Experimental Studies on Some Clays Leading to Instability

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Abstract

The landslide problem in the city of Constantine (Algeria) is mostly due to clayey soil, which is covering the whole area. Therefore, all the structures and foundations built over such soil are quite vulnerable. The sensitivity of this soil increases during the rainfall season, which might reach an extreme magnitude and cause damage to the structures. To understand some aspects of the failure of the mentioned soil and its mechanism, a large campaign of geotechnical characterization was undertaken using samples from the area. The specific gravity, Atterberg limits, and coefficient of lateral earth pressure at rest were determined. Samples were tested in triaxial stress path cells over a range of stresses, with the test being either stress- or strain-controlled, leading to sliding phenomena under different stress levels. The clays can be classified as over-consolidated soils. The test program and the characterization study indicated that K_o is directly related to the stress history of the soil and also dependent upon the state of the sample before the test (i.e., undisturbed or disturbed). The clay minerals in the Mio-Pliocene landslide zones in Constantine are mostly montmorillonite and kaolinite, which have the lowest frictional resistance. As the montmorillonite content increases, the angle of internal friction decreases. The present study focuses on the critical state analysis since all the behavior problems of these tested samples show a peak shear strength, which is characteristic of over-consolidation materials that could lead to instability in this area.

Keywords: Constantine (Algeria); Triaxial Tests; Clay Soil; Landslide; Critical State Model.

1. Introduction

Landslides occur around the world and can significantly harm people and properties. The damage increases as the built environment spreads into hillside regions. The majority of the Northern–Western Constantine area in Algeria, including over fifteen sites, faces this problem. In that area, clayey formations are affected by the swelling and sliding phenomena. The loss of strength is frequently cited as one of the causes of sliding. According to DeGroot (2003) [1], Benaissa et al. (1989) [2], Belouar et al. (2004) [3], Lafifi et al. (2008) [4], and Ykhlef et al. (2014) [5], the distribution of different strengths within a slope may have a significant impact on the time to slope failure. To understand landslide events and to consider proportionate geotechnical countermeasures, a proper calculation of the strength parameters and their spatial distribution within a slope is fundamentally necessary [6]. In the case of Constantine, the issue is mostly caused by at least these factors:

- The seasonal rainfall in the area for which the greatest magnitude is recorded by Paulsen et al. (1999) [7]. Consequently, the water content of the soil significantly varies, and the swelling and sliding that occur in such soils are at their highest, according to Khemissa (2016) [8] and DeGroot et al. (2019) [9].

- The deposits of clay are characterized by several specific features. Shear strength and vulnerability of the soil structure to instability, when loaded and wetted, are two key distinguishing characteristics.

The majority of issues that impact structures and foundations in the Constantine area are caused by the Mio-Pliocene geological formations. These formations cause instability and settlement issues. As reported by Gasparre et al. (2011) [10], trials conducted all around the world demonstrate that most disaster scenarios could be avoided if straightforward and cost-effective construction norms are followed. According to this perspective, clay formations are anticipated to be susceptible to creating disaster in the Constantine region, as shown in Figure 1.
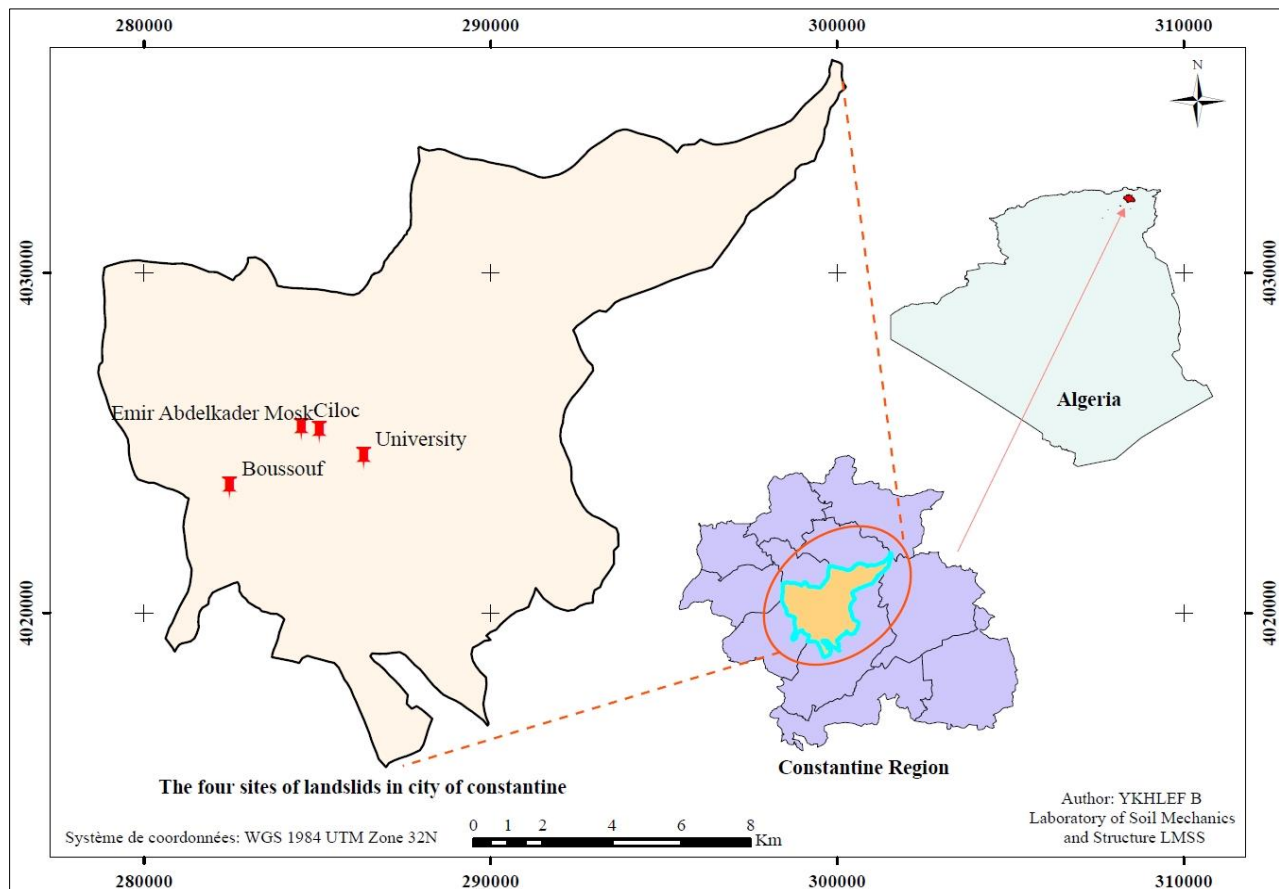

Figure 1. Location map of study area

In general, the structure and density (or void ratio) of soil determine its strength. In saturated soil, these factors are correlated with water content. The procedures used for sample preparation and testing must also be taken into account when determining laboratory strengths, according to Skempton (1969) [11], Kézdi (1988) [12], Atkinson (1993) [13], Scott (1994) [14], and Terzaghi (1996) [15]. It is crucial to understand how the shear strength changes as the strain increases. According to Jamiolkowski et al. (1985) [16], the mechanical behavior of samples under compression must be investigated to comprehend the mechanism of a landslide. However, it is difficult to investigate the stress-strain relations directly in the field. Hence, determining the mechanism of a landslide might be done according to the findings of laboratory analysis.

