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**Original Research Article**

# An Alternative Method for the Estimation of Coefficient of Vertical Consolidation of Coarse-Grained Lateritic Soils

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## ABSTRACT

*Coefficient of consolidation ($c_v$) is a very important parameter in the estimation and monitoring of settlement characteristics of soil for foundation design and other purposes. It is usually determined through an oedometer test using various methods. However, the irregularity of the dial reading versus root of time curve used to determine $c_v$ using Taylor's method has been a challenge especially for soils with high sand content. Ten (10) lateritic soil samples were tested in the laboratory using an oedometer apparatus. The dial reading versus root of time graph was plotted for all the samples and $c_v$ determined in both normal fit (free hand method) and power function fit (power series method). The coefficient of determination ($R^2$) was quite significant for the curves when fitted to power function with average values ranging from 0.871 to 0.986 across all samples. With the power curve closely approximating the theoretical U-T curve, it is expected that $c_v$ values from such would be more reliable. Descriptive statistics carried out for the $c_v$ values show high standard error, standard deviation and variance for $c_v$ values determined from free hand method when compared to power series method. Thus, fitting dial reading versus root of time curve to a power function could be a more reliable way of determining $c_v$ for coarse-grained soils using Taylor series method.*

## 1. INTRODUCTION

Consolidation is a term that is applicable to soils experiencing pressure or load. Such soil mass is said to be under compressive force which in effect causes its volume to decrease. This property is called compressibility of soils (Arora, 2014). The compression of soil is a multifaceted phenomenon. The first phase involves the compression of solid particles and water in voids. This is negligible as solid particles and water are nearly incompressible. So, there is no significant work done here. On the other hand, there is a second phase which involves the compression and expulsion of pore air. This is usually rapid as air is very compressible. The last phase is very important as it is time-dependent. It involves the expulsion of pore water collected in the voids and the solid particles are forced to adjust and pack together vertically. This is known as settlement (Arora, 2014). Settlement is a critical term in the design of foundations. A stable structure is not expected to undergo excessive settlement or unequal settlement. The analysis of consolidation settlement is very important as it helps to forecast the magnitude of settlement and the time/rate of settlement of structure. This rate of settlement is usually found upon the determination of coefficient of consolidation ($c_v$ or $c_r$) (Pillai and Gandhi, 2016). The coefficient of consolidation ($c_v$) depends on the amount of water squeezed out and hydraulic conductivity and it is measured in square centimeter/millimeter per second ($cm^2/s$ or $mm^2/s$). $C_v$ is determined in the laboratory using an oedometer apparatus on undisturbed soil samples based on one-dimensional Terzaghi's consolidation theory (Sharma *et al*, 2019).

There are two commonly used methods for the estimation of the value of coefficient of consolidation from oedometer data. These are known as the Casagrande's logarithm of time fitting method (Casagrande and Fadum, 1940) and the Taylor's square root of time fitting method (Taylor, 1948). With these methods the experimental deflection - time curves are fitted to the theoretical degree of consolidation - time factor curves.

The Casagrande's logarithm of time fitting method and Taylor's square root of time fitting method are based on the fact that there is a similarity in shape between the curve obtained from the plot of compression dial reading ($\delta$) against time (t) and theoretical curve between average degree of consolidation (U) and Time factor ($T_v$) from consolidation theory (Pillai and Ghandi, 2016). Both methods are effective for estimating $c_v$. However, there are slight differences due to soil type (Arora, 2014). These curves are constructed with oedometer data.

Pillai and Ghandi (2016) reported that Casagrande's logarithm of time method can be used for soils that does not have initial straight portion but it takes time for secondary compression portion to be established. Su (1958) and Robinson and Allam (1996) worked to improve on the logarithm of time fitting method by developing a method which makes use of early logarithm of time (log t) and requires time compression data for only short duration. However, both the original Casagrande method and modified methods are not suitable for soils exhibiting additional secondary compression. Taylor's method proved better alternative for such types of soil but both methods are not suitable for rapidly consolidating soils. Taylor's methods have also undergone modifications. Tewatia and Venkatachalam (1997) improved Taylor method such that $c_v$ value at any U between 70 and 95% can be determined. Parkin (1978) developed the more efficient velocity method and was improved by Pandian *et al* (1993). However, both methods don't place emphasis on the initial and final parts of the consolidation curve. Sridharan *et al* (1981) developed the rectangular hyperbola method in which the initial compression was not affected and was improved by Sridharan (1985). Other researchers such as Sridharan and Prakash (1995) and Mesri *et al* (1999) have made significant contributions at improving the way $c_v$ can be computed from oedometer test results but no perfect method have been developed.

