Undrained shear strength of Lisbon Miocene clay: a reappraisal based on triaxial and pressuremeter test results

Mafalda Lopes Laranjo¹ · Manuel Matos Fernandes²

Received: 12 July 2021 / Accepted: 12 October 2021

Published online: 11 December 2021

© The Author(s) 2021

Abstract

Undrained shear strength plays a fundamental role on the behaviour of clays. In overconsolidated clays, this parameter is largely influenced by test conditions, namely consolidation stress. “Prazeres Clay” is a Miocene overconsolidated formation, that can be found in a significant part of Lisbon area. Over the last decades a number of very relevant constructions have generated a large database for physical and mechanical properties of Miocene clays. Included in a broader study at the Faculty of Engineering of Porto University about Miocene clay’s physical and mechanical properties, existing data was gathered, treated and critically analysed, in order to establish a useful framework for geotechnical designers. This paper presents the results obtained for undrained shear strength, obtained from triaxial tests and Ménard Pressuremeter tests. It addresses the main difficulties associated with test’s interpretation and presents a discussion on how theoretical values relate to experimental ones. The paper proposes a range of variation for *Prazeres Clay*’ undrained shear strength based on a significant amount of test results, that is considered to be useful for geotechnical design.

Article Highlights

- Undrained Shear strength is a relevant parameter for clays, and is usually derived from triaxial tests
- For overconsolidated clays, this parameter is highly dependent on preconsolidation stress, and on its relation to in situ stress.
- Based on a significant set of data, the paper presents a simple methodology for estimating this parameter

Keywords Miocene clays · Undrained shear strength · Overconsolidation · Triaxial tests · Pressuremeter tests

1 Introduction

Lisbon Miocene series addresses an almost continuous sedimentation process over the last 16 million years on the vestibular area of Tagus River Basin [1]. The sequence of transgressive and regressive geological events, associated with changes in the position of the shore line on Lisbon and Setubal region, has led to the deposition of

a nearly 300 m thick layer of sediments [2]. This Miocene Series is considered as a referential for Western Europe, due to its geographical location and to the alternate presence of continental and marine sediment layers, allowing to establish a rigorous series of events [3].

From the depositional cycle’s sequence and the variation of continental and marine environments, it is possible to observe throughout Lisbon Miocene Series a broad

✉ Mafalda Lopes Laranjo, mlopes@estg.ipv.pt; Manuel Matos Fernandes, mfern@fe.up.pt | ¹Polytechnic Institute of Viana do Castelo; Av. Atlântico, 4900-348 Viana do Castelo, Portugal. ²Faculty of Engineering, University Porto, Rua Dr. Roberto Frias, s/n, 4200-465 Porto, Portugal.

range of soils and rocks, where the first prevail. Among these soils, *Prazeres* Clay has a particular relevance. It comprises argillite, silty argillite, marly argillite and marls [4].

In Lisbon Geological Chart [5] this formation encompasses a significant part of the City subsoil, namely in areas of high patrimonial and historical interest. Therefore, geotechnical design for these locations requires deep knowledge on the shear strength and deformation properties of this formation.

Included in a broader study performed at the Faculty of Engineering of the University of Porto, concerning Lisbon Miocene clayey soils, a large amount of data on *Prazeres* Clay layer has been collected and compiled, from geotechnical surveys undergone during the last decades, as the City has developed and inevitably expanding itself underground. This valuable set of data includes results from in situ and laboratory tests concerning a number of relevant structures and infrastructures, hence performed at different locations. Its systematic study and treatment allowed to deepen the knowledge on both physical and mechanical properties of *Prazeres* Clay [6, 7]. Additionally, some data was directly obtained by the authors from an Experimental Site, where field tests have been carried out together with undisturbed block sampling for laboratory testing [6, 7].

This paper focuses on undrained shear strength obtained from triaxial tests in the laboratory and Ménard pressuremeter in situ tests. The comparison of thoroughly analysed and organized data from laboratory and in situ tests, allowed to establish trends on the change in depth of undrained shear strength, with two straight lines representing its lower and upper limits. Before presenting and discussing the undrained shear strength results, a summary of the basic physical properties is presented, in order to establish a reference framework for the mechanical parameters.

2 Methodology

Gathering and treatment of existing data is a very complex and long-lasting task, as information comes from different sources and is organized in distinct ways. All geological and geotechnical reports that have been analysed, mainly referring to works undergone during Lisbon Subway expansion since 1976, present a clear location of the site where samples were collected and the depth at which tests were performed. These locations have to be overlaid on the Lisbon Geological Chart to assure that they refer to the formation under analysis. After this initial screening, it is necessary to choose the reports that include *Prazeres* Clay and check if there is more information than borehole

logs or Standard Penetration Test results. Data is then organized by location and content.

