# Degree of Initial Saturation and Shear Strength of Soil in a Triaxial Loading

[1]Palani Kumar A, [2]Suresh Maurya, [3]Dr. Manish Gupta, [4]Dr. R.Chitra

[1]Scientist B, [2]Scientist D, [3]Scientist E, [4]Scientist E
[1]Central Soil and Materials Research Station,
[1]Ministry of Jal Shakti, D/o Water Resources, RD & GR, New Delhi.

***Abstract:*** Shear strength is the basic strength properties of soil which controls the stability of a soil mass under loads. The shear strength of soil helps to design the foundation of all the civil engineering structures, stability of slopes, earth retaining walls, embankments, man-made excavation etc. All the problems of soil engineering are related in one way or the other with shear strength of the soil. The triaxial shear test is one of the precise methods to determine the shear strength of the soil. Here the loading is more often triaxial than biaxial which simulates, the field loading. In field saturation level of soil may vary and depends upon climatic condition, drainage condition, porosity, density of particle etc., which may affect the shear strength of soils. The shear strength parameters of partially saturated to nearly saturated compacted soils under varying degree of initial saturation, S (%) were determined through triaxial shear test. In doing so consolidated undrained (CU) test was carried out through measurement of B-parameter coefficient (B-factor). This study helps to plot the variation of shear strength under different S (%) of silty-clay type of soil.

***Index Terms:*** **Triaxial, Cohesion, Saturation, Soil.**

## 1. INTRODUCTION

The top of earth crust is filled with soil. The soil is the primary construction material for earth retaining structures, embankment dams and all foundation of the Civil Engineering structures resting on the soil. Shear strength is the most important geotechnical property of soils which helps in stability of structures on or below the earth. Hence, study of the shear strength of soil is very important for stability of the structure.

Shear strength is an important strength property of soil, which helps designer to design any geotechnical structures. The shear strength is an internal resisting force to arrest movement of the soil particles. The shear stress is developed both by applied load and overlying soil itself. If the shear stress in soil exceeds the shear strength, soil structure will fail. The soil resist external load by its shear resistance in terms of cohesion '*c*' (cementing or bonding between particles) and angle of internal friction '$\phi$' (interlocking of particles). The shear strength can be evaluated in terms of total stress and effective stress. In consolidated drained test, effective stress result is obtained because during shearing pore water is released. Whereas in consolidated un-drained test with measurement of pore water pressure the drainage valve is closed during shear and pore water pressure is noted, in which total and effective strength parameters can be obtained.

The shear strength is influenced by soil density, mineralogy, grain size, shape of particle and drainage condition. If the density of sample is more, the shear strength also increases. The presence of montmorillonite minerals in soil particles, increases cohesive force '*c*'. When clay is dominant in soil mass, the maximum shear strength is taken by cohesive force 'c' and when sand is dominant in soil mass, then the maximum shear strength is taken by angle of internal friction '$\phi$'. The test results may vary and depends largely on the drainage condition and therefore adopting or simulating the field drainage condition in laboratory can yield better results.

Mohr-Coulomb failure theory is valuable function in analysis of the shear strength of soil. According to Mohr, the failure is caused by a critical combination of the normal and shear stress. Figure 1 shows the Mohr's graphical representation of stresses when a soil element is subjected to principal stresses ($\sigma_1$ and $\sigma_3$).

