DSHANSEP: Alternative considerations of the SHANSEP approach

A B Tsegaye¹,², A Gylland², S A Degago³, J S Gloppestad², A Montafia²

¹Norwegian Geotechnical Institute, Trondheim, Norway
²Multiconsult Norge AS, Trondheim, Norway
³Norwegian Public Roads Administration (NPRA), Trondheim, Norway

Abstract. Since it was first proposed by Ladd and Foott in 1974, the SHANSEP approach has become a quite useful and convenient approach for the evaluation of the undrained shear strength of clay soils in-situ. SHANSEP is commonly practiced with determination of its parameters using various correlations based on consistency limits. While seemingly handy, such correlations hide some details of the behaviour of clayey soils that are useful for engineering judgement. In this article, an alternative way of looking at the SHANSEP method is presented by examining an integrated framework for strength behaviour of clays. This resulted in a new approach called DSHANSEP, where the D represents the so-called “dilatancy parameter” which measures changes in pore pressure due to changes in shear stresses during undrained loading. The resulting association with dilatancy is logical as this parameter reflects failure mechanisms of the clays which is integral aspect of the undrained shear strength of clays. The implication of this relation is explored by looking at its dependence on the overconsolidation ratio, the mobilized stress ratio and sample quality. Furthermore, a formalized system of classification for clays based on the dilatancy parameter and their overconsolidation ratio is proposed and applied for Norwegian clays along with suggested parameters. Use of DSHANSEP and the proposed classifications are in line with the safety philosophy adopted for stability of slopes involving soft clays in Norway.

1. Introduction

The short-term stability and the bearing capacity of clay soil deposits depend on their undrained shear strength. The search for a simple and capable approach for the determination of undrained shear strength has therefore been a continuous interest among geotechnical engineers.

One of the approaches that has been commonly used in the state of the practice is the SHANSEP approach. SHANSEP is a theory first suggested by Ladd and Foott [1] where the name is acronym for Stress History and Normalized Soil Engineering Properties. SHANSEP is based on observation of laboratory tests and the concept is built on the premise that the normalized undrained shear strength of clay relates uniquely to their overconsolidation ratio (OCR). The approach consists of establishing normalised undrained shear strength as a function of OCR from a series of laboratory tests. This relationship is then used to estimate the in-situ undrained shear strength after evaluating the in-situ vertical effective stress. With this proposal, Ladd and Foott [1] arrived at the conclusion that the undrained shear strength of clay soils can reasonably be estimated using the empirical relation
\[ c_u = s \sigma'_{vo}^{OCR} \]  \hspace{1cm} (1)

where \( s \) and \( \alpha \) are the SHANSEP parameters which are constant for a given material and test type; \( c_u \) is the undrained shear strength; \( \sigma'_{vo} \) is the in-situ vertical effective stress. The parameter \( s \) is also referred to as undrained shear strength ratio in normally consolidated state (OCR=1).

**Table 1.** SHANSEP parameter estimated for reconstituted Norwegian clays, based on data from [2]

| Parameters | HE | BE | LE |
|------------|----|----|----|
| \( \alpha \) | 0.568 | 0.568 | 0.568 |
| \( s \) | 0.45 | 0.33 | 0.24 |

*HE: = BE+1.645 standard deviation; LE: =BE-1.645 standard deviation

**Figure 1.** Plots of low, high and best estimate Normalized Active Undrained Shear Strength (NAUSS) curves together with data for Norwegian clays: Left: in a log-log space, Right: actual space. Data from Karlsrud and Hernandez-Martinez [2]

Ladd and Footh [1] suggested to establish the parameters \( s \) and \( \alpha \), for a specific soil type and given mode of shearing, based on curve fitting of measurements from a series of high quality laboratory tests. However, in absence of such a rigorous tests scheme, several correlations have been suggested in literature. For example, the parameter \( s \) has been set a constant value (e.g. Mesri [3]; Jamiolkowski et al. [4]; D’Ignazio et al [5]). It has also been correlated to various parameters such as plasticity index \( (I_p) \) (e.g Skempton [6]; Larsson [7]), liquid limit \( (w_L) \) (Hansbo [8]; Larsson et al. [9]), sensitivity (Chiang and Phoon [10]), friction angle or the stress ratio at the critical state, \( M_c \), (e.g., Mayne [11]; Wood [12]) and water content of the soil \( (w) \) (e.g., Karlsrud and Hernandez-Martinez [2]).

