Determination of cyclic soil parameters for offshore foundation design from an existing data base

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ABSTRACT

Determination of the soil parameters in the foundation design analyses for offshore and nearshore platforms subjected to cyclic loading from wind and waves requires extensive advanced laboratory testing. The amount of such testing can be reduced by drawing on the experience that has been gained during the design of offshore structures in the past. It is outlined how an existing data base can be used to estimate the soil parameters needed in the foundation design analyses based on conventional parameters, like undrained static shear strength, plasticity index and overconsolidation ratio for clays, and relative density and/or water content, fines content and overconsolidation ratio for sand and silt. The estimated soil parameters can be used in feasibility analyses before site-specific parameters are available and to reduce the amount of site-specific advanced laboratory testing in the final design phase. The application of the data base is demonstrated by examples for clay and for sand with different fines content.

1. Introduction

Foundation design is an essential part of the design of offshore and nearshore fixed and floating platforms, both for wind turbines and for the continued development of oil and gas fields. The platforms are subjected to significant cyclic loading from wind and wave loading, and it is imperative for a safe and economical design to establish soil parameters that account for the effect of these cyclic loads.

Offshore platforms for oil and gas development have a more than 50-year-long history, with foundation design as an essential part. This has significantly contributed to geotechnical experience with soil investigation, laboratory testing, soil design parameters and design analyses. Prototype instrumentation and model testing have also provided considerable insight and learning. This experience from the oil and gas development is invaluable in the foundation design of future platforms, both offshore and nearshore.

The experience includes data bases with soil parameters that are required in the foundation design related to both capacity, displacements and dynamic behaviour. One such data base is the NGI data base related to the cyclic contour diagram concept and presented in several papers by e.g. Andersen (2004, 2015) and Andersen and Schjetne (2013). The data base covers different soil types, like clay, silt and sand with different plasticity, relative density, water content, particle size distribution and overconsolidation history. It includes shear strength, deformation and consolidation characteristics, for both monotonic and cyclic loading. The parameters are valid for different types of foundations, including skirted foundations, monopiles, gravity based, jack-ups, suction anchors and piles.

The parameters can be determined based on conventional soil properties and used for feasibility studies before site-specific data are available. The parameters may also be used in the final design, but then they should ideally be verified by some site-specific tests or used with great caution. Especially for wind parks with a large number of units and for nearshore conditions, the soil profiles can be very layered, and detailed testing of all layers may be prohibitive, also for the final design stage.

This paper will demonstrate how the data base mentioned above can be used to provide the soil parameters that are required for the foundation design of platforms under cyclic loading by means of examples for clay and for sand.

2. Foundation design aspects

The major requirements to be addressed in cyclic foundation design...
are: (1) ensuring sufficient bearing capacity; (2) making sure that cyclic displacements are tolerable; (3) providing equivalent soil spring stiffnesses and damping for use in dynamic soil-structure analyses; (4) assessing whether long term displacements due to permanent straining during cyclic loading are tolerable, (5) considering displacements developed during and after cycling through creep and pore pressure dissipation, and (6) assessing how soil reaction stresses developed at the soil-structure interface may change due to cycling. The importance of the different requirements depends on the type of foundation. The requirements are outlined and discussed in more detail by Andersen et al. (2013) and Andersen (2015).

3. Required soil parameters for foundation design

A number of soil parameters are required to address the design requirements. The influence of cyclic loading is especially important, and the data base described herein is related to the cyclic contour diagram concept (e.g. Andersen, 2015).

The soil parameters listed below may be required, but the list can be reduced for some foundation types and soil types.

- Cyclic shear strength
- Deformation parameters
- Pore pressure generation due to cyclic loading
- Initial shear modulus, $G_{\text{max}}$
- Consolidation characteristics (permeability, $k$ and constrained modulus, $M$)
- Effective stress strength parameters, $q'_u$ and $\alpha'$.
- Damping

The cyclic shear strength, deformation and pore pressure parameters depend on the average and cyclic shear stresses and the cyclic load history. A convenient way to express these parameters is in the form of contour diagrams where contours of number of cycles to failure, contours of average and cyclic shear strains and permanent pore pressure are plotted as functions of average and cyclic shear stresses for given number of cycles (e.g. $N = 1, 10$ and $100$). The set of contours should also include cyclic shear strength, average and cyclic shear strains, and permanent pore pressure as functions of the number of cycles for a given average shear stress. Examples of contour diagrams are presented in the figures in Table 3. Andersen (2015) presents more examples and shows how the contour diagrams can be constructed and the laboratory testing that is required.

Both the shear strength and the deformation parameters are stress path dependent, and contour diagrams for both direct simple shear (DSS) and triaxial type of loading are required to take stress-path dependency into account. Stress-path dependency is referred to as anisotropy in this paper, without distinguishing between inherent and load-path induced anisotropies. As discussed later, anisotropy ratios may be used in simplified analyses if triaxial contour diagrams are not available.

Later sections show how site-specific parameters can be determined from the data bases for clay, silt and sand. The input parameters to the data base are vertical effective stress ($\sigma_z$), undrained shear strength ($s_u$), plasticity index ($I_p$) and overconsolidation ratio (OCR) for clays, and $\sigma_{vc}$, relative density ($D_r$) and/or water content ($w$), fines content and OCR for sand and silt. A reference stress, $\sigma'_{ref}$, is used instead of $\sigma_{vc}$ in the data base since the normalized shear strength $s_u/\sigma'_{ref}$ will be more independent of $\sigma_{vc}$ than $s_u/\sigma_{vc}$. The reference stress is defined as $\sigma'_{ref} = p_a (\sigma_{vc}/p_a)^n$, where $p_a$ is the atmospheric pressure ($\approx 100$ kPa), and $n$ is a function of the normalized undrained static shear strength, $s_u/\sigma'_{ref}$, of the soil in its normally consolidated state. The $n$ can be set to 0.9 for clays. Values of $n$ for silt and sand are given in Andersen (2015).

The cyclic parameters should be based on cautiously estimated values of these input parameters, with the intention that they cover the uncertainties associated with the site- or location-specific soil profile.

The cyclic parameters are determined as best estimate values in the examples in this paper, and the correlations expressed as equations express best estimate values from the parameter plots. The uncertainties associated with establishing the best estimate values from the data base are beyond the scope of this paper. It is recommended to visit the diagrams in Andersen (2015) to see the scatter in the data to evaluate the uncertainty behind the equations. It is also recommended to visit Engin et al. (2021) who discuss uncertainties in design analyses, including uncertainties in the soil parameters and the consequences for the design analyses.