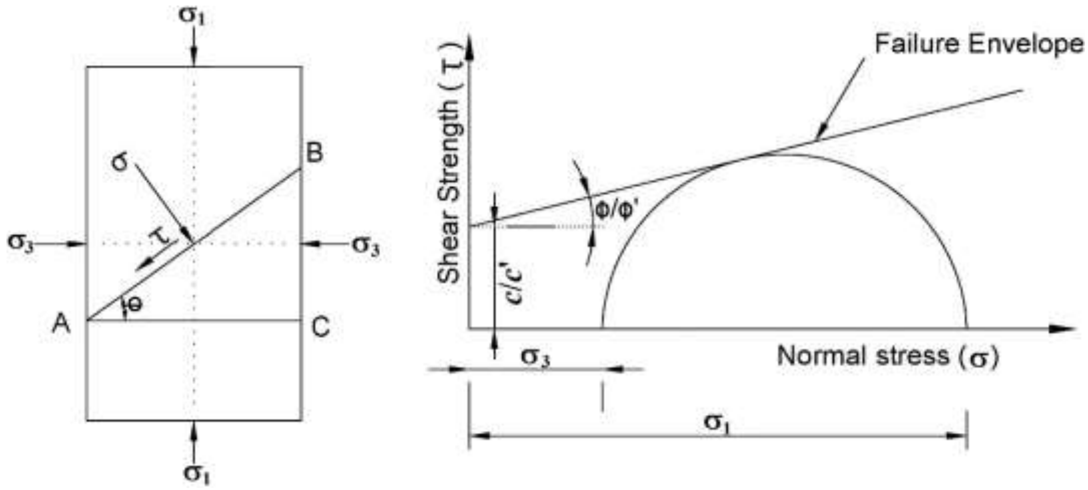



**Fig. 1.** Mohr's graphical representation of stresses

Mohr-Coulomb failure envelope is represented by a straight line. It represents shearing resistance of soil linearly related with '$\sigma$'. Here '*c*', equals to the intercept on $\tau$-axis and '$\phi$' is the angle which the envelope makes with the $\sigma$-axis.

$$\text{Total shear strength, } \tau = c + \sigma \tan \phi$$

Parameter 'c' and '$\phi$' depends upon the number of factor such as water content, drainage conditions and conditions of testing. Actual stresses which control the shear strength of a saturated soil (i.e. when degree of saturation is 100 %) are effective stresses and not the total stresses. Thus above equation is modified as Mohr-Coloumb equation for the shear strength of soil as.

$$\text{Effective shear strength, } \tau' = c' + \sigma' \tan \phi'$$

Where,

$c/c'$ = Cohesion for total and effective stress

$\phi/\phi'$ = Angle of internal friction for total and effective stress

$\sigma_3$ = Minor principle stress

$\sigma_1$ = Major principle stress

Hossne Garcia et al. [1] studied the relationship between the shear strength and compaction of sandy loam soils under varying water content. Soil strength increased as compaction increased in the soil compaction-water characteristic curve, the optimal soil shear strength took place before the optimal compaction occurred. The effect of moistness, weakening the shear strength was greater than the effect of dry bulk density strengthening shear strength.

Rohit Ghosh [2] studied the effect of moisture content of undrained shear strength of compacted clayey soil by vane shear test. Shear strength curve drawn for different compaction effort showed exponential decrease in the shear strength with gradual rise in water content.

Yong Wang et al. [3] conducted triaxial test of loess soil under different drainage condition (CD and CU test). Stress-strain curves obtained from CD and CU test show similar characteristics. However, Strength parameter obtained from CU test are larger than CD test. Result from the tests shows that with an increase in moisture content, the stress-strain curve of loess shows strain softening behaviour. The cohesion decrease slightly with the increase of moisture content, when it is lower than the plastic limit, while when the water content exceeds the plastic limit, the cohesion decreases significantly. The internal friction angle did not decrease significantly with the change of water content. When the water content is less than 20 %, the secant modulus increases significantly with the increase of confining pressure, while the secant modulus changes slightly at the moisture content of saturated condition.

Sakuro Murayama et al. [4] conducted large number of tests such as U-test, vane test and triaxial compression test to research the effect of the moisture content on the strength characteristic of undisturbed saturated clay. Liquid limit can be taken as a principal factor which influences the strength of clay with the moisture content. The compressive strength of fully saturated clay has a linear relationship to the moisture content on the semi-logarithmic paper, and the above linear plotting is parallel to the virgin compression line of the consolidation test.

Y. Luo et al. [5] conducted direct shear test on loess soil and studied the relationship between shear strength and moisture content. The results show that when the moisture content is greater than the limit moisture content, the matric suction and shear strength decrease with the increase in moisture content of loess. However, when the moisture content is lower than the limit moisture content, the current understanding on the relationship between water content and matrix suction is not appropriate, and results show that the matric suction and shear strength increase with the increase in loess moisture content.

Muawla A. Dafalla [6] studied the effect of different proportions of clay content to sand with moisture content using direct shear test. The cohesion of the mixture was found to increase consistently with the increase of clay content. Increase in moisture content was found to cause a drop in both cohesion and angle of internal friction.

