Transportation Geotechnics 14 (2018) 157–168

Contents lists available at ScienceDirect

Transportation Geotechnics

journal homepage: www.elsevier.com/locate/trgeo

Laboratory investigation on the behaviour of an overconsolidated expansive clay in intact and compacted states

Mohamed Khemissa *, Lakhdar Mekki, Abdelkrim Mahamedi

Geomaterials Development Laboratory, Civil Engineering Department, Faculty of Technology, University of M'sila, P.O. Box 166 Ichbilia, 28000 M'sila, Algeria

ARTICLE INFO

Article history:
Received 16 July 2017
Revised 6 December 2017
Accepted 17 December 2017
Available online 19 December 2017

Keywords:
Expansive clay
Compaction
Compressibility-consolidation
Swelling-shrinkage
Shear-failure

ABSTRACT

Natural climatic phenomena and human activity frequently cause disorders in the masses of fine-grained soils characterized by very significant volume variation as soon as the conditions of their equilibrium are modified. A better description of the behaviour of fine-grained soils can be seen with respect to drying-wetting cycles. This paper presents a series of laboratory test results obtained on a heavily overconsolidated expansive clay, for which significant damages frequently appear in road and motorway infrastructures, in urban public utilities, as well as in civil and industrial low-rise structures. The effects of compaction and drying-wetting cycles on the mechanical parameters of this clay are analyzed to establish a predictive model of the soil movement following the in-situ water table variation. Comparative analysis between the deformability and strength characteristics of the clay in intact and compacted states are then presented. Tests results show that the values of the geotechnical parameters derived from these tests are depending on some experimental aspects such as the compaction energy, drying time, initial deformability and soil saturation. In all cases, the behaviour of intact and compacted clay samples is governed by the same laws of compressibility and consolidation, shrinkage and swelling as well as shear and failure.

Introduction

Swelling soils are present worldwide and indexed in several countries [24]. These soils cover an important part of Algerian arid and semi-arid regions, which are delimited by the Tellian Atlas in the North and the Saharian Atlas in the South and extend from East to West until the bordering Maghreb countries. Some characterization studies of soils located in these zones have been carried out, among which the works of Derriche and Kebaili [8] on In-Aménas clays, Hachichi and Fleureau [14] on Oran clays, Djedid et al. [10] on Tlemcen clays, and Medjnoun et al. [27] on Médéa clays. These studies confirm the expansive character of the soils and measure the extent of damages caused to civil and industrial structures in such sites. In Algeria, urban works are nowadays booming due to a population growth amplified by a considerable economic and social development. This is the case of the province of M'sila, *inter alia*, of which the urban fabric extends towards virgin zones often less favorable than those already urbanized. This extension transforms villages into cities and cities into metropolises, which require the construction of new road and rail networks. The rapid development and the high demand for the natural deposits of good quality are becoming more and more rare and we have often recourse to problematic soils which do not always meet regulatory requirements: the expansive clays are a typical example. Existing road construction standards require the physico-chemical and mechanical properties of such soils to be identified. Thus, their use could be considered in construction of embankments and subbase layers with a secure margin [23].

Intact and compacted expansive soils have been the subject of many experimental researches for several decades. It was concluded that the behaviour of natural stiff clays and natural marls or compacted fine-grained soils can be described by responses characterized of different forms, but the behaviour laws which characterize them seem to be sufficiently similar so that we can extrapolate some aspects from one soil to another [26,24]. However, the constitutive parameters of the laws of soil behaviour depend on the validity criteria of oedometric and triaxial tests results [17]. Damages affecting the geotechnical structures can be explained by the mechanical behaviour of compacted soils and by variations of hydric conditions that they undergo at their boundaries. Corresponding deformations can be attributed to creep related to the viscosity of intact or compacted materials, to swelling-shrinkage related to the presence of expansive clayey

* Corresponding author.
E-mail address: khemissa@univ-msila.dz (M. Khemissa).

https://doi.org/10.1016/j.trgeo.2017.12.003
2214-3912/© 2017 Published by Elsevier Ltd.

minerals, and to the effect of hydric fluctuations, which induce deformations of unsaturated compacted materials.

The behaviour of highly-compacted highly-swelling clays has already been the subject of numerous theoretical and experimental investigations over the last three decades [4,12,15,6,20,28,32,5,33,42,40,34,13,37,36,41,44]. These research studies focus mainly on the characterization of barriers in the framework of nuclear, domestic or industrial waste storage likely to pollute the groundwater. Their purpose was to select the natural deposits whose materials have the appropriate physicochemical and mechanical characteristics to the construction of engineered barriers. Obtained results highlight the influence of the temperature on the hydric behaviour of tested soils. It will retain especially that the local hydration of the barriers and the highly heat generated by the radioactive wastes modify the transport coefficients and induce swelling or shrinkage strains. However, some aspects of the behaviour of heavily overconsolidated expansive natural clays located on surface are not yet clearly established for their valorization in road construction. Therefore, this paper aims at characterizing the physical and mechanical properties of an overconsolidated expansive clay from Sidi-Hadjrès city (Province of M'sila, Algeria), which can play an important role in the variation of its volume in order to establish a predictive model of the soil movement following a hydric variation of the site. Some aspects, such as the compaction energy, drying time, initial deformability and saturation of tested soil samples are firstly presented and discussed. Their compressibility and consolidation, swelling and shrinkage, and shear and failure characteristics are then presented and analyzed. Also, the comparative analysis between these characteristics in intact and compacted states is performed.

Brief description of Sidi-Hadjrès clay

The clay samples come from Sidi-Hadjrès city (Province of M'sila, Algeria), where important disorders frequently appear in road and motorway infrastructures, in urban public utilities, as well as in civil and industrial low-rise structures. The city is located in a semi-arid zone characterized by low precipitations rates and a significant variation in temperature between the winter and summer

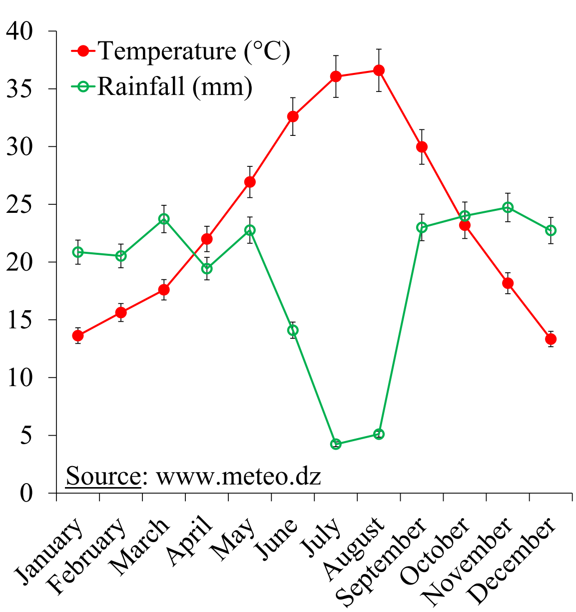

Fig. 1. Typical meteorology of the province of M'sila (Algeria).

seasons (cold and wet winter and hot and dry summer, see Fig. 1). Khemissa et al. [18] indicate that the geology of this zone includes clays formations characterized by high volume change when the conditions of their equilibrium are modified (natural climatic phenomena due to a prolonged drought period, human activity by the modification of the groundwater level due to excessive pumping, configuration of constructions in their environment).

The sampling was carried out starting from four boreholes at a depth of 6.5 m by means of a 100 mm diameter sampler. Visual examination of soil samples and qualitative analysis of their physical properties made it possible to establish a geotechnical profile starting from the surface with a topsoil crust of 30 cm of thickness surmounting a thick marl layer, which is more or less altered at depth with gypsum interbeddings. The boreholes did not indicate the presence of any groundwater table.

Fig. 2 shows the evolution of main geotechnical parameters with depth for the tested soil samples This figure seems to indicate that the soil mass can be considered as homogeneous with depth. Table 1 gives the variation ranges and average values of the main geotechnical parameters of the tested soil samples between 1.3 and 1.7 m of depth. Table 2 gives their chemical composition. According to the Unified Soil Classification System, these soil samples can be described as highly plastic clay (CH), very consistent with important activity based on its clayey fraction ($A_c > 0.6$) due to presence of calcic montmorrillonite. The geochemical analysis conducted on this natural clay shows that the dominating elements are silica (46.63%), alumina (13.92%), carbonates (13.35%), ferrite (5.66%) and magnesia (2.81%). X-ray diffraction analysis illustrates that the main crystalline phases are quartz (21%) and calcite (79%), while the secondary phases are dolomite, feldspars and plagioclases. The oriented spectra technique shows that the clayey phase consists of kaolinite (10%), muscovite-illite (10%), montmorillonite (25%) and sepiolite (55%). Its montmorillonite content being high, this highly plastic marly clay is thus sensitive to water content and is likely to undergo significant volumetric strains (shrinkage or swelling according to the variations of water content). Various identification methods of expansive soils are based on the evaluation of their shrinkage or swelling potentials starting from the measurement of their physical parameters. Fig. 3 presents various classifications of Sidi-Hadjrès clay according to its swelling potential [35,7,43,3] and shrinkage potential [2]. These classifications are all in good agreement and conclude that the Sidi-Hadjrès clay is characterized by high-to-very-high shrinkage and swelling potentials.

