Shear strength behaviour of different energy in compaction using consolidated drained triaxial test

A S A Rahman¹, M J M Noor¹, J Ahmad¹, N H M Zain¹ and M I F Rosli¹

¹Faculty of Civil Engineering, Universiti Teknologi MARA, Shah Alam, Selangor, Malaysia

*Corresponding author: abdulsamad@salam.uitm.edu.my

Abstract. The concept of compaction has been widely used in the construction activities such as highways embankments, earth dams and others engineering structures where the loose soils must be compacted to increase the unit weights and thus increase the bearing capacity of the subjected soils. However, little intention has been directed towards understanding the influence of compaction energy on the behaviour of shear strength using consolidated drained triaxial test. This research was conducted to study the effect of different energy in compaction for 25 number of blows compare with 40 number of blows using Standard Proctor Test. Results shows that the effect of different energy in compaction is not too significant where the changes is merely about 1° only in effective friction angles. It is because during consolidation stage in triaxial, the effect of different energy has no influence on the specimens. During consolidation stages in triaxial, the soil particles rearrange to each other with different applied pressure thus eliminates the effect of different energy on the specimens due to compaction effort.

1. Introduction

Ground improvement works is necessary before starting construction works at project site especially with weak soil condition. Prior to construction, soil upgrading will be performed to improve its mechanical behaviours in line with the requirements for construction [1]. Soil upgrading can be either by modification, stabilization of soil or both. Rahman et al. [2] stated that soil stabilization is carried out in order to improve the bearing capacity of the soil to accommodate construction needs while soil modification involves the addition of any modifier into the soil to modify the properties of soil such as its strength. The methods available to improve ground soils includes compaction process, pilling work, soil stabilization, vertical drain and load reduction method.

Compaction can be defined as the densification process of unsaturated soil due to decrease in air volume without any changes in volume of water or liquid, by mechanical forces [3]. Compacted residual soil has been widely accepted in tropical or semi-tropical areas as a filling materials where the soil is used as a civil engineering structures such as road based, embankments, slope stability, lateral earth pressure and landfill [4].

Soil compaction was necessary in many engineering works in order to ensure that the soil can sustain loadings from the infrastructure. However, compaction will also decrease the permeability of soil, causing erosion and run off. According to Akshaya [5], when compaction energy is increased the value of the optimum moisture content (OMC) will decrease, while the value of maximum dry density (MDD) will increase due to the increase in soil compaction energy.
There are various geotechnical properties of granitic residual soil that need to be considered in the design of structures to ensure that it is safe and robust [6]. Townsend [7] stated that residual soils are formed from chemical weathering therefore the engineering properties of residual soil depend on climate, parent rock, topography and the time it is formed. Based on Brand [8] and Fookes [9], residual soil is found partially saturated consisting of partly entrapped air and water due to the elevation of the ground from sea level causing the shear strength to vary.

Compaction of granitic residual soil are most important as they are vital in geotechnical design of shallow foundations. The shear resistance is not the main issue but the expansion and reduction of volume due to drying and wetting will affect the compactness of the granitic residual soil [10]. For the design of shallow foundations, embankments, dams and many other structures while consolidation is the change in volume due to increase of overburden pressure at a prolonged time causing reduction of soil volume and relates to the clay content that exist within the soil.

Residual soils are weathering products of rock that usually formed under unsaturated condition. It is formed through the chemical and mechanical weathering of parent rocks in situ. More than 80% of land in Malaysia is covered by the residual granite soil. The low level of ground water table causes the soil to be unsaturated and when infiltrated by rainwater the moisture content increases a soil because wetter. These soils generally belong to the residual category that may exhibit collapse settlement upon wetting [11]. When the soil in wetted condition i.e. inundation, there are possibilities that the soil is expose to large and sudden reduction in volume and known as collapsible soil. The are two main characteristic of the collapsible soil which is consists of loose cemented deposits and the soil is normally in dry condition. Apart of the two main characteristic of the collapsible soil, it also can sustain with larger vertical stresses and with a little effect in compression. However, when the collapsible soil is in wetted condition, then it will results in larger settlement without increase in vertical stress [12].