Soil mass generally referred to as a three-phase system consists of solid particles, water and air. It becomes a two-phase system when the soil is absolutely dry or when the soil is fully saturated. It is the relative proportion of these three constituents and their interaction that governs the behavior and properties of soils. It is obvious to say that with an increase in the confining pressure there is decrease of air in voids and degree of saturation increases. This result, in reduced shear stress to cause failure.

In field saturation level of soil may vary and depends upon climatic condition, drainage condition, porosity, density of particle etc., which may affect the shear strength of soils. If the volume of air is relatively small, the soil may get saturated itself under stresses. If the air content is very large, the soil remains unsaturated and undergoes large volume changes even in undrained conditions. Therefore it is important to understand the relationship of varying water content and shear strength of soil in partially saturated to saturated soils in terms of total stresses. As a result consolidated-undrained triaxial tests are conducted on a number of samples with a given degree of initial saturation, S (%).

## 2. MATERIALS AND PREPARATION OF SPECIMENS

The packing of remoulded samples, its extraction from mould and preparation of specimen in the triaxial cell must be carried out with limited disturbance. Sample disturbance can significantly affect the results obtained from the test.

### 2.1 MATERIALS

To study the effect of varying degree of initial saturation on the shear strength of soil, series of triaxial tests were conducted on silty clay of medium compressibility soil. From the test results of specific gravity (2.73) and maximum dry density (1.65 g/cc), the saturated moisture content (SMC) is calculated as 25 %.

### 2.2 PREPARATION OF SPECIMENS

To appropriate the field condition, samples are packed at 98 % relative compaction with utmost care. The poor quality samples may affect laboratory results. The nominal diameters of the samples prepared are 38 mm and 76 mm height. Height to diameter ratio of 2 is maintained for all sample specimen as per IS code [7]. It is also ensured that ratio of diameter of specimen to maximum particle size should not be less than 5. A de-aired, porous stone along with filter paper disc is placed at both the end of specimen. De-aired vertical filter strips are also place at regular spacing around the entire periphery touching both porous stones to induce radial drainage. Porous stones provide free drainage surface to the pore water. Filter paper disc prevent the stones from becoming clogged. Cylindrical specimen is placed between rigid caps, covered with latex membrane and secured with rubber O-ring as shown in Fig. 2. Transparent perspex cylinder is then secured on a triaxial cell apparatus which is filled with water.

**Fig. 2.** Preparation of specimen in triaxial cell apparatus

Initially disturbed sample is processed (mixing water with sample) and kept in air tight container for minimum 16 hours for proper maturing. Four soil samples from same material are prepared with different water content. Fourth sample is prepared at SMC. Four specimens for each sample were packed at their respective sample water content. The initial degree of saturation, S (%) obtained from B-par coefficient for the first three samples were less than 90 %, while in the fourth sample it was more than 90 % as shown in Table 1. This is to investigate the shear strength of compacted soil under varying degree of initial saturation and model it somewhere between the partially saturated state to nearly saturated state of soil.

**Table 1.** Varying degree of initial saturation S (%)

| Sample no. | Degree of initial saturation, S (%) |
|---|---|
| 1 | 14 |
| 2 | 34 |
| 3 | 54 |
| 4 | 95 |

## 3. TESTING MACHINE AND TEST METHOD

To substantiate the effect of degree of initial saturation, S (%) on the shear strength of compacted soil, series of triaxial tests were conducted as per IS-2720 (Part 12) in semi-automatic triaxial compression machine. In doing so consolidated undrained (CU) test were carried out, with measurement of S (%) through B-parameter coefficient, before shearing of specimens.

### 3.1 TESTING MACHINE

Semi-automatic triaxial compression machine consist of a strain controlled loading frame operating in the range 0.00096 to 3 mm/minute, a triaxial cell setup, and pressure chambers. Machine is attached with readout devices like load cell, LVDT and pressure transducer for measuring load, strain and stress respectively. Load cell has a least count of 0.01 kgf, its capacity varies from 0 to 250 kgf, LVDT (Linear variable differential transformer) with least count of 0.001 mm, its measuring range varies from 0 to 40 mm and pressure transducer with least count of 0.001 psi, its pressure range varies from 0 to 200 psi. All the three readouts devices are connected with data acquisition system (DAS). DAS is interfaced with a computer system along with a software which displays test parameters, graphs and diagnostic messages etc. Figure 3 shows the semi-automatic triaxial machine setup. Triaxial laboratory consist of four machines in one row to test the specimens at four different confining pressures.

