Strength and deformation characterization of Norwegian organic cohesive soil (gyttja)

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Abstract. In several road projects in western Norway, it is common to encounter a soil consisting of mineral soils with an organic part and high water content. The organic soil is referred to as gyttja and the organic part originates from remains of plants and animals rich in fats and proteins. It is common to find up to 14 m thick soil layers containing gyttja. The amount of organic content is much lower than peats however this is observed to significantly alter the engineering property of mineral soils involving clay and silt. However, there exists no systematic study on characterization of strength and deformation properties of gyttja in Norway. As a result, it is a common practice to excavate and replace such soils. However, this shall not be a viable engineering solution as removal of organic soils gives a significant CO\textsubscript{2} emission. This implies that it is important to find ways to deal with such types of soils apart from always replacing them. To achieve this, it is essential to establish engineering properties for characterization of strength and deformation behaviour of gyttja. This research work thus focuses on looking at these aspects based on extensive field and laboratory tests. Effect of cement as a stabilization method to improve strength is also experimentally looked at. Results are also compared with available data from literature documenting other countries experiences with organic soils. This work presents and discusses main trends, correlations and characteristics related to strength and deformation behaviours of Norwegian gyttja.

1. Introduction

During the planning and development of transport infrastructure projects in western Norway, soils with a high organic- and water content are commonly encountered. The organic part is mixed with clay and silt and this cohesive organic soil is referred to as gyttja. The organic constituent originates from decomposed plant and animal remains and a detailed definition is given in next section. In western Norway, it is common to find up to 10-14 m thick soil layers containing gyttja.

Due to topography in the western part of Norway, road constructions often involve high embankment fillings and rock cuts to provide a suitable road alignment. High embankments normally result in settlement and stability/bearing capacity challenges if the soil below is not sufficiently competent. Time constraints on the construction of transportation infrastructure often results in the inapplicability of solutions to these challenges, such as stage loading and pre-loading to improve stability/bearing capacity and settlement of high embankments. The common practice in Norway has as a result been to remove organic soil by excavation and replace it with crushed rock from cuts. This is practiced even if the planned filling has minimal height, e.g. 1-2 m, mainly due to uncertainty in the engineering properties of gyttja. Environmental considerations demand that projects limit or avoid the removal of organic soils as much as possible due to CO\textsubscript{2} emissions from continued decomposition [1,2].
This work aims to explore the possibility of building small fillings on gyttja without excavation and removal. To achieve this, it is important to study the engineering properties of gyttja with respect to stability and settlements so that the NPRA requirements for acceptable stability safety factors as well as acceptable settlements are satisfied. This is done based on extensive field and laboratory research. Results are also evaluated with results from literature. A detailed definition of gyttja is first given followed by details of the field and laboratory work along with their corresponding results.

2. Definition of gyttja
Gyttja is not a common or widely studied soil type, and the following definition by Larson [3] is provided for context:

“Gyttja originates from remains of plants and animals rich in fats and proteins, in contrast to peat which is formed from remains of plants rich in carbohydrates. Dead microscopic aquatic animals are dissolved and decomposed with the aid of bacteria to a flocculent substance, in which mineral particles and less composed remains of plants and animals are embedded. Further decomposition occurs with the aid of organisms living in the substance, such as worms and larvae. Fermentation processes generating sulphuretted hydrogen and methane complete the formation of gyttja. Gyttja formed in nutritious water is greenish in colour. In less nutritious environments, the gyttja becomes brown from mixing with brown-black dy. Gyttja has a more or less elastic consistency which is sometimes jellylike. Gyttja formed in areas with calcareous soil often occurs as a transitional form between gyttja and marled called calcareous gyttja. Depending on the content of mineral particles there are a number of various soils, such as clayey gyttja, organic clay etc.”

3. The test site and overview of investigations
The test site is located in Kopervik in Rogaland county, in western Norway. The site selected for this research contained clayey silty gyttja to depths of up to 10 m. Field investigations include total sounding, CPTu and field vane shear test with 0.5 m interval. In addition, 13 soil samples with 54 mm diameter were extracted. As described in subsequent sections of this article, the laboratory tests include routine tests, hydrometry, ignition test, uniaxial compression tests, oedometer and triaxial tests.

Based on the total soundings in the mire, area topography and total soundings between the wetlands and the nearby fjord, the area is believed to have been connected to the fjord. Following the last period of glaciation mineral soils have been deposited together with decomposed microscopic aquatic animals and plants in stagnant seawater with moderate waterflow between the fjord and the previous bay, filling the area highlighted in Figure 1a and 1b. The total soundings show that the rock surface is approximately 0.5 m under current sea level closer to the fjord.

