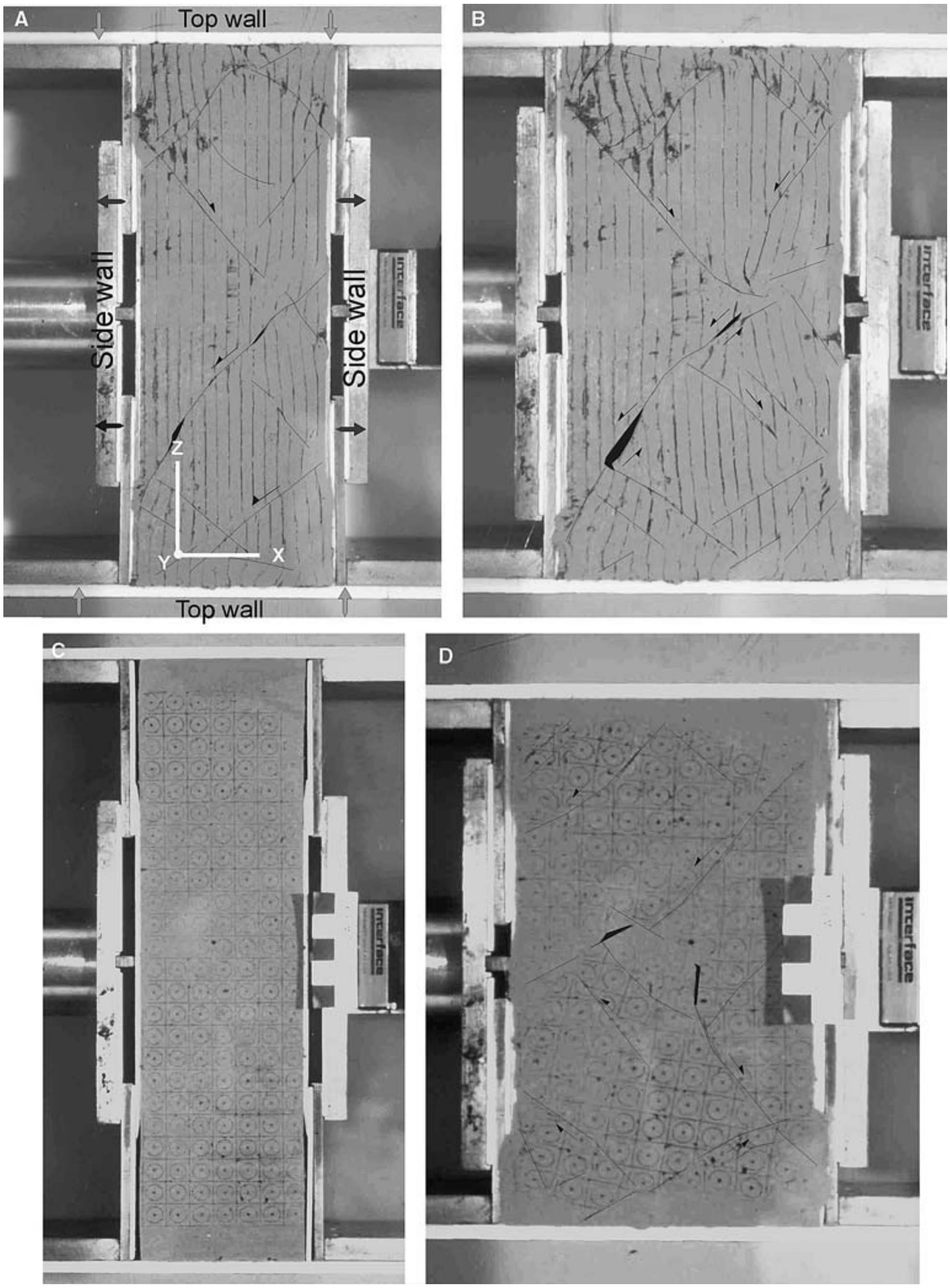


Fig. 6 Photographs of XZ plane (plan view) of results of pure shear tests. **a** and **c** are initial stages, **a** and **b** have lower water content, and **c** and **d** have higher water content

sets was typically a conflict zone, more or less complex depending on equal or unequal development of conjugate sets and timing of intersections. Fault geometry was very simple (Fig. 6), but variable according to water content and shortening percentage: they were (at least) initially rectilinear, and oriented at ± 30–$35°$ to σ_1 in the experiments with less water content in the clay (Fig. 6a, b), and at greater angles with higher water content (Fig. 6c, d). At later stages, some faults rotated and bent significantly. The central section of the fault gradually rotated, clockwise in the sinistral faults and anticlockwise in the dextral, and the overall shape became sigmoid. In some cases, the terminations of faults asymptotically curved towards the main conjugate.