The present work deals with determining the strength parameters of clayey soils in the Constantine landslide area and studying their variability with depth. A common consolidated undrained (CU) stress path cell triaxial test with pore-pressure measurement was used. In addition, some physical features were determined and used for soil classification and description. For this purpose, undisturbed and disturbed samples representing several parts of the mentioned area in Constantine were collected and tested in the geotechnical laboratory of the *University of Mentouri Constantine 1* (Laboratory of Soil Mechanics and Structure, LMSS). It proposes some triaxial test results that aim to appreciate the influence on the limit and state behavior of clayey soils and to report what we can draw from the experimental data available on the city of Constantine (Algeria).

The variation range of geotechnical characteristics of this natural clay, as well as their mean values, are shown in Table 1. Therefore, it is crucial to recognize these soils and comprehend their mechanical characteristics because of the potential risk posed to engineering projects founded on sliding and collapsing soils. This study is the first attempt to fill the knowledge gap on these clays in Constantine and to help the Algerians with future construction on such soils.

Table 1. Geotechnical properties of some specimens in the area of Constantine

Description	Depth m	LL	LP	PI	K_o	Gs	Initial m/c	Failure m/c	φ'	$K_{jaky} = 1-\sin\varphi'$	$K_{brooker} = 0.95-\sin\varphi'$	$K_o = tg^2 (45-(\varphi'/2))$
Firm grey fissured silty clay (brittle certainly desiccated) - Undisturbed	2.6	60	25	35	*0.90	2.696	33.591	32.17	25°	0.58	0.53	0.614
Grey silty clay with some rootlets (soft) - Undisturbed	6.1	74	31	43	0.49	2.696	51.94	39.071	27.5°	0.54	0.48	0.57
Soft grey peaty clay (lots of peat inclusion) - Undisturbed	4.7	167	71	96	*0.64	2.372	79.38	114.15	19.5°	0.67	0.62	0.72
Firm grey silty clay (Brittle and desiccated) - Undisturbed	9.0	63	27	36	*0.85	2.696	48.12	37.65	25.5°	0.59	0.54	0.62
Soft river alluvium sandy clay Undisturbed	7.2	33	17	16	0.47	2.663	25	16.73	27°	0.55	0.49	0.58
Uniform stiff grey fissured clay Disturbed	26.2	73	20	47	0.69	2.702	26.50	34.38	15.5°	0.73	0.68	0.83
Stiff orangey brown clay	1	110	30	80	0.71	2.759	43.0	40.07	13°	0.77	0.72	0.88

2. Experimental Procedure

Several specimens of clay soils in the Constantine area were subjected to an experimental study. Their behavior was determined using stress path cell triaxial tests, as shown in Figures 2 and 3, following different stress or strain paths.
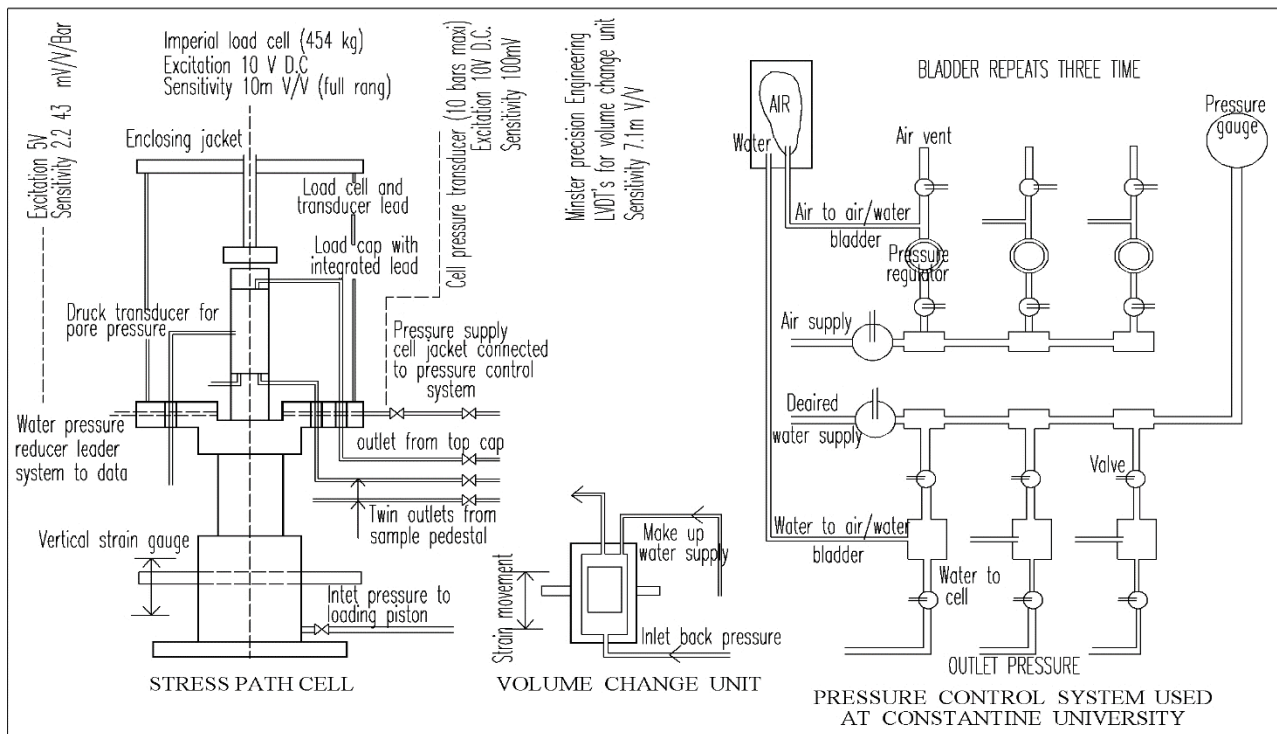

Figure 2. General arrangement for the stress path cell with pressure control system
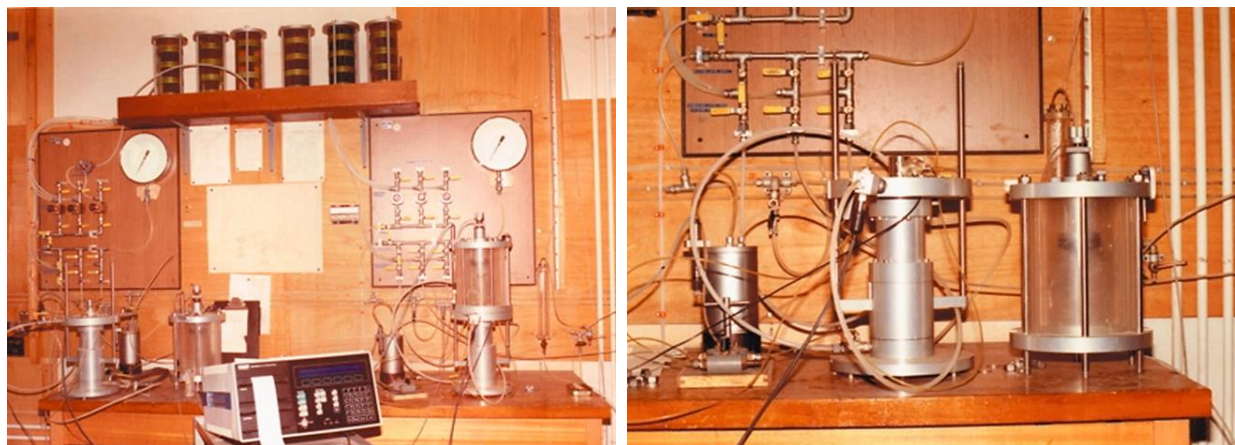

Figure 3. The set of stress path cell apparatus

These tests allow a fairly accurate description of the soil behavior and are conducted on specimens to which the concepts of continuum mechanics can be applied. This indicates the notable success of the test, in which the effective stress or followed strain courses are known. The literature of geotechnical engineering publications, such as Cheng et al. (2018) [17], exposed the research test results on equipment's laboratory tests. The specific gravity, Atterberg limits, and coefficient of lateral earth pressure at rest were measured. In a triaxial test that was either stress-controlled or strain-controlled, samples were put under different stresses that caused sliding phenomena at various stress levels. Both the undisturbed or disturbed samples were placed between two filter papers and porous discs at the top and bottom, and then they were put on the pedestal. On one side of the sample, a piece of filter paper was used to separate the small druck transducer from the clay specimen.

