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Liquefaction and post-liquefaction behaviour of a soft natural clayey soil

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Abstract. The paper presents the results of identification, monotonous and cyclic triaxial tests on a potentially liquefiable soil from the Guadeloupe island. The material is a very soft clayey soil whose susceptibility to liquefaction is not clear when referring to index properties such as grain size distribution, plasticity, etc. The classifications found in the literature indicate that the material has rather a "clay-like" behaviour, i.e., is not very susceptible to liquefaction, but its properties are very close to the threshold values given by the authors. Cyclic triaxial tests carried out on the material under different conditions show that liquefaction is possible for a relatively important level of cyclic deviator or number of cycles. The second part of the paper is devoted to the study of the recovery of the soil after liquefaction and possibly reconsolidation. For the specimens tested without reconsolidation, that simulated the soil immediately after an earthquake, the recovery is nearly non-existent but the drop in pore pressure during extension results in a small available strength. On the contrary, after reconsolidation, the increase in strength of the liquefied specimens is quite large, compared to the initial state, but with unchanged failure envelopes.

Keywords: soft clayey soil; index properties; cyclic triaxial tests; post-liquefaction recovery.

1. Introduction

Protection of the populations and goods against seismic solicitations requires a good knowledge of the direct and induced effects of earthquakes. Among the induced effects, liquefaction of soils is a spectacular phenomenon that can provoke important damages in the buildings and infrastructures exposed to the earthquake. It is necessary to evaluate the risk of liquefaction of soils and, for the established risks, determine the answer of the structures under the conditions of post-liquefaction and, if possible, propose mitigation methods. The first point is to really understand the behavior of the natural soils and to have predictive modeling tools, validated from in-situ observations. It must be noted that the critical cases are often related to the fact that the soils are "intermediate sediments" (clayey silts, etc.), precisely in the domains in which the present methods lack precision.

To answer these questions, a collaborative project, financed by the French National Research

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Agency (BRGM 2010), was undergone with the two following main objectives:

- to precisely characterize and instrument a research site in the Antilles region (site of Belle Plaine, township of Gosier, Guadeloupe), presenting a significant risk of liquefaction in case of earthquake, in order to improve the knowledge of the liquefaction phenomenon.
- to validate modeling techniques allowing to get predictive tools to study post-liquefaction behavior of soils.

Liquefaction potential of sands and silts has been extensively studied during the past decades. Seed and Idriss (1982), Seed *et al.* (1985), Youd *et al.* (2001), for instance, proposed simple procedures to determine the boundary line between the liquefaction and no-liquefaction zones, based on *SPT* and *CPT* results. The effect of non-plastic fines on liquefaction and pore pressure generation characteristics of saturated sands was also studied by Lade *et al*. (2009) and Dash and Sitharam (2011). However, the investigations concerning clayey sands and clayey silts are far less frequent and the boundary lines for these soils are not so well established (Li *et al.* 2007). The presence of clay increases the plasticity of the soil, modifies its microfabric and decreases its permeability. As long as the quantity of clay or its plasticity is small, it has no effect on the resistance of the soil to liquefaction, whereas, for larger percentages of clay, the resistance to liquefaction increases (Guo and Prakash 1999). This conclusion is confirmed by studies on artificial mixtures of sand and clays (Chang and Hong 2008). The limit between the two behaviors was placed by the authors at PI between 2 and 4% but these values are discussed by many researchers. Most of them agree that it is necessary to distinguish between "clay-like behavior" and "sand-like behavior", but the boundary between both domains is still not very clear. Gratchev *et al.* (2006) related the "sand-like" and "clay-like" behaviors to the microfabric of the soil derived from SEM observations. They studied both artificial mixtures of clay and sand and natural soils, and concluded that soils vulnerable to liquefaction tended to have an open microfabric, in which clay aggregations were generally gathered at the sand particle contact points, serving as low strength "connectors". On the other hand, the microfabric of soils resistant to liquefaction appeared to be more compact, with clay producing a matrix that prevented sand grains from rearranging themselves during cyclic loading. Obviously, the initial density and compressibility of the soil plays a major part in the liquefaction phenomenon and in the increase of excess pore pressure under undrained conditions.

Many authors proposed to evaluate the susceptibility of soils to liquefaction using index properties. A few criteria are indicated in Table 1. According to Boulanger and Idriss (2006), the liquid limit and plasticity index are significant parameters of liquefaction susceptibility, whereas the percentage of fines (< 2 μm or < 5 μm), the w/w_L ratio or the liquidity index are not. The conclusion is that there is no agreement either in the use of index properties for assessing the susceptibility of a clayey soil to liquefaction nor in the parameters to use. Additional experimental results are therefore needed to answer this question.

This paper presents the results of laboratory tests performed on one of the materials of the Belle-Plaine site, a very soft clayey silt, including identification tests, oedometric tests, monotonous and cyclic triaxial tests. The objective is :

- to confront the empirical liquefaction index criteria with the results of the cyclic triaxial tests on this material,
- to examine the possibilities of recovery of the material after liquefaction and reconsolidation.