**Fig. 3.** Semi-automatic triaxial machine

### 3.2 TEST METHOD

Triaxial test involves four process 1) Preparing of specimen at desired density in a mould and packing of specimen in triaxial cell apparatus, 2) Checking the degree of saturation by B-parameter co-efficient, degree of saturation is preferably 100% but should not be less than 90% for saturated state condition (if needed assistant of back pressure system may be taken to saturate the specimen), 3) Consolidation at incremental confining pressures ($\sigma_3$) and 4) Shearing the specimen using strain control compression machine.

Specimen in the triaxial setup is subjected to the confining pressure ($\sigma_3$) by applying pressure to water in the cell through air compressor. During the CU test condition, the specimen is allowed to consolidate in the first stage and drainage is permitted until the consolidation is completed. In the second stage when specimen is sheared, deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) is recorded in the DAS with the help of pressure transducer until the specimen fails (see Fig. 4). No drainage is permitted in this stage.

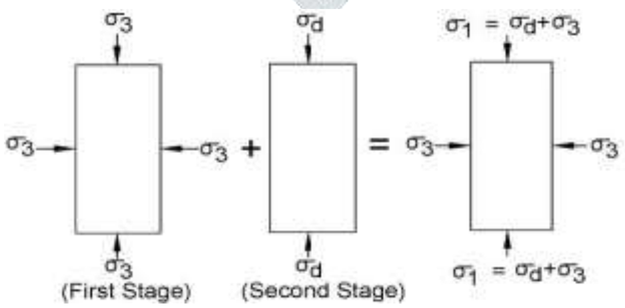



**Fig. 4.** Cylindrical specimen subjected to stresses in CU test

Here,

$\sigma_1$ = Major principle stress acting on major principle plane
$\sigma_3$ = Minor principle stress acting on minor principle plane (confining pressure)
$\sigma_d$ = Deviator stress ($\sigma_d = \sigma_1 - \sigma_3$)

The tests are performed on minimum three or more number of identical samples at incremental confining pressures and deviator stress is recorded until the specimen fails. Failure is defined usually when the deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) reaches the maximum (peak) value. When the peak stress was not well defined stress at 20 % strain may be considered as failure stresses Results are plotted and '*c*' and 'ϕ' are determined from the best fit line of modified failure envelope. Figure 5 shows the condition of specimen before and after the test.

**Fig. 5.** Specimen before and after the test

## 4. CALCULATIONS

The deviator stress-strain graph obtained during the test are helpful to understand the behaviour of soil during the test. A modified failure envelope is plotted between 'p' and 'q' values at failure for each incremental confining pressures and their respective peak deviator stress as shown in Figure 6.

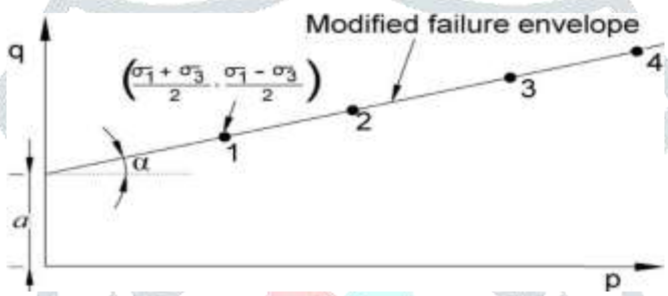



**Fig. 6.** Modified failure envelope, (p-q plot)

The coordinates of the top point of the Mohr circle corresponding to the maximum stresses, represented as 'p' and 'q'.

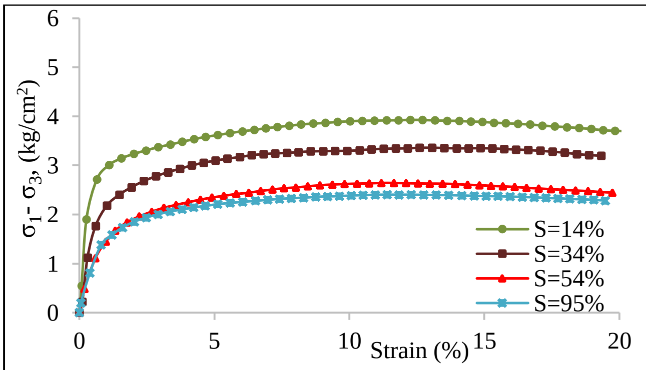
$$\text{Where, } p = \frac{(\sigma_1+\sigma_3)}{2} \text{ and } q = \frac{(\sigma_1-\sigma_3)}{2} \quad \dots (1)$$

A line is drawn through theses points to make an angle 'α' with the p-axis and an intercept '*a*' on the q-axis and has the line equations as q = *a* + p tan α. Comparing with the coulomb equation of $\tau = c + \sigma \tan \phi$, the values of shear strength parameters '*c*' and 'ϕ' are obtained from the intercept '*a*' and slope 'α', using below equations.