Figure 4. Boreholes and specimen from Constantine area

It was discovered to offer a smoother pore water pressure response and prevent smearing on the face of ceramic. The junction for the pore pressure transducer was then embodied in a specific pore pressure sheath, which covers the sample. The upper portion of the stress-path cell was installed on the lower portion once all connections had been established. Then, the test was started after the cell was filled with de-aired water. The drains were open, and the five recording devices started to work before applying any pressure. The constant temperature of the test was 18 to 20 °C.

Firstly, the samples were consolidated under an effective overburden pressure. The back pressure through the volume change unit was set to a minimum value of 2.5 bars to dissolve any air that might have been trapped while setting up the sample. To maintain a constant cross-sectional area of the sample during K_o consolidation, both cell pressure and ram pressure were gradually changed. This was achieved by monitoring the water being expelled from the specimen and the vertical displacement. The tests were executed at pressures ranging from the effective overburden pressure to the full-scale capacity of the air supply compressor (9 bars), Skempton (1961) [18], and Dai et al. (2022) [19].

The time duration between successive increments of pressure was roughly 10 to 12 hours to allow equalization of pore pressures. The consolidation was considered finished (90%) when the variation of the volume change did not exceed 0.2 cm3 per 24 hours, or when the pore water pressure transducer reduced to the back-pressure value. At this stage, the K_o part of the test was considered to be completed. After closing the drainage, a shearing test including measurements of the pore water pressure under an undrained state was conducted. This was accomplished by maintaining the cell pressure while gradually raising the ram pressure until failure, as is customary.

The total time to carry out the full K_o consolidation and shear was approximately two weeks for each specimen. The shear strength of both undisturbed and disturbed clay samples with various pollution durations was assessed using a stress-path cell triaxial apparatus which was controlled digitally, for the consolidated-undrained triaxial shear test. These samples were tested with the shear rate regulated at 0.1% per minute. Axial and volumetric strains were at their highest points at 5 and 15%, respectively. The consolidated undrained (CU) triaxial compression test with pore pressure measurement was carried out employing a strain-controlled loading device at a rate of 0.05% per minute. This made it possible to calculate the shear strength parameters of saturated material in terms of effective stress. The result of the X-Ray diffraction on the undisturbed specimen is given in Figure 5, Athmania et al. (2009) [20] and Xu et al. (2019) [21]. The former mineral of the undisturbed clay includes quartz, montmorillonite, kaolinite, and illite, Stevenson & Gurnick (2016) [22] and Ducasse et al. (2020) [23].
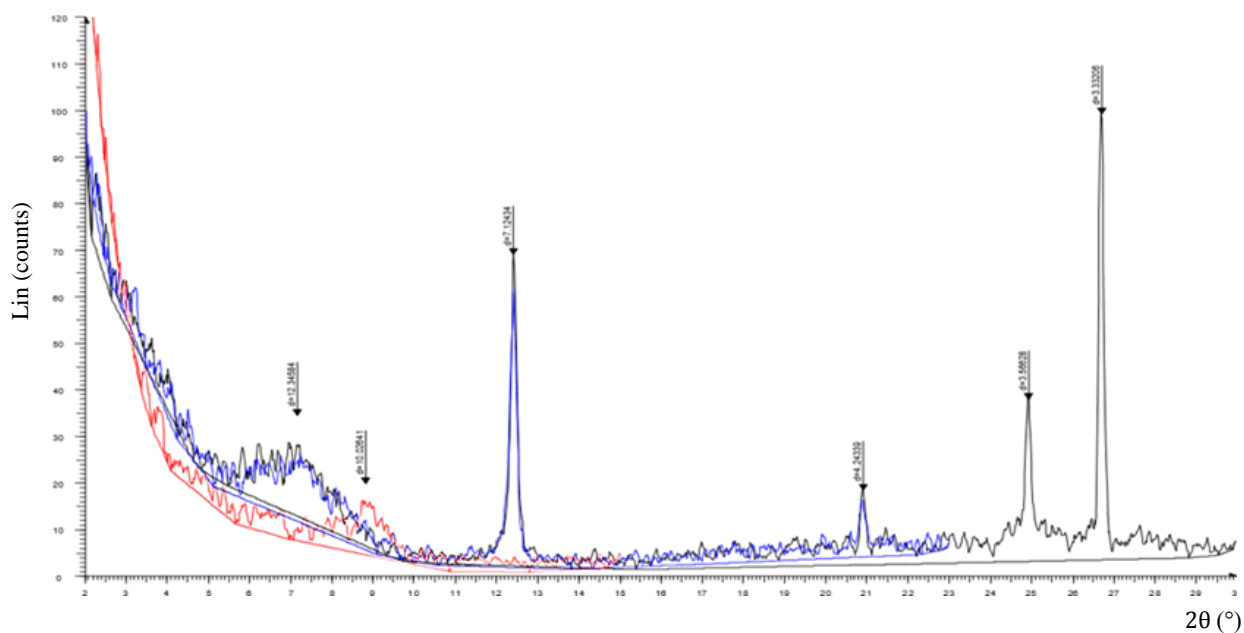
Figure 5. X-ray diffraction of undisturbed clay [20]

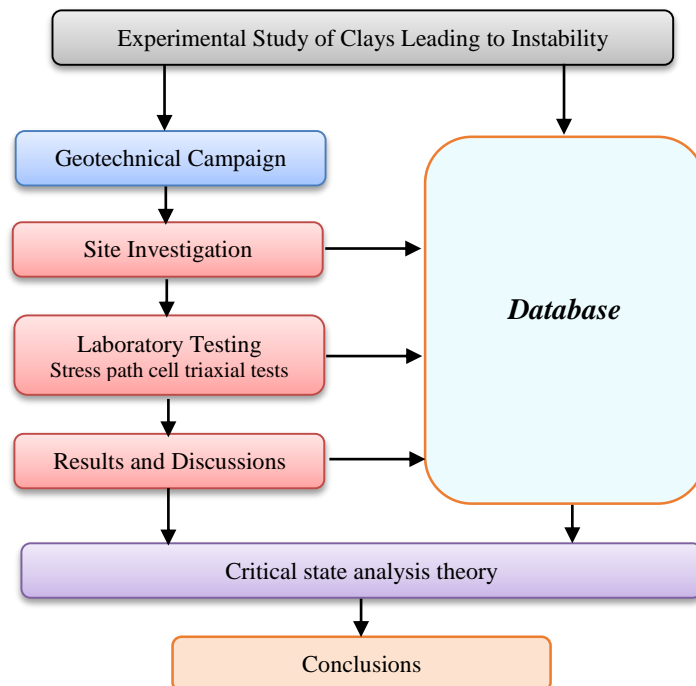The following methodological process is shown in the flowchart of this research (Figure6).