Experimental program and testing procedures

In addition to identification conventional tests of Sidi-Hadjrès clay, the experimental program comprises Proctor compaction tests, California bearing ratio tests, oedometric compressibility and consolidation tests, swelling and shrinkage tests, ultrasonic tests, unconfined compression tests, and direct shear tests. These conventional tests require the following comments:

- Standard Proctor test (SPT) and modified Proctor test (MPT) aim, *inter alia*, to examine the impact of compaction energy on the mechanical properties of reconstituted clay and, consequently, help choosing the energy which is most appropriate to it.
- California bearing ratio tests were achieved before (unsoaked CBR) and after (soaked CBR) imbibition of soil samples in order to know the short and long terms bearing capacity of the reconstituted clay and, consequently, check the durability with respect to drying-wetting cycles resulting from a probable variation of the groundwater table.

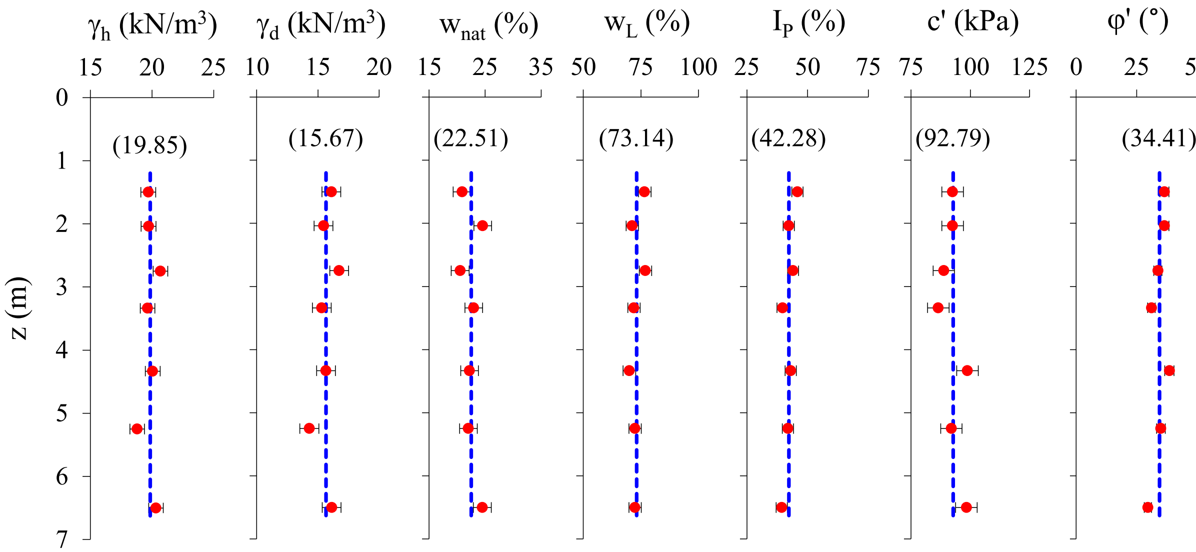

Fig. 2. Geotechnical profile of Sidi-Hadjrès clay.

Table 1
Geotechnical parameters of Sidi-Hadjrès clay.

Parameters	Variation ranges	Mean values
Depth, z (m)	1.30–1.70	1.50
Natural water content, w_{nat} (%)	18.0–23.0	20.9
Specific unit weight, γ_s (kN/m^3)	27.1–28.4	27.8
Humid unit weight, γ_h (kN/m^3)	17.2–21.4	19.8
Dry unit weight, γ_d (kN/m^3)	14.4–18.6	16.8
Degree of saturation, S_r (%)	57.4–91.1	80.9
Liquid limit, w_L (%)	67.2–86.7	78.0
Plastic limit, w_P (%)	25.3–36.6	30.4
Plasticity index, I_P (%)	34.2–65.3	49.9
Consistency index, I_c (%)	1.16–1.28	1.21
Methylene blue value, MBV (%)	7.4–12.0	9.5
Clay content, $C_{2\mu m}$ (%)	17.7–71.9	49.6
Clay activity, A_c	0.60–5.95	1.65

Table 2
Chemical composition of Sidi-Hadjrès clay.

Constituents	Variation ranges (%)	Mean values (%)
SiO_2	41.24–51.84	46.63
Al_2O_3	11.06–15.12	13.92
CaO	9.56–14.66	11.35
Fe_2O_3	4.02–6.22	5.66
MgO	2.14–3.69	2.81
K_2O	1.45–2.81	1.98
SO_3	0.03–5.48	0.79
Na_2O	0.13–1.12	0.40
Cl	0.00–0.37	0.03
Loss of ignition	7.80–19.68	13.25

- Oedometric compressibility and consolidation tests aim at determining the compressibility and consolidation parameters of both intact and compacted clays and, therefore, derive their overconsolidation ratio values in order to control the most appropriate compaction energy to reconstitute the soil by compaction with a performance comparable to the intact soil. They were performed by using oedometers on test specimens of 60 mm in diameter and 20 mm in height, in accordance with the French LPC test methods [25,11]. These testing methods were chosen because of their simplicity of implementation and the experience acquired in the Algerian geotechnical laboratories during several decades. In addition, an analysis of the variability of the results of oedometric tests on a homogeneous series of natural clay specimens confirmed the validity of these test methods [19]. The principle of oedometric compressibility and consolidation tests is described as follows:
  - An incremental compressibility test consists in subjecting a previously saturated soil specimen to a system of loads applied successively in 24 h-increments according to loading and unloading rates which depend on the in-situ soil preconsolidation state. The soil saturation is achieved by supplying water, the level of which will be maintained approximately constant throughout the test. If the soil swells during its saturation, it will gradually be loaded so that no movement can occur. The interpretation of the test results is based on the exploitation of the compressibility, permeability and consolidation curves. The compressibility curve represents the variation of the soil void ratio measured at the end of each loading increment as a function of the logarithm of the effective vertical stress, from which the following compressibility parameters are derived: preconsolidation pressure σ'_p, compression index C_c, swell index C_s, in-situ void ratio e_o, compression ratio $C_c/(1 + e_o)$ and overconsolidation ratio $OCR = \sigma'_p/\sigma'_{vo}$; where σ'_{vo} is the effective overburden pressure. The consolidation curve represents, for a given loading level, the variations of the soil specimen height as a function of the square-root-of-time (Taylor's method), from which the following consolidation parameters are derived: coefficient of consolidation c_v, vertical permeability coefficient k_v, time t_{90} corresponding to 90% of primary consolidation. The permeability curve represents the variations of the soil vertical permeability coefficient as a function of the logarithm of the effective vertical stress corresponding to the stresses greater than the soil preconsolidation pressure, from which the following permeability parameters are derived: variation rate of the soil permeability C_k, initial permeability coefficient k_{vo} and void ratio e_k corresponding to vertical permeability coefficient $k_v = 1$ m/s.
  - A creep test follows the same procedure as the compressibility test described above, but the test specimen is subjected to only one load kept constant until stabilization of the deformations. It takes as many creep tests as loading levels corresponding to stresses greater than the preconsolidation pressure. The interpretation of the creep test

