Shear strength and compressibility of reconstituted Boom clay, a stiff clay from the Paleogene

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ABSTRACT

Boom clay is a stiff marine clay from the Oligocene epoch of the Paleogene period. It outcrops along the rivers Rupel and Scheldt in Northern Belgium, nearby the city and the port of Antwerp. Most of the existing geotechnical investigation around the Antwerp area, where the clay is located at relatively shallow depths, consists of standard laboratory tests on undisturbed samples performed at stress levels around the estimated in situ stress. Although the geological preconsolidation pressure of this clay is relatively well known, the clay yields at much higher stresses in 1D compression. Moreover, due to the brittle behavior of the stiff clay, it is often difficult to evaluate its shear strength behavior beyond peak strength. To this end, evaluating intrinsic parameters of compressibility, critical state shear angle and residual strength is of great relevance. In the present research, the mechanical behavior of reconstituted normally consolidated Boom clay was examined in a series of oedometer, CRS, CU triaxial, and multi-reversal direct shear tests. The mechanical behavior of the reconstituted clay was compared to the behavior of undisturbed clay samples. The results allow for a better evaluation of the structure effects of the natural samples on their mechanical response.

Keywords: stiff clay; shear strength; stiffness; reconstitution.

1. Introduction

Boom clay is a thick medium plastic clay deposit outcropping in the northern part of Belgium along the rivers Rupel and Scheldt. This Paleogene clay belongs to the Oligocene epoch.

According to Vandenberghe et al. (2014) Boom clay has been studied since the nineteenth century. Geologists studied the fossils of the Boom Clay derived from the deposits in the outcrop areas. Studies became more relevant with the advent of the research to evaluate the potential of the Boom clay as a host formation for the long-term geological disposal of highly radioactive waste. Laboratory facilities at 225 m below the ground surface were built in the 80s in Mol to study the mechanical, chemical and biological interaction of the Boom clay and radioactive waste.

Similarly, extensive research has been performed in the Antwerp area to evaluate the mechanical behavior of the clay as many civil constructions, tunnels, and port facilities are founded on the Boom clay. Today, ongoing large projects around Antwerp demand further geotechnical characterization with emphasis on compressibility, swelling and shear strength.

In the Antwerp area, the Boom clay has a thickness of 60 m to 80 m. It dips 1-2 % towards the northeast and it increases in thickness in that direction. In the Campine area and close to the border with The Netherlands, the top of the Boom clay may be located at a depth between 200m and 300m reaching a thickness of about 150m.

As suggested by Schittekat (2001) the thickness of about 150m may have also been present in the Antwerp area, but due to later erosion processes it was reduced to about 60 m to 80 m. Based on these geological considerations Schittekat (2001) estimates a removed overburden of about 90m for the Antwerp area.

Fig. 1 illustrates a geological cross section from South to North along the Antwerp area. The Boom formation has been subdivided into up to 4 main stratigraphic units. From bottom to top, the Belsele-Waas Member (BmBw), which is the siltiest part of the Boom Formation; the Terhagen Member (BmTe), characterized by pale grey clay; the Putte Member (BmPu), characterized by dark clay and the systematic presence of organic matter; and finally the Boeretang Member which is rich in silt and more prominent in the Campine area.

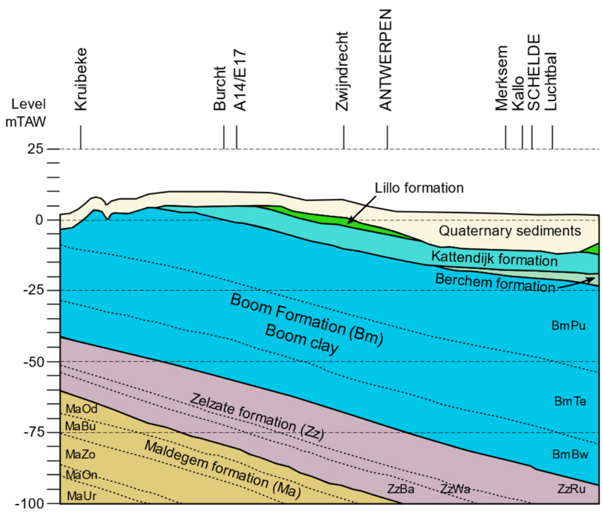


Figure 1. Geological section from South to North along the Antwerp area (adapted from Matthijs et al., 2003).

The mechanical behavior of Boom clay is often complex to evaluate due to its brittle behavior and tendency to swell. To this end, the determination of intrinsic parameters of compressibility, critical state shear angle, and residual strength are of great relevance to evaluate the effect of structure and stress history.

In the present research, the mechanical behavior of reconstituted normally consolidated Boom clay was examined in a series of oedometer, CRS, CU triaxial, and multi-reversal direct shear tests. The material was sampled from an outcrop nearby Kruibeke (Fig. 1).