Relative rotations about axes parallel to conjugate's intersection were commonly observed in faults and blocks limited by faults. Rotation of faults was achieved in two distinct ways: by flattening of the clay, or by rigid rotations, or by a mixture of both mechanisms. Rigid rotation of blocks of considerable dimension (to as much as 20° from the original position) was typically associated with significant rotation of limiting faults and with very low values of strain of the clay. Mode of fault propagation and rotation of faults gave rise to open gashes that persisted throughout the experiment.

The analysis of the stress (σ_1) strain (% shortening) graphs (Fig. 7), at constant strain rate, shows an early rapid increase of σ_1 to a peak value. This part of the curve means that the early stages of deformation of the clay are accompanied by strain hardening (increase in σ_1 at constant strain rate). After the maximum of σ_1 is reached, strain softening slowly develops, deduced from the negative slope of the curve. From the analyses of the stress/strain graphs, at constant strain rate, only the behavior of the whole model is apparent, which is the sum of the behavior of the clay itself and faults. The initial strain hardening can be due to the elastic deformation of the clay, because there are no faults. But from the peak σ_1 onwards, the stress drop should be related with fracture/fault formation and ductile behavior of the clay. Coincidence of fault initiation with peak stress in the experiments suggests that faulting triggers strain softening. This should be equivalent to the drastic stress drop observed in experiments with natural rocks (e.g. Reches and Dieterich 1983). A more detailed look at the graphs shows two important features: (1) the peak σ_1 has an inverse dependence upon degree of water content; ca. 100 kPa at 250% water content and ca. 75 kPa at 400% water content (Fig. 7a, c, respectively); (2) the stress drop at ca. 5–10% shortening is sharper with less water content. Together, these two features seem to indicate an increasing ductile behavior of the analyzed clay with increasing water content.

When left to dry after deformation, to simulate the effects of the dry season on the deformed Na-bentonite, it retracts significantly and the fractures become wide open (Fig. 8).

Conclusions

The analyzed Na-bentonite has characteristics of expansibility and, possibly, of permeability adequate for its use as an isolation liner. However, the difficult and uneven saturation of the clay shown by the geotechnical

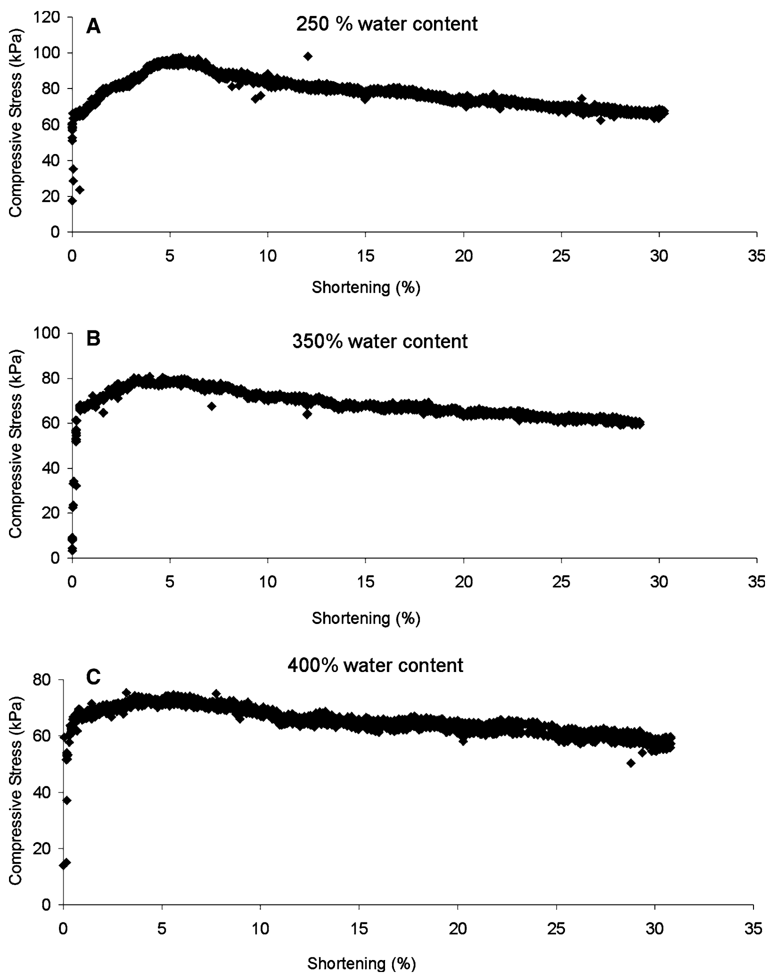


Fig. 7 Most typical σ_1 versus % shortening graphs of experiments, at constant ca. 30 kPa of side stress. **a** Initial strain hardening to ca. 100 kPa, and then strain softening. **b** Initial strain hardening to ca. 82 kPa, then strain softening. **c** Initial strain hardening to ca. 75 kPa, and then strain softening