4. Available data

This paper mainly refers to the data base described in Andersen (2004, 2015) and Andersen and Schjette (2013) and demonstrates how contour diagrams and other required design parameters can be determined as functions of conventional parameters and stress histories from this data base. The sand and silt parameters in the data base are generally based on silica soils with a coefficient of uniformity less than about $C_u = 12$ and $D_{50} < 0.2$ mm. The correlations may be less reliable for soils with other characteristics.

Additional data sets with cyclic contour diagrams for single, specific soils are available for clay (e.g. Andersen et al., 1988a, Andersen et al., 1988b, Andersen et al., 1989, Jeannen et al., 1998, Kleven and Andersen, 1991, Andersen et al., 1993, Wichmann et al., 2013, Liedtke et al., 2019, Liu et al., 2020a, b, He et al., 2021), sand and silt (e.g. Andersen and Berre, 1999; Yang et al., 2022; Blaker and Andersen, 2019) and calcareous soils (e.g. Finnie et al., 1999; Colleavy et al., 2022). The additional data sets give contour diagrams for one specific soil, however, and not as functions of conventional properties, and may not contain all the parameters that are needed. The information in the individual data sets prior to 2013 is also included in the general data base.

Damping is not part of the existing data base and is discussed in a separate section later in this paper.

5. Parameters for clay

The parameters needed to establish the cyclic parameters from the data base are the static DSS shear strength ratio ($s_u/\sigma'_{ref}$), the plasticity index ($I_p$) and the overconsolidation ratio (OCR).

The following cyclic correlations are needed to perform a design:

- Number of cycles to failure, $N_f$, as a function of normalized average and cyclic shear stresses ($\tau_{cy}/s_u$ and $\tau_{cy}/s_{ref}$)
- Cyclic and average shear strains ($\gamma_{cy}$ and $\gamma_{cy}$) as functions of normalized cyclic and average shear stresses ($\tau_{cy}/s_u$ and $\tau_{cy}/s_{ref}$) for different number of cycles, e.g. $N = 1, 10$ and $100$.
- Cyclic shear strain ($\gamma_{cy}$) as a function of normalized cyclic shear stress ($\tau_{cy}/s_u$) and number of cycles ($N$) for $\tau_{cy} = 0$.

These correlations are needed for DSS and triaxial stress paths and can be expressed in the form of contour diagrams. Examples can be seen in e.g. Andersen (2004). The contour diagrams will depend on OCR and $I_p$ values. One may use a simplified approach where the contour diagrams are established for DSS type loading and empirical anisotropy factors are used to account for triaxial type stress paths. Anisotropy factors are discussed in a subsequent section.

The normalization of the contour diagrams is done with respect to the undrained static shear strength, since a representative undrained shear strength profile is normally established for clays. Normalization to $\sigma'_{ref}$ can also be used, but the contour diagrams will then become very sensitive to OCR and may be more difficult to apply in practice (Andersen, 2015).

The correlations listed above are presented for Drammen Clay in Andersen (2004, 2015). The Drammen Clay data have often been found
to agree well with data from actual sites. The correlations are made more generally valid by including a correction factor for Ip different from Drammen Clay (Ip = 27%).

The procedure to establish site-specific cyclic parameters can be done as explained in the following. The procedure is illustrated by two examples, Clay 1 and Clay 2, with different Ip and σud/σ′ ref as shown in Table 1. The correction factors are determined based on DSS tests, but these parameters shall also be applied for triaxial conditions.

1. Determine the equivalent OCR value for Drammen Clay based on the measured normalized static DSS strength (σad/σ′ ref) through the SHANSEP equation (σad/σ′ refOCR/σud/σ′ refOCR = OCR) where (σad/σ′ refOCR = 0.21 and m = 0.78, also presented graphically in Andersen (2004, 2015)). This OCR should be in line with the measured OCR value if oedometer data is available.

2. Find the Drammen Clay DSS and triaxial contour diagrams for the OCR closest to the OCR determined in Step 1. The Drammen Clay data base contains contour diagrams for OCR of 1, 4 and 40 and can be found in Andersen (2004) and Andersen et al. (1988a, 1988b).

The normalized cyclic shear strength at failure, τycy/σ′ ref, is not very sensitive to the OCR, but the average and cyclic shear strains depend strongly on OCR. Construction of new contour diagrams by interpolation between the diagrams may therefore be necessary if the site-specific OCR is not close to one of the Drammen Clay OCR values. Alternatively, the analyses can be done with contours for different OCR-values around the actual OCR and interpolating between derived stress strain curves or calculated displacements and calculated capacities afterwards.

3. Correct the contour diagrams for the effect of difference in Ip between the actual clay and Drammen Clay (Ip = 27%) by the following factor that shall be applied on the vertical axis of the contour diagrams

\[ F_{Ip} = \frac{(0.41 \pm 0.224)}{(0.41 \pm 0.224)} \approx 0.48 \]  

The equation is based on the relationship between the normalized DSS cyclic shear strength at 10 cycles and Ip of (τcy/σ′ ref)N = 10 = 0.41 Ip /224 (Andersen, 2015).

This scaling may give too low normalized stiffness for clays less plastic than Drammen Clay (i.e. Ip > 27%) and higher normalized stiffness for Ip > 27%. One should therefore consider adjusting for this. This can be done by adjusting the strain values on the strain contours in the contour diagrams, but it can be more convenient to perform this correction by adjusting the stress-strain curves derived from the contour diagrams or the calculated load-displacement curves. The adjustment is done by dividing the shear strains by a correction factor.

The adjustment at small strain or displacement level can be done based on the equation for the normalized initial shear modulus

\[ G_{max} / G_{ud} = (30 + 300(Ip/100 + 0.03)) \cdot OCR^{-0.25} \]  

(Andersen, 2015)

### Table 1

Cyclic shear strength and strain parameters for clay based on the data base in Andersen (2004, 2015).