Sharma *et al* (2019) suggested that $c_v$ can be influenced by the disturbance induced during sampling or by the inaccuracy of measurements from oedometer test. As noted elsewhere, the Casagrande's logarithm of time fitting method and Square root of time fitting method are based on the similarity in shape between the curve obtained from the plot of compression dial reading ($\delta$) against time (t) and theoretical curve between average degree of consolidation (U) and Time factor ($T_v$) from consolidation theory (Pillai and Ghandi, 2016). Over the years, efforts have been made to improve on the method of determining $c_v$ based on different reasons as outlined above. One point method, developed by Sridharan *et al.* (1995) was particularly useful at obtaining an experimental curve that coincides better in shape to the theoretical curve. It was observed based on laboratory tests carried out by the authors in the past that the experimental curve of dial reading versus root of time obtained from oedometer test for coarse-grained lateritic soils is usually irregular and does not come similar in shape to the theoretical consolidation curve. This irregularity could be due to soil properties or experimental errors. Power function which are normally used to model the relationship between two quantities which in this case are dial reading and time was sought to correct the irregularity. The power series is simply an innovation for handling consolidation curves for coarse-grained soils. Arora (2014) pointed out that Taylor's square root of time fitting method is not usually suitable for soils that does not undergo secondary compression. Among such soils are sands and coarse-grained soils. This innovation is a modification that utilizes the Taylor's method for coarse-grained soils.

The aim of this paper is to examine the variation of $c_v$ values when it is determined from irregular points fitted with power function curve compared to those determined from the normal irregular curve.

## 2. MATERIALS AND METHODS

### 2.1. Materials Collection

Ten (10) lateritic soil samples were used in this study. They were obtained from various locations in Anambra State, Nigeria. The samples were packaged in water-tight polythene bags (to avoid moisture loss) and transported to the geotechnical engineering laboratory of Nnamdi Azikiwe University Awka, Nigeria for geotechnical analysis. The sample designation, location and depth are shown in Table 1. The sampling locations were chosen because they contain popular borrow pits while the sampling depths were adopted to obtain undisturbed samples.

Table 1: Sample designation, location and depth

| Samples | Location | Depth |
|---|---|---|
| 1 | Agu Awka 1 | 1.5 |
| 2 | Agu Awka 2 | 1.6 |
| 3 | Nawfia | 2.8 |
| 4 | Awkuzu- Nkwelle | 1.8 |
| 5 | Awkuzu Ifite 1 | 2.5 |
| 6 | Awkuzu Ifite 2 | 1.8 |
| 7 | Ring Road 1 | 1.5 |
| 8 | Ring Road 2 | 1.6 |
| 9 | Nteje | 1.8 |
| 10 | Nkwelle-Ezunaka | 2.5 |

### 2.2. Methods

Index property tests and consolidation tests were carried out on the soils.

### 2.2.1. Index property tests

The index tests were done according BS 1377-2:1990 (BS 1990a) to enable classification of the soil. These include natural moisture content test, Atterberg limits tests, specific gravity and particle size distribution tests. The grading modulus (GM), was computed by using Equation (1) (Osinubi *et al*, 2012):

$$\text{GM} = \frac{300 - \%\,passing\ 2.4\ mm + \% < 0.425mm + \% 0.075mm}{100} \tag{1}$$

% passing 2.4 mm = Percentage of particle sizes passing sieve with 2.4 mm aperture; % < 0.425 mm = Percentage of particle sizes less than 0.425 mm; % < 0.075 mm = Percentage of particle sizes less than 0.075 mm.

Derived plasticity parameters such as plasticity modulus (PM) and plasticity product (PP) are used to represent the effective contribution of the plasticity of the fines to the performance of the whole material and they depend on the proportion of fines in the material (Osinubi *et al*, 2012).

Plasticity Modulus (PM) is defined as the product of plasticity index (PI) and percentage of soil fraction passing BS No 40 sieve (i.e., % < 425 μm):

$$PM = PI \times (\% < 425\ \mu m) \tag{2}$$

% < 425 μm = percentage of particle sizes less than 0.425 mm.

Plasticity product (PP) is defined as the product of plasticity index (PI) and percentage of fines less than BS No 200 sieve (i.e., % < 75 μm): % < 75 μm = percentage of particle sizes less than 0.075 mm

$$PP = PI \times (\% < 75\ \mu m) \tag{3}$$

### 2.2.2. Consolidation tests

Laboratory consolidation test was done in accordance to (BS 1377-6:1990) (BS 1990b). The ten (10) samples were loaded in sequence with the six (6) different loads namely; 12.5 kPa, 25 kPa, 50 kPa, 100 kPa, 200 kPa and 400 kPa. For all loads applied, the initial and final extensometer readings are taken at (15, 25, 30 and 60 seconds) and (2.25, 4, 9, 16, 25, 36, 49, 64, 81, 100, 121, 144, 169 and 1440 minutes) until load time was completed. The square root of time fitting technique was employed in the determination of the coefficient of vertical consolidation for the soils (Taylor, 1948).

### 2.2.3. Statistical analysis

Statistical analysis was carried out using t-test: two sample assuming unequal variances and descriptive statistics. The t-test tool was used to determine whether there is any indication of a difference between the means of the $c_v$ results obtained from the two methods. The descriptive statistics was used to examine the standard error, standard deviation, variance and coefficient of variability between the two results.

## 3. RESULTS AND DISCUSSION

The results from the index properties tests and classification of the ten soil samples are shown in Table 2. The soils were classified based on Unified Soil Classification System (ASTM D 2487-16) and most of the soils are clayey sands (SC) hence the name coarse-grained lateritic soils. Except for sample 10, all other samples have fines content less than 50 % thus they fall under coarse-grained soils and their behaviour would be dominated by the sand fraction. Such soils are semi-pervious to impervious and have low compressibility. These properties could contribute to irregularity of consolidation curves for the soils.

