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Geotechnical Properties on Residual Soil of Sedimentary Rock

Farah Wahida Mohd Latib[1,*], Anuar Kasa[2], Mohd Fairuz Bachok[3]

[1] Civil Engineering Studies, College of Engineering, Universiti Teknologi MARA Pahang Branch, Bandar Pusat Jengka, Pahang, Malaysia
[2] Department of Civil and Structural Engineering, Universiti Kebangsaan Malaysia, Bangi, Selangor, Malaysia
[3] Civil Engineering Studies, College of Engineering, Universiti Teknologi MARA Johor Branch, Bandar Seri Alam, Masai, Johor, Malaysia

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ABSTRACT

The residual soil is one of the major soils in the peninsular region of Malaysia. Because of this, engineering activities can employ this kind of residual soil in the building industry, such as retaining walls and high-rise building. Yet, because it is structurally complicated and has substantial weathering in most locations, the residual soil often brings in heterogeneities, which can lead to the failure of a structure. Finding the value of the shear strength parameters as a geotechnical properties is the key to solving this problem, which may be done here. After the soil samples had been compacted, they were put through a series triaxial tests, namely Consolidated Drained (CD) and Consolidated Undrained (CU) tests. A total of eight soil samples, four for each test were used for this study. As a consequence of the tests that have been carried out, the value of the cohesion parameter, c', as recorded by the CD test is greater than that recorded by the CU test. On the other hand, the value of the friction angle, ϕ ', as recorded by the CU test is greater than that recorded by the CD test. In the meantime, with regard to the stress-strain behaviour, the brittle failure pattern was seen in each and every sample that was put through the CD test. The pattern that was seen in the results of the CU test was also seen in the results of the CD test. The output of this study makes it abundantly evident that the maximum values for pore water pressure, lateral stress, and axial strain would be significantly increased if the effective confining stress that was applied to the sample was increased to a higher value.

1. Introduction

Geotechnical engineering is the most essential subfield of engineering in the building industry. This is due to the fact that the even distribution of loads on the ground plays an important part in maintaining the stability of the created structure [1]. Residual soil is one of the forms of soil that is frequently utilized, although there are certain issues with using this type of soil. According to the findings of many researchers [2-11], residual soil has a lot of potential for uncertainty because the degree to which weathering progresses at varying rates throughout the various layers of this soil. On the other hand, given that [2] found that over 70 percent of the land in Malaysia is composed of residual soil, construction projects are compelled to make use of this soil as the primary medium.

* *Corresponding author.*
E-mail address: farahwahidaml@uitm.edu.my

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According to Phai and Amin [12], soil that has weak geotechnical properties will influence the structure that is built on it. As a result, the soil either needs to be replaced with other materials that are stronger or it needs to be stabilized.

Soils are residual if they are generated from rocks or accumulations of organic matter and then continue to exist in the same location in which they were initially formed without migrating to another location. According to Blight and Eng [13] residual soil is produced because of a complicated naturally occurring in-situ weathering process. This weathering of rocks may be broken down into three distinct categories: physical weathering, chemical weathering, and biological weathering. The production of residual soil will be affected differently depending on the level of weathering that occurred. Clay will be created in greater quantities in direct proportion to the depth of weathering, which in turn is directly proportional to the amount of weathering that has occurred [14]. The greater the depth of the residual soil, the greater the degree of weathering, and the greater the proportion of clay in the soil. Because of this, the sizes of the soil distribution will vary according to depth as a result of the influence of the degree of saturation and the type of minerals that were generated [15].

The weathering process of sedimentary rocks results in the formation of residual sedimentary soil. Sedimentary rocks are generated because of the build-up of sediment, hence weathering of these rocks results in the formation of sedimentary soil. The sedimentary rocks can be divided into three distinct types: organic, clastic, and chemical. The lithification process of weathered rock fragments that have been physically moved and deposited in lower places such as rivers, lakes, or seas results in the formation of these rocks. Figure 1 shows the division and classification of land in Peninsular Malaysia. It is made abundantly evident here that residual soil can be split up into three distinct groups, namely granite residual soil, sedimentary residual soil, and soil that was created from volcanic ash.