tests causes the formation of extensive fissuring, which is a major concern and an obvious limitation of the material to provide a very low permeability layer and reducing the liner self-repairing capacity. Still, the thickness about 12 mm after expansion (in conditions of total saturation and devoid of confining pressure) is on the lower limit of the thickness interval recommended in the literature (Bagchi 1994). The effects of load on the Na-bentonite, as shown by the mechanical tests under pure shear, are extensive fracturing with local formation of wide open gashes. When subjected to decreases of water content, the Na-bentonite retracts and the frac-

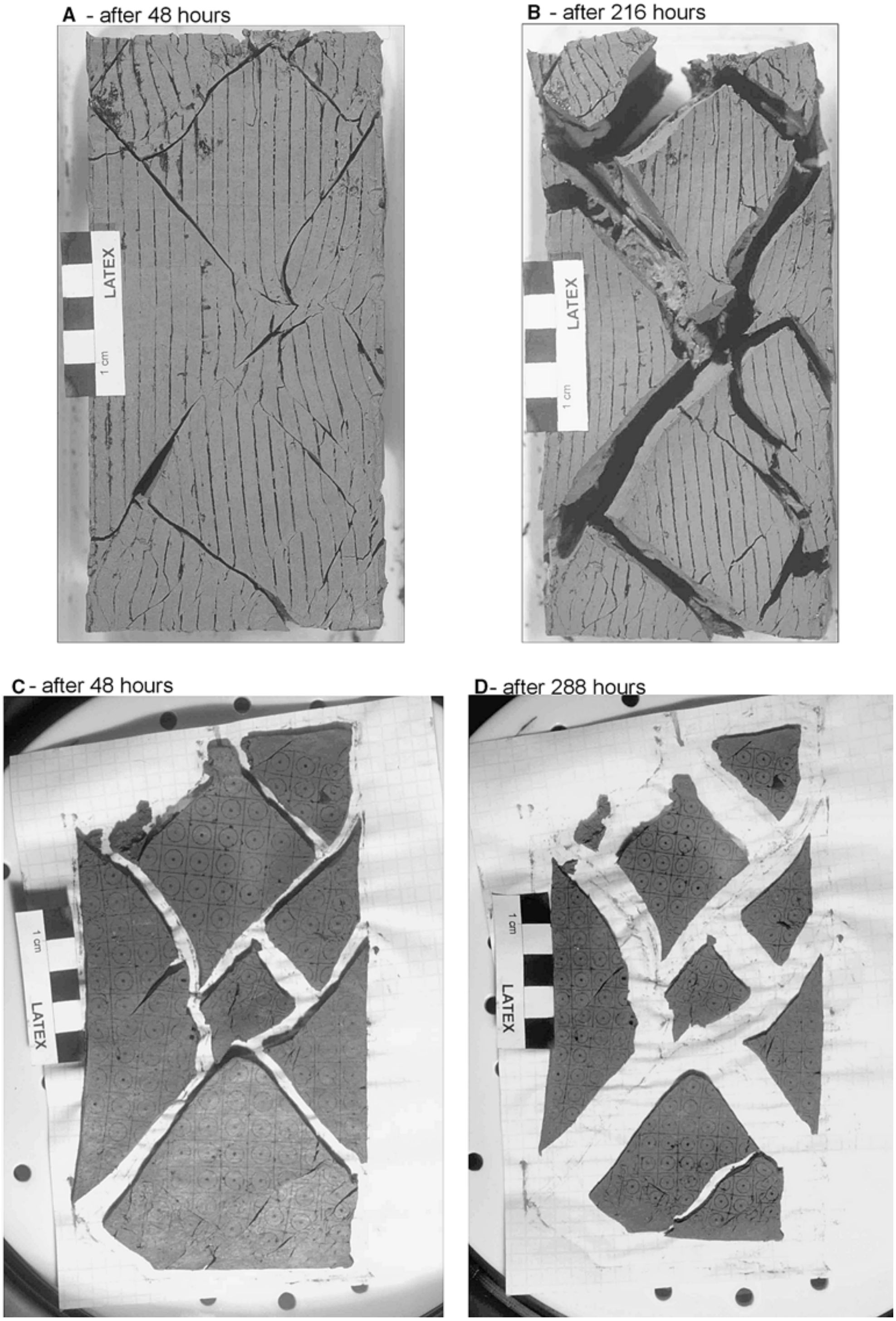


Fig. 8 Photographs of the deformed clay cake left to dry in the open air. **a** after 48 h; **b** after 216 h. **c** and **d** are similar, but cut to a thickness similar to that used in the GCL and between overlapping GCL. Note the great amount of widening of the faults during desiccation

tures become also wide open. Therefore, any seepage through the overlying HDPE will explore the fissuring of the clay, which will not prevent downflow and escape of pollutants. In this case, even the self-healing properties of the clay can be not enough to recover the initial low permeability to sustain the leakage.