Table 2: Classification and index properties of soil samples used

| Samples | USCS class | NMC (%) | SG | FC (%) | SC (%) | GC (%) | LL (%) | PI (%) | GM | PM (%) | PP (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Clayey sand (SC) | 10.10 | 2.58 | 45.64 | 54.31 | 0.05 | 34.70 | 16.86 | 0.855 | 16.860 | 769.490 |
| 2 | Silty sand (SM) | 9.30 | 2.48 | 35.93 | 63.93 | 0.14 | 51.30 | 18.75 | 1.088 | 37.500 | 673.688 |
| 3 | Clayey sand (SC) | 6.50 | 2.55 | 19.79 | 80.21 | 0.00 | 28.00 | 12.02 | 1.134 | 36.060 | 237.876 |
| 4 | Clayey sand (SC) | 7.62 | 2.51 | 30.09 | 69.87 | 0.04 | 40.50 | 21.01 | 0.999 | 84.040 | 632.191 |
| 5 | Clayey sand (SC) | 9.11 | 2.59 | 35.87 | 64.05 | 0.08 | 24.40 | 11.03 | 1.085 | 55.150 | 395.646 |
| 6 | Clayey sand (SC) | 5.79 | 2.49 | 25.28 | 74.72 | 0.00 | 20.90 | 12.32 | 1.171 | 73.920 | 311.450 |
| 7 | Clayey sand (SC) | 9.14 | 2.42 | 34.42 | 64.30 | 1.28 | 24.60 | 10.20 | 0.950 | 71.400 | 351.084 |
| 8 | Clayey sand (SC) | 9.03 | 2.46 | 27.76 | 65.56 | 6.68 | 23.60 | 10.39 | 1.059 | 83.120 | 288.426 |
| 9 | Clayey sand (SC) | 8.29 | 2.42 | 27.75 | 72.25 | 0.00 | 22.25 | 6.47 | 1.030 | 58.230 | 179.543 |
| 10 | Sandy lean clay (CL) | 17.73 | 2.30 | 63.74 | 27.12 | 9.14 | 23.00 | 11.44 | 0.711 | 114.40 | 729.186 |

NMC = natural moisture content, SG = specific gravity, FC = fines content, SC = sand content, GC = gravel content, LL = liquid limit, PI = plasticity index, GM = grading modulus, PM = plasticity modulus, PP = plasticity product

### 3.1. Power Function Curves

Power function curves were fitted to the scatter plot of dial reading versus root of time for the ten samples. However, for brevity, only the results for two samples are shown (Figure 1a and 1b) to drive home the purpose. The summary of equations and corresponding values of coefficient of determination ($R^2$) for the ten samples are given in Table 3. The average values of $R^2$ from power function plots are also summarized in Table 4.

