



Research article

Engineering properties of low to medium overconsolidation ratio offshore clays

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Abstract: This paper presents a summary of the geotechnical engineering properties of low to medium overconsolidation ratio offshore inorganic clays derived from a high quality database. These properties, such as preconsolidation stress and undrained shear strength, are required for design of most offshore infrastructure and for assessment of offshore geohazards. The database was developed through a series of Joint Industry Projects using results from offshore site investigations performed worldwide. The key feature of the database is that it only contains results obtained using good to excellent quality samples that were tested using advanced laboratory procedures such as constant rate of strain consolidation and consolidated undrained triaxial and direct simple shear. A secondary objective of the paper was to examine common empirical correlations between index tests (e.g., water content, Atterberg limits) and soil design parameters using the new database. Such empirical correlations between simple and inexpensive index tests and more costly advanced laboratory tests can serve a valuable purpose in offshore infrastructure design. This is particularly the case for preliminary design at early stages of projects where little information is known about soil properties, for small projects with limited site characterization budgets, and international projects at locations where advanced laboratory tests performed to international standards are not available. The paper describes development of the database and presents summary results and plots for undrained shear

strength, in situ stress state, and consolidation and flow parameters. Results from the empirical correlations investigated are presented and the paper concludes with recommendations on use of the data and correlations in practice. The recommendations are limited to clays of low to medium overconsolidation ratio and are not applicable to highly overconsolidated and desiccated clays. While the database primarily consists of offshore clays, the correlations presented should also be applicable to terrestrial clays.

Keywords: clay; empirical correlations; laboratory; preconsolidation stress; undrained shear strength

1. Introduction

The primary objective of this paper is to present a summary of the geotechnical engineering properties of low to medium overconsolidation ratio offshore clays derived from a high quality database. These properties, such as preconsolidation stress (σ'_p) and undrained shear strength (s_u), are required for design of most offshore infrastructure and for assessment of offshore geohazards such as submarine landslides. The collection of data presented is from a database that was developed using results from a large number of offshore site investigations performed worldwide. The key feature of the database is that it only contains results obtained using good to excellent quality samples that were tested using more advanced laboratory procedures. The adverse effects of sample disturbance on the reliability of laboratory measured soil properties is well established and it was an important goal of this study to exclude results from tests performed on poor to very poor quality samples. Furthermore, data were only used from tests that are considered to produce a more reliable measurement of soil properties for design (e.g., constant rate of strain consolidation, anisotropically consolidated undrained triaxial and direct simple shear) and that were performed in a consistent manner. The database and results presented herein provide a valuable collection of engineering properties for offshore clays that can be used for comparing and evaluating results obtained from new offshore investigations and especially those being performed in new, previously unexplored, regions.

A secondary objective of the study was to examine common empirical correlations between index tests (e.g., water content, Atterberg limits) and soil design parameters using the new database. Such empirical correlations between simple and inexpensive index tests and more costly advanced laboratory tests can serve a valuable purpose in the design of offshore infrastructure. This is particularly the case for preliminary design at early stages of projects where little information is known about soil properties, for small projects with limited site characterization budgets, and international projects at locations where advanced laboratory tests performed to international standards are not available. However, the usefulness of any correlation is strongly dependent on the reliability of the database that is used to develop the correlation. Mixing data from a variety of sources can often result in significant scatter and a decrease in the reliability of the correlations because of differences in measurement techniques and the quality of the data used. This is a particular problem for offshore soils since obtaining high quality undisturbed samples is often a significant challenge.

The results presented in this paper are from a series of Joint Industry Projects (JIP) undertaken by the Norwegian Geotechnical Institute (NGI) over the past two decades. The overarching goal of these projects was to develop a new high quality soil properties reference database for offshore clays and to explore the possibility of developing empirical correlations between index tests and engineering design parameters. The vast majority of the data used to develop the database are from projects performed by NGI thus providing a high level of consistency in how samples were processed, evaluated and tested. While the JIP projects investigated many engineering parameters, the focus of this paper is primarily on undrained shear strength, preconsolidation stress, and consolidation parameters and for low to medium overconsolidation ratio (i.e., $OCR < 4$) offshore clays since these are the most important for design for clays. For the empirical correlations investigated, the index properties considered were natural water content and Atterberg limits. The fall cone strength index test was also considered. The paper describes development of the database and presents summary results and plots for undrained shear strength, in situ stress state, and consolidation and flow parameters. Results from the empirical correlations investigated are presented and the paper concludes with recommendations on use of the data and correlations in practice. Some initial results of the database analysis were presented in [1–3].

2. Background

2.1. Empirical correlations

The geotechnical engineering literature contains numerous examples of empirical correlations between basic index properties and soil design parameters (e.g., [4–11]). The key index properties for clays are water content (w) and Atterberg limits including plastic limit (w_p), liquid limit (w_L), plasticity index (I_p), and liquidity index (I_L). For correlations that use Atterberg limits it is important to consider that they are performed on completely remoulded soil. Therefore any natural structure that existed in situ is destroyed and yet it is the in situ structure of the soil that largely controls its behaviour and hence is of most interest for design. Although, Holtz et al. [12] note that Atterberg limits can correlate well with some engineering properties because both are affected by many similar factors including clay mineralogy, pore water chemistry and geologic history. Thus in a very general sense differences in the Atterberg limits of clays imply differences in their engineering behaviour. In terms of water content, it is anticipated, again in a very general sense, that if the natural water content is close to the liquid limit the clay will typically be of lower strength and more compressible (i.e., like a low OCR clay) as compared to a much stronger and stiffer response (i.e., like a higher OCR clay) when the water content is close to the plastic limit.

Some of the better published empirical correlations for clays involve consolidation parameters (e.g., compressibility, coefficient of consolidation) while the weaker ones tend to be for stress state and shear strength parameters such as preconsolidation stress (σ'_p) and undrained shear strength (s_u). For these latter parameters there is often a strong interrelationship between two design parameters (e.g., the strong link between s_u and σ'_p) thus making it difficult to develop a simple correlation using a single index parameter. The following describes some of the more common correlations for clays

found in the literature and served as a starting point in the JIP projects for investigating such correlations using the new database.

Skempton [13] showed that for normally consolidated clays there is a unique relationship between the in situ vertical effective stress (σ'_{v0}) and I_L . However, most clays do not exist in a truly normally consolidated state and the potential existence of such a relationship for overconsolidated clays is often observed empirically. That is, lightly overconsolidated clays often have a water content around the liquid limit ($I_L = 1$) while heavily overconsolidated clays often have a water content around the plastic limit ($I_L = 0$). In terms of σ'_p , it is only for the case of removal of overburden that it would be expected that overconsolidated clays might also exhibit a unique relationship between σ'_p and I_L . Other factors such as weathering, ageing, diagenetic bonding and other physico-chemical processes also change the in situ stress state of a clay [14] and the relationship between σ'_p and I_L becomes weaker and more scattered. Yet in spite of these complicating factors, most empirical correlations for σ'_p use I_L (e.g., [4,14,15]). NAVFAC [4] presents a relationship between σ'_p and I_L that makes use of the sensitivity (S_t) to refine the correlation. Stas and Kulhawy [14] reviewed data for clays with sensitivities ranging from 1 to 10 and suggested a direct correlation between σ'_p and I_L and contrary to NAVFAC, found no influence of sensitivity on the correlation.

Many correlations have been presented between the 1-D compressibility of clays, as for example expressed by the compression index ($C_c = \Delta e / \Delta \log \sigma'_v$, where Δe is the change in void ratio), and either the natural water content (w_n) or the plasticity index. Terzaghi et al. [7] present a correlation between C_c and w_n for a large variety of clays and suggest that such a direct relationship should exist because both properties are controlled by composition and structure unlike I_p and I_L . Leroueil et al. [16] found that S_t strongly influences the value of C_c and present a correlation that links C_c with the in situ void ratio (e_0) and S_t for sensitive clays. The coefficient of consolidation (c_v) for the normally consolidated state has been found to correlate well with w_n [17] while NAVFAC [4] presents a correlation between c_v and w_L for normally consolidated, overconsolidated and remoulded states.

The literature presents numerous examples of a good correlation between I_L and the remoulded undrained shear strength (s_{ur}) (e.g., [7,10,16,18]). Bjerrum [19], subsequently confirmed by others, found a good correlation between S_t and I_L . Unlike s_{ur} , the intact or in situ s_u typically does not correlate well with any of the basic index parameters and most correlations for s_u focus on normalisation by either σ'_{v0} or σ'_p . Skempton [20] first suggested a correlation between s_u / σ'_{v0} and I_p based on field vane shear strengths for normally consolidated marine clays. However, Bjerrum and Simons [21] showed that this relationship did not work well for normally consolidated sensitive Norwegian marine clays. Ladd [22] and Terzaghi et al. [7] present data on the variation of s_u normalised by the laboratory consolidation stress σ'_{vc} (or s_u / σ'_p in [7] if overconsolidated) with I_p as measured in triaxial compression (TC; s_{uC}), direct simple shear (DSS; s_{uD}), and triaxial extension (TE; s_{uE}) for normally to lightly overconsolidated clays. These data show no trend in s_{uC} / σ'_{vc} with I_p while there is some increase in s_{uD} / σ'_{vc} and s_{uE} / σ'_{vc} with an increase in I_p showing that the degree of anisotropy of clays is greater for low plasticity clays. Ladd [22] suggests that the ratio of extension to compression undrained shear strength (K_s) can be estimated as $K_s = 0.37 + 0.0072I_p$. Mesri [23,24] analysed field and laboratory data and concluded that the mobilised undrained shear strength $s_u(\text{mob})$ for stability problems (e.g., embankments, footings and excavations) is related to σ'_p as $s_u(\text{mob}) =$

$0.22\sigma'_p$ independent of I_p . Larsson [25] proposed a similar relationship with s_u independent of I_p for inorganic clays.

In summary, the literature presents numerous examples of empirical correlations between index parameters and soil design parameters. However, the quality of some correlations is difficult to evaluate as the data used are often from a wide variety of sources, involve different measurement techniques and in some cases no consideration was given to the quality of the samples used to obtain the measured design parameters. This is an especially important issue for offshore clays since obtaining high quality undisturbed samples is a challenge. It is only correlations for soil properties representing a normally consolidated state (i.e., properties measured from samples of natural clays that were loaded into the normally consolidated range in the laboratory) that the issue of sample quality is of less importance.

2.2. Intrinsic properties

Some of the properties of the clays in the database were evaluated relative to their intrinsic state following the work of Nagaraj as summarized in [26–29]. Nagaraj and Srinivasa Murthy [30,31] developed a framework for assessing the properties of natural clays relative to their remoulded state (also known as reconstituted or intrinsic state) as determined from measuring the behaviour of the remoulded clay prepared to a w somewhat greater than w_L . They defined the intrinsic state parameter as e divided by e at the liquid limit (e_L) and found that the one-dimensional compression curve of many remoulded uncemented clays plot along a near unique intrinsic state line in $e/e_L-\sigma'_v$ space (Figure 1a). Burland [27] introduced the in situ void index $I_{v0} = (e_0 - e^*_{100})/C^*_c$, where the intrinsic properties C^*_c represents the slope of the 1-D compression curve of a remoulded clay (between $\sigma'_v = 100$ & 1000 kPa) and e^*_{100} is the void ratio at $\sigma'_v = 100$ kPa. The superscript * denotes a soil property on the remoulded 1-D compression curve. The intrinsic compression line (ICL) is a reference line for remoulded compression curves in $I_v-\sigma'_v$ space and the sedimentation compression line (SCL) is another reference line developed by Burland that represents where data from 1-D consolidation tests performed on intact samples clustered for some selected geologically normally consolidated natural clays (i.e., they have not been subjected to mechanical overconsolidation, desiccation, cementation, etc.). Burland [27], Burland et al. [28], and Chandler [29] suggest that clays with an in situ state which plots above the SCL tend to be sensitive, cemented whereas clays that plot below to well below the ICL tend to be overconsolidated (Figure 1b).