For Norwegian clays, experimental data from Karlsrud and Hernandez-Martinez [2] are digitised and plotted in log-log space as shown in Figure 1. Applying statistics in the log-log space, the high estimate (HE), the best estimate (BE) and the low estimate (LE) SHANSEP parameters are obtained and are given in Table 1. It is difficult to find other relevant parameters that may have a role in the scatter of the data. It is worthwhile to mention that Karlsrud and Hernandez-Martinez [2] interpreted SHANSEP parameters that are different from the ones given in Table 1.

The aim of the current work is to look into fundamentals of deformation behaviour of clay soils under undrained loading in a triaxial stress state and try to come up with ways to correlate normalised undrained shear strength with relevant variables and soil properties. In doing so, the work aims to bring upfront important elements that are hidden in the shadow of the SHANSEP approach.
2. Undrained strength-dilatancy relationship

This section lays down the basics of a new approach called DSHA NSEP, where SHANSEP is as abbreviated in Ladd and Foott [1] and $D$ is for dilatancy.

Consider shearing in a triaxial compression under undrained conditions. The change in pore pressure ($\Delta u$) in the shearing phase of the test is conveniently written as the sum of the contribution of isotropic compression stresses ($\Delta \sigma_m$) and contraction or dilatation due to the change in deviatoric stresses ($\Delta \sigma_d$), i.e.,

$$\Delta u = B(\Delta \sigma_m - D \Delta \sigma_d),$$

in which $D$ is called dilatancy parameter [13] that defines the pore pressure contribution from shear. We consider the sign convention in which $D > 0$ is dilative (suction pressure due to shear), $D < 0$ is contractive (pore pressure due to shear).

The dilatancy parameter $D$ can be measured from undrained triaxial compression tests. As long as our interest is specifying the ultimate strength, the nonlinearity of the value of $D$ should not have a significant discourse. We are therefore considering the secant value.

The parameter $B$ depends on the saturation level of the sample. We shall consider a fully saturated condition in which $B$ can assume a value of almost 1 in the following.

Further assumptions and considerations are illustrated with schematics for the strength-dilatancy relations as given in Figure 2. It is assumed that the ultimate undrained strength of clays in the effective confining stress-deviatoric stress space is defined by using a line with a slope $M$. This line is referred to as the $M$-line, where $M$ is the stress ratio (defined as deviatoric stress divided by mean effective stress). Assuming the ultimate strength of clays can fairly be estimated using the Mohr-Coulomb strength criterion, for a triaxial compression condition we have:

$$M = \frac{6 \sin \phi}{3 - \sin \phi},$$

where $\phi$ is the effective characteristic friction angle.

Let the shearing in a triaxial compression test under undrained condition begin from a mean effective stress $p'_0$. Let the dilatancy parameter $D_0$ be the slope of the line that goes from the initial effective stress projection on the isotropic axis to the point of intersection of the undrained effective stress path with the $M$-line. The line with the slope, $D_0$, is referred to as the $D_0$-line, Figure 2. The $D_0$-line and the $M$-line meet at a unique point (that is for unique $M$ and $D_0$ values).
Figure 2. Schematics for the strength-dilatancy relations.
This point can be analytically determined and in principle can be used to determine the undrained shear strength, $c_u^A$, as

$$c_u^A = \frac{1}{2} \frac{M}{1 - D_0 M} (p'_0 + a),$$

(4)

where $M$ is the stress ratio and $a$ is attraction and $D_0$ is the dilatancy parameter measured from the isotropic axis.