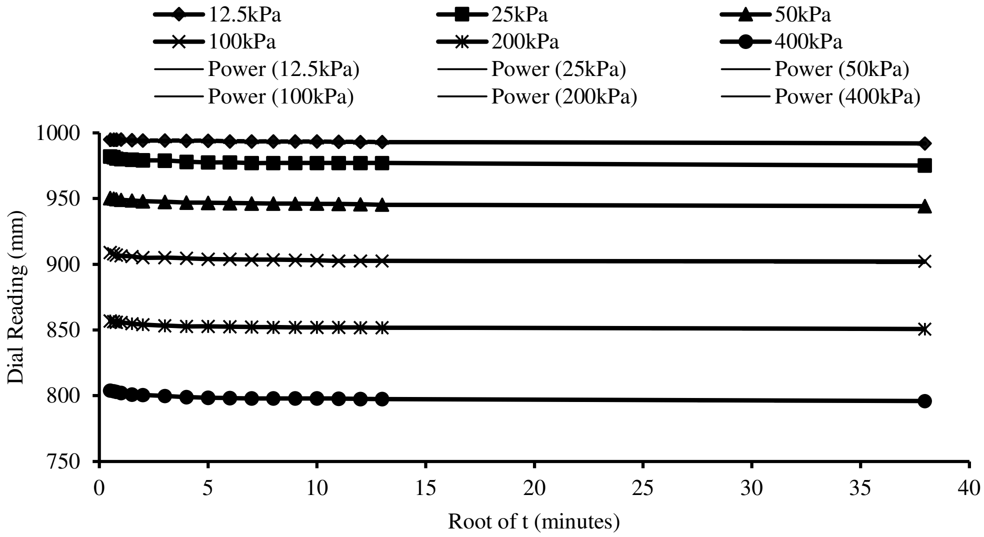

Figure 1(a): Power function plots for sample 6

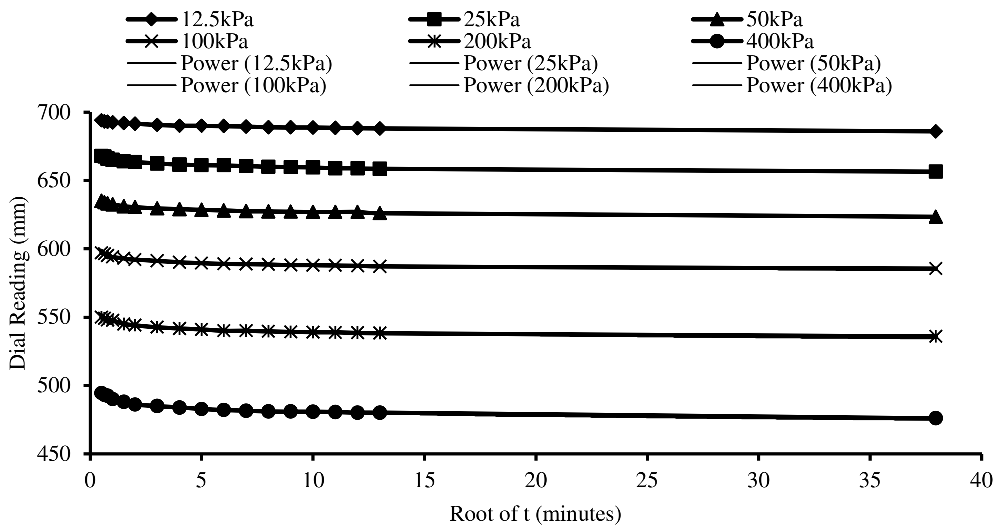

Figure 1(b): Power function plots for sample 10

Table 3: Equations of power function curves and corresponding $R^2$

| Loading (kPa) | | 12.5 | 25 | 50 | 100 | 200 | 400 |
|---|---|---|---|---|---|---|---|
| Sample 1 | Equation | $Y = 798.9X^{-2E-04}$ | $Y = 795.8X^{-5E-04}$ | $Y = 783.4X^{-0.001}$ | $Y = 747.1X^{-0.001}$ | $Y = 701.3X^{-0.002}$ | $Y = 649.8X^{-0.003}$ |
| | $R^2$ | 0.805 | 0.929 | 0.951 | 0.968 | 0.947 | 0.943 |
| Sample 2 | Equation | $Y = 799.2X^{-2E-04}$ | $Y = 796.9X^{-4E-04}$ | $Y = 788.9X^{-9E-04}$ | $Y = 767.8X^{-0.002}$ | $Y = 733.2X^{-0.001}$ | $Y = 692.7X^{-0.002}$ |
| | $R^2$ | 0.581 | 0.865 | 0.972 | 0.939 | 0.921 | 0.945 |
| Sample 3 | Equation | $Y = 698.8X^{-3E-04}$ | $Y = 686.7X^{-0.001}$ | $Y = 649.9X^{-8E-04}$ | $Y = 593.6X^{-0.002}$ | $Y = 536.5X^{-0.002}$ | $Y = 486.9X^{-0.002}$ |
| | $R^2$ | 0.917 | 0.981 | 0.878 | 0.954 | 0.935 | 0.966 |
| Sample 4 | Equation | $Y = 899.0X^{-7E-05}$ | $Y = 896.3X^{-5E-04}$ | $Y = 878.2X^{-0.002}$ | $Y = 828.6X^{-0.002}$ | $Y = 755X^{-0.003}$ | $Y = 719.4X^{-0.004}$ |
| | $R^2$ | 0.818 | 0.828 | 0.934 | 0.918 | 0.947 | 0.921 |
| Sample 5 | Equation | $Y = 598.4X^{-4E-04}$ | $Y = 540.3X^{-2E-04}$ | $Y = 529.9X^{-0.003}$ | $Y = 487.3X^{-0.003}$ | $Y = 440.0X^{-0.004}$ | $Y = 382.3X^{-0.005}$ |
| | $R^2$ | 0.872 | 0.854 | 0.961 | 0.971 | 0.965 | 0.971 |
| Sample 6 | Equation | $Y = 994.7X^{-7E-04}$ | $Y = 980.3X^{-0.002}$ | $Y = 949.1X^{-0.002}$ | $Y = 906.8X^{-0.002}$ | $Y = 855.5X^{-0.002}$ | $Y = 802.1X^{-0.002}$ |
| | $R^2$ | 0.967 | 0.965 | 0.992 | 0.942 | 0.964 | 0.969 |
| Sample 7 | Equation | $Y = 897.4X^{-0.001}$ | $Y = 858.1X^{-0.003}$ | $Y = 806.8X^{-0.004}$ | $Y = 753.8X^{-0.004}$ | $Y = 686.4X^{-0.003}$ | |
| | $R^2$ | 0.993 | 0.969 | 0.975 | 0.979 | 0.98 | |
| Sample 8 | Equation | $Y = 894.9X^{-7E-04}$ | $Y = 872.2X^{-0.001}$ | $Y = 819.3X^{-0.003}$ | $Y = 747.7X^{-0.002}$ | $Y = 702.1X^{-0.006}$ | |
| | $R^2$ | 0.859 | 0.893 | 0.962 | 0.94 | 0.975 | |
| Sample 9 | Equation | $Y = 698.0X^{-6E-04}$ | $Y = 682.4X^{-0.002}$ | $Y = 660.0X^{-0.001}$ | $Y = 635.7X^{-0.002}$ | $Y = 608.8X^{-0.002}$ | $Y = 577.1X^{-0.002}$ |
| | $R^2$ | 0.975 | 0.889 | 0.938 | 0.951 | 0.951 | 0.97 |
| Sample 10 | Equation | $Y = 692.5X^{-0.003}$ | $Y = 665.5X^{-0.004}$ | $Y = 632.5X^{-0.004}$ | $Y = 594.2X^{-0.005}$ | $Y = 546.9X^{-0.006}$ | $Y = 490.3X^{-0.009}$ |
| | $R^2$ | 0.993 | 0.987 | 0.99 | 0.982 | 0.982 | 0.98 |