<table>
<thead>
<tr>
<th>Step</th>
<th>Input</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>3</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>Ip (%)</td>
<td>σud/σ′ ref</td>
<td>OCR</td>
<td>Find contour diagram</td>
<td>Corr. factor for Ip on vertical axis, FIp</td>
<td>Gmax/σ′ ref corr. factor for Ip</td>
</tr>
<tr>
<td>Clay 1</td>
<td>27</td>
<td>0.62</td>
<td>4</td>
<td>Andersen (1988a, b, and 2004)</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Clay 2</td>
<td>15</td>
<td>0.43</td>
<td>2.5</td>
<td>Andersen (2004, 2015)</td>
<td>0.88</td>
<td>1.64</td>
</tr>
</tbody>
</table>

* Correction factor shall be applied by dividing the shear strain by the specified factor.

The correction relative to Drammen Clay (Ip = 27%) will be independent of OCR and becomes

\[ F_{Ip \ small\ strain} = 0.029 \cdot (1 + 10 \cdot (0.01 \cdot Ip + 0.03)^{-1}) \]

The adjustment is expected to be smaller at higher shear stresses, and it is suggested to apply half the correction above at 50% of the failure load i.e. a correction factor of

\[ F_{Ip \ small\ strain} = 1 + (F_{Ip \ small\ strain} - 1)/2 \]

The rest of the strain contours, the stress-strain curves or the load-displacement curves must be interpolated between the corrected values of failure shear stress, small strain stiffness and stiffness at half the failure load based on engineering judgement.

### 6. Parameters for sand and silt

The input parameters needed to establish the cyclic correlations from the data base are relative density (D), fines content (FC) and overconsolidation ratio (OCR).

The following cyclic correlations are needed to perform a design:

- Number of cycles to failure (N) as a function of average and cyclic shear stresses (τa/σ′ ref and τcy/σ′ ref) for drained and undrained values of ∆ta = τa – τ0.
- Cyclic and average shear strains (ta and τa) as functions of cyclic and average shear stresses (τa/σ′ ref and τcy/σ′ ref) for different number of cycles, e.g. N = 1, 10 and 100 for drained and undrained values of ∆ta = τa – τ0.
- Cyclic shear strain (τcy) as a function of cyclic shear stress and number of cycles (N) for τa = 0.
- Permanent pore pressure (up/σ′ ref) as a function of cyclic shear stress and number of cycles for τa = 0.
- Consolidation characteristics, covering permeability and moduli for virgin loading, unloading and reloading conditions. The determination of consolidation characteristics is discussed separately in a later section.

Ideally, the cyclic correlations should be established for both DSS and triaxial conditions, but one may take a simplified approach to establish the contour diagrams for DSS type loading and use anisotropy factors to account for triaxial type stress paths. Anisotropy factors are discussed in a subsequent section.

A data base with cyclic correlations for sand and silt is presented in Andersen (2015). Examples for DSS conditions can be seen in Table 3. The correlations are given for a range of the input parameters, but the cyclic correlations may need adjustment to the exact site-specific conditions. The determination of the cyclic correlations can, with reference to Tables 2 and 3, be done by the steps explained in the following. Two examples for DSS conditions, Case A and Case B, with different Dc, w, FC and OCR as given in Table 2 are selected as illustrations. Both examples require scaling of the contour diagrams to be representative for the specified site-specific conditions. The examples follow and refer to the steps below.

1. Establish input parameters (Dc, w, FC and OCR)

2. Determine the normally consolidated undrained static DSS shear strength (τa/σ′ ref) by entering the diagram for undrained static shear strength with Dc and FC. The strength can also be determined from a similar diagram with water content as input instead of Dc. This can give a valuable supplement to determination based on Dc.

3. Determine the normally consolidated undrained cyclic DSS shear strength for N = 10 at τa = 0 (τcy/σ′ ref)N = 10, by entering the cyclic shear strength diagram with Dc and FC. The cyclic strength can also be determined from a diagram with w as input instead of Dc, as for the static shear strength.
4. Determine the slope of the failure line (α′) in the effective stress path for drained DSS tests. Andersen (2015) presented plots of α′ vs. Dv and water content. The slope for a consolidation stress of σ′vc = 100 kPa in this diagram can be expressed as a function of relative density by

\[ \alpha'_{100} = 0.21 - D_v + 23 \]

or as function of water content by

\[ \alpha'_{100} = 70 - 1.3w \]

The slope for an arbitrary consolidation stress, σ′vc, can be determined from

\[ \alpha'_{\text{vc}} = 3 \times 10^{-6} \cdot \sigma_{\text{ref}}^2 - 0.0023 \cdot \sigma_{\text{vc}} - 1.21 \]

The α′–value to be used in the following is the value for a consolidation stress of σ′vc = 100 kPa because this will be consistent with normalization to σ′ref (σ′ref = σ′vc for σ′vc = 100 kPa).

5. Determine the OCR correction factor. The values determined in the previous steps are for normally consolidated soils. The OCR correction depends on whether the soil contracts or dilates in the normally consolidated state. The normalized undrained static DSS strength, \( \tau/\sigma_{\text{ref}} \), is used to evaluate this. A low normally consolidated \( \tau/\sigma_{\text{ref}} \) indicates a strongly contracting soil, like a clay or a loose sand or silt, and the undrained shear strength will be strongly influenced by OCR. A high normally consolidated \( \tau/\sigma_{\text{ref}} \) indicates a strongly dilatant soil, like a very dense sand or silt, and will be marginally influenced by OCR.

Andersen (2015) presented a plot of the ratio between the normalized undrained shear strengths of overconsolidated and normally consolidated soils that can be estimated by the following expressions:

\[ (\tau/\sigma_{\text{ref}})_{OC} / (\tau/\sigma_{\text{ref}})_{NC} = OCR^{m} \]

where

\[ m = 0.78 \text{ for clay} \]
\[ m = 1.13 - 1.45 \cdot (\tau/\sigma_{\text{ref}})_{NC} \text{ with } m_{\text{max}} = 0.8 \text{ for sand/silt when } (\tau/\sigma_{\text{ref}})_{NC} < 0.44 \]
\[ m = 0.54 - 0.12 \cdot (\tau/\sigma_{\text{ref}})_{NC} \text{ with } m_{\text{max}} = 0 \text{ for sand/silt when } (\tau/\sigma_{\text{ref}})_{NC} > 0.44 \]

The OCR correction shall be applied on both cyclic and static undrained shear strengths. In the case of drained average shear stress, however, the OCR correction can be applied on the cyclic shear stress, but the OCR correction shall not be applied on the part of the failure curve where the failure mode is governed by large average shear strain or on the average shear strain contours (Andersen, 2015).