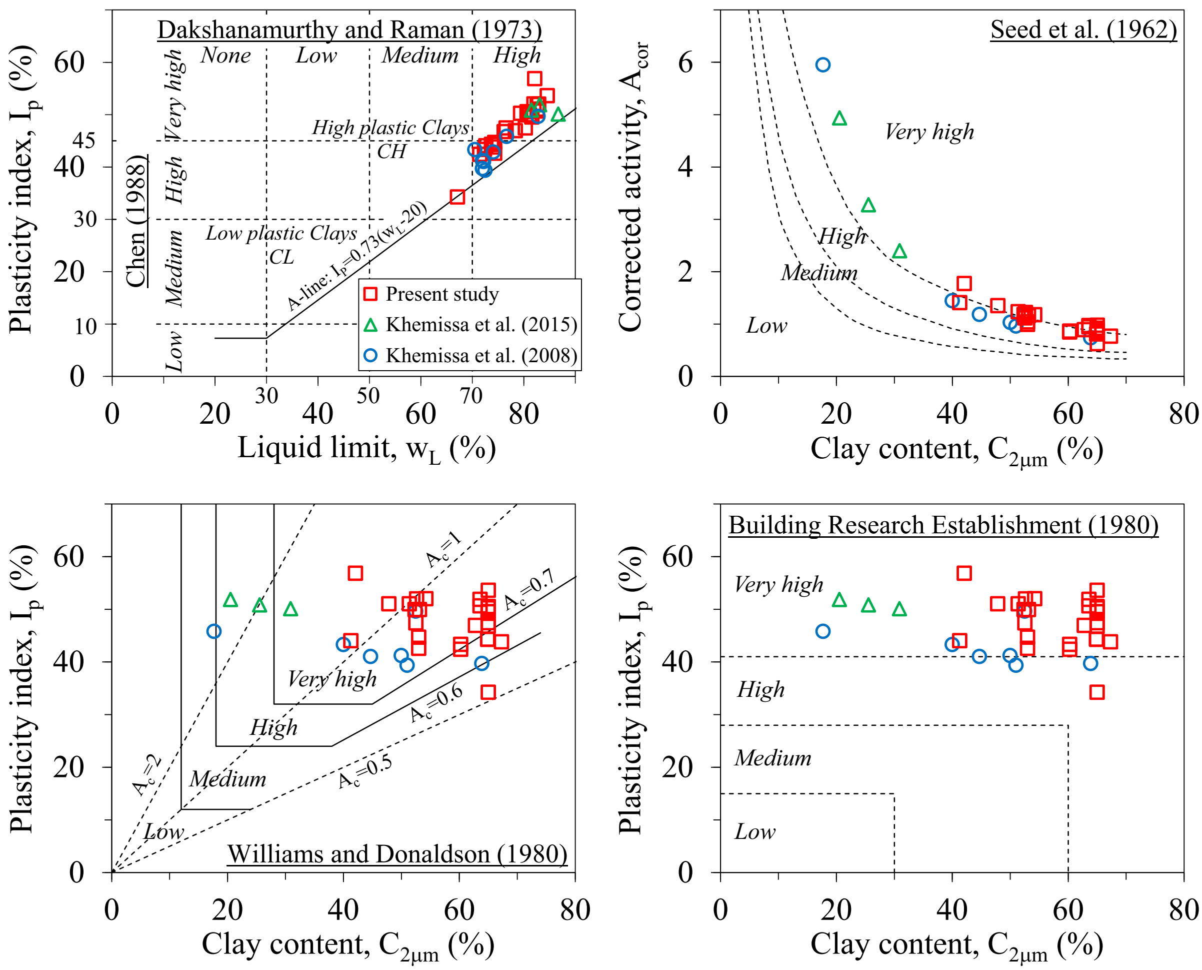

Fig. 3. Classification of Sidi-Hadjrès clay according to its swelling and shrinkage potentials.

results is based on the exploitation of creep curves representing the variations of the test specimen height under applied loads as a function of the logarithm-of-time (Casagrande's method), from which the time t_{100} corresponding to 100% of primary consolidation and creep index $C_{\alpha e}$ will be derived.

- Swelling tests aim at determining the swelling parameters of intact and compacted clay and, therefore, to check the compaction effect on the corresponding swelling potential. They were performed in accordance with the French CEBTP method and the French AFNOR standard [31] by means of conventional oedometers. The execution mode of these tests and their interpretation are explained by several authors [9,38,1,45]. The principle of these two types of swelling tests is described as follows:
  - A swelling test according to the CEBTP method, called "free swelling test", is a direct result of the traditional oedometric test procedure. It is carried out by loading a single test specimen with several loading levels. The test specimen is first subjected to imbibition under weight of the piston. Once the swelling phenomenon is stabilized, the loading is carried out in levels at a suitable loading rate until the deformation is stabilized under each stage. The swelling pressure corresponds to the load which it is necessary to bring the test specimen back to its initial height. By analogy with the construction of an oedometric diagram, the interpretation of the free swelling test results is based on the exploitation of the curve representing the variations of the void ratio as a function of the logarithm of the applied effective stress, from which the swelling pressure σ_g corresponding to the initial void ratio e_i is derived. It is also based on the exploitation of swelling curves representing, for a given load, the variations in the test specimen height as a function of the logarithm-of-time, from which the secondary swelling rate $C_{\alpha s}$ and the time t_{100} corresponding to 100% of primary swelling (the time between primary and secondary swelling) are derived. These curves characterize the kinetic of swelling. They show shapes similar to the consolidation curves, but in the opposite direction. They include a primary swelling phase followed by a secondary swelling phase. The primary swelling phase is due to the migration of water into the test specimen from its ends (diffusion process more or less slow depending on the load, the nature and the material state). The secondary swelling phase is such that the direction of the deformation is opposite to that of the loading (the kinetics of the secondary swelling is generally very slow according to the considered loading level). Let us simply note that the swelling amplitude, known as free swelling, is the maximum deformation undergone by the test specimen without loading (an initial load of 5–10 kPa depending on the soil consistency is applied to the test specimen to restore its initial state in-situ). In the case of medium to high swelling soils, the free swelling test may cause a structural change during the swelling before returning to zero strain. This is why it tends to overestimate the swelling pressure.

– A swelling test according to the AFNOR standard, called "swelling test in parallel", consists in using several test specimens of the same material in different oedometric cells. Each test specimen is loaded in its natural state up to an appropriately chosen axial stress according to a predetermined rate of load increase. The swelling of each test specimen is obtained by carrying out its imbibition under stress until stabilization of the deformations. A load kept constant is therefore applied to each test specimen. The axial deformation corresponding to each of them is first measured without adding water and then with added water. By definition, the swelling pressure is the one corresponding to zero deformation. However, this technique has the disadvantage of testing simultaneously test specimens of possibly heterogeneous materials. It is mainly used in the case of compacted materials, for which it is easier to prepare similar specimens. The interpretation of the test results of the "swelling test in parallel" is based on the exploitation of the curve representing the variations of the soil void ratio as a function of the logarithm of applied effective stress, from which the swelling pressure σ_g corresponding to a zero deformation and the swelling factor R_g are derived. It is also based on the exploitation of the swelling curves identical to those corresponding to the free swelling tests described above.

- A desiccation tests aim at determining the shrinkage parameters of intact and compacted clay and, therefore, to check the compaction effect on the corresponding shrinkage potential. They were performed in accordance with the French AFNOR standards on disturbed [29] and undisturbed [30] specimens. The principle of the desiccation tests on undisturbed specimens consists in allowing an undisturbed soil sample to dry freely and gradually and to periodically measure the variation of its height and mass. The execution mode of these tests and their interpretation are explained by several authors [9,38,1,45]. The interpretation of the desiccation test results is based on the exploitation of the shrinkage curve representing the variations of soil compaction as a function of its water content, from which the desiccation parameters are derived: effective limit shrinkage w_{RE}, linear shrinkage factor R_l and natural water content w_{nat}.
- Unconfined compression tests were performed on cylindrical soil samples (10 cm in diameter and 11 cm in height) by using a hydraulic press, where continuous increasing loads are applied until the failure point of the material. The strain rate was chosen sufficiently high and equal to 1.6 mm/min to ensure that no drainage can occur during the tests.
- Non-destructive ultrasonic tests make it possible to determine the ultrasonic wave celerity in both intact and compacted clay samples. They give therefore an important indication on their deformability and verify the compaction performances on their elasticity moduli. They cannot represent the real behaviour of intact or compacted soils considered as macro-porous and micro-cracked materials (i.e. discontinuous, heterogeneous and anisotropic materials).
- Direct shear tests were performed on saturated and unsaturated soil samples by using a shear box similar to Casagrande shear box. After a possible phase of saturation, the test specimens were consolidated isotropically at various confining pressures (50, 100, 200, 300, and 400 kPa), and then sheared until soil failure. The shear velocity was chosen equal to 1 mm/min for the undrained tests (CU-tests) and to 0.6 mm/min for the drained tests (CD-tests). It should be noted that the saturation of the test specimens was carried out by immersion in the water for 24 h under the only piston pressure of the shear box.

Experimental procedures followed in each type of test and interpretation techniques of the corresponding test results were in accordance as much as possible with usual Algerian geotechnical standards [16] derived from the French geotechnical standards mentioned above (LPC methods, CEBTP method, AFNOR standard). The test specimens were cut out in samples of intact and compacted clay. Compacted soil samples were made starting from the necessary quantity of finely crushed dried soil, then humidified at the optimum water content w_{opt} (i.e. maximum unit weight $\gamma_{d\text{-}max}$) corresponding to a given standard or modified Proctor density. The paste was remixed thoroughly before its compaction and dried at the room temperature ranging between 20 and 25 °C during 7, 14 and 28 days in plastic bags to prevent any possible moisture change.

Test results and discussion

To restrict hereafter the discussion only to representative and significant experimental data dealing with characterization parameters of intact and compacted clays, a preliminary qualitative analysis of all mechanical test results obtained on used soil samples is necessary. Some outcomes can therefore be commented as follows:

- Fig. 4 shows the typical standard and modified Proctor compaction curves of Sidi-Hadjrès clay. Their peaks correspond to the maximum dry density (i.e. the optimum water content) obtained for the considered compaction energy. It can be observed that the modified Proctor compaction test gives a higher maximum dry unit weight $\gamma_{d\text{-}max}$ (i.e. a lower optimum water content w_{opt}) than the standard Proctor compaction test. It can also be noted that the saturation line (SL) corresponding to 75% of soil saturation joins the optimum water content for various compaction energies. The saturation line corresponding to 100% of soil saturation is the envelope of the compaction curves. Experience shows moreover that the compaction mode can also have a considerable influence on the orientation of the soil structure and therefore on its swelling.

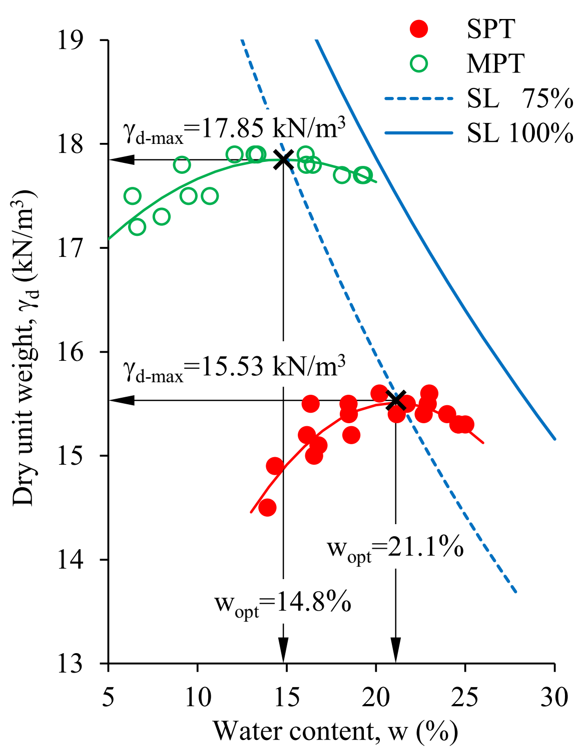

Fig. 4. Typical Proctor compaction curves of Sidi-Hadjrès clay.