Burland et al. [28] introduced the normalised vertical yield stress $\sigma'_{vy}/\sigma^*_{ve}$ where σ'_{vy} is the vertical yield stress for one-dimensional loading and σ^*_{ve} is the equivalent σ'_v on the remoulded compression curve at e_0 as shown in Figure 1a in $e/e_L-\sigma'_v$ space. The yield stress ratio YSR was defined by [28] as $\sigma'_{vy}/\sigma'_{v0}$ to distinguish it from the historical definition of OCR ($= \sigma'_p/\sigma'_{v0}$) as established from geologic evidence (e.g., mechanical overconsolidation such as erosion, glacial loading, etc.). This distinction is important when trying to understand the evolution of the yield stress for a clay [29] although there is often not enough geologic evidence to do so. In practice the distinction is not as important as ultimately it is σ'_{vy} that is of interest for design. In this paper the more familiar σ'_p (and OCR) is used as the value of the vertical yield stress determined from laboratory 1-D consolidation tests and to represent all preconsolidation mechanisms.

Cotecchia and Chandler [32] and Chandler [29] termed $\sigma'_{vy}/\sigma'^*_{ve}$ the stress sensitivity (S_σ). A new parameter, the void ratio sensitivity (S_e), is introduced here as

$$S_e = \Delta e^*/e_L = (e_0 - e^*)/e_L \quad (1)$$

for a given σ'_v and where Δe^* expresses how different e_0 is compared to a soil's remoulded or intrinsic state (Figure 1a) at σ'_{v0} . For highly structured, aged clays without mechanical overconsolidation, Δe^* at σ'_{v0} is positive and can be quite large whereas for moderate to heavily overconsolidated clays it is small and can in fact be negative. The void ratio sensitivity is analogous to the metastability index of [33] which evaluates the difference in the liquidity index of the undisturbed and destructured soil at $\sigma'_v = \sigma'_p$. Shibuya [34] also uses the metastability index but defined it in terms of the shear modulus instead of σ'_p .

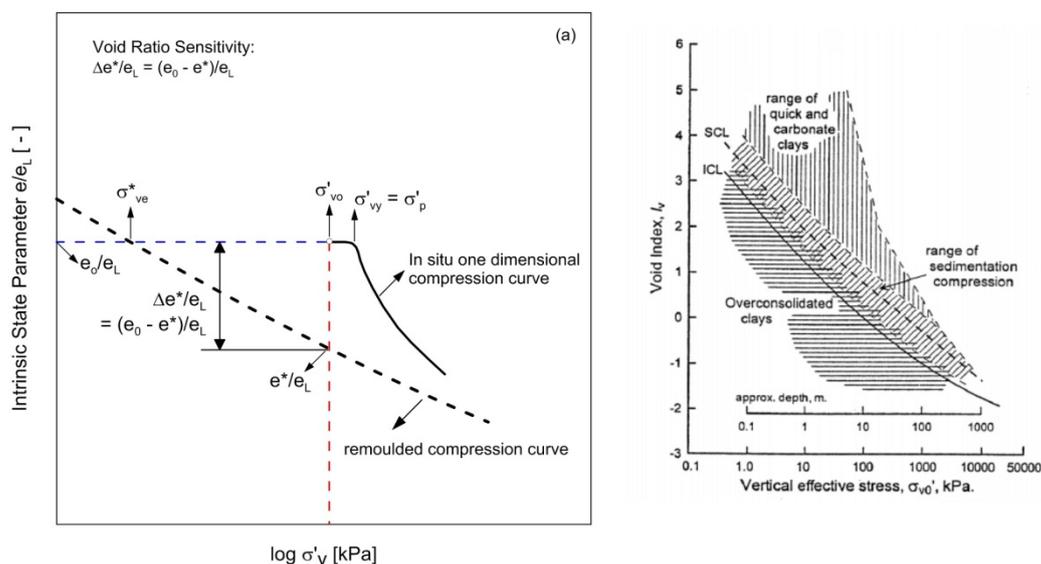


Figure 1. (a) Nagaraj intrinsic state parameter e/e_L framework [26] with introduction of the void ratio sensitivity parameter S_e (Eq 1) and (b) different stress history states in terms of Burland void index (from [29]).

3. Database development

A major objective of the JIP projects was to develop a high quality database of soil properties. To this end it was considered essential that the site investigation data collected were objectively screened to ensure that (1) the undrained shear strength data were only from consolidated strength tests that were performed using the same laboratory consolidation procedure, and (2) that results from tests conducted on samples that suffered excessive disturbance were not included. The following describes the screening procedure used in developing the database. It is also noted that the vast majority of the data collected were from tests performed at NGI thus providing a high level of consistency in how the samples were handled and tested. The majority of the offshore samples were collected using a thin walled fixed piston sampler; for the onshore sites, a majority of the samples were collected using a Sherbrooke block sampler.

3.1. Test methods

Reliable determination of undrained shear strength values for design should focus on use of equipment that can conduct consolidated-undrained (CU) tests (e.g., TC, DSS). These tests are often referred to as advanced laboratory tests but in fact this is a relative classification since these tests are routinely conducted at laboratories such as NGI while other laboratories that rarely or cannot perform them would consider them advanced. Undrained shear strength anisotropy is an important design consideration for stability problems and data from test programs using a combination of TC, DSS and TE provide reliable data on s_u anisotropy. If good quality samples are available then NGI conducts such tests with anisotropic (A) consolidation to the estimated in situ effective stress state σ'_{v0} , σ'_{h0} (i.e., Recompression method, [35]). Isotropically consolidated undrained (CIU) triaxial compression tests are common in practice rather than CAU tests because of the widespread availability of basic triaxial equipment and the relative simplicity of the isotropic consolidation procedure. But an isotropic state of stress does not accurately replicate in situ K_0 stress conditions especially for low OCR clays. The use of isotropic consolidation to the mean in situ effective stress [i.e., $\sigma'_c = \sigma'_m = 1/3(\sigma'_{v0} + 2K_0\sigma'_{v0})$] is used by some laboratories as a substitution for CAU consolidation. However, for low OCR soils, with K_0 values well below 1, such an isotropic stress state will apply too low a vertical effective consolidation stress (σ'_{vc}) relative to the in situ stress state. This can result in important differences in the measured stress-strain-strength behaviour because of undrained shear strength anisotropy. Thus only s_u data from CAUC (TC for short), CAUE (TE for short), and consolidated undrained DSS tests that were consolidated to within $\pm 5\%$ of the best estimate in situ effective stress state were included in the database. The only exception to this was that for several sites some SHANSEP [22,36,37] OCR = 1 test results were included as being representative of the normally consolidated behaviour for those soils.

The TC and TE Recompression tests were performed in accordance with the equipment and procedures described in Lacasse and Berre [38]. Anisotropic consolidation was performed by estimating the in situ K_0 using the empirical correlation among K_0 , OCR and I_p developed by Brooker and Ireland [39] and consolidating to a final vertical effective axial stress $\sigma'_{ac} = \sigma'_{v0}$ and final radial effective stress $\sigma'_{rc} = K_0\sigma'_{v0}$. Hydrostatic in situ pore pressure (u_0) conditions were assumed for computing σ'_{v0} ; while not certain, it is believed that u_0 was not significantly different than hydrostatic for all the sites. The DSS tests were performed using the equipment described by Bjerrum and Landva [40] and with $\sigma'_{ac} = \sigma'_{v0}$. For DSS tests on overconsolidated specimens, σ'_v was first preloaded up to a stress level greater than σ'_{v0} (typically around $0.8\sigma'_p$) and then unloaded to $\sigma'_{ac} = \sigma'_{v0}$ because simple reconsolidation to σ'_{v0} in the semi-rigid wire reinforced DSS membrane would result in too low a radial stress relative to in situ conditions [e.g., 41,42].

Strength index tests such as the torvane, pocket penetrometer, laboratory vane, fall cone and unconfined compression (UUC) tests are very common in offshore practice. While these tests are quick and easy to conduct, they can be significantly influenced by sample disturbance, presence of fissures, use of fast shear rates, and different modes of shear. As a result, data collected using these devices at best represent relative strengths rather than values suitable for design [22,37]. Furthermore, data from such tests do not always result in so-called conservative s_u values. Germaine and Ladd [43] present four examples of data from sites with high quality sampling and well developed reference

strengths. In two of the examples the UUC strengths were considered unsafe and Germaine and Ladd [43] conclude that UUC data can be low, reasonable or unsafe for design.

Data from strength index tests were not included in the database with the exception of the fall cone for measurement of s_u and the remoulded undrained shear strength s_{ur} . At NGI the fall cone index s_u is typically measured on an unconfined specimen cut from a sample during extrusion from the sample tube. Thereafter the specimen is typically remoulded by thoroughly hand mixing with a spatula until a constant maximum fall cone penetration value is achieved. The fall cone penetration is converted to s_u or s_{ur} using the empirical correlations given in the Norwegian Standard NS8015 [44] which are from the 1946 Swedish Geotechnical Institute (SGI) calibrations [45] and differ from that given in the ISO 17892-6:2017 Fall cone test standard [46]. For the standard 10g and 60g cones with an apex angle of 60° the difference in the cone factors is significant [47].

For measurement of consolidation behavior, NGI prefers to use the constant rate of deformation or strain (CRS) test as it gives better definition of the consolidation (ε - σ'_v) and flow (ε - k_v - c_v) characteristics of a test specimen. Better defined compression curves allow for more accurate determination of σ'_p . However, data from some incremental load (IL) consolidation tests were also included in the database since this type of test was used for a few sites. The CRS and IL tests were performed in accordance with the equipment and procedures described in Sandbækken et al. [48]. All σ'_p values were determined using Casagrande's construction but none of the values were adjusted for possible strain rate effects in the CRS tests. These tests were typically performed with an axial strain rate of 1.4 to $2.8 \times 10^{-6}/s$ resulting in a ratio of the base pore pressure to total axial stress in the range of 2 to 7%. The low base excess pore pressure ratio indicates that the selected strain rate was less than approximately 10 times the strain rate corresponding to the end-of-primary (EOP) consolidation [49]. For tests performed at 10 times the EOP strain rate Mesri et al. [49] suggest that it may be necessary to adjust CRS σ'_p values by a factor of approximately 0.9. However, while this possible strain rate effect is recognized, no such adjustments to σ'_p values were made and it is noted that the reported σ'_p values could be between approximately 1.0 to 1.1 times the equivalent EOP σ'_p depending on specific test conditions.

All liquid limit data and thus correlations presented in this paper using Atterberg limits are based on the liquid limit as measured using the fall cone (FC) apparatus, which can give a different value than that determined using the US Casagrande cup of the American Society for Testing and Materials (ASTM) International standard [50] and the Vasiliev cone of the Russian standard [51]. NGI [52] summarises several published relationships between w_L (Note: herein w_L is the default notation used to indicate measurement of liquid limit using the fall cone) and the Casagrande cup $w_{L,CC}$ to which an average fit to these data gives

$$w_L = 5.0 + 0.96w_{L,CC} \quad (2)$$

for $w_L < 125\%$. This equation, or any other available site/soil specific correlation, can be used to convert $w_{L,CC}$ to w_L for using the correlations in this paper. Likewise in the Russian laboratory testing system, w_L is determined using the Vasiliev cone (VC; 51) which also gives different values than the Fall Cone. Based on the literature (e.g., [53]) and additional data collected and analysed by NGI [54] the following relationship is suggested for converting values between the two systems

$$w_L = 1.21w_{L,VC} \quad (3)$$

3.2. Sample quality

The adverse effects of sample disturbance on laboratory measurement of clay design properties is well documented (e.g., [7,9,37,55,56]). It was considered essential that the database eliminated, in an objective manner, any results from tests performed on samples of poor to very poor quality. Thus sample quality for all consolidation (IL and CRS), CAU triaxial, and DSS tests was evaluated using NGI's criteria for sample disturbance which is the measure of $\Delta e/e_0$ during reconsolidation to the estimated in situ effective stress state as described by [55,56]. In this method, samples are ranked using four categories as very good to excellent (1), fair to good (2), poor (3) and very poor (4) based on the measured value of $\Delta e/e_0$. Only data from tests performed on samples that were determined to be in the top two categories (very good to excellent and fair to good) were included in the database. Additionally any specimens that had net swelling during laboratory reconsolidation were excluded. The $\Delta e/e_0$ criteria were developed based on results from laboratory tests performed on marine clays collected from depths below the seafloor of 0 m to 25 m and range in properties of 6% to 43% for plasticity index, 20% to 67% for water content, and 1 to 4 for OCR [56]. Some of the collected data are outside of these ranges, especially depth, but the criteria were nevertheless applied in such cases.