![Figure 1. a) Overview of the investigation site](image1)

![Figure 1. b) Section showing planned road, and boreholes for the planned road as well as testing referred to in this work](image2)
4. Routine laboratory tests
Detailed routine/index tests were carried out to provide soil identification and classification data including determination of natural water content, consistency limits, unit weight, organic content and shear strength using fall cone and unconfined compression testing [4].

Results from selected routine tests are presented in Figure 2. The water content (measured as the ratio of mass of water to the mass of solid particles) of the soil varies mainly between 80 and 370%. The organic content in the soil varies from 5 to 18%. The unit weight of the soil is between 11.3 kN/m³ and 14.6 kN/m³. The liquid limit w_L varies between 89% and 286% and decreases with reduction in natural water content as well as organic content. The ground water table at the investigation site is approximately at terrain level, which in combination with the low unit weight results in a low effective stress situation. Most of the samples are shown to generally be homogenous and of good quality as shown in Figure 3. In the 5-6 m depth interval the soil has higher organic- and water content, is black-brown in colour and is defined as peat (Figure 2).

![Figure 2. Soil layering and some index test results](image)

5. Strength parameters
The undrained shear strength presented in this section is based on field vane shear tests, uniaxial compression tests, fall cone tests, active triaxial tests and CPTu. In the presentation of field vane shear tests and fall cone tests correction factors based on correlations from the Swedish practice as given by Larson et. al [5] have been applied. A discussion of all the results is presented in section 5.5.

![Figure 3. Sample 54mm, 2.0-2.8m depth.](image)
5.1. Undrained shear strength from uniaxial compression shear tests

Figure 4 shows the results from the uniaxial compression shear tests. The results show higher undrained shear strength than the triaxial test results given in section 5.3 and are also in similar range to the uncorrected test results of the fall cone test (Figure 5), indicating a potential need for data correction to data from uniaxial compression test. Currently there is no correction suggested in literature for uniaxial tests as given for fall cone and field vane test results.

5.2. Undrained shear strength from fall cone test and field vane test

The correlations for interpretation of fall cone tests used in Norway is intended for inorganic clay with low plasticity. As a result, correlations utilized in Sweden for interpretation of fall cone tests on clay, organic clay and gyttja with higher plasticity as presented by Larson et. al [5] were used, as shown in Figure 5. These corrections are meant to give direct shear strength of the organic soil. For interpretation with the Norwegian method reference is made to Holstad [4]. Corrections from Larson et. al [5] based on liquid limit $w_l$ were also applied to the results of the field vane tests. A similar range of undrained shear strength is observed in results of the corrected fall cone test and field vane tests. However, undrained shear strength from fall cone test results are generally seen to show more scatter with depth than from field vane test. The correction applied on the measured field vane test is seen to give results comparable with the strength results obtained from triaxial test as given in section 5.3.

![Figure 4. Results uniaxial compression test](image1)

![Figure 5. Fall cone test results](image2)

![Figure 6. Field vane test results](image3)
5.3. Undrained shear strength from triaxial compression test

The triaxial samples were consolidated for in-situ effective stress levels with $K_0' = 0.6$. Use of $K_0' = 0.6$ is based on results from various literatures [e.g. 3, 6, 7]. One oedo-triaxial test was also performed to see if $K_0' = 0.6$ was representative for the material in this work. Oedo-triaxial is a test procedure using triaxial equipment where the consolidation phase is run drained and at the same time ensuring that the cross-sectional area of the sample is kept constant like an oedometer by increasing cell pressure. Details of the oedo-triaxial test is presented in Holstad [4] and the test gave interpreted $K_0' = 0.57$ which is fairly similar to the assumed value of $K_0' = 0.6$. With this set up, eight triaxial compression tests were performed. Seven out of the eight triaxial compression tests were classified as acceptable quality based on Norwegian evaluation practice [8]. The sample at 2.3 m depth was classified as disturbed but the result is still presented in Figure 7. Peak undrained shear strength at failure as well as undrained shear strength at 2% strain are presented in Figure 7.

Results show that failures occur at strains higher than 2%, which is considered to be an upper limit for inorganic Norwegian clays. The undrained shear strength results are seen to be significantly lower than expected for soft Norwegian clays. Another important factor of the results is that failure happens when the increase in pore pressure has brought the effective horizontal stress down to zero, and failure is primarily expected to occur as the due to vertical cracking of the sample (tensile failure). As a result, it was not possible to reliably interpret the drained strength parameters in the triaxial tests conducted in this study, as the Mohr-Coulomb criteria does not allow interpretation of drained strength parameters for such mode of failure.