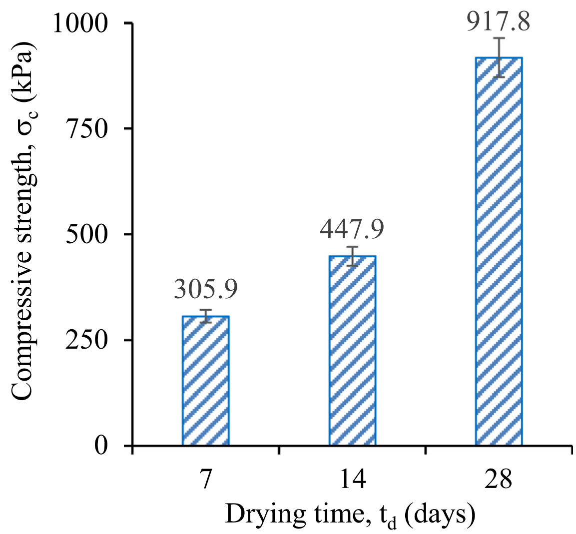

Fig. 5. Drying effect on the compressive strength of the clay compacted at the standard Proctor density.

- The drying time is also a determining parameter of deformability and strength characteristics of the compacted clay. Fig. 5 shows the distribution histogram of compressive strength at various drying times (7, 14 and 28 days) of the clay samples compacted at the standard Proctor density. It can be noted that the compressive strength of the clay increases with increasing drying time. It can be assumed that the drying of the test specimens is accompanied by a shrinkage of the soil. This shrinkage results in a tightening of the soil's grains and, consequently, a reduction of its void ratio, hence the increase of its unconfined compressive strength. It should be noted that the unconfined compression tests carried out on soil samples on the same day of their reconstitution yield no results. As a consequence, only the test results corresponding to soil samples of the same age after a given drying delay can be compared.
- The saturation of intact and compacted soils can affect their resistance and deformability properties. Direct shear tests were therefore performed on soil samples saturated before consolidation and shearing as well in drained as in undrained conditions. Fig. 6 shows the drained and undrained direct shear curves and the corresponding failure envelopes (direct shear curves peaks) obtained on soil samples compacted at the standard Proctor density. The degree of saturation corresponding to unsaturated soil samples used in shear tests varies between 64.1 and 74.6% with an average value of 69.6%. It remains constant because there has been no supply or loss of water from the specimens subjected to the shear tests. These curves confirm the saturation effect of the clay on its shear strength. They show moreover that, for both drained and undrained shear conditions, the sat-

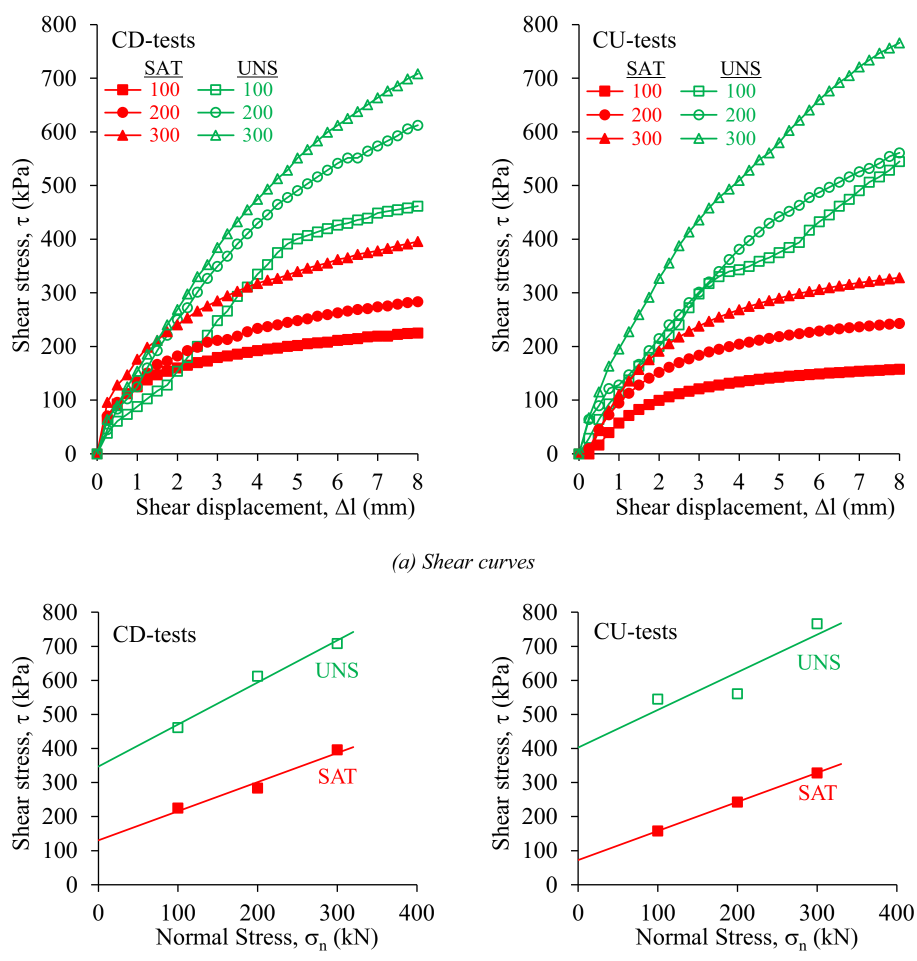

Fig. 6. Soil sample saturation effect on the drained and undrained shear strength of the clay compacted at the standard Proctor density.

urated soil samples (SAT) are characterized by lower shear strength values than those of unsaturated soil samples (UNS). It will be noted that, at the last point of each shear curve, the relative deformation of the corresponding test specimen reaches 13%. The standard used in this study recommends stopping the direct shear test when the shear stress has become constant, or if it continues to grow when a relative deformation of about 10% has been reached. At this deformation level, the soil failure is therefore occurred. Also, it can be noted that, unlike the cohesion, the internal friction angle does not seem to be affected by the saturation of the soil sample.

- Fig. 7 shows the oedometric compressibility and consolidation curves obtained on an intact soil sample. These results seem to show that the adopted LPC methods apply well for studying the oedometric compressibility and consolidation behaviour of this natural clay. However, the corresponding compressibility and consolidation parameters values depend on the considered depth and the applied loading level (for loading levels lower than the swelling pressure, there is no measurable settlement because the soil continues to swell). The parameters that are obtained call for some remarks. The initial and in-situ void ratios values are substantially equal ($e_i = 0.60$ and $e_o = 0.61$ respectively). These results seem to indicate that the soil has not disturbed. They testify the good storage of the used soil samples, which retained the same in-situ and laboratory geotechnical properties. The overconsolidation ratio $OCR = 13.1$ confirms that it is a stiff natural clay. The compression ratio $C_c/(1 + e_o) = 0.14$ confirms that it is not very compressible. The coefficient of consolidation c_v varying between 2.72×10^{-8} and 6.12×10^{-8} m^2/s and the corresponding permeability coefficient k_v varying between 4×10^{-12} and 18×10^{-12} m/s show that it has a very low permeability. These test results indicate that this plastic clay is therefore heavily overconsolidated and has a low permeability. Its overconsolidation seems to be due to a shrinkage phenomenon resulting from a more or less intense desiccation.
- Fig. 8 shows the oedometric creep curves for various loading levels surrounding the soil preconsolidation pressure obtained on intact soil samples corresponding to two different boreholes and depths from the same site. These results seem also to show that the adopted LPC methods apply well for studying the oedometric creep behaviour of this natural clay. The creep index $c_{\alpha e}$ varying between 0.001 and 0.004 does not show a significant influence of the time effect on the behaviour of this stiff natural clay that seem therefore less sensitive to creep.
- Fig. 9 shows the oedometric swelling curves obtained on intact soil samples by two different French testing methods (CEBTP method and AFNOR standard). These results show that the values of the swelling parameters depend on the testing method. The values of the preconsolidation pressure and the free swelling amplitude obtained by the AFNOR method ($\sigma'_p = 437$ kPa and $\varepsilon_{fs} = 63.6\%$) are higher than the on obtained by the CEBTP method ($\sigma'_p = 425$ kPa and $\varepsilon_{fs} = 54.0\%$). While the values of the secondary swelling rate and the time corresponding to 100% of primary swelling obtained by the AFNOR method ($C_{\alpha s} = 0.029$ and $t_{100} = 2959$ min) are lower than the on obtained by the CEBTP method ($C_{\alpha s} = 0.076$ and $t_{100} = 4096$ min). These results are in good agreement with the conclusions given by several authors ([39,14,22], etc.). In all cases, according to Komornik and David [21] and Seed et al. [35] classifications, the swelling pressure values higher than 300 kPa and the free swelling values higher than 25% correspond to a very high swelling potential for this clay. Corresponding values also confirm the previous classifications based on the physical parameters.
- Fig. 10 shows the desiccation curves also obtained on intact soil samples taken from two different boreholes and two different depths from the same site. These results show that the values