Concerning *Prazeres* Clay physical properties, a total of 495 test results were analysed, from 59 locations. The physical characteristics addressed were grain size distribution, Atterberg limits, natural water content, unit weight and void ratio.

Triaxial test results were available for both compression and extension undrained tests. In this paper, just the undrained compression tests have been considered, corresponding to tests performed on 75 samples from 10 different sites. The reports provide clear information on the test consolidation conditions and on the complete deviator stress and excess pore pressure *versus* axial strain curve, permitting to identify the value of the deviator stress at failure. All the specimens were 50 mm diameter.

The specimens from the Experimental Site were carefully trimmed from large block samples (approximately $0.35 \times 0.35 \times 0.30 \text{ m}^3$). Saturation of the specimens involved the conventional use of backpressure up to 300 kPa. The tests on these specimens involved a large range of consolidation stresses, which in some cases required the use of a high-pressure triaxial cell. Specimens for conventional triaxial tests were 50 mm diameter, whereas for the high-pressure cell we used 70 mm diameter specimens. Prior to these tests, a set of oedometer tests were also performed, to assess preconsolidation stress and overconsolidation ratio.

Ménard Pressuremeter test results were also gathered from existing reports, that provide pressuremeter modulus (E_M) and limit pressure (p_l). Some results had to be discarded as the reports mentioned membrane failure. In addition, three more Ménard Pressuremeter tests were carried out at the Experimental Site mentioned above, the results of which are also presented. Undrained shear strength was obtained from the ratio between the net limit pressure and an empirical nondimensional parameter β [8, 9].

After a thorough analysis and discussion of the results from laboratory and in situ tests, a profile for the variation in depth of the undrained shear strength is proposed.

3 Physical properties

Prazeres Clay present a wide diversity on grain size distribution, as shown by Fig. 1a, which contains Feret's classification, and by Fig. 1b, which refers to the percent distribution of particles by size [7]. It should be noticed that sedimentation results were only available in some reports; thus, clay fraction information is more limited. Silt percentage is the highest, whereas sand and clay fractions vary in a wide interval. From a set of 140 samples, composition

Fig. 1 Grain size distribution for Prazeres Clay: **a** Feret's triangle; **b** percentual distribution

mean values are 48% silt, 35% clay and 17% sand. Therefore, it seems reasonable to classify these soils mainly as silty clays (approximately 47%) and clayey silts (approximately 17%).

Figure 2 presents Atterberg limits, natural water content and consistency index for the gathered samples. Although the depth until which samples refer to is quite large, natural water content does not reveal significant variation over depth, due to the fact that this is a highly overconsolidated

formation. Consistency index is equal to or higher than 1.0 for the majority of samples, as expected.

Casagrande's plasticity chart, presented in Fig. 3, shows that the majority of samples can be classified as CL or CH, and that Plasticity Index can be derived from the following equation:

$$I_p = 0.7 * (w_L - 13.2) \quad (1)$$

Fig. 2 Prazeres Clay indexes: **a** Atterberg limits and natural water content; **b** consistency index

Fig. 3 Casagrande plasticity chart for *Prazeres Clay*

Fig. 4 SEM observations of *Prazeres Clays*, showing compact domains of clay particles [6]

In spite of the high percentage of silt, almost all samples lie above A-line. This is a result from the clay minerals present in the soil. Mineralogical composition by X-ray diffraction [6] shows illite as the predominant clay mineral, although smectite, kaolinite and chlorite are also present, as it occurs in many other marine and estuarine clays [10–12]. Further, the presence of plastic particles which may have been considered as silt in the sedimentation test interpretation, appears to be corroborated by SEM observations [6]. As shown by Fig. 4, the latter revealed a packed

natural clay fabric, with particles aggregated in domains, which is consistent with soil composition and depositional environment [6, 13].

4 Undrained shear strength

4.1 Results of triaxial compression tests collected from pre-existing geotechnical investigation reports

Data concerning undrained shear strength values from triaxial tests in the lab selected from existing reports is summarized in Table 1. It corresponds to tests performed on 75 samples from 11 different sites, collected between 2 and 33 m depth. Undrained shear strength was considered as the half of the maximum deviator stress.

Figure 5 shows raw results, directly from geotechnical reports, for undrained shear strength in depth. Scatter is considerable, mainly due to inconsistencies found in the testing programmes for each case. Actually, it was common practice to request the laboratories to test three specimens, for each sample from a certain depth, using distinct consolidation stresses, in order to obtain a “Mohr Coulomb failure envelope” in effective stresses [14]. As Fig. 5 clearly shows, this leads to s_u values that grow with increasing consolidation stresses, but with no real usefulness, as they refer to distinct overconsolidation ratios.