3.3. Void index and void ratio sensitivity

Determination of the void index and void ratio sensitivity ideally requires one-dimensional consolidation tests performed on specimens remoulded to w between $1.0w_L$ to $1.5w_L$ as suggested by Burland [27]. However, no such tests were available for the database sites and thus the empirical correlations recommended by Burland [27] for C_c^* and e^*_{100} were used. These correlations are a function of e_L which was computed for all the soils using w_L , measured specific gravity (G_s) and assuming degree of saturation $S = 100\%$

$$e^*_{100} = 0.109 + 0.679e_L - 0.089e_L^2 + 0.016e_L^3 \quad (4)$$

$$C^*_c = 0.256e_L - 0.04 \quad (5)$$

where e_L = void ratio at liquid limit which was taken as equal to $G_s w_L / 100$. The void ratio sensitivity $S_e = \Delta e^* / e_L$ correlation for σ'_p (introduced in Section 5.1) required an estimate on the remoulded compression curve of e^* at σ'_{v0} and σ^*_{ve} at e_0 (Figure 1a) and were computed as

$$e^* = e^*_{100} + \log(100/\sigma'_{v0})C^*_c \quad (6)$$

$$\log(\sigma^*_{ve}) = 2.0 - (e_0 - e^*_{100})/C^*_c \quad (7)$$

4. Database characteristics—summary of soil properties

The database consists of 25 offshore and 5 well characterized onshore sites as summarised in Table 1. For the offshore sites water depths cover a significant range from a few 10s m to greater than 1500 m and sample depths range from near seafloor up to 150 m below seafloor but with a majority of samples being collected within 50 m of the seafloor. Natural water content ranges from 15 to 150% (number of data points $n = 1322$) with the majority of values in the range of 20 to 80%.

Plasticity index ranges from approximately 6 to 100% ($n = 1212$) and most of the liquidity index values are between -0.4 to 2.0 ($n = 975$). The Atterberg limits cover a wide range of values with most of the data plotting above the A-line in a Casagrande plasticity chart as presented in Figure 2. The corresponding Unified Soil Classification System (USCS) designations are mainly low plasticity clay (CL) and high plasticity clay (CH) with an occasional high plasticity silt (MH). The range of σ'_p values ($n = 269$) is very large but a majority of the data are less than 600 kPa and the corresponding OCRs range from 1 to 3. Most of the offshore sites in the database can be considered geologically normally consolidated in that they have not been subjected to post-depositional changes in total stress or pore pressure, although other preconsolidation mechanisms such as aging and physical chemical or biological phenomena may have taken place. The CAU s_u data consists of $n = 301$ for TC, $n = 203$ for DSS, and $n = 99$ for TE with s_u taken for triaxial tests as equal to $q = 0.5(\sigma_1 - \sigma_3)$ at 10% axial strain or at peak q_f if it occurred before this strain and for DSS tests at the horizontal shear stress τ_h at 15% shear strain or $\tau_{h,max}$ if it occurred before this strain.

The data plotted in Figures 3 and 4 shows that collectively the sites cover a very broad region in Nagaraj $e/e_L - \sigma'_{v0}$ and Burland $I_{v0} - \sigma'_{v0}$ space which implies significant differences in the composition, depositional environment and subsequent geologic history of the sites and even various clay units at individual sites. Considerable effort was put into trying to partition the database into groups of common geology and geologic history but this proved difficult as there ended up being so many variables to consider. Burland's $I_{v0} - \sigma'_{v0}$ space was used as an objective proxy and the database was simply partitioned into three nominal groups based on where data plotted in Burland's space, i.e., below the ICL, around the ICL, and around/above the SCL. Partitioning the database into the three "Burland" groups was found to have separated the sites, and in some cases two or more different clay units at some individual sites, into groups that were reasonably consistent with what knowledge was available about each site's geology. But overall, consideration of the separate Burland groupings did not result in significant differences in the correlations investigated.

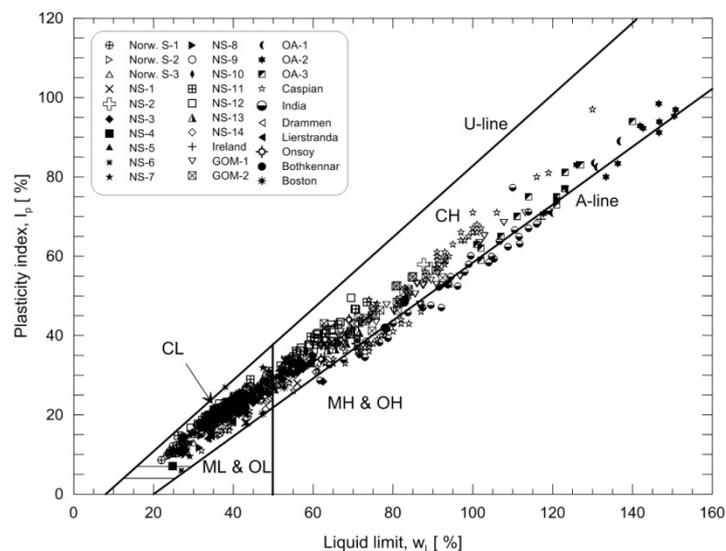


Figure 2. Atterberg Limits data plotted in Casagrande plasticity chart (note: all liquid limit values measured using the fall cone apparatus).

Table 1. List of sites included in the database and range of basic index properties.

Area	Location	Graph Symbol	Sample Depth (m)	w _n (%)	w _L (%)	I _p (%)	I _L (-)	Clay Fraction (% < 2 μm)
Norwegian Sea	Norw. S-1	⊕	0–8	14–24	22–65	9–38	0.02–0.83	23–46
	Norw. S-2	▷	3–24	21–50	35–57	15–32	0.19–0.53	33–52
	Norw. S-3	△	2–88	15–28	30–53	13–31	–0.07–0.69	27–51
North Sea	NS-1	✕	1–127	15–44	28–63	10–42	–0.30–2.40	n/a
Norwegian Sector	NS-2	⊕	2–4	73–83	88–89	57–58	0.75–0.83	51
	NS-3	◆	1–106	16–28	32–63	17–35	–0.21–0.53	19–64
	NS-4	■	0–23	17–26	25–43	7–22	–0.17–0.24	16–40
	NS-5	▲	1–140	15–49	35–60	16–35	–0.04–1.91	16–55
	NS-6	✱	14–106	13–25	27–43	6–28	–1.17–0.28	13–42
	NS-7	★	2–99	16–57	29–49	10–32	0.06–1.95	12–38
	NS-8	▶	0–60	14–26	26–45	11–26	–0.19–0.56	13–37
	NS-9	○	1–106	16–58	31–55	15–33	–0.04–1.40	3–62
	NS-10	◆	0–39	17–26	32–58	17–36	0.10–1.54	15–40
	NS-11	⊞	6–67	14–33	26–74	15–48	0.05–0.36	n/a
	NS-12	□	0–99	16–76	29–70	17–50	0.06–1.3	20–73
	NS-13	▲	1–25	20–82	35–72	19–41	–0.14–1.49	32–67
	North Sea-Danish	NS-14	◇	6–11	19–33	47–63	21–34	–0.29–0.09
Offshore Ireland	Ireland	⊕	0–59	27–63	37–117	19–69	0.18–1.1	13–59
Gulf of Mexico	GOM-1	▽	8–89	36–80	54–113	31–71	0.17–1.13	42–69
	GOM-2	⊞	4–16	53–104	57–93	33–59	0.62–1.61	42–59
Offshore Africa	OA-1	☾	1–17	105–147	120–137	71–89	0.75–1.20	64–75
	OA-2	★	5–19	115–147	118–151	71–98	0.68–1.25	44–68
	OA-3	◼	1–17	85–155	94–140	53–94	0.70–1.46	49–83
Caspian Sea	Caspian	☆	4–149	21–68	32–130	11–97	0.05–1.48	n/a
Offshore India	India	◐	1–23	39–126	62–119	29–77	0.04–1.33	31–80
Onshore Norway	Drammen	◁	8–17	32–53	37–54	17–29	0.67–0.98	35–41
	Lierstranda	◄	6–22	33–42	34–43	14–20	0.86–1.40	31–36
	Onsøy	⊕	3–25	52–72	57–74	32–44	0.71–1.10	21–38
Onshore UK	Bothkennar	●	6–8	64–75	78–83	42–49	0.68–0.94	17–26
Onshore US	Boston	✱	13–40	32–54	42–57	20–34	0.24–1.05	45–54

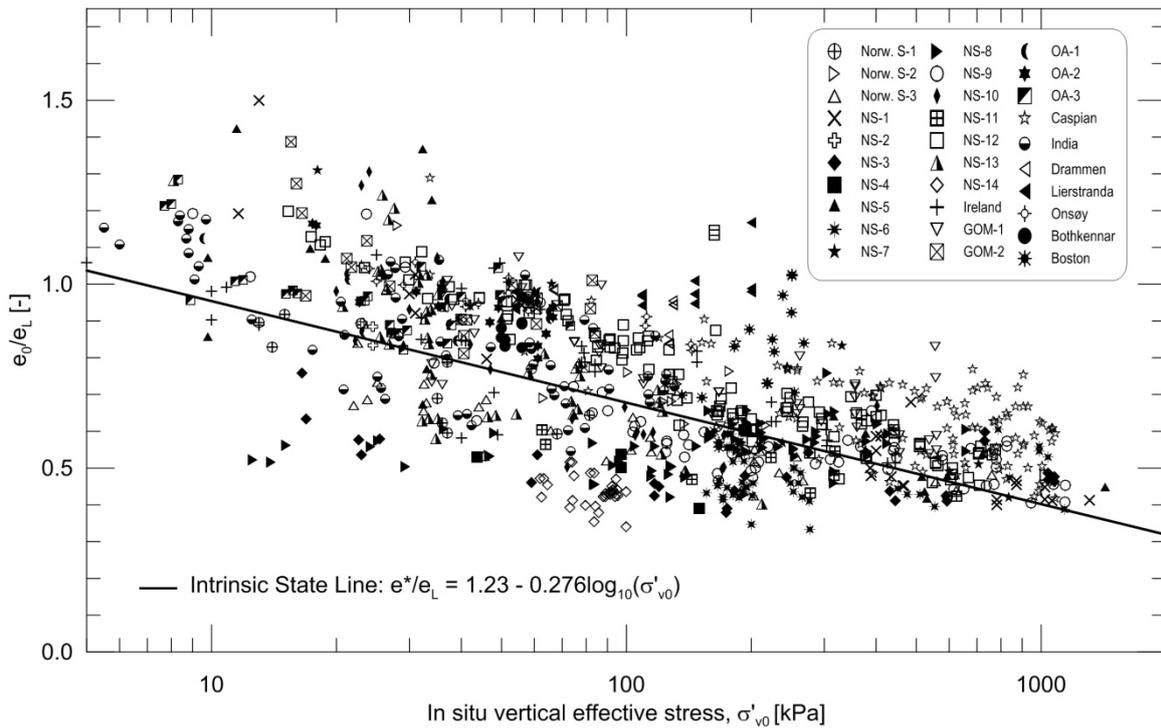


Figure 3. Data from all sites plotted in Nagaraj intrinsic state parameter—in situ vertical effective stress space.

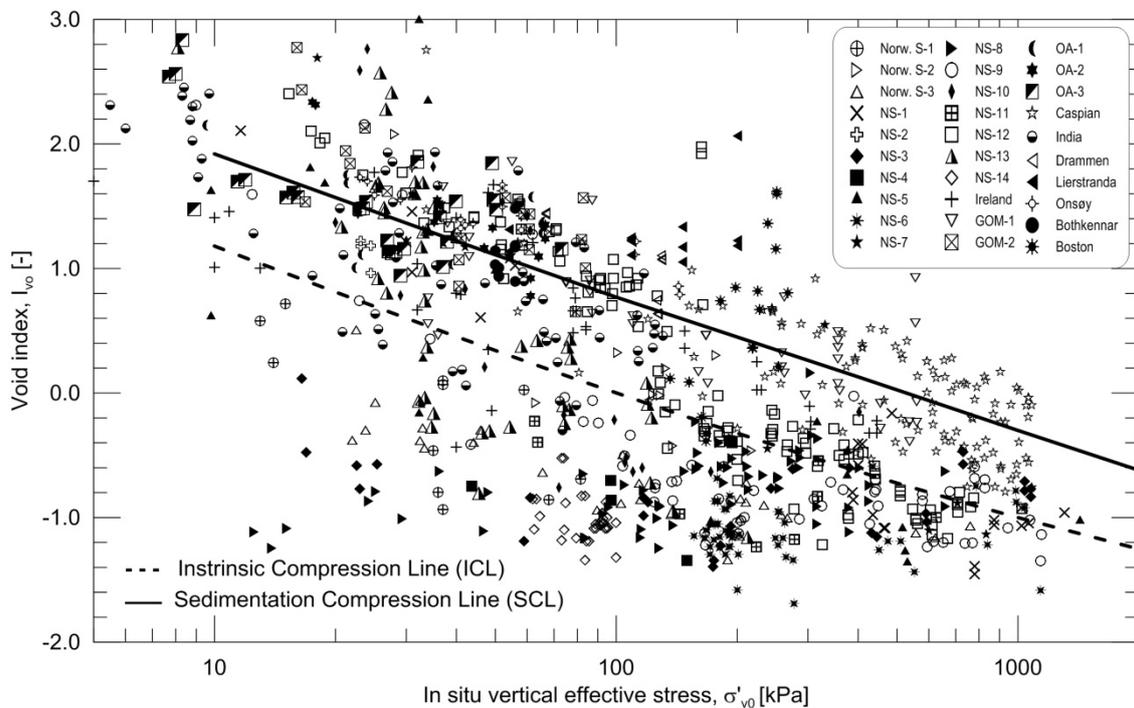


Figure 4. Data from all sites plotted in Burland void index—in situ vertical effective stress space. SCL from [27] ICL from [29].