Y=dial reading, X= square root of time

Table 4 show significant coefficient of determination for all the samples. The least value for sample 2 is 0.581 with some values as high as 0.993 (sample 10). The minimum average value across all loads is 0.871 (sample 2) while the maximum is 0.986 (sample 10). Coefficient of correlation is normally used to examine the relationship between two variables (one dependent and one independent variable). Since there is no similar work done on this method before, the $R^2$ is the only tool to explain this correlation. The $R^2$ values were observed to improve when the experimental curves were fitted to power function with the curves becoming more similar to the theoretical U-T curve. This shows that power function could correct the errors due to high sand content in the soils.

Table 4: Average values of $R^2$ obtained from power series

| Samples | Range of $R^2$ (across six (6) loads) | Average value of $R^2$ |
|---|---|---|
| 1 | 0.805 – 0.968 | 0.924 |
| 2 | 0.581 – 0.972 | 0.871 |
| 3 | 0.878 – 0.983 | 0.939 |
| 4 | 0.818 – 0.947 | 0.894 |
| 5 | 0.854 – 0.971 | 0.932 |
| 6 | 0.942 – 0.992 | 0.967 |
| 7 | 0.969 – 0.993 | 0.979 |
| 8 | 0.893 – 0.975 | 0.926 |
| 9 | 0.889 – 0.975 | 0.946 |
| 10 | 0.980 – 0.993 | 0.986 |

### 3.2. Coefficient of Consolidation ($C_v$) Values from Power Series Method and Free Hand Method

Although Taylor's method has proved to be one of the most versatile methods for determining $c_v$ values, the irregularity of dial reading versus square of time curve is a challenge especially for coarse-grained soils. To determine $c_v$ from experimental compression ($\delta$) versus time (t) curve, the experimental curve (Figure 3) is usually matched to the theoretical U versus T curve (Figure 2) (Pillai and Ghandi, 2016).

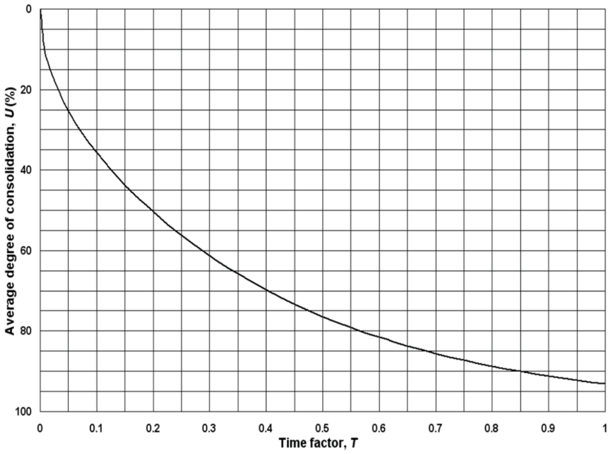

Figure 2: Graphical representation of U-T relationship (Shukla *et al*, 2009)

Taking the load 12.5 kPa in sample 10 for instance, Figure 4 show experimental curve of dial reading versus root of time. One can notice that there are slight deviations of the power function curve from the original plotted points based on the oedometer readings. When this curve (Figure 3) is matched to Figure 2 one can also see that it is similar to the theoretical curve. This could be the reason while high values of $R^2$ were obtained for the power function curves (Table 4).

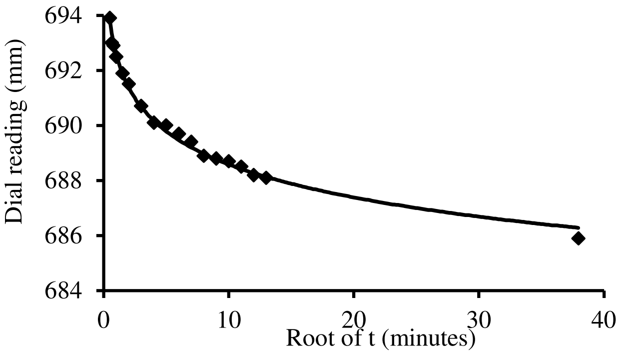

Figure 3: Experimental dial reading versus root of time curve when fitted to power function

### 3.3. Coefficient of Consolidation ($C_v$) Versus Log of Pressure Plots

Figures 4a to 4j shows the $c_v$ versus log P curves for the ten soil samples tested. $C_v$ values from power series method show more regular pattern in response to log P when compared to values from free hand method. Muntohar (2009) in his work pointed out that the general concern about Taylor's method is that the consolidation curve near 90% consolidation and the shape of the √t consolidation curve may be affected by secondary compression. The coefficient of consolidation was also observed to decrease with increasing applied pressure. This trend was more regular with $c_v$ values determined from PSM when compared to values determined from FHM. Wide variability was also observed for $c_v$ values determined from FHM when compared to those determined from PSM. This could be due to the fact that the length of the measured secondary compression segment was not sufficient. It is anticipated that fitting the experimental consolidation curves to power function would help to resolve these irregularities. Arora (2014) also opined that Taylor's square of time method is more suitable for soils exhibiting high secondary compression. Such soils are peat or clay. Based on this, it would be concluded that secondary compression is small or negligible in sands or soils with high sand content such as coarse-grained lateritic soil. This could be the reason why there is much irregularity in the curve and this effect can be corrected by fitting the irregular curve to power function.