The behavior of the reconstituted clay was then compared to the behavior of undisturbed Boom clay samples from the area of Antwerp coming from depths of about 30m.

2. Reconstituted sample preparation

Reconstitution was done by mixing the clay and demineralized water into a slurry and then consolidating the material under selected vertical stresses. Table 1 summarizes some index properties of the clay.

To obtain a uniformly textured slurry, the natural stiff clay blocks were first reduced to thin shavings with the help of a knife. The clay shavings were allowed to soak in demineralized water for a period of 24h. Initially, a slurry water content between 1.25 and 1.5 times the liquid limit (Burland, 1990) was aimed. But to improve the disaggregation of the clay and uniformity of the slurry during mixing, the water content was raised to 1.8 times the liquid limit. After soaking, the slurry was mechanically mixed in a dough mixer for a period of about 8h until a smooth texture was obtained.

Next, the slurry was brought into a 15cm diameter cylindrical mold provided with drainage at the top and bottom (Fig. 2). Then, the sample was slowly consolidated under a vertical load of either 40 kPa or 80 kPa. Samples consolidated at 40 kPa were used for mechanical tests under low normal stresses to preserve the normally consolidated (NC) nature of the soil samples. The consistency of the sample consolidated to 40 kPa (Fig. 1) was stiff enough to allow further subsampling and handling. Table 1 summarizes some properties of the reconstituted samples.

Figure 2. Consolidation and extrusion of a reconstituted Boom clay sample.

Table 1. Reconstituted soil properties

	Value
Plastic limit (%)	22
Liquid limit (%)	72
Plasticity index (%)	50
Preconsolidation stress (kPa)	40 or 80
Water content (%)	64.5 or 51.5
Unit weight (kN/m^3)	16.3 or 16.8
Void ratio	1.6 or 1.3

3. Methods

The reconstituted soil samples were subjected to the following mechanical tests:

- Triaxial compression CU, on specimens with a diameter of 50mm and height of 100mm. Procedure according to EN ISO 17892-9
- Multi-reversal direct shear test, on a 60x60x20 mm specimen. Procedure according to EN ISO 17892-10
- Constant rate of strain test on a specimen with a diameter of 70mm and height of 20mm. Procedure according to ASTM D4186M-12
- Incremental load oedometer test on a specimen with a diameter of 63mm and a height of 20mm. Procedure according to EN ISO 17892-5.

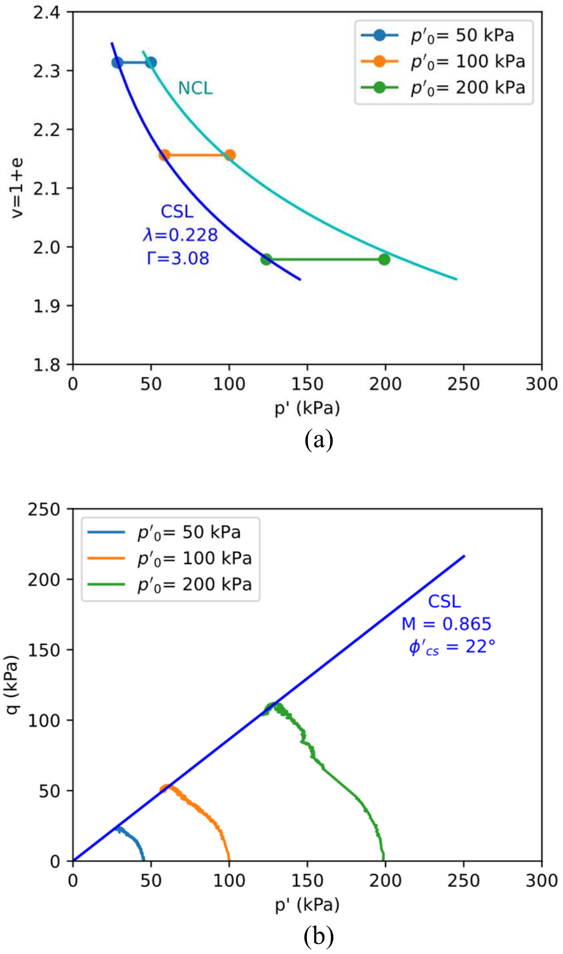
4. Results on reconstituted samples

4.1. Shear strength and stiffness

The shear strength and stiffness of the reconstituted Boom clay were determined through a series of CU triaxial tests. After a saturation phase at a backpressure of 500 kPa under an effective stress of about 5 kPa (reaching B-values > 0.97), the samples were isotropically consolidated to stresses of 50 kPa, 100 kPa and 200 kPa. The state after consolidation defines the normal compression line (NCL) as depicted in Fig. 3a.

Volume change is not allowed during undrained shearing, therefore as expected for NC soil, an increase of pore water pressure was recorded. This leads to a decrease of the mean effective stress (p') until a failure state is asymptotically reached at the critical state. The end state either in the specific volume (v) vs. p' or deviatoric stress (q) vs. p' diagrams (Fig. 3a and 3b respectively) defines the critical state line (CSL).