Figure 6. Flowchart of the research methodology

3. Test Results

To determine the coefficient of earth pressure at rest and the effective strength parameters, the tests using the stress-path cell were executed on the following samples from Constantine city, as it is shown in Figure 1.

- Four undisturbed samples out of U4 cores from the University site.

- One undisturbed sample from Emir Abdelkader Mosk's site.

- One remolded clay from the Ciloc site.

- One remolded clay from the Boussouf site.

All 7 samples were 38 mm in diameter and 84 mm in length. The properties of the seven samples tested in the laboratory are given in the following figures: The relationship between the stresses and strains is shown in Figures 7 to 10.
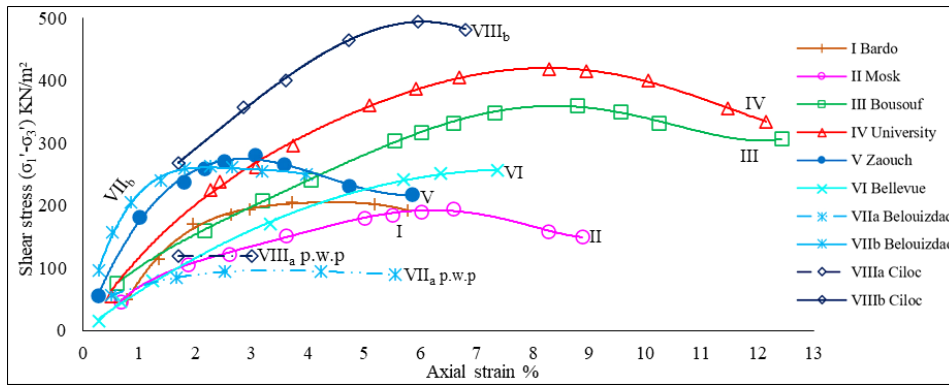
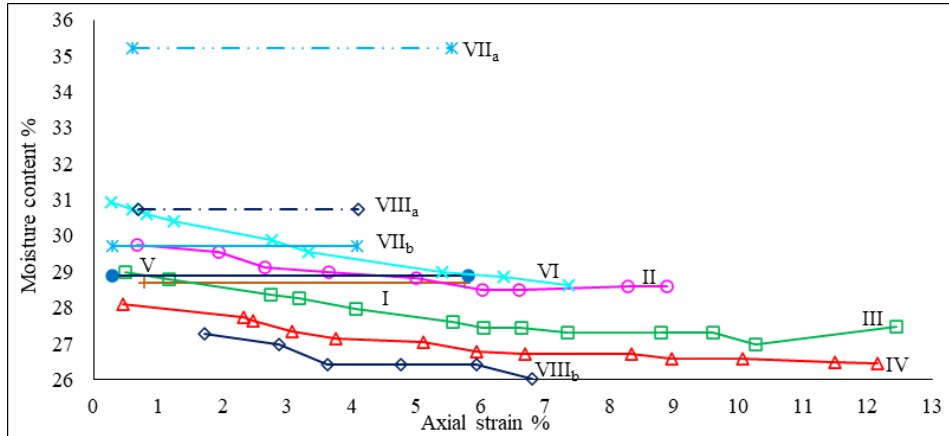**Figure 7. Shear stress versus axial strain of the testing samples**

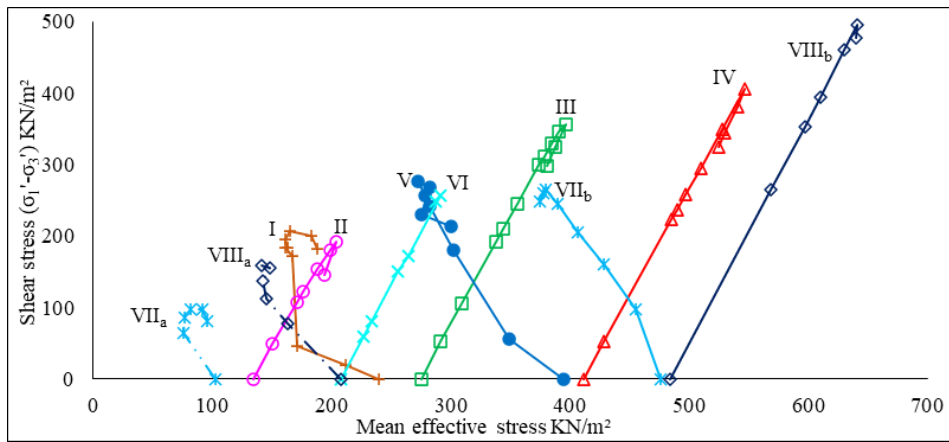**Figure 8. Percentage of moisture content versus axial strain of the testing samples**

Figure 9. Shear stress versus effective mean stress of the testing samples
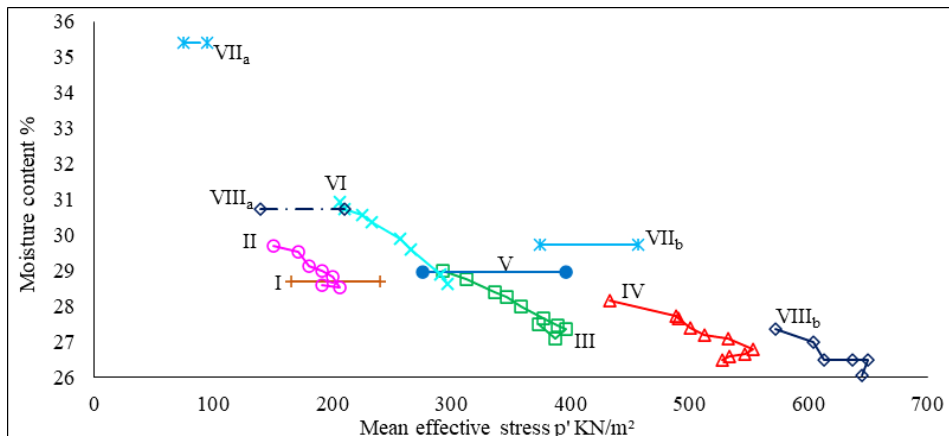

Figure 10. Percentage moisture content versus effective mean stress of the testing samples

Except for sample II (Figure 7), a recognizable straight-line trend is visible between the effective horizontal and vertical stress. In the former case (Emir Abdelkader Mosk), the few points which varied significantly from the straight line were recorded shortly after a rise in the all-around cell pressure when the pore pressure response was considerably lower (i.e., Δσ > Δu).

The ratio σ'_h / σ'_v then returned to the K_o line after roughly 8 hours. Figure 10 shows the volumetric strain versus log p' relationship for all tests. It also indicates that the samples follow the virgin consolidation curves through the loading sequence. The overconsolidated pressure could be determined for all the samples of the region except the sample of Boussouf which be due to the sampling procedure. This is mainly caused by the breaking structure of the clayey particles.

The desiccated zone would have a higher overconsolidated ratio. The desiccated zone would have. The relationship between stress/strain is shown in Figures 7 to 10 for the undrained triaxial test stage to failure as it is discussed by Jelinek et al. (2002) [24] and Ozbay & Cabalar (2016) [25]. The large strains were due to the fact that the tests were stress controlled through the ram piston.