$D_0$ can be related with a dilatancy parameter defined from any other initial stress state. An explicit relation is given as follows. Let the dilatancy parameter measured from an intermediate $M_k$-line be $D_k$ (where the $M_k$ -line has the same vertex as the $M$-line but has a slope of $M_k$). For a triaxial compression condition, the transformation

$$D_0 = \frac{D_k}{M} \left( \frac{M - M_k}{1 - D_k M_k} \right)$$

(5)

holds assuming both $D_0$ and $D_k$ lead to the same undrained shear strength, where $M$ is the stress ratio as defined earlier and $M_k$ is the stress ratio at any other arbitrary stress state.

The in-situ effective stress state is then defined from weight considerations and the at-rest earth pressure coefficient. Bishop [14] defined that “coefficient of earth pressure at rest is the ratio of the lateral to the vertical effective stresses in a soil consolidated under the condition of the no lateral deformation, the stresses being principal stresses with no shear stress applied on planes which the stresses act.” i.e.,

$$K_0 = \sigma_{oh} / \sigma_{ov} ;$$

(6)

where $\sigma_{oh}$ is the at-rest lateral effective stress assumed equal in all lateral directions and $\sigma_{ov}$ is the at-rest vertical effective stress.

The vertical effective stresses in soils are estimated from the overlying weight. In spite of lack rigorosity in its derivation, Jaky’s [15] formulation of $K_0$ as

$$K_0 = 1 - \sin \varphi$$

(7)

is widely applied both for sand and normally consolidated clays. For clays, and assuming horizontal terrain, Jaky’s formulation has been extended to accommodate effect of overconsolidation ratio as [16]

$$K_{0,OC} = (1 - \sin \varphi) OCR^m ;$$

(8)

and it falls between the active stress ratio and the passive stress ratio. Various suggestion from literature for the parameter $m$ are presented in Table 2. L’ Heureux et al.[17] presented data for Norwegian clays showing dependency of $K_{0,OC}$ on the OCR in line with the formulation given in Eq.8.

By assuming that Eq.8 holds for both normally consolidated and over consolidated clays, the undrained shear strength may be given in terms of the initial effective vertical stress as:

$$c_u^A = \frac{1}{6} \frac{M f_i}{1 - MD_0} OCR^m (\sigma'_{v0} + \bar{a}), \quad D_0 < \frac{1}{M}$$

(9)

where $f_i = OCR^{-m} + 2 - 2 \sin \varphi$, $\sigma'_{v0}$ is the initial effective vertical stress, $\varphi$ is the characteristic friction angle, $OCR$ is the over consolidation ratio, $\bar{a} = 3a/(1 + 2K_0)$ is the modified attraction. If we collect the terms before OCR$^m$ into
\[ s_D = \frac{1}{6} \frac{M f_i}{1 - M D_0} \]  

Eq.10 gives a relation in the form similar to that of the SHANSEP formulation and is here referred to as DSHANSEP. In the DSHANSEP formulation, the \( s_D \) parameter, analogous to \( s \) in the SHANSEP, depends on the specified dilatancy parameter \( D_0 \), the overconsolidation ratio and the friction angle of the material. There also comes the additional term due to attraction in the DSHANSEP. Due also to the way Eq.9 is deduced, the parameter \( m \) should be the same parameter as that which is used for the OCR dependence of the \( K_0 \) value.

| \( m \) | Reference |
|--------|-----------|
| 0.4 - 0.5 | Schmertmann [18] |
| \( \sin \varphi \) | Mayne and Kulhawy [19] |
| 0.47 | L'Heureux et al. [17] (for Norwegian clays, with \( 1 - \sin \varphi = 0.43 \)) |