A critical stress ratio M=0.865, equivalent to a shear angle $\varphi'_{cs} \approx 22°$, was determined. In the v:p' diagram the CSL is defined by a gradient $\lambda = 0.228$ and a specific volume intercept at $p' = 1$ kPa of $\Gamma = 3.08$.



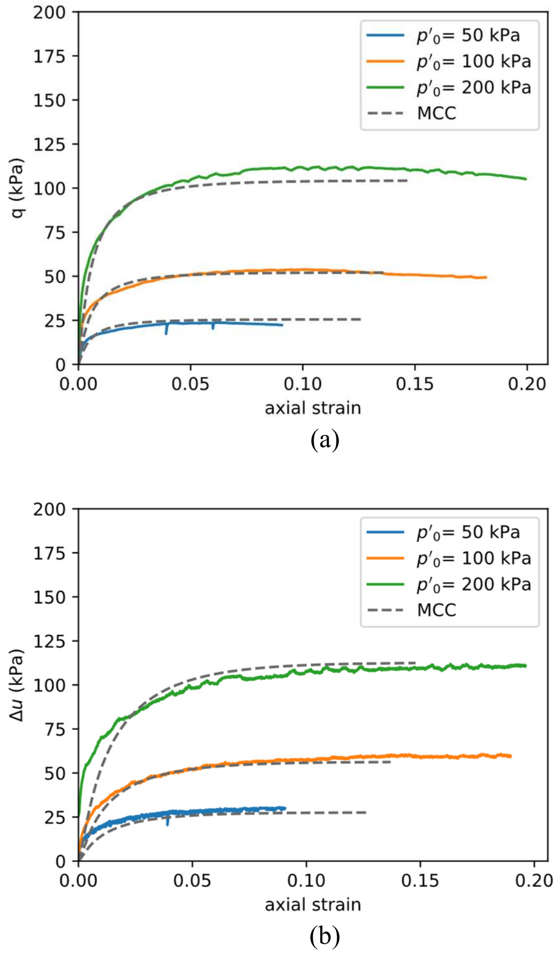


Figure 3. Stress paths of triaxial CU tests (a) specific volume vs. p′ (b) deviatoric stress vs. p′.

Figure 4. Triaxial compression CU tests (a) stress-strain curve (b) excess pore pressure.

The evaluated shear angle $\varphi'_{cs} \approx 22°$ agrees well with data for the clay of similar plasticity as reported by Mitchell & Soga (2005). Still, it is slightly higher than the value of 18.5° reported by Bouazza et al. (1996) for reconstituted Boom clay (starting from a slurry at a water content 1.26 times its liquid limit). However, Bouazza et al. (1996) also reported a cohesion intercept of c′=10 kPa. Moreover, all their stress-strain curves exhibited a brittle behavior with a distinct peak undrained strength which, as the authors relate, is rare for NC reconstituted soil. Their results were probably affected by insufficient saturation of the specimens (Bouazza et al., 1996).

The shear behavior of reconstituted Boom clay in this research seems to show typical features of ideal textbook shearing. In fact the modified Cam Clay model (MCC) fits rather accurately the stress-strain response and excess pore water pressure mobilization of all three specimens as illustrated in Fig. 4.

Fig. 5 illustrates the evaluated undrained modulus E_{u50} as a function of the consolidation stress p'_0, both normalized with respect to the atmospheric pressure p_a = 100 kPa. An almost linear relationship could be observed.

Another relevant shear parameter is the residual strength shear angle φ'_r.

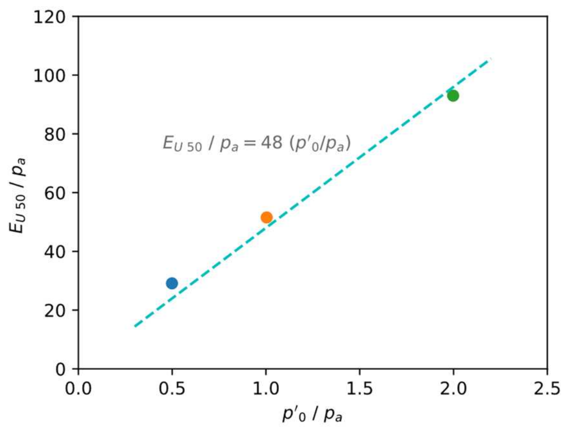


Figure 5. Undrained modulus E_{u50} vs. p'_0 normalized by the atmospheric pressure p_a = 100 kPa.

The importance of the residual strength of soils is well recognized in several geotechnical problems, such as the reactivation of landslides and soil structure interfaces where large deformations are expected.

