Factors Influencing the Shear Strength of Clays: A review

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Abstract. The shearing strength of clays is greatly affected by some factors. These factors are including the composition of soil minerals, the initial moisture content, the degree of saturation, the over-consolidation ratio, the initial void ratio, the porosity, the loading conditions (drained conditions versus undrained conditions), the pore fluid composition, weathering, the number of freeze-thaw cycles the soil is subjected as well as any stabilization techniques used on the clay mass. In this study, the effect of three factors (mineralogy, over-consolidation ratio, and drained versus undrained loading) are examined through the available technical literature. The conclusions made in the literature show that (a) the presence of montmorillonite in the soil mass can affect the shear strength in a more pronounced manner than other clay minerals, (b) the higher the over-consolidation ratio (OCR), the larger the shearing resistance of a soil mass, (c) soils loaded under drained conditions tend to exhibit higher strengths than those loaded under undrained conditions, (d) there are many methods to raise the un-drained shearing strength of the soil mass.

Keywords: Mineralogy, montmorillonite, friction angle, clay, drainage, shearing strength.

1. Introduction
The shear strength of fine-grained soil such as clay through un-drained loading is a measured response and it is assuming has no change in volume. For fine-grained soils, the un-drained shearing strength and water content can be depicted as a nonlinear relation where the classification of soil is known by two characteristics. These parameters are related on the mineral compositions of soils [1]. Generally, the mechanical properties of cohesive soil depend on more important characteristic as plasticity, water content and mineral composition of soil. The relation between many factors as plasticity index, mineral composition over-consolidation ratio (OCR) and soils loaded under drained conditions with shear strength were studied by several researchers [1-4]. The aim of this study is to discover the effect of three factors (mineralogy, over-consolidation ratio, and drained versus undrained loading) on shear strength of clays by reviewing the available preceding studies.

2. Influence of mineralogy
This section is concerned with the composition of soil minerals and its influence on the shear strength of soil. Limited researches are obtainable concerning the effect of mineralogy of clay on the shear strength.
Correlations between the geotechnical properties of different mineralogical components of soil and shear strength are included in the past studies. Most of the tested samples are mixtures of montmorillonite, kaolinite and quartz. It was found that the influence of montmorillonite could greatly reduce the angle of friction and increase the liquid limit and plasticity index. The literature below provides an overview of the influence of the mineralogy on the shear strength of clay soils.

2.1. Tiwari and Marui (2005)
Depending on the results for more than 35 soil samples, Tiwari and Marui [5] concluded that the relations between residual angle of friction and some soil properties as liquid limit (LL), plastic limit (PL), plasticity index (PI), and clay content fraction (CF) are significantly useful to calculate the residual shear strength parameter of soil. Figure 1 contains the relation between the residual frictional angle and the plasticity index and figure 2 contains the correlation between the residual frictional angle and the liquid limit as proposed by Tiwari and Marui [5]. Kang et al [2] found that the residual frictional angle depends on the proportion of both the smectite and the total minerals of clay. LL and residual frictional angle of the soil are greatly varied due to the clay minerals type despite particle size distribution analysis shows same CF.
2.2. Smith et al (2006)

To describe earth materials, which do not well fit into the explanation of either soil or rock, a new term, Intermediate Geotechnical Material (IGM) is introduced [6]. It is typically used for very hard soils or very soft rocks. The main difference between over-consolidated clays and clay shales is only the amount of pressure that was applied to get them to their present state. These materials can be degraded from rock or IGM, which helps to design shear strengths. According to Stark et al [7], the residual shearing strength is directly related to the frictional resistance of the particles, which is function of clay mineralogy. At residual strength, shear displacement plays a vital role for the clay's strength, which is just a function of its mineralogy. Due to the shear displacement there is parallel alignment of the individual clay particles. Clay minerals flocculated into a hard mass in a saturated environment. That type of hard mass has low permeability. When these materials are unloaded, it bounces back to some extent and the tendency to expand causes high negative pore pressure. As a result, increase its un-drained strengths. Strength design of these materials depends basically on the mineralogy of the clay.

The clays from south of the current Potomac River Virginia contains high percentages of Montmorillonite minerals, which is typically classified as CH materials. Some of the properties of this type of clays are LL 70+ to 100+, PL 30%, moisture contents near the PL [8], Standard penetration test (SPT) blows counts 20 to 40 blows per foot. Unconfined compression shear strength of this type of clay varies between 4 to 6 ksf [8]. In these types of soil, generally, progressive failure occurs and lateral earth pressure coefficient values at rest varies from 1.5 to 2.3. These values were determined by the correlation developed for the similar type of hard clays. In addition, measured residual shear friction angles are lies between 10 to 15 degrees. In the failure surface, the cohesion is reduced to zero.