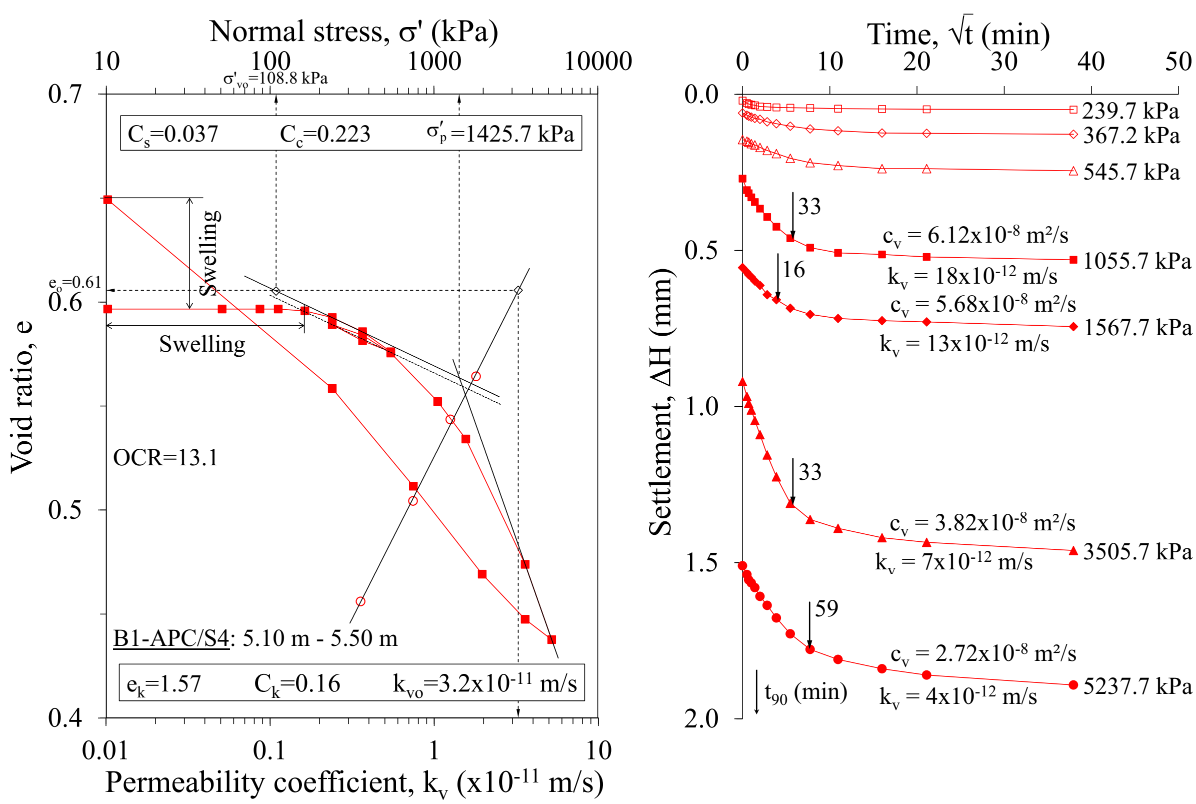

Fig. 7. Oedometric compressibility and consolidation test results obtained on an intact soil sample.

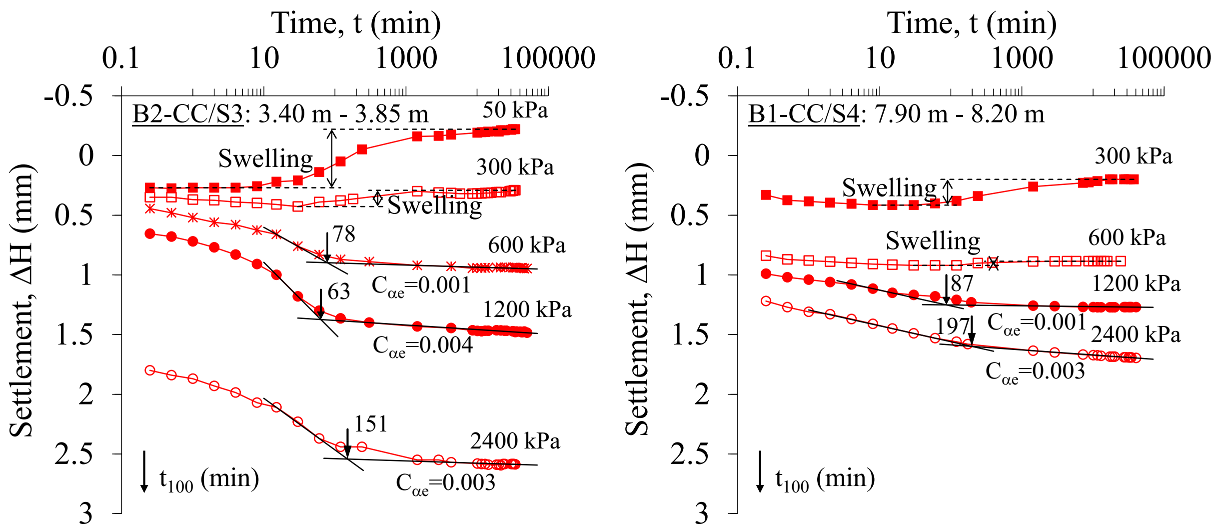

Fig. 8. Oedometric creep test results for two intact soil samples taken in two different boreholes and depths.

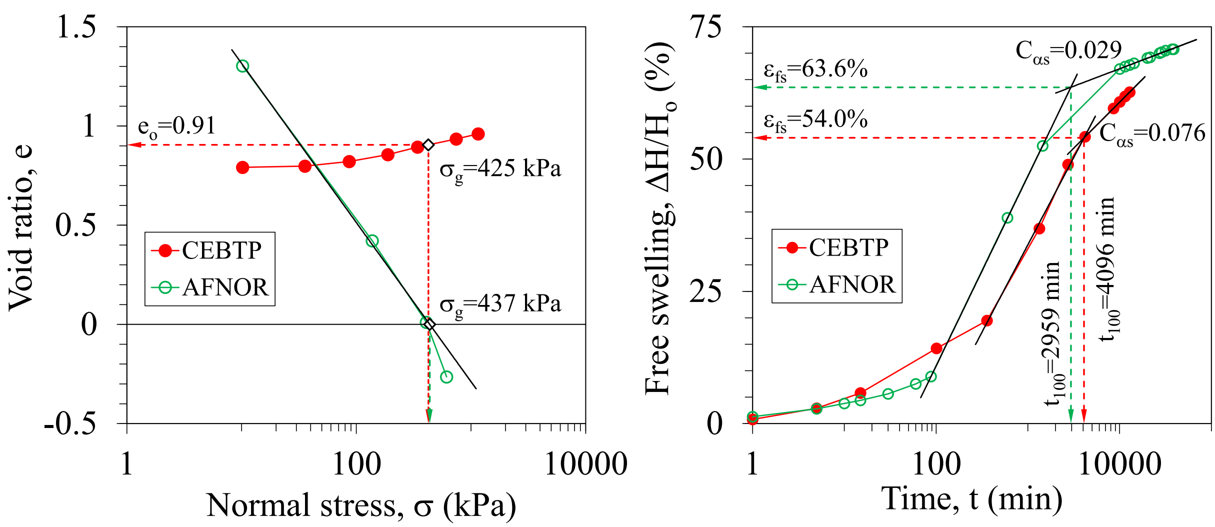

Fig. 9. Oedometric swelling test results obtained on intact soil samples.

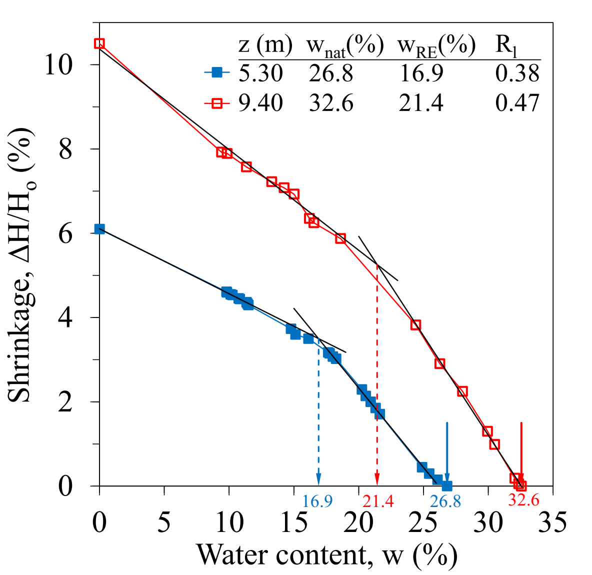

Fig. 10. Oedometric desiccation test results obtained on intact soil samples.

of the effective shrinkage and the limit linear shrinkage factor are also in agreement with previous identification test results. However, these values increase with the natural water content and with the considered depth. In all cases, high swelling and shrinkage potentials values can be explained by the high plasticity of this natural clay.