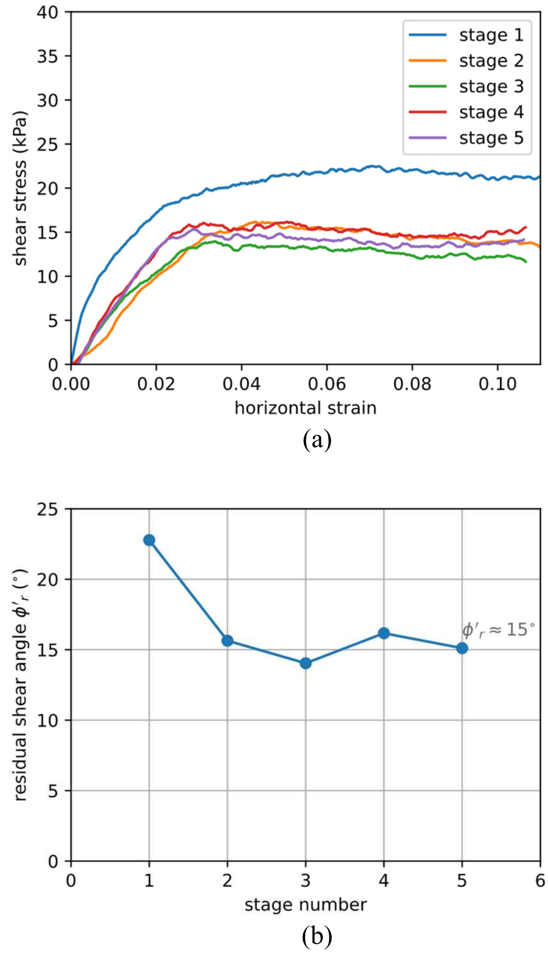


Figure 6. Residual strength of reconstituted Boom clay under a normal effective stress of 50 kPa.

To this end multi-reverse direct shear tests were performed on one specimen subjected to a normal stress of 50 kPa. Fig. 6a illustrates the stress-strain curves of each of the five shearing stages at a deformation rate of 0.0006 mm/min. After each stage the sample was brought back to its original position at the same deformation rate and it was allowed to reconsolidate. Fig. 6b shows the evaluated shear angle from each shearing stage. The evaluated shear angle of the first stage approximately matches the critical state angle, but then it quickly decreases and seems to oscillate around $\varphi'_r \approx 15°$.

This value agrees well with general data reported in the literature for clay of similar plasticity and subjected to the same normal stress (Stark & Hussain, 2013). It is well known that the residual shear angle tends to decrease with increasing normal effective stress. Merchan et al (2011) reported $\varphi'_r = 13°$ for saturated Boom clay under normal stresses up to 200 kPa. De Beer (1967) reported a broader range of values, $\varphi'_r = 10°$ to $15°$ over a broader range of normal stresses, up to 400 kPa, on pre-cut Boom clay intact samples. The lowest residual shear angle corresponds to the higher normal stress range. De Beer (1967) highlighted the impact of normal stress and difficulties of testing of intact samples on the determination of φ'_r of stiff clay.

4.2. 1D compression

The one-dimensional compressibility of reconstituted Boom clay was evaluated by means of a CRS test and an incremental loading oedometer test. As illustrated in Fig. 7a, both samples were first loaded from their initial state to a normal effective stress of about 1000 kPa. Next, the samples were unloaded to about 60 kPa and finally reloaded to about 2000 kPa. This is a typical stress range often implemented in local practice for undisturbed sample testing. The results of both, the CRS and oedometer test, show very good agreement.

These samples were reconstituted under a normal stress of 40 kPa, so the compression curve displays a stiffer response for $\sigma'_v < 40$ kPa. Beyond a normal stress of about 60 kPa the reconstituted sample shows a well-defined normal compression line, also called the intrinsic compression line (ICL) as the properties of a reconstituted sample refer to the inherent or intrinsic properties of the clay and they are independent of the natural state of the clay (Burland, 1990).

The following intrinsic parameters can be evaluated: e^*_{100}=1.24 and e^*_{1000}=0.72, which correspond to the void ratio on the ICL at reference normal stresses of 100 kPa and 1000 kPa respectively.