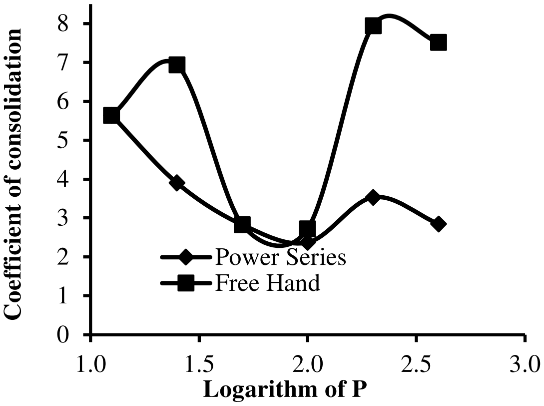

Figure 4a: $c_v$ versus log P for sample 1

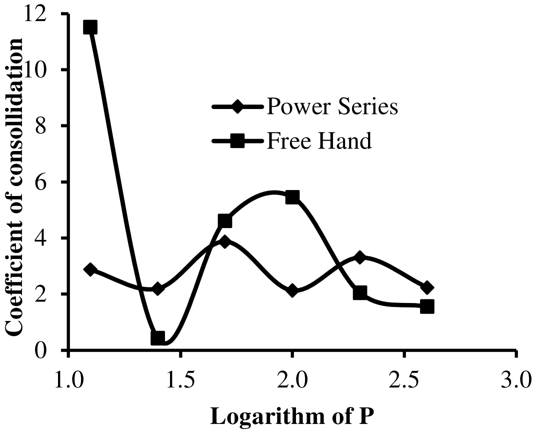

Figure 4b: $c_v$ versus log P for sample 2

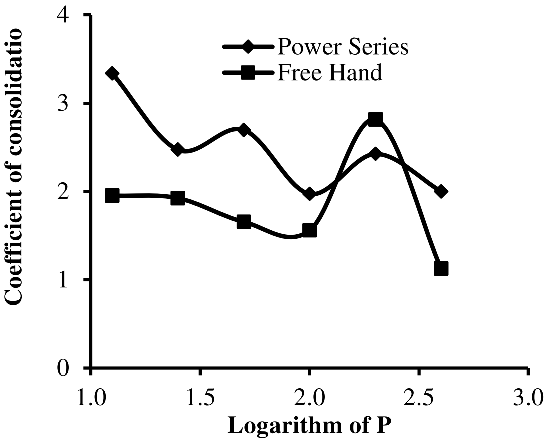

Figure 4c: $c_v$ versus log P for sample 3

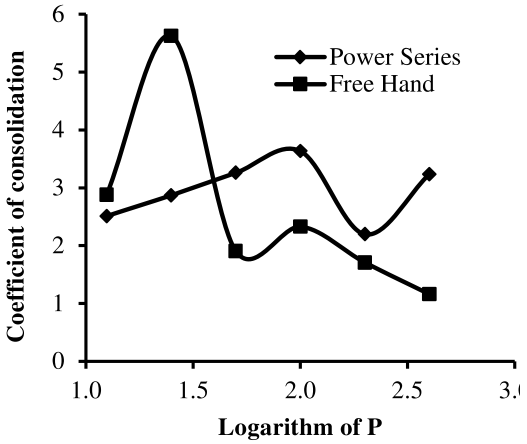

Figure 4d: $c_v$ versus log P for sample 4

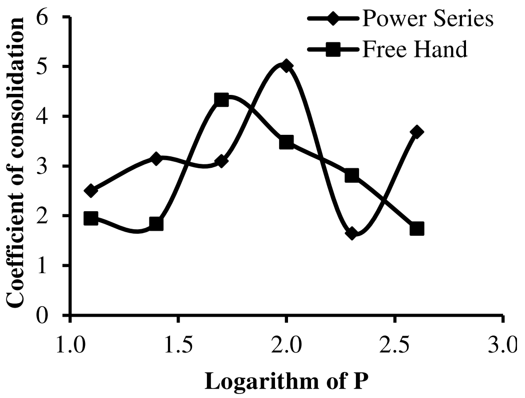

Figure 4e: $c_v$ versus log P for sample 5

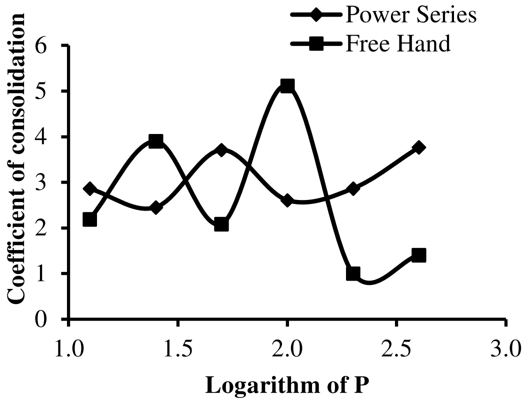

Figure 4f: $c_v$ versus log P for sample 6

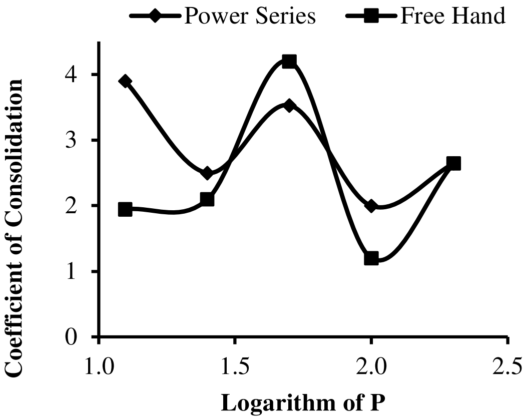

Figure 4g: $c_v$ versus log P for sample 7

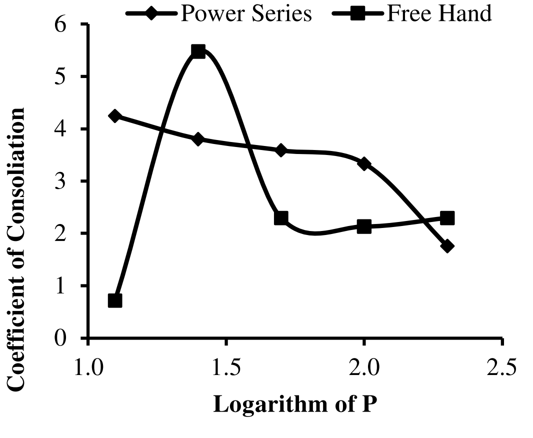

Figure 4h: $c_v$ versus log P for sample 8

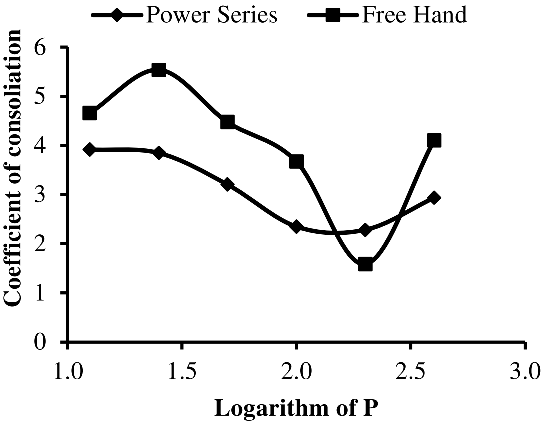

Figure 4i: $c_v$ versus log P for sample 9

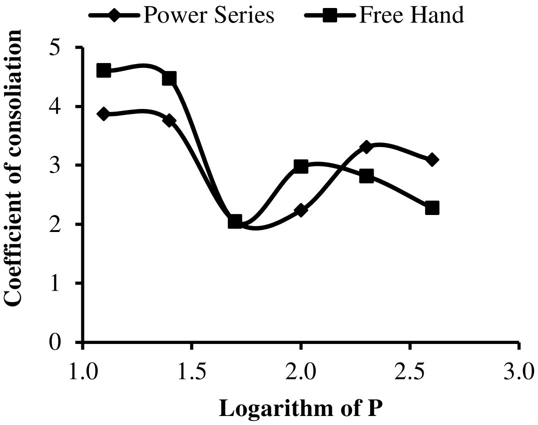

Figure 4j: $c_v$ versus log P for sample 10

## 3.4. Statistical Analysis

### 3.4.1. T-test: two sample assuming unequal variances

The test was done to examine whether there is any indication of a difference between the means of two different populations. Tables 5 and 6 shows the results from the t-test analysis. As noted elsewhere (Muntohar, 2009), wide variability is a problem for soils that does not undergo secondary compression. The results of the t-test (Tables 5 and 6) showed high variances for $c_v$ values determined from FHM when compared to those determined from PSM. Thus, the novel method has been able to reduce the effects of low secondary compression on $c_v$ values for coarse-grained soils. It was also observed that there was no significant difference in the means of each sample for power series and free hand method (two-tail) since $P > 0.05$.