Table 1 Liquefaction susceptibility criteria from the literature

Reference	Criteria for soils susceptible to liquefaction according to the literature	Values measured for the soil of the present study
"Chinese criteria" Wang 1979	Soils with less than 15-20% fines < 5 μm; $w/w_L > 0.9$	15-25% < 5 μm $w/w_L = 1.1\text{-}1.2$
Seed and Idriss 1982	Soils with less than 15-20% fines < 5 μm; $w_L < 35\%$; $w/w_L > 0.9$	15-25% < 5 μm $w_L = 37\text{-}42\%$; $w/w_L = 1.1\text{-}1.2$
PS 92 guide 1995	For silty sands and silts: $S_r = 100\%$; $C_u = d_{60}/d_{10} < 15$; 0.05 mm < d_{50} < 1.5 mm for clays: $d_{15} > 0.005$ mm; $w_L < 35$; $w/w_L > 0.9$; point situated to the right of the A-line of the plasticity diagram	$S_r = 100\%$; $C_u = 10\text{-}15$; $d_{50} = 0.012\text{-}0.020$ mm; $d_{15} = 0.001\text{-}0.006$ mm $w_L = 37\text{-}42\%$; $w/w_L = 1.1\text{-}1.2$ Point situated on the A-line of the plasticity diagram
Youd 1998	Point situated to the right of the A-line of the plasticity diagram; PI < 7; soils with *L* in USCS classification	Point situated on the A-line of the plasticity diagram; CL-ML
Andrews and Martin 2000	Soils with less than 10% < 2 μm; $w_L < 32\%$	8-16% < 2 μm; $w_L = 37\text{-}42\%$
Polito 2001	High susceptibility: $w_L < 25\%$; PI < 7 medium susceptibility: $25\% < w_L < 35\%$; 7 < PI < 10	$w_L = 37\text{-}42\%$; PI = 11-18
Seed *et al*. 2003	High susceptibility: PI < 12; $w_L < 37\%$; $w/w_L > 0.8$ medium susceptibility: 12 < PI < 20; $37\% < w_L < 47\%$; $w > 0.85\, w_L$	$w_L = 37\text{-}42\%$; PI = 11-18 $w/w_L = 1.1\text{-}1.2$
Boulanger and Idriss 2004, 2006	"Sand-like" behavior: PI < 7	PI = 11-18
Bray and Sancio 2006	High susceptibility: PI < 12; $w/w_L > 0.85$ medium susceptibility: 12 < PI < 20; $w/w_L > 0.8$	PI = 11-18; $w/w_L = 1.1\text{-}1.2$
Chang and Hong 2008	"Sand-like" behavior: soils with less than 20-25% < 2 μm	8-16% < 2 μm

2. Material and methods

2.1 Geological description of the material

The studied material comes from 2 samples (SC1 and SC2) taken on the site of Belle-Plaine, close to the water treatment plant of the city of Gosier, on Guadeloupe "Grande-Terre" (Fig. 1). The Guadeloupe island, in the French Antilles, presents the geodynamic particularity to be astride two sub-meridian volcanic bows. The old bow, to the East, is surmounted by the carbonated platform constituted by the island of Grande Terre whereas, to the west, the Basse Terre is an emergence of the present bow, and the Soufrière volcano a witness of the volcanic activity. To this day, no boring reached the pedestal of the Grande Terre, the oldest deposits known from borings being the

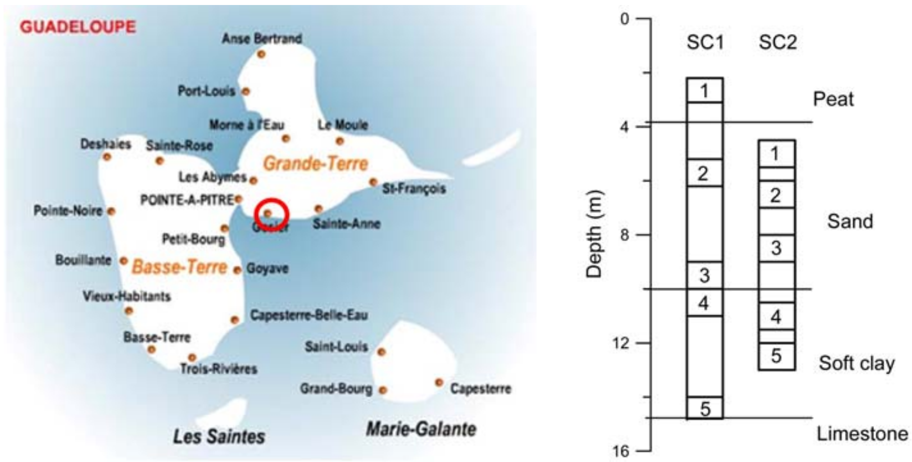

Fig. 1 Study site and localization of the samples

limestones of the lower Pliocène. The plio-pléistocènes carbonated deposits constitute the rocky substratum. In recent Quaternary, the surrection of the Grande Terre led to the emersion of the Deep Funds permitting the development of phenomena of erosion and alteration at the same time as the surface formations were deposited. Among the deposited present formations, one finds the products of the dismantling of the northern volcanic chain of Basse Terre. One also notes the presence of detritic rocks with volcanic terrigen elements, carbonated cementation products (tufite) and consolidated ashes (cinérites) with a marked rocky character, whose extension is limited to the western part of the Pointe isthmus. Above these formations are located the pyroclastic deposits (ashes and lapillis) extensively altered and clayed. Finally, below the present marine level, formations of mangrove swamp of "soft clay" and "clayey sand" types are deposited (Monge *et al.* 1997).

The samples show the existence of 4 levels (Fig. 1): (a) a higher level between 2 and 4 m depth composed of peat with many organic remnants, (b) a level of sand between 4 and 10 m becoming more and more clayey as it goes down, (c) a level of soft clay containing many shellfish between 10 and 14.5 m and (d) the chalk substratum towards 14.5 m. The study that follows was carried out on the SC1- 2, 3, 4 and SC2- 4, 5 samples of level (c) soft clay. Similar studies have been done on the sand and peat but are not presented here.