The geology of these heavily over-consolidated plastic clays (mineralogical and tectonically) make slope failures in the area and these landslides are triggered by ground water conditions. Design strength values must also consider as risk tolerance and economics.
2.3. Dimitrova, R. S., & Yanful, E. K. (2012)

Dimitrova and Yanful [3] arranged artificial mixed samples of mine tailings/clay in the research facility, the sample were created by adding kaolinite or bentonite clay to hard rock mine tailings. The all content of clay from obtained samples differed from 4% in the reference mine tailings to 16%, with the upper limit comparing to the greatest clay rate found in many tailings in Canada. They found that the effective angle of friction was found to shift somewhere in the range 35.2° and 40.4° relying upon the level of clay in the mixed samples and the type of the clay substance. Shearing under somewhat drained state yielded a total angle of friction of the mixed samples, which is consistently lower than the active friction angle and ranged around 15.1° and 23.3°. In addition, it was concluded that clay, which was added to mine tailings generally caused a lessening in the friction strength of the last mentioned; nonetheless, the extent of this reduction was more when clay was bentonite and lower when kaolinite.

2.4. Mehta, B. & Sachan, A. (2017)

Mehta and Sachan [4] conducted experimental study on the correlation of mineralogical properties (CEC, SSA, montmorillonite content) for expansive soil and the mechanical properties. The mechanical properties include shear strength, compressibility, swelling potential and index characteristic. All experiments have been conducted on soil which is considering expansive (called black cotton soil) existing large areas of Bhavnagar, extended along the beach line of Gulf of Khambhat in Gujarat city (India) (as shown in table 1), which has genuine construction state due to its hard shrinkage–swelling. Figure 3 shows mineralogical properties of Bhavnagar soil.

| Table 1. Properties of expansive soil in Bhavnagar area [4]. |
|-------------------------------------------------------------|
| **SN** | **Soil region** | **Distribution of grain size (%)** | **Mineral of clay (%)** | **Atterberg limits (%)** | **"DFSI" (%)** | **Soil class** |
|       |                | **S** | **G** | **M** | **C** | **Mon** | **Ka** | **LL** | **PI** |
| S1    | "Sidsar"      | 10   | 0    | 40   | 50   | 50     | 0     | 81    | 51    | 100   | CH    |
| S2    | "Mamsa"       | 7    | 1    | 42   | 50   | 48     | 2     | 72    | 50    | 97    | CH    |
| S3    | "Sanes"       | 4    | 0    | 59   | 37   | 37     | 0     | 83    | 61    | 68    | CH    |
| S4    | "Tagdi"       | 20   | 1    | 35   | 44   | 37     | 7     | 67    | 43    | 67    | CH    |
| S5    | "Virani"      | 18   | 0    | 44   | 38   | 30     | 8     | 66    | 44    | 58    | CH    |
| S6    | "Nari"        | 38   | 4    | 20   | 28   | 28     | 0     | 64    | 42    | 55    | CH    |
| S7    | "Aavaniya"    | 23   | 2    | 41   | 34   | 28     | 6     | 53    | 32    | 54    | CH    |
| S8    | "Bhuteswar"   | 44   | 3    | 28   | 25   | 25     | 0     | 58    | 37    | 52    | CH    |
| S9    | "Akwada"      | 40   | 0    | 42   | 18   | 18     | 0     | 34    | 14    | 23    | CH    |
| S10   | "Harinagar"   | 68   | 0    | 11   | 21   | 21     | 0     | 37    | 19    | 29    | CH    |

*DFSI is the differential free swell index*
Figure 3. Mineralogical parameters of soil in Bhavnagar region. (a) Typical XRD pattern of soil in Bhavnagar, (b) UV spectroscopy results of soil in Bhavnagar for CEC estimations [4].

Table 2 shows the impact of mineralogical parameters of soil in Bhavnagar region on compressibility index and shearing strength limits. Figure 4 explains impact of content of montmorillonite mineral on geotechnical parameters of expansive soil in Bhavnagar region. It is shown that increasing the montmorillonite content leads to increasing L.L, PI, compression index and Shear strength.