As reported by Zhao [13], the dominants minerals that exhibit in the granite are quartz and feldspar where the content of the minerals were at 30% and 65% respectively. Almost every part of the Peninsular Malaysia was covered by residual soil where the colour of the soil is in red due to reaction of natural lateritic and laterite form because of the disintegration of fossils under climatic conditions [9].

Chiu and Ng [14] found that when the process of weathering take place for the granite residual soil, the soil will be in sandy state as sand-sized particles of quartz and partially weathered feldspar. Eventually, the weathered feldspar grain will gradually be transformed into fined-grained clay minerals for sometimes. However, the quartz minerals remain in contact due to its resistance to the weathering and results in having the soil in both sand-sized quartz and clay.

In this research, a compaction process at different energy level was performed to improve the shear strength of granitic residual soil. This was carried out on soil specimens using the consolidated drained triaxial test. Hypothetically, the shear strength should increase proportionally with the higher energy in compaction. Results should show that the process of compaction using 40 number of blows should produce higher effective friction angles in the soil specimen compared to 25 number of blows.

2. Research methodology
A laboratory study was done to investigate the effect of shear strength of granite residual soil when tested with consolidated drained triaxial test with compacted of different specimens with different energy of 25 number of blows and 40 number of blows. There are two types of data collected from the laboratory procedure which is index properties and engineering properties. For index properties test, the test that will be conducted were moisture content, wet and dry sieving, sedimentation and the Atterberg limit test. On the other hand, the consolidated drained triaxial test (CD) was conducted to determine the effective minimum internal friction angle at failure, \( \phi'_{min} \), transition shear strength, \( \tau_t \),
and transition effective stress, \((\sigma-u_w)\), based to the curved surface shear strength envelope model. About four (4) different effective stresses were applied at 50, 100, 200 and 300kPa to obtain a series of Mohr circles.

The soil samples were taken at Hutan Lipur Jeram Toi, Kuala Klawang, Negeri Sembilan at coordinates of 2°52’09.1"N; 102°00’50.6"E. Samples were collected at the site location and put in polyethylene bags and bring the soil to the laboratory in UiTM as soon as possible for testing.

About eight (8) cylindrical remoulded specimens with size of 50 mm diameter with 100 mm in height were used for the test. In order to make the remoulded specimens in identical conditions, the same optimum moisture content with maximum dry density were used for 25 number of blows and 40 number of blows consecutively. The aim is to evaluate the effect of different compaction energy in number of blows to the shear strength of the granitic residual soil.

3. Analysis of results
Granitic residual soil was taken from Hutan Lipur Jeram Toi, Kuala Klawang, Negeri Sembilan and classified as silty SAND with 11% of gravel, 60% of sand, 18% of silt and 11% of clay. Moreover, during compaction test, results indicate that the optimum water content for 25 number of blows was at 15% with maximum dry density at 1790 kg/m\(^3\) while for 40 number of blows, the optimum water content was 12.5% with maximum dry density at 1880 kg/m\(^3\). Result reveals that there is a significantly reduction in water content of 17% with an increasing in dry density of 5% when the energy in compaction is increase from 25 number of blows to 40 number of blows. Figure 1 illustrate the optimum moisture content with respect to maximum dry density for the 25 and 40 number of blows.

![Figure 1](image)

**Figure 1.** Optimum moisture content against maximum dry density for standard Proctor compaction at different number of blows.

Basically, when the soil is compacted with higher number of blows, it will re-orientate to denser state of packing and thus will reduce the void ratio. This will reduce the ability of water to seep through the soil and results in reduction of water content in the soil. This explain why the water content for 40 number of blows is less compared to 25 number of blows.

Effective internal friction angle of the saturated specimens can be plotted using the stress-strain graph tabulated from consolidated drained triaxial test. Table 1 illustrate the tabulated data for shear strength parameters of 25 number of blows with respect to deviator stress, pore water pressure and cell pressure. Results reveal that the effective friction angle at failure is 37° with transition shear strength, \(\tau_t\) is 66kPa and transition effective stress, \((\sigma-u_w)\), is 62kPa. In order to determine the transition shear strength, \(\tau_t\) and transition effective stress, \((\sigma-u_w)\), the non-linear shear strength envelope need to be
plotted as shown in Figure 2(b). The values of the transition shear strength, $\tau_t$, and transition effective stress, $(\sigma-U_w)_t$, can be calculated using the plotted Mohr envelope especially at the lower shear-stress value i.e. at 50kPa effective stress Mohr envelope.