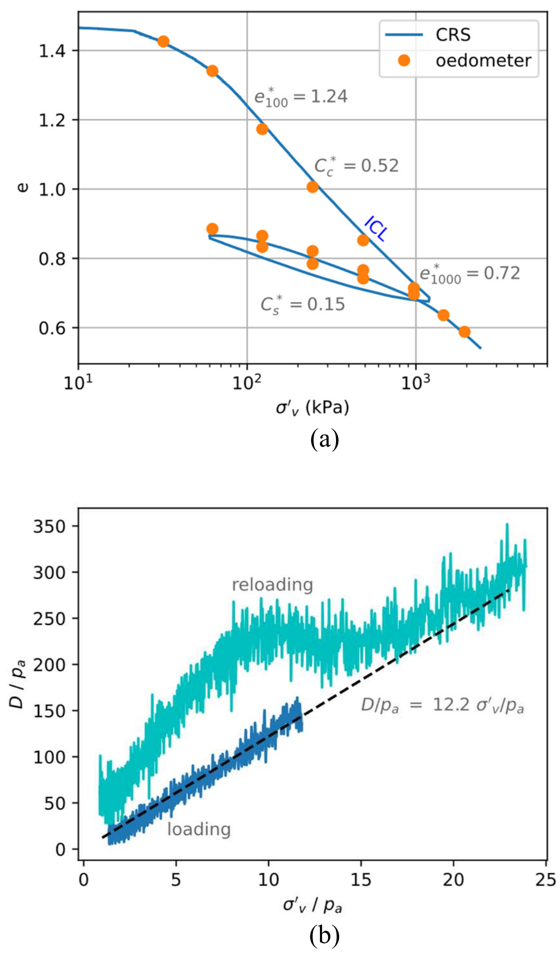


Figure 7. 1D compression of reconstituted Boom clay (a) out of CRS and oedometer (b) constrained modulus D.

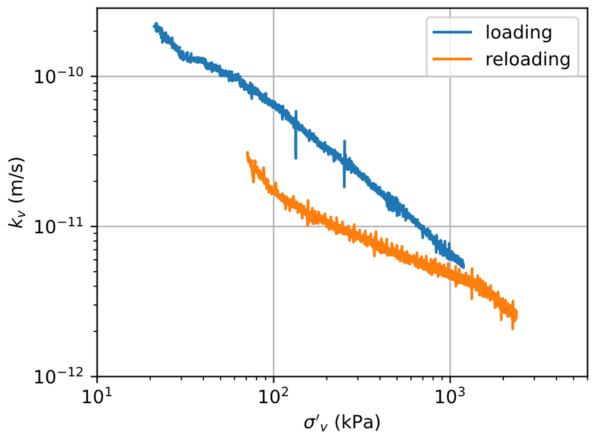


Figure 8. Evaluation of the vertical hydraulic conductivity of reconstituted Boom clay out of CRS test.

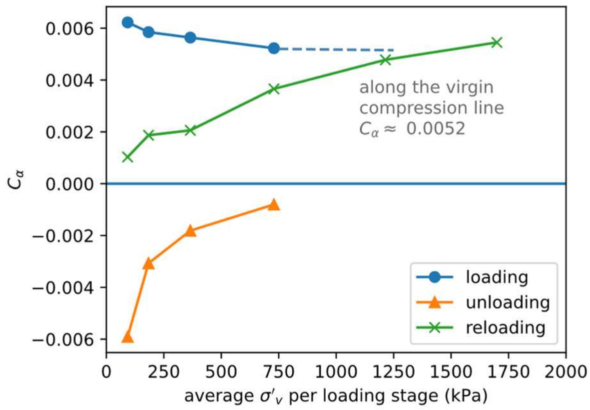


Figure 9. Secondary compression and swelling index of reconstituted Boom clay out of oedometer test.

Also, the intrinsic compression and swelling index were estimated as $C_c^* = e^*_{100} - e^*_{1000} = 0.52$ and $C_s^* = 0.15$. The evaluated C_c^* agrees well with the CSL gradient λ estimated out of undrained shear testing (Fig. 3a), as $\lambda \approx C_c^*/\ln(10)$.

The constrained modulus (symbolized here as D to avoid confusion with the stress ratio M) out of the CRS test is illustrated in Fig. 7b. As expected, a rather linear relationship between D and σ'_v along the ICL was observed, also after reloading.

Moreover, Fig. 8 illustrates the vertical hydraulic conductivity k as a function of σ'_v. The relationship resembles the compression curve and confirms the low permeability of Boom clay often reported in the literature (e.g. Yu et al., 2013 on undisturbed Boom clay) even at its reconstituted state.

The secondary compression of the reconstituted clay was also evaluated out of oedometer tests. The secondary compression index C_α here is given by (CUR, 2005):

$$C_\alpha = \frac{\Delta H / H_i}{log\left(\frac{t_i + \Delta t}{t_i}\right)} \tag{1}$$

where ΔH is the change of sample height over the linear secondary compression portion of the compression vs. (log) time curve, H_i is the sample height at the start of the current loading step and t_i and $t_i + \Delta t$ represent the time interval over which ΔH takes place.

Fig. 9 illustrates the evaluated C_α for every stage of loading, unloading and reloading vs. the mid stress of every stage. The C_α values during unloading were plotted as negative to differentiate them from loading stages.

The results show that C_α is affected by the stress state and loading path. During the first loading stage, which takes place along to the ICL, C_α shows high values, starting at $C_\alpha \approx 0.0060$ and converging towards $C_\alpha \approx 0.0052$ with increasing stress. During unloading, C_α initially shows a five-fold decrease but it quickly increases in magnitude to similar values as on the ICL, in spite of the increased overconsolidation ratio. On reloading, C_α shows again a five-fold decrease but this time it gradually increases with increasing stress (decreasing overconsolidation ratio) back to the almost constant value evaluated along the ICL. These results suggest that secondary swelling could be significant when the clay is considerably unloaded.

