Cyclic Response of Natural Onsøy Clay
Part I: Experimental Analysis

Nina Müthing(✉), Thomas Barciaga, and Tom Schanz

Department of Civil and Environmental Engineering,
Ruhr-Universität Bochum, Universitätsstr. 150, 44780 Bochum, Germany
nina.muething@rub.de

Abstract. In geotechnical applications cyclic loading occurs frequently caused by earthquake shaking, traffic or wind and wave loading. The consideration of cyclic loading effects finds increasing attention nowadays. This particularly holds true for structures, which are of civil importance and involve high investment costs. Sophisticated calculation approaches are applied within the design process of these boundary value problems. However, many of the calculation models assume undrained stress paths, where cyclic loading leads to a continuous generation of excess pore water pressure. When soft, marine clays under slower loading are involved, the dissipation of excess pore water pressure becomes relevant. The transient consolidation process needs to be considered. Thus, in the present paper the consolidation behaviour of Norwegian Onsøy clay as a typical representative of natural, marine clay under cyclic loading is analysed. Part I of this paper presents the experimental study. Testing results from monotonic and cyclic oedometer tests on natural as well as remoulded clay are introduced. The differences in the compression behaviour and pore water pressure dissipation of structured and remoulded clay are illuminated. Furthermore, the effect of cyclic loading characteristics, as e.g. the load amplitude, on consolidation is analysed. Part II of the paper comprises a numerical study. Modelling the cyclic consolidation processes by use of FEM, the focus of the analysis is set on the necessity of different features of a hierarchical model to analyse this type of boundary value problems.

1 Motivation: Natural Clay Behaviour Under Oedometric, Cyclic Loading Conditions

In many geotechnical applications cyclic loading of structures plays an important role and needs to be considered within the design process. Cyclic loading may for example be caused by traffic loads or earthquake shaking. Of cause, it is also of particular importance in the framework of foundation design for offshore installations, as e.g. pipeline or pile foundations, wind turbines and gravity platforms [24]. When dealing with soft soils under slower loading, the dissipation of excess pore water pressure needs to be considered and the transient consolidation of the soil comes into picture.

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Consolidation processes under cyclic loading have been dealt with in a multitude of studies in literature. Analytical solutions considering consolidation under cyclic loading conditions are e.g. available in [18,20,21]. Admittedly, most of these solutions are based on Terzaghi's classical theory assuming geometrical and constitutive linearity. Due to this linearisation only idealized boundary value problems can be analysed. In [16,17,19] an experimental study dealing with cyclic consolidation processes of soft Kaolin clay is introduced. Within this study the excess pore water pressure build-up and dissipation occurring during consolidation is analysed.

Based on this study, the present paper deals with the consolidation of natural, structured clay under cyclic loading. As representative material in the present paper natural, marine Onsøy clay from Norway is considered. Hereby, the focus is set on the behaviour of structured clay loaded to a stress level below and above its yield stress as well as on the difference in consolidation of the natural and identical, but remoulded clay.

Structure of soil includes fabric and bonding. While fabric is the arrangement of soil particles, bonding characterises the connections of the soil skeleton, which are not of frictional kind. When structured clay is subjected to an increasing stress, initially the bonding will procure a relatively stiff behaviour of the soil matrix. Only when the yield stress, the maximum bearable stress, is exceeded the bonding is destroyed and the soil experiences a so-called destructuration. This destructuration is reflected in the stress-strain behaviour by a sudden drop of soil stiffness thus a significant compression caused by a rearrangement of the soil particles [2–4]. Beyond that, the destructuration of natural clay may also be caused by a mechanical destruction of the bonding induced by simple remoulding of the natural clay. From the example of *quick clay* the effect is well known, that sensitive, marine clays subjected to remoulding show a dramatic change in their material behaviour. This effect can be observed in the oedometric compression behaviour of structured clays as due to the mechanical distortion of the structure the remoulded clay shows the same compression behaviour in terms of stiffness in low and high stress ranges [22].

The present paper is subdivided into two parts: Part I presents the experimental study and testing results, while Part II deals with the numerical modelling of the boundary-value problem.

The experimental study, presented in this Part I of the paper, is opened by a characterisation of the tested soil, Onsøy clay. To analyse the compression behaviour of natural and remoulded clay under monotonic loading, stepwise oedometer tests are conducted. To meet the special requirements of the testing, a particular oedometer cell designed and constructed at Ruhr-Universität Bochum is used. This oedometer cell introduced in [16] allows the testing of slurries due to a cell sealing system and enables the measurement of pore water pressure, water in- and outflow, radial stress as well as frictional loss. Regarding the analysis of the settlements differences between natural and remoulded clay are illuminated. The analysis of the compression behaviour of the natural samples additionally allows for determination of the material's yield stress. The knowledge of yield stress is required for the following cyclic consolidation analysis, as within this study natural and remoulded samples are subjected to cyclic loading with various loading amplitudes below and above yield stress. Thereby, the influence of load amplitude on the cyclic consolidation of structured clay is analysed.

Part II of the paper deals with the numerical modelling. As an important role falls upon the constitutive behaviour of cyclically loaded soils, the necessity of different features of a hierarchical model to analyse this type of boundary value problems is studied.

