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Laboratory Geotechnical Investigations on Five Silty Soils Sampled Along the Banks of the Lubumbashi River/Haut-Katanga/DR Congo

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Abstract. An adequate design of port structures along waterways requires a good knowledge of the geotechnical properties of the surrounding soils. This study aims to characterize the geotechnical laboratory properties of alluvial soils along the Lubumbashi river (section between the Tshombe and Tshondo bridges, approximately 3 km long) in the city of Lubumbashi, province of Haut Katanga in DR Congo. To meet this objective, five sampling zones were selected from which samples were taken with a hand auger and various geotechnical laboratory tests were performed. Identification tests reveal that the tested soils have low to moderate plasticity and are in the range of sandy silt to silty sand with a low fraction of clay particles, essentially inactive. Oedometric compression tests reveal that the soils in place are (moderately) compressible. The consolidated undrained (CU) triaxial compression and direct shear tests indicate effective values of internal friction angle and negligible drained cohesion that, are consistent for this kind of soil. In the end, the compressibility and strength parameters of the five tested soils are correlated with their plasticity index, showing a decrease in the shear strength and an increase in the compressibility when the plasticity index increase.

Introduction

Geotechnical investigation dedicated to the determination of the geomechanical parameters of the encountered soils is of paramount importance in many domains, such as road infrastructure [1], geotechnical projects [2], earthen hydraulic work [3], waste management [4], earthen constructions [5], amongst many others. Soil is a natural material that is a priori unknown and heterogeneous by nature. Consequently, soil investigation is a fundamental step for the success of any project related to soil geomechanical behavior. An orderly and organized planning of the tasks requires the determination of the soil characteristics before the execution of the construction works [2].
Unlike other civil engineering materials, the soils already exist on the site where the work will be carried out. Therefore, it is necessary to carry out adequate geotechnical studies in order to define the nature and the state of the soils on the site. The realization and operation of port works involve geotechnical studies that go through thorough analyses to ensure their safety while minimizing the risks of instability [6]. Any geotechnical study requires in-situ and laboratory tests [7]. The purpose is to determine the strength and deformability parameters such that the geostructures can be designed to satisfy the ultimate and serviceability limit state [8]. Neglecting the geotechnical investigation step and ignoring the geotechnical characteristics of the soil (from the point of view of resistance and deformability) would be a permanent danger to the structure [9].
The present study characterizes by laboratory tests of the alluvial soils collected on five sites along the Lubumbashi river canal, on a section crossing the city of Lubumbashi between Lake Tshombe and the Tshondo bridge. This characterization of local soils is an essential preliminary to the future construction of docking and mooring structures.To achieve the objective of our research, we opted for a laboratory characterization method via a series of sampling of five soils taken from our study area, followed by a test campaign carried out at the Geotechnology Laboratory of the University of Liege (Belgium).

Materials and Methods

Zone of interest. The Lubumbashi River crosses the city of Lubumbashi, capital of the province of Haut Katanga. This province is located in the South-East of the Democratic Republic of Congo (DR. Congo) between parallels -11°30' and -11°50' and meridians 27°17' and 27°40'. It's the second largest city in the country after the city of Kinshasa. It's located in a vast depression bounded to the northeast by the anticline of the Star (1275 m altitude) and southwest by the anticline of Kisanga (1346 m altitude) [10]. The city of Lubumbashi has seven communes that are drained by the Lubumbashi River. The Lubumbashi River is one of the major rivers of the city and has its source in the village Tumbwe in the northwest of the city of Lubumbashi. It crosses the western part of the territory of Kipushi and the city of Lubumbashi where it passes through several communes and neighborhoods before flowing into the Kafubu . The ground along the Lubumbashi river is made of alluvial soils that may have various geomechanical characteristics from stiff and highly bearing gravel to soft and plastic clays. This work considered the section between Lake Tshombe and the Tshondo bridge as a case study (Fig. 1).

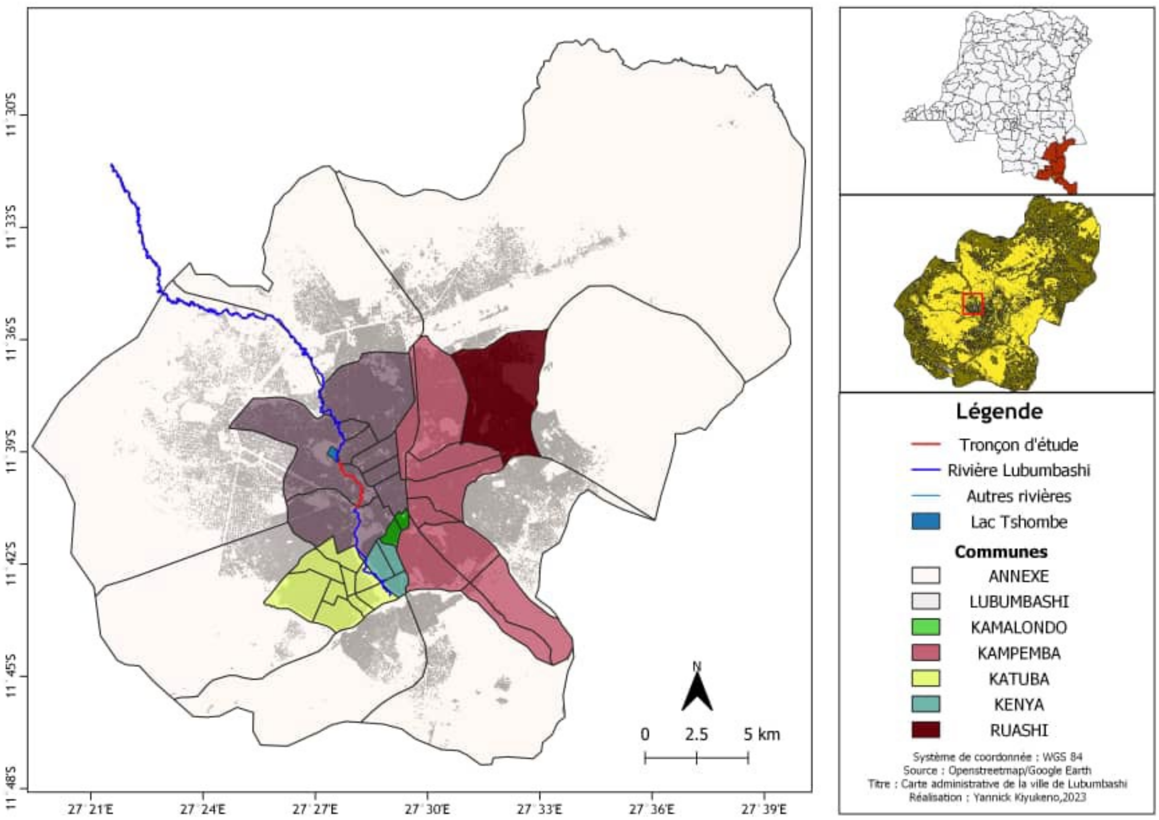


Figure 1: Presentation of the study area.

Methodology. The methodology followed in this study consisted of a sampling campaign along the Lubumbashi river channel (Lake Tshombe-Tshondo Bridge). Samples extracted by hand auger at different depths (Table 1) at 5 different sites were subjected to geotechnical tests in laboratory. The tests were carried out on soil samples previously dried, homogenized and reconstituted to the average dry density in place.

A range of sieve mesh sizes of: 80 mm; 50 mm; 32 mm; 20 mm; 10 mm; 5 mm; 2 mm; 1 mm; 0.4mm; 0.1 mm; 0.08 mm (80 µm) was used for particle size analysis (dry and wet), particle size analysisis continued by sedimentation, according to NF EN ISO 17892-4 [11]. The plasticity index (PI) of the materials was determined by the Atterberg limit test [12]. The methylene blue value (MBV) was determined according to NF P 94-068 [13]. The helium gas pycnometer, allowed us to determine thedensity of solid grains γ_s) [14]. To characterize the soils along the banks of the Lubumbashi River, our study used mechanical tests which included direct shear tests, undrained consolidated triaxial compression tests (UC) as well as oedometer compression tests [15].

Table 1: Geographic coordinates and depth of sampling points.