Only the performed tests regarding the deformability and strength characteristics of the clay in intact and compacted states and their comparative analysis are presented and discussed hereafter.

Compressibility and consolidation characteristics

Fig. 11 shows the oedometric compressibility curves obtained on three soil samples: an intact soil sample and two others reconstituted by compaction at the standard and modified Proctor densities. Fig. 12 shows the corresponding oedometric creep curves under the 1200 kPa and 2400 kPa loading levels. Comparative analysis of the compressibility and consolidation characteristics of Sidi-Hadjrès clay leads to the following comments:

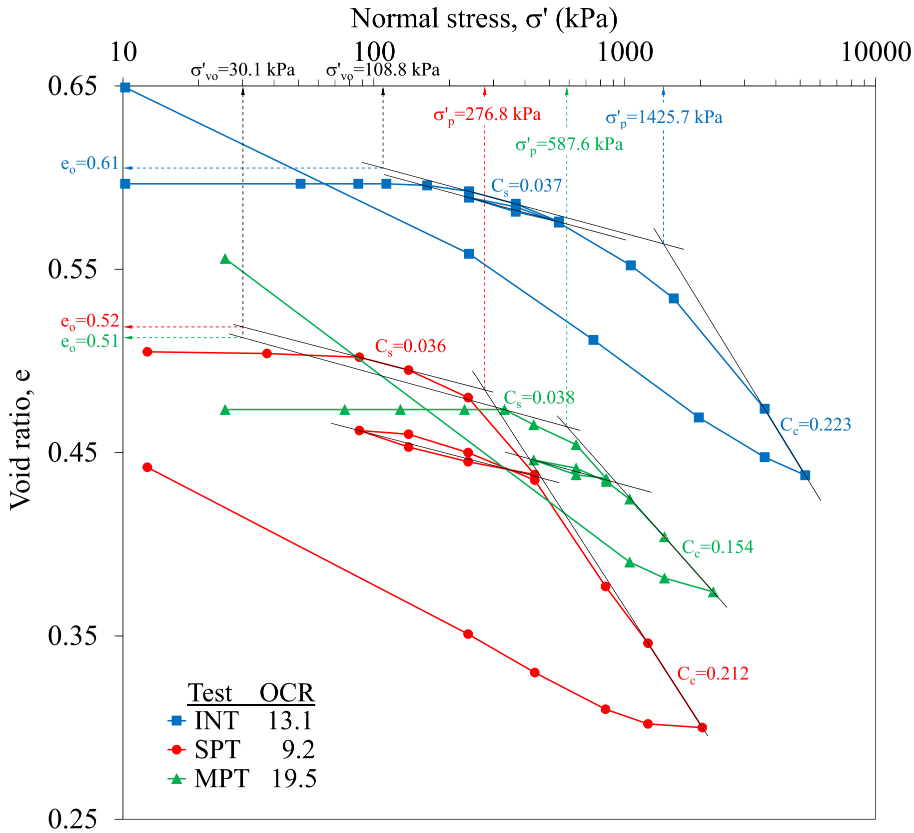

Fig. 11. Oedometric compressibility curves obtained on intact and compacted soil samples.

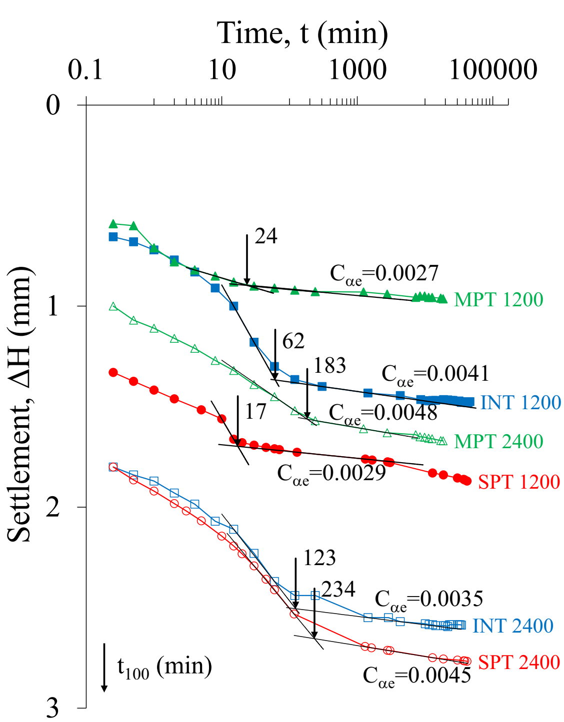

Fig. 12. Oedometric creep curves obtained on intact and compacted soil samples under the 1200 kPa and 2400 kPa loading stages.

- In the loading phase, the compressibility curves of intact and compacted test specimens are characterized by a settlement which begins to appear only after a few loading levels exceeding the overburden pressure during which the soil continues to swell (four loading levels in the case of the intact soil sample). In the unloading-reloading phase, these curves are characterized by highly accentuated slopes to which correspond high values of the swelling index. Final unloading curves that intersect the initial loading curves thus show the expansive nature of the soil. Linearizable sections of the compressibility curves also confirm the existence of two domains limited by the soil preconsolidation pressure for both intact and compacted soil samples: the overconsolidated domain (corresponding to low loads where deformations are small and reversible) and the normally consolidated domain (corresponding to high loads where deformations are more important and largely irreversible).
- In the overconsolidated domain where the loading levels are lower than the soil preconsolidation pressure, the consolidation and creep curves do not make it possible to distinguish the primary and secondary consolidation phases (see Figs. 7 and 8 for the intact soil sample, for example). However, their shape indicates a slight soil swelling (more or less important depending on the considered loading level) making it possible to define the pressure to be applied to soil in order to prevent it from swelling.
- In the normally consolidated domain where the loading levels are higher than the soil preconsolidation pressure, these curves clearly show the primary and secondary consolidation phases (Fig. 12). However, linearizable sections of the consolidation and creep curves corresponding to secondary consolidation phase are characterized by steep slopes and, consequently, by low secondary compression ratio values (i.e. low creep index values).
- Corresponding compressibility and consolidation parameters values depend on the soil compacity state. Experience also shows that the preconsolidation pressure increases with depth,

but not the compression and recompression indices. The comparison can therefore be made in terms of the soil overconsolidation ratio. The overconsolidation ratio value of the soil sample compacted at the modified Proctor density (OCR = 19.5) is then higher than that for the soil sample compacted at the standard Proctor density (OCR = 9.2), but both constitute an envelope for the intact soil sample (OCR = 13.1). This can be directly attributed to the compaction energy: a high compaction energy overestimates the OCR-value and vice versa. Furthermore, the compression index values show that the intact soil sample (C_c = 0.223) is more compressible than the soil sample compacted at the standard Proctor density (C_c = 0.212), which is more compressible than the soil sample compacted at the modified Proctor density (C_c = 0.154). On the other hand, the swelling index values of the intact (C_s = 0.037) and compacted soil samples at standard (C_s = 0.037) and modified (C_s = 0.038) Proctor densities confirm the swelling character of this natural clay. Also, the very low creep index values ($0.0016 < C_{\alpha e} < 0.0045$) and the time corresponding to the end of the primary consolidation (17 min < t_{100} < 234 min) suggest that the load applied to the soil is the only one that can have significant influence.

Moreover, these results confirm the conclusions made earlier that this clay remains heavily overconsolidated, with a low permeability and little sensitive to creep after its reconstitution by compaction at the standard and modified Proctor densities.

Shrinkage and swelling characteristics

Fig. 13 shows the free swelling and shrinkage curves obtained on three soil samples: an intact soil sample and two others reconstituted by compaction at standard and modified Proctor densities. Comparative analysis of the shrinkage and swelling characteristics of Sidi-Hadjrès clay leads to the following comments:

- Free swelling curves have similar shapes for both intact and compacted clays. They show two successive swelling phases: a primary swelling phase characterized by an increasing deformation rate and a phase of secondary swelling with a substantially constant deformation rate. Corresponding swelling parameters values depend strongly on the soil compacity: the compacted clay is less expansive than the intact clay (ε_{fs} = 63.6% and $C_{\alpha s}$ = 0.029), and more true for compaction at the modified Proctor density (ε_{fs} = 19.0% and $C_{\alpha s}$ = 0.008) than for compaction at the standard Proctor density (ε_{fs} = 19.6% and $C_{\alpha s}$ = 0.014).
- Desiccation curves also show similar shapes for both intact and compacted clays. Corresponding desiccation parameters values also depend on the soil compacity: the intact clay samples (w_{RE} = 16.9%) are more retractable than the compacted clay samples, and more true for compaction at the modified Proctor density (w_{RE} = 15.3%) than for compaction at the standard Proctor density (w_{RE} = 14.3%).

These results show that the clay is governed by the same swelling and shrinkage laws in both intact and compacted states. However, they show that the compacted clay is less sensitive to swelling-shrinkage cycles than the intact clay.