2.2 Geotechnical identification of the material

The samples were taken from the site by gently pushing a thin wall sampler into the soil, thus reducing the disturbance of the very soft soil to a minimum. They were put in Perspex tubes, which were carefully sealed by wax to maintain the moisture content of the samples, and the tubes were put in wooden boxes to be air transported to France. All the identification tests were performed according to the procedures of the AFNOR standards. The main plasticity and grain size distribution properties, indicated in Table 2 and Fig. 2, show that the soil can be classified in the category *ML* of the clayey low plasticity silts. The corresponding point is situated close to the A line of the Casagrande diagram. This material is very little dense, with a natural water content slightly above the liquid limit.

The permeability of the silt, directly derived from steady state constant head measures, or

Table 2 Geotechnical identification of the material

Liquid limit w_L, %	Plastic limit w_P, %	Plasticity index PI, %	Density of grains γ_s/γ_w	Natural water content w_{nat}, %	Natural dry unit weight γ_{dnat}, kN/m^3
37-42.2	24-26.7	11-18	2.72	40-53	1.05-1.17
Grain size distribution					
< 80 μm %	< 2 μm %	d_{10} mm	d_{50} mm	d_{60} mm	$C_u = d_{60}/d_{10}$
66-94	8-16	0.001-0.003	0.012-0.020	0.015-0.030	10-15

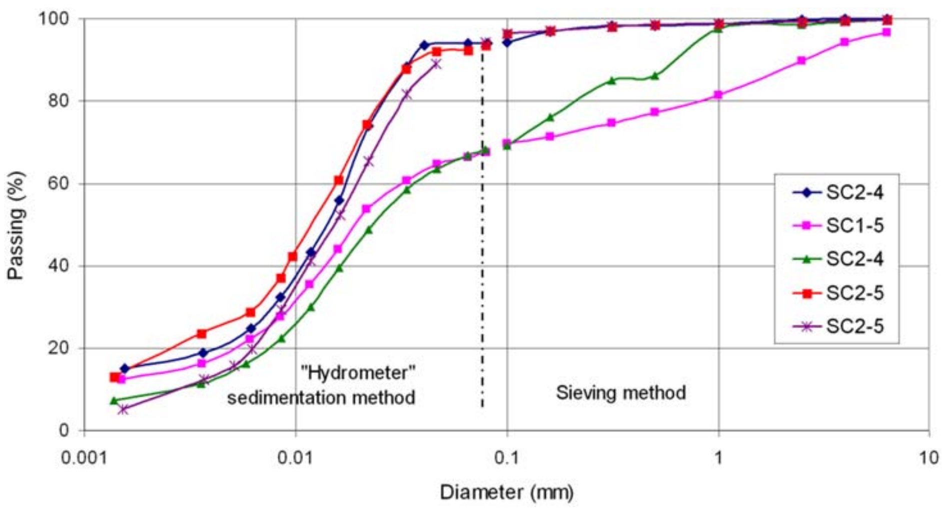

Fig. 2 Grain size analysis of the Belle-Plaine material

indirectly from oedometric tests (using the classical Casagrande method), is comprised between 2 and 5.10^{-10} m/s under low stresses (50 to 100 kPa).

2.3 Triaxial tests

For the triaxial tests, the samples were extracted from their mold and cut to the chosen height, their diameter being equal to that of the core samples. The dimensions of the samples thus obtained are 92 mm in diameter and 160 to 170 mm in height. Before putting the sample, the circuits of the cell were saturated. After setting the sample on the pedestal of the cell and applying the initial conditions (i.e., $\sigma_3 = 35$ kPa and $u = 15$ kPa), saturation was carried out by increasing the confining pressure and the back-pressure simultaneously in a continuous way, while maintaining a difference of 20 kPa between the two, up to a radial pressure of 370 kPa and a back-pressure of 350 kPa. At the end of this step, the value of Skempton B coefficient was checked and the tests were carried out only on specimens with B values larger than 0.95.

In the case of the cyclic tests, the phase of isotropic consolidation was followed by up to 90 stress-controlled undrained compression-extension cycles. Some of the samples that liquefied were re-consolidated under the same value of isotropic effective stress as at the beginning, then they were submitted to a strain-controlled undrained compression-extension test. Others were submitted to undrained compression-extension tests without re-consolidation. For all the tests, loading rates were adjusted according to the pore water pressure equalization criteria of Germaine and Ladd (1988) to obtain reliable assessment of effective stresses.

3. Results

3.1 Oedometric tests

The result of two oedometric tests is shown in Fig. 3. They lead to the following values of the parameters

$$C_c = 0.22;\ C_s = 0.01;\ \sigma'_p = 80 - 100\ \text{kPa}$$

The value of C_c is significantly lower than that given by Skempton's correlation

$$C_c = 0.013\ (w_L - 13) = 0.31 - 0.38$$

The consolidation curve is not very far from the mean *NC* consolidation line of Biarez and Favre (1975) defined by the points $w = w_L$ for $\sigma'_v = 7$ kPa and $w = w_P$ for $\sigma'_v = 1000$ kPa, but its slope is slightly larger. Comparing the initial water content and density of the tested material ($w = 30\%$ and $e = 0.92$) to those of the samples on which the triaxial tests had been performed three months earlier ($w = 50\%$ and $e = 1.30$) shows that desiccation of the material occurred, entailing shrinkage without desaturation of the soil. Putting the samples in contact with water under very low stress at the beginning of the test did not provoke any measurable swelling of the material. Desiccation could be

