GEOTECHNICAL CONSTRUCTIONS ON SOFT SENSITIVE CLAYS: SOME ESSENTIAL ASPECTS AND EXAMPLES

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ABSTRACT: Soft sensitive clay deposits exhibit strain-softening, which can affect, for example, the stability of an overlaying fill and the capacity of a foundation resting on this type of clay. Design based on the peak strength of such clays, even with a factor of safety more than one, may still result in the failure of the geotechnical constructions due to progressive failure. On the other hand, the design may be overly conservative if residual shear strength is used in the design calculations. Therefore, geotechnical constructions on soft sensitive clays require additional efforts with respect to safe and economical design. In this paper, some of the essential aspects related to soft sensitive clays are presented using real examples from Norway.

INTRODUCTION

The mechanical behavior of soft sensitive (SS) clays is greatly influenced by the clays’ sensitivity ($S_t$), i.e., the ratio of the undisturbed undrained shear strength to the remolded undrained shear strength. SS clays are mainly marine deposits and can be found onshore or offshore. SS clay exhibits strain-softening, i.e., monotonous reduction in shear strength beyond the peak with increasing strain, ref. Fig. 1. The strain-softening behavior of SS clays may, in some cases, lead to a progressive development of a failure mechanism. One of the characteristic features of progressive failures is that a relatively small and local disturbance in a highly sensitive clay creates an extensive landslide, often in areas with only modest inclination.

Fig. 1 Typical stress-strain relationship for non-sensitive clays and SS clays (τ_{peak} is the peak intact soil strength and τ_{rem} is the residual remolded shear strength)

Six thousand-km-long coastlines in India are covered with marine clays. According to [1, 2 and many more], marine clays along the Indian coastlines are moderately sensitive in nature but are less sensitive than those found in Norway, Sweden and Canada. The majority of available literature regarding the SS clays in India has focused on mineralogical characteristics, index properties and compressibility. To provide insight into other aspects, this paper focuses on the strength characteristics of soft sensitive clays. The following sections present some essential aspects and observations from Norway with regard to the mechanical behavior of SS clays.

ANISOTROPY

The short-term stability of geotechnical constructions on SS deposits should be calculated based on undrained shear strength. The undrained shear strength of an SS clay is not an absolute value but varies depending on its stress history. SS clays can be highly anisotropic in nature. In other words, this type of clay exhibits different shear strength depending on the orientation of the principal stresses. The failure plane of an SS clay slope can be divided into three main zones: the active zone, the direct zone and the passive zone, ref. Fig. 2.

Fig. 2 Stress conditions (a) prior to erosion and (b) after the construction of geotechnical structures

Figure 2 (a) shows stress conditions in these three zones. The mobilized shear stress ($\tau$) can be defined in terms of the earth pressure at rest ($K_0$) and the effective vertical stress ($P_e$). With geological-scale time, the terrain may alter due to erosion. This will cause a reversal of the shear stress, i.e., the lateral stresses will exceed the axial at point P, whereas there will be no influence on the active zone, at point A, and a light increase in the mobilization will take place in
the direct zone. When geotechnical constructions such as fills or houses are built on the top of the altered terrain, $\tau$ along the potential failure plane will be calculated as shown in Fig. 2 (b), i.e., at point $P$, $\tau$ will be calculated using the coefficient of passive earth pressure ($K_p$), whereas at point $A$ will be calculated using the coefficient of active earth pressure ($K_a$).

**Critical Slope Angles**

Hazard mapping and hazard assessment are very important prior to constructing a structure over SS clays because a relatively small and local disturbance in a highly sensitive clay creates an extensive landslide, often in areas with only modest inclination. Among other parameters, the critical slope angle ($\theta$) is an important parameter used in hazard mapping. $\theta$ primarily corresponds to SF and the soil condition, i.e., drained or undrained. $\theta$ can be obtained using the following equations for the drained and undrained conditions.

**Drained condition:**
$$\tan(\theta) = \frac{\gamma'}{\gamma} \times \tan(\phi) \times \frac{1}{SF}$$

**Undrained condition:**
$$\sin(\theta) = \frac{\gamma'}{\gamma} \times \alpha \times \frac{1}{SF}$$

These equations can be illustrated using a simple slope stability calculation of a long natural slope where the failure surface is parallel to the ground. The ground water level was assumed to be on the surface level. The input parameters are shown in Table 1, and the results are presented in Table 2.

### Table 1 Input parameters

<table>
<thead>
<tr>
<th>Soil density-total (kN/m$^3$)</th>
<th>Soil density-effective (kN/m$^3$)</th>
<th>Friction angle ($\phi$)</th>
<th>Strength factor for SS clays ($\alpha$)</th>
<th>Average shear strength (undrained) ($Su_m$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>10</td>
<td>30</td>
<td>0.2</td>
<td>$\alpha x p_0'$</td>
</tr>
</tbody>
</table>

### Table 2 Results

<table>
<thead>
<tr>
<th>Condition</th>
<th>SF</th>
<th>$\theta$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained</td>
<td>1.0</td>
<td>16.1</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>11.7</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>10.2</td>
</tr>
<tr>
<td>Undrained</td>
<td>1.0</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>3.6</td>
</tr>
</tbody>
</table>