2 Material Characteristics of Onsøy Clay

The present study was performed using marine, natural Onsøy clay from a site close to Fredrikstad, which is located approximately 100 km south-east of Oslo, Norway. The deposit was formed during glaciation and early post glaciation (Holocene). During the isostatic uplift, caused by the following de-glaciation, the depositional environment changed from marine to estuarine [14]. The material properties of this clay have been characterized in detail by [13,14] as well as other studies at *Norwegian Institute of Technology (NGI)*. The main characteristics are described in the following. A more detailed characterisation can be found in [14,15].

The block sample used for the present study was taken in approx. 10 m depth. For the sampling a Sherbrooke sampler was used in order to prevent disturbance of the sensitive clay material. Details on this sampling procedure can be found in [5,12].

The natural water content of the tested clay was determined to be approximately $w_{nat} = 65\%$ with a mean void ratio of approximately $e_{nat} = 1.77$, as the specific gravity ρ_s was determined according to [1] by pycnometer method to be $\rho_s = 2.77\text{g/cm}^3$. These values coincide with the range for the natural water content given by [14] for samples from similar depths. The pH value was determined by pH probe to be $pH = 7.3$. With reference to tests in [24] on the same material from a different sampling depth, the salt concentration was assumed to be 32.5 g/l, which is in accordance with [14] suggesting an average value of 30 g/l for material from depths larger than 7 m. Table 1 summarises the most important parameters describing the natural state of the sample block used within the present study.

Table 1 additionally gives the plasticity index and Atterberg limits for the tested clay, determined according to [6–8]. With reference to the plasticity diagram after Arthur Casagrande Onsøy clay tested in the present study lies slightly above the A-line and therefore can be characterised as a pronounced plastic clay. The activity I_A can be calculated to $I_A = 1.9$[-].

The grain size distribution was analysed using two different techniques: sedimentation technique according to [9] and laser diffraction method. Figure 1 shows the determined grain size distribution curves. Hereby, Onsøy clay can be characterised as a dark grey clayey silt, with a clay content of 40%. The organic content can be approximated to 0.6% [14].

Table 1. Material characteristics of tested Onsøy clay material

	Onsøy clay
Natural water content, w_{nat}[%]	65
Natural degree of saturation, S_r[%]	96–100
Natural void ratio, e_{nat}[-]	1.77
pH value, pH[-]	7.31
Salt concentration in pore water, Ψ_{Salt}[g/l]	25–29
Liquid limit, w_L[%]	67.4
Plastic limit, w_P[%]	29.6
Plasticity index, I_P[%]	37.8
Shrinkage limit, w_S[%]	28.8
Activity, I_A[-]	1.9

Further material characteristics, e.g. mineralogy, chemical composition, CEC and specific surface area, can be found in [15].

3 Experimental Methods

The experimental study presented in the following covers oedometer tests on natural and remoulded Onsøy clay under monotonic and cyclic loading. All tests were carried out in an oedometer cell designed and constructed at Ruhr-Universität Bochum. The device was introduced in [17,19]. Details on the functionality, construction and calibration of the device in addition can be found in [15]. In the following section the main features of the oedometer device are illustrated together with the applied loading and hydraulic boundary conditions. As sensitive natural and remoulded soil samples are tested, particular consideration is required regarding the sample preparation and installation technique. The description of these closes this chapter.

Oedometer Device

For the experimental study of the consolidation behaviour of clays under cyclic loading an oedometer device was designed and constructed at Ruhr-Universität Bochum, introduced in [17,19].

Figure 2 shows the device, which enables testing of samples with a diameter of 70 mm. For the recent study samples with a height of 20 mm were used. Thereby, the diameter-to-height-ratio of 3.5 follows [10] and limits the friction between sample and oedometer ring to a tolerable extent.

The sample is placed between two filter plates within a stainless-steel oedometer ring, which is fixed in vertical position and sealed against the top and bottom plate of the device by rubber rings. Thus, it allows testing of slurry and paste-like material as required in the present study. The thin-walled oedometer ring is equipped with strain gauges allowing the measurement of radial stresses during