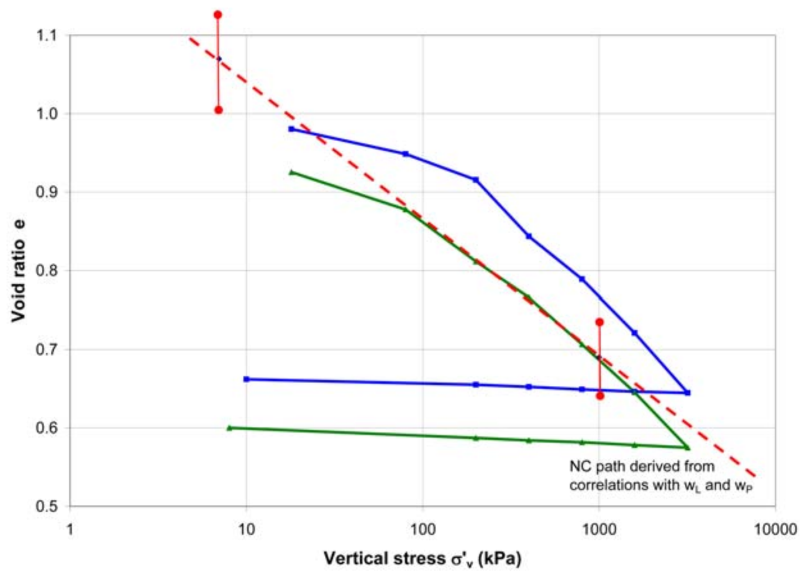

Fig. 3 Oedometric tests on Belle-Plaine material

Table 3 Initial and final conditions of monotonous triaxial tests

	NR1	NR5	NR14	NR2	NR15
Specimen	SC2-4	SC1-5	SC2-5	SC1-5	SC2-4
Depth (m)	11.0	14.2	12.9	14.0	11.1
Consolidation stress (kPa)	50	100	150	200	300
Type of test	CD	CD	CU	CD	CU
Initial water content (%)	50.7	51.9	50.9	52.4	50.0
Initial void ratio	1.30	1.09	1.32	1.13	1.30
Water content at the end of consolidation (%)	36.6	35.1	33.9	34.0	30.6
Void ratio at the end of consolidation	1.00	0.96	0.92	0.92	0.83
Final water content (%)	33.3	30.5	33.9	29.3	30.6
Final void ratio	0.91	0.83	0.92	0.80	0.83

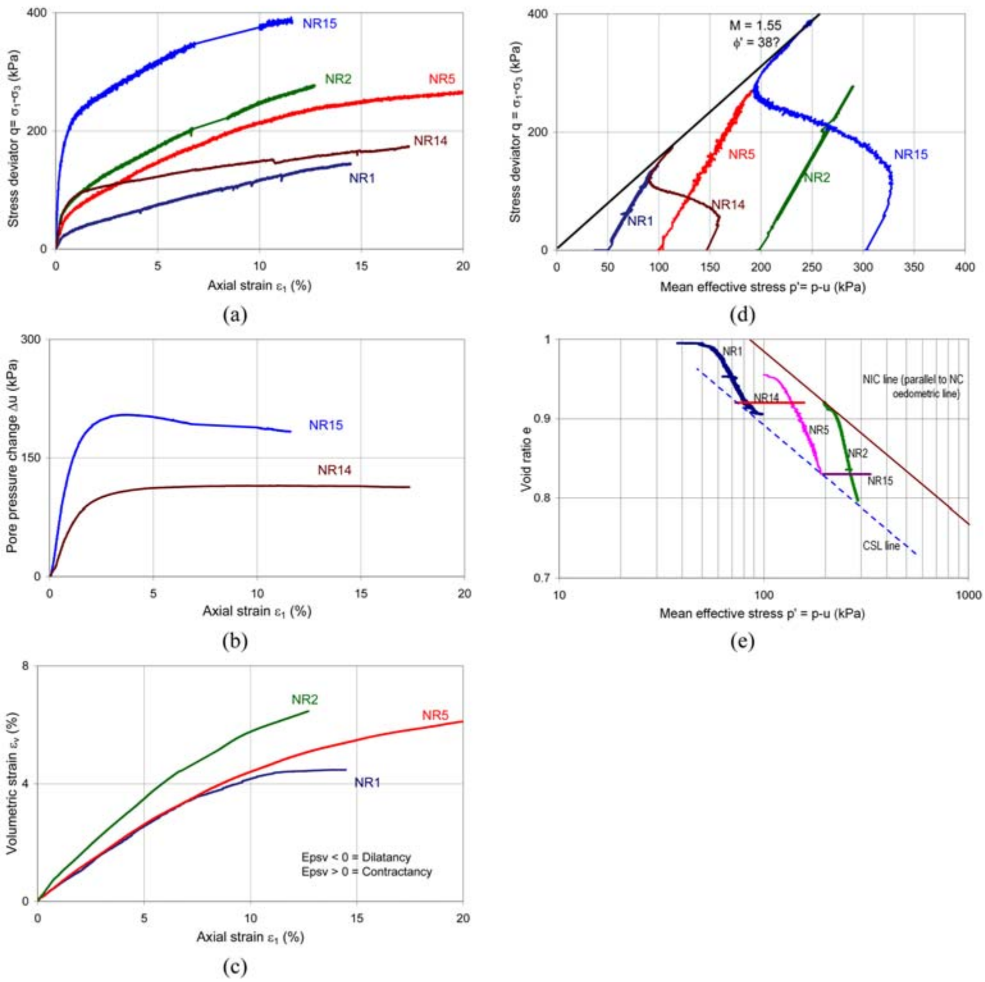

Fig 4 Results of the monotonous drained and undrained triaxial tests (a) Strain-stress curves, (b) Pore pressure versus axial strain for undrained tests, (c) Volumetric strain versus axial strain for drained tests, (d) Stress paths and failure criterion and (e) Void ratio versus effective mean stress and *CSL*