**Table 2.** Impact of mineralogical characteristics of soil in Bhavnagar area on the compressibility and shear strength properties [4].

| SN | DFSI (%) | Mineralogical parameters | Compressibility properties | Swell pressure | Parameters of shear strength |
|----|----------|--------------------------|----------------------------|----------------|-----------------------------|
|    |          | CEC (meq/100gm) | SSA (m²/gm) | Cₛ | Cᵣ | SP (KPa) | C (KPa) | φ (deg) | Sᵤ (KPa) |
| S1 | 100      | 148 | 572 | 0.28 | 0.04 | 750 | 187 | 46 | 208 |
| S2 | 97       | 135 | 538 | 0.26 | 0.06 | 550 | 221 | 44 | 240 |
| S3 | 68       | 112 | 394 | 0.23 | 0.06 | 420 | 201 | 41 | 218 |
| S4 | 67       | 123 | 405 | 0.17 | 0.05 | 400 | 197 | 37 | 212 |
| S5 | 58       | 120 | 415 | 0.20 | 0.05 | 390 | 178 | 36 | 193 |
| S6 | 55       | 102 | 389 | 0.16 | 0.02 | 230 | 115 | 39 | 131 |
| S7 | 54       | 109 | 383 | 0.17 | 0.03 | 200 | 115 | 31 | 127 |
| S8 | 52       | 93  | 388 | 0.20 | 0.02 | 180 | 108 | 36 | 123 |
| S9 | 23       | 80  | 375 | 0.10 | 0.02 | 100 | 80  | 28 | 91  |
| S10| 29       | 92  | 380 | 0.16 | 0.01 | 80  | 36  | 28 | 47  |
Influence of content montmorillonite mineral on geotechnical parameters of expansive soil in Bhavnagar. (a) LL, (b) PI, (c) swell pressure, (d) c index, (e) shearing strength [4]

3. Influence of over-consolidation ratio
All the experiments and theories show that the over-consolidation ratio (OCR) has positive effect on the shearing strength. That is, shear strength increases with increasing OCR. The first to deal with this method were [9]. Indraratna et al [10] have studied the variation of shear strength with different values of the height of the thickness of infill proportion (t/a) with respect to OCR. The relation between the over consolidation ratio and the shear strength was also studied by [11] and [12].

3.1. Graig and Chua (1990)
It is considered that the dimensionless group (S_u/γD) is important and provides information about the shear strength [12]. S_u represents the un-drained shearing strength for the soil, γ represents effective density of
the soil and D represents dimension of foundation. It is recognized as the stability number and is useful analyze the over-consolidation ratio of clays. Equation 1 expresses the plan strain un-drained shearing strength of the clay soil in the form provided by [13].

\[
\frac{S_{u}}{\sigma'_{v}} = \left[\frac{S_{u}}{\sigma'_{v}}\right]_{n_{c}} \cdot \text{OCR}^{A}
\]

The value for A is typically 0.8 and taken to be 0.8, while the value of the normally consolidated shear strength is generally 0.3. Thus, the previous form will be as follows with these values (equation 2):

\[
S_{u} = 0.3 \sigma'_{v} \cdot \text{OCR}^{0.8}
\]

The effective normal stress at the top surficial area of clay layer will be \(\gamma D\), so the shear strength at the top surface will be (equation 3):

\[
\frac{S_{u}}{\gamma D} = 0.3 \cdot \text{OCR}^{0.8}
\]

The shear strength ratio can be evaluated depending on the appropriate values for the over-consolidation ratio. Table 3 provides the estimation of shearing strength ratio and corresponding over-consolidation ratio.

| OCR  | \(S_{u} / \gamma D\) | \(G_{c} / S_{u}\) |
|------|----------------------|-------------------|
| 1    | 0.3                  | 160               |
| 4.5  | 1                    | 79                |
| 25   | 4                    | 28                |
| 60   | 8                    | 25                |
| 100  | 12                   | 23                |

3.2. Indraratna et al. (2008)

Indraratna et al [10] studied the influence of the height with infill thickness ratio (\(t/a\)) on the shearing strength of clays. If the value of \(t/a\) is determined to be less that the critical value, then part of the failure plane can possibly move across the interface. Conversely, failure surface will be within the infill itself. Indraratna et al [10] noted that for smaller values of \(t/a\) the shear strength of asperities was affected by normal stress, which was applied on the joint. They said that for high normal stress, shearing could be occurred in the asperities.