![Figure 7. Results of active triaxial tests and strain at failure](image)

5.4. Undrained shear strength from cone penetration test (CPTu)

In Norwegian practice CPTu correlations for organic cohesive soil such as gyttja do not exist. Interpretation of CPTu data for undrained shear strength based on suggestions by Larsson [9], Mlynarek et. al [10, 11] and Lauesen [12] are given in Figure 8. These suggestions are meant to give an equivalent direct shear strength to take anisotropy into considerations [3]. The resulting strength is seen to vary between the different suggestions from literature. The method suggested by Larsson [9] is seen to yield results most comparable to the results obtained from the field vane shear tests conducted in this study. The other methods suggested by Mlynarek et. al [10, 11] and Lauesen [12], overestimated the results.

In an attempt to find local correlations applicable for active shear strength, $N_{kt}$ values are proposed in light of the triaxial test and are given in Figure 9. It is proposed to use $N_{kt} = 15$ (for $w_L > 200\%$) and $N_{kt} = 13$ (for $w_L < 150\%$) to find the active undrained shear strength.
Discussion on undrained shear strength
The results from different field and laboratory tests show a significantly lower undrained shear strength compared to normal soft inorganic Norwegian clays. The experience from Sweden as reported by Larsson [3] is that there is some anisotropy of undrained shear strength based on the organic and water content. The corrected results from fall cone and field vane test are meant to reflect direct shear strength. Whereas the triaxial test give active undrained shear strength. It is thus important to compare the results in light of this effect. Uniaxial compression shear tests are seen to give significantly higher undrained shear strength as compared to fall cone and field vane test. This imply the need for a correction factor of undrained shear strength results obtained with uniaxial compression test. The results from the fall cone test is interpreted based on Swedish correlations and correction factors generally show a higher scatter than the field vane shear test, but in similar range as the field vane shear test. When corrected undrained shear strength from fall cone and field vane test are compared with triaxial tests, it was observed that the undrained shear strength from triaxial interpreted at failure were somehow higher. This is expected to possibly be caused by anisotropy. When it comes to CPTu data, the Swedish correlation for direct shear strength matched well with the field vane shear test results. To obtain active undrained shear strength from the Swedish correlations, it is important to take into account anisotropy. This is detailed in Holstad [4].

Figure 8. Undrained shear strength interpreted from CPTu based on correlations from literature and field vane shear data from current study

Figure 9. Undrained shear strength interpreted with proposed $N_{kt}$-factors based on active triaxial test
5.6. Undrained shear strength after cement stabilization

In most projects where construction time is limited, stage loading and pre-loading are less viable options, especially for projects with higher embankments, and soil stabilization could be a potential alternative. Experimental investigation has been performed on 31 samples stabilized with cement. Samples were mixed with both 100 kg/m³ (i.e. 100 kg cement per cubic meter of gyttja) and with 300 kg/m³ of cement to investigate the effect of soil stabilization. Following stabilization, samples were subjected to uniaxial compression testing after 28 days. The results are presented with data prior to stabilization in Figure 10. The results clearly show that use of cement gives a significant increase in the strength of gyttja, and also indicate that the improvement is related to water content and amount of cement added. The results indicate that for water content higher than 150% the stabilizing effect is reduced for a given cement mix, implying that more cement is necessary with increasing organic- and water content to achieve the same strengthening effect. The mixing ratio is proposed to be determined as a function of water content or liquid limit.

6. Settlement parameters

To establish settlement parameters incremental (IL) as well as continuous rate oedometer tests (CRS) were conducted. In total 8 IL and 6 CRS oedometer tests were performed. The IL tests are done with 24-hour load steps and the CRS tests were done with rate of strain 0.0020 mm/min (0.6%). Settlement calculations in Norway are primarily based on Janbu’s approach [13, 14] and important parameters for this approach, modulus number \( m (-) \), consolidation coefficient \( c_v \) (m²/year) and time resistance number \( r_s (-) \), are emphasized.

6.1. Modulus number \( m (-) \)

The modulus number is defined as \( M = m \cdot (\sigma^* - \sigma_r^*) \), where \( M \) (kPa) is the deformation modulus, \( m \) (-) is the modulus number, \( \sigma^* \) (kPa) is the effective stress and \( \sigma_r^* \) is the reference stress for \( M = 0 \). The material is highly compressible and test results show very high strain, and the use of natural strain is therefore appropriate in the context of this study. To showcase the effect of using linear strain, which is common for inorganic soils, an interpretation using linear strain was also completed. As expected, the material is highly compressible, and the test results show very high strain. Both sets of results are presented in Figure 11.

Larson [3] presented results from Vallda-Kirkbyn site where the modulus number \( m (-) \) is interpreted between 7 and 9 for samples with organic content of 10-12% and water content approximately 160-180%. Larsson [3] did not explicitly mention which strain measure was used.
Therefore, results of this study based on linear and natural strains are given in Figure 11 for the sake of comparison. The interpreted modulus number is given based on organic and water content, and results show a clear trend of a low modulus number that is further reduced with increasing organic- and water content.