Collectio npoints	Designation	Geographic coordinates	Depth range (meters)	Sample types
SPTSHB (Ech1)	Bridge floor Tshombe	11°40' 28'' S 27°28' 07'' E 1195 m	0.30 to 1.63	Remolded
SPRWE (Ech2)	Bridge floorRuwe	11°39'46'' S 27°28'54'' E 1216 m	0.22 to 1.61	Remolded
SDOM (Ech3)	Bridge floor domaine marial	11°33'54'' S 27°28'13'' E 1211 m	0.43 to 1.80	Remolded
SPLDO (Ech4)	Bridge floorlido	11°40'11'' S 27°28'12'' E 1201 m	0.37 to 1.54	Remolded
SPTSHD (Ech5)	Bridge floor tshondo	11°40'28'' S 27°28'07'' E 1195 m	0.27 to 1.59	Remolded

Grain size distribution. The grain size analysis by sieving of the soils was carried out up to 80 µm and continued by sedimentation for particles smaller than 80 µm [11]. At the end of this analysis, the grain size distribution curves were realized (Fig. 2).

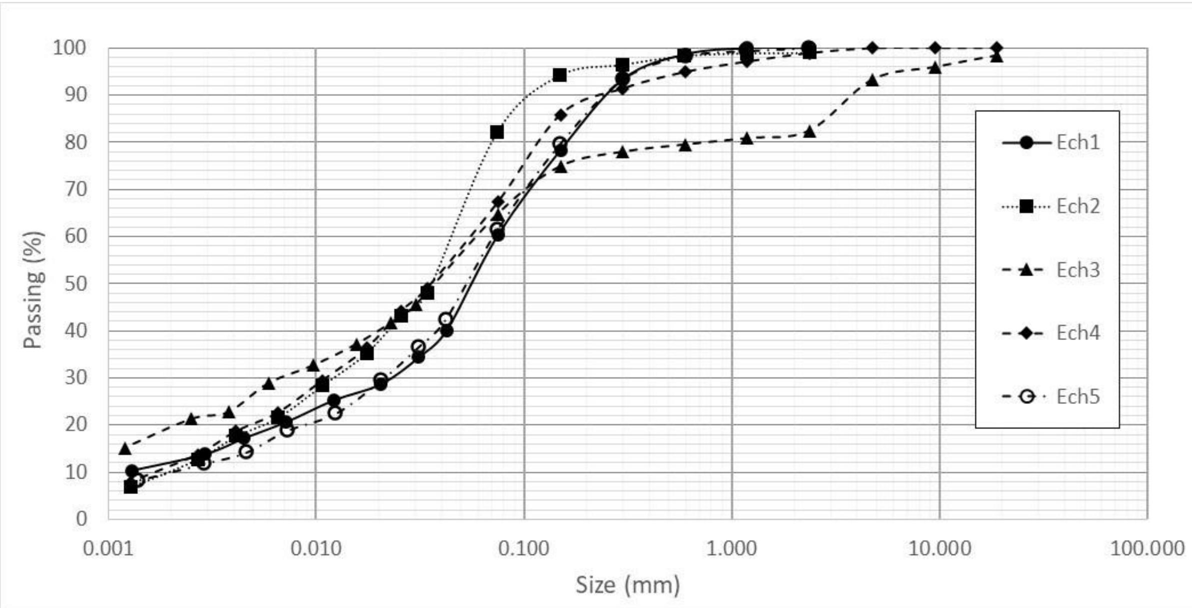


Figure 2: Grain size distribution curves

It appears from these grain size distribution curves that Ech3 has slightly more clay particles (< 2 µm) compared to Ech 1,2,4 and 5.

Consistency limits. The consistency limits were measured with the objective to determine the plasticity index (PI) which corresponds to the difference between the liquid limit (LL) and the plastic limit (PL) in order to define the extent of the plastic domain of the soil. These tests were carried out using the Casagrande cup for LL and the rolling technique for PL, according to the NF P 94-051 standard [12].

Table 2 summarizes the results of this analysis.

Table 2: Results of Atterberg limits

Samples	LL (%)	PL (%)	PI (%)
Ech1	25.4	20.2	5.2
Ech2	31.3	19.8	11.5
Ech3	32.9	23.8	9.2
Ech4	43.4	28.4	15
Ech5	24.5	21.0	3.5

Atterberg's limit tests performed on our materials gave high plasticity index (PI) for Ech2 and 4 (PI<10%) compared to Ech1,3 and 5 (PI>10%).

Table 3: Classification of soils according to PI

Samples	PI (%)	ASTM	NFP
Ech1	5.2	MH (silt highplasticity).	Non plastic clayeysand.
Ech2	11.5	CL-ML (clay lowplasticity).	Clayey silt withlow plasticity.
Ech3	9.2	CL-ML (clay lowplasticity).	Clayey silt withlow plasticity.
Ech4	15	CL-ML (clay lowplasticity).	Clayey silt withlow plasticity.
Ech5	3.5	CL-ML (clay lowplasticity).	Non plastic clayeysand.

Table 3 shows that Ech2, 3, 4, and 5 are classified as moderately plastic while sample Ech 1 is classified as plastic according to the ASTM standard. The NF P 11 300 standards [16] classifies Ech2, 3, and 4 as moderately plastic soils while Ech1 and 5 are plastic.

Methylene blue value (MBV). Known for its ability to absorb, clay soils are materials that have several characteristics. However, some clays do not have the capacity to absorb chemical solutions, hence to determine the cleanliness of materials in relation to their clay constitution the methylene blue test becomes essential in geotechnical engineering. Table 4 shows the results obtained for the variation of the blue quantity of the five soils.
The results obtained (Table 4), show that the clay fraction is inactive in the soils studied and no great difference for the values of methylene blue.

Table 4: Results of methyl blue value

Samples	MBV(/100gr)	Clay fraction of soil
Ech1	1.3	Inactive
Ech2	1.5	Inactive
Ech3	1.3	Inactive
Ech4	2.5	Inactive
Ech5	1.3	Inactive

Solid grain density. The density of solid grains is the mass per unit volume of the material without taking into account the voids in or between the grains. For our five soils, the helium gas pycnometer was used to determine γ_s. The results obtained for this test are shown in Table 5.

Table 5. Density values solid grains

Samples	γ_s (kg/m^3)
Ech1	2678
Ech2	2720
Ech3	2852
Ech4	2600
Ech5	2670

From the analysis of the results in Table 5, it follows that the solid grain density value is quite similar for all our materials, excepted Ech 3 that has a density of solid grains significantly higher thanthe others due to a higher presence of clay particles (< 2 µm).

Geomechanical tests. The oedometric compression tests were carried out to characterize the compressibility of soils. It thus makes it possible to define the degree of settlement of soil under a certain level of vertical load. The tests have been performed on home-made test benchs dedicated to oedometric compression tests.
The direct shear tests were performed, on the AUTOSHEAR 27-WF21A60 apparatus from Wykeham Farrance producer, according to ASTM D3080 [17] to obtain the soil shear strength parameters (cohesion and friction angle).
The triaxial tests were carried out, on TriSCAN Pro 50 Advanced Load Frame and VJT0549 triaxial cell provided by VJTech company, according to the ASTM D 4767- 11 standard [18].

Results and Discussions

Generally speaking, the geomechanical tests were carried out on cylindrical samples manufactured at a water content close to the plasticity limit and at the natural dry density (as measured on fragments of soil in place), see Table 6. Then, depending on the case, these samples were either tested in this condition or resaturated and then tested. The size of the samples is 100 mm in height and 50 mm in diameter for the triaxial tests, 20 mm in height, and 50 mm in diameter for the direct shear and oedometric compression tests.

Table 6: Density values and water content of samples

Samples	**Dry density (g/cm^3)**	**Initial water content (W(%)i) (%)**
Ech1	1.74	20.4
Ech2	1.70	20.2
Ech3	1.71	22.2
Ech4	1.70	20.1
Ech5	1.73	17.0

Oedometric compression tests. For this study, the oedometric compressibility curves in the semi-logarithmic scale of vertical deformations according to the vertical stresses are presented in Figure 3.