responsible for the relatively high value of the preconsolidation stress, that is difficult to explain, considering the small thickness of the layer above and the fact that the silt has the appearance of a normally consolidated soil.

3.2 Monotonous triaxial tests

The initial and final conditions of the monotonous tests are indicated in Table 3. The initial water contents are relatively homogenous but they decrease sharply after consolidation, which is in agreement with the low density of the samples in their natural state. Comparisons were made with the in-situ water contents at depths corresponding to the same vertical effective stress and were found to be similar. After consolidation, the void ratios and water contents of the specimens are correctly classified with respect to the applied confining stress, despite the initial scatter of densities.

The results of the drained and undrained tests performed on the material are shown in Fig. 4 in the usual planes: stress deviator $q = \sigma_1 - \sigma_3$ versus axial strain ε_1 (Fig. 4(a)), change in pore pressure δu versus axial strain ε_1 for the undrained tests (Fig. 4(b)), volumetric strain σ_v versus axial strain ε_1 for the drained tests (Fig. 4(c)), stress deviator q versus effective mean stress $p' = \frac{\sigma_1 + 2\sigma_3}{3} - u$ (Fig. 4(d)) and void ratio versus the logarithm of effective mean stress (Fig. 4(e)).

The shape of the curves is more or less normal for the NR1 and NR5 tests, but not for the NR2 test that does not seem to have been continued far enough to reach perfect plasticity. The same conclusion may be drawn from the normalization of the curves by the effective consolidation stress, where the curves for the NR1 and NR5 tests are regrouped whereas it is not the case for the curve of the NR2 test. For this test, one does not get a plateau on the volumetric strains vs. axial strain curve, and even less on the stress-strain curve. The plot of the failure criterion is therefore based on the results of the NR1, NR5, NR14 and NR15 tests. The end points of these tests are well enough aligned on a straight line passing by the origin, whose properties are

$$c' = 0;\ M = 1.55;\ \phi' = 38°$$

In the [log(p'); q] plane, it is possible to derive the critical state line (*CSL*) of the material, parallel to the *NC* line of the oedometric tests with a shift of about 0.09 between both, which appears conform to the conclusions of Biarez and Hicher (1994).

3.3 Cyclic triaxial tests

Table 4 Initial and final conditions of the cyclic tests

	NR4	NR22	NR6	NR10	NR21	NR12	NR20
Specimen	SC2-5	SC1-4	SC2-5	SC2-4	SC1-4	SC2-4	SC1-4
Depth (m)	12.0	10.5	12.4	10.9	10.3	10.2	10.1
Consolidation stress (kPa)	50	50	100	100	100	200	200
Type of test	CU	CU	CU	CU	CU	CU	CU
Norm. half cyclic deviator $q_c/2\sigma'_3$	0.2	0.3 0.38	0.4	0.35	0.35	(a) 0.2 (b) 0.3	0.3
Initial water content (%)	52.1	41.5	50.8	38.0	38.0	40.5	35.4
Final water content (%)	44.2	39.6	38.4	32.6	33.4	31.7	31.7
Final void ratio	1.20	1.08	1.02	0.89	0.91	0.84	0.86

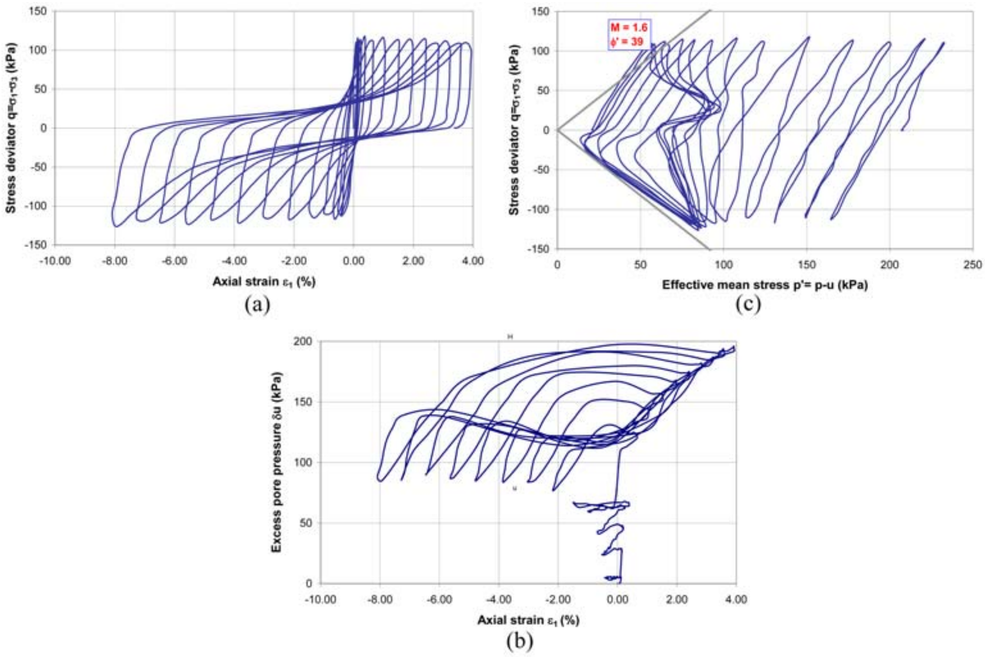

Fig. 5 Results of the cyclic undrained triaxial test on specimen NR20 (a) Strain-stress curves, (b) Pore pressure versus axial strain for undrained tests and (c) Stress paths and failure criterion