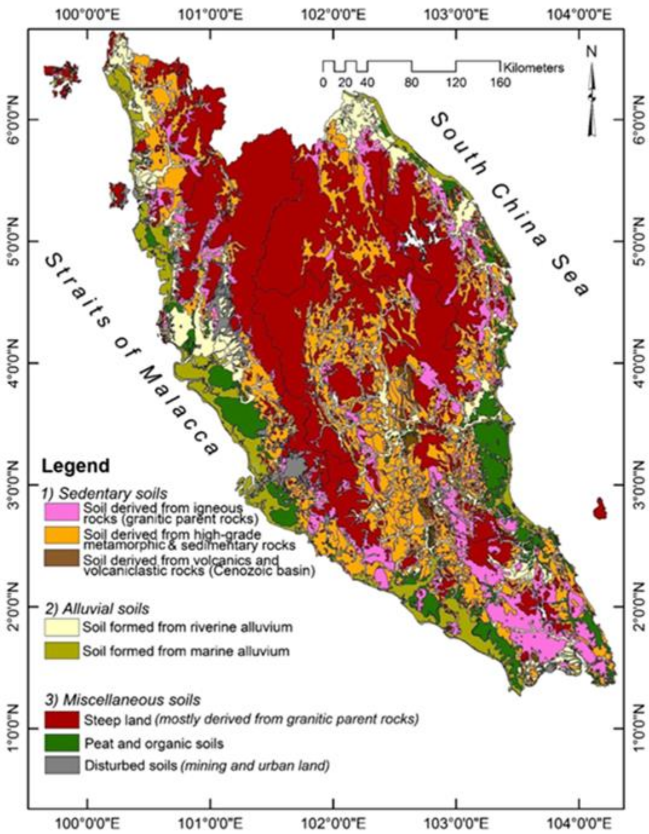

Fig. 1. Classification of land in Peninsular Malaysia [16]

Shear strength is one of the most important engineering features in soil, and the strength of soil is the result of the resistance that failure brings to the movement of soil molecules that are connected to one another [17,18]. In geotechnical engineering, shear strength parameters such as the value of apparent cohesion, c' and effective friction angle, ϕ' are extremely helpful in the process of designing geotechnical structures for both safety and economic geotechnical structure design. These parameters are found in shear strength values.

The values of these shear strength parameters are affected by a wide variety of factors, including the degree of weathering [19,20], pore pressure, disturbance of the soil structure [21], stress history, time, and environmental conditions, natural moisture contents, and percentage of clay particle [22,23]. The most popular ways to measure the shear strength parameters are those that take place in the field and in the laboratory [23]. In the field, common methods for determining shear strength parameters include the standard penetration test (SPT), the cone penetration test (CPT), and the pressure meter test. In the laboratory, popular methods for determining shear strength parameters include the triaxial test and the shear box test. [24].

The Mohr-Coulomb failure criterion was used to determine two shear strength parameters for saturated sedimentary residual soils. These parameters are denoted by the symbols, i.e c' and ϕ'. Using the consolidated drained (CD) and consolidated undrained (CU) triaxial tests, this research investigated the effective shear strength characteristics of remolded saturated residual soil of sedimentary rock.

2. Methodology

The samples of disturbed residual soil were collected from an open space near to the Infra Science Tech (IST) block at the Universiti Teknologi MARA (UiTM) Pahang Branch. The soil samples were obtained by digging down with a shovel between one to one and half meters below the surface of the ground. Before the soil sample was taken to the laboratory to be dried for 24 hours in an oven heated to 105°C, all the topsoil and humus were removed. Figure 2 shows the location of a soil sample.

Fig. 2. Location of soil sampling

The soil sample will go through a physical properties test and compaction test before the triaxial drained (CD) and triaxial undrained (CU) tests can be carried out. The evaluation was carried out in accordance with the British Standard (1990), Methods of Test for Soils for Civil Engineering Purposes (BS137, Part1-9-1990) and Head: Volume 1:2006. To confirm that the sample can get consistent compaction, the Proctor compaction test is widely applied with automatic compaction equipment. A sample of 3.0 kg of soil that passed through a 20 mm sieve was obtained then water was added with amounting to 5% of the soil weight. After that, this sample is kept in an airtight container for 12 hours. The test started by inserting a soil sample at 1/3 of the height of the mold and then it was compacted 27 times. The soil sample is filled again until the height of 2/3 of the mold and the process continues. At the end of the scale, the soil sample is added up to the height of the mold or past the neck of the mold.