Different critical \(t/a\) ratios that exceed unity could be obtained for different types of infill. The over-consolidation ratio can also influence the ratio, which will also affect the shear strength. Figure 5 represents how the normalized shearing strength varies with \(t/a\) proportion of a specific in filled joint for different values of the over-consolidation ratio. It is obvious from figure 5 that the critical \(t/a\) ratio decreases if over-consolidation ratio increases. The critical ratio \((t/a)_{cr}\) of infilled saw-toothed joints with variety of over consolidation ratio could be clarified in relationship as shown in equation 4:

\[
(t/a)_{cr,n} = f ((t/a)_{cr,1}, \text{OCR})
\]

In Equation 5, \((t/a)_{cr,1}\) is the critical value of \(t/a\) for OCR equal to 1 and thus, \((t/a)_{cr,n}\) is the critical value of \(t/a\) for OCR equal to n. The ratio \(k_{oc,n}\), which is shown in the Figure 6, is showed as

\[
k_{oc,n} = (t/a)_{oc,n} / (t/a)_{cr,n}
\]

where,
(t/a)_{oc,n} is t/a ratio of the given ratio with OCR equal to n, and
(t/a)_{cr,n} is critical t/a ratio

The shear strength, which has different values with different over-consolidation ratios as illustrated in Figure 6, could be classified as two main zones (interfering and non-interfering) based on the values of K_{oc,n} ratio.
If \( K_{oc,n} \) is more than 1 (non-interfering), the behavior of joint will be a function of infill alone. Figure 6 shows the relation between normalized shear strength and the OCR for infilled joints which have high infill thickness that was proposed. The shearing strength in the non-interfering region can be expressed as in Equations 6 and 7, where \( \alpha \) is an empirical constant.

\[
\log \left( \frac{W_p}{V_n} \right)_{oc,n} = \log \left( \frac{W_p}{V_n} \right)_{oc,1} + D \log(OCR)
\]

(6)

That is,

\[
\left( \frac{W_p}{V_n} \right)_{oc,n} = \left( \frac{W_p}{V_n} \right)_{oc,1} \times OCR^\alpha
\]

(7)

In the region of \( K_{oc,n} < 1 \), the shear strength of infilled saw-tooth joints is revised for over-consolidation ratio value of \( n \) with using the algebraic functions \( A_n \) and \( B_n \) shown in Figure 6.

3.3. Jamiolkowski et al. (1985)

Jamiolkowski [11] preformed field vane shear tests to estimate the un-drained shear strength. The results they obtained were plotted as the OCR versus un-drained shearing strength to the vertical consolidation stress \( C_u/\sigma_v' \). The testing was conducted at nine different sites for several types of clays. The log-log plots shown in Figure 7 contain the results they obtained and the corresponding equations. The results were quite similar to the results obtained by [13] using simple shear test. The studies reveal that the higher the value of over-consolidation ratio, the larger the strength obtained from the vane shear test.

3.4. Amin et al. (1997)

Amin et al [14] discussed SHANSEP procedure to predict the undrained shear strength of soil. SHANSEP procedure is introduced firstly by Ladd [9]. SHANSEP is based on the theory that soil parameters can be normalized by reduce them to non-dimensional numbers. For example, by dividing \( S_u \) by the confining pressure of the test or the field effective stress, a normalized parameter can be obtained. In this method, as SHANSEP shows, the over-consolidation ratio, which represents the stress past behavior, has a clear effect on the shearing strength of soil. A graph of \( S_u \) versus over-consolidation ratio is required for use in this method. Over-consolidation ratio can be estimated by determining the maximum consolidation pressure applied in the past. The prediction of \( S_u \) could be done by Plotting normalized \( S_u \) versus OCR. SHANSEP study was completed in order to plot \( S_u \) versus the OCR for soft marine clay and this plot was used for the
design and construction. Figure 8 illustrates OCR versus $S_u$ relation for Klang clay using the Direct Simple Shear device. Other common clays are also included in the figure showing that the Klang clay plot follows a pattern similar to other curves.

![Figure 8. Normalized Su versus OCR Curves for Several Commonly Reported Clays [14].](image)

4. Effect of Drained Versus Undrained Loading
The following summaries of experiments explain the basic ideas behind the behavior of drained and undrained clay specimens.