5. Engineering properties and correlations

5.1. In situ state

There is a strong trend between water content and total unit weight (γ_t) as presented in Figure 5 for which the best fit equation gives

$$\gamma_t = (26.06 + 0.254w_n)/(1 + 0.0256w_n) \quad n = 1191, r^2 = 0.97, \sigma = 0.42 \quad (8)$$

where n equals the number of data pairs, r^2 equals the coefficient of determination, and σ equals one standard deviation. Equation 8 follows the theoretical relationship between total unit weight, water content and an assumed density of solids which infers that in general the clays tested had a high degree of saturation.

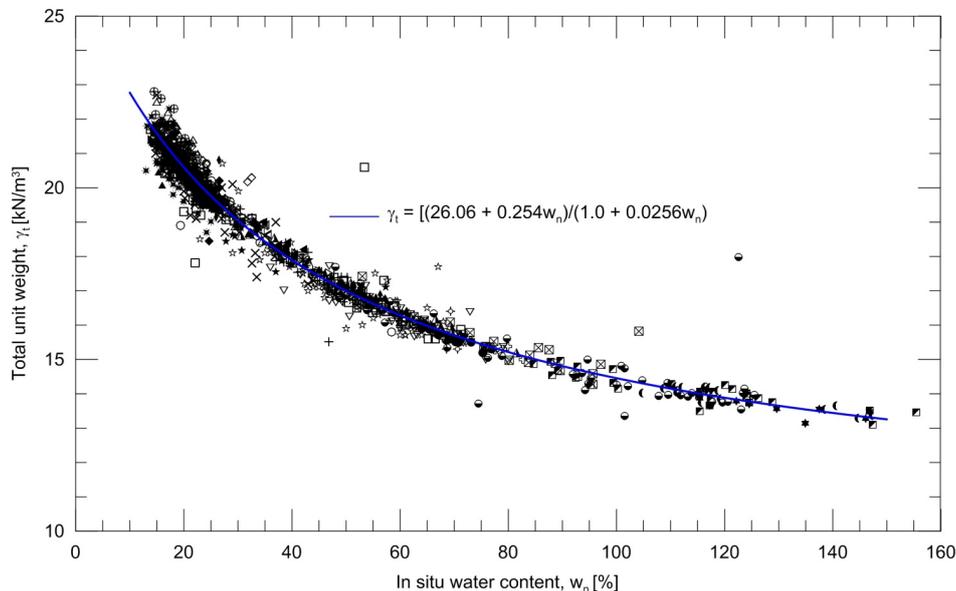


Figure 5. Relationship between total unit weight and in situ water content.

Significant effort was made to determine if a reasonable correlation could be found between σ'_p and various index parameters, this also included consideration of multi-variable regression analyses using two or more index parameters. However, in all cases no universal reliable correlation was found directly with any of the traditional index parameters investigated. Given the complex and highly variable nature of the composition, depositional environment, geology and stress history among the sites worldwide, and even sometimes within a site, this finding is perhaps not surprising. Even the well-known traditional correlation between σ'_p vs I_L was overall relatively poor. Plots of σ'_p vs I_L whether examined for the full database, Burland groupings or individual sites do indeed show a general trend of decreasing σ'_p with increasing I_L . However, there is considerable scatter in the data, especially for I_L less than 0.5. For example, Figure 6 plots the data for the full database for which a least squares regression results in

$$\sigma'_p = 10^{(2.94 - 1.09I_L)} \quad n = 181, r^2 = 0.66, \sigma = 0.74 \quad (9)$$

The efficacy of this correlation was checked by predicting σ'_p profiles for several sites not in the database and it was generally found to be a fair to poor predictor of σ'_p in comparison to the measured data for these trial sites and especially so with increasing depth. Adding in S_t as a second variable showed some general trends but overall resulted in little to no improvement in this correlation for σ'_p .

While exploring various ideas for developing a correlation for σ'_p , a better predictor than I_L was found by linking Chandler's [29] stress sensitivity σ'_p/σ_{ve}^* to the void ratio sensitivity $\Delta e^*/e_L$ introduced in Eq 1 and presented schematically in Figure 1a. Figure 7 plots σ'_p/σ_{ve}^* versus $\Delta e^*/e_L$ for the full database and shows a strong trend between the two parameters as anticipated. A best fit regression to the data results in

$$\sigma'_p/\sigma_{ve}^* = 10^{(0.29 + 3.96\Delta e^*/e_L)} \quad n = 181, r^2 = 0.96, \sigma = 0.34 \quad (10)$$

While this approach is merely empirical, Eq 10 was found to be a better predictor of σ'_p profiles than use of I_L in Eq 9 for trial sites not in the database and especially as depths extended beyond approximately 35 m below seabed. For the grouping of soil units that plotted around and above the SCL and were from less than 35 m below the seabed, the best fit regression equation is

$$\sigma'_p/\sigma_{ve}^* = 10^{(0.30 + 4.04\Delta e^*/e_L)} \quad n = 86, r^2 = 0.88, \sigma = 0.34 \quad (11)$$

A separate database consisting of results from terrestrial investigations was created in the project to examine empirical correlations for the coefficient of lateral earth pressure at rest K_0 [52]. However, no suitable empirical correlation was found between K_0 and any index property. A strong trend was found between K_0 and OCR (Figure 8) for which a best fit gave

$$K_0 = 0.52(\text{OCR})^{0.47} \quad n = 33, r^2 = 0.78, \sigma = 0.076 \quad (12)$$

However, there is some considerable scatter in the data at any given OCR. This fit implies a representative constant volume friction angle $\phi'_{cv} = 29^\circ$ for $K_{0,NC} = 1 - \sin\phi'_{cv}$ as per Mesri and Hayat [42].

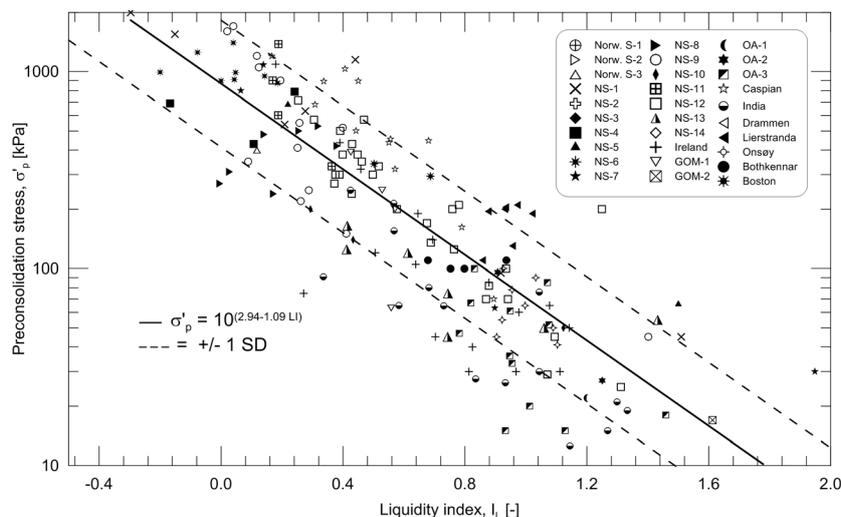


Figure 6. Preconsolidation stress versus liquidity index.

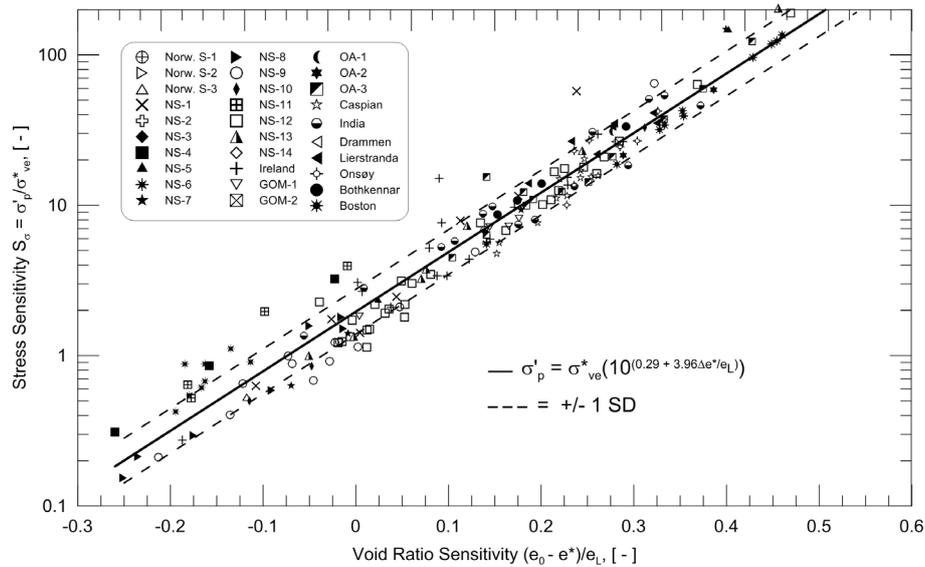


Figure 7. Stress sensitivity $S_\sigma = \sigma'_p/\sigma^*_{ve}$ versus void ratio sensitivity $S_e = \Delta e^*/e_L$ (Note: equations for computing σ^*_{ve} and Δe^* are given in Section 3.3).

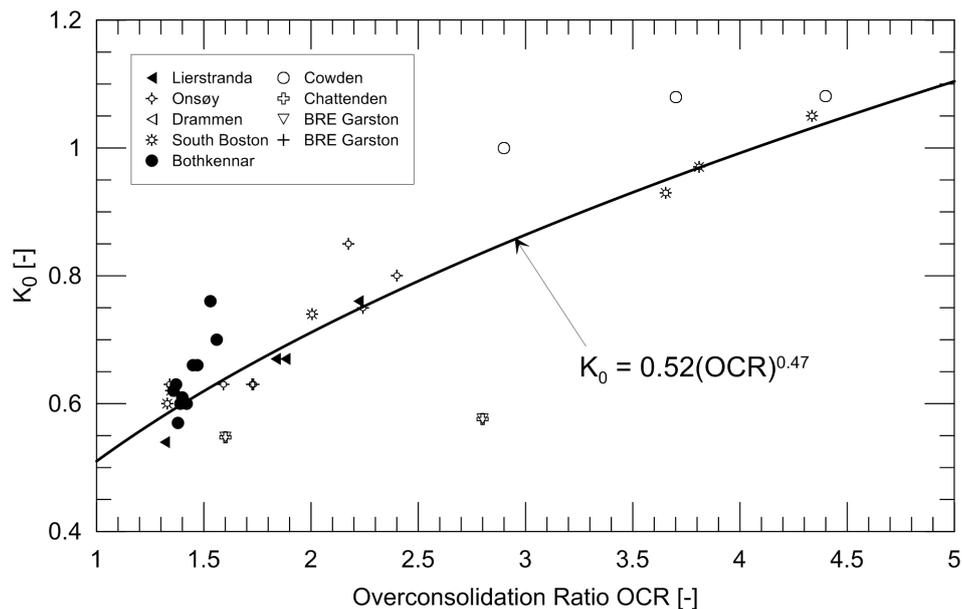


Figure 8. Relationship between K_0 and OCR based on data from terrestrial sites.