![Figure 11. Results of Janbu’s modulus number (m) versus organic and water content.](image1)

6.2. Consolidation coefficient \( c_v \)

The consolidation coefficient \( c_v \) is an important parameter indicating the duration of primary consolidations. The consolidation coefficient is defined as \( c_v = M \cdot k / \gamma_w \), where \( k \) (m/year) is the permeability coefficient, \( M \) is the deformation modulus and \( \gamma_w \) is the unit weight of water. Figure 12 presents the results from interpreted results for the consolidation coefficient for stress levels above the preconsolidation stress (\( \sigma'_c \)). Most results show lower \( c_v \) parameters compared to soft inorganic Norwegian clays which typically have a range of 0.5-15 m\(^2\)/year [8]. The results can also be compared with results from Swedish investigations presented by Larson [3] which show consolidation coefficient \( c_v \) in the 0.06 and 0.6m\(^2\)/year range. The results indicate that primary settlements in such soils could take a very long time.

![Figure 12. Results of consolidation coefficient \( c_v \) for various water content](image2)
6.3. Time resistance number \( r_s \) (-)

The time resistance \( R \) is defined as \( R = \frac{\Delta t}{\Delta \varepsilon} \) where \( \Delta t \) is the change of time and \( \Delta \varepsilon \) is the change of strain. The slope on the curve for time resistance \( R \) plotted against time gives the time resistant number \( r_s \). The time resistant number \( r_s \) can be compared to the coefficient of secondary compression \( \alpha_s \) with the equation \( r_s = \frac{\ln(10)}{\alpha_s} \) [15]. For large strains, it is appropriate to use natural strain as a strain measure, and the time resistance number is mainly interpreted based on natural strain. Interpretation based on linear strain is also performed in order to make a direct comparison with data from literature, i.e. it is not explicitly mentioned in Tk. Geo [6] if a natural strain measure was used, so it is assumed that results given in Tk. Geo [6] are based on linear strain measure. The \( r_s \) results interpreted based on 8 IL tests are presented in Figure 13. The two lowest \( r_s \) values above the preconsolidation stress (\( \sigma_c' \)) from each test were selected for presentation. The results are plotted against water content.

The figure also provides correlations from the Swedish practice as given in in Tk. Geo [6]. The results show that the soils resistance against deformation over time is lower than normal inorganic Norwegian clays which ranges between 100-500 [14]. Janbu [14] also provides data for peat which is in the range of 25-75 and this is in the same range as the data observed in this study, implying that creep settlements are higher. The results from this study are also shown to follow similar trends to Tk. Geo [6] and yields values that are slightly lower for \( r_s \) than the values from Tk. Geo [6]. It is worthwhile to mention that the data from Tk. Geo [6] is based on several investigation sites and that the sites generally have higher effective stress compared to the site in this study.

![Figure 13. Results of time resistance number \( r_s \) (-)](image)

The ratio between the modulus number and the time resistance is presented in Figure 14. This kind of result could be used to get estimate of the creep parameter, \( r_s \), based on CRS test which gives the modulus number, \( m \). The results in Figure 14 are based on natural strain for both modulus number and the time resistance number. The ratio of \( m/r_s \) is in range 0.07-0.14 when the organic part is lower than 14 % and 0.16-0.25 when the organic part is between 17% and 21%.

![Figure 14. Relation between modulus number \( m \) (-) and time resistance number \( r_s \) (-)](image)
7. Final remarks

Field and laboratory tests conducted in this study along with review of literature has provided an increased understanding of the behaviour of organic soils particularly gyttja. The study has provided previously non-existent/scarc reference data for characterization of strength and deformation parameters of gyttja in Norway, and results and correlations have been compared to data from other countries on similar soil materials.

The organic content and the high water content of gyttja studied is observed to significantly affect the strength and deformation parameters in unfavourable way. The strength/bearing capacity are limited and settlements over a long period of time is expected following loading. For smaller embankments, the use of stage- and pre-loading could be possible, and should be considered given sufficient construction time. Allowing for the time needed in the planning of transportation infrastructure projects would also result in reduced environmental impact and emissions. Higher embankments still necessitate excavation and replacement of gyttja and associated organic soils with rock fill due to time constraints, but the results of the cement stabilization tests indicate that the use of cement columns or mass stabilization with cement are viable alternatives. Economic and environmental aspects for both methods should be evaluated before a ground improvement method is chosen.

This work attempted to systematically investigate engineering properties of Norwegian gyttja, and is considered to be a useful preliminary contribution towards a more comprehensive understanding of this soil type. Based on lessons learned from the current research further experimental studies on different sites with significant gyttja deposits are recommended, enabling the creation of a database of Norwegian gyttja and its properties. This could be valuable for engineering design of embankments on such materials.

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