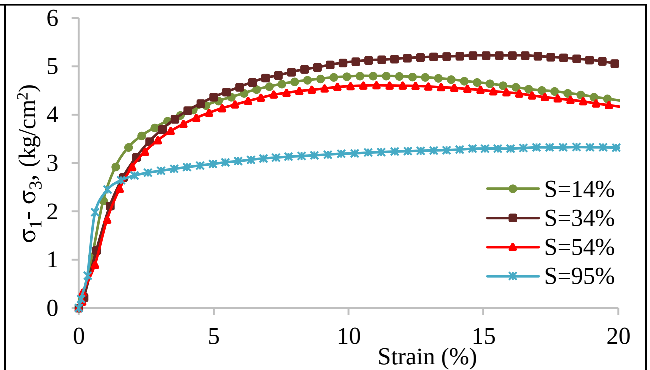
$$c = \frac{a}{\cos\phi} \text{ and } \phi = \sin^{-1}(\tan \alpha) \quad \dots (2)$$

## 5. RESULTS AND DISCUSSION

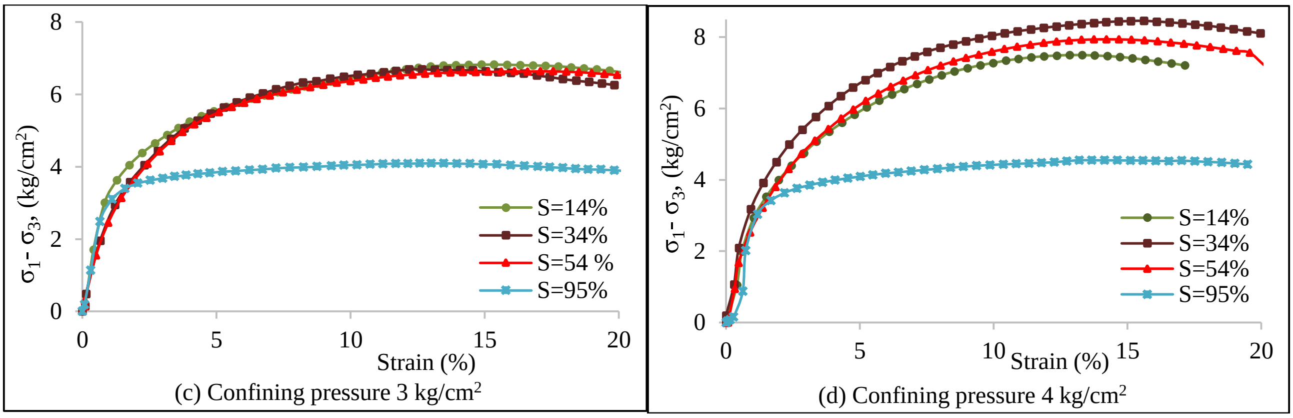
Deviator stress-strain graph obtain from experimental work for different samples at confining pressures of (a) 1 kg/cm$^2$ (b) 2 kg/cm$^2$ (c) 3 kg/cm$^2$ and (d) 4 kg/cm$^2$ are shown in Fig. 7.




(a) Confining pressure 1 kg/cm$^2$




(b) Confining pressure 2 kg/cm$^2$




(c) Confining pressure 3 kg/cm²  (d) Confining pressure 4 kg/cm²

**Fig. 7.** Deviator stress-strain graph for different samples at confining pressures of (a) 1 kg/cm² (b) 2 kg/cm² (c) 3 kg/cm² and (d) 4 kg/cm².

For partially saturated state of samples having S (%) 14 %, 34 % and 54 %, the strain softening characteristics of stress-strain curves are similar, and the reduction of deviator stress is not very large. When the S (%) for nearly saturated sample is 95 %, the strain hardening takes place early and the deviator stress to cause failure decreases sharply. The deformation characteristic of sample with S (95 %) are mainly dominant by moisture content and behave differently than other three samples having lesser S (i.e., 14 %, 34 % & 54 %).