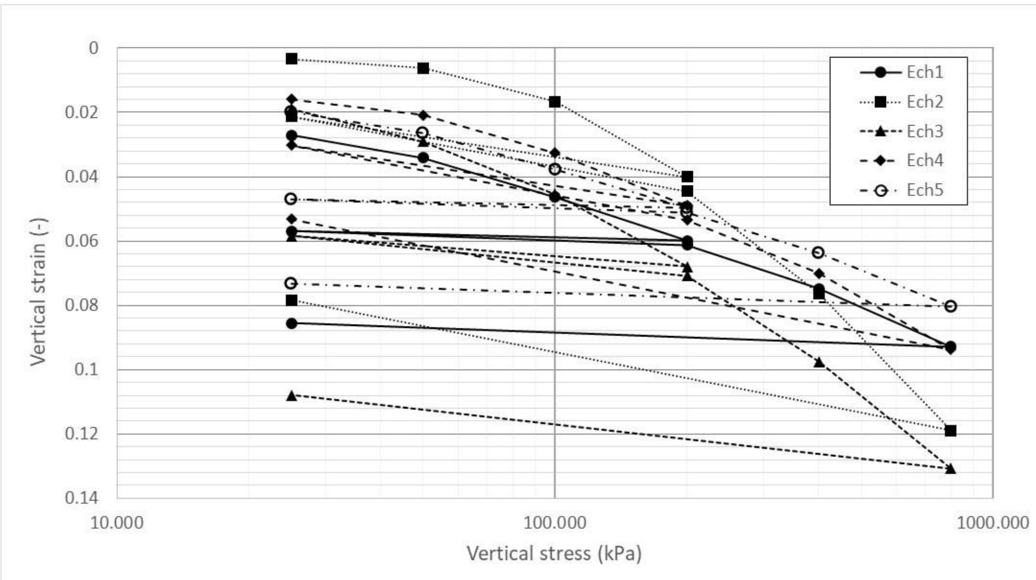


Figure 3: Oedometric compressibility curves of Ech1, Ech2, Ech3, Ech4 and Ech5.

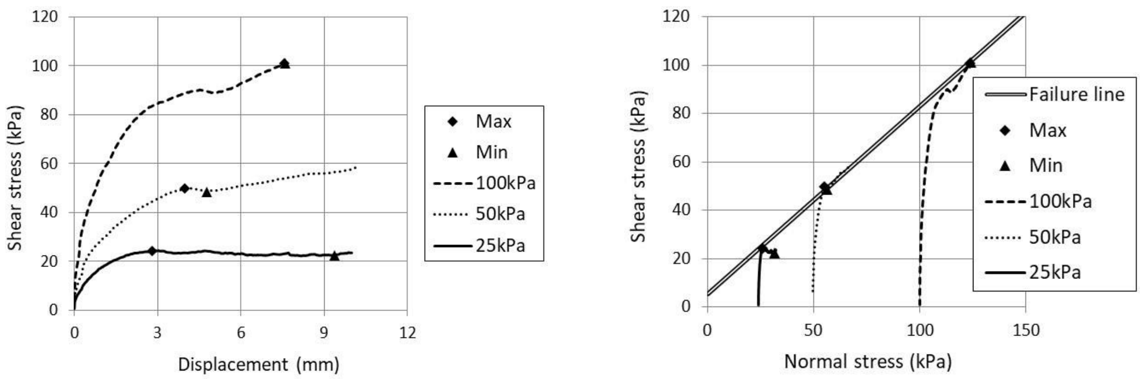
Based on the oedometric compressibility curves, the parameters C_c and C_s were determined and summarized in Table 6.

Table 6: Results of odometer tests

Samples	**CcCs**
Ech1	0.110.02
Ech2	0.310.04
Ech3	0.180.02
Ech4	0.170.03
Ech5	0.120.02

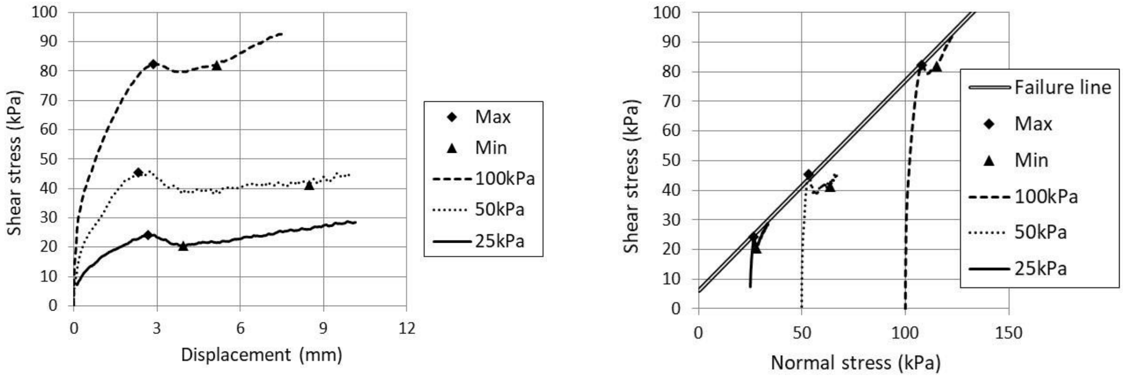
With regard to the synthesis of the results of Table 6, it results that the soils of the banks of the Lubumbashi river are moderately compressible for the Ech 1, 3, 4 and 5; rather strongly compressible for the Ech 2.

Direct shear tests. The direct shear tests were performed at three normal stresses (25, 50, and 100 kPa) on three similar specimens of each soil at initial water content conditions (W(%)i) and at resaturated water content conditions (W(%)ir). Each test performed includes the consolidation and shear phases.
The typical curves giving the deformations of the soils tested show the non-linearity of these curves almost at all levels of these deformations. These curves are consistent with the typical behavior of compressible clays.
From the curves obtained (Figs. 4 to 13) , the peak and residual strength parameters can be determined. The peak strength is determined as the point on the curve with the maximum Tau/sigmaN ratio while the residual strength state is taken as the minimum value of the Tau/sigmaN ratio in the post-peak part. When those stress states under peak and residual strength are reported in the Mohr plane, it is then possible to deduce the strength parameters: cohesion (c) and friction angle (ϕ).



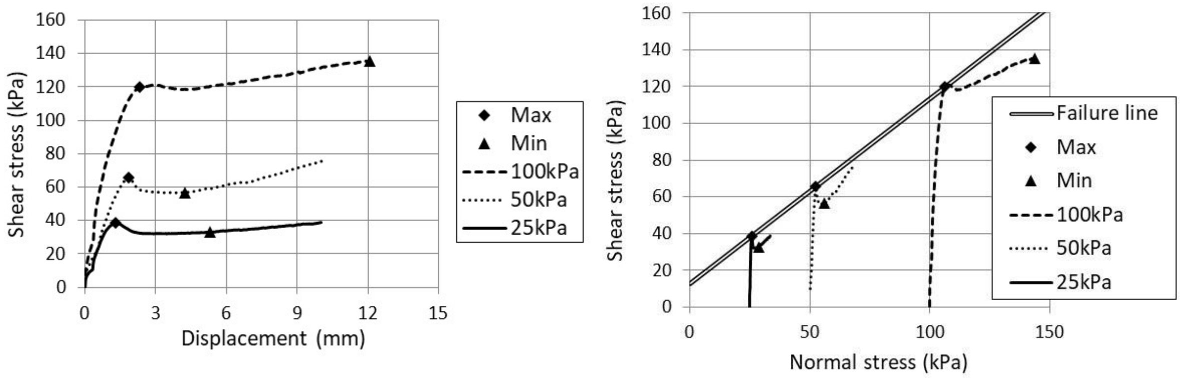
(a) (b)

Figure 4: Shear curve for Ech1 prepared at W (%)i: (a) displacement-shear stress curve; (b) normal stress-shear stress curve.