The weight of the compacted soil sample and the mold is then weighed. The test continued with another 3% increase in the amount of water for the next sample, and the compaction test was continued until the weight of the soil, and the mold was reduced. In this test, the compaction curve is plotted, and the optimum dry density and optimum moisture content values can be determined which will then be used to produce samples for the triaxial test.

In triaxial test, four different samples were subjected to a saturated triaxial compression test under consolidated drained, and undrained conditions, respectively. After that, the soil samples were saturated until they had a value of pore pressure coefficient (B_w) that was greater than 0.95. When the saturation process was complete, the soil samples were consolidated under an effective confining stress of 50, 100, 200, and 400 kPa while maintaining the same back pressure of 400 kPa. The samples of soil were then sheared at a strain rate of 0.029mm/min.

3. Results

3.1 Physical Properties of Soil

The residual soil was undergo dry sieving test to determine the percentage breakdown of the soil size as shown in Figure 3. Based on the Unified Soil Classification System (USCS), the soil sample was classified as silty sandy soil (SM). Therefore, the Atterberg test needs to be done to determine the plastic limit value, liquid, and plastic index of the residual soil.

Table 1 shows the summary of physical properties of residual soil and compaction test value based on BS5930:1981, BS 5930:2015 and USCS. The soil sample was classified as silty sand (SM) with intermediate plasticity (MI). These values are then compared with data obtained by previous studies such as [18, 25 - 30] found that the values obtained are close or in the same range.

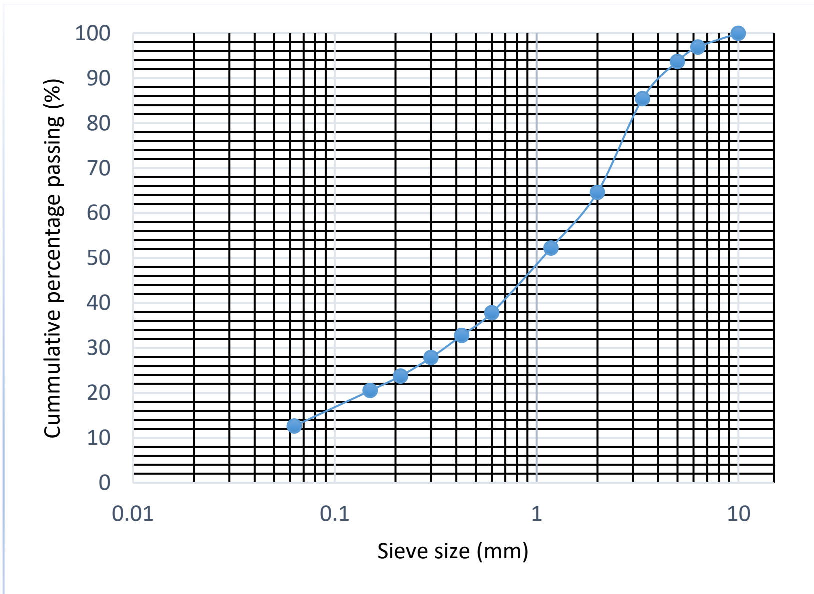

Fig. 3. Grading curve distribution

Table 1
Physical properties of sedimentary residual soil

Items	Value
Depth (m)	1.0 -1.5
Colour	brownish brown
Maximum dry density, MDD (kg/m^3)	1788
Optimum moisture content (OMC) (%)	15.8
Specific gravity, (Gs)	2.62
Liquid Limit, LL (%)	39.85
Plastic Limit, PL (%)	25.08
Plasticity Index, PI (%)	14.77
Gravel (%)	35.42
Sand (%)	51.93
Silt (%)	12.65
Clay (%)	0
Soil Classification	Silty SAND, SM (Silty sand with intermediate plasticity)

3.2 Failure Envelope

The Mohr-Coulomb method can be used to determine the failure envelope, which indicates the soil strength for the residual saturated soil that was used in this investigation. The following equation can be used to get the soil strength parameter. In this equation, the slope of the failure surface is denoted by *tan* ϕ' and the intercept on the x-axis is denoted by c'

$$\tau = c' + \sigma tan\phi' \tag{1}$$

where τ is shear stress, c' is cohesion, σ is normal stress and ϕ' is the value of friction angle.

Figure 3 shows the failure envelope for the CD test. The soil strength parameters c' and ϕ' recorded are 26 kPa and 29°. While Figure 4 shows the failure envelope for the CU test. The recorded soil strength parameters c' and ϕ' are 13 kPa and 32°.