The complexity of this problem can be easily understood with the help of an e - $\log \sigma'_v$ diagram, as shown in Fig. 6. Point 1 refers to sedimentation and point 2 represents the end of the Miocene era, for which effective

Table 1 Summary of triaxial compression tests selected from existing reports

Site	Type of consolidation	Number of specimens	Depth (m)
Red Line L.S.	Isotropic	6	17.0–22.7
Blue Line L.S.		3	24.95
Yellow Line L.S.		20	7.2–28.8
Alameda tunnel L.S.		4	13.6–30.9
Campo Pequeno Square	K_0	8	19.0–31.2
Duarte Pacheco Avenue		2	20.8–21.15
Rossio tunnel		3	2.15–2.75
Rosa Araújo Street		9	2.15–6.6
Capuchos Hospital	Isotropic	1	9.5
Calhariz Benfca Street		3	3.0–4.65
Baixa-Chiado Station L.S.		3	5.15–6.95
		13	15.9–33.9

L.S. – Lisbon Subway

vertical stress corresponds to the pre-consolidation stress (σ'_p). Point 3 represents the at-rest effective vertical stress, assuming a certain overconsolidation ratio. If the sample is intact, it does not experiment any volume change, being submitted to an isotropic unknown effective stress state, whereas total stresses are null. This is what point 4 is meant to show. When consolidating in the lab a soil sample prior to shear, we are installing a certain effective stress state, represented by point 5. This position will match the at-rest stress state, represented by point 3, just for the cases in which a K_0 consolidation (or an isotropic consolidation equivalent to the mean effective at-rest stress) was adopted. The path followed between positions 3, 4 and 5 on the figure is represented using a dashed line.

In the same figure, some stress ranges are indicated, namely: (i) the effective vertical stress at-rest (σ'_{v0}); (ii) the effective vertical pre-consolidation stress (σ'_p); (iii) the consolidation stress in the triaxial tests described in Table 1 (σ'_c). The effective vertical pre-consolidation stress was evaluated on the basis of oedometer tests [6].

Considering the stress ranges shown in Fig. 6, it becomes clear that all the tests of Table 1 were

Fig. 5 Undrained shear strength in depth for the entire set of results of triaxial compression tests

Fig. 6 e - $\log \sigma'_v$ diagram showing stress state changes that occur on a sample prior to triaxial testing [6]

performed on overconsolidated specimens, though the respective lab overconsolidation ratio was very variable. In some cases, OCR^{test} was lower than the $OCR^{in situ}$, while in other cases it was higher, which conveys further difficulties to test interpretation.

With the previous considerations in mind, from the set of results available and shown in Fig. 5, we considered useful for further analysis only those for which consolidation stresses fulfilled the following conditions:

- for anisotropically consolidated tests:

$$\sigma'_{vc} \approx \sigma'_{v0} \quad (2)$$

- for isotropically consolidated tests:

$$\sigma'_c \approx \frac{\sigma'_{v0} + 2K_0\sigma'_{v0}}{3} \quad (3)$$

In brief, we have only considered the results from tests in which the OCR is close to the in situ value.

Figure 7 shows s_u values from those tests, and one additional result obtained from the Experimental Site. It can be observed that the scatter drastically reduced, allowing to depict an in situ undrained shear strength range in depth. These values naturally correspond to an overconsolidated state, in accordance to *Prazeres Clay* geological history [3].

Fig. 7 Undrained shear strength in depth from triaxial compression tests with consolidation stresses close to effective at-rest stresses

4.2 Results obtained by applying a theoretical equation

A complementary way to check the validity of the trends expressed by Fig. 7, is to apply the theoretical equation for s_u in triaxial compression [15]:

$$s_u = \frac{c' \cos(\phi')}{1 - \sin(\phi') + 2A_f \sin(\phi')} + \sigma'_{v0} \frac{\sin(\phi') (K_0 + A_f - K_0 A_f)}{1 - \sin(\phi') + 2A_f \sin(\phi')} \quad (4)$$

as function of the effective strength parameters, c' and ϕ' , the at-rest pressure coefficient, K_0 , and the Skempton's pore pressure parameter, A_f , and to compare those with experimental values.