6. The contour diagrams depend on the drainage conditions during the storm. The contour diagrams assume undrained conditions within each cycle, but the average shear stress can be drained or undrained, and diagrams for both drained and undrained average shear stress may be required. The drainage under average shear stress can be calculated when the variation of the average load during the peak part of the storm is known. Both drained and undrained static shear stress conditions are presented in Tables 2 and 3. The effect of drainage under the average shear stress can be significant. Comparison between the static shear strengths shows whether drained or undrained average shear stress condition will be critical. One may choose to use the most conservative in cases where drainage is uncertain.

It is important to check that the assumption of undrained conditions under a single cycle is valid, especially for dense sand where the effective stress due to dilatancy can be lost if drainage occurs.

7. When the cyclic and static shear strengths are established, the cyclic parameters are established by determining scaling factors to the cyclic contour diagram with the static and cyclic shear strengths closest to the cyclic and static shear strengths established above. The scaling factors are given for both drained and undrained average shear stress (\( \Delta \tau_{\text{a}} \)) conditions. The scaling factors for the data base refer to the diagrams in Figures 12.15, 12.16, 13.1.1 to 13.11 and 14.1 in Andersen (2015), each subnumbered by letter a to e in alphabetic order from the top. Scaling from a diagram with a higher cyclic strength will give stiffnesses on the low side, whereas scaling from a diagram with lower cyclic shear strength will give stiffnesses on the high side.

8. The contours in the diagrams in the data base are for normally consolidated sands and silts. The normalized shear stiffness will decrease with increasing OCR, and the strain or stiffnesses based on these contours should be corrected if they are used for overconsolidated soils. Andersen (2015) presented a plot of the ratio between the normalized shear stiffnesses of overconsolidated and normally consolidated soils. The ratio is higher for the small strain modulus than for the modulus at shear stress levels above 20% shear strength mobilization. The ratios can be estimated by the following expressions:

\[ (G_{\text{max}}/\tau)_{OC} / (G_{\text{max}}/\tau)_{NC} = OCR^{p} \text{ where } p = 0.32 - m \]
Table 3
Example cases A and B. Figures are simplified versions of figures in Andersen (2015).

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Establish input parameters ($D_r$, $w$, FC, OCR)</td>
</tr>
<tr>
<td>2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Case A: $\alpha'<em>{100} = 0.21 \cdot D_r + 23$ or $\alpha'</em>{100} = 70 - 1.3 \cdot w$</td>
</tr>
<tr>
<td>5</td>
<td>Case B: $\tau_f/\sigma_{ref}' = \tau_f/\sigma_{ref}' = OCR'$</td>
</tr>
<tr>
<td>6/7</td>
<td>Failure contours. Undrained $\Delta \tau_a$</td>
</tr>
<tr>
<td></td>
<td>Failure contours. Drained $\Delta \tau_a$</td>
</tr>
</tbody>
</table>

(continued on next page)
Table 3 (continued)

Cyclic shear strain = \( f(\log N, \tau_{cy}/\sigma_{ref}') \) for \( \Delta\tau_a = 0 \)

\( u_p/\sigma_{ref}' = f(\log N, \tau_{cy}/\sigma_{ref}') \) for \( \Delta\tau_a = 0 \)

\( \gamma_a \) and \( \gamma_{cy} = f(\tau_a/\sigma_{ref}', \tau_{cy}/\sigma_{ref}') \) for \( N = 10 \), Undrained \( \Delta\tau_a \)

(continued on next page)
The ratio between the cyclic and the average shear stresses will depend on the weight of the structure and the ratio between the cyclic and average components in the load history. The simplest approach is to assume that the ratio between the cyclic and average shear stresses in the soil are \( \tau_{cy}/(\tau_a - \tau_0) = P_{cy}/P_a \), where \( \tau_0 \) is the initial shear stress in the soil, and \( P \) are the loads from the platform to the soil. A clay will normally not be consolidated under the platform weight prior to the design event, and \( \tau_0 \) will then be due to the soil overburden. A sand may be consolidated, and the weight should then be included in the \( \tau_0 \) calculation for the soil beneath the platform.

The assumption of a constant \( \tau_{cy}/(\tau_a - \tau_0) \) will give the stress paths indicated by the full line in the DSS diagram and the dotted lines in the triaxial diagrams in Fig. 1. The example assumes a ratio of \( \tau_{cy}/(\tau_a - \tau_0) = 1 \). Inspection of the shear strain combination where the different paths intersect the failure envelope will usually show that the average and cyclic shear strains at failure are very different in the DSS and the triaxial contours and that there will not be strain compatibility along a failure surface that involves compression, DSS and extension type elements. In order to achieve strain compatibility, both average and cyclic shear stresses need to be redistributed, and the stress paths will look more like the fully drawn curves in Fig. 1. It can be discussed whether full strain compatibility will occur, but with that assumption, the cyclic shear strength can be defined. The example assumes DSS failure mode to be dominating, but the same exercise can be done with other assumptions.

The cyclic shear strength for the different stress paths is defined by the intersection between the load path and the failure envelope. This procedure requires both DSS and triaxial contour diagrams. The data bases normally contain more DSS than triaxial type contours, and examples of anisotropy ratios that one may apply as approximations in lieu of triaxial contours are presented in Tables 4 and 5. The tables
contain both static and cyclic anisotropy ratios. The cyclic anisotropy ratios have been developed for soils where both DSS and triaxial contours have been available. The strength anisotropy ratios are generally conservative (low) best estimate values but can also be on the optimistic side in a few cases.

The static strength anisotropy ratios for clay in Table 4 are based on experience from onshore and offshore soil investigations (Lunne and Andersen, 2007) and Drammen Clay (e.g. Andersen, 2004). The cyclic shear strength anisotropy ratios are based on contour diagrams for Drammen Clay (e.g. Andersen, 2004) and are valid over the full range of $\frac{\tau_{cy}}{\Delta \tau_a}$ ratios. They are developed for $N = 10$, but are reasonably independent of $N$.

The static and cyclic shear strength anisotropy ratios for sand and silt are based on Andersen (2015), supplemented with data from Dogger.

Table 4
Approximate anisotropy ratios for clay (undrained).