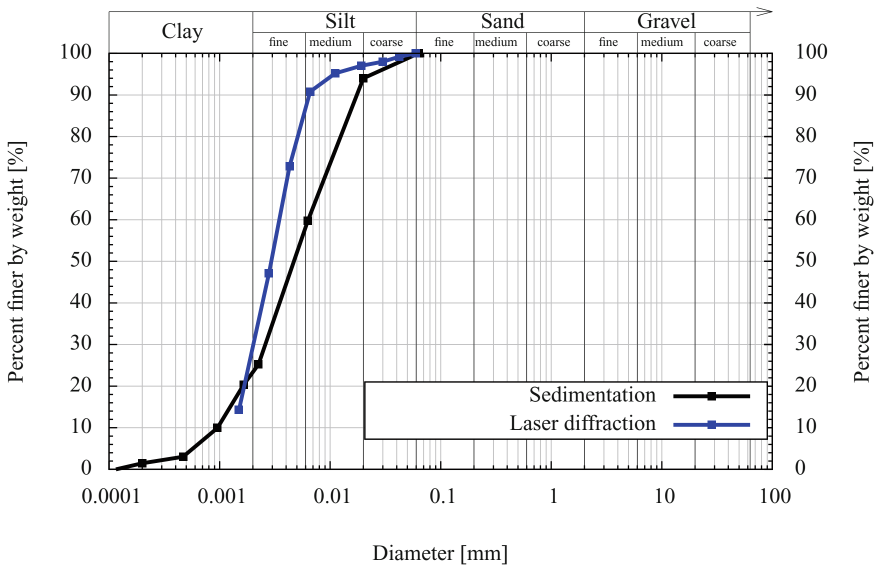

Fig. 1. Grain size distribution of Onsøy clay

the consolidation process. Drainage is allowed through the filter plates embracing the sample. Therefore, in general different drainage conditions are possible. In the present study drainage is executed through the sample top. A pore pressure transducer included in the bottom part of the device is used to measure transient pore water pressure during testing. It is able to measure positive and negative pore water pressures up to $1000/-100\,\text{kPa}$.

A load cell at the top and bottom of the device allows the measurement of vertical force above and below the sample and thereby facilitates the evaluation of frictional loss.

Applied Loading

The present study comprises oedometer tests under monotonic as well as cyclic loading.

In the monotonic loading tests the load is applied stepwise with a magnitude of 10–20–50–100–200–400 kPa during loading phase and 200–100–50–20–10 kPa during unloading phase. The load steps are applied quasi-instantaneously. The vertical stress application is limited to a maximum of 400 kPa due to the measurement range of the strain gauges applied on the oedometer ring.

For the cyclic tests a loading function of sinusoidal type was chosen according to [16,21] as a typical loading pattern in geotechnical applications. The loading function in given in Eq. 1.

$$L(t) = q\ \sin^2 \frac{\pi t}{d} \tag{1}$$

where $L(t) =$ applied loading as a function of time t, $q =$ load amplitude and $d =$ load period.

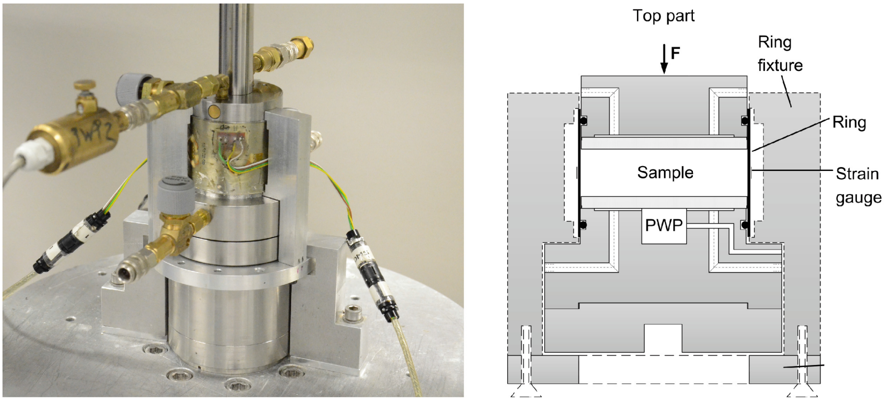

Fig. 2. **A** Photograph and **B** Sketch of the new oedometer device [17]

Figure 3 shows the haversine loading function over time as applied in the experimental testing.

For cyclic consolidation studies the chosen load period and amplitude are of significant relevance. In the present study the load amplitude was varied with reference to the yield stress of the tested natural material. Details are given in Sect. 5. In agreement with [16] the load period d was chosen to be $d = 120\,\mathrm{s}$. It can be calculated with reference to the time $t^{ref}(T_0 = 1)$. $t^{ref}(T_0 = 1)$ is the time an equivalent sample (identical initial state) under static loading needs to consolidate.

$$d = \frac{T_0 \cdot H^2}{c_v} = \frac{0.0075 \cdot (0.02\,\mathrm{m})^2}{2.5 \cdot 10^{-8}\mathrm{m}^2/\mathrm{s}} = 120\,\mathrm{s} \tag{2}$$

where T_0 is the chosen dimensionless period, c_V is the material-dependent coefficient of consolidation assumed to be a constant, stress-independent value determined from monotonic consolidation tests on equivalent sample material. For details see [17].

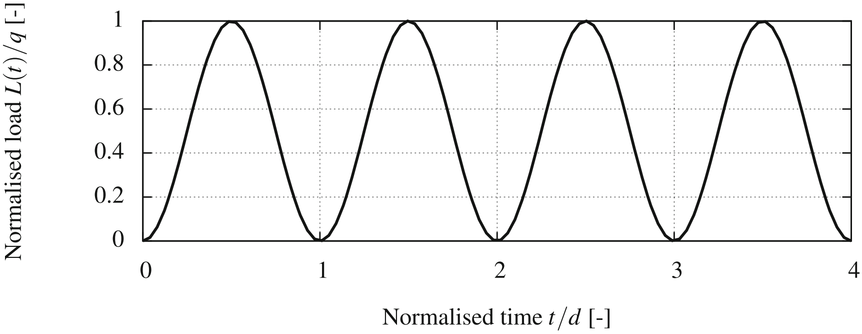

Fig. 3. Haver-sine loading function

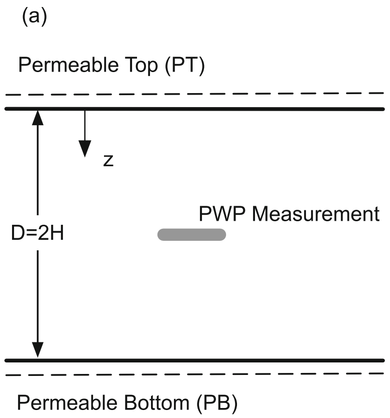

Fig. 4. Equivalence of **(a)** PTPB and **(b)** PTIB oedometer drainage configurations [16]