5.2. Consolidation and flow parameters

At NGI the compressibility of clays is often expressed using the constrained modulus ($M = \Delta\sigma'_v/\Delta\varepsilon$) which is the tangent modulus of the stress-strain curve for one-dimensional compression as described by Janbu [17]. For normally consolidated to lightly overconsolidated stress states Janbu [17] showed that M can be taken as linearly increasing with increasing σ'_v such that

$$M = m(\sigma'_v - \sigma'_r) \quad (13)$$

for $\sigma'_v > \sigma'_p$ and where m = modulus number and σ'_r = reference stress (intercept on the σ'_v axis of M vs. σ'_v for σ'_v stress levels $> \sigma'_p$). If σ'_r is equal to zero then the compression index $C_c = 2.3(1 + e_0)/m$. Figure 9 plots the constrained modulus M_1 for the stress increment from σ'_{v0} to σ'_p versus w_n and Figure 10 plots m (to compute M using Eq 13 for $\sigma'_v > \sigma'_p$) versus water content. There is a marked decrease in M_1 and m with increasing w_n . For low values of w_n there is significant variability in the values of M_1 and m . Figure 11 plots $c_{v,min}$ versus w_n with $c_{v,min}$ taken as the minimum value of c_v measured for the normally consolidated stress state, which typically occurred at σ'_v just beyond σ'_p . The data display general trend of decreasing $c_{v,min}$ with increasing w_n . The data for c_v for the stress range σ'_{v0} to σ'_p showed too much scatter to be considered. Terzaghi et al. [7] note the recompression values of c_v (i.e., loading from σ'_{v0} towards σ'_p) can be from 1 to as much as 100 times the normally consolidated value although for most soft clays the ratio typically ranges from around 5 to 10. Furthermore for soft clays and silts c_v is more or less constant in the compression range from σ'_p to $5\sigma'_p$. The parameters M , m and c_v also showed trends of decreasing values versus increasing I_p and I_L but overall the best correlations for these parameters were using w_n as plotted in Figures 9 to 11.

Figure 12 plots the vertical hydraulic conductivity as determined by direct constant head measurement during CRS (during a constant stress phase) or IL testing [48] versus void ratio. Consideration of plasticity index and clay fraction ($CF = \% < 2 \mu m$) indicates some nominal trend in the results. Overall the majority of the data are within the range of 1×10^{-10} to 5×10^{-9} m/s which is somewhat larger than the range for most soft clays reported by Terzaghi et al. [7] of 5×10^{-10} to 5×10^{-9} m/s.

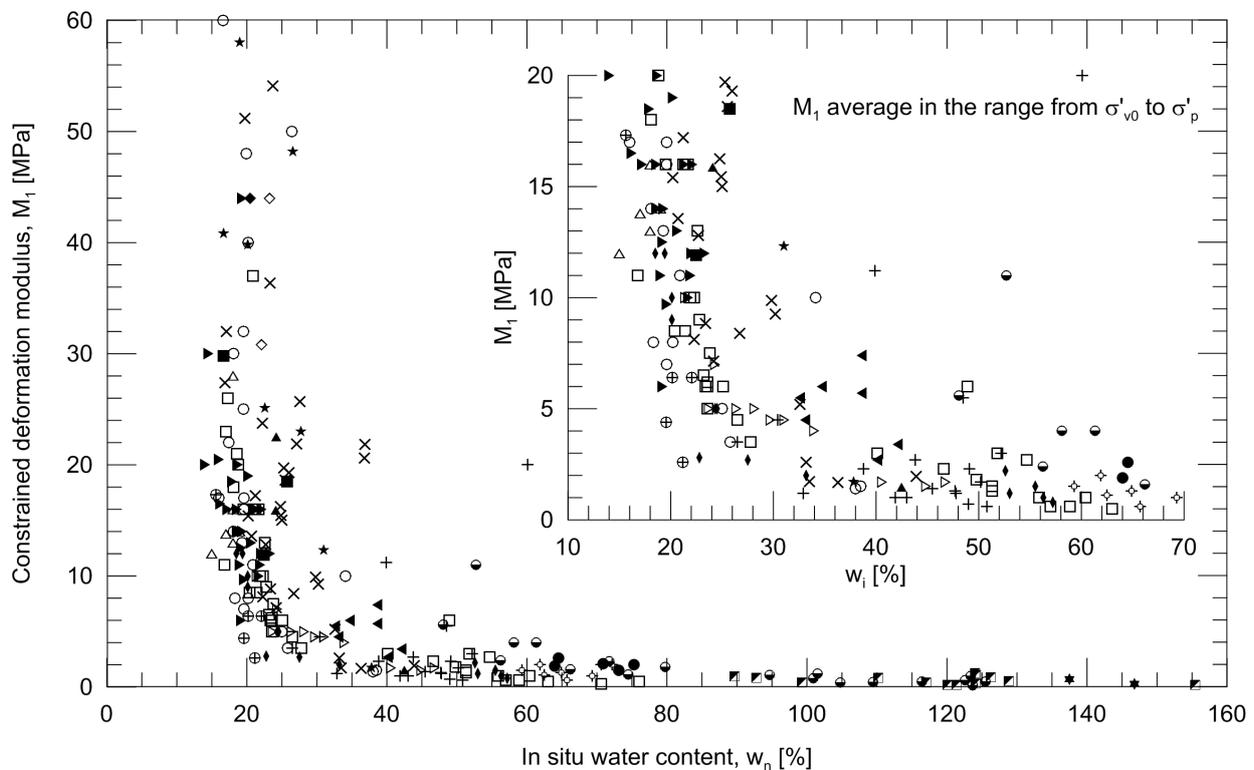


Figure 9. Constrained deformation modulus M_1 versus water content.

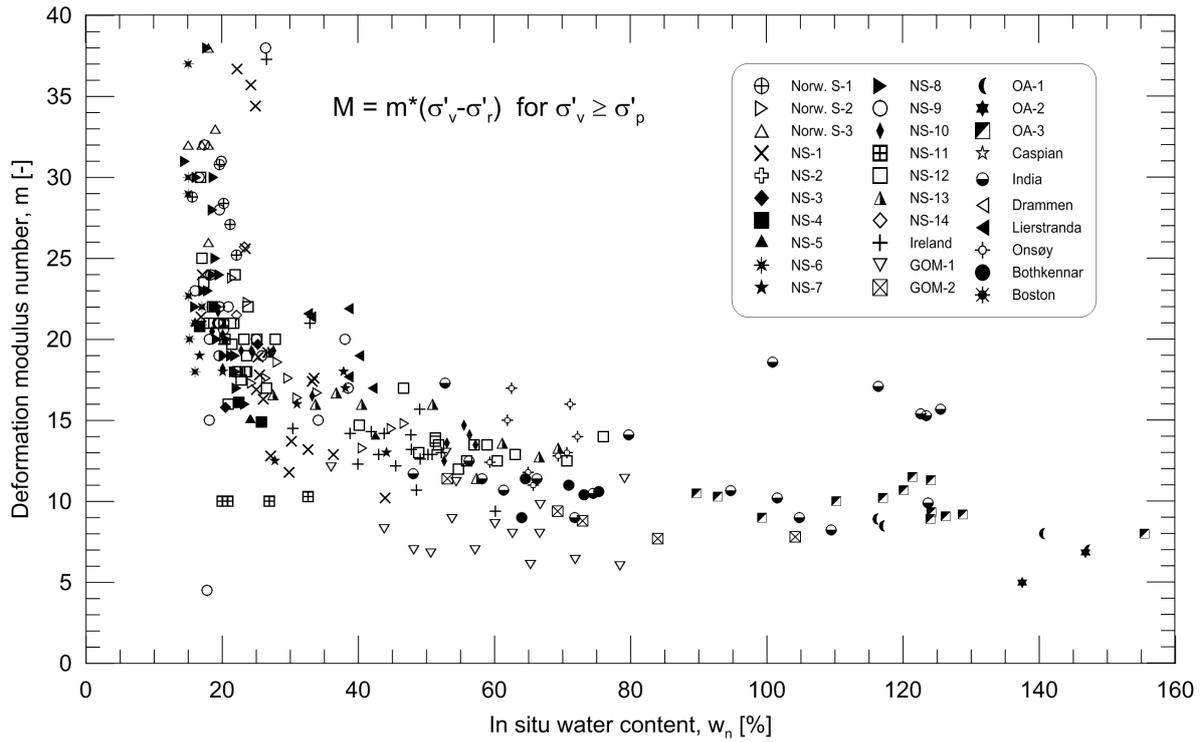


Figure 10. Deformation modulus number “m” (Eq 13) versus water content.

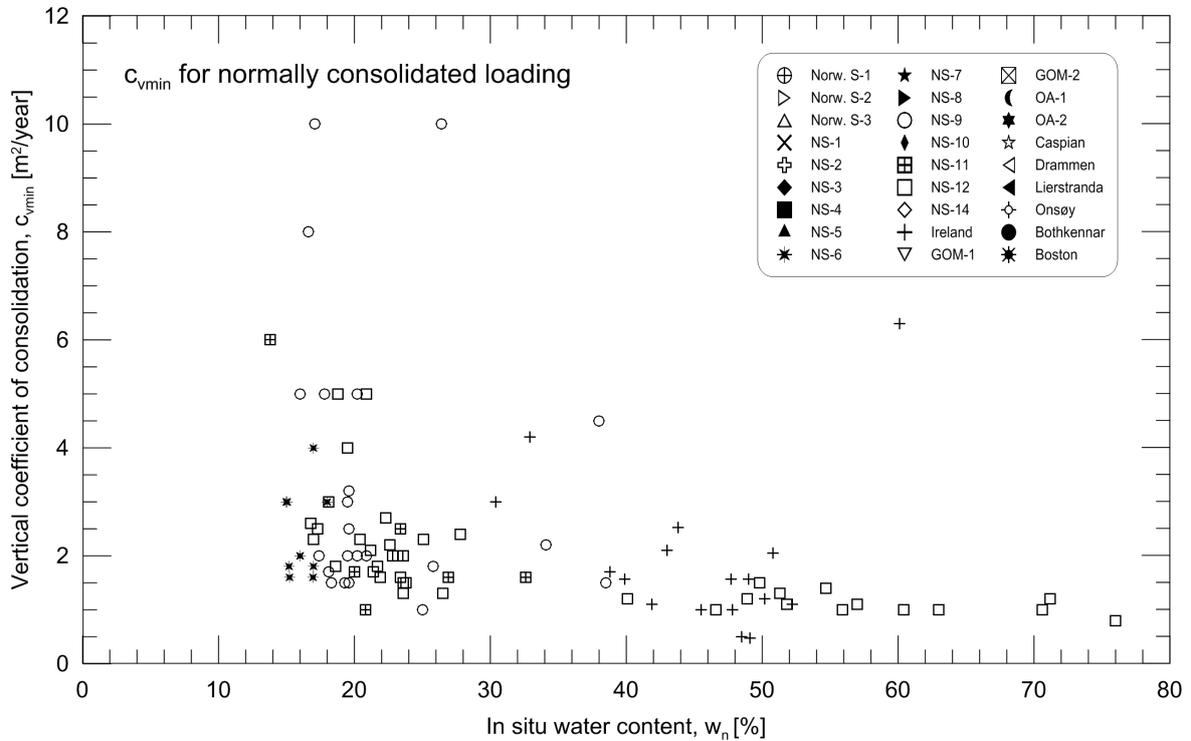


Figure 11. Minimum vertical coefficient of consolidation $c_{v,min}$ during laboratory induced normally consolidated stress state versus water content.

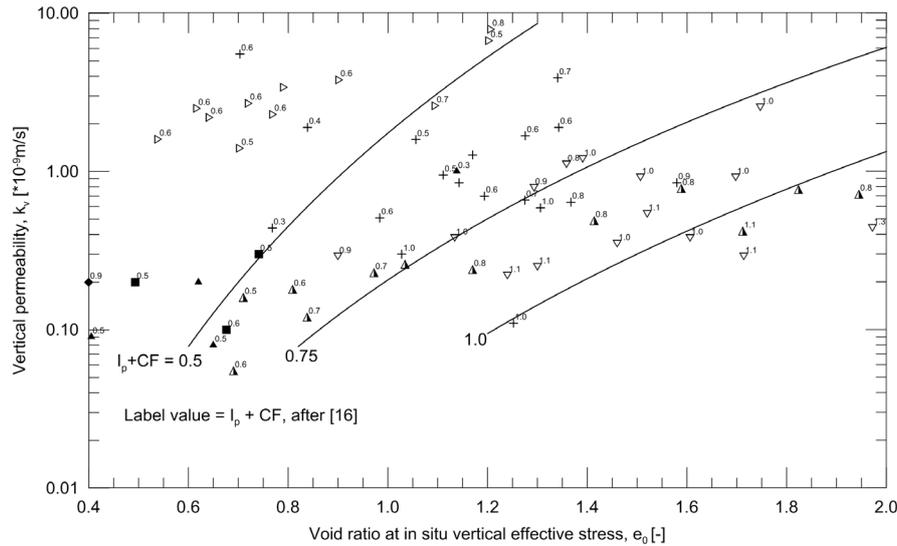


Figure 12. Vertical hydraulic conductivity versus void ratio at in situ vertical effective stress (excludes high void ratio clays from offshore Africa).