<table>
<thead>
<tr>
<th>Loading</th>
<th>OCR</th>
<th>CAUC/DSS</th>
<th>CAUE/DSS</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Total</td>
<td>Cyclic</td>
<td></td>
</tr>
<tr>
<td>Static</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.25$^a$</td>
<td>–</td>
<td>0.78$^a$</td>
<td>–</td>
<td>Lunne and Andersen (2007)</td>
</tr>
<tr>
<td>1.45$^b$</td>
<td>–</td>
<td>0.61$^b$</td>
<td>–</td>
<td>Andersen (2004)</td>
</tr>
<tr>
<td>1-40</td>
<td>1.45</td>
<td>0.78</td>
<td>–</td>
<td>Drammen, Andersen (2004)</td>
</tr>
<tr>
<td>Cyclic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.25</td>
<td>1</td>
<td>0.5</td>
<td>0.65</td>
</tr>
<tr>
<td>4</td>
<td>1.25</td>
<td>1</td>
<td>0.75</td>
<td>1</td>
</tr>
<tr>
<td>40</td>
<td>1</td>
<td>1</td>
<td>0.75</td>
<td>1</td>
</tr>
</tbody>
</table>

$^a$ Offshore samples.
$^b$ High quality samples.

Table 5
Approximate anisotropy ratios for sand (U means $\Delta \tau_a$ applied undrained. D+ means $\Delta \tau_a$ applied drained by increasing the normal stress. D- means $\Delta \tau_a$ applied drained by decreasing the normal stress).

<table>
<thead>
<tr>
<th>Loading</th>
<th>$D_0$ (%)</th>
<th>Drainage</th>
<th>CAUC/DSS</th>
<th>CAUE/DSS</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Total</td>
<td>Cyclic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static</td>
<td>≥80%</td>
<td>U</td>
<td>4</td>
<td>–</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>70-70%</td>
<td>U</td>
<td>3</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>60-70%</td>
<td>U</td>
<td>2</td>
<td>–</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>&lt;50%</td>
<td>U</td>
<td>1.45</td>
<td>–</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>All $D_0$</td>
<td>D+</td>
<td>1$^c$ 2.25$^b$</td>
<td>–</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>All $D_0$</td>
<td>D-</td>
<td>0.45</td>
<td>–</td>
<td>0.2$^c$ 0.45$^c$</td>
</tr>
</tbody>
</table>

| Cyclic  | ≥80%      | U | 2$^i$ | – | 1.35$^c$ | – | Fig 12.1 Andersen (2015). |
|         | ≥80%      | D+ | 2.5$^i$ | 2.7 | 1.5$^c$ | – |
|         | ≥80%      | D- | 1.5 | – | 0.6$^c$ | 1.5 |
|         | 80-60%    | U, D+, D- | (1.1-1.8)$^i$ | – | (0.4-0.75)$^i$ | – |
| <60%    | U        | 1.25 | 1 | 0.5 | 0.65 | $D_0 < 80\%$ is especially uncertain; must be used with great care! |

$^a$ $K_0' = 0.5$.
$^b$ $K_0' = 1.0$.
$^c$ Best estimate.
$^d$ Range.
$^e$ Scale linearly between 80% and 60%.
It is necessary to determine whether there can be drainage under the average shear stress ($\Delta \tau_a$) in sand and silt since this will decide what type of contour diagram that shall be used. Examples showing the effect drainage can have on the contour diagrams are presented in Figs. 2 and 3. It is important to note that the drained static triaxial shear strength depends strongly on whether the average shear stress ($\Delta \tau_a = 0.5(\Delta \sigma_v - \Delta \sigma_h)$) is applied by increasing or decreasing the normal stress. The static shear strength defines the intersection point at the horizontal axis and has a governing influence on the contours. Fig. 3b shows the contours for the case where the drained $\Delta \tau_a$ is applied by changing the vertical normal stress. The intersection points of the contours at the horizontal axis for the case with changing horizontal normal stress are also indicated. The contours for the case with drained change in horizontal normal stress need to be consistent with these intersection points. Examples are shown in Andersen (2015). The DSS examples in Fig. 2 show that drained conditions generally give lower shear strengths than undrained for very dense sand. The opposite will be the case for low density.

The undrained static strength ratios for sand and silt in Table 5 can be established from Figure 10.4b in Andersen (2015). The static strength anisotropy ratios for drained conditions can be established using the strength formula for the different stress paths (Andersen, 2015) with values of $\alpha'$ and $\phi'$ as functions of relative density according to the equations in Sections 6 and 10, respectively.

Tables 4 and 5 show that there can be significant differences between the triaxial compression ($s_{uc}$), DSS ($s_{uD}$) and triaxial extension ($s_{uE}$) strengths, for both static and cyclic loading. In many cases, the DSS strength can be a reasonable first estimate of the average strength, but this requires that the three strengths contribute equally much to the capacity. The triaxial compression strength can be significantly higher than the DSS strength in dense sand and silt, and the triaxial extension strength can also be higher than the DSS strength. This can have a significant influence on the failure mechanism and the capacity and stiffness of a foundation. Therefore, anisotropy should be given attention in design. The capacity and stiffnesses of structures on dense sands and silts can be significantly underestimated if the design is based on DSS strength without consideration of anisotropy.

7.2. Stress-strain anisotropy ratios

The anisotropy factors described above can be applied in limiting equilibrium analyses and in finite element analyses where the cyclic shear strengths for triaxial compression, DSS and triaxial extension are given as input (e.g. Andersen and Jostad, 1999; Andersen et al., 2005; Jostad and Andersen, 2015).

Fig. 2. Comparison of contours for Nf = 10 in DSS tests for undrained and drained conditions (based on Andersen, 2015).

Fig. 3. Example of difference in triaxial contour diagrams between undrained and drained $\Delta \tau_a$ conditions for very dense sand (based on Andersen, 2015).
Finite element analyses to calculate displacements and stiffnesses require stress-strain input in addition to shear strength. The stress-strain curves can be established by reading corresponding values of shear stress and shear strain along the curves in Fig. 1. This can be done separately for cyclic and average components ($\tau_{cy}$ vs. $\gamma_{cy}$ and $\tau_a$ vs. $\gamma_a$). The average and cyclic components can be added to establish curves for $(\tau_a+\tau_{cy})$ vs. $(\gamma_a+\gamma_{cy})$ if one wants to calculate maximum displacements under maximum load. Ideally, anisotropy should be accounted for by using different curves for DSS, triaxial compression and triaxial extension. Examples of stress-strain curves derived from the contour diagrams as described above are given in Andersen (2015) and Engin et al. (2021).