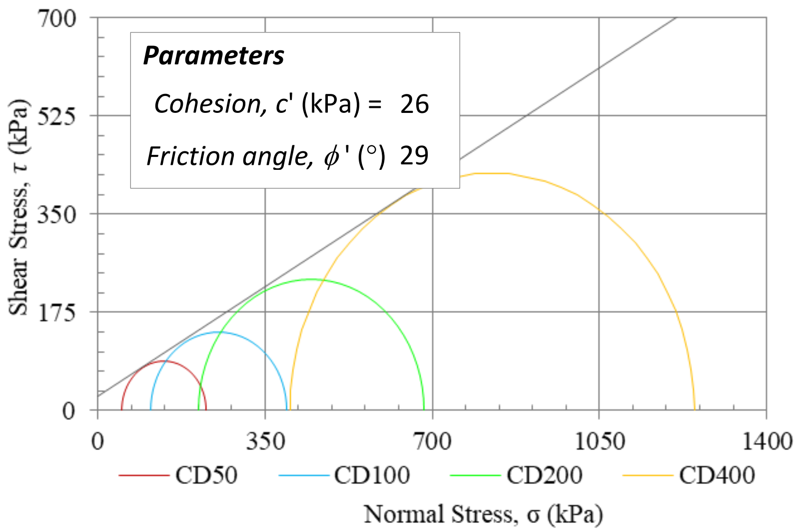

Fig. 3. Mohr-Coulomb failure envelope for CD test

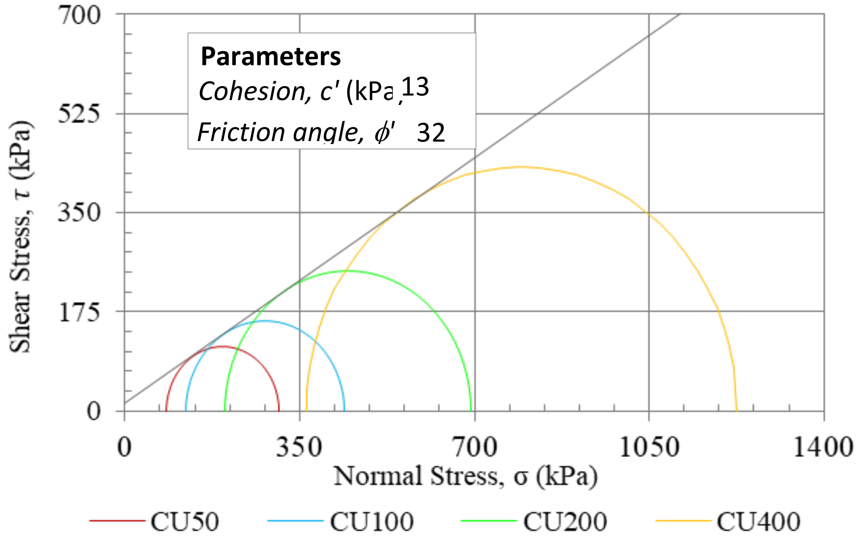

Fig. 4. Mohr-Coulomb failure envelope for CU test

According to the findings of the research conducted by [6], who conducted a CD test on residual soil on sedimentary rock acquired in Sepang, Selangor, they found that the range of soil strength parameters c' and ϕ' recorded were 0 -10 kPa and 26 - 33°. Whereas [31] reported that the range of soil strength parameters c' and ϕ' recorded utilising CU tests with soil samples taken in Kenny Hill, Kuala Lumpur were 0-12 kPa and 23-28° respectively. While CD value recorded for c' is lower than CU, while CU value recorded for ϕ' is higher than CD. Moreover, the similar trend was discovered by [22,24].

According to [25], the value of the residual soil strength parameter acquired will change depending on the depth and weathering grade, which will either lead to a drop or an increase in the value obtained. This may be seen as a consequence of the previous statement. Because the majority of the residual soil has become clay, the value of c' will be higher if the grade of weathering that was recorded was higher. This will make the value of ϕ' will be lower.