Figure 8 shows s_u results from triaxial compression tests represented in Fig. 7, together with the ones obtained by using the theoretical equation (Eq. 4). The latter have been obtained by considering $c'=0$, $\phi'=36^\circ$, A_f in the range -0.1 to 0.3 and K_0 equal to 1.0 for CIU tests and 0.8 for CK0U tests, and assuming linear variation in depth for σ'_{v0} . It should be noticed that the effective strength parameters have been obtained from the treatment of the experimental results with a high degree of confidence [6]. The values assumed for A_f and K_0 are also experimentally based [6].

The lines obtained from the theoretical equation reveal a trend over depth that seems to be consistent with the experimental results. It should be noticed that, since they are based on values of $c'=0$, they correspond to null values of s_u at ground surface. Bearing in mind that we are dealing with overconsolidated soils, it would be expectable that the effective cohesion is, in fact, not null. However, small values of this parameter, which might arise from a distinct treatment of the experimental results in terms of the effective strength parameters, would not alter in a

significant extent the position of the theoretical lines considering the actual scale of s_u in Fig. 8.

4.3 Results of triaxial compression tests from an Experimental Site

In addition to the results obtained from pre-existing geotechnical reports, other results have been obtained from tests performed on samples collected from an Experimental Site at Visconde Valmor Avenue in Lisbon [6]. Block samples were collected at 4.0 m depth, with an effective at-rest vertical stress of 80 kPa.

The testing programme included 8 triaxial compression tests. In addition to the conventional triaxial cell, a high-pressure cell was also used. The main aspects of this testing programme are described in Table 2.

Prior to the triaxial tests, a set of oedometer tests was carried out with an apparatus allowing the application of vertical effective stresses up to 24 MPa, that is well beyond the expected pre-consolidation stress. The results for this variable and the respective OCR values are summarized in Table 3, applying the Casagrande [16] and Butterfield [17] methods.

Fig. 8 Theoretical and experimental values of s_u from triaxial compression tests

Table 2 Summary of CIU triaxial compression tests carried out on samples from the Experimental Site [6]

Test	Sample	Type	Equipment	σ'_c (kPa)
C1	7	CIU	Conventional	40
C2	10	CIU	Conventional	80
C3	1	CIU	Conventional	120
C4	B2A7	CIU	Conventional	500
C5	3	CIU	Conventional	1000
HP1	11	CIU	High-pressure	4700
HP3	9	CIU	High-pressure	9800
HP4	B2A5	CIU	High-pressure	6500

Table 3 Pre-consolidation stress and OCR for samples from the Experimental Site tested on the high-pressure oedometer test

Test	σ'_{v0} (kPa)	Casagrande method		Butterfield method	
		σ'_p (kPa)	OCR	σ'_p (kPa)	OCR
OED1	80	1127	14.1	1245	15.6
OED2	80	1219	15.2	1278	16.0
OED3	80	1369	17.1	1477	18.3

From the results presented in Table 3, it can be concluded that all the triaxial tests of Table 2 using the high-pressure triaxial cell were performed on specimens that can be considered normally consolidated.

The representation of triaxial test results in the p' - q space according to critical state soil mechanics, as shown in Fig. 9, evidences the differences between the behaviour of the normally and the overconsolidated samples. For samples tested under lower consolidation stresses, corresponding to high OCR values, negative excess pore pressures are developed, and then p' progressively increases. On the contrary, for samples tested under higher consolidation stresses, corresponding to the normally consolidated state, positive excess pore pressures are developed, leading to decreasing p' . This is the general trend, with the exception of the sample tested with $\sigma'_c = 500$ kPa. The sample tested with $\sigma'_c = 1000$ kPa reveals a transitional behaviour, with very small excess pore pressures and p' essentially constant.

The following equation is commonly used to describe the relationship between undrained shear strength and overconsolidation ratio:

$$\frac{s_u}{\sigma'_{v0}} = \left(\frac{s_u}{\sigma'_{v0}} \right)_{NC} OCR^\Lambda \quad (5)$$

where $(s_u/\sigma'_{v0})_{NC}$ is the normalized undrained shear strength for the normally consolidated soil and Λ is the plastic volumetric strain ratio [18, 19].

As shown in Fig. 10, this equation has been applied to the triaxial test results, assuming a preconsolidation effective vertical stress of 1200 kPa, according to Table 3, and relating σ'_c used in the CIU tests and σ'_{v0} through equation (Eq. 3). This analysis provides $\left(\frac{s_u}{\sigma'_{v0}} \right)_{NC} = 0.33$ and $\Lambda = 0.46$,

Fig. 9 Stress paths in undrained triaxial tests in the p' - q space: **a** overconsolidated samples; **b** normally consolidated samples

Fig. 10 Normalized undrained shear strength as a function of OCR

which fall in the ranges described in the literature [20]. It should be noticed that the result from the test with $\sigma'_c = 1000$ kPa was not considered, since it reveals a significant deviation from the remainder.