A simpler, but more approximate approach is to use the strength anisotropy factors to scale the DSS stress-strain curves. Examples of anisotropy ratios as functions of shear strain are shown for different sands with a load path ratio of 1.5 and $N = 10$ in Fig. 4. The figures show that the anisotropy ratio tends to decrease with increasing shear strain. Using the anisotropy ratio at failure for the whole stress-strain curve may thus underestimate the stiffness at stresses below failure. It is important to note, however, that the stress-strain curves may have significantly different shapes in DSS, triaxial compression and triaxial extension, especially for the average components, which show differences similar to what is observed in monotonic tests (e.g. Andersen, 2015).

7.3. Alternative methods

In more sophisticated analyses, the cyclic shear strengths and shear strains are determined as part of the analysis. These alternatives require input in the form of contour diagrams for both DSS and triaxial tests. Andersen and Lauritzen (1988a) proposed a limiting equilibrium approach where the shear stress redistribution is accounted for. The method is based on the assumption that the combination of average and cyclic shear strains is the same along the potential failure surface (strain compatibility), and on the condition that the average shear stress along the potential failure surface is in equilibrium with the average loads. The critical $\gamma_{cy}/\gamma_a$ combination can be determined by iteration.

The most advanced alternative is to use a finite element code where the stress path is calculated in each integration point and the stress-strain characteristics are defined by input in the form of contour diagrams. Such finite element codes (UDCAM and PDCAM) are described by Jostad et al. (2014, 2015). The strain or pore pressure accumulation is also taken care of in the codes. UDCAM is developed for undrained conditions, whereas PDCAM can account for the drainage and pore pressure redistribution that can occur during the cyclic load history. The $\tau_{cy}/(\tau_a-\tau_0)$-ratio can vary from one integration point to the next in these codes.

8. Initial shear modulus

The initial shear modulus may be required to calculate the foundation stiffness under small loads. The initial shear modulus can be used to supplement the contour diagrams with contours for smaller strains than those in the existing diagrams or to adjust the initial part of stress-strain curves derived from the contour diagrams.

The initial shear modulus for clays can be calculated by the expressions (Andersen, 2015)

$$G_{max}/\sigma_{ref} = (30 + 75/(I_p/100 + 0.03)) \cdot OCR^{0.5}$$

There are several formulas that express the initial shear modulus for sand based on void ratio and mean effective stress, like the one from Hardin and Drnevich (1972):

$$G_{max}/\sigma_{oct}^{0.5} = 3222 \cdot (3-e)^2/(1+e) \quad \text{(parameters in kPa)}$$

9. Consolidation characteristics

The consolidation characteristics are needed to calculate pore pressure dissipation, effective stress changes and settlements due to the...
platform weight, dissipation of pore pressure during the cyclic load history, and to check that the conditions are undrained during individual cycles. Such calculations require consolidation characteristics in the form of permeability coefficient and modulus. The modulus may be needed for both virgin loading, unloading and reloading, depending on the project.

The coefficient of permeability and the moduli can be estimated from data bases based on water content (or void ratio), fines content, clay content and $D_{10}$. A data base for permeability can be found in Andersen and Schjønning (2013), which also gives the framework for moduli under both virgin loading, unloading and reloading. The latest parameter correlations for this moduli framework are given in Andersen (2015).

10. Friction angle

It is most convenient to use $\phi_v'$ and $\alpha'$ at a consolidation stress of 100 kPa to determine the drained strengths to apply when constructing contour diagrams. The strengths will then be consistent with normalization to $\sigma'_{vc}$ since $\sigma'_{ref} = \sigma'_{vc}$ when $\sigma'_{vc} = 100$ kPa. The effect of $\sigma'_{vc}$ will then be accounted for in normalization to $\sigma'_{ref}$. The correlation of $\phi_v'$ and $\alpha'$ with $D_r$ can be expressed as

$\phi_v' = 32.4 + 0.077 \cdot D_r - 0.00036 \cdot D_r^2$ (for $\sigma'_{vc} = 100$–199 kPa)

$\alpha'_{100} = 0.21 \cdot D_r + 23$ or $\alpha'_{100} = 70 - 1.3 \cdot w$ for $\sigma'_{vc} = 100$ kPa

The drained peak friction angle is also required to calculate e.g. the skirt penetration resistance by the bearing capacity approach. The empirical constants in Andersen et al. (2008) are based on $\phi_v'$ at $\sigma'_{vc} = 100$–250 kPa. The equation above is thus also applicable for skirt penetration resistance.

11. Damping

Damping in the soil has not been an important issue in the foundation design of offshore platforms so far. Soil damping parameters have therefore not received the same attention as strength and moduli and are not part of the database in Andersen (2015). Soil damping can be more important for wind power foundations, due to different cyclic load characteristics and platform design. Soil damping information may therefore need to be considered. Rather than developing contour diagrams for damping, however, it seems more practical to relate the damping ratio ($D$) to $\gamma_{cy}$, which is the way it has been expressed in the literature. In the approach proposed in this paper, $D$ can then be related to the $\gamma_{cy}$ determined as described in previous sections.

11.1. Literature

Several authors have published empirical correlations expressing damping, $D$, as a function of $\gamma_{cy}$. One of the early references which has been widely used, especially in earthquake engineering, is Seed and Idriss (1970) who present separate correlations for sand and clay. Seed and Idriss (1970) partly drew upon a contemporary study by Hardin and Drnevich (1970).

Seed et al. (1986) reinterpreted the Seed and Idriss (1970) curves and confirmed the damping curves for sand. Sun et al. (1988) developed a correlation for clay which was in agreement with Seed and Idriss (1970). Idriss (1990) presented a common curve for sand and clay similar to Seed and Idriss (1970) for clay, which is lower than Seed and Idriss (1970) for sand.

More recently, Vucetic and Dobry (1991) and Darendeli (2001) have published correlations for sand and clay with $I_p$ as an important parameter.

Darendeli (2001) rates the importance of different parameters and states that:

- effective stress, soil type, plasticity and number of cycles can be very important
- load period can be important
- OCR, void ratio, and grain characteristics, size, shape, gradation and mineralogy can be less important
- fines content is not important

However, the Darendeli (2001) formulas do not always seem to support this ranking.

Vucetic and Dobry (1991) state that the $I_p$ is the most important parameter, and that there is little influence of test type, OCR and N. They found a relatively large variation in $D_{min}$ of 1%–5.5% with no clear relation to $I_p$, however.