Maximum normal stress (p) and maximum shear stress (q) values are calculated using Eq. 1 and tabulated in Table 2. A modified failure envelope is plotted between 'p' and 'q' values as shown in Figure 8. Failure envelope for fourth sample at higher S (%) is more towards horizontal than other three samples having lesser S (%). With substantially increase in degree of saturation, the shear stress to cause failure decreases considerably and failure envelope tends to come down.

**Table 2.** Maximum normal stress (p) and maximum shear stress (q)

| Confining pressure, $\sigma_3$ (kg/cm²) | Sample 1 | | Sample 2 | | Sample 3 | | Sample 4 | |
|---|---|---|---|---|---|---|---|---|
| | At = 14 % | | At = 34 % | | At =54 % | | At = 95 % | |
| | p | q | p | q | p | q | p | q |
| 1.00 | 2.96 | 1.96 | 2.68 | 1.68 | - | - | 2.20 | 1.00 |
| 2.00 | 4.40 | 2.40 | 4.61 | 2.61 | 4.30 | 2.30 | 3.66 | 1.60 |
| 3.00 | 6.41 | 3.41 | 6.35 | 3.35 | 6.32 | 3.32 | 5.05 | 2.05 |
| 4.00 | 7.75 | 3.75 | 8.23 | 4.23 | 7.97 | 3.97 | 6.28 | 2.28 |

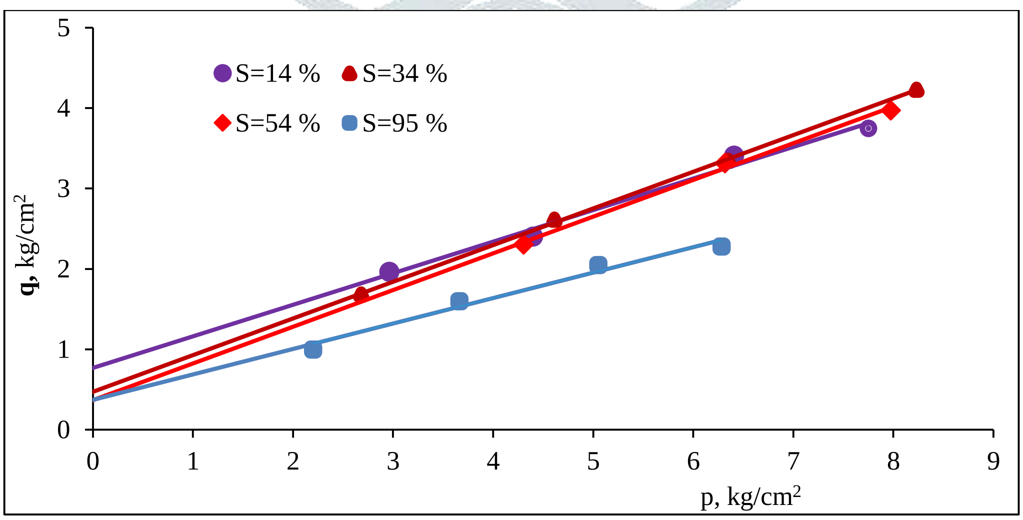



**Fig. 8.** 'p-q' plot

Shear strength parameters '*c*' and 'ϕ' are obtained from the intercept '*a*' and slope 'α' of modified failure envelopes using Eq. 2 for all the samples tested at varying degree of S (%). Shear strength parameters are tabulated in Table 3.

**Table 3.** Shear strength parameters

| Sample no. | Degree of saturation, S (%) | $c$ (kg/cm$^2$) | $\phi$ (Deg.) | Assumed normal stress, $\sigma_n$ (kg/cm$^2$) | Shear strength, $\tau$ (kg/cm$^2$) | Effect of '$\phi$' on $\tau$ |
|---|---|---|---|---|---|---|
| 1 | 14 | 0.835 | 23.10 | 4.0 | 2.543 | 67.1 % |
| 2 | 34 | 0.532 | 27.14 | | 2.582 | 79.4 % |
| 3 | 54 | 0.414 | 27.20 | | 2.467 | 83.2 % |
| 4 | 95 | 0.302 | 18.46 | | 1.637 | 81.6 % |