5. Undisturbed vs. reconstituted behavior

Many tests on undisturbed Boom clay from various sites around Antwerp were performed by the geotechnical laboratory of the Flemish Government. In this section only a few results are shown to illustrate some common behavior features and to compare them to that of reconstituted clay samples, assuming that the composition of the reconstituted clay material is representative of the clay in all the area.

The shear strength of undisturbed Boom clay samples from Antwerp (sampling depth of about 30 m) out of consolidated undrained triaxial tests on samples with a diameter of 38 mm is illustrated in Fig. 10. A compilation of peak shear strength states is illustrated in Fig. 10a. As expected, the peak states plot above the evaluated CSL, defining a peak envelope. The peak states should gradually approach the CSL at much higher stress levels as the sample structure degrades and the clay reaches a normally consolidated state. Moreover, the residual strength line plots below the CSL and its gradient is expected to decrease with increasing normal stress.

Fig. 10b illustrates some examples of normalized stress paths. Here, p′ has been normalized with respect to the critical pressure p'_c and q has been normalized with respect to Mp'_c. The critical pressure p'_c is the stress on the critical state line (Fig. 3a) corresponding to the specific volume of the clay sample before shearing. The critical state in this normalized chart plots at a single point at $p'/p'_c = 1$ and $q/Mp'_c = 1$. All reconstituted normally consolidated samples follow a similar path rising and rapidly bending towards the left because of a gradual increase of excess of pore water pressure until they reach the critical state. On the contrary, undisturbed samples show different stress paths (typical of overconsolidated clay), rising almost vertically and slightly bending towards the right before reaching a peak state. After the peak state the stress paths decline probably due to the development of shear bands and degradation of the structure.

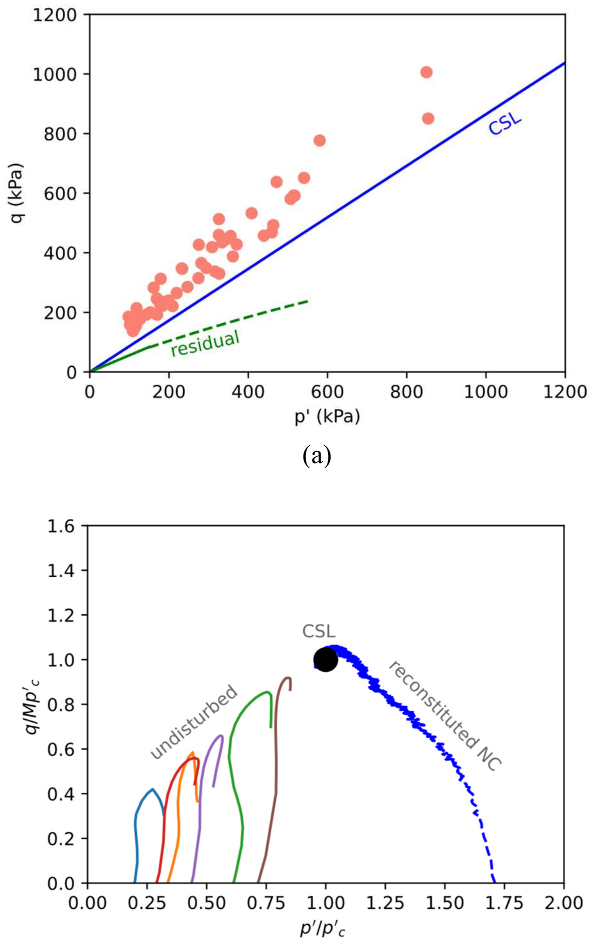


Figure 10. (a) Peak states of undisturbed Boom clay samples from Antwerp vs CSL (b) normalized stress paths.

The 1D compression behavior of 3 samples of undisturbed Boom clay from Antwerp (sampling depth of about 30 m) out of oedometer tests is illustrated in Fig. 11. Unfortunately most of the existing data is limited to to relatively low stresses. Here, the compression curves have been normalized with respect to the intrinsic properties of the clay by replacing the void ratio (e) with the void index (I_v), introduced by Burland (1990), to allow for a better comparison of undisturbed and reconstituted behavior. The void index is given by:

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C^*_c} \tag{2}$$

In such normalized compression chart, the ICL can be rendered as a unique line passing through $I_v = 0$ at $\sigma'_v = 100$ kPa and $I_v = -1$ at $\sigma'_v = 1000$ kPa, for all clays. However, as demonstrated by Burland (1990), the normal compression line of a natural (structured) clay, denoted here as sedimentation compression curve (SCC), would plot above the ICL owing to the clay structure.

Fig. 11 also illustrates the SCC of the natural Boom clay, as suggested by Chandler (2010), and the compression curve of an undisturbed Boom clay sample, obtained from a depth of about 250 m ($\sigma'_v \approx 2.5$ MPa), from Mol (Horseman et al., 1987). This sample, compressed to a maximum stress of 32 MPa, yields on the SCC at a yield stress of about 6 MPa.