Figure 11 compares the ratio s_u/σ'_c obtained from the tests with the curve derived from equation (Eq. 5), assuming a preconsolidation stress of 1200 kPa and Λ equal to 0.46. The curve fits reasonably well the experimental data (with the exception of the test with $\sigma'_c = 1000$ kPa, as mentioned above). In what normally consolidated samples are concerned, shown in Fig. 11b, normalized undrained shear strength of 0.33 seems to fit the results reasonably well.

Fig. 11 Undrained shear strength normalized by the test effective consolidation stress versus effective consolidation stress: **a** overconsolidated samples; **b** normally consolidated samples

4.4 Results from Ménard Pressuremeter tests (PMT)

Results of Ménard pressuremeter tests (PMT) from existing reports were also part of the entire set of analysed data [6]. Data concerning *Prazeres Clay* comprise 146 PMT tests performed at 16 different sites, at depth between 3.5 and 46.5 m. Some reports mention that failure of the membrane occurred, hence providing unrealistic values for the limit pressure. After a careful initial screening, 98 results were considered in the analysis.

Soil type can be evaluated on the basis of the ratio between the pressuremeter modulus (E_M) and the limit pressure (p_l) [21]. Despite the scatter, it was found an average value of 16.9 for that ratio, confirming that the soils under study are stiff to very stiff clays [21, 22].

The estimation of the undrained shear strength from the limit pressure is usually done through the equation:

$$s_u = \frac{p_\ell^*}{\beta} \quad (6)$$

where p_ℓ^* is the net limit pressure, given by

$$p_\ell^* = p_\ell - \sigma_{h0} \quad (7)$$

and β is a dimensionless empirical parameter. For the soils under study, $\beta = 15$ is recommended [9, 23].

Figure 12 shows the undrained shear strength values obtained according to this methodology from the 98 tests selected for analysis from existing reports, together with the results found by the authors at the Experimental Site [24]. It is possible to see that Experimental Site results for

Fig. 12 Undrained shear strength obtained from PMT test results

s_u are consistent with the ones obtained from the existing reports.

5 Discussion

Undrained shear strength variability has been described for other stiff clays [25–28]. The attempt to establish a trend on undrained shear strength for *Prazeres* Clay is a rather arduous task, since we are dealing with a formation and not a soil, and natural variability may occur both laterally and in depth [26, 29]. In sedimentary soils this variability may arise from depositional environment, post depositional and late stage processes, composition, stress history and stress state, among others [26, 29–32]. Moreover, it is necessary to consider that testing conditions, whether in situ or in the laboratory, also add variability to results [29, 30, 33].

Although triaxial test results may be considered more reliable than Ménard pressuremeter tests, since boundary conditions are imposed and directly controllable, there are several factors affecting its results. Sampling in

overconsolidated clays may produce changes in the mean effective stress in the sample, cause water content redistribution and modify soil structure [30, 31, 34]. In other stiff clays, the presence of unfavourably oriented fissures and shear strength anisotropy are also known to introduce variability [25, 35, 36].

On the other hand, the estimation of undrained shear strength from the pressuremeter theoretical limit pressure, although commonly adopted, is not absent of inconsistencies. Since these tests are performed on a pre-drilled borehole, minimizing soil disturbance is a critical issue [22, 37]. Theoretical limit pressure may not be reached in some tests, given the limited water volume of the control unit [38]. Several authors have reported that although the cylindrical cavity expansion theory should provide a sound basis for obtaining s_u , the interpreted strength results are often inconsistent with high-quality laboratory results [39]. Factors like the presence of a disturbed annular zone of remoulded soil around the probe, an irregular non-cylindrical cavity initial shape, high strain rate and distinct deformation patterns and stress paths are pointed out as causes for poor estimations [22, 38, 40–42].

The comparison of thoroughly analysed and organized data from laboratory triaxial and in situ pressuremeter tests concerning *Prazeres* Clay allowed to establish trends on the change in depth of the undrained shear strength, summarized in Fig. 13. It is rather curious to mention that the limits indicated in the figure are the same already presented in Fig. 7 for the triaxial test results.

In fact, although there is some scatter, s_u values from pressuremeter tests seem to evolve in depth in a very good agreement with the range found from the analysis of the triaxial tests results. It is worth to notice that, for a given depth, effective at-rest stress may range in a given interval, depending on the position of the water table. For the areas where the majority of the results was obtained, the values of the median of the depth of the water table is in the range 4.0–5.0 m [43, 44]. The results above the upper limit line from pressuremeter tests at shallower depths might probably correspond to tests performed above the water table suffering the influence of suction [42].