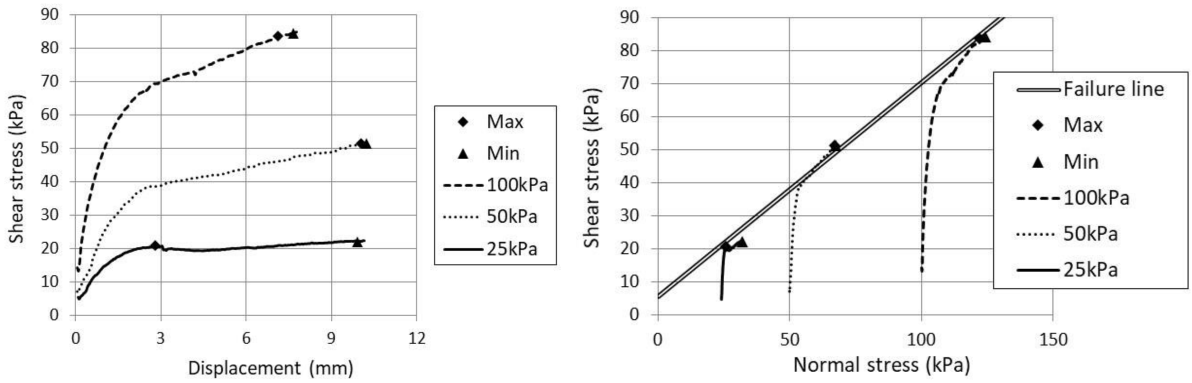
(a) (b)

Figure 5: Shear curve for Ech1 prepared at W (%)ir: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



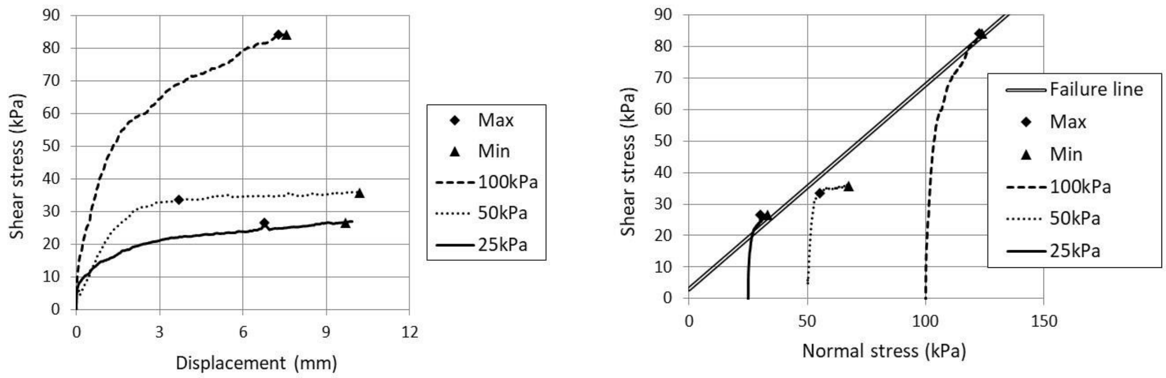
(a) (b)

Figure 6: Shear curve for Ech2 prepared at W (%)i: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



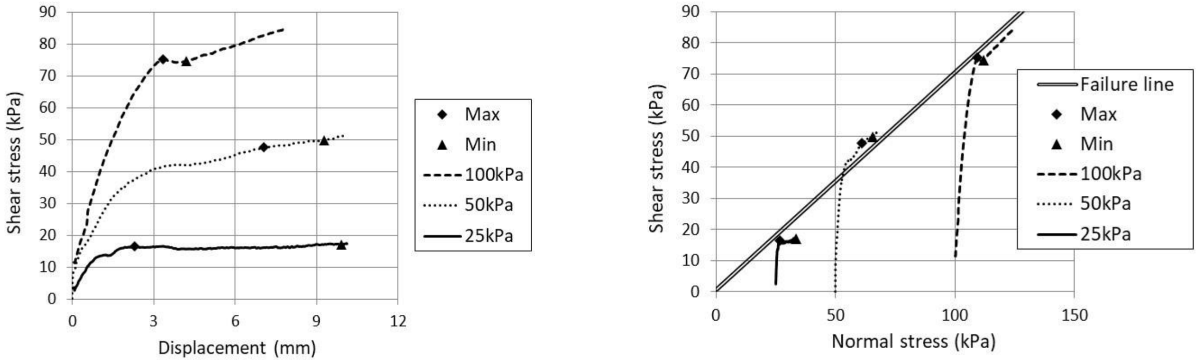
(a) (b)

Figure 7: Shear curve for Ech2 prepared at W(%)ir: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



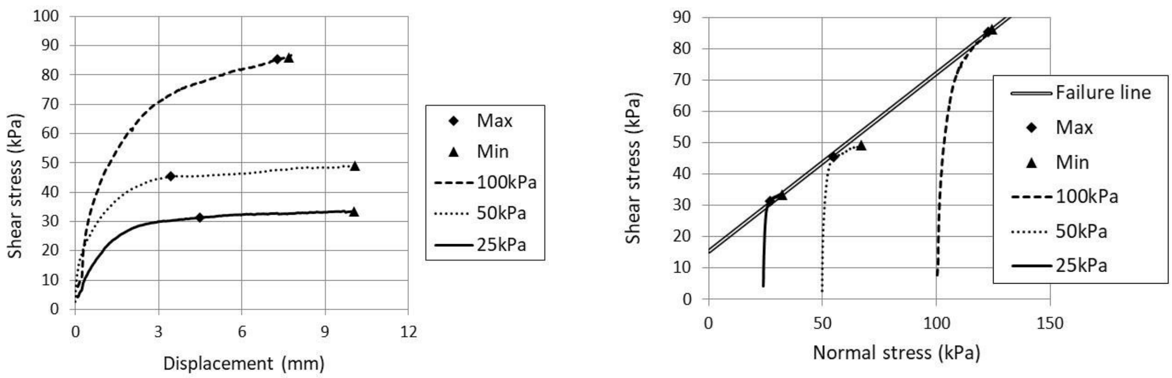
(a) (b)

Figure 8: Shear curve for Ech3 prepared at W(%)i: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



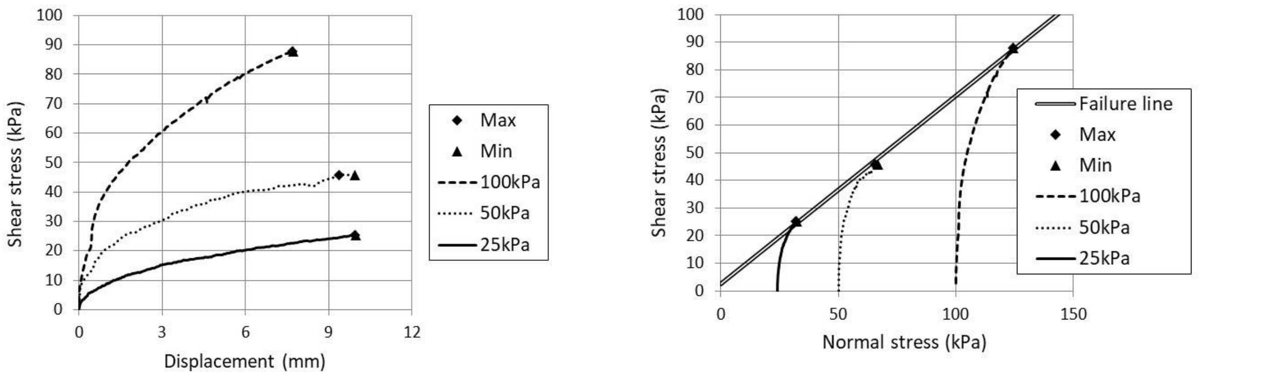
(a) (b)

Figure 9: Shear curve for Ech3 prepared at W(%)ir: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