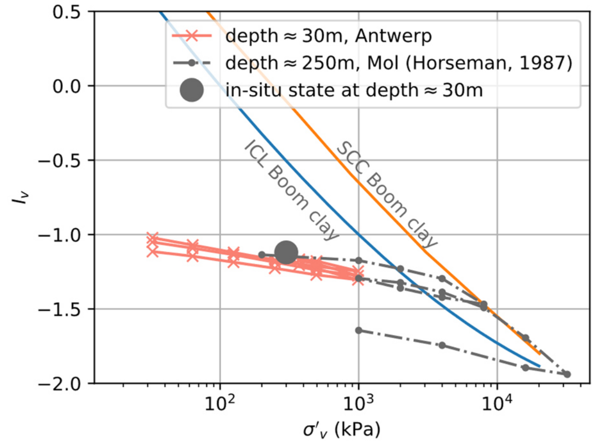


Figure 11. Normalized compression curves of undisturbed Boom clay samples.

The in-situ state of the samples from Antwerp plots far to the left of the ICL clearly indicating their geologically overconsolidated nature. The in-situ state of the deep Boom clay ($\sigma'_v \approx 2.5$ MPa) would plot just next to the SCC suggesting light overconsolidation.

The compression curves of the samples from Antwerp show similar stiffness to the deep Boom clay sample from Mol and are expected to follow a similar path and probably yield at some point beyond the ICL. The expected yield stress would then most probably be higher than the estimated geological preconsolidation pressure of about 900 kPa. As suggested by Burland (1990) and Chandler (2010), this is also a typical feature of overconsolidated stiff clay with a post-sedimentation structure.

The vertical hydraulic conductivity of the samples from Antwerp was evaluated out of the oedometer test results. As illustrated in Fig. 12 the hydraulic conductivity of the undisturbed samples from Antwerp is lower than that of the reconstituted sample, probably due to the lower void ratio, structure and higher yield stress. But both, undisturbed and reconstituted (reloading path), seem to decrease with increasing stress at a comparable rate. The evaluated hydraulic conductivity is about the same order of magnitude as the values reported by Yu et al. (2013) for the Putte and Terhagen members of the Boom formation.

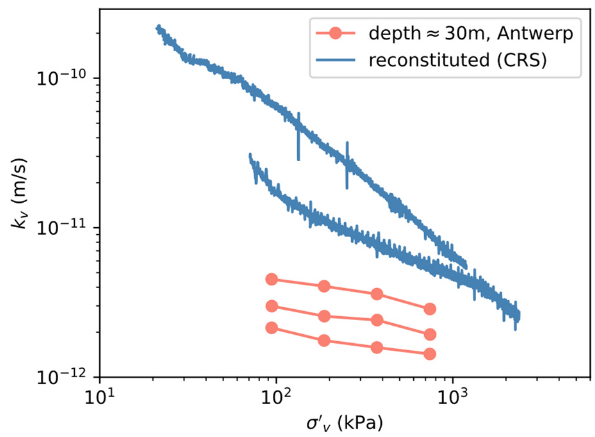


Figure 12. Vertical hydraulic conductivity of undisturbed Boom clay out of oedometer tests.

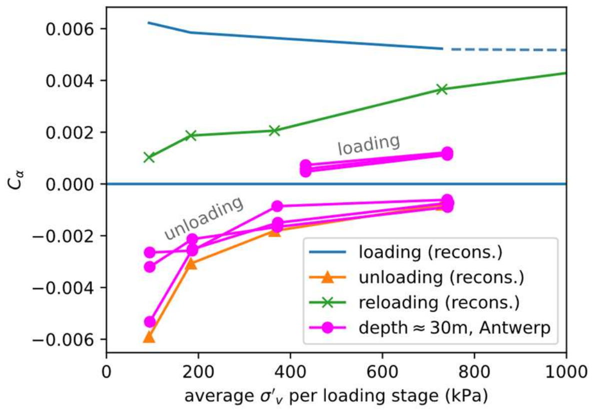


Figure 13. Secondary compression and swelling index of undisturbed Boom clay out of oedometer tests.

Finally, the secondary compression and swelling of the undisturbed samples from Antwerp was also evaluated out of the oedometer test results (Fig. 13). An initial swelling pressure of about 300 to 400 kPa was observed on all samples, therefore, the loading path started from that stress level. Along the loading path, C_α of undisturbed samples is significantly lower than C_α of the reconstituted sample along the reloading path; however, both gradually increase with increasing stress at different rates. The secondary swelling (negative C_α) along the unloading path initially shows small C_α values as well but, despite some scatter, C_α is observed to rapidly increase in magnitude as the unloading becomes more severe reaching values close to those of the reconstituted clay perhaps as a result of destructuration. These results suggest that the effect of secondary swelling is relatively small and controlled for stresses higher than the swelling pressure, but when unloading results in a stress lower than the samples' swell pressure, secondary swelling could become quite significant.