Hydraulic Boundary Condition

In a standard oedometer configuration usually both top and bottom of the sample are drained (abbreviated PTPB), the maximum pore water pressure occurs in the mid-plane of the sample. In this location, measurement is technically difficult and would cause strong disturbance of the sample and flow paths. Thus, in the present experimental study tests were conducted with permeable top and impermeable bottom (abbreviated PTIB). This configuration is equivalent to a standard PTPB oedometer configuration as illustrated in Fig. 4 [16].

Sample Preparation and Installation

Sample material preparation and installation of the sample in the oedometer ring are distinctly different for remoulded and natural clay. Important details of these procedures are illustrated in the following.

The sample preparation requires particular elaborateness and precision as mainly the natural clay is very sensitive and testing results are susceptible to variations in initial and boundary conditions, e.g. in the initial material state and saturation of the sample. Therefore, throughout the whole installation process, special caution was taken to guarantee homogeneity of the sample, to avoid inclusion of air bubbles and to ensure full saturation of the testing system.

For the tests on natural samples, larger, so-called pre-samples were cut from the block avoiding inclusion of shell fragments, sand inclusions and disturbed boundary areas. The sample itself was then installed by pushing the thin-walled oedometer ring into the pre-trimmed material. This preparation and installation technique was selected to guarantee the least disturbance of the sensitive natural clay as well as close contact between sample and oedometer ring for measurement of horizontal stresses.

The remoulded clay was prepared by mixing soil material at the natural water content of $w_{L,nat} = 65\%$, without prior air or oven drying. The mechanical distortion during mixing is presumed to a cause a complete destruction of the existing structure in the natural state. Afterwards, the paste-like, remoulded clay was placed in the oedometer ring using a spatula taking special caution to avoid the inclusion of air bubbles during the installation of sample material into the ring.

Pre-moistened filter paper is placed below and above the soil sample to prevent finest particles to be flushed out of the sample into the filter plate clogging its pores.

4 Consolidation Under Monotonic Loading

To analyse the consolidation of Onsøy clay under monotonic loading, stepwise oedometer tests on natural and remoulded samples were carried out. The compression behaviour is analysed regarding the destructuration of the natural clay. A comparison to testing data from literature is drawn in order to evaluate the present test results. Besides, material parameters as for instance compression and swelling index as well as the yield stress are determined from the compression curves. A closer look is taken at the time-dependent settlement evolution and pore water dissipation of natural and remoulded clay. From this comparison conclusions regarding the material behaviour of natural samples before and after exceeding the yield stress can be drawn.

Compression Behaviour

Figure 5 illustrates the compression behaviour of natural and remoulded Onsøy clay from the conducted oedometer tests. The natural clay initially acts very stiff and shows a pronounced destructuration after exceeding the yield stress. Under the applied maximum load of 400 kPa the two materials still show a difference in axial strain or void ratio respectively. However, from tests in a high stress oedometer cell it is known that the compression curves for natural and remoulded clay converge when subjected to a stress of about 1000 kPa [15].

The yield stress σ_y marks the stress at transition point where the compression of natural soil changes from elastic to elasto-plastic. From the test data it is determined to be $\sigma_y \approx 50{-}60\,kPa$. However, additional tests in a high stress oedometer cell suggest a slightly higher yield stress of approx. 80 kPa [15], which is in better accordance with data from literature [14].

The elasto-plastic compression index C_c and elastic swelling index C_s can be calculated from the testing data with reference to Eqs. 3 and 4 to be $C_c = 0.425$ and $C_s = 0.055$.

$$C_\mathrm{c} = -\frac{\Delta e}{\Delta \log\left(\sigma_v'\right)} \tag{3}$$

$$C_\mathrm{s} = -\frac{\Delta e^\mathrm{e}}{\Delta \log\left(\sigma_v'\right)} \tag{4}$$

The compression ratio CR according to [11] is defined as

$$CR = \frac{C_c}{(e_0 + 1)} \tag{5}$$

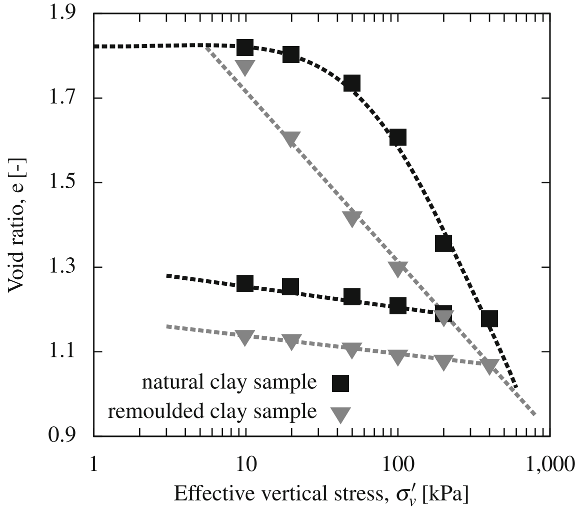

Fig. 5. Compression behaviour of natural and remoulded Onsøy clay

and can be calculated to be $CR = 0.15$. It is in good agreement with data from literature [11].