Seed and Idriss (1970) state that the effect of number of cycles is small and that the effect of consolidation stress is important for sand.

The correlations above are generally valid for symmetrical cyclic loading with $N \leq 10$ at a load period of 1s on normally consolidated soils. None of the references give guidance for non-symmetrical cyclic loading.

11.2. NGI tests

NGI has interpreted damping from cyclic DSS, triaxial and resonant column laboratory tests on clay and cyclic DSS and triaxial laboratory tests on dense sand (e.g. Blaker and Andersen, 2019; Lovholt et al., 2020). The cyclic loading was applied load controlled with a load period of 10s for most tests. The damping ratio was derived by using an improved method for interpretation of the damping, taking the influence of permanent strain accumulation into account (Lovholt et al., 2020).

The tests on clay were run on intact samples with OCR in the range 1.35–1.5 of high, medium and low plasticity with $I_p$ of about 80%, 37% and 18%, respectively as well as on a quick clay.

The tests on sand were run on two batches of fine to medium Dogger Bank sand (Blaker and Andersen, 2019). Batch A had essentially no fines, and Batch B had 20% fines. Triaxial and DSS tests were run on two relative densities, $D_r = 80\%$ and $100\%$, in a normally consolidated state. The $D_r = 80\%$ specimens from Batch A were also tested at OCR = 4.

11.3. Comparisons

The different correlations are plotted for high plasticity clay, low plasticity clay, sand with 20% fines, and clean sand, in Figs. 5 and 6. The results from the NGI tests are included, both as results from individual tests and as contours based on the tests. The illustration of the effect of parameters like OCR, $\tau_a$, load period, $\sigma'_{vc}$ and triaxial vs. DSS in the NGI tests is limited due to space limitations.

Comparison of the correlations show similarities, but also considerable differences in some cases. The agreement between the NGI tests and the correlations depends on the soil. Some main findings are listed below. General:

- The range between the upper and lower Seed and Idriss (1970) curves is very wide. It would be very conservative to use the most unfavourable limit in some cases.
- The correlations show that $N$ has a small effect on $D$. This does not agree with the NGI tests, as discussed below. The effect of $N$ is important to note especially for fatigue analyses where the number of representative cycles can be high.
- Seed and Idriss (1970) mean curve, Vucetic and Dobry (1991) and Darendeli (2001) give similar $D$ for clay with $I_p = 15\%$–30\%, but the effect of $I_p$ is much more significant in Vucetic and Dobry (1991) which gives lower $D$ than the two other at high $I_p$. Seed and Idriss (1970) is independent of $I_p$.
- The correlations do not discuss the effect of $\tau_a$. The NGI tests show that $D$ increases with increasing average shear stress in the clays, and it will be conservative (low $D$) to use $D$ from DSS tests with...
Fig. 5. Empirical correlations of $D$ with $\gamma_{cy}$ from literature and results from NGI tests for high and low plasticity clays.

Fig. 6. Empirical correlations of $D$ with $\gamma_{cy}$ from literature and results from NGI tests for clean sand and sand with FC = 20% at $D_r = 80\%$ and 100\%.
symmetrical cyclic loading. The effect is less conclusive in the dense sand.

- D tends to increase with increasing load period.
- Correlations for sand give higher D than for clay. D for sand is the same as for clay with I_p = 0 in Vucetic and Dobry (1991) and Darendeli (2001).

  - High plasticity clay:
    - Vucetic and Dobry (1991) give a smaller D than Darendeli (2001) at γ<sub>cy</sub> > 0.03%, smaller than Seed and Idriss (1970) mean curve and close to Seed and Idriss (1970) lower curve.
    - NGI tests give a somewhat higher D than Vucetic and Dobry (1991), but smaller than Darendeli (2001) and Seed and Idriss (1970) mean curve. NGI tests give D close to Seed and Idriss (1970) lower curve.
    - NGI tests give a small effect of N, as in the literature correlations.

  - Low plasticity clay:
    - Vucetic and Dobry (1991), Darendeli (2001) and Seed and Idriss (1970) mean curve give similar D for γ<sub>cy</sub> > 0.1%, but Darendeli (2001) gives lower D than the others for γ<sub>cy</sub> < 0.1%.
    - NGI tests give significant effect of N with highest D for low N. This is not captured by the literature correlations.
    - NGI tests are similar to Vucetic and Dobry (1991) and Seed and Idriss (1970) mean curve for γ<sub>cy</sub> < 0.1%, and higher than Darendeli (2001), especially at low γ<sub>cy</sub>.

- Sand:
  - The literature correlations give D independent of D<sub>r</sub> and a small effect of N.
  - Vucetic and Dobry (1991) and Seed and Idriss (1970) mean curves are similar.
  - Darendeli (2001) gives a lower D which is similar to Seed and Idriss (1970) lower curve.
  - The NGI tests indicate that D depends significantly on N, but also on FC and D<sub>r</sub>. This is in contradiction with the literature correlations.
  - Darendeli (2001) predicts no effect of overconsolidation ratio, which agrees with only a small tendency for D to decrease with increasing overconsolidation ratio in the NGI tests.

- Clean dense sand:
  - NGI tests give D for N = 1 similar to Vucetic and Dobry (1991) and Seed and Idriss (1970) mean curve and is higher than Darendeli (2001). However, D decreases with N in the NGI tests, and the NGI tests give lower D than Vucetic and Dobry (1991) and Seed and Idriss (1970) mean curve when N > 1. The effect of N is more important for Dr = 80% than for Dr = 100%.

- Dense sand with 20% fines:
  - NGI tests give lower D than Vucetic and Dobry (1991) and Seed and Idriss (1970) mean curve and is similar to Darendeli (2001) for N = 1. However, D decreases with N in the NGI tests, and the NGI tests give even lower D than Darendeli (2001) when N > 1. The effect of N is more important for Dr = 100% than for Dr = 80%.

11.4. Summary and recommendations

There can be significant differences between D from the different literature sources. The agreement with the NGI tests varies from one case to the other, and the NGI tests do not clearly support one source in favour of the others. It is recommended to use the NGI curves for the different soil types in Figs. 5 and 6 with some engineering judgement as best estimate D. One may consider using D from the lowest correlation as a conservative estimate.