### 3.4.2. Descriptive statistics

Table 7 shows the descriptive statistics for $c_v$ values determined from free hand method and power series method. The aim is to examine the standard error, standard deviation, variance and coefficient of variability between the two results. The standard error, standard deviation, sample variance and coefficient of variation are shown in Table 7.

Table 5: T-test (assuming unequal variances) for samples 1 to 5

| | Sample 1 | | Sample 2 | | Sample 3 | | Sample 4 | | Sample 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Power series | Free hand | Power series | Free hand | Power series | Free hand | Power series | Free hand | Power series | Free hand |
| Mean | 3.521 | 5.599 | 2.773 | 4.272 | 2.484 | 1.838 | 2.950 | 2.601 | 3.186 | 2.695 |
| Variance | 1.379 | 5.386 | 0.507 | 16.19 | 0.255 | 0.318 | 0.283 | 2.531 | 1.288 | 1.101 |
| Observations | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 | 6.000 |
| Hypothesized mean difference | 0.000 | | 0.000 | | 0.000 | | 0.000 | | 0.000 | |
| df | 7.000 | | 5.000 | | 10.000 | | 6.000 | | 10.000 | |
| t Stat | -1.958 | | -0.899 | | 2.089 | | 0.510 | | 0.777 | |

There were high error values, high standard deviation and high sample variance for $c_v$ values determined from free hand method when compared to those obtained from power series method. The coefficient of variation also shows wide variability for $c_v$ values determined from free hand method when compared to the ones determined from power series method. These high error values, standard deviations and wide variations are due to the high sand content in the soils and the shortcomings in the Taylor's normal fit method used to determine the $c_v$ previously. This shows that power series method could be more reliable.