5.3. Intact undrained shear strength

The undrained shear strength and both measures of the normalized undrained shear strength s_u/σ'_{v0} and s_u/σ'_p did not correlate well with any of the index parameters as was generally expected. It was only for s_u versus I_L that a noticeable trend was evident but even this correlation had a large degree of variability especially for I_L less than 0.5. Figure 13 plots s_u/σ'_{v0} versus OCR for the TC, DSS, and TE modes of shear and in general show trends, albeit with a considerable amount of variability and low r^2 values. These Recompression tests can be represented by the SHANSEP equation [22]

$$s_u/\sigma'_{v0} = S(\text{OCR})^m \quad (14)$$

where $S = s_u/\sigma'_{v0}$ for $\text{OCR} = 1$ and $m = \text{increase in } s_u/\sigma'_{v0} \text{ with increase in OCR}$. For OCR less than 3 the data yield

$$s_{uC}/\sigma'_{v0} = 0.33(\text{OCR})^{0.71} \quad n = 97, r^2 = 0.43, \sigma = 0.18 \quad (15)$$

$$s_{uD}/\sigma'_{v0} = 0.24(\text{OCR})^{0.88} \quad n = 64, r^2 = 0.46, \sigma = 0.20 \quad (16)$$

$$s_{uE}/\sigma'_{v0} = 0.17(\text{OCR})^{1.04} \quad n = 32, r^2 = 0.41, \sigma = 0.25 \quad (17)$$

The OCR value for each s_u/σ'_{v0} data point was determined using the σ'_p value from a companion CRS or IL tests performed on the same sample. The different S values in Eqs 15 to 17 reflect the significant s_u anisotropy although the m values imply that the degree of anisotropy decreases with increasing OCR as m increases from TC to DSS to TE (for which a cap of 1.00 would normally be placed on the value of m in Eq 17).

Figure 14 plots the s_u/σ'_p data versus I_p and also includes the SHANSEP $\text{OCR} = 1$ s_u/σ'_{vc} data (where σ'_{vc} is the $\sigma'_v \gg \sigma'_p$ used in the laboratory to create an $\text{OCR} = 1$ state). The Figure 14 data

imply that while the S value in Eqs. 15 to 17 is statistically representative of the full data set whereas in fact S varies with I_p ; the least amount for TC and the greatest amount for TE. The data presented in Figures 13 and 14 for offshore clays are similar to that found for the mostly terrestrial cohesive soils presented by Ladd [22,37] for OCR = 1 SHANSEP data and Terzaghi et al. [7] for both SHANSEP and Recompression (s_u/σ'_p) data. The scatter in the data for each mode of shear is greater than that of Ladd [22] and Ladd and DeGroot [37] but their data were all OCR = 1 SHANSEP tests and as such would inherently have less scatter because of potential uncertainty in σ'_p for the Recompression data since the s_u and σ'_p were determined from two different test specimens. The scatter in Figure 14 is similar to that for the s_u/σ'_p data presented in Terzaghi et al. [7]. Linear regression to the data results in

$$s_{uC}/\sigma'_p = 0.27 + 0.00043I_p \quad n = 99, r^2 = 0.02, \sigma = 0.053 \quad (18)$$

$$s_{uD}/\sigma'_p = 0.22 + 0.00065I_p \quad n = 88, r^2 = 0.04, \sigma = 0.046 \quad (19)$$

$$s_{uE}/\sigma'_p = 0.15 + 0.0011I_p \quad n = 38, r^2 = 0.08, \sigma = 0.049 \quad (20)$$

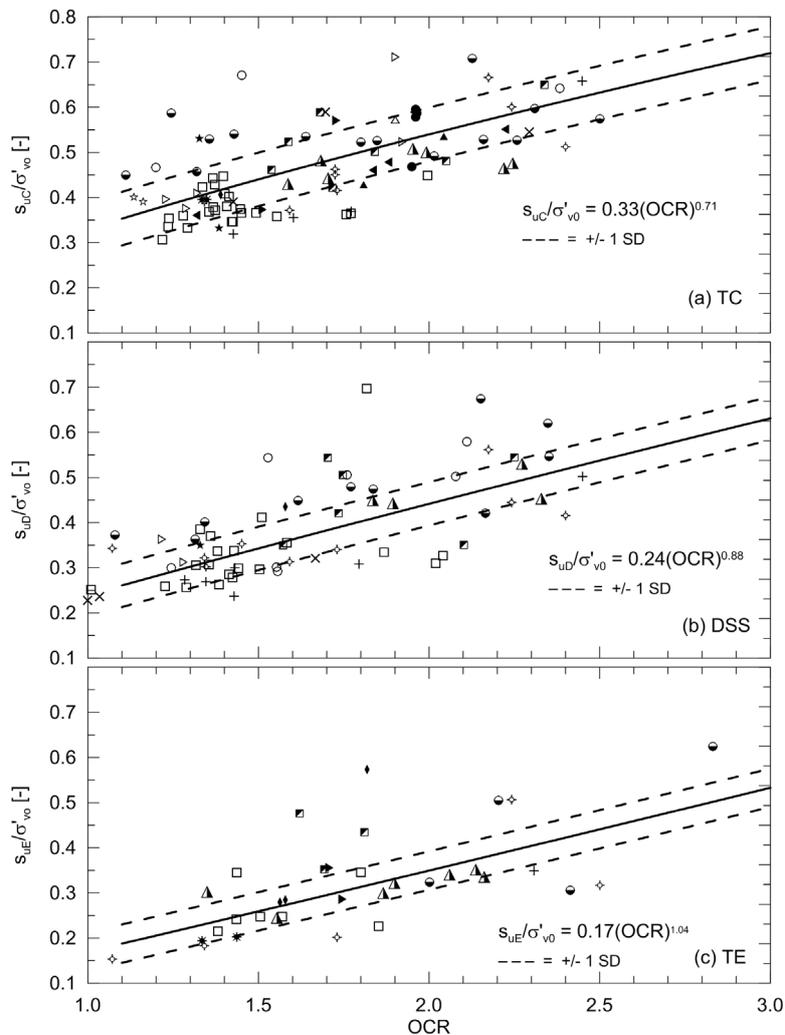


Figure 13. Normalised undrained shear strength s_u/σ'_{v0} versus overconsolidation ratio for a) TC, b) DSS and c) TE modes of shear. Data from Recompression tests only.

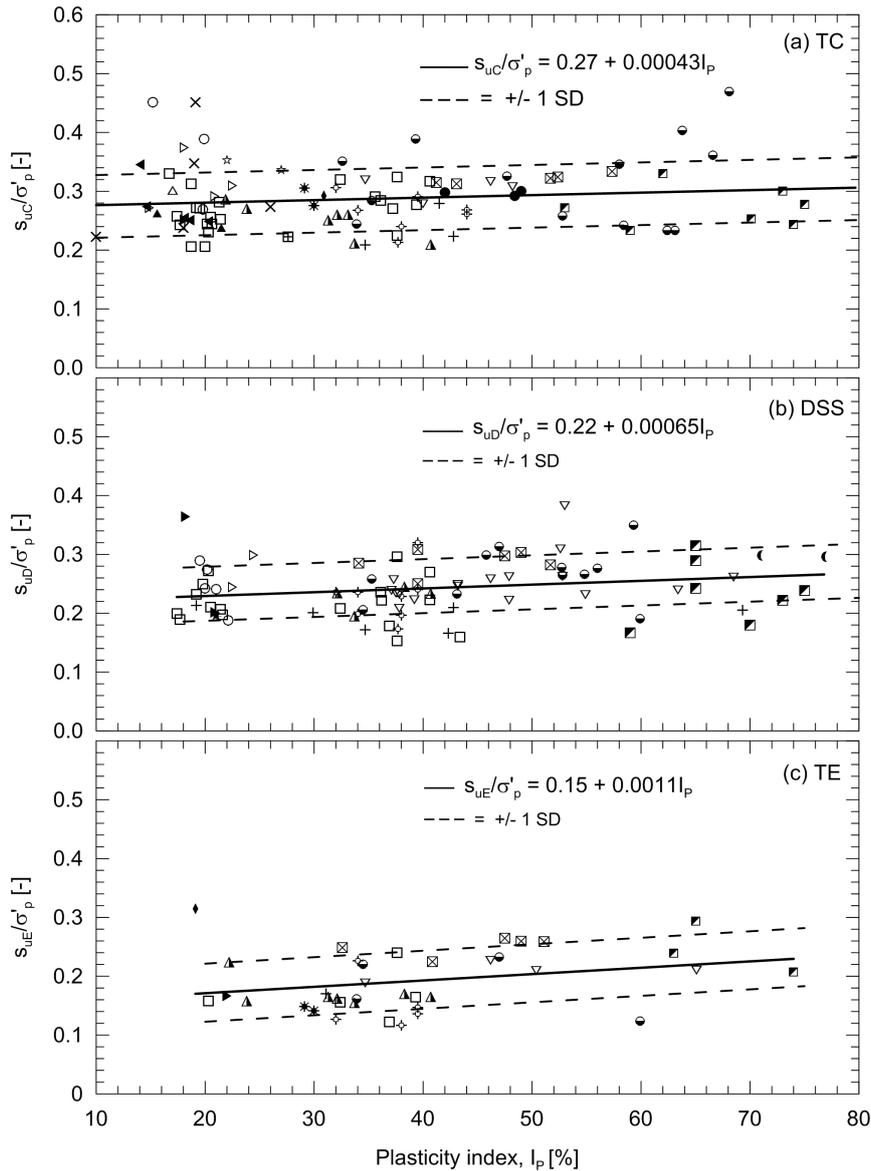


Figure 14. Normalised undrained shear strength s_u/σ'_p versus plasticity index for a) TC, b) DSS and c) TE modes of shear (includes both Recompression and SHANSEP test data with $\sigma'_p = \sigma'_{vc}$).

The anisotropy ratios K_s implied by the data in Figure 14 for $I_p = 20$ to 80% are

$$K_s = s_{uD}/s_{uC} = 0.82 + 0.00095I_p \quad (21)$$

$$K_s = s_{uE}/s_{uC} = 0.56 + 0.0027I_p \quad (22)$$

which reflect the decrease in s_u anisotropy with increasing I_p . The average undrained shear strength

$$s_{u,ave} = 1/3[s_{uC} + s_{uD} + s_{uE}] \quad (23)$$

implied by the data in Figure 14 gives

$$s_{u,ave}/\sigma'_p = 0.21 + 0.00073I_p \quad (24)$$

which reflects the progressive decrease in s_u anisotropy with increasing I_p and furthermore is near identical to that for s_{uD} (Eq 19). If the rate reduction factor proposed by Mesri [24] to obtain $s_{u,mob}$ for stability problems from $s_{u,ave}$ is applied to the data in Figure 14 then the resultant gives $s_{u,mob}$ equal to $0.22\sigma'_p$ independent of I_p as per Mesri [23,24] and Terzaghi et al. [7].