The two linear limits, drawn in Fig. 13, that encompass the great majority of the results, can be expressed as:

$$s_{u \min} = 36.4 + 4.5 \cdot z \text{ (kPa, m)} \quad (8)$$

$$s_{u \max} = 79.7 + 22.8 \cdot z \text{ (kPa, m)} \quad (9)$$

Figure 14 presents undrained shear strength for *Prazeres* Clay together with data for London Clay obtained from triaxial tests adapted from other authors [30, 35]. For the latter, the figure shows a range for undrained shear strength from triaxial compression tests (TC) [35] whereas the

Fig. 13 Proposed trends for undrained shear strength in depth for *Prazeres* Clay

short-dashed lines refer to an estimated s_u profile based on effective stress considerations [30]. It is important to notice that these estimated profiles for London Clay derive from scattered values, that besides natural variability were attributed to sampling disturbance [30, 45]. Despite the scatter, there is a trend to increasing strength with depth. London Clay lower limit and estimated profiles, stay close to the lower limit for *Prazeres* Clay, given by Eq. 8. Upper limit falls short on the results for *Prazeres* Clay, particularly for depth beyond 20 m.

6 Conclusions

Prazeres Clay is a Miocene clayey formation with great importance which underlies the city of Lisbon. Its geological history is complex as it results from 16 million years of continuous sedimentation. These deposits, around 300 m thick at some locations, have suffered a series of transgressive and regressive geological events, associated with sea water level variations. This has led to high values of the overconsolidation ratio.

In this paper, a large amount of results has been treated both from triaxial compression and PMT tests. The results

obtained reveal a reasonable overall agreement between the results from laboratory and in situ tests. It was confirmed that the use of $\beta = 15$ to derive s_u from the limit pressure of PMT is appropriate.

The change in depth obtained from the theoretical approach, using effective strength parameters c' and ϕ' , seems to fit reasonably well the triaxial test results, when consolidation stresses are close to in situ stresses.

Normalized undrained shear strength for the normally consolidated soil and plastic volumetric strain ratio were determined for samples from an Experimental Site, and fall in the ranges described in the literature. However, these results are limited, thus they should be carefully considered.

A range of variation with depth of in situ undrained shear strength, that encompasses a large majority of the results, is proposed. Scatter of results suggests that these limits should be considered with caution. However, they might be useful for preliminary design calculations as well as to check the reasonableness of new experimental values.

Naturally, more experimental and field data are needed to improve the applicability of the proposed trends. The effects of overconsolidation ratio on undrained shear strength determined from laboratory tests should be

Fig. 14 Undrained shear strength from triaxial tests for *Prazeres* Clay and London Clay (adapted from [30, 35])

deepened. The suitability for normalization of *Prazeres* Clay should be investigated by means of systematic laboratory tests, involving undisturbed and reconstituted samples. SEM analysis and mineralogical composition might be helpful in clarifying some aspects related to structure that also influence mechanical behaviour.

Acknowledgements This work was developed within the scope of the project proMetheus – Research Unit on Materials, Energy and Environment for Sustainability, FCT Ref. UID/05975/2020, financed by national funds through the FCT/MCTES. This work was financially supported by: Base Funding - UIDB/04708/2020 of the CONSTRUCT—Instituto de I&D em Estruturas e Construções, funded by national funds through FCT/MCTES (PIDDAC). Laboratory tests were performed in the Geotechnical Laboratory of FEUP.

Authors' contribution All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by Mafalda Lopes Laranjo. The first draft of the manuscript was written by Mafalda Lopes Laranjo and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

Funding This work was developed within the scope of the project proMetheus – Research Unit on Materials, Energy and Environment for Sustainability, FCT Ref. UID/05975/2020, financed by national funds through the FCT/MCTES. This work was also supported by

UID/ECI/04708/2019—CONSTRUCT—Instituto de I&D em Estruturas e Construções, funded by national funds through FCT/MCTES (PIDDAC).

Declarations

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

Availability of data and material: Authors can confirm that all relevant data are included in the article.

Ethics approval This article does not have any potential conflicts of interest.

Consent to participate The authors have no relevant financial or non-financial interests to disclose. The authors have no financial or proprietary interests in any material discussed in this article.

Consent for publication The authors have no conflicts of interest to declare that are relevant to the content of this article. All authors certify that they have no affiliations with or involvement in any organization or entity with any financial interest or non-financial interest in the subject matter or materials discussed in this manuscript.