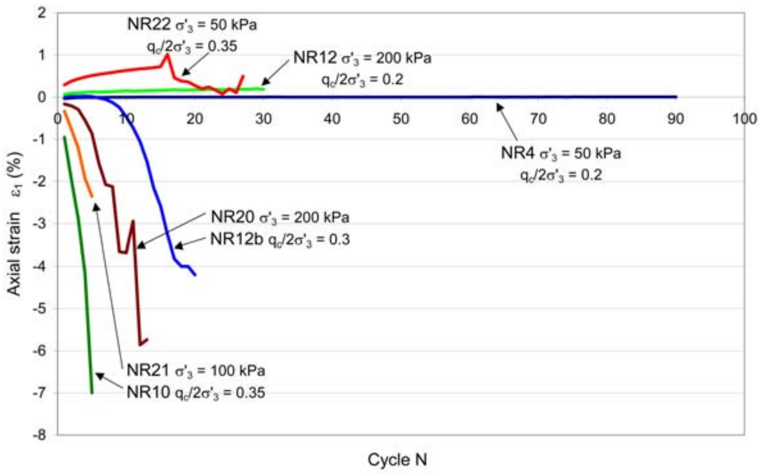

Fig. 6 Axial strain versus number of cycles

The initial and final conditions of the cyclic triaxial tests are indicated in Table 4. A typical result is shown in Fig. 5. According to Boulanger and Idriss (2006), the shape of the strain-stress curve is intermediate between that of a sand and that of a clay. The cyclic tests were performed at a frequency of 1 Hz, in order to be in the same frequency range as the expected earthquake in the region. Various tests were performed on similar specimens at frequencies of 0.1, 0.3, 1 and 3 Hz and no significant difference was found either in the pore pressure build-up or in the axial strain after 3 loading steps corresponding to 100 cycles at $q_c/2\sigma'_{30}$ = 0.15, 0.2 and 0.25. It was therefore concluded that this parameter had not a noticeable influence on the behavior of the soil in this range of values.

Figs. 6 and 7 present the evolution of axial strain and excess pore pressure ratio $r'_u = \Delta u/\sigma'_{30}$ (where Δu is the excess pore pressure and σ'_{30} is the consolidation stress) at the end of each cycle, i.e., when the stress deviator becomes equal to 0. Fig. 8 and Table 5 present the synthesis of the liquefaction tests, i.e., the relationship between the cyclic resistance ratio ($CRR = q/2\sigma'_3$) and the number of cycles leading to liquefaction. It appears that liquefaction of this soil occurs only for *CRR* larger than or equal to 0.3, which means that the material is susceptible to liquefaction, but only in the case of high magnitude earthquakes.

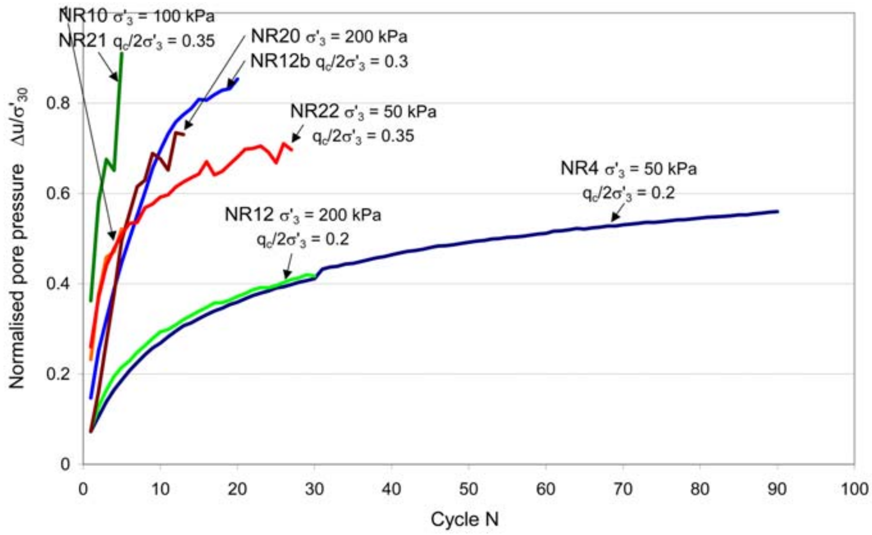

Fig. 7 Pore pressure versus number of cycles

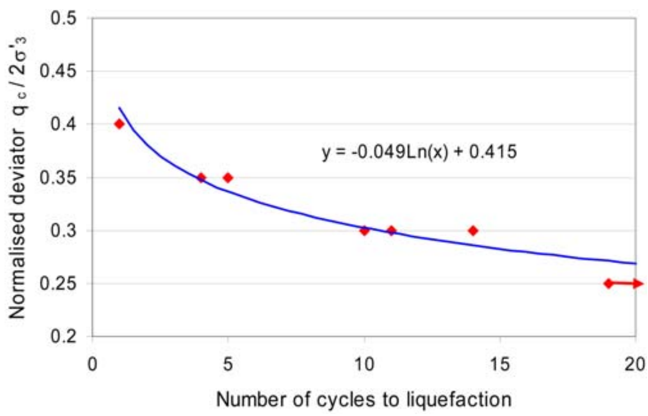

Fig. 8 Normalised cyclic deviator (Cyclic Resistance Ratio, CRR) versus the number of cycles leading to liquefaction of the material