3.3 Stress- Strain Behaviour

Figure 5 shows the relationship of deviator stress, q against axial strain ε for the CD test. All tested samples showed brittle failure. When brittle failure occurs, the sample will experience lateral stress up to the maximum extent before decreasing until it reaches a uniform lateral stress value. The higher the effective confining stress applied to the sample, the higher the maximum lateral stress and axial strain values recorded. Sample CD400 shows the maximum lateral stress value of 848.5 kPa at an axial strain of 12.5% before decreasing to 700 kPa. Samples with an effective confining pressure of 50 kPa and 100 kPa showed an insignificant brittle failure pattern when compared to the sample with an effective confining pressure of 400 kPa. All samples were tested up to 20% axial strain. As stated by [32], the critical state of the soil is defined when the pore water pressure starts to become constant or when the soil is sheared up to 20% of the axial strain (whichever happens first).

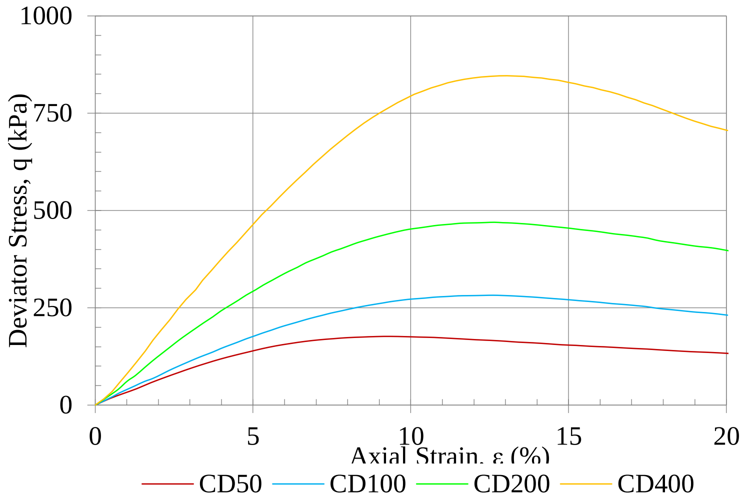

Fig. 5. Stress – strain behaviour for CD test

While Figure 6 shows the relationship of deviator stress, *q* against axial strain ε for the CU test. The same pattern was shown in the CU experiment as in the CD experiment. However, the brittle failure pattern is not so significant for all the samples in the CU test when compared to the CD test sample. The lateral stress value increases gradually before reaching the peak lateral stress and the lateral stress value will decrease until it becomes constant for all the samples tested. The maximum lateral stress value becomes higher as the effective confining stress increases. The same relationship can also be seen to occur in the axial strain, where the axial strain at the effective confining stress also increases starting from sample CU50 up to CU400.

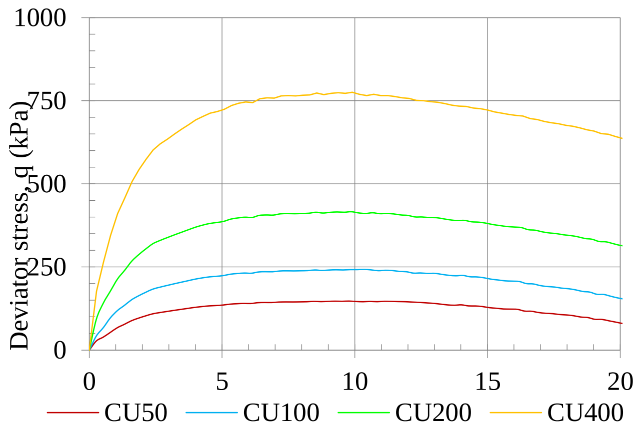

Fig. 6. Stress – strain behaviour for CU test

Figure 7 below shows the volume strain curve, *v* against the axial strain, ε for the CD test. For the CU experiment this curve was not obtained because the volume change could not be recorded. All samples showed almost identical volumetric strain patterns where none of the recorded samples had uniform volumetric strain readings. For example, for sample CD400, the negative volume strain increases rapidly until the volume strain reaches -2.05% at 9% axial strain. After that the curve will reverse with a decrease in negative volume strain where there is an increase in positive volume strain but not up to the initial state. This shows that all the samples experienced a shrinkage process before the expansion process occurred. Deviator stress, *q* and axial strain, ε curve patterns of CD test is then

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compared with this volumetric strain, *v* and axial strain, ε curves. Both curves show the same pattern, and all samples are categorized as overconsolidated.