The attempt to show a relationship between K_o and the plasticity index gave a scatter of values that did not confirm the close relationship previously mentioned by Bjerrum (1954) [26], Zhu & Wang (2019) [27], and Silva et al. (2022) [28]. The exception related primarily to the undisturbed alluvial clays from University's site and was attributed to the presence of organic material. A decent linear relationship between K_o and φ' is shown in Figure 11 and this agrees better with the formula proposed by Brooker & Ireland (1965) [29], for cohesive soils. The samples from the site of the University which departed completely from the graph were recovered.
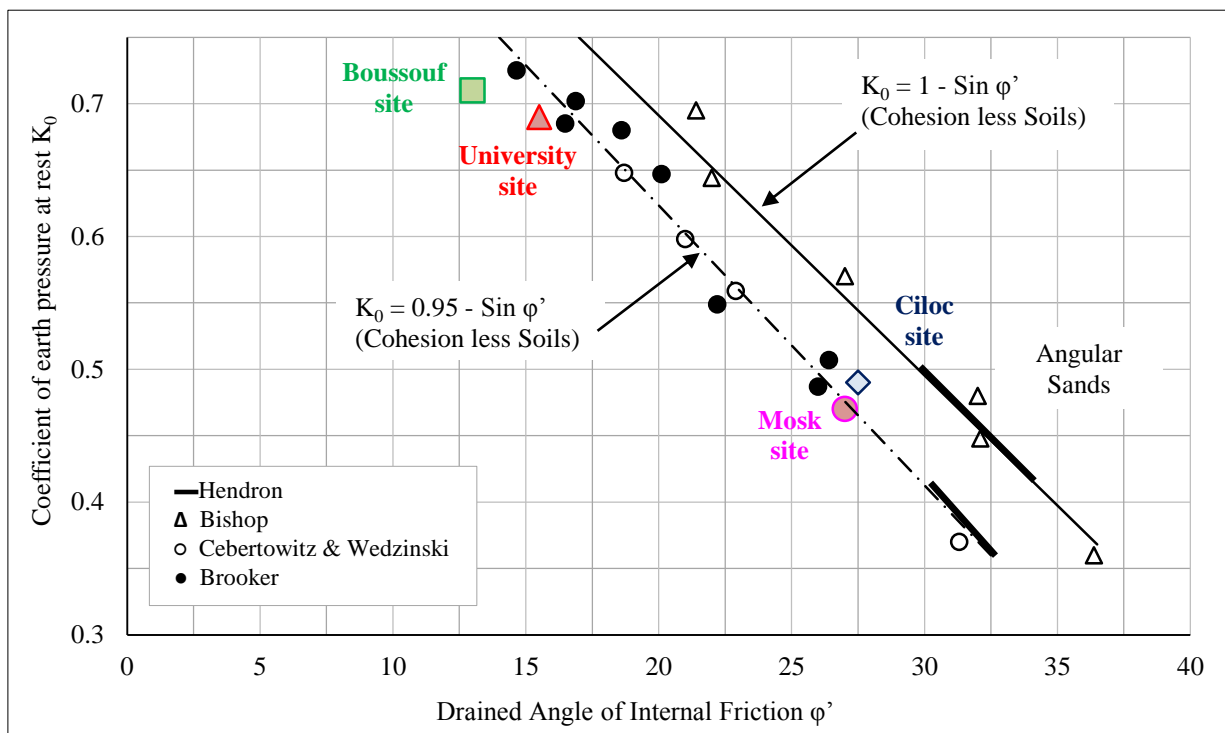

Figure 11. Results of K_o versus φ' for all tests in Constantine area reproduced from Chandler (1972) [30]

It is worth mentioning that all samples which were tested in the stress path cell from the Constantine area were found to lie on the Brooker's and Ireland curve.

4. Analysis and Discussion according to Critical State Analysis

A series of ten consolidated slow drained and undrained tests on 38 mm diameter undisturbed samples from a site in the Constantine area, were analyzed according to the critical state model. The Constantine area includes deposits of weathered clays of the Mio-Pliocene series. A first behavior appreciation of these tested samples is presented in Figure 7. A peak shear strength is observed, which is the characteristic of over-consolidation and occurred at 7% - 10% axial strain. With increased strain rate, strength reduced as it is shown in Figure 7. Both peak and final shear are followed through the effective stress path plots in Figure 9. In Figure 9, a fairly consistent relationship between the consolidation and failure lines is visible; although departures from a smooth curve in both cases occur with a minor variation in the moisture content. However, it was found that a better curve fit could be achieved, if moisture contents at the start and failure were given as a ratio of the initial moisture content, as it is shown in Figure 12.
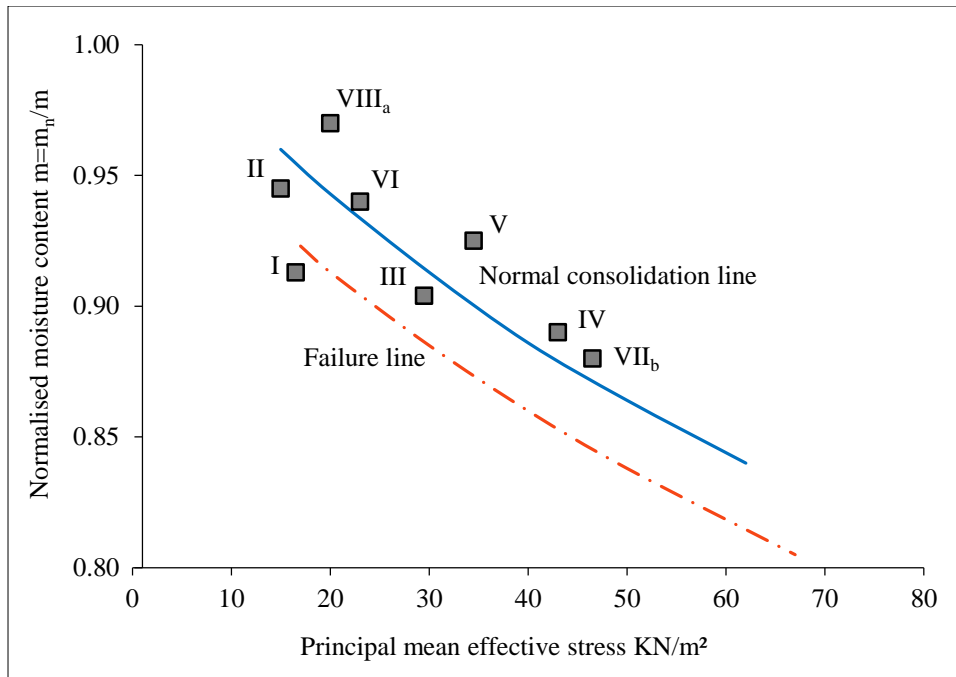
Figure 12. Percentage clay consolidation line and failure points for normalized moisture content