Figure 6 compares the present test data with test data given in [14] for samples from 6.10 m and 6.23 m depth. In general, the present results are in good agreement with data from literature. The slightly higher yield stress corresponds to the higher sampling depth of the clay tested within the present study.

Settlements

Figure 7 (top) shows the normalised time-dependent settlements of a natural clay sample under oedometric compression. It can be recognised that in the first three load steps from 1–10 kPa, from 10–20 kPa and from 20–50 kPa settlements occur fast compared to the later load steps. In Terzaghi's theory [23] the material's ability to consolidate and the consolidation rate are characterised by the consolidation coefficient c_v. The consolidation coefficient can be described by the following equation

$$c_v = \frac{k \cdot E_s}{\gamma_w} \tag{6}$$

where k is the hydraulic permeability, E_s is the stiffness and γ_w is the unit weight of water. Using c_v the time where 99% of the consolidation settlement has been reached from Terzaghi's theory can be calculated to be

$$t_{99\%} = \frac{2 \cdot H^2}{c_v} \tag{7}$$

where H is the drainage length.

In the first three load steps the natural soil is structured. The high permeability due to high void ratio together with a high stiffness due to intact bonding

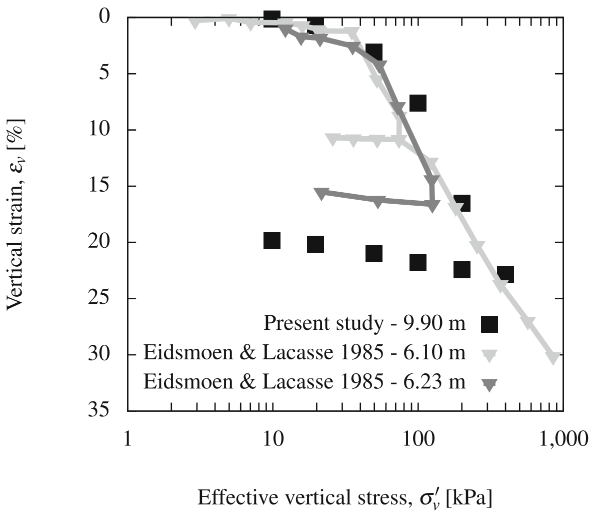

Fig. 6. Compression behaviour of natural and remoulded Onsøy clay - comparison with literature

result in a high c_v and in a fast consolidation respectively. After exceeding the yield stress, the stiffness suddenly drops due to the destructuration of the soil, while the void ratio and permeability experience a slower change. This significantly retards the consolidation process or in terms of c_v causes a decrease in c_v. In the higher load steps the soil matrix is compressed. The decrease in void ratio causes a decrease in permeability. However, the increase in stiffness is more dominant, so that for higher loading the consolidation occurs faster. This can also be observed in Fig. 7 (bottom) illustrating the normalized time-dependent settlements of the remoulded clay sample under oedometric compression. Here, for all load steps it holds true that with ongoing compression the consolidation is accelerated.

Figure 8 compares the normalized time-dependent settlements of natural and remoulded clay experiencing oedometric decompression. It can be observed that due to the experienced destructuration and elastic decompression stress path both clays show an equivalent decompression. Caused by the higher stiffness the swelling deformation occurs faster in the higher than in the lower stress ranges. As the change in stress-dependent stiffness is congruent for remoulded and natural sample (reflected in almost identical inclination of the decompression branch C_S), the swelling rate in the particular load steps likewise is the same.

A detailed study on the evolution of c_v during soil compression and its dependency on changing permeability and stiffness can be found in [15].