### 3. Quantitative aspects of the dilatancy parameter, \( D_0 \)
#### 3.1 Dependence on OCR
Looking at trends of the dilatancy parameter in the stable and metastable region of triaxial compression tests, \( D_0 \) may be estimated from the mathematical equation

\[ D_0 = a \left( \frac{M - M_k}{M_c - M_k} \right) \left( \frac{M}{M_c} - \frac{b}{OCR} \right) \leq 1 \]

where \( \langle \quad \rangle \) is the Macaulay bracket, \( a \) and \( b \) are fitting parameters, \( M_k \) is the stress ratio at the \( K_0 \) stress state and \( M_c \) is the critical state stress ratio. Considering the condition \( D_0 = 0 \), one finds \( M = \frac{b M_c}{OCR} \). The parameter \( b \) is therefore to be fixed for \( b = OCR M_{D=0}/M_c \). When \( M = \frac{b M_c}{OCR} \), the state is a dilatancy neutral state and the strength is the same as that predicted with an elastic perfectly plastic Mohr-Coulomb model. In other words, shear strain and volumetric strain are decoupled. When \( M > \frac{b M_c}{OCR} \) the dilatancy parameter is positive and the state is said to be a dilative state.

Let us investigate the Eq.11 by setting \( M = M_c \), i.e., let \( D_0 \) be measured where the undrained effective stress path first touches the \( M_c \)-line. Substituting \( M_c \) in terms of \( M \), we are led to

\[ D_0 = a \left( 1 - \frac{b}{OCR} \right) \leq \frac{1}{M_c} \]

(12)

This is similar to the relationship derived by Muir Wood [12] from the modified Cam Clay model, which is given as:

\[ D_0 = \frac{1}{M_c} \left( 1 - \left( \frac{r}{OCR} \right)^{\Lambda} \right) \]

(13)

where, \( \Lambda \) is a model parameter \( M_c \) is the stress ratio at the critical state; and

\[ r = e^{\frac{\nu - \Gamma}{\lambda - \kappa}} \]

(14)

in which \( \nu, \Gamma, \lambda \) and \( \kappa \) are model parameters. Accordingly, a state with \( \frac{OCR}{r} < 1 \) is contractive, \( \frac{OCR}{r} > 1 \) is dilative, while \( \frac{OCR}{r} = 1 \) is neutral.
From geometric considerations of the modified Cam clay yield function, one obtains $r = 2$. Therefore, $D_0 = 0$ is obtained for an $OCR = 2.0$. Wood [12] compared the results from Eq.13 with data from Bishop and Henkel [20] using $M = 0.95$, $\lambda = 0.091$, $k = 0.034$ and $\lambda = 0.63$ and found good agreement particularly at lower overconsolidation ratios. Additional data from literature that further justified the trend are added. Using the relationship in Eq.12 and setting $a = 0.7$ and $b = 1.9$, a reasonable fit is obtained, Figure 4. Note that the data from Amundsen [21] and Lunne et al. [22] are adjusted according to the transformation rule in Eq.5.

Considering Eq.12 into Eq.9, plots of normalized active undrained shear strength versus $OCR$ are produced for various values of $a$ and $b$. The plots are shown along with data for Norwegian clays, Figure 4. The trends capture well of that indicated by the data. It is also possible to produce exponential trends of the normalized undrained shear strength with $OCR$ for some other sets of $a$ and $b$ parameters. In that case, the trend will be non-SHANSEP (in the traditional sense that the exponent, $\alpha$, in the SHANSEP formulation falls below 1.)

**Figure 3.** Dilatancy normalized stress ratio relationships for various $OCR$ levels for $a=0.5$ and $b=2$, Eq.11.

**Figure 4.** a) $D_0 - OCR$ relationships, data for the validation of Eq.12, b) DSHANSEP predictions for different values of $a$ and $b$ in Eq.12. NAUSS= Normalized Active Undrained Shear Strength, Data from Karlsrud and Hernandez-Martinez [2].
3.2 Dependence on \( K_0 \)
The empirical relationship laid down in Eq.12 implies an obvious relationship between \( D_0 \) and \( K_0 \). Law and Holtz [23] collected data from literature and plotted Skempton’s pore pressure parameter against the coefficient of consolidation. The trend shows that the value of Skempton’s pore pressure parameter, \( A \), decreases with increasing \( K_0 \). The trend is reproduced for the dilatancy parameter \( D_0 \) in Figure 5. The original values are transformed according to Eq.5 assuming the pore pressure coefficients were originally calculated from the \( K_0 \). The trend shows that \( D_0 \) increases with increasing \( K_0 \) which is expected if the empirical relations in Eq. 8 and Eq.11 should hold at the same time. More data is needed on this to have a statistically significant amount.