While a long natural slope may exhibit SF more than 1 for the slopes up to 16.1$^\circ$ under drained condition, according to Table 2, a geotechnical construction on any slope steeper than 5.7$^\circ$ will be subjected to failure if the stability of slope is not improved. This is due to the fact that SS clays usually exhibit rather low shear strength under undrained conditions ($Su_m = 0.2 x p_0'$). To have a good safety margin, the Eurocode 7 recommends a partial factor of 1.4 on the undrained parameters, meaning that any slope that is steeper than 4.1$^\circ$ should have a necessary measure prior to the geotechnical construction over it. In Norway, $SF = 1.6$ is recommended for large fills or complex geotechnical constructions on SS clays. This simple exercise illustrates how an SS clay slope steeper than 5.7$^\circ$ can become critical against failure under an undrained condition.
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i.e., $c = a \tan \phi$. As per Eqn. 3, the failure line for SS clays is unique regardless of the type of loading, i.e., drained or undrained. From this perspective, the pore pressure response plays a key role in defining the failure mechanism of SS clays under an undrained condition. Pore pressure-induced softening is often observed for SS clays where the strain-softening occurs without degrading the strength parameters, such as $c$ and $\phi$ of the soil. This strain-softening is observed due to shear-induced pore pressure resulting from the contracting behavior of the material [6]. This is demonstrated in Fig. 4 using results from the two triaxial tests carried out for two SS clays ($S_t = 8$ and 18). The normalized pore pressure ($n_{pw}$) plots (the lines with dark circles on the Fig. 4) show that there is a continuous increase in pore pressure with increasing deformation ($n_i$) even beyond the shear stress ($\tau$) at the peak. This is due to the contractant behaviors of SS clays. The effective stress paths show no softening in the strength parameters. The degree of mobilization plots ($n_f$) show that the peak shear strength was achieved when $n_f$ was around 0.8. The strain-softening response was observed in the hardening regime itself, $n_f < 1$, due to the shear-induced pore pressure. A full mobilization is achieved when $S = S_f$ and consequently $\tan \phi = \tan \phi$. More details about pore-pressure-induced softening can be obtained from [7].

SHEAR BAND THICKNESS-DEPENDENT CAPACITY

Strain-softening in SS clays is associated with the formation and propagation of shear bands, i.e., bands with plastic deformation. A study by [8] shows that the physical thickness of shear band ($t_{sb}$) is around 3.0 mm for SS clays. At the onset of localization, the negative second-order work is concentrated within the shear bands alone. Considering the direct shear test condition, the work done ($W$) by the shear band can be expressed as

$$W = B \times n_\delta \left( \frac{1}{\gamma} \int \tau \cdot d\gamma \right)$$  \hspace{1cm} (4)

where $\frac{1}{\gamma} \int \tau \cdot d\gamma$ gives the average shear stress and $\gamma$ is shear strain.

The average shear stress is inversely proportional to the shear strain. For a given $n_\delta$, a thinner shear band will have more concentrated plastic strain; hence, the average shear stress will be lower than in a wider shear band. Thus, a narrow shear band would exhibit a large reduction in shear resistance in the post-peak regime due to the large concentration of plastic strain within the SB. Consequently,

Fig. 4 Undrained triaxial test results for two SS clays. Here, $W$ is the water content, LL is the liquid limit, PI is the plasticity index, $\gamma_{sat}$ is the saturated unit weight, $S_u$ is the remolded shear strength, $S_u$ is the uniaxial strength and OCR is the over consolidation ratio of the given clay samples.

Fig. 5 Shear band thickness ($t_{sb}$)-dependent undrained capacity of a slope consisting of SS clay ($S_t = 2$). Here, $S_s$ is synonymous to $S_{ua}$. Courtesy [9].
a narrower shear band will cause a higher softening rate and more brittleness in general. The thickness of shear band has a direct influence on the capacity of soft (sensitive) clays. Figure 5 shows numerical results from [9], where an SS clay ($S_d = 2$) exhibits a reduction in the undrained capacity by 30% when the thickness of shear band reduced from 150 cm to 1 cm. Therefore, it is important to know the physical thickness of shear bands and the realistic capacity of SS clays. Observations show that initial slides often take place due to the over-prediction of the capacity of SS clays. It is not always necessary that SS clays show a maximum capacity equal to $(\pi + 2) \times S_u$ under an undrained condition.

**SLIDING PROCESS**

The mechanism of landslides in long natural slopes of SS clays is a complex problem because vast areas of seemingly stable ground may become engaged in landslides triggered by some trivial local disturbance agents. Figure 7 shows a schematic view of a coastal area prior to a slide that took place in 2009 in Norway. The slide started by an initial slide due to remolding of SS clay by a stone piece in area 1 (see Fig. 7), and within a matter of minutes, the slide progressed to points 2 and 3 and so forth. There were several houses in the area that were displaced in the sea by several hundred meters.

**CONCLUSIONS**

This paper presented a brief overview of the strength-related characteristics of SS clays. An emphasis was placed on anisotropy, pore pressure response, failure mechanism (shear band) and sample disturbances that may be associated with SS clays. These aspects must be taken into account in the design of geotechnical constructions on this kind of material.

**REFERENCES**