Furthermore, plotting specific volume against log p' for both consolidation and failure points provided parallel lines with the same slope ($\lambda = 0.09$) and both values of N = 2.272 and $\Gamma = 2.252$ as it is shown in Figure 13. This linearity agrees properly with the critical state theory of Yin et al. (2013) [31] and Hamidi et al. (2015) [32].
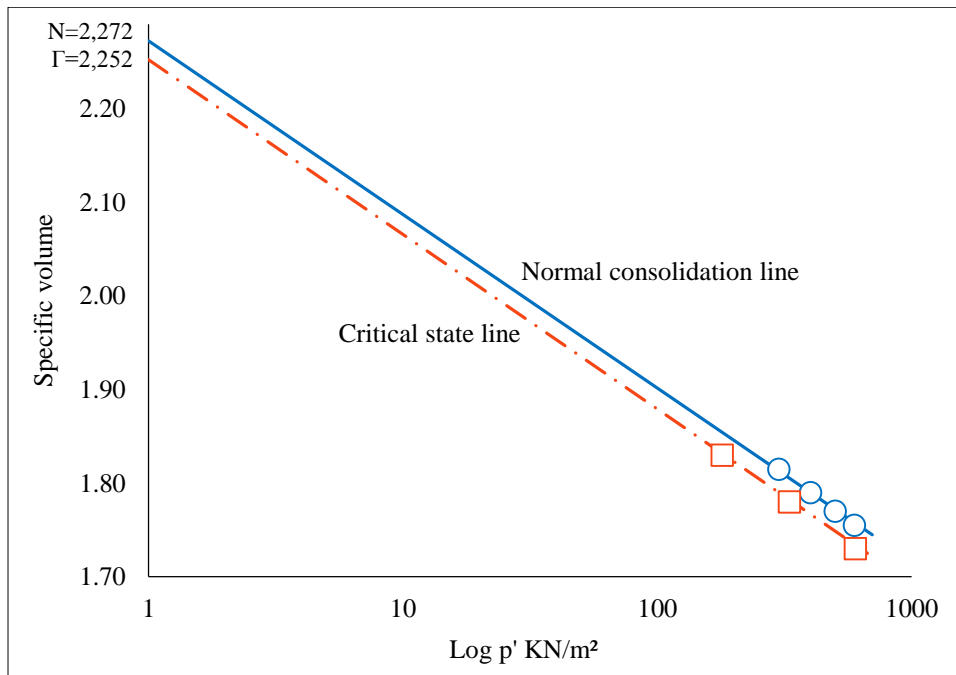

Figure 13. Constantine's normal consolidation and critical state line in specific volume versus Log p'

The main clay minerals in the Mio-Pliocene landslide zones of Constantine are montmorillonite and kaolinite, which have the lowest frictional resistance among clay minerals. As the montmorillonite content increases, the internal angle of friction decreases and becomes a residual angle which can create landslides in this area (Figure 11).

Figure 14 shows the critical state line for both the Hvorslev surface and Roscoe surface in the q' - p' plane. The best straight-line fit was drawn passing between the peak and final values, back plotted through the origin. This defines the critical state line. It was also possible to trace the Hvorslev surface and the Roscoe surface in the q' - p' plane at failure for three samples.
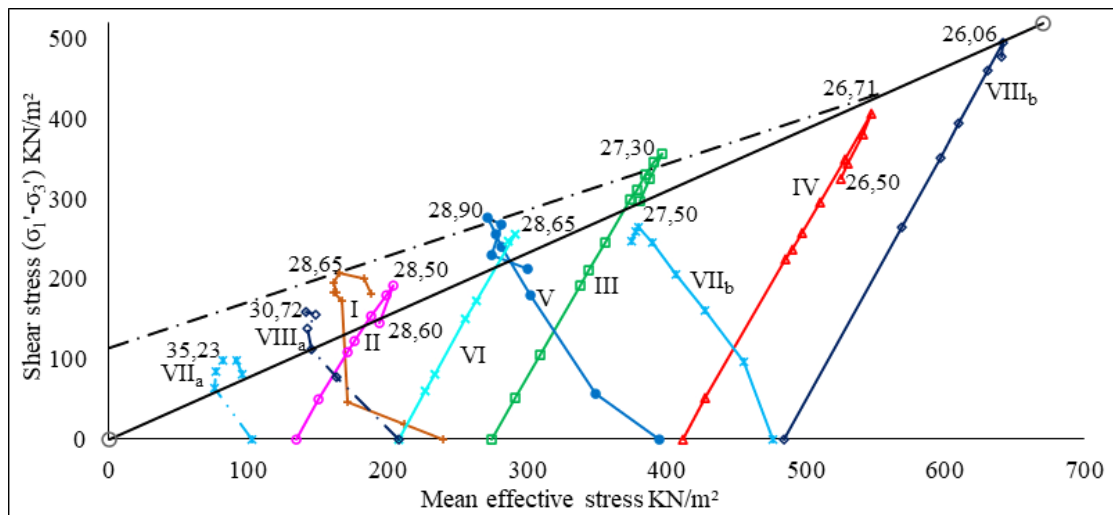
Figure 14. Deviator stress versus mean effective stress for Constantine's samples

Critical state theory is a concept related to the state-boundary surface inside which sample behavior is confined. This was obtained by normalizing both the deviator and the mean effective stresses by the equivalent pressure (pe'). The value of equivalent pressure was then calculated as the specific volume varied or stayed constant for each test, either drained or undrained, applying the following equation stated by Rampello et al. (1997) [33]:

$$p_e' = \exp\left(\frac{N-V}{\lambda}\right) \tag{1}$$

It was observed that most samples lie in the slightly over-consolidated part of the boundary state as it is shown in Figure 15. The zero effective stress condition, as described by Carey & Petley (2014) [34], and Yin et al. (2021) [35], is represented by a "tension cut off" that is drawn in the q/p' plane with an inclination of q'/p' = 3 across the origin to help define the failure behavior. The path followed by both drained and undrained tests had roughly the same curved shape in the plane q'/p'e - p'/p'e. However, most samples tend towards the critical state line from both parts of the state boundary surface.
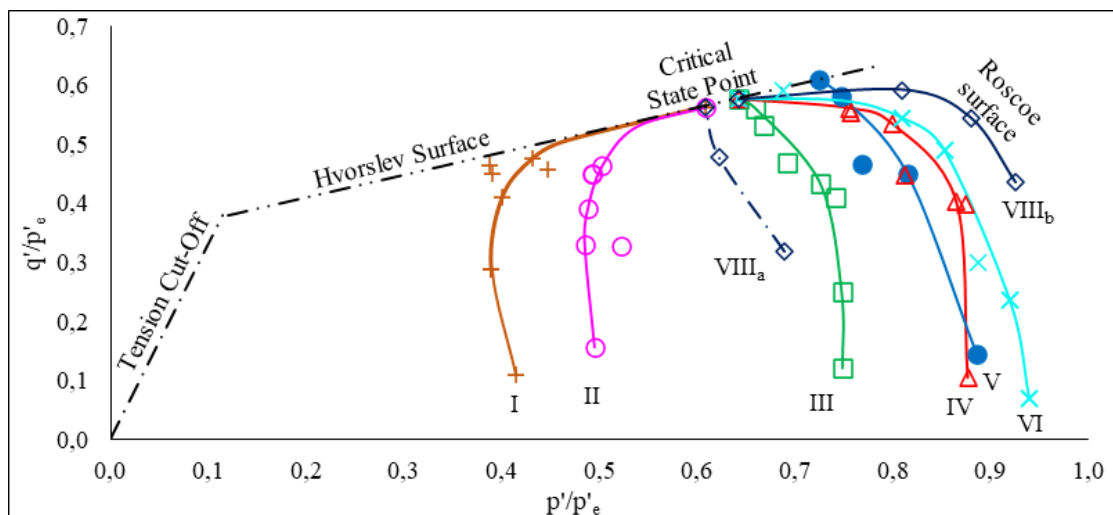