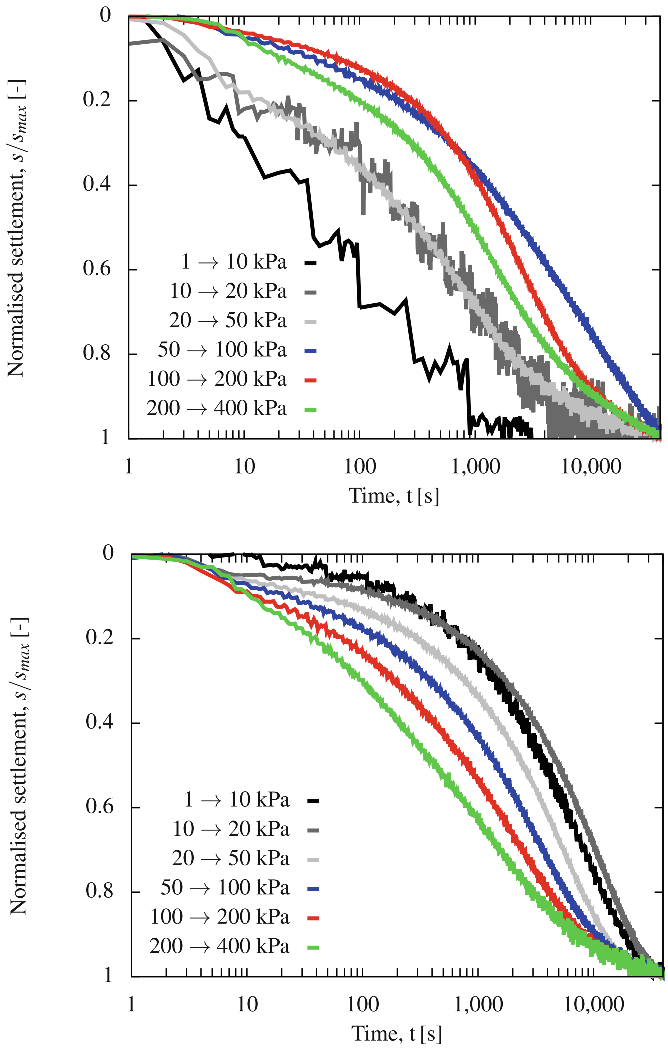

Fig. 7. Time-dependent settlements of natural (top) and remoulded (bottom) Onsøy clay

Pore Water Dissipation

The pore water dissipation during the monotonic load steps reflects the typical behaviour in terms of stiffness and permeability changes found from the time-dependent settlements. A detailed study of the measured pore water pressures during monotonic load application and consolidation phase is beyond the scope of this paper, but can be found in [15].

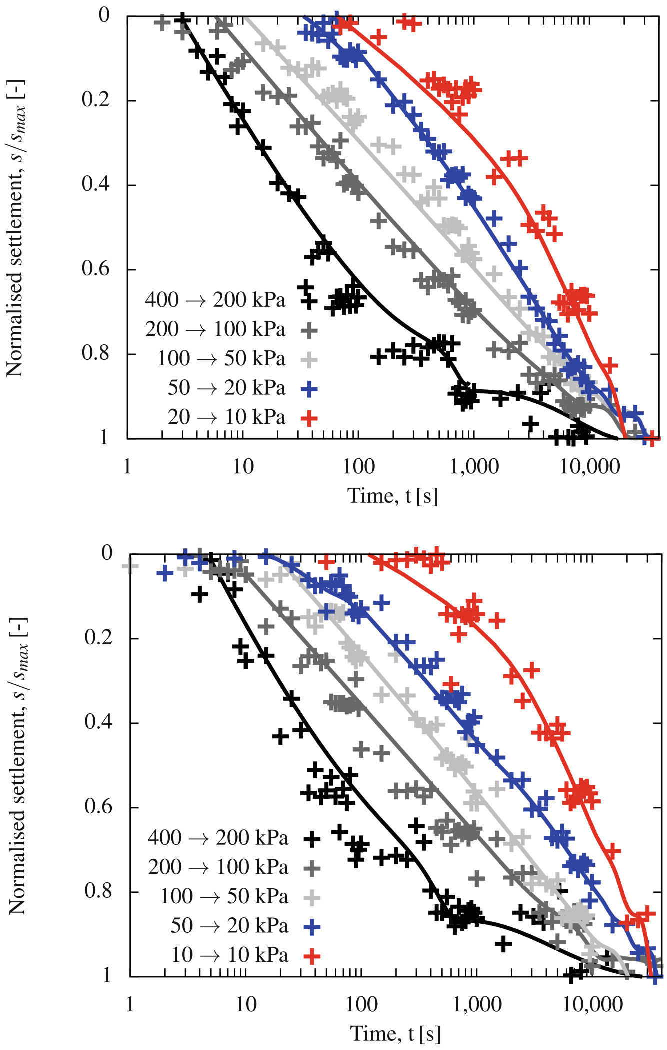

Fig. 8. Time-dependent decompression of natural (top) and remoulded (bottom) Onsøy clay

5 Cyclic Consolidation

For the present study on the consolidation of marine Onsøy clay under cyclic loading six oedometer tests are evaluated to analyse the influence of the applied load amplitude on the consolidation. Thereby, the load amplitude is chosen with reference to the yield stress. While one load amplitude was set to 50 kPa and hence chosen to be smaller than the yield stress, the other two amplitudes were set to be 100 kPa and 200 kPa exceeding the yield stress. The influence of the