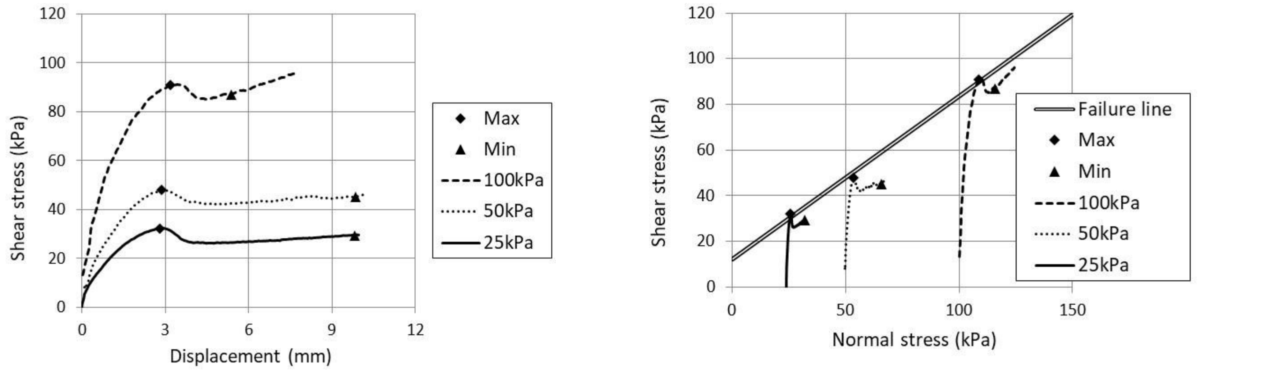
(a) (b)

Figure 10: Shear curve for Ech4 prepared at W(%)i: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



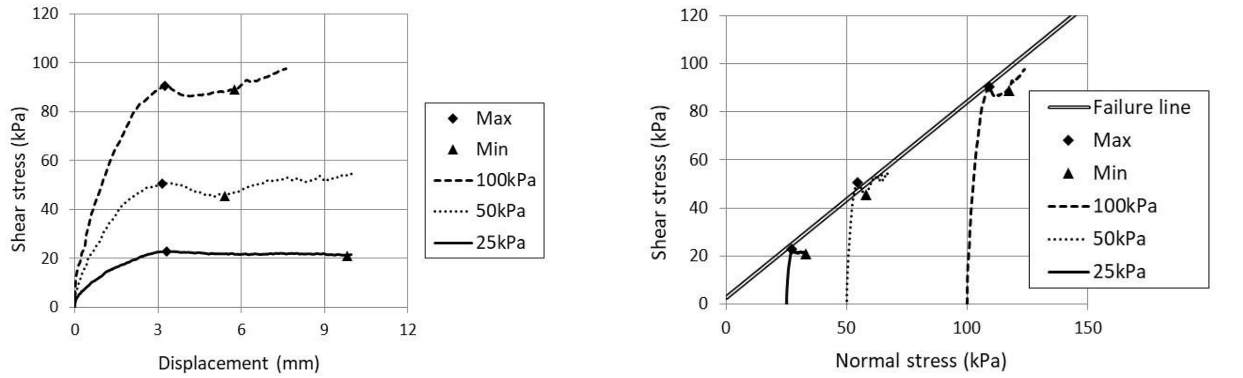
(a) (b)

Figure 11: Shear curve for Ech4 prepared at W (%)ir: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



(a) (b)

Figure 12: Shear curve for Ech5 prepared at W (%)i: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.



(a) (b)

Figure 13: Shear curve for Ech5 prepared at W(%)ir: (a) displacement- shear stress curve; (b) normal stress-shear stress curve.

Table 7 summarizes the obtained effective strength parameters in terms of cohesion and friction angle in the peak state.

Table 7: Results of direct shear tests at peak strength state

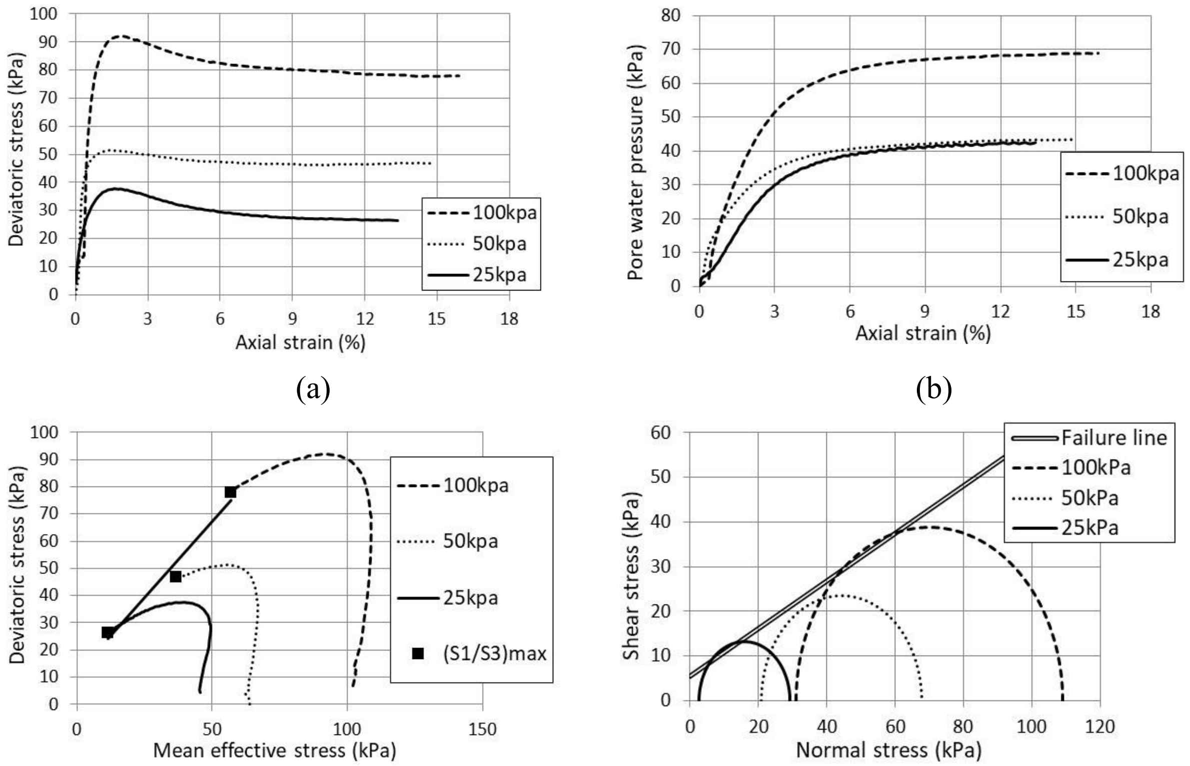
Samples	*Direct shear*			
	W(%)i		W(%)ir	
	Cmax (kPa)	ϕmax (°)	Cmax (kPa)	ϕmax (°)
Ech1	0.5	37.3	4.2	36.0
Ech2	12.9	45.2	4.0	33.0
Ech3	0.5	32.4	0.0	34.3
Ech4	14.9	29.5	0.0	35.3
Ech5	0.7	39.9	0.0	38.1

The mechanical parameters (C and φ), show variable strength, with cohesion ranging from 0.5 to 14.9 kPa and internal friction angle ranging from 22.9 to 45.2° for samples tested at water content corresponding to the initial water content while samples tested at the initial resaturated water content show almost no cohesion (0 to 4.2 kPa) and internal friction angle ranging from 33.0 to 38.1°. It can be seen from these results that there is a small variation in the shear parameters depending on whether the initial water content or the resaturated initial water content is present.
All these results demonstrate that the shear strength parameters are a function of the initial state parameters.