![Figure 5. Trend of the dilatancy parameter with the at-rest stress ratio. The data are taken from Law and Holtz [23].](image)

3.3 Shear strain dependency of the dilatancy parameter
Higher shear strain mobilizations will lead either to a significant accumulation of pore pressure in the case of contractive states or to development of suction pressure in the case of dilative states and hence lower \( D_0 \) in the case of the former and higher \( D_0 \) in the case of the later. The strength contribution due to suction can also have significant uncertainty. In both cases, limiting the strain level while choosing characteristic undrained strength is wise. In many cases, a shear strain of 1-2% may be considered. From constitutive modelling point of view, the dependence of \( D_0 \) on the shear strain level can be considered through the dependence of the stress ratio \( M \) on the shear strain; a hardening rule as it is formally called.

3.4 Effect of sample disturbance and sampling method
One of the criteria that are used as a signature for sample disturbance is the change of void ratio, \( \Delta e_0 \), during the consolidation phase of triaxial compression tests normalized by the initial void ratio, \( e_0 \). Accordingly, samples are characterized as very good to excellent, good to fair, poor and very poor [22]. In Figure 6, various data from literature are shown in dilatancy parameter versus the normalized void ratio change (\( \Delta e_0 / e_0 \)) plots. The overall picture is a link between the dilatancy parameter and sample quality. However, when looking at data isolated for each sampling technique, there is no apparent trend in this relationship. As a result, a reliable correlation between the relative change in void ratio and the dilatancy parameter is not deduced here. It can be seen from the figure that sampling techniques that are known to incur less disturbance give less scatter in the dilatancy parameter. This might carry an implication that the dilatancy parameter itself may be a good indicator of the level of the sample disturbance and thus the quality of the sample. Lunne et al.[22] also stated that the dilatancy parameter is one of the signatures of sample disturbance on laboratory tests.
4. Classification system based on the dilatancy parameter and OCR

We will now proceed to establish a classification system for clays based on the ranges of the dilatancy parameter $D_0$ and the overconsolidation ratio such that qualitative descriptions can be more precisely quantified.

The bases of classifications using $D_0$ and using $OCR$ are presented in Table 3. A given sample can be described by a combination of these two. This system of classification yields a 4 x 6 matrix of specified properties, Table 3. For instance, NC-HC will be a normally consolidated and highly contractive clay soil. Accordingly, it has an $OCR = 1$ and a dilatancy parameter value less than -0.5.

For Norwegian clays, the values of $s_D$ for various ranges of $D_0$ and overconsolidation ratio are produced in

Table 4 and Figure 7. In the table, the mid $OCR$ values and a friction angle of 30 degrees are considered for calculating the $s_D$ values. The values increase/decrease for higher/lower values of friction angle. Most data are concentrated between MC and D across the ranges of $OCR$ between 1 and 6. Based on the data, the values in the shaded cells of

Table 4 are the most probable values of $s_D$ for the Norwegian clays under the various classifications. The states towards the left down corner and the right up corner in the table can be rare to find. It should also be noted that, the $s_D$ values calculated for the data from [2] in Figure 7 show a significant scatter. The scatter can be due to other factors and sample disturbances that have influence on the dilatancy parameter and possible differences in the friction angle (or differences on the determination of the undrained shear strength).