The available geophysical and geochemical data, with their clear evidence for the beginning of a leak under the facility only two years after starting operation, suggest that the processes observed in the laboratory have already occurred in the field. More investigation and testing of the synthetic clays are required to support its use as an effective ultimate leachate barrier in landfill base liners. The data obtained indicate that the clay should not be subjected to large variations in water content and high stresses during operation. This aspect is particularly important in the studied case, where the clay was used as a seal between adjacent strips of GCL, as a replacement of mechanically stable and strong seams. The extensive cracking observed during saturation indicates that the material cannot be confidently used to seal adjoining strips of GCL without any sort of permanent and strong seam.

It is also debatable the efficiency of GCL liners to replace completely compacted clay liners (CCL), what could lead to more conservative projects with the combination of GCL and CCL to ensure a safer and long lasting landfill isolation.

Due to the importance of the impacts on groundwater, efficient and adequate environmental monitoring should be conducted from the beginning of the facilities construction, so that eventual failures may be detected and remedied in their initial stages.

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References

ASTM D427-98 (1998) Standard test method for shrinkage factors of soils by the mercury method. ASTM International, USA

ASTM D4318-00 (2000) Standard test methods for liquid limit, plastic limit, and plasticity index of soils. ASTM International, USA

Bagchi A (1994) Design, construction and monitoring of landfills, 2nd edn. Wiley, New York

Bowles J E (1984) Physical and geotechnical properties of soils, 2nd edn. McGraw-Hill Int, Ed

Egloffstein AT (2001) Natural bentonites-influence of the ion exchange and partial dessication on permeability and self-healing capacity of bentonites used in GCLs. Geotextiles and Geomembranes 19:427–444

Instituto Nacional de Resíduos (INR) (2000) Produção de resíduos sólidos urbanos/2000. Ministério do Ambiente. Lisboa

LNEC (1967) Soil swelling test. Especificação LNEC E200–1967. Laboratório Nacional de Engenharia Civil, Portugal

Mancktelow NS (1988) An automated machine for pure shear deformation of analogue materials in plane strain. J Struct Geol 10:101–108

Marques FO (2001) Flow and fracturing of clay: analogue experiments in bulk pure shear. In: Koyi HA, Mancktelow NS (eds) Tectonic modeling: a volume in honor of Hans Ramberg. Geological Society of America, Memoir 193:261–270

Marques FO, Mateus A, Amaral H, Gonçalves MA, Tassinari C, Silva P, Miranda JM (2001) The Amares basin: an ENE-WSW graben formed by recent reactivation of the late-Variscan fracture network? Comunicações Instituto Geológico e Mineiro 88:35–50

Marques FO, Mateus A, Tassinari C (2002) The late-Variscan fault network in central-northern Portugal (NW Iberia): a re-evaluation. Tectonophysics 359:255–270

Mateus A, Marques FO, Monteiro Santos F, Figueiras J, Gonçalves MA, Mota R, Amaral H, Silva P, Miranda M, Almeida C (2002) Fluid flow in fractured rocks and its implications for waste disposal criteria. Fundação para a Ciência e Tecnologia –Portugal, Research Project PRAXIS XXI/P/CTE/11028/98, Final Report

Mateus A, Gonçalves MA, Marques FO, Monteiro Santos F (2003) Multi-scale geometrical analysis of fracture networks in granite massifs and its application to landfill site selection. "A Geologia de Engenharia e os Recursos Geológicos", Coimbra, Portugal. Eng Geol 1:79–91

Mota R, Monteiro Santos FA, Mateus A, Marques FO, Gonçalves MA, Figueiras J, Amaral H (2004) Granite fracturing and incipient pollution beneath a recent landfill facility as detected by several geoelectrical surveys. J Appl Geophys 57:11–22

Reches Z, Dieterich JH (1983) Faulting of rocks in three-dimensional strain fields: I. Failure of rocks in polyaxial, servo-control experiments. Tectonophysics 95:111–132

Rowe RK, Sangam HP (2002) Durability of HDPE geomembranes. Geotextiles Geomembranes 20:77–95