Triaxial compression tests. These tests were used for the purpose of determining the shear strength parameters,as an alternative to direct shear. They are also part of the reference tests for studying geomechanical soil behavior and, mainly, the deformability of soils under conditions that are very different from oedometric tests. In this study, undrained consolidated type (CU) triaxial tests were carried out at three different confinement stresses (25, 50 and 100 kPa) of each soil. All the specimen were tested under saturation conditions.
The curves in the p-q plane allowed us to determine cohesion and the angle of friction based on the regression line of the failure points (q=k+Mp'), the slope M allowed us to obtain phi via the equation 1 and then the cohesion was obtained by the equation 2.

$$\phi = \text{Arcsin}\,(3M / (6 + M)) \qquad (1)$$
$$c = k \,.\, \text{Tan(phi)} / M \qquad (2)$$

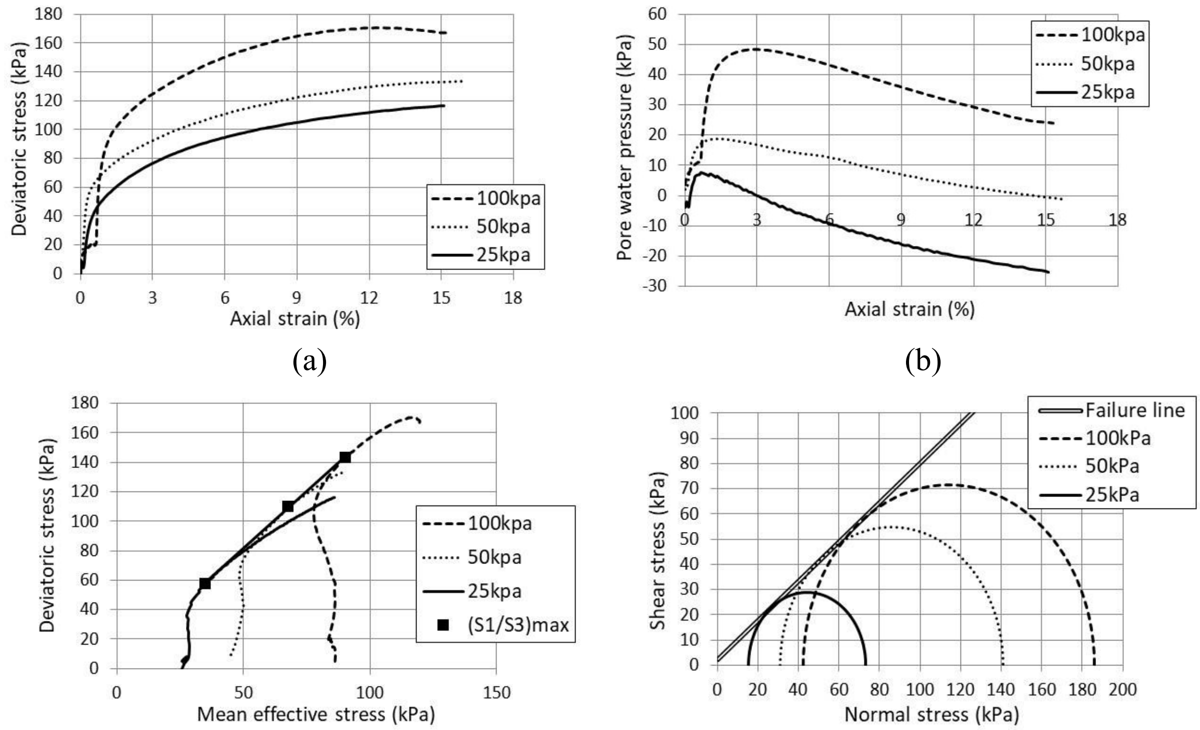
At the end of these tests the axial stress-axial strain and pore pressure-axial strain curves were obtained (Figs 14 to 18).



(a) (b)

(c)(d)

Figure 14: CU triaxial test curve for Ech1: (a) def-q; (b) def-U; (c) q-p; (d) Mohr circle



(a) (b)

(c) (d)

Figure 15: CU triaxial test curve for Ech2: (a) def-q; (b) def-U; (c) q-p; (d) Mohr circle

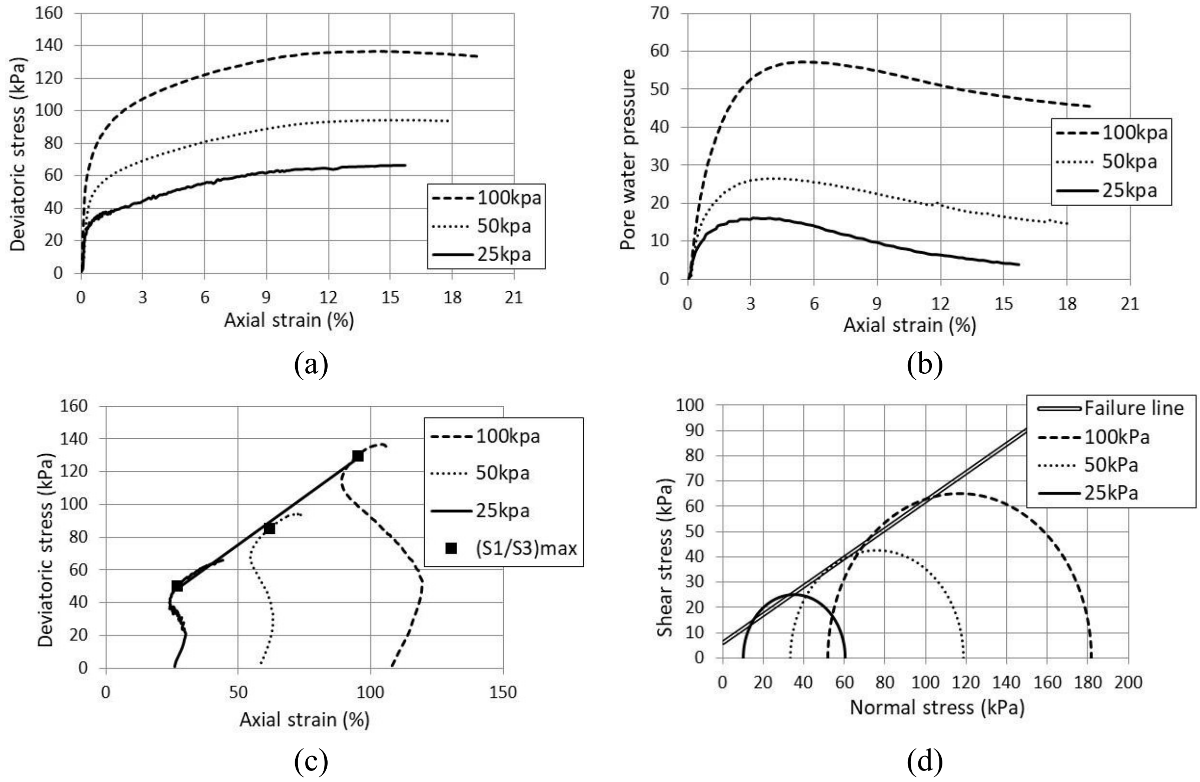


Figure 16: CU triaxial test curve for Ech3: (a) def-q; (b) def-U; (c) q-p; (d) Mohr circle

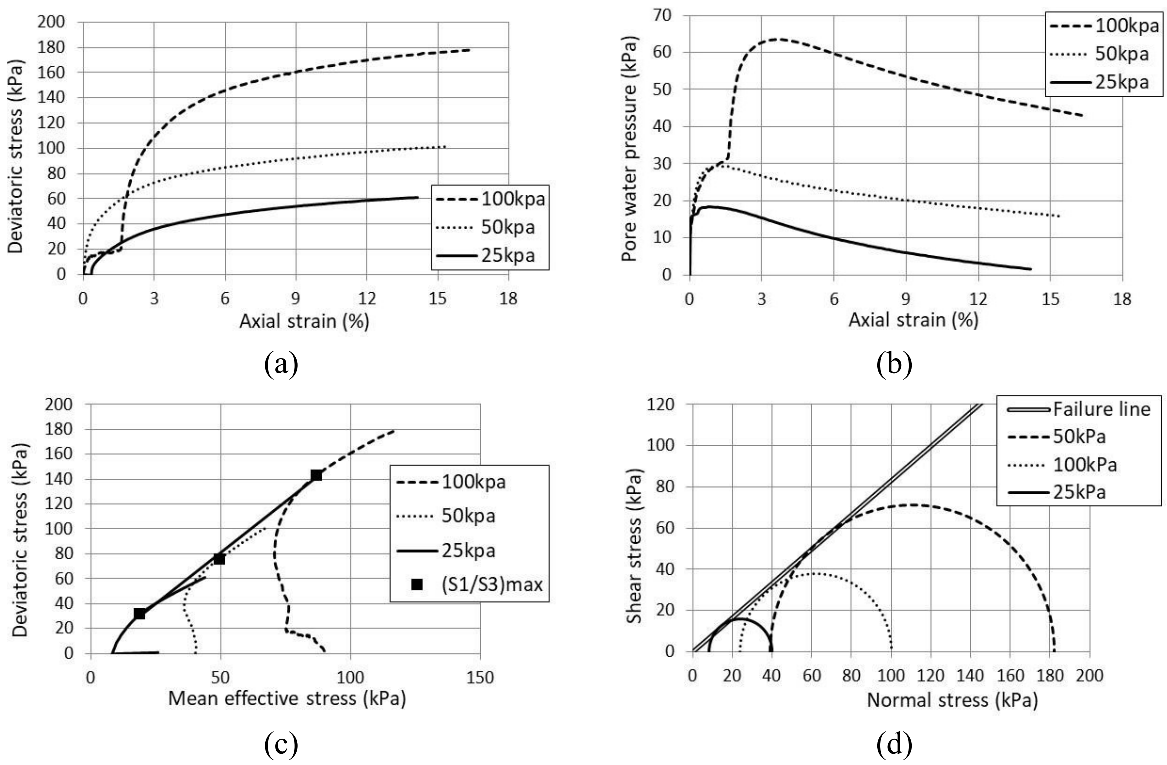


Figure 17: CU triaxial test curve for Ech4: (a) def-q; (b) def-U; (c) q-p; (d) Mohr circle