4.1. Bobet (2010)
Bobet [15] conducted numerous simulations to evaluate the deformation of a rectangular tunnel under far-field shear stress in an elastic medium for drained and undrained response. Moreover, the slipping situation between the soil and structure boundary (full-slip and no-slip) was taken in considerations. He states that the opening shape has little effect on the structures deformation, the full-slip situation results in less deformation, the undrained conditions decrease deformation when the structures is elastic, and the deformation increases with rigid structures. Simulations were also conducted for circular structures to have a comparison for the rectangular structures results (Figure 9). The drained conditions for these simulations were achieved when no excess pore water pressure existed in the ground. The undrained conditions for these simulations were conducted when the excess pore water pressure was produced and did not disperse fast enough. Static conditions were assumed for the formulations and simulations. The simulation results showed that the shape of opening was neglected in the deformation on rectangular structure in case of both drainage condition, drained and undrained. The principle factor that needs to be considered for subterranean structures is the relative elasticity between the structure and soil. The structure that is rigid will have less deformation than a structure that is more elastic. The paper uses Wang’s flexibility proportion to represent the relative elasticity between the soil and structures (Figure 10).
Overall, it can be noticed that the deformation of the rectangular structures in case of full-slip is less than that of no-slip medium. There is also a higher deformation in drained conditions, for all full-slip and no-slip simulations, in comparison to the undrained conditions. As shown in the above figures, the flexibility verses deformation with drained or undrained conditions is different for circular structures and rectangular structures. The results showed that the slip conditions have negligible effect on the deformation value under undrained condition whilst this the effect of slip conditions is clearer under drained conditions. Since the full-slip medium of the circular structure produces higher deformation than that of no-slip one

4.2. Alawaji et al. (1992)

Alawaji et al [16] studied the stress and strain variables for isotropic plasticity soils. Both drainage conditions, drained and undrained, are taken in consideration. To verify equilibrium of pre-estimated stresses that correspond with the strain response variables, the drained conditions response variables are determined by iterations. Then again, the undrained conditions strains and pore water pressures are calculated by simultaneous iterations to fulfill equilibrium and the incompressibility condition. The paper applies these conditions to generalized cam-clay models with various iterations.

The control mode is initiated as the triple \( C_i \), where \( C_i = 1 \) means strain control, whereas \( C_i = 0 \) means stress control in the principal direction. The different loading programs are defined as drained conventional triaxial compression (DCTC) and drained true triaxial (DTT). It follows that DTT1 is defined by pure strain control, whereas DTT2a and DTT2b are defined by pure stress control, which represents the least confined of any control mode. Figure 11 shows the converged stress-strain behavior for DTT2a and DTT2b. The remaining assessments concern the iteration efficiency in various modes of mixed control.

Finally, a comparison is made for DTT2b of the convergence behavior for different control modes. The various loading programs that have been investigated for un-drained behavior are: un-drained triaxial compression (UCTC), un-drained plane strain (UPS), and un-drained true triaxial (UTT). The same initial stress state and material parameter values as for the drained case DCTC are used. Figure 12 shows the
converged undrained response functions for the loading programs UCTC and UPS. The stress-strain curves that are shown in Figure 12, especially for UPS, are characteristic for undrained behavior of normally consolidated clay. Finally, a comparison of the convergence behavior for different control modes is carried out for the program UTT2b, the last three control models are carried out for the program UTT2b, where the last three control modes involve two prescribed strain components.

Since the incompressibility condition directly gives the third strain component, this case effectively corresponds to pure strain control. The discussed integration technique for mixed control is based on a strain-driven core algorithm, while equilibrium and incompressibility (for the undrained case) are satisfied iteratively. The algorithm was found to be efficient and robust at the application to a generalized cam-clay model.

The general conclusion for drained behavior seems to be independent of the degree of stress control, although the number of required iterations increases for all iteration methods with increasing degree of stress control. For undrained behavior the difference in efficiency between the various iteration methods seems less dramatic. Generally speaking, the constraint of incompressibility, that is, pertinent to undrained behavior, has a beneficial influence in the sense that the required number of iterations is smaller than for drained behavior.

![Figure 11. Converged Stress-Strain Response for the Drained True Triaxial Tests Defined in Loading Programs DTT2a, e=1.0 and DTT2b, e=0.65 [16].](image1)

![Figure 12. Converged Un-drained Stress-Strain Response for Un-drained True Triaxial Tests, Defined in Loading Programs UTT2a and UTT2b [16].](image2)

5. Conclusion

Three of the most significant factors could affect the shear strength of clay were examined in this study. These factors are:

1. Mineralogy.
2. Over-consolidation ratio.
3. Drained versus undrained loading.

The conclusion of the main points of this literature could be summarized in the following points:

1. There are number of techniques to increase the shear strength of soil mass
2. The montmorillonite presence in the soil mass has greatest effect on the shear strength of clay than other clay minerals.
3. Liquid limit and residual friction angle of the soil are greatly varied due to the clay minerals type despite particle size distribution analysis shows same CF.

4. The over consolidation ratios OCR have a positive influence on the shearing strength of soil, in other words, higher OCR higher shear strength.

5. Soils loaded under drained conditions exhibits higher shear strengths than those loaded under undrained conditions.

6. References

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