Table 5 Number of cycles to reach liquefaction

Test	NR6	NR10	NR21	NR22	NR20	NR12b	NR12	NR4
CRR = $q/2\sigma'_3$	0.4	0.35	0.35	0.3	0.3	0.3	0.2	0.2
N_{liq}	1	4	5	11	8	14	> 30	> 90

3.4 Undrained compression-extension tests after liquefaction of soil

Three undrained triaxial compression-extension tests (NR 20-21-22) were carried out after reconsolidation on liquefied specimens. Two additional tests (NR 10-12b) were also carried out on liquefied specimens, but without reconsolidation, to simulate the behavior of the soil immediately after the earthquake. The results of these tests are shown in Fig. 9.

If the specimens are allowed to reconsolidate after liquefaction, the conclusion is that there occurs a complete strength recovery of the material, with nearly the same failure criterion as for the initial sample (M = 1.60 instead of 1.55). Comparing the maximum stress deviator in both cases shows that this parameter reaches higher values after liquefaction and reconsolidation than before: the maximum deviators are initially of 180 kPa and 400 kPa under confining stresses of 100 kPa and 300 kPa, whereas they reach 300 kPa and 600 kPa under confining stresses of 100 kPa and 200 kPa, respectively, after liquefaction (Fig. 10). This strength increase is partly due to the additional consolidation of the specimens after liquefaction though the decrease in void ratio due to re-

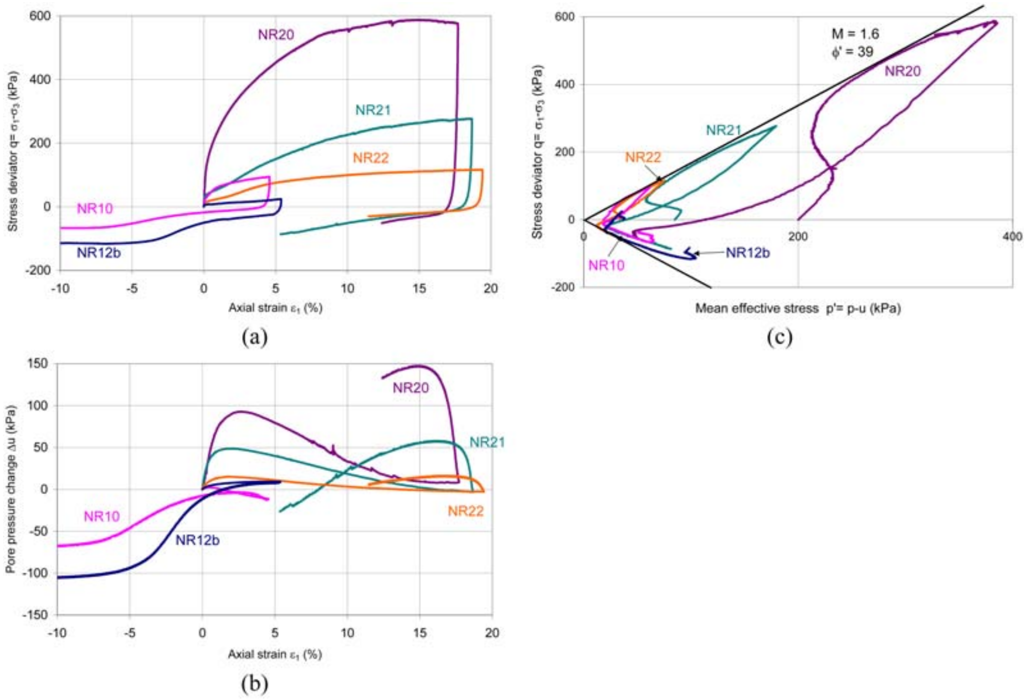

Fig. 9 Results of undrained triaxial compression-extension tests carried out on liquefied specimens without reconsolidation (NR10-12b) or with reconsolidation (NR20-21-22) (a) stress-stain curve, (b) change in pore pressure vs. axial strain and (c) stress paths