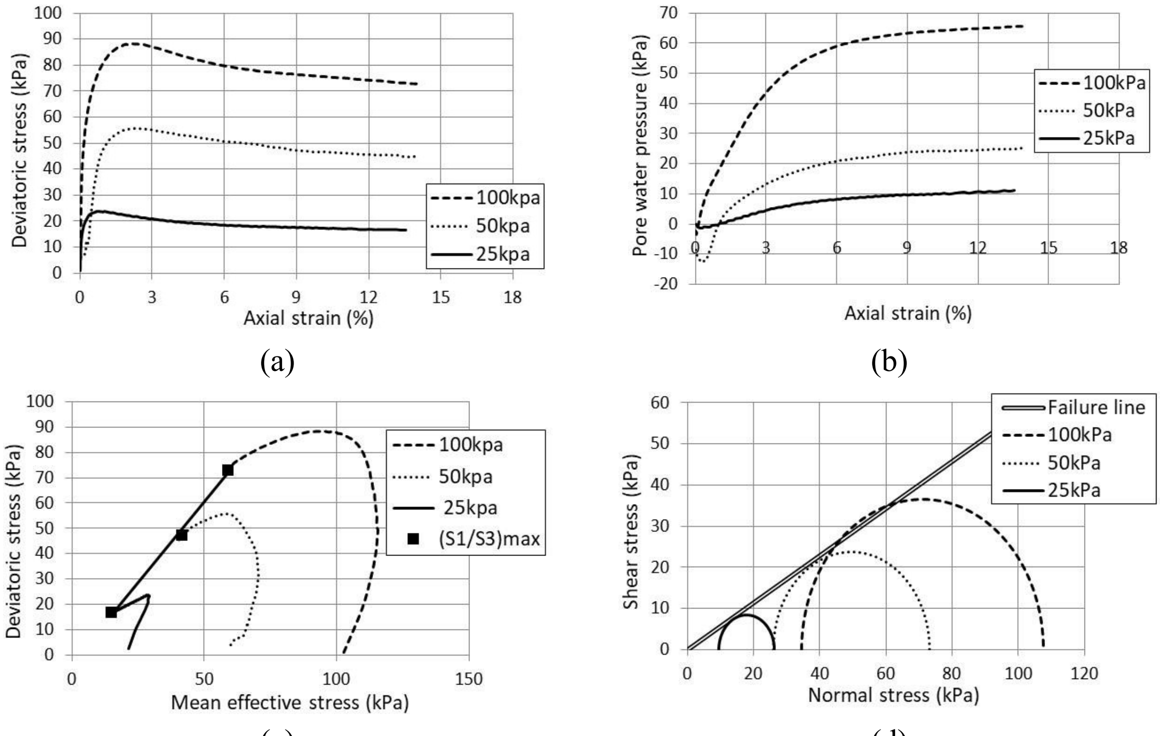


Figure 18: CU triaxial test curve for Ech5: (a) def-q; (b) def-U; (c) q-p; (d) Mohr circle

From the results of the triaxial consolidated undrained CU tests, the values of cohesion (C) and internal friction angle (ϕ) were determined (Table 8). The undrained Young's modulus (Eu), corresponding to 3 times the shear modulus ($E_u = 3G$) is obtained by taking the maximum slope of the axial stress-axial strain curve, over a range of axial strain of 0.1%. The Young modulus (E) was calculated using equation (3) for the Poison coefficient (υ) equal to 0.3.

$$G = E / 2(1 + \upsilon) \qquad (3)$$

Table 8: Results of mechanical parameters obtained from triaxial tests

Samples	C (kPa)	φ (°)	E (Mpa)			G (Mpa)		
			25 (kPa)	50 (kPa)	100 (kPa)	25 (kPa)	50 (kPa)	100 (kPa)
Ech1	5.5	28.0	7.9	18.9	25.4	3.0	7.3	9.8
Ech2	2.3	37.9	13.5	21.4	30.6	5.2	8.2	11.8
Ech3	6.0	29.3	15.8	16.6	29.1	6.1	6.4	11.2
Ech4	0.0	39.6	6.5	14.2	13.6	2.5	5.4	5.2
Ech5	0.0	30	10.2	8.9	33.2	3.9	3.4	12.8

The results in Table 8 show internal friction angles in the range of 28.0 to 40.1° with cohesion ranging from 0.0 to 6.0 kPa.

The cohesion and friction angle values obtained by the triaxial tests and those obtained by the direct shear tests under saturated conditions are quite similar while they are different from those obtained on the specimen tested at the initial unsaturated water content (Figs. 19 and 20). The difference between the strength parameters under saturated state and unsaturated state is due to the capillary strengthening of the soils when it is drier. While the slight difference between parameters obtained from consolidated undrained triaxial tests and direct shear tests could be due to the unperfectdrainage conditions for direct shear tests or unperfect re-saturation for triaxial compression.

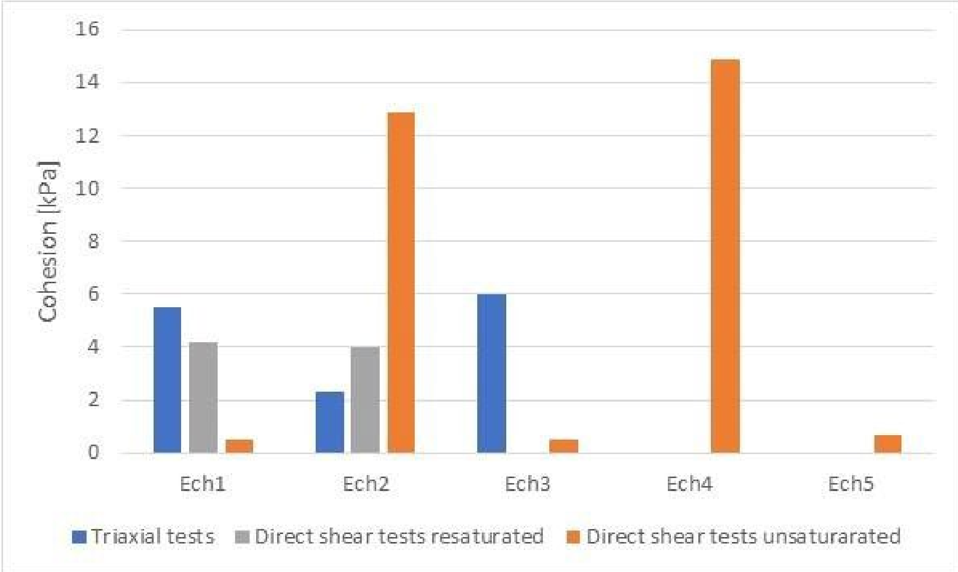


Figure 19: Cohesion variation as a function of triaxial tests and resaturated and unsaturated direct shear