If it is assumed that $m = 1$ in Eq 14 then this implies that s_u is simply directly proportional to σ'_p as suggested by Terzaghi et al. [7] for an individual site or soil unit. Considering all the data together, and not on a site by site basis, Figure 15 plots the Figure 13 data in terms of s_u versus σ'_p and linear regression set through the origin to these data results in

$$s_{uC} = 0.28\sigma'_p \quad n = 101, r^2 = 0.96, \sigma = 18.7 \quad (25)$$

$$s_{uD} = 0.22\sigma'_p \quad n = 74, r^2 = 0.98, \sigma = 13.3 \quad (26)$$

$$s_{uE} = 0.18\sigma'_p \quad n = 34, r^2 = 0.96, \sigma = 7.2 \quad (27)$$

with $s_{u,ave} = 0.23\sigma'_p$ and the corresponding anisotropy ratios

$$K_s = s_{uD}/s_{uC} = 0.79 \quad (28)$$

$$K_s = s_{uE}/s_{uC} = 0.64 \quad (29)$$

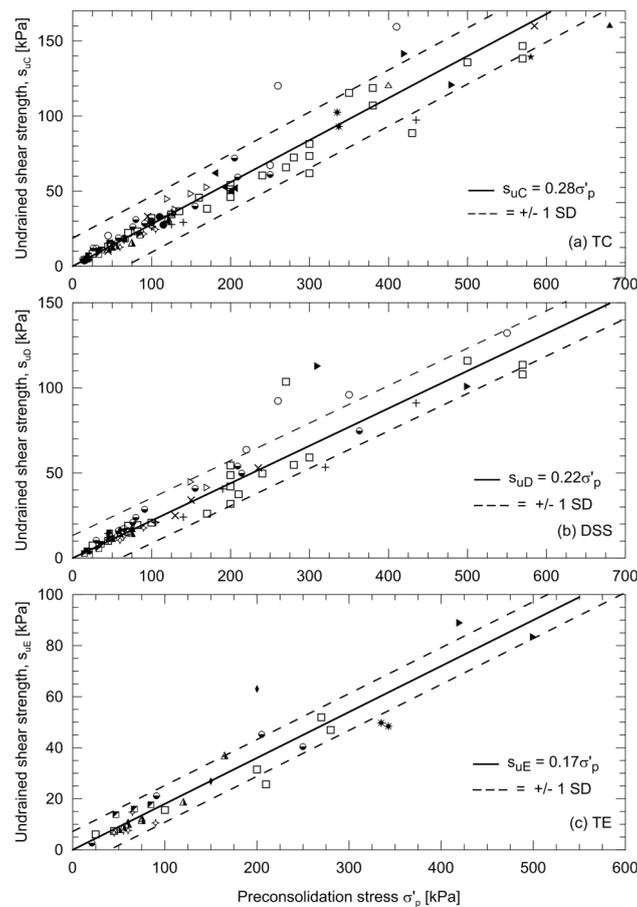


Figure 15. Undrained shear strength versus preconsolidation stress for a) TC, b) DSS and c) TE modes of shear. Data from Recompression tests only.

The TE and DSS coefficients in Eqs 26 and 27 are as expected close to the S values in Eqs 16 and 17 since the m values in these latter two equations are close to 1 whereas for TC with a m value of 0.71 the Eq 25 coefficient of 0.28 is much smaller than the Eq 15 S value of 0.33.

For all of the s_u equations presented above there was little practical difference among the correlations based on separation by different Burland Groups and by separation between the full and shallow (< 35 m) databases.

At NGI, the index undrained shear strength is typically measured using the fall cone (FC) on an unconfined specimen. Figure 16 presents s_u as measured in the CAUC test versus that measured with the fall cone on the same sample for which a linear regression set through the origin gives

$$s_{uC} = 1.15s_{uFC} \quad n = 97, r^2 = 0.94, \sigma = 22.8 \quad (30)$$

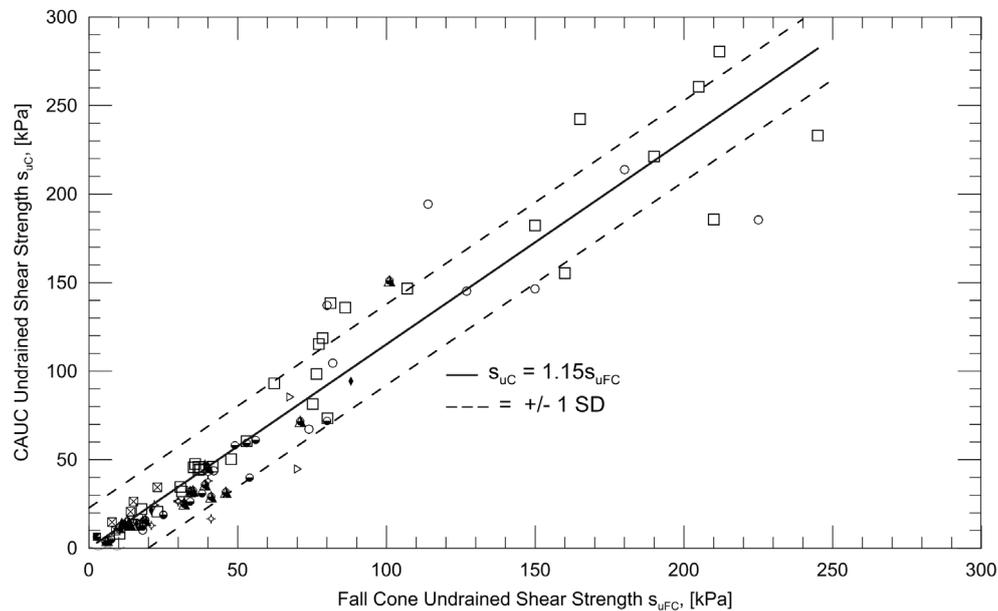


Figure 16. Undrained shear strength as measured in Recompression CAUC test versus fall cone (note: fall cone results were converted to s_{uFC} using NS8015, Norsk Standard 1988).

5.4. Remoulded undrained shear strength

The S_t data correlates with I_L as per Bjerrum [19] but with significant scatter which could be in part due to sample quality issues for the measure of s_u . Furthermore, the sensitivity values for the Gulf of Mexico and Offshore Africa sites are lower than historical experience suggests. For these soft, low OCR clays the index test measure of s_u used for the sensitivity test was likely too low due to sample disturbance. A regression fit to the data from all the other sites plotted in Figure 17 with the intercept forced to give $S_t = 1$ for $LI = 0$ gives

$$S_t = 10^{(0.65I_L)} = \exp(1.50I_L) \quad n = 166, r^2 = 0.27, \sigma = 1.86 \quad (31)$$

for I_L between 0 to 1.5. This correlation gives less increase in S_t with increasing I_L than that proposed by Bjerrum [19] which is likely due in part to the fact that Bjerrum's data set included many sensitive

soils with high to very high S_t values ($S_t > 20$). Alternatively s_{ur} correlates well with I_L as shown in Figure 18 and for which the data can be represented by

$$s_{ur} = 4.5(I_L)^{-1.5} \quad n = 229, r^2 = 0.73, \sigma = 0.55 \quad (32)$$

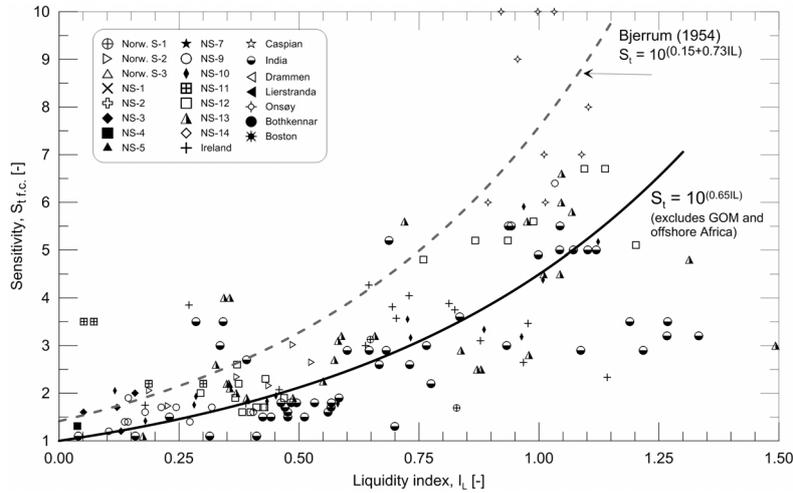


Figure 17. Sensitivity as measured using the Fall cone apparatus versus liquidity index (note: fall cone results were converted to s_{uFC} using NS8015, 44).

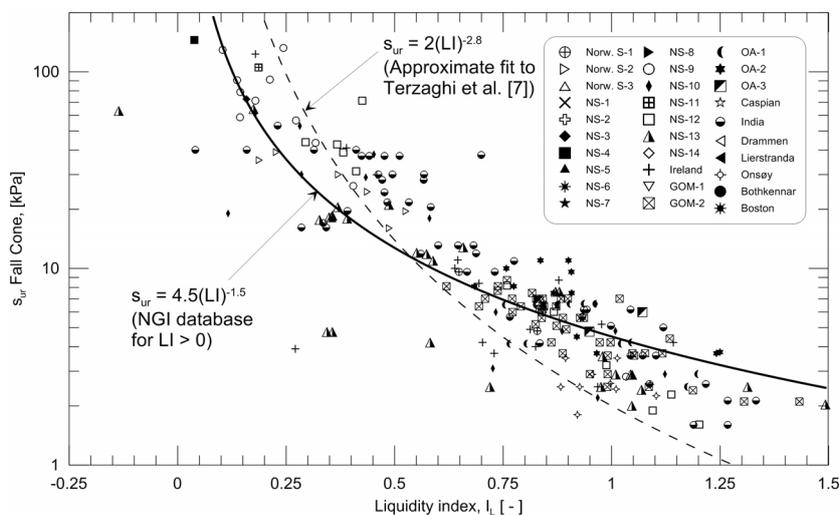


Figure 18. Remoulded undrained shear strength as measured using the fall cone apparatus versus liquidity index (note: fall cone results were converted to s_{uFC} using NS8015, 44).

6. Recommendations for use of correlations in practice

The Authors are strong advocates of performing in situ testing, high quality soil sampling and advanced laboratory testing as part of competent and reliable site characterisation programs for clays (e.g., [2,3,37,57,58]). The data and correlations presented here are primarily intended for

preliminary design considerations, comparing and evaluating results obtained from new offshore investigations, and for small, low risk projects with limited site characterisation budgets. This section gives recommendations for use of the data and correlations in practice in consideration of three levels of available information on soil properties from laboratory and in situ tests which include:

Level 1: For cases where only information from index tests such as water content and Atterberg Limits are available.

Level 2: Same information as Level 1 with additional data from index s_u tests such as fall cone and in situ tests such as piezocone (CPTU) or field vane (FV).

Level 3: Same information as Level 2 with additional data from advanced laboratory tests (e.g., oedometer, DSS, triaxial) conducted on undisturbed samples.

Some offshore soil investigations often combine in situ testing such as CPTU and soil sampling during the same campaign. Furthermore, preliminary tests are at times conducted using an offshore laboratory. This is especially the case for many offshore energy projects and in such circumstances Levels 1 and 2 would be combined.

Several limitations must be considered when using the correlations including:

1. The correlations are for inorganic clays that plot above the A-line in a Casagrande Plasticity Chart, i.e., either CL or CH soils in the USCS.
2. Almost all of the soil units used to develop the correlations are low to medium OCR soils ($OCR < 3$). The correlations are not applicable to higher OCR soils that result from such mechanisms as significant mechanical unloading, desiccation, etc. Such soils typically plot below to well below the ISL or ICL (Figure 1). Low values of $I_L < -0.1$ or $\Delta e^*/e_L < -0.1$ at shallow depths below seabed are also indicators of these soils.
3. The correlations are not intended for soils with significant cementation and quick clays. Such soils typically plot well above the ISL or SCL (Figure 1) with high values of I_L or $\Delta e^*/e_L$.
4. The recommended correlation equations were developed based on the data points presented in the figures and should not be used beyond the range of that data.

6.1. Preconsolidation stress and OCR

All important aspects of clay behavior are influenced by stress history (σ'_{v0} , σ'_p , OCR) and as such knowledge of σ'_p is arguably the most important parameter for assessing a suite of design parameters for a project. There are a variety of mechanisms that cause an initially normally consolidated soil to become overconsolidated including mechanical stress changes, pore water pressure changes, desiccation, drained creep, and physical chemical phenomena [59]. Very few natural clay deposits are truly normally consolidated, unless either recently loaded by fill or pumping if on land or by recent deposition if located under water. It is also common for two or more mechanisms to have occurred during the geologic history of a deposit. These factors highlight the importance of developing an understanding of the geologic history of a site so that stress history data from in situ and laboratory testing can be properly evaluated and understood [37].

Unfortunately no universal reliable correlation for estimating the preconsolidation stress was found directly with any of the traditional index parameters investigated. The following presents an hierarchy of options for estimating σ'_p in site characterisation practice. It is presented starting from

the basic correlations developed in this work and in increasing order of preference which is also generally in increasing order of reliability but as well as cost and time. In all cases knowledge of site geologic stress history if available should guide the decision process.