**Table 1.** Effective shear stress parameters at failure for 25 number of blows.

| Effective Stress, kPa | Condition of Failure | Shear Strength Parameters |
|----------------------|----------------------|---------------------------|
|                      | Deviator Stress (kPa) | Pore Water Pressure (kPa) | Cell Pressure (kPa) | $\phi'_f$ | $\tau_t$ | $(\sigma-U_w)_t$ |
| 50                   | 190                  | 400                        | 450               |          |          |             |
| 100                  | 350                  | 400                        | 500               |          |          |             |
| 200                  | 648                  | 400                        | 600               | $37^\circ$ | 66        | 62          |
| 300                  | 956                  | 400                        | 700               |          |          |             |

Figure 2(a) illustrate the deviator stress for each stress-strain curve at maximum for the saturated specimens at 25 number of blows. Nevertheless, the recorded data at peak for the deviator stress are at 190, 350, 648 and 956kPa for an at effective stresses of 50, 100, 200 and 300kPa respectively. Finally, about four (4) failure Mohr envelopes can be drawn using non-linear failure envelope as shown in Figure 2(b).

**Figure 2.** (a) Illustrate the stress-strain curves for 25 number of blows at 50, 100, 200 and 300kPa effective stress while in Figure 2(b) illustrate the non-linear shear strength envelope at 50, 100, 200 and 300kPa effective stress.

The results of shear strength parameters at 40 number of blows in compaction was recorded and tabulated in Table 2. From the tabulated table, that the effective friction angle at failure was at $38^\circ$ with the transition shear strength, $\tau_t$, is 68kPa and transition effective stress, $(\sigma-U_w)_t$, is 61kPa. On the other hand, Figure 3(a) illustrate the deviator stress at failure for each of the stress-strain curve for the saturated specimens at 40 number of blows. The deviator stress at maximum were recorded as 195, 355, 660 and 995kPa for the stress-strain curves at effective stresses of 50, 100, 200 and 300kPa respectively. Four failure Mohr envelopes were drawn using non-linear failure envelope as shown in Figure 3(b).
Table 2. Effective shear stress parameters at failure for 40 number of blows in compaction.

| Effective Stress, kPa | Deviator Stress (kPa) | Pore Water Pressure (kPa) | Cell Pressure (kPa) | $\phi'_f$ | $\tau_k$ kPa | $(\sigma-U_w)_k$ kPa |
|----------------------|-----------------------|---------------------------|---------------------|----------|---------------|---------------------|
| 50                   | 195                   | 400                       | 450                 |          |               |                     |
| 100                  | 355                   | 400                       | 500                 |          |               |                     |
| 200                  | 660                   | 400                       | 600                 | 38°      | 68            | 61                  |
| 300                  | 995                   | 400                       | 700                 |          |               |                     |

Figure 3. (a) Illustrate the stress-strain curves for 40 number of blows at 50, 100, 200 and 300kPa effective stress while in Figure 3(b) illustrate the non-linear shear strength envelope at 50, 100, 200 and 300kPa effective stress.

The difference in effective friction angles between 25 number of blows and 40 number of blows was only about 1°, that is between 37° and 38°. This small change in effective friction angle have insufficient influence on the shear strength of the soil. Even though the increase in compaction from 25 to 40 number of blows significantly increase the water content and dry density, it did not give significant contribution to the shear strength. This means that an increasing in number of blows for compaction did not increase the shear strength of the soil at all.

Figure 4(a) shows the combination of recorded maximum deviator stress for each stress-strain curve of the saturated specimens at 25 and 40 number of blows. Figure 4(b) on the other hand shows the combination of failure envelope of 25 and 40 number of blows.
Figure 4. (a) Shows the combination of stress-strain curves for 25 and 40 number of blows at 50, 100, 200 and 300kPa effective stress while in Figure 4(b) shows the non-linear shear strength envelope at 50, 100, 200 and 300kPa effective stress.