**Table 3. Classification based on ranges of $D_0$ and OCR (*Norwegian clays)**

| Descriptions↓ | Based on $OCR$→ | Normally consolidated (NC) | Lightly overconsolidated (LOC) | Moderately overconsolidated (MOC) | Heavily overconsolidated (HOC) |
|---------------|-----------------|-----------------------------|--------------------------------|----------------------------------|---------------------------------|
| Based on $D_0$ ↓ | Criteria | OCR=1 | $s_{D_0=0}$ ≥ $BE - s_D$ | $s_{D_0=0}$ < $s_{D_0=1/3}$ ≥ $HE - s_D$ | $s_{D_0=1/3}$ < $HE - s_D$ |
| Highly contractive (HC) | $D_0$~0.5 | NC-HC | LOC-HC | MOC-HC | HOC-HC |
| Medium contractive (MC) | $-0.5 ≤ D_0 < 0$ | NC-MC | LOC-MC | MOC-MC | HOC-MC |
| Contractive (C) | $-0.25 ≤ D_0 < 0$ | NC-C | LOC-C | MOC-C | HOC-C |
| Neutral (N) | $D_0 = 0$ | NC-N | LOC-N | MOC-N | HOC-N |
| Dilative (D) | $0 ≤ D_0 < 0.25$ | NC-D | LOC-D | MOC-D | HOC-D |
| Highly dilative (HD) | $0.25 ≤ D_0 < 0.5$ | NC-HD | LOC-HD | MOC-HD | HOC-HD |

Figure 6. Dilatancy parameter versus relative change of void ratio in the compression phase of undrained triaxial compression tests, classification based on $\Delta e/e_0$ [22]. Data from Amundsen [21].
Table 4. Ranges of sD for Norwegian clays

| Descriptions | NC     | LOC    | MOC     | HOC     |
|--------------|--------|--------|---------|---------|
| HC           | 0.18 - 0.25 | 0.16 - 0.22 | 0.13 - 0.18 | 0.12 - 0.17 |
| MC           | 0.25 - 0.3  | 0.22 - 0.28 | 0.18 - 0.22 | 0.17 - 0.21 |
| C            | 0.3 - 0.38  | 0.28 - 0.36 | 0.22 - 0.28 | 0.21 - 0.27 |
| N            | 0.38       | 0.36    | 0.28    | 0.27    |
| D            | 0.38 - 0.57 | 0.36 - 0.51 | 0.28 - 0.42 | 0.26 - 0.39 |
| HD           | 0.57 - 1   | 0.51 - 0.9 | 0.42 - 0.73 | 0.39 - 0.68 |

Figure 7. a) The values of sD according to the classification system in Table 3 together with data for Norwegian clays, the best, the low and the high estimate sD values for the data [2] and lines for the dilatancy neutral (D₀=0) and for the dilatancy D₀=1/3 which gives undrained strength equal to that of the drained capacity in the triaxial compression mode, b) sD according to the Eq.10, with D₀ from Eq.12.

5. Summary and Conclusions

In this paper, a method for the determination of the undrained shear strength of clays based on the dilatancy parameter and overconsolidation ratio is presented. The method is called DSHANSEP. Central to the development is the dilatancy parameter, which measures the pore pressure contribution due to shear. It is shown that the parameter s in the SHANSEP approach depends on friction angle and the dilatancy parameter and therefore not a soil constant. The dependence of the undrained shear strength on the overconsolidation ratio comes because of the dependence of both the initial effective stress and the dilatancy parameter on the overconsolidation ratio. The relationship between the dilatancy parameter and the overconsolidation ratio proposed in this paper is shown to reasonably follow the trends in experimental findings. The dilatancy parameter is also highly sensitive to disturbance and as a result needs to be calibrated based on high quality samples for doing a reasonable prediction of the undrained...
shear strength of clays in the field. Furthermore, the proposed classification system for characterization of clay soils based on the dilatancy parameter and the overconsolidation ratio gives a better description of the likely mechanical response of clay soils under undrained loading. It may also give a better foundation for further research and characterization. The conceptual framework now tuned from the point of view of the practicing engineer, can be implemented into a constitutive model.

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