Figure 9 shows, the plot of variation of shear strength parameters with degree of saturation, where '$\phi$' curve achieved similar profile of shear strength ($\tau$) and '$c$' curve achieved inverse profile. Figure shows that '$\phi$' component plays an active role in controlling the shear strength of soil (see effect of '$\phi$' on '$\tau$' in Table 3). Change in '$\phi$' is similar to the compaction curve of soil. When S (%) reaches optimal degree of saturation, '$\phi$' is the largest. When S (%) is lower than the optimal, '$\phi$' increases with increase in S (%). When S (%) is higher than the optimal, '$\phi$' decreases with increase in S (%). To certain extent there is a linear relationship between '$\phi$' and shear strength. However, cohesion decreases continuously with increase in S (%) and has little effect on shear strength of soil.

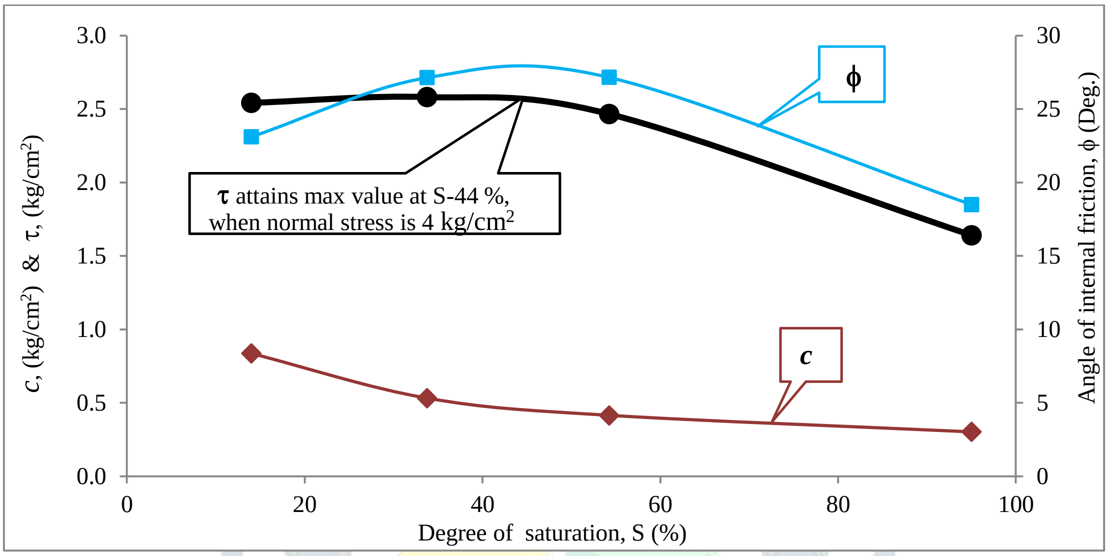



**Fig. 9.** Variation of shear strength parameters with degree of saturation

There is a slight increase in shear strength initially with an increase in degree of saturation and reach maximum value at optimal degree of saturation. With further increase beyond optimal value, shear strength decreases drastically. Figure 9 shows, shear strength curve of a soil under varying degree of S (%) when subjected to a normal stress of 4 kg/cm$^2$.

## 6. CONCLUSIONS

This study helps to plot the variation in shear strength of silty-clay type of soil under varying degree of saturation by triaxial compression test under consolidated undrained (CU) condition. Strain hardening reaches early at lower shear stress in nearly saturated state of soil when compared with partially saturated state of soil. The deformation characteristic are mainly dominated by increasing confining pressure and effected by increasing degree of saturation. Shear strength initially increases with an increase in degree of saturation and reach maximum value at optimal degree of saturation. With further increase beyond optimal value, shear strength decreases drastically. Change in shear strength of soil follow similar profile of angle of internal friction ($\phi$). For the tested silty-clay type of soil, maximum '$\phi$' (27.3$^{\circ}$) occurs at optimal degree of saturation (44 %). Increase or decrease in water content reduces '$\phi$' value. Angle of internal friction ($\phi$) curve achieved similar profile of shear strength ($\tau$) and cohesion ($c$) curve achieved inverse profile. It indicates that '$\phi$' component plays an active role in controlling the shear strength of silty-clay type of soil. However, cohesion decreases continuously with increase in S (%) and has little effect on shear strength of soil.

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