4. Conclusion and recommendation

Based on the results presented, the compaction for 40 number of blows gave a higher value of dry density but lower value water content compared with 20 number of blows. It can be said that the higher energy applied in the compaction would result in better dry density but lower permeability. Different energy in compaction can influence the shear strength of the soil. However, in consolidated drained triaxial test the results of shear strength between 25 and 40 number of blows shows that the values of the effective friction angle is almost the same which is 37° and 38° respectively. The difference in 1° of the shear strength will not give any significant impact on the soil. Even though there is different energy in compaction between 25 and 40 number of blows, the difference in energy did not influence the shear strength of the soil when tested in the consolidated drained triaxial test. This is could be due to the specimens being saturated in consolidated stages, resulting in the soil skeleton re-arranging and repacking itself to a denser state according to the desired effective stress. Therefore, even if the specimens were compacted using different energy i.e. different in number of blows, the shear strength will not increase since the specimens will have to consolidated with the desired effective stress in the consolidation stages. As for the recommendations for further research, studies can be carried out on the impact of different energy in the form of number of blows on the shear strength using direct shear strength method since this this method will not involve any consolidation stages. The results could then be compared with the outcome from the triaxial test.

References

[1] Rahman Z A, Lee J Y Y, Rahim S A , Lihan T and Idris W M R 2015 Application of gypsum and fly ash as additives in stabilization of tropical peat soil Journal of Applied Science 15 (7) 1007-1012
[2] Rahman Z A, Ashari H H, Sahibin A R, Tukimat L and Razi I W M 2014 Effect of rice husk ash addition on geotechnical characteristics of treated residual soil Journal of Agriculture & Environment Sciences 14 (12) 1368 - 1377
[3] Ghosh R 2013 Effect of soil moisture in the analysis of undrained shear strength of compacted clayey soil Journal of Civil Engineering and Construction Technology 4(1) 23-31
[4] Taha M R, Mofiz S A and Hossain M K Mohamad A 2000 Stress-strain behaviour of compacted residual soil in direct shear test *Proceedings of ISRM International Symposium, Melbourne, Australia*

[5] Akshaya K S and Ranjan K M 2015 Effect of compaction energy on engineering properties of fly ash-granite dust stabilized expansive soil *International Journal of Engineering and Technology* 71617-1624

[6] Marto K and Kassim F 2003 Characterisation of Malaysian residual soil for geotechnical and construction engineering *Project Report Vote NO:72256 UTM, Malaysia*

[7] Townsend F C 1985 Geotechnical characteristics of residual soils *Journal of Geotechnical Engineering* 111 77-92

[8] Brand E W 1982 Analysis and design in residual soils *Proceedings of the Conference on Engineering and Construction in Tropical and Residual Soils* ASCE, Honolulu, Hawaii 89-129

[9] Fookes P G 1997 *Tropical residual soils*, 1st. ed. London: The Geological Society London.

[10] Brand E W 1982 Analysis and design in residual soils *Proceedings of the Conference on Engineering and Construction in Tropical and Residual Soils*. ASCE, Honolulu, Hawaii 89-129

[11] Ayadat T and Hanna A M 2011 Assesment of soil collapse prediction methods *International Journal of Engineering* 25 19-26

[12] Ahmed F A, Yahaya A S and Farooq M A 2006 Characterization and geotechnical properties of Penang residual soils with emphasis on landslides *American Journal of Environmental Sciences* 2 121-128

[13] Zhao J 1994 Engineering properties of the weathered Bukit Timah granite and residual soils *Regional Conference in Geotechnical Engineering* 94 Melaka, Malaysia

[14] Chiu C F and Ng C W 2014 Relationships between chemical weathering indices and physical and mechanical properties of decomposed granite *Engineering Geology* 179 76-89

[15] Little A L 1969 The engineering classification of residual tropical soils *Proceedings 7th International Conference Soil Mechanics and Foundation Engineering, Mexico* 1 1-10