Figure 15. Critical state in the *q':p'* plane for Constantine's samples

The critical state line is bounded to the left by the Hvorslev surface which started from the tension cut-off with c = 0.33 and a slope m = 0.38 and bounded to the right by the Roscoe surface which in fact must commence from the normal consolidation line. Since soil (in general) is unable to withstand tensile stresses, the tension failure began from the origin and traveled to the Hvorslev surface with a slope of 3. These three surfaces formed the complete state boundary surface as it is shown in Figure 15. It was impossible to directly determine the over-consolidation ratio of the tested samples because in each case it varied considerably due to the weathering effects with depth. By comparing to the other published works by Abdelhamid and Krizek (1976) [36], Mesri et al. (1993) [37], and Qiu et al. (2020) [38], tested samples were found to lie in the landslip materials classification (Figure 16). The degree of weathering of this material is evident therefore.
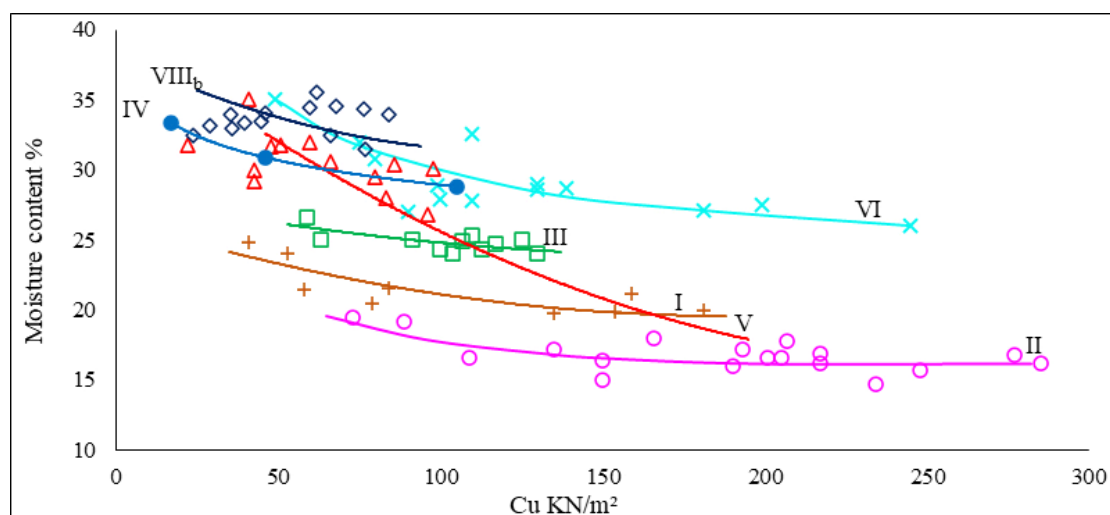
Figure 16. Moisture content versus undrained compression

5. Conclusions

The purpose of the present experimental investigation was to learn more about the mechanism of induced landslides in the Constantine area. To determine the mechanical characteristics of clay under stress path cell loading, some triaxial undrained and drained tests were carried out on undisturbed and disturbed samples. Axial strain, excess pore pressure, and shear strength have been discovered by interpreting test results from the stress path.

The study provides a better understanding of some aspects of the failure mechanism and the soil behavior in the landslide area of Constantine. The test program and the characterization study undertaken indicate that K_o is directly related to the stress history of the soil and also dependent upon the state of the sample before the test (i.e., undisturbed or disturbed). Additionally, the results indicate a linear relationship between K_o and σ' at stress levels over any over-consolidation pressure. This concurs with the studies of other authors. However, unlike the prior works, no correlation between K_o and the plasticity index (PI) was found. It was impossible to make any precise determination of K_o values for material in an over-consolidated state. This applies to some degree to the early stages of the tests reported here. The difficulties surrounding K_o measurements of over-consolidated materials have been proved in this study.

Landslides and slope failure are frequently caused by rainfall and groundwater tables in clay deposits and clay bed regions around the northwest of Constantine city. The clay primarily contains some mineral substance that is easy to weather and has low permeability, while the water leads to a decrease in the shear strength of the expansive clay.

The study was accomplished in critical state analysis because all the problems related to the behavior of the tested samples showed a peak shear strength. This is the characteristic of over-consolidation material that could cause instability in this area. Moreover, it was possible to retrace the Hvorslev surface in the q' - p' plane at failure for three samples. The tests were performed in the Laboratory of Soil Mechanics and Structure (LMSS) of the University of Mentouri Constantine 1, for the first time. The purpose of this experiment is to find out more about how a triggered landslide happens in the area around Constantine. In order to determine the mechanical characteristics of clay under stress path cell loading, a number of triaxial undrained and drained tests have been carried out on undisturbed and disturbed samples. Axial strain, excess pore pressure, and shear strength have been discovered by interpreting test results from the stress route.

6. Declarations

6.1. Author Contributions

Conceptualization, B.Y., Ab.B. and Az.B.; methodology, B.Y. and Ab.B.; software, B.Y.; validation, B.Y. and Ab.B.; formal analysis, B.Y., Ab.B. and Az.B.; investigation, B.Y. and Ab.B.; data curation, B.Y.; writing—original draft preparation, B.Y. and Ab.B.; writing—review and editing, B.Y., Ab.B. and Az.B. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in the article.

6.3. Funding

6.4. Acknowledgements

6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References

[1] DeGroot, D. J. (2003). Laboratory Measurement and Interpretation of Soft Clay Mechanical Behavior. Soil Behavior and Soft Ground Construction. Symposium on Soil Behavior and Soft Ground Construction Honoring Charles C. "Chuck" Ladd, 167-200. doi:10.1061/40659(2003)6.

[2] Benaissa, A., Cordary, D., & Gioraud, A. (1989). Les mouvements de terrain dans la zone urbaine de Constantine (Algérie). Bulletin of the International Association of Engineering Geology, 40(1), 85–90. doi:10.1007/bf02590344.

[3] Belouar, A., Boulfoul, A., & Seridi, A. Determination des caracteristiques geotechniques d'une zone potentiellement instable– Cas de Boussouf, Constantine". Sciences & Technologie. B, Sciences de l'ingénieur, 114-121. (In French).

[4] Lafifi, B., Darve, F., Nouaouria, M. S., & Guenfoud, M. (2008). Application du critère de hill dans la modélisation par éléments finis des glissements de terrain de la région de Constantine (Algérie). European Journal of Environmental and Civil Engineering, 12(6), 747–769. doi:10.1080/19648189.2008.9693042.

[5] Ykhlef, B., Belouar, A., & Boubidi, S. (2014). Plan for Prevention of Natural Hazards in Urban Areas. Case of the City of Constantine (Algeria). MATEC Web of Conferences, 11, 03008. doi:10.1051/matecconf/20141103008.

[6] Lin, Q., Steger, S., Pittore, M., Zhang, J., Wang, L., Jiang, T., & Wang, Y. (2022). Evaluation of potential changes in landslide susceptibility and landslide occurrence frequency in China under climate change. Science of the total environment, 850, 158049. doi:10.1016/j.scitotenv.2022.158049.

[7] Paulsen, S., Krauter, E., & Hanisch, J. (1999). Expert report on landslides in the city of Constantine (Algeria). Federal Institute for Geosciences and Natural Resources Hannover (FRG), Narch, 117989. (In French).

[8] Khemissa, M. (2016). Validity Criteria of Oedometric and Triaxial Test Results. Geotechnical and Geological Engineering, 34(4), 1171–1180. doi:10.1007/s10706-016-0036-4.