Shear and failure characteristics

Fig. 14 shows the ultrasonic wave celerity measurements obtained on three soil samples: an intact soil sample and two others reconstituted by compaction at the standard and modified Proctor densities. Fig. 15 shows the corresponding drained direct shear test results. Comparative analysis of the shear and failure characteristics of Sidi-Hadjrès clay leads to the following comments:

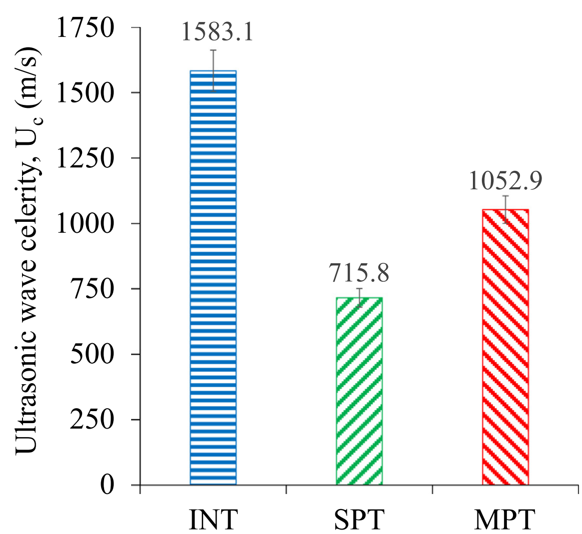

Fig. 14. Celerity of ultrasonic waves obtained on intact and compacted soil samples.

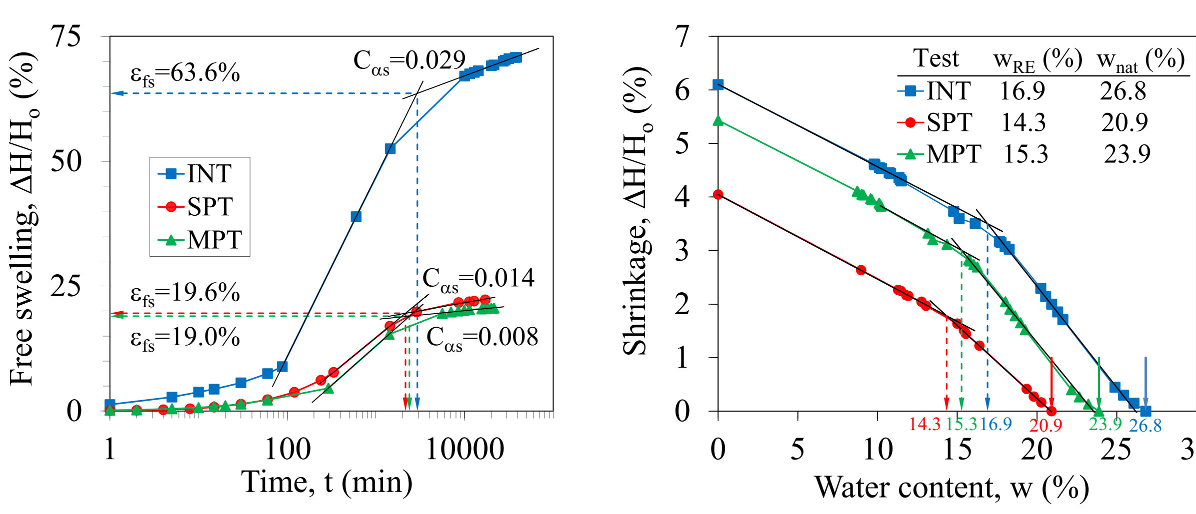

Fig. 13. Free swelling and shrinkage curves obtained on intact and compacted soil samples.

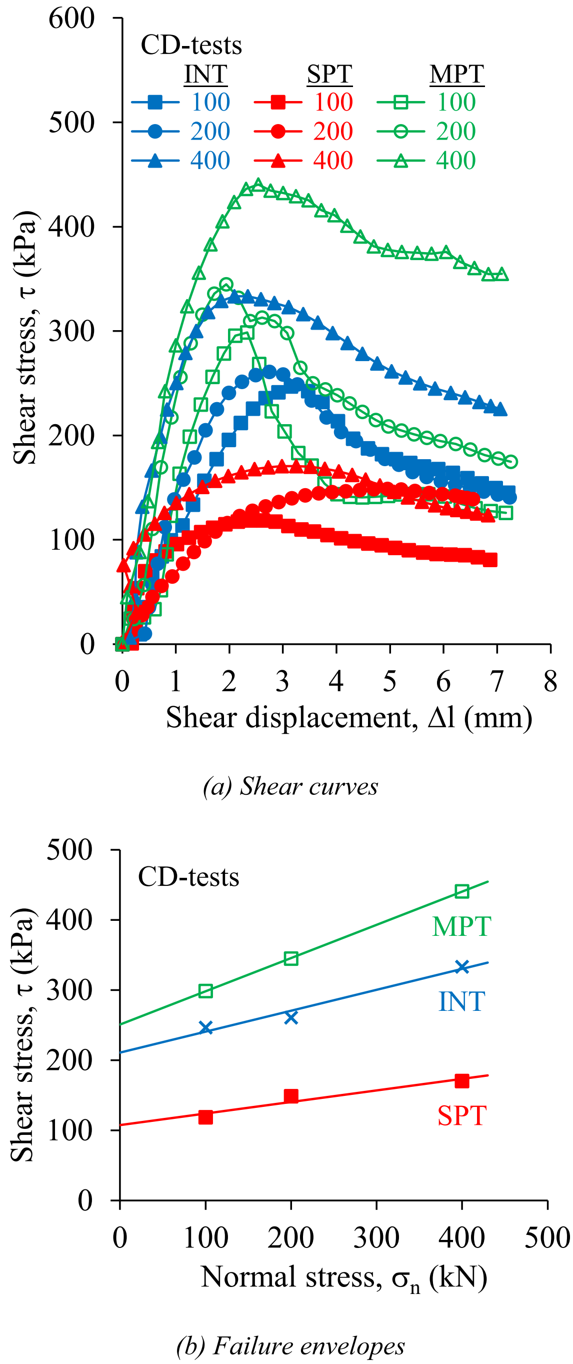

Fig. 15. Drained direct shear test results obtained on intact and compacted soil samples.

- Compared to intact soil samples, the celerity of ultrasonic waves depends on their compacity (i.e. on their compaction energy). It will be noted that the celerity values of ultrasonic waves determined on soil samples compacted at the modified Proctor density (U_c = 1052.9 m/s) are higher than those determined on soil samples compacted at the standard Proctor density (U_c = 715.8 m/s): the higher is the compaction energy, the better is the soil compacity (i.e. its dry density). But, the highest celerity values are obtained on intact soil samples (U_c = 1583.1 m/s). These results seem to indicate that the compaction of clayey soils at the modified Proctor density is more efficient than that at the standard Proctor density. It should also be noted that the higher is the compaction energy, the less deformable is the clay.
- The drained direct shear curves and the corresponding failure envelopes obtained on the intact and compacted soil samples confirm the compaction energy effect on the shear strength of the clay. They show moreover that the soil samples compacted at the modified Proctor density are characterized by a quasi-similar behaviour to that of intact soil samples. The drained shear curves describe a typical behaviour of stiff soils characterized by increasing shear strength followed by a more or less pronounced peak according to the level of the considered normal stresses. It will be noted that the shear strength values of soil samples compacted at the standard and modified Proctor densities constitute an envelope for the intact soil sample.

Summary and conclusions

This paper aims at characterizing the laboratory behaviour of soil samples come from the Sidi-Hadjrès city (Province of M'sila, Algeria). The choice of this urban site was justified because of its extension towards hazardous zones, where significant damages frequently appear in road and motorway infrastructures, in urban public utilities, as well as in civil and industrial low-rise structures.

Based on the results obtained on these soil samples, a comparative analysis was carried out between their strength and deformability characteristics in both intact and compacted states. The main conclusions are as follows:

- The tested soil was identified as a highly plastic marly clay. Various classifications based on the geotechnical properties show that this natural clay is characterized by a very high swelling potential. The swelling of this clay is partly due to its mineralogical structure (high montmorillonite content) and variations in its water content (desiccation-humidification cycles).
- The test results show that the geotechnical parameters values derived from tests performed on this clay are in good agreement, but depend on some experimental aspects already verified on others clayey soils, such as:
  - the compaction energy: the higher it is, the better is the shear strength of the clay and vice versa;
  - the drying time: the older is the clay, the better is its shear strength;
  - the initial deformability has the effect of reducing the shear strength of the clay;
  - the degree of saturation of the clay affects its shear strength: the saturated clay is less resistant to shear than the unsaturated clay.
- The compressibility and consolidation characteristics show that this highly plastic natural clay is heavily overconsolidated, with a low permeability and not very sensitive to creep. Its overconsolidation is due to the phenomenon of shrinkage resulting from a more or less intense desiccation.
- The shrinkage and swelling characteristics show that this heavily overconsolidated natural clay is characterized by very high values of swelling pressure and free swelling, as well as by very high conventional and effective shrinkage values. These values confirm the different classifications based on the physical parameters, but they strongly depend on the test method considered.
- The shear and failure characteristics show that this heavily overconsolidated expansive clay is characterized by a typical behaviour of stiff clays.

- For a given overconsolidation ratio, the macroscopic behaviour of intact and compacted clays is governed by the same laws of compressibility and consolidation, shrinkage and swelling, as well as shear and failure.

In addition, these results will be used to accumulate knowledge for better understanding the macroscopic behaviour of heavily overconsolidated expansive soils. For practical applications, the obtained data will be used to determine the parameters for calculating geotechnical structures.