Open Access This article is licensed under a Creative Commons Attribution 4.0 International License, which permits use, sharing, adaptation, distribution and reproduction in any medium or format, as

long as you give appropriate credit to the original author(s) and the source, provide a link to the Creative Commons licence, and indicate if changes were made. The images or other third party material in this article are included in the article's Creative Commons licence, unless indicated otherwise in a credit line to the material. If material is not included in the article's Creative Commons licence and your intended use is not permitted by statutory regulation or exceeds the permitted use, you will need to obtain permission directly from the copyright holder. To view a copy of this licence, visit <http://creativecommons.org/licenses/by/4.0/>.

References

- Antunes MT, Legoinha P, Cunha PP, Pais J (2000) High resolution stratigraphy and miocene facies correlation in Lisbon and Setúbal Peninsula (Lower Tagus basin, Portugal). *Ciências da Terra (UNL)* 14:183–190
- Pais J, Moniz C, Cabral J, Cardoso JL, Legoinha P, Machado S, Morais MA, Lourenço C, Ribeiro ML, Henriques P, and Falé P (2006) Geological Chart of Portugal, Explanatory News of Folha 34D Lisbon
- Almeida IM (1991) Características geotécnicas dos solos de Lisboa. University of Lisboa, Geologia
- Zbyszewski G (1947) Panorama sur la Geologie de la Ville de Lisbonne. *Boletim da Sociedade de Geografia de Lisboa*, 65 Série(9–10)
- Moitinho de Almeida F (1986) Carta Geológica do Concelho de Lisboa. S.G.d. Portugal, Editor, Serviços Geológicos de Portugal
- Lopes Laranjo M (2013) Argilas Miocénicas de Lisboa. Civil Engineering, University of Porto, Parametrisação para o Dimensionamento de Estruturas Geotécnicas
- Lopes Laranjo M, Matos Fernandes M, and Almeida e Sousa J (2011) Some basic trends on the geotechnical characteristics of Lisbon Miocene Clays. *Proc. of XV European Conference on Soil Mechanics and Geotechnical Engineering*:351–356. <https://doi.org/10.3233/978-1-60750-801-4-351>
- Briaud J-L (2013) Geotechnical engineering: unsaturated and saturated soils. Wiley
- Briaud JL (1992) The pressuremeter. A.A. Balkema
- Skempton A (1953) The colloidal activity of clays. *Selected papers on soil mechanics*:106–118
- Tan T, Phoon K, Lee F, Tanaka H, Locat J, and Chong P (2003) A characterisation study of Singapore lower marine clay. *Characterization and engineering properties of natural soils*, AA Balkema, Lisse, the Netherlands, 1:429–454
- Tanaka H, Ritoh F, and Omukai N (2003) Geotechnical properties of clay deposits of the Osaka Basin, *Characterisation and engineering properties of natural soils*. AA Balkema, Lisse, the Netherlands, 1:455–474
- Cotecchia F, Mitaritonna G, and Vitone C (2011) Investigating the influence of microstructure, fissuring on the clay response. In: *proc. of deformation characteristics of geomaterials: proceedings of the fifth international symposium on deformation characteristics of geomaterials*, IS-Seoul 2011, 1–3 September 2011, Seoul, Korea:53
- Mayne P (2007) In-situ test calibrations for evaluating soil parameters. *Characterization & engineering properties of natural soils*, AA Balkema, Taylor & Francis 3:1601–1652
- Matos Fernandes M (2006) *Mecânica dos Solos. Princípios e Conceitos Fundamentais*. Edições FEUP
- Casagrande A (1936) The determination of the preconsolidation load and its practical significance. In: *proc. of 1st int. conf. on soil mechanics and foundation engineering*, 3:60–64
- Butterfield R (1979) A natural compression law for soils (an advance on e -log p'). *Géotechnique* 29(4):469–480. <https://doi.org/10.1680/geot.1979.29.4.469?journalCode=jgeot>
- Ladd CC, Foott R, Ishihara K, Schlosser F, and Poulos HG (1977) Stress-deformation and strength characteristics. In: *Proc. of 9th international conference on soil mechanics and foundations engineering*, 2:421–494
- Mayne PW (1988) Determining OCR in Clays from Laboratory Strength. *Journal of Geotechnical Engineering* 114(1):76–92
- Gutierrez M, Nygård R, Høeg H, Berre T (2008) Normalized undrained shear strength of clay shales. *Eng Geol* 99:31–39. <https://doi.org/10.1016/j.enggeo.2008.02.002>
- Clarke GB (1995) *Pressuremeters in geotechnical design*. C. Press Chapman and Hall
- Baguelin F, Jezequel J, and Shields D (1978) *The pressuremeter and foundation engineering*.
- Cassan M (1978) *Les Essais in situ en Mécanique des Sols*. Tome 1: Réalisations et Interpretation.
- Lopes Laranjo M, Matos Fernandes M, and Viana da Fonseca A (2015) Selfboring and Ménard Pressuremeter Tests in Lisbon Miocene Prazeres Clay. *Proc. of Symposium International ISP7/ PRESSIO* 2015:165–172
- Hight D, McMillan F, Powell J, Jardine R, and Allenou C (2003) Some characteristics of London clay. *Characterisation and engineering properties of Natural Soils*, AA Balkema, Lisse, the Netherlands 2:851–946
- Powell J and Butcher A (2003) Characterisation of a glacial clay till at Cowden, Humberside. *Characterisation and engineering properties of Natural Soils*, AA Balkema, Lisse, the Netherlands 2:983–1020
- Picarelli L, Olivares L, Di Maio C, Silvestri F, Di Nocera S, and Urcioli G (2003) Structure properties and mechanical behaviour of the highly plastic intensely fissured Bisaccia clay shale. *Characterisation and engineering properties of Natural Soils*, AA Balkema, Lisse, the Netherlands 2:947–982
- Uzielli M, Lacasse S, Nadim F, and Phoon K (2007) Soil variability analysis for geotechnical practice. *Characterisation and engineering properties of Natural Soils* AA Balkema, Taylor & Francis 3:1653–1754
- Clayton C and Hight D (2007) Laboratory testing of natural soils—Some factors affecting performance. *Characterisation and Engineering Properties of Natural Soils*, AA Balkema, Taylor & Francis 3:1535–1599
- Hight D and Leroueil S (2003) Characterisation of soils for engineering purposes. *Characterisation and engineering properties of Natural Soils*, AA Balkema, Lisse, the Netherlands 1:255–360
- Gasparre A, Nishimura S, Coop M, Jardine R (2007) The influence of structure on the behaviour of London Clay. *Géotechnique* 57(1):19–31
- Locat J, Tanaka H, Tan T, Dasari G, and Lee F (2003) Natural soils: geotechnical behavior and geological knowledge. *Characterisation and engineering properties of Natural Soils*, AA Balkema, Lisse, the Netherlands 1:3–28
- Pantelidou H, Simpson B (2007) Geotechnical variation of London clay across central London. *Géotechnique* 57(1):101–112
- Sorensen KK, Baudet B, Simpson B (2007) Influence of structure on the time-dependent behaviour of a stiff sedimentary clay. *Géotechnique* 57(1):113–124
- Nishimura S, Minh N, Jardine R (2007) Shear strength anisotropy of natural London clay. *Géotechnique* 57(1):49–62
- Cotecchia F, Vitone C, Cafaro F, and Santaloia F (2006) The mechanical behaviour of intensely fissured high plasticity clays from Daunia. In: *proc. of the second international workshop on characterization and engineering properties of natural soils*, invited lecture, Singapore, 3:1975–2003