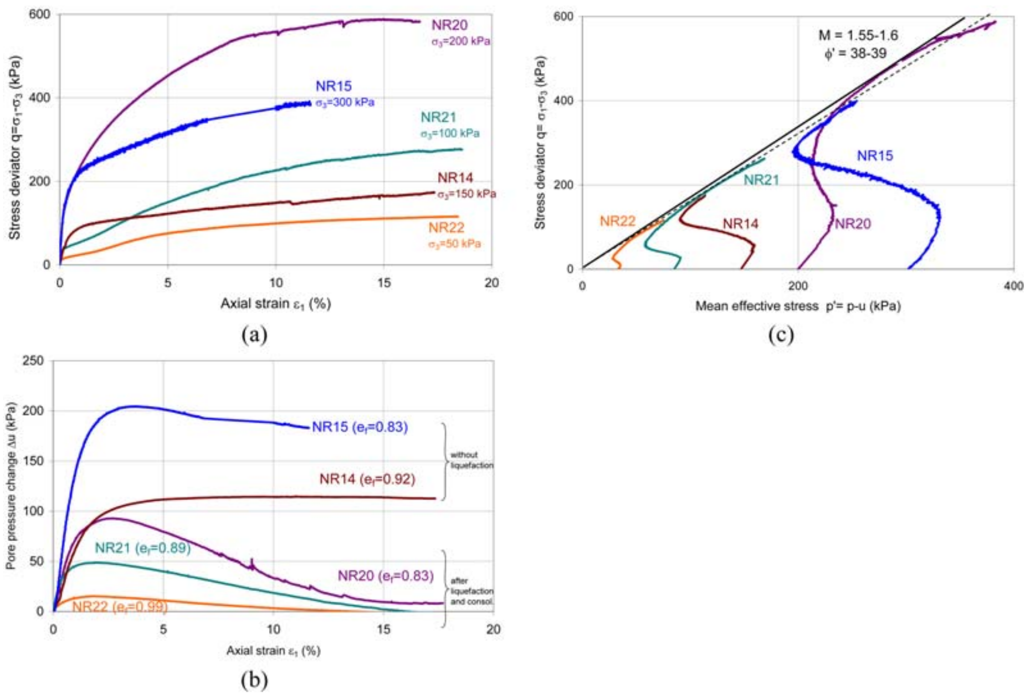

Fig. 10 Comparison between the results of undrained triaxial compression tests carried out on liquefied specimens with reconsolidation (NR20-21-22) and those on the soil in its initial state (NR14-15): (a) stress-stain curve, (b) change in pore pressure vs. axial strain and (c) stress paths

consolidation is moderate: from 1.08 to 0.99 for NR22 (50 kPa), from 0.91 to 0.89 for NR21 (100 kPa) and from 0.86 to 0.79 for NR20 (200 kPa). The effect of consolidation also appears in the shape of the pore pressure vs. axial strain curves: before liquefaction, there is a continuous increase in pore pressure during the compression and the curves reach a plateau whereas the pore pressure curves present a noticeable maximum followed by a sharp decrease after liquefaction, which is usually the result of a high soil density. The contractant behavior which appeared in the specimens before liquefaction evolves towards a dilatant behavior after liquefaction and re-consolidation. Considering the small increase in density due to re-consolidation, this effect may also be the result of a rearrangement of the grains after the disturbance provoked by liquefaction.

On the contrary, if the specimens are not allowed to reconsolidate after liquefaction (NR 10-12b), there is no recovery and the stress deviator remains very low, which is not surprising considering that the fabric of the material has been severely shaken during liquefaction and that its density remains constant. However, during extension, the decrease in pore pressure results in a small increase in stress deviator, but this increase remains very small compared to the initial value before liquefaction.

4. Susceptibility of the soil to liquefaction

Based on the measured index properties, it is clear that the material does not strictly obey the criteria of clays even though it is very near to it. The soil is saturated, with a water content to liquid limit ratio larger than 1. Its grain size distribution is intermediate between those of “sand-like” and “clay-like” soils with a fine content between 10 and 25% and a wide grain size distribution. Its plasticity is rather high, with a liquid limit of 37 to 42% and a plastic limit between 11 and 18%. Looking back on Table 1, where the measured values have been put in front of the criteria found in the literature, it appears that the material nearly satisfies the criteria of Wang (1979), Seed and Idriss (1982), PS92 (1995), Youd (1998), Chang and Hong (2009), always remaining near the threshold vales of most of these criteria. It is further from those of Andrew and Martin (2000) and Boulanger and Idriss (2004, 2006). It is classified in the soils moderately susceptible to liquefaction by Polito (2001) and Bray and Sancio (2006).

The cyclic triaxial tests confirm that liquefaction is possible, but only for a relatively important level of cyclic deviator or number of cycles. As a conclusion, the soft soil from the “Belle-Plaine” site appears as moderately susceptible to liquefaction, as indicated for instance by Polito (2001) and Bray and Sancio (2006).

5. Conclusions

The material considered in this study is a very soft clayey soil from marine origin, for which very few data exist concerning its susceptibility to liquefaction during an earthquake. The criteria based on index properties proposed by several authors give dispersed answers because its properties are intermediate between those of silts and clays. In the case of the studied material, index-based criteria indicate that the soil is moderately susceptible to liquefaction as most of the criteria are met whereas others are only half met, the properties being just below the threshold values of these criteria. In that case, cyclic tests are necessary to assess precisely the risk of liquefaction. The stress-controlled undrained cyclic tests carried out on the material under different conditions proved that, indeed, the soil could be liquefied, but only under relatively high cyclic deviators, or relatively large magnitudes of earthquakes, i.e. for cyclic resistance ratio larger than 0.25. This behavior is consistent with the criteria of Polito (2001) and Bray and Sancio (2006). The conclusion of this study seems to be that the criteria based on plastic properties are best suited than those based on grain size.

The second point of the work was to study the recovery of the soil after liquefaction and, possibly, reconsolidation. Monotonous undrained compression-extension tests showed that the soil was able to fully recover after liquefaction, presenting a higher strength than originally, partly due to a small increase in its dry density resulting from the re-consolidation, and partly to the rearrangement of the grains. On the contrary, for the specimens tested without reconsolidation, that simulated the soil immediately after an earthquake, the recovery is nearly non-existent but the drop in pore pressure during extension tests results in a small available strength. In all the cases, the failure criterion remains the same.

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