Acknowledgements

Authors are grateful to the Editor and Reviewers for their useful comments and suggestions that contributed to the improvement of this paper. They also wish to express their gratitude to Prof. Roger Frank (Ecole des Ponts ParisTech, France), Immediate Past President of ISSMGE, and to Prof. Mounir Bouassida (ENIT & University of Tunis El-Manar, Tunisia), former Vice-President of ISSMGE for Africa, for the proofreading of the final version of the manuscript.

Funding

This study was financially supported by the Ministry of Higher Education and Scientific Research, Algeria (CNEPRU project No. J0405620110006 from January 2012 to December 2014).

Appendix A. Supplementary material

Supplementary data associated with this article can be found, in the online version, at https://doi.org/10.1016/j.trgeo.2017.12.003.

References

[1] Bigot G, Zerhouni MI. Retrait, gonflement et tassement des sols fins. Bulletin des Laboratoires des Ponts et Chaussées France 2000;229:105–14.
[2] BRE-UK. UK building regulations. Watford, UK: The Building Research Establishment; 1980.
[3] Chen FH. Foundations on expansive soils, vol. 54. Amsterdam: Developments in Geotechnical Engineering Elsevier; 1988. p. 464.
[4] Coulon H. Propriétés physico-chimiques de sédiments argileux français: contribution au stockage de déchets radioactifs. Doctoral Thesis, Université des Sciences et Techniques de Lille Flandres-Artois, France; 1987.
[5] Cui YJ, Yahia-Aissa M, Delage P. A model for the volume change behavior of heavily compacted swelling clays. Eng Geol 2002;64(2–3):233–50.
[6] Delage P, Howat MD, Cui YJ. The relationship between suction and swelling properties in a heavily compacted unsaturated clay. Eng Geol 1998;50:31–48.
[7] Dakshanamurthy V, Raman V. A simple method of identifying an expansive soil. Soils Found 1973;13:97–104.
[8] Derriche Z, Kebaili M. Prévision du gonflement des argiles d'In-Aménas. Bulletin des Laboratoires des Ponts et Chaussées France 1998;218:15–23.
[9] Didier G, Soyez B, Héritier B, Parez L. Etude à l'œdomètre du gonflement des sols. Comptes rendus du 9ème Congrès Européen de Mécanique des Sols et des Travaux de Fondations 1987;5:549–52.
[10] Djedid A, Bekkouche A, Aissa Mamoune SM. Identification et prévision de quelques sols gonflants de la région de Tlemcen (Algérie). Bulletin des Laboratoires des Ponts et Chaussées France 2001;233:67–75.
[11] Ducasse P, Mieussens C, Moreau M, Soyez B. Oedometric testing in Laboratoires des Ponts et Chaussées, France. In: Young RN, Townsend FC, editors. Consolidation of soils: testing and evaluation, ASTM STP 892, Philadelphia; 1986. p. 282–98.
[12] Gens A. Constitutive modelling: application to compacted soils. Unsaturated Clays 1995;3:1179–200.
[13] Gueddouda MK. Comportement hydro-mécanique des sols compactés : Application à la conception d'une barrière ouvragée « sable de dune – bentonite » Doctoral Thesis. Algeria: University of Tlemcen; 2011.
[14] Hachichi A, Fleureau JM. Caractérisation et stabilisation de quelques sols gonflants d'Algérie. Revue Française de Géotechnique 1999;86:37–51.
[15] Hueckel T, Peano A. Thermo-mechanics of clays and clay barriers. Spec Issue Eng Geol 1996;41:1–301.
[16] IANOR. Recueil des normes algériennes: secteur des travaux publics. Algérie: Institut algérien de normalisation; 2010.
[17] Khemissa M. Validity criteria of oedometric and triaxial tests results. Geotech Geol Eng 2016;34(4):1171–80.
[18] Khemissa M, Mekki L, Bakir N. Comportement œdométrique des argiles expansives de M'sila (Algérie). In: Actes du Symposium International Sécheresse et Constructions, Presses du Laboratoire Central des Ponts et Chaussées, France; 2008. p. 229–34.
[19] Khemissa M, Magnan JP. Validité des résultats d'essais œdométriques sur l'argile molle de Guiche. Bulletin des Laboratoires des Ponts et Chaussées France 2000;227:41–50.
[20] Khemissa M. Mesure de la perméabilité des argiles sous contraintes et température. Revue française de Géotechnique 1998;82:1–12.
[21] Komornik A, David D. Prediction of swelling pressure of clays. J Soil Mech Found Eng Div 1969;95(1):209–25.
[22] Lancelot L, Shahrour I, Khaddaj S. Etude en laboratoire du gonflement de l'argile des Flandres. *PARAM 2002*, Laboratoire Central des Ponts et Chaussées et Presses de l'Ecole Nationale des Ponts et Chaussées, France; 2002. p. 145–51.
[23] LCPC-SETRA. Guide des terrassements routiers: Réalisation des remblais et des couches de forme. Guide technique, Service d'Etudes Techniques des Routes et Autoroutes, Bagneux, France; 2000.
[24] Magnan JP. Panorama des sols gonflants en géotechnique. Bulletin des Laboratoires des Ponts et Chaussées France 2013;280–281:85–103.
[25] Magnan JP, Mieussens C, Soyez B, Vautrain J. Essais œdométriques. Méthodes d'essais LPC, Laboratoire Central des Ponts et Chaussées France, vol. 13; 1985. p. 83.
[26] Mestat P. De la rhéologie des sols à la modélisation des ouvrages géotechniques. Etudes et recherches des Laboratoires des Ponts et Chaussées France; 2000; GT 67. p. 235.
[27] Medjnoun A, Khiatine M, Bahar R. Caractérisation minéralogique et géotechnique des argiles marneuses gonflantes de la région de Médéa, Algérie. Bull Eng Geol Environ 2014;73:1259–72.
[28] Navarro V, Alonso EE. Modeling swelling soils for disposals barriers. Comput Geotech 2000;27:19–43.
[29] Norme XP P94-060-1. Sols : Reconnaissance et essais – Essai de dessiccation – Partie 1: Détermination conventionnelle de la limite de retrait sur le passant à 400 µm d'un matériau. Association Française de Normalisation, AFNOR, France; 1997.
[30] Norme XP P94-060-2. Sols : Reconnaissance et essais – Essai de dessiccation – Partie 2 : Détermination effective de la limite de retrait sur un prélèvement non remanié. Association Française de Normalisation, AFNOR, France; 1997.
[31] Norme XP P94-091. Sols: Reconnaissance et essais – Essai de gonflement à l'œdomètre – Détermination des déformations par chargement de plusieurs éprouvettes. Association Française de Normalisation, AFNOR, France; 1995.
[32] Romero E, Gens A, Lloret A. Temperature effects on the hydraulic behaviour of an unsaturated clay. Geotech Geol Eng 2001;19(3–4):311–32.
[33] Romero E, Villar MV, Lloret A. Thermo-hydro-mechanical behaviour of two heavily overconsolidated clays. Eng Geol 2005;81(3):255–68.
[34] Sanchez M, Gens A, Olivella S. Effect of thermo-coupled processes on the behaviour of a clay barrier submitted to heating and hydration. Ann Brazilian Acad Sci 2010;82(1):153–68.
[35] Seed HB, Woodward RJ, Lundgren R. Prediction of swelling potential for compacted clays. J Soil Mech Found Div 1962;88(SM 4):107–31.
[36] Sellin P, Leupin OX. The use of clay as an engineered barrier in radioactive-waste management – a review. Clays Clay Miner 2013;61(6):477–98.
[37] Selvadurai APS, Letendre A, Hekimi B. Axial flow hydraulic pulse testing of an argillaceous limestone. Environ Earth Sci 2011;64(8):2047–58.
[38] Serratrice JF, Soyez B. Les essais de gonflement. Bulletin des Laboratoires des Ponts et Chaussées France 1996;204:65–85.
[39] Sridharan A, Sreepada R, Sivapullahia PV. Swelling pressure of clays. Geotech Test J 1986;9(1):24–33.
[40] Tang AM, Cui YJ, Barnel N. Thermo-mechanical behaviour of a compacted swelling clay. Géotechnique 2008;58(1):45–54.
[41] Tournassat C, Steefel C, Bourg I, Bergaya F. Natural and engineered clay barriers. Development in clay science, vol. 6, 1st ed. Elsevier; 2015.
[42] Villar MV, Pérez del Villar L, Martin PL, Pelayo M, Fernandez AM, Garralon A, et al. The study of Spanish clays for their use as sealing materials in nuclear waste repositories: 20 years of progress. J Iberian Geol 2006;32(1):15–36.
[43] Williams AB, Donaldson GW. Developments related to building on expansive soils in South Africa: 1973–1980. In: Proc. of the 4th Int. Conf. on Soil Mech. Found. Eng. vol. 2, Denver; 1980. p. 834–44.
[44] Yong RN, Pusch R, Nakano M. Containment of high-level radioactive and hazardous solid wastes with clay barriers. Spon Research, CRC Press; 2017.
[45] Zerhouni MI, Dhouib A, Hubert B. Paramètres de gonflement et retrait des argiles – essais normalisés en France. *PARAM 2002*, Laboratoire Central des Ponts et Chaussées et Presses de l'Ecole Nationale des Ponts et Chaussées, France; 2002, p. 167–71.