37. Masoud Z, Khan AH (2019) An improved technique for prebored pressuremeter tests. *KSCE J Civ Eng* 23(7):2839–2846. <https://doi.org/10.1007/s12205-019-1448-5>
38. Isik NS, Ulusay R, Doyuran V (2015) Comparison of undrained shear strength by pressuremeter and other tests, and numerical assessment of the effect of finite probe length in pressuremeter tests. *Bull Eng Geol Env* 74(3):685–695
39. Aubeny C, Whittle A, Ladd C (2000) Effects of disturbance on undrained strengths interpreted from pressuremeter tests. *J Geotech Geoenviron Eng* 126(12):1133–1144
40. Yeung S and Carter JP (1990) Interpretation of the pressuremeter test in clay allowing for membrane end effects and material non-homogeneity. In: *proc of the third international symposium on pressuremeters, british geotechnical society, oxford university, 2–6 April 1990*:199–208
41. Houlsby G, Carter J (1993) The effects of pressuremeter geometry on the results of tests in clay. *Géotechnique* 43(4):567–576
42. Briaud JL (2013) Ménard Lecture - The pressuremeter test: Expanding its use. In: *proc. of the 18th international conference on soil mechanics and geotechnical Engineering*, I:107–121
43. Pinto CN (2020) Analysis and modeling of geological and geotechnical data aiming geotechnical mapping in Lisbon. University of Lisbon
44. Fernandes MM (2020) Analysis and design of geotechnical structures. C.P.T. Francis
45. Hight D (2003) Behaviour and properties of natural soils and soft rocks. Characterisation and engineering properties of Natural Soils, AA Balkema, Lisse, the Netherlands 1:29

Publisher's Note Springer Nature remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.