[9] DeGroot, D.J., E. Landon, M., & E. Poirier, S. (2019). Geology and engineering properties of sensitive Boston Blue Clay at Newbury, Massachusetts. AIMS Geosciences, 5(3), 412–447. doi:10.3934/geosci.2019.3.412.

[10] Gasparre, A., Nishimura, S., Coop, M. R., & Jardine, R. J. (2007). The influence of structure on the behaviour of London Clay. Geotechnique, 57(1), 19–31. doi:10.1680/geot.2007.57.1.19.

[11] Skempton, A. W. (1969). The consolidation of clays by gravitational compaction. Quarterly Journal of the Geological Society of London, 125(1–4), 373–408. doi:10.1144/gsjgs.125.1.0373.

[12] Kézdi, Á., & Rétháti, L. (1988). Handbook of soil mechanics, Soil mechanics of earthworks, foundations, and highway engineering. Elsevier Science, Amsterdam. doi:10.1016/C2009-0-15474-0.

[13] Atkinson, J. (1993). An introduction to the mechanics of soils and foundations: through critical state soil mechanics. McGraw-Hill Book, London, United Kingdom.

[14] Scott, C. R. (1994). An Introduction to Soil Mechanics and Foundations. Springer Science Business, Dordrecht, Netherlands. doi:10.1007/978-1-4899-7250-7.

[15] Terzaghi, A. (1996). Soil management for improvement of soil physical characteristics related to erosion in Uruguay. Wageningen University and Research, Wageningen, Netherlands.

[16] Jamiolkowski, M., Ladd, C. C., Germaine, J. T., & Lancellotta, R. (1985). New developments in field and laboratory testing of soils. 11th International Conference on Soil Mechanics and Foundation Engineering, 12-16 August, 1985, San Francisco, United States.

[17] Cheng, T., Zhang, Y., & Yan, K. (2018). Experimental Study of Silty Clay Plane Strain Tri-axial Test under RTC Path and Modified Cam-clay Model. Civil Engineering Journal, 4(3), 518. doi:10.28991/cej-0309112.

[18] Skempton, A. W. (1984). Horizontal Stresses in an Over-Consolidated Eocene Clay. Selected Papers on Soil Mechanics, 119, 119–125. doi:10.1680/sposm.02050.0015.

[19] Dai, M., Li, X., Gao, Z., & Li, M. (2022). Post-cyclic undrained shear behavior of soft marine clay under anisotropic consolidation stress paths. Marine Georesources & Geotechnology, 40(7), 882-894. doi:10.1080/1064119X.2021.1953649.

[20] Athmania, D., Benaissa, A., & Bouassida, M. (2009). Mineralogical properties of swelling clays of the wilaya of Mila. Colloque international Sol Non Saturés et Environnement. Tlemcen-Algérie-27 et, 425-435. (In French).

[21] Xu, S., Lu, H., Liu, J., & Li, J. (2019). An experimental study on the microstructure and triaxial shear of structured clay in contact with landfill leachate. Bulletin of Engineering Geology and the Environment, 78(6), 4611–4622. doi:10.1007/s10064-018-1409-0.

[22] Stevenson, C. M., & Gurnick, M. (2016). Structural collapse in kaolinite, montmorilonite and illite clay and its role in the ceramic rehydroxylation dating of low-fired earthenware. Journal of Archaeological Science, 69, 54–63. doi:10.1016/j.jas.2016.03.004.

[23] Ducasse, E., Adeline, K., Briottet, X., Hohmann, A., Bourguignon, A., & Grandjean, G. (2020). Montmorillonite estimation in clay-quartz-calcite samples from laboratory SWIR imaging spectroscopy: A comparative study of spectral preprocessing and unmixing methods. Remote Sensing, 12(11). doi:10.3390/rs12111723.

[24] Jelinek, R., Omura, H., Morita, K., Nagasawa, T., & Kubota, T. (2002). Mechanical properties and behaviour of soils under the triaxial test, the Yamauchi landslide, Japan. Bulletin of Engineering Geology and the Environment, 61(3), 207–212. doi:10.1007/s10064-002-0159-0.

[25] Ozbay, A., & Cabalar, A. F. (2016). Effects of triaxial confining pressure and strain rate on stick-slip behavior of a dry granular material. Granular Matter, 18(3), 1–9. doi:10.1007/s10035-016-0664-7.

[26] Bjerrum, L. (1954). Theoretical and experimental investigations on the shear strength of soils. Ph.D. Thesis, Swiss Federal Institute of Technology, Zürich, Switzerland.

[27] Zhu, E., & Wang, S. (2019). Modeling Confined Compression on Structured Soils. IOP Conference Series: Earth and Environmental Science, 371(2), 22091. doi:10.1088/1755-1315/371/2/022091.

[28] Silva, I. N., Indraratna, B., Nguyen, T. T., & Rujikiatkamjorn, C. (2022). Shear behaviour of subgrade soil with reference to varying initial shear stress and plasticity index. Acta Geotechnica, 17(9), 4207–4216. doi:10.1007/s11440-022-01477-w.

[29] Brooker, E. W., & Ireland, H. O. (1965). Earth Pressures at Rest Related to Stress History. Canadian Geotechnical Journal, 2(1), 1–15. doi:10.1139/t65-001.

[30] Chandler, R. J., & Nicholls, R. A. (1973). Discussion: Lias clay: Weathering processes and their effect on shear strength. Geotechnique, 23(1), 131–133. doi:10.1680/geot.1973.23.1.131.

[31] Yin, Z. Y., Xu, Q., & Hicher, P. Y. (2013). A simple critical-state-based double-yield-surface model for clay behavior under complex loading. Acta Geotechnica, 8(5), 509–523. doi:10.1007/s11440-013-0206-y.

[32] Hamidi, A., Tourchi, S., & Khazaei, C. (2015). Thermomechanical Constitutive Model for Saturated Clays Based on Critical State Theory. International Journal of Geomechanics, 15(1), 4014038. doi:10.1061/(asce)gm.1943-5622.0000402.

[33] Rampello, S., Viggiani, G. M. B., & Amorosi, A. (1997). Small-strain stiffness of reconstituted clay compressed along constant triaxial effective stress ratio paths. Geotechnique, 47(3), 475–489. doi:10.1680/geot.1997.47.3.475.

[34] Carey, J. M., & Petley, D. N. (2014). Progressive shear-surface development in cohesive materials; implications for landslide behaviour. Engineering Geology, 177, 54–65. doi:10.1016/j.enggeo.2014.05.009.

[35] Yin, J., Zhang, K., Geng, W., Gaamom, A., & Xiao, J. (2021). Effect of initial water content on undrained shear strength of K0 consolidated clay. Soils and Foundations, 61(5), 1453–1463. doi:10.1016/j.sandf.2021.08.010.

[36] Abdelhamid, M. S., & Krizek, R. J. (1976). At-Rest Lateral Earth Pressure of Consolidating Clay. Journal of the Geotechnical Engineering Division, 102(7), 721–738. doi:10.1061/ajgeb6.0000295.

[37] Mesri, G., & Hayat, T. M. (1993). The coefficient of earth pressure at rest. Canadian Geotechnical Journal, 30(4), 647–666. doi:10.1139/t93-056.

[38] Qiu, Z., Wang, J., Chen, Y., Yang, Y., & Li, Y. (2020). Effects of periodic saturation on K0 for a sandstone-mudstone particle mixture. Marine Georesources & Geotechnology, 38(7), 867–875. doi:10.1080/1064119X.2019.1637486.