Table 6: T-test (assuming unequal variances) for samples 6 to 10

| | Sample 6 | | Sample 7 | | Sample 8 | | Sample 9 | | Sample 10 | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Power series | Free hand | Power series | Free hand | Power series | Free hand | Power series | Free hand | Power series | Free hand |
| Mean | 3.044 | 2.616 | 2.914 | 2.417 | 3.346 | 2.584 | 3.087 | 4.002 | 3.052 | 3.199 |
| Variance | 0.315 | 2.483 | 0.610 | 1.259 | 0.901 | 3.060 | 0.499 | 1.795 | 0.585 | 1.198 |
| Observations | 6.000 | 6.000 | 5.000 | 5.000 | 5.000 | 5.000 | 6.000 | 6.000 | 6.000 | 6.000 |
| Hypothesized mean difference | 0.000 | | 0.000 | | 0.000 | | 0.000 | | 0.000 | |
| df | 6.000 | | 7.000 | | 6.000 | | 8.000 | | 9.000 | |
| t Stat | 0.627 | | 0.813 | | 0.856 | | -1.480 | | -0.269 | |
| P(T<=t) one-tail | 0.277 | | 0.222 | | 0.212 | | 0.089 | | 0.397 | |
| t Critical one-tail | 1.943 | | 1.895 | | 1.943 | | 1.860 | | 1.833 | |
| P(T<=t) two-tail | 0.554 | | 0.443 | | 0.425 | | 0.177 | | 0.794 | |
| t Critical two-tail | 2.447 | | 2.365 | | 2.447 | | 2.306 | | 2.262 | |

Table 7: Descriptive statistics for $c_v$ values from power series method and free hand method

| | | Mean | Standard | Standard | Sample | Coefficient of |
|---|---|---|---|---|---|---|
| Sample 1 | Power series | 3.521 | 0.479 | 1.174 | 1.379 | 33.340 |
| | Free hand | 5.600 | 0.947 | 2.321 | 5.386 | 41.450 |
| Sample 2 | Power series | 2.773 | 0.291 | 0.712 | 0.507 | 25.676 |
| | Free hand | 4.272 | 1.643 | 4.024 | 16.192 | 94.195 |
| Sample 3 | Power series | 2.484 | 0.206 | 0.504 | 0.255 | 20.290 |
| | Free hand | 1.838 | 0.230 | 0.564 | 0.318 | 30.686 |
| Sample 4 | Power series | 2.950 | 0.217 | 0.532 | 0.283 | 18.034 |
| | Free hand | 2.601 | 0.650 | 1.591 | 2.531 | 61.169 |
| Sample 5 | Power series | 3.186 | 0.463 | 1.135 | 1.288 | 35.625 |
| | Free hand | 2.695 | 0.428 | 1.049 | 1.101 | 38.924 |
| Sample 6 | Power series | 3.044 | 0.229 | 0.561 | 0.315 | 18.430 |
| | Free hand | 2.616 | 0.643 | 1.576 | 2.482 | 60.245 |
| Sample 7 | Power series | 2.914 | 0.349 | 0.781 | 0.610 | 26.802 |
| | Free hand | 2.417 | 0.502 | 1.122 | 1.259 | 46.421 |
| Sample 8 | Power series | 3.346 | 0.424 | 0.949 | 0.901 | 28.362 |
| | Free hand | 2.584 | 0.782 | 1.749 | 3.060 | 67.686 |
| Sample 9 | Power series | 3.087 | 0.288 | 0.706 | 0.499 | 22.870 |
| | Free hand | 4.002 | 0.547 | 1.340 | 1.795 | 33.483 |
| Sample 10 | Power series | 3.052 | 0.312 | 0.765 | 585.000 | 25.066 |
| | Free hand | 3.199 | 0.447 | 1.095 | 1.198 | 34.229 |

## 4. CONCLUSION

Power function were fitted to a set of dial reading versus root of time curves obtained from laboratory oedometer tests for ten (10) soil samples. The soils have high sand content which made the normal dial reading versus root of time curve irregular and thus liable to give unreliable results for coefficient of consolidation ($c_v$). When the power function curve was fitted to the normal curve, it modelled closely, the theoretical U-T consolidation curve to which the original curves are matched with significant improvement in the value of coefficient of determination ($R^2$). The $c_v$ values were determined based on the construction in the normal curves and power function curves. T-test shows that there is no significant difference between the means of the samples for all the samples (two-tail). This shows that power function curves can be used as modification of Taylor's method to determine more reliable $c_v$. Descriptive statistics on the samples also show that there is high standard error, standard deviation and variance for $c_v$ values determined using free hand method. This shows that power function curves should be used to determine $c_v$ values when the soil have high sand content ($\geq 50\%$).

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## 6. CONFLICT OF INTEREST

There is no conflict of interest associated with this work.

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