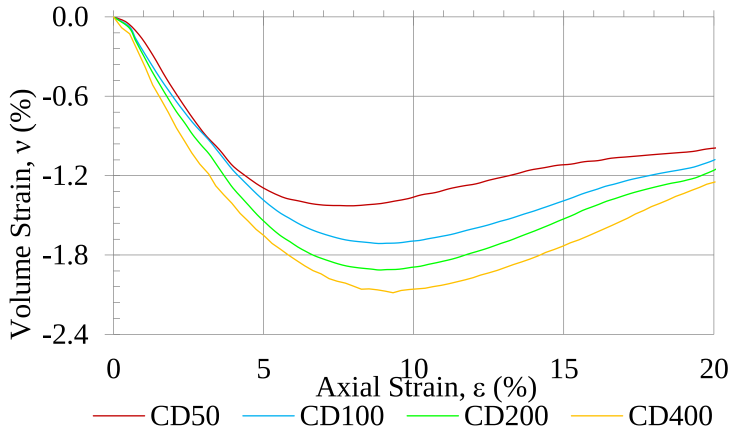

Fig. 7. Volumetric strain – axial strain for CD test

For the CU test, the volume change does not occur thus make only the change in pore water pressure against the axial strain can be observed. The change in pore water pressure occurs due to the water given during the triaxial test being held out of the sample which eventually causes the pore water pressure value to increase in the soil pores. Figure 8 shows the porewater pressure, u versus axial strain, ε curve recorded for each sample. The higher the effective confining stress given, the higher the pore water pressure reading recorded. This may occur because the soil sample is compacted during the sample preparation process which causes the soil microstructure to compact too. The maximum strain values recorded were 19.9, 46.2, 69.3 & 110.7% for CU50,100,200 & 400 samples.

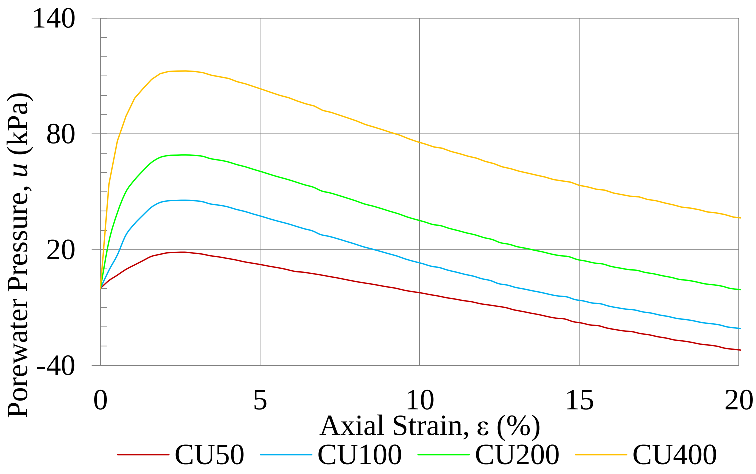

Fig. 8. Porewater pressure – axial strain for CU test

4. Conclusions

Based on the results being mentioned above, the numerous conclusions can be found below

i. It was discovered that the value of c' recorded in the CD test was greater when compared to the previous studies whereas the value of ϕ' was found to be within the same range.

ii. This could be the result of different sampling locations, heterogeneity brought on by weathering (for example, different weathering grades), or differential weathering rates, all of which contribute to the reported values being slightly inflated.
iii. When it comes to the CU test, the data that was collected is not noticeably distinct from the previous studies.
iv. It can also be seen, the cohesion value, c' is higher recorded using the CD test than the CU, and for the friction angle value, ϕ' is higher recorded in the CU test compare than the CD test.
v. While in the stress- strain analysis, the brittle failure pattern was observed in every single sample evaluated by CD test.
vi. The results of the CU test showed the same pattern as those of the CD test. On the other hand, when contrasted with the CD test sample, the brittle failure pattern is not nearly as prominent for any of the samples in the CU test.
vii. If the effective confining stress that was applied to the sample was larger, then the maximum lateral stress, axial strain and pore water pressure values that were recorded would also be higher.
viii. The volumetric strain and axial strain curve patterns are then compared with the results of the CD test of the deviator stress and the axial strain. Both curves exhibit the same trend, and hence, all the samples are considered to be overconsolidated.

As a result of the research and observations conducted, the geotechnical parameters of residual soil on sedimentary rock are of utmost significance before any construction work can begin. In the event there are any issues with the soil, other measures, such as soil improvement, might be taken after that.

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