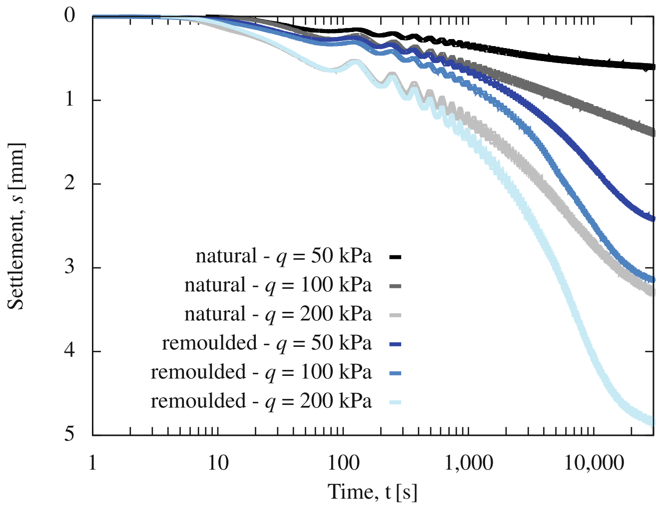

Fig. 9. Time-dependent settlements of natural and remoulded Onsøy clay under cyclic loading

load amplitude on the destructuration and thus on the consolidation process is analysed in terms of time-dependent settlements and pore water dissipation. Other loading characteristics, as e.g. the loading frequency, have an additional influence on the consolidation. A study on these influences can be found in [15], but is not considered in the present paper.

Compressibility and Time-Dependent Settlements

Figure 9 compares the time-dependent settlements of natural and remoulded samples under haversine loading with different amplitudes.

During the cyclic consolidation process the settlements accumulate and after a finite number of load steps reach a quasi stationary state. The number of cycles required for accomplishment of steady state is material and load dependent and accounts for approx. 200–250 cycles in the present testing. The increment of settlement accumulation is significantly decreasing from larger values in the first cycles to smaller ones in quasi stationary state.

Comparing the final settlements after 250 cycles or 500 min to the results from the monotonic oedometer tests (see Fig. 10), it can be observed that the compression under cyclic loading corresponds with the compression under monotonic loading. The generally slightly smaller settlement in cyclic tests can be explained by remaining pore water pressures and a corresponding smaller effective compression stress. Details can be found in [15].

However, it can be observed that compression of all tests on remoulded clay exceed the settlements of the natural samples significantly under loading with same load amplitude. This corresponds to the compression behaviour in monotonic oedometer tests.

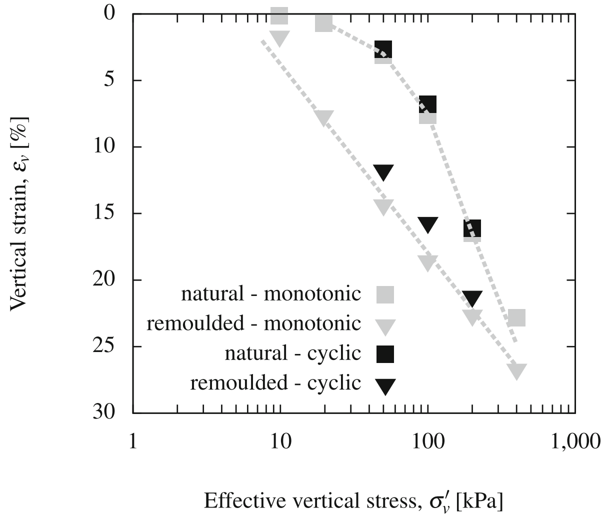

Fig. 10. Compression behaviour of natural and remoulded Onsøy clay under monotonic and cyclic loading

Figure 11 illustrates the normalized time-dependent settlements under cyclic loading. It becomes obvious, that the remoulded samples all show a similar time-dependent behaviour, while the time-dependent compression behaviour of the natural clay depends on the applied load amplitude. The natural sample loaded

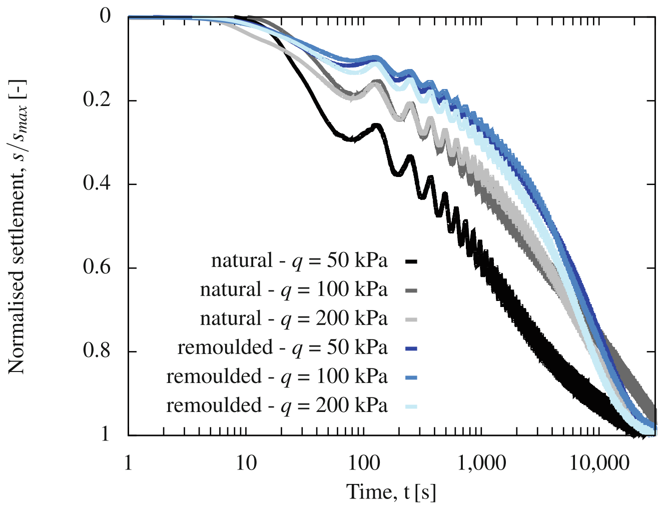

Fig. 11. Normalised time-dependent settlements of natural and remoulded Onsøy clay under cyclic loading

with a load amplitude below yield stress shows a much faster consolidation than the natural samples loaded with a load amplitude above yield stress. This also corresponds to the findings shown for consolidation under monotonic loading. However, it was to be expected from the monotonic testing results that the test under 100 kPa consolidates slower than the test under 200 kPa. This is assumed to be the case as the shape of the settlement curve for 100 kPa suggests that full consolidation is not yet reached.