6. Conclusions

Boom clay is a stiff marine clay from the Oligocene epoch of the Paleogene period. In the present research the mechanical behavior of reconstituted normally consolidated Boom clay was examined in a series of oedometer, CRS, CU triaxial and multi-reversal direct shear tests. The following parameters were obtained:

- critical state shear angle: $\varphi'_{cs} = 22°$.
- CSL parameters: M = 0.865, $\lambda = 0.228$, $\Gamma = 3.08$.
- residual shear angle: $\varphi'_r = 15°$ under $\sigma'_v = 50$ kPa.
- intrinsic compression parameters: $e^*_{100}=1.24$, $e^*_{1000}=0.72$, $C_c^* = 0.52$ and $C_s^* = 0.15$.
- secondary compression index C_α = 0.0060 to 0.0058 along the ICL.

The mechanical behavior of the reconstituted clay was compared to the behavior of undisturbed clay samples tested over the years for the design and construction of infrastructure around Antwerp. The undisturbed clay samples show stiffer and brittle behavior which can be attributed to the clay structure and its overconsolidated nature. Shearing tests on overconsolidated reconstituted clay were out of the scope of this research but would be useful to evaluate in more detail the effect of structure of the natural clay.

References

ASTM D4186M-12. 2012. "Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading". ASTM International.

Bouazza, A., Van Impe, W.F., Haegeman, W. 1996. "Some mechanical properties of reconstituted Boom clay" Geotechnical & Geological Engineering 14, pp. 341-352. https://doi.org/10.1007/BF00421948

Burland, J.B. 1990. "On the compressibility and shear strength of natural clays" Geotechnique 40(3), pp. 329-378. https://doi.org/10.1680/geot.1990.40.3.329

Chandler, R.J. 2010. "Stiff sedimentary clays: geological origins and engineering properties" Geotechnique, 60(12), pp. 891-902. https://doi.org/10.1680/geot.07.KP.001

CUR recommendation 101. 2005. Procedure and interpretation of the oedometer test. CROW, Aeneas Media (in Dutch).

De Beer, E.E. 1967. "Shear strength characteristics of the Boom Clay", In: Proceedings of the Geotechnical Conference Oslo, 1967, Vol. 1, p. 83-88.

EN ISO 17892-5. 2017. "Geotechnical investigation and testing – Laboratory testing of soil – Part 5: Incremental loading oedometer test". European Committee for Standardization.

EN ISO 17892-9. 2018. "Geotechnical investigation and testing – Laboratory testing of soil – Part 9: Consolidated triaxial compression tests on water saturated soils". European Committee for Standardization.

EN ISO 17892-10. 2018. "Geotechnical investigation and testing – Laboratory testing of soil – Part 10: Direct shear tests". European Committee for Standardization.

Horseman, S.T., Winter, M.G., Entwistle, D.C. 1987. "Geotechnical characterization of Boom clay in relation to the disposal of radioactive waste", Commission of the European Communities, Nuclear Science and Technology, Report EUR 10987.

Matthijs, J., Buffel, Ph., Leroi, S. 2003. "Geological cross-sections through the tertiary in Flanders, scale 1:100000". Technical Note. Geological Service Company bvba (GCS), commissioned by the Ministry of the Flemish Community - Department of Natural Resources and Energy (in Dutch).

Merchan, V, Romero, E., Vaunat, J. 2011. "An Adapted Ring Shear Apparatus for Testing Partly Saturated Soils in the High Suction Range" Geotechnical Testing Journal 34(5), pp. 1-12, https://doi.org/10.1520/GTJ103638.

Mitchell, J.K., Soga, K. 2005. "Fundamentals of Soil Behavior", 3rd ed., John Wiley & Sons, Inc., Hoboken, New Jersey.

Schittekat, J. 2001. "Engineering geology of the Boom clay in the Antwerp area", In: Proc. of Screw Piles – Installation and Design in Stiff Clay, pp. 11-17. Swets & Zeitlinger, Lisse.

Stark, T.D., Hussain, M. 2013. "Empirical correlations: Drained shear strength for slope stability analyses" Journal of Geotechnical and Geoenvironmental Engineering. 139(6), https://doi.org/10.1061/(ASCE)GT.1943-5606.0000824

Vandenberghe, N, De Craen, M, Wouters, L. 2014. "The Boom Clay Geology. From sedimentation to present-day occurrence: a review", Memoirs of the Geological Survey of Belgium, 60, Royal Belgian Institute of Natural Sciences, Geological Survey of Belgium.

Yu, L., Rogiers, B., Gedeon, M., Marivoet, J., De Craen, M., Mallants, D. 2013. "A critical review of laboratory and in-situ hydraulic conductivity measurements for the Boom clay in Belgium" Applied clay Science 75, pp. 1-12. https://doi.org/10.1016/j.clay.2013.02.018