12. Discussion

The cyclic contour diagrams for sand and silt in the data base are generally based on predominantly silica soils, a coefficient of uniformity less than about C_u = 12, D<sub>50</sub> < 0.2 mm, a load period of 10s, and a modest preshearing of 400 cycles with a cyclic shear stress of 4% of the vertical consolidation stress. Yang et al. (2022) showed that the contours did not seem to be significantly influenced by mineralogy when about half of the quartz content was replaced by K-Feldspar and Plagioclase, but the coefficient of uniformity and D<sub>50</sub> could influence both the undrained static and cyclic shear strengths.

For sand and silt the relative density and the water content are used as a basis to select cyclic contour diagrams. The relative density is also used to select additional parameters for sand, such as φ<sup>'</sup> and α<sup>'</sup>. Relative density in situ is often estimated from cone penetration test (CPT) correlations according to e.g. Jamiolkowski et al. (2003). Estimating D<sub>r</sub> from laboratory testing is subject to the determination of minimum and maximum dry densities. Both the in situ and the laboratory determination of D<sub>r</sub> involve uncertainty. The water content is a simpler parameter, but reliable in situ measurements can be a challenge, especially in sand. The data base in Andersen (2015) offers the advantage of comparing the D<sub>r</sub>-based estimate of parameters with a parallel estimation based on the water content. Potential differences in parameters determined based on D<sub>r</sub> and water content should be evaluated based on engineering judgment. One should also keep in mind the scatter in the data that the correlations are based on.

Andersen (2015) includes diagrams that can be used to estimate corrections for load period and level of preshearing. Modest preshearing will normally cause increased strength and stiffness, but it should be noted that laboratory tests show that preshearing can cause degradation of strength and stiffness of overconsolidated soils (e.g. Andersen, 2015). This may not be a problem for the soil beneath a foundation since the soil beneath the foundation will be strengthened by the simultaneous increase in effective stresses from the weight of the platform. The consolidation under the weight of the structure will also reduce the OCR prior to the design event and thus the potential for negative preshearing effect. Outside the platform, however, there may not be increased effective stresses, neither in clay or in sand, and the OCR may remain high. This can lead to a reduction in strength and stiffnesses outside the foundation, increasing with time.

Foundation design is often based on strength and stiffnesses from DSS tests, assuming that DSS data represent a reasonable average of triaxial compression, DSS and triaxial extension data. This may not always be true, especially for dense silt and sand, as discussed in Section 7.2. DSS testing requires less soil material, and time and costs are saved by limiting testing to DSS. The anisotropy data presented herein can be used to estimate the effect of anisotropy and whether it will be worth the extra effort to include monotonic and cyclic triaxial tests.

13. Summary and conclusions

The soil parameters needed to perform a foundation design of an offshore or a nearshore structure under cyclic loading from wind and/or waves can be estimated from an available data base. This includes cyclic shear strength, deformation parameters, pore pressure generation due to cyclic loading, initial shear modulus, consolidation characteristics, effective stress strength parameters, φ<sup>'</sup> and α<sup>'</sup> and damping.

The input to the data base are conventional parameters, like undrained static shear strength, plasticity index and overconsolidation ratio for clays, and relative density and/or water content, fines content and overconsolidation ratio for sand and silt.

The data base includes correlations and parameters from triaxial compression, direct simple shear and triaxial extension tests, for both static and cyclic loading, thus covering the typical stress-paths that
characterize different types of soil-foundation interaction behaviour. Therefore, the parameters are valid for a wide range of foundations, including skirted foundations, monopiles, gravity bases, jack-ups, suction anchors and piles. The data base does not include damping. Interpretation and guidance on damping parameters are therefore included in this paper.

The estimated soil parameters can be used in feasibility analyses before site-specific parameters are available and to reduce the amount of interpretation and guidance on damping parameters are therefore included in this paper. The application of the data base is demonstrated by examples for clay and for sand with different fines content.

CRediT authorship contribution statement

Knut H. Andersen: Writing – original draft, Prepared the manuscript. Harun Kursat Engin: Writing – review & editing, Reviewed the manuscript. Marco D’Ignazio: Writing – review & editing, Reviewed the manuscript. Shaoli Yang: Writing – review & editing, Reviewed the manuscript.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data is available in the referenced publications.

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References


List of Symbols and Abbreviations

\(C_u\): Coefficient of uniformity, \(C_u = D_{60}/D_{10}\)
\(D\): Damping ratio
\(D_r\): Relative density
\(D_{10}\): Particle diameter where 10% of the population is smaller
\(D_{60}\): Particle diameter where 60% of the population is smaller
\(DSS\): Direct simple shear
\(FC\): Fines content
\(F_{p,small\,area}\): Correction factor for plasticity on cyclic shear stress at failure
\(F_{p,50\%\,area}\): Correction factor for plasticity on shear modulus at 50% of failure
\(G_{max}\): Initial shear modulus
\(I_p\): Plasticity index
\(M\): Constrained modulus, tangent
\(m\): Exponent in SHANSEP equation

\(N\): Number of cycles
\(NC\): Normally consolidated
\(N_c\,max\): Number of maximum load cycles that is equivalent to the full load history
\(N_c\): Number of cycles to failure
\(NGI\): Norwegian Geotechnical Institute
\(OC\): Overconsolidated
\(OCR\): Overconsolidation ratio
\(P\): Load
\(\mu\): Exponent in equation for G as function of OCR
\(s_{u0}, s_{uc}, s_{uE}\): Undrained shear strength in DSS, triaxial compression and triaxial extension, respectively
\(u_p\): Permanent pore pressure (pore pressure at end of a cycle)
\(w\): Water content
\(a\): Slope of failure line in DSS effective stress path plot
\(\psi_{pl}\): Peak drained friction angle
\(\gamma_{ave}, \gamma_{cy}\): Average and cyclic shear strain, respectively
\(\gamma_{ref}\): Vertical effective stress
\(\sigma_{ref}^{n}\): Reference stress, \(\sigma_{ref}^{n} = p_a(\sigma_{vc}^{u}/p_a)^n\), where \(p_a\) is the atmospheric pressure (\(-100\,kPa\)), and exponent \(n\) is a function of normalized undrained shear strength of the soil in its normally consolidated state
\(\tau_{a}, \tau_{cy}\): Average and cyclic shear stress, respectively
\(\tau_{a,f}, \tau_{cy,f}\): Average and cyclic shear stress components at failure, respectively
\(\tau_f\): Shear stress at failure
\(\tau_{cy,f}\): Cyclic shear stress at failure, \(\tau_{cy,f} = \tau_{a,f} + \tau_{cy,f}\)