1. Empirical correlations between σ'_p and index parameters: with no laboratory 1-D consolidation tests or CPTU/FV results use the universal I_L (Eq 9, Figure 6) or void ratio sensitivity (Eqs 10 and 11, Figure 7) correlations presented in Section 5.1. As noted above, consideration of site geology and past experience are especially important in assessing the validity of any such empirically based estimate of σ'_p . This is part of a Level 1 approach. Assuming an OCR (either 1 or some other value deemed appropriate) would also be part of a Level 1 approach.
2. Universal CPTU/FV Correlations: conduct in situ tests such as either the CPTU or the FV (with the CPTU being the default standard of practice offshore) and use common universal CPTU or FV correlations to estimate σ'_p profiles. This is part of a Level 2 approach.
3. Soil Type/Region Specific Correlations: from past projects develop soil type or region specific correlations between data from laboratory 1-D consolidation tests performed on good quality undisturbed samples and parallel in situ tests such as either the CPTU or FV. Once such correlations are established future projects involving the specific soil type or region can rely on data from the in situ tests to estimate σ'_p profiles. This represents part of a Level 2 approach but with use of soil type or region specific correlations.
4. Site Specific Integrated Characterisation Program: combine in situ testing with collection of good-quality undisturbed samples and perform laboratory 1-D consolidation tests (e.g. CRS). Evaluate sample quality using the $\Delta e/e_0$ at σ'_{v0} method of Lunne et al. [55]. Develop site/soil specific correlations between the laboratory CRS and CPTU/FV results to determine recommended σ'_p profiles. This represents part of a Level 3 approach.

6.2. Level 1

In this level, the only information available is that from index tests such as water content and Atterberg Limits. Again, it is important to iterate that data from these correlations should only be used for feasibility studies and preliminary analyses. Final design should always involve in situ testing and laboratory stress-strain-strength tests on undisturbed samples.

It is important to note that while some of the correlations recommended here are good, others are not as good and in some cases have significant scatter. These weaker correlations are nevertheless presented because nothing better was found in the study. They should at least provide a guide for selection of parameters in the absence of any other information. Users of the correlations are encouraged to always inspect the scatter associated with any estimate as opposed to blindly using the best-fit equations provided. Given project considerations, appropriate decisions can be made as to whether to select values along the mean trend of the correlations or nearer to the lower or upper bounds, or a range.

There are three parameters that must be known in order to use the recommended Level 1 correlations including

- (1) in situ water content (w_n),

- (2) Atterberg Limits (w_L and w_P),
 (3) in situ pore pressure conditions (u_0).

Table 2 presents the recommended correlations for three categories of soil properties together with the Authors' rating of each correlation. As noted in Section 5, in some cases the correlations are based on basic index properties only whereas others are directly linked to another design soil property (e.g., s_u as function of σ'_p).

Table 2. Summary of correlations investigated.

Category	Soil Property	Index Parameter	Other Parameter	Figure/Equation	Correlation Rating
In Situ Stress State	γ_t	w_n	G_s	Eq 33	excellent
	γ_t	w_n	-	Figure 5, Eq 8	excellent
	σ'_p	I_L	-	Figure 6, Eq 9	fair, significant scatter at low I_L
	σ'_p	w_n, w_L $\sigma^*_{ve}, \Delta e^*/e_L$	G_s, σ'_{v0}	Figure 7, Eq 10 or Eq 11 (for $z < 35m$ & around SCL)	good to very good
	K_0	-	σ'_p, σ'_{v0}	Figure 8, Eq 12	good
Consolidation-Permeability	M_1	w_n	-	Figure 9	good, large scatter at $w_n < 30\%$
	m	w_n	-	Figure 10	very good, large scatter at $w_n < 30\%$
	c_{vmin}	w_n	-	Figure 11	fair, large scatter for $w_n < 30\%$
	c_v	-	c_{vmin}	-	typically 5 to 10 times $c_{v,min}$
	k_{v0}	e_0	-	Figure 12	can refine with CF and I_p
Shear strength	s_{uC}, s_{uD}, s_{uE}	-	σ'_{v0}, σ'_p	Figure 13, Eqs 15, 16 and 17	good, $m < 1$
	s_{uC}, s_{uD}, s_{uE}	-	σ'_p	Figure 15, Eqs 25, 26 and 27	good, assumes $m = 1$
	s_{uC}, s_{uD}, s_{uE}	I_p	σ'_p	Figure 14, Eqs 18, 19 and 20	good, $I_p = 20$ to 80%
	$s_{u,ave}$	I_p	σ'_p	Eq 24	good, $I_p = 20$ to 80%
	$s_{u,mob}$	-	σ'_p	$0.22\sigma'_p$	good
	$K_s = s_{uD}/s_{uC}$	I_p	-	Eqs 21 and 22 or simply s_{uD}/s_{uC}	very good
	$K_s = s_{uE}/s_{uC}$			$= 0.79$ and $s_{uE}/s_{uC} = 0.64$	for $I_p = 20$ to 80%
				independent of I_p	
	s_{uC}	-	s_{uFC}	Figure 16, Eq 30	very good
S_t	I_L	-	Figure 17, Eq 31	good	
s_{ur}	I_L	-	Figure 18, Eq 32	very good	

6.2.1. In situ state

Computation of the in situ vertical effective stress (σ'_{v0}) requires estimates of the in situ equilibrium pore water pressure (u_0) and the total unit weight to compute the in situ vertical total stress (σ_{v0}). The total unit weight can be estimated using the in situ water content w_n , an assumed value for the specific gravity and assuming 100% saturation as

$$\gamma_t = (1 + w_n/100)(G_s\gamma_w)/(1 + w_nG_s/100) \quad (33)$$

where γ_w is the unit weight of water. Alternatively the total unit weight can be directly estimated using w_n without assuming a G_s value as presented in Figure 5 and Eq 8. For most, but not all sites, it is reasonable to assume hydrostatic pore pressure conditions, as is the case of most of the sites in the databases. However, regions that have undergone recent deposition, submarine slides, or other mechanisms may have pore pressures different than hydrostatic. In cases where excess pore pressures are suspected the correlations recommended herein should be used with caution.

Level 1 estimates of σ'_p must rely on the empirical correlations between σ'_p and I_L or $\Delta e^*/e_L$ (Figures 6 and 7, Eqs 9 to 11) with the latter correlation considered more reliable. The σ'_p estimates and corresponding OCR values should be evaluated relative to any knowledge of site geology as discussed in Section 6.1. Estimates of K_0 can be made using Figure 8 and Eq 12 and should be evaluated in comparison to that using the Brooker and Ireland [39] K_0 - I_p -OCR correlation.

6.2.2. Consolidation-permeability

No empirical equations are created for any of the consolidation-permeability parameters given the relatively large degree of scatter, especially at low water contents. Estimates of M_1 , m , $c_{v,min}$ and k_v can be made using Figures 9 to 12 and in consideration of the scatter in the figures.

6.2.3. Shear-strength

All the equations for estimating s_u presented in Section 5 require an estimate of σ'_p . The recommended correlations are the relationships between s_u/σ'_p and I_p given in Eqs 18 to 20 and plotted in Figure 14 as they best capture changes in s_u anisotropy with increasing I_p . If a project only requires $s_{u,ave}$ then Eq 24 as function of I_p is recommended or if only $s_{u,mob}$ is required (e.g., stability problems) then simply use $0.22\sigma'_p$ independent of I_p . Overall the differences in using Eqs 18 to 20 versus the $s_u/\sigma'_{v0} = f(\text{OCR})$ (Eqs 15 to 17) or $s_u/\sigma'_p = \text{constant}$ (Eqs 25 to 27) equations can be as much as approximately 25% depending on mode of shear and specific combinations of OCR and I_p with the greatest differences being for high I_p (particularly for TE mode of shear which has the greatest increase in s_u/σ'_p with an increase in I_p) and high OCR (e.g., ≈ 4) particularly for TC which has the lowest m value in Eqs 15 to 17). Accordingly it is recommended that estimates of s_u no matter which set of the correlations are used should be evaluated by comparing the estimates with the database values in Figures 13 to 15 before making final selections. Final estimates can be based on either the average from the correlations, or a range of upper and lower bound values, or for conservative estimates just using the lower bound values only when this is conservative (e.g.,

stability problems). Alternatively, when it is conservative to use the upper bound (e.g., skirt penetration, pile drivability) this should be considered.

Figure 17 and Eq 31 can be used to estimate the sensitivity based on I_L . An alternative approach is estimate s_{ur} using Figure 18 and Eq 32. If a reliable estimate of the s_u is known then the sensitivity is computed as

$$S_t = s_u/s_{ur} \quad (34)$$

6.3. Level 2

Level 2 is when additional information is available from laboratory tests such total unit weight and index strength tests (e.g., Fall Cone) and also in situ tests (e.g., CPTU, and field vane). Interpretation of soil properties from in situ tests should be used to evaluate and update estimates made using Level 1 correlations. Details on use of in situ tests for soil property estimates are given elsewhere (e.g., [3,9,37,57,60,61]).

Measurements from the index strength tests can be used to supplement s_u estimates made using the Level 1 correlations. If fall cone data are available, Figure 16 and Eq 30 can be used to estimate s_{uC} or simply a conservative estimate of s_{uC} can be made assuming it is equal to s_{uFC} . Updated s_{uD} and s_{uE} estimates can be made using the updated s_{uC} and the anisotropy ratios of Eqs 21 and 22. Updated s_{uC} estimates based on index strength tests and in situ data can be used to evaluate the original σ'_p estimates from Level 1. Final σ'_p and s_u estimates should be made based on a collective evaluation of the Level 1 and Level 2 data. Again, as noted for Level 1, engineers can then choose to report final estimates based on an overall assessment of the estimates in consideration of the design problem being evaluated.

6.4. Level 3

In Level 3, advanced laboratory tests (CRS, triaxial, DSS) are conducted on undisturbed samples. The quality of the laboratory test samples should be evaluated using the $\Delta e/e_0$ sample quality criteria [55] which ranks samples in one of four categories: 1) very good to excellent, good to fair, poor, and very poor. If the samples tested and evaluated can be classified to be in the very good to excellent or good to fair classes, the results of the advanced laboratory tests should be used as the major input to the recommended soil design parameters. The correlation plots presented herein and Level 1 and 2 estimates can be used to evaluate the soundness of data measured in Level 3 and for comparison with past experience. For test programs with limited data from advanced tests, the correlation plots can be used as a guide for interpretation of the measured data. They are also useful for estimating appropriate consolidation stress conditions for advanced laboratory tests.

If the samples tested and evaluated are classified to be in the 'Very poor' to 'Poor' categories, relatively more reliance should be placed on the soil parameters derived from the index correlations. Additionally or alternatively SHANSEP type tests can be conducted to derive the S and m parameters in Eqs 15 to 17 for each relevant mode of shear. Furthermore the SHANSEP data together with results from CPTU or FV tests can be used to check the consistency of undrained shear strength and stress history estimates as described for example in DeGroot [62] and Yang et al. [63] for the

CPTU and Chandler [64] and Ladd and DeGroot [37] for the FV. Details on recommended procedures for conduct of SHANSEP tests and interpretation of results are given in Ladd and DeGroot [37] and DeGroot and Ladd [58].

7. Conclusions

The paper presents results from an analysis of a comprehensive database consisting of high quality advanced laboratory and soil index test results. The main database includes 25 offshore and 5 onshore sites from around the world. The database was used to develop correlations between index parameters and soil design parameters. Most of the correlations only require information from natural water content and Atterberg Limits tests. A framework based on intrinsic soil parameters was found to be useful in developing some of the correlations. Intrinsic properties can be derived from oedometer tests on remoulded soil specimens or from empirical correlations with the liquid limit. Useful correlations were found between the intrinsic parameter, void ratio sensitivity, and the preconsolidation stress (σ'_p). Correlations were also developed between σ'_p (or OCR) and K_0 and between σ'_p and s_u that accounts for undrained shear strength anisotropy. Good correlations were found between the liquidity index and the sensitivity and the remoulded undrained shear strength. Good trends were found between settlement parameters and water content but with significant scatter at low water contents. The correlations are limited to soils that plot above the A-line in a Casagrande plasticity chart and that are of low to medium overconsolidation (< 4). They are not intended for soils that have high overconsolidation from such mechanisms as mechanical unloading or desiccation. Nor are they intended for use with soils that have significant cementation bonding. Recommendations are provided on how to use the correlations in practice in consideration of three levels of available site investigation data.

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Conflict of interest

All authors declare no conflicts of interest in this paper.

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