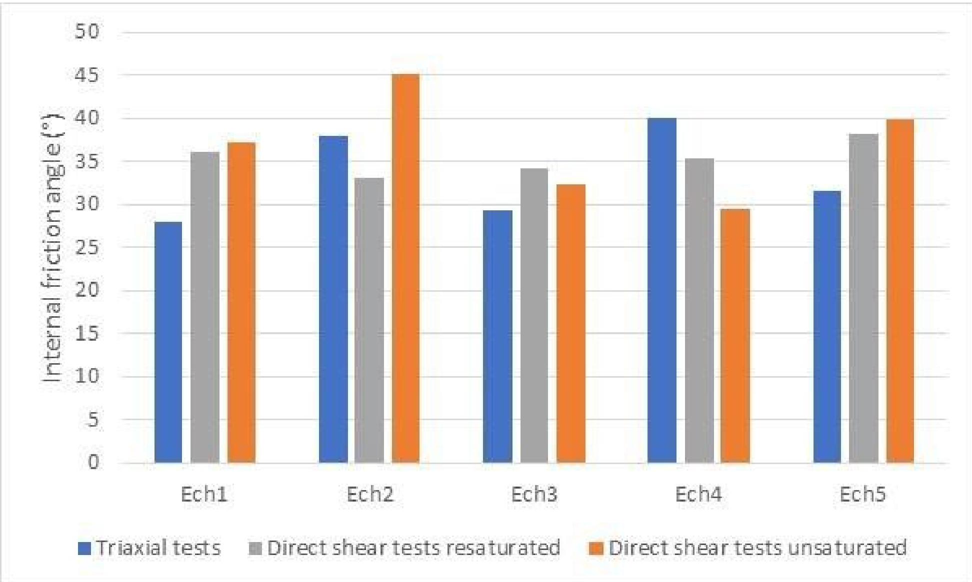


Figure 20: Variation of the internal friction angle as a function of triaxial tests and resaturated and unsaturated direct shear tests

The analysis of the results showed that after the triaxial tests the Young modulus increases with the confining pressure. These results also indicate a typical behavior of a silty clay after the tests carried out.

Discussions. From the geomechanical parameters obtained on the five tested soils, we can now analyze the results and evaluate their consistency with respect to the identification parameters. As indicated in the identification test results (see Section 3.1), the five soils exhibit low to moderate plasticity. Two soils (Ech2 and Ech4) have a plasticity index (PI) higher than 10% while the plasticity index is lower than 10 for the three other soils. It is commonly admitted that the plasticity index is a good indicator of the geomechanical properties of soils [19]. It is then proposed here to check the relation between the plasticity index and the strength and compressibility parameters of the tested soils (Fig. 21).

First of all, the plasticity index is well-correlated with the fraction of particles smaller than 80 µm (clay and silt fractions) (Fig. 21a). This is consistent with the fact that the plastic character of the soil is provided by the fine-grained part of the soil particles. Then, plastic compressibility (Cc parameter, determined from oedometric compression tests) increases with the plasticity index (Fig. 21b). More the soil is plastic and more it is compressible. Inversely, the Young modulus (E, determined from the triaxial compression test) diminishes when the plasticity indexes increase. The stiffness being the inverse of the compressibility, the trend is perfectly coherent. Note that the Young modulus is affected by the confining pressure. As a general trend, the Young modulus increases when the confining pressure increases. In Fig. 21c, the trend line is drawn for the average Young modulus obtained from the three confining pressures. Finally, the friction angle (ϕ, determined from the direct shear test under saturated conditions) is decreasing while the plasticity index increases (Fig. 21d). For all those trends, it is worth noting that the soil Ech4 is usually a bit out of the trend because it shows the highest plasticity index while the geomechanical properties are not the worst.

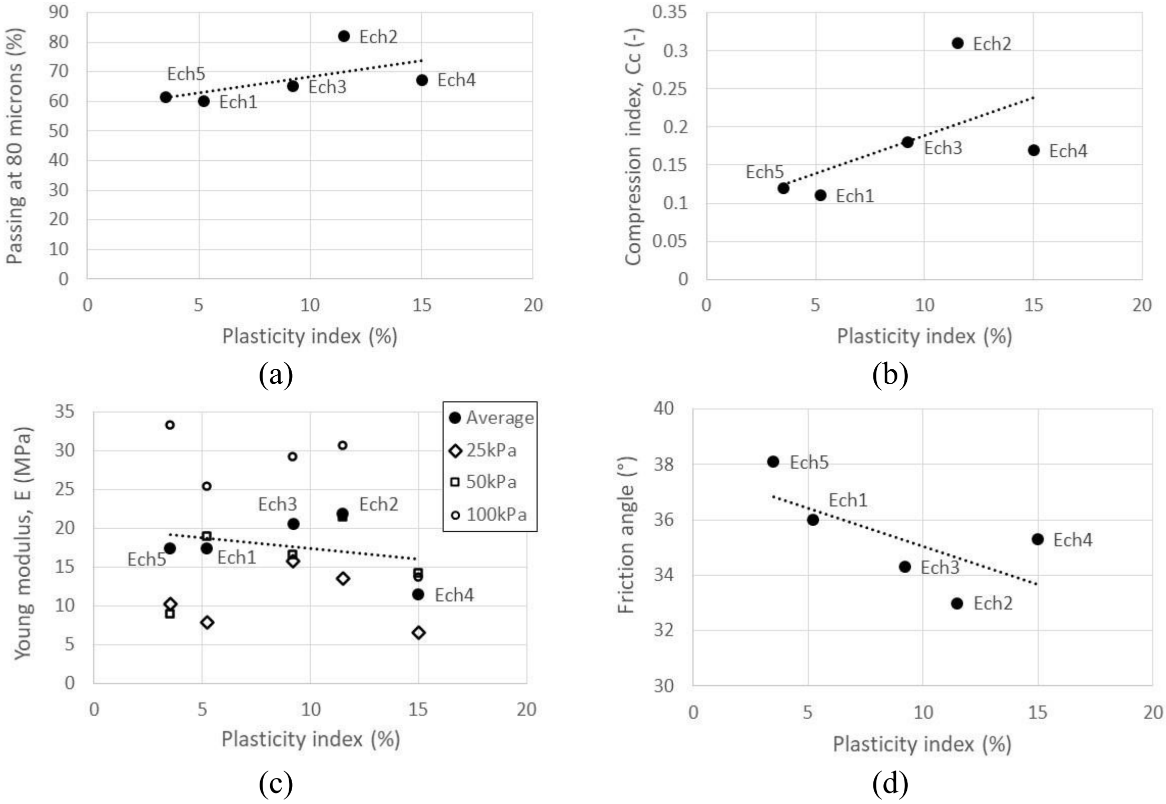


Figure 21: Correlations between various geomechanical parameters and the plasticity index: (a) Passing at 80µm, (b) plasticity compressibility, (c) Young modulus, (d) friction angle.

Conclusions

The objective of this study was to characterize by geotechnical laboratory tests the alluvial soils collected along the banks of the Lubumbashi river in the area between lake Tshombe and the Tshondo bridge in the city of Lubumbashi, in the perspective of carrying out retaining geotechnical structures for docking operations along the river.

For the region concerned by our study, the identification tests showed similar characteristics for the five tested soils that are alluvial in nature with a dominance of clayey silt and clayey sand. The methylene blue values show that the clay fraction in the soils is inactive. According to the plasticity index, the soils along the banks of the Lubumbashi River are lowly plastic.

The oedometric tests showed that the soils of the banks of the Lubumbashi river are moderately compressible for the samples (Ech1, 3, 4, and 5) and are quite strongly compressible for the sample (Ech2).

The direct shear tests performed on samples at initial water content reveal higher strength parameters than on the samples that have been resaturated. The comparison between effective shear strength parameters obtained from direct shear tests and undrained triaxial compression tests shows similar

trends despite some slight discrepancies that could be due to the unperfect drainage conditions for direct shear tests or unperfect re-saturation.
At the end of this study, we check the correlation between the obtained geomechanical parameters (in terms of compressibility and strength) with the plasticity index of the tested soils. Consistently, the strength is reduced and the compressibility is increased as long as the plasticity index increases. In other words, more plastic is the soil, and more soft and weak it is. This trend is observed for all the tested soils, except Ech4 which shows the highest plasticity index while the strength parameters remain very good.
From the analysis of the results obtained after conducting different laboratory tests, the studied soils appear to be unproblematic soils with low plasticity and good strength properties. They would adequately accommodate port geotechnical structures, built on the banks of the Lubumbashi river.

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