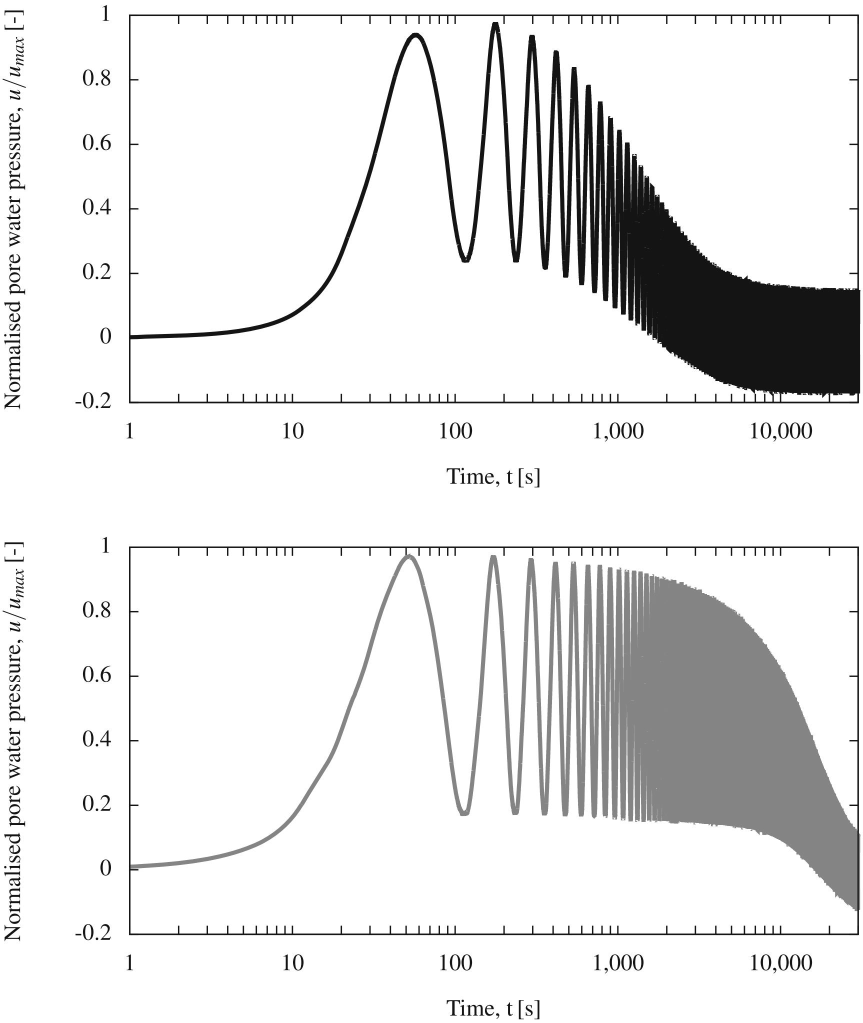

Fig. 12. Pore Water Dissipation of natural (top) and remoulded (bottom) Onsøy clay under cyclic loading of 50 kPa amplitude

Pore Water Dissipation

The difference in consolidation rate for natural and remoulded clay tested with a load amplitude below yield stress is clearly reflected in the pore water dissipation. Figure 12 compares the normalised time-dependent dissipation of pore water pressures for the two tests with a load amplitude of 50 kPa.

Generally, it can be seen that within the first cycles the excess pore water is build up. With ongoing consolidation time the excess pore water pressure dissipates, reaching a quasi stationary state after a finite number of cycles. This number of cycles is equal to the number of cycles necessary to reach a steady settlement state. It is important to notice that the amplitude of excess pore water pressures u is strongly damped during the consolidation process. Moreover, the pore water pressures in the quasi stationary state reach slight negative values at the beginning and end of a loading cycle cycling around 0.

A significantly faster decay of mean pore water pressure is observed for natural than for remoulded clay. Analogue to the monotonic loading the bonding in the soil structure of the natural clay causes a higher stiffness of the material and thus a must faster consolidation. The effect is even more pronounced in the cyclic testing as here due to the alternating loading the effective stress increases slower and thus seems to have a less distinct effect on the destructuration.

6 Conclusion and Outlook

In the present paper the consolidation of Norwegian Onsøy clay as a typical representative of natural, marine clay is analysed.

Part I of this paper comprises the experimental study. To analyse the material behaviour of natural and remoulded clay, Onsøy clay was characterised regarding its soil mechanical properties. In a specially designed oedometer cell tests under monotonic as well as cyclic loading were conducted. Particular considerations and elaborateness are required for sample preparation, installation as well as for the choice and measurement of the hydraulic and mechanical boundary conditions.

The consolidation in terms of compression, time-dependent settlements and pore water dissipation was analysed. From the experimental data it was shown, that Onsøy clay under monotonic loading behaves as a typical structured clay, showing destructuration after reaching its yield stress of approximately 60 kPa. From the cyclic oedometer tests it becomes clear that the monotonic compression is reflected in the cyclic behaviour as well. The influence of structure and loading conditions on the behaviour of natural and remoulded clay was scrutinized by comparing both material types under cyclic loading with load amplitudes below and above yield stress.

The following Part II of this paper comprises a numerical study modelling the cyclic oedometer tests on Onsøy clay. In geotechnical design processes the need for sophisticated constitutive models allowing the simulation of complex boundary value problems is growing. Therefore, the necessity and ability of different features of a hierarchical model to analyse this type of cyclic consolidation boundary value problems are studied.

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