THE IMPACT OF THE DIAMETER TO HEIGHT RATIO ON THE COMPRESSIBILITY PARAMETERS OF SATURATED FINE-GRAINED SOILS

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Abstract

Compressibility parameters of fine-grained soils are mainly influenced by soil mineralogy, moisture content and soil diameter to height ratio (D/H). The British and American standards suggested that to obtain accurate engineering properties; it is necessary to use D/H ratio of 4 and 2.5 respectively to eliminate friction between the soil and the structure. In the current study, various D/H ratios were adopted ranging from 0.5 to 11. The D/H ratios effect on some compressibility parameters such as coefficient of consolidation (c_v), compression index (c_c) and coefficient of volume compressibility (m_v) were analysed. Additionally, the impact of the D/H ratio on the acquire c_v values were also presented where three methods were used namely: Casagrande, Taylor and Inflection method. The scaling effects based on c_v ratio [$c_v(\sqrt{t}) / c_v(\log t)$] from Oedometer tests using different D/H ratios are also presented. The results showed that Taylor's method is the most appropriate way to achieve an accurate c_v and an increase in pressure leads to a reduction in c_c and a gradual decrease in m_v. The validation of the experimental results on a finite element software package PLAXIS was completed.

***Keywords**: Compressibility, D/H ratio, Fine-grained soil, Friction*

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1. INTRODUCTION

The diameter to height (D/H) ratio criteria is used to reduce friction between the soil and structure. The American Standard suggested a minimum value of 2.5 with a value greater than four most suitable [5]. While the British Standard proposed a value of 4 with a cell diameter of 75mm and height 20mm are mostly applicable [10]. No available data were found to validate the D/H ratio proposed by the American Standard. Morris and Lockington [21] conducted a self-weight consolidation test on fine-grained marine, riverine and lacustrine soils with a diameter to height (D/H) ratio ranging from 1.9 to 4.1. The findings showed that the final void ratio were relatively comparable and was due to the sample particle sizes. Morris and Lockington neglected the effect of D/H ratio from the self-weight consolidation test. Berry and Reid [8] stated that the greater the soil thickness, the more likely friction will occur, and smaller soil sample have no friction.

During conventional consolidation test, the lateral pressure acting on the side walls of the sample container produce frictional resistance to the compression of the sample. Taylor [26] investigated the effect of side friction in consolidation tests and showed that the frictional force varies from 12-22% of the applied pressure for remoulded clay and 10-15% for undisturbed clay. Therefore, the thinner the sample the less side friction due to the small lateral surface area in contact with the wall of the Oedometer apparatus. Leonards and Girault [19] used steel Oedometer cell and show that friction can significantly alter the result. Thus, by applying greased Teflon, it eliminates friction provided the D/H ratios exceed six. Sivrikaya and Togrol [25] investigated the effect of clay thickness on secondary compression. Sivrikaya and Togrol [25] designed a new Oedometer cell capable to measure the frictional effect between the specimen and the cell. The outcomes revealed that side friction does not remain constant with time but slightly increases at a decreased rate. Kolay and Bhattacharya [17] conducted consolidation tests on kaolin soil with Teflon large diameter (120mm) with a D/H ratio six. The comparison of the results of previous tests (D/H ratio 3) with Kolay's investigation showed that c_v is affected by the apparatus side friction and a correction factor for different consolidation characteristics was applied. The correction factor derived by Kolay and Bhattacharya [17] was to reduce side friction.

The effect of D/H ratio scale was not previously investigated on c_c and m_v. The importance of c_c is in the direct calculation of settlement of structures from the relationship of pressure and void ratio. It is used as its value does not change with a change in confining pressure for normally consolidated clays [1, 24]. On the other hand, m_v is also the most suitable compressibility parameters for direct settlement calculation. Its variability with confining pressure makes it less useful when correlating with some engineering properties [24]. Hence, c_c and m_v have been studied to obtain a relationship with some design parameters. These parameters include internal friction angle, recompression index (c_r), moisture content, liquid limit (LL) and specific gravity (G_s).

On the other hand, the effect of D/H ratio on some engineering properties such as shear strength (τ), modulus of elasticity (E) and the stress-strain relationship was previously investigated. These design parameters were studied using Triaxial and Oedometer tests. Dirgeliene and Stragys [14] proposed a reduction in the standard D/H ratio in Triaxial test from 2 to 1 to eliminate friction. While Grisso et al. [15] showed that the D/H ratio of a Triaxial soil sample has little effect on the compacted soil as compared to smaller D/H ratio. On the other hand, during Triaxial testing, it was observed that at constant cell pressure, the modulus of elasticity decreases with an increase in the D/H ratio [18].
In the current study, the effect of D/H ratios on compressibility parameters is presented and it effect of c_v ratio [$c_v(\sqrt{t})$ / c_v (log t)]. More emphasis are employed in the different method used to obtain c_v and the how much impact the D/H ratio has on the rate of consolidation of fine-grained soils. Numerical simulation was also carried out to model test results, using PLAXIS, to show the compatibility of the numerical analysis with the test. During numerical modelling, the Mohr-Coulomb (MC) model that is probably the most used in the modelling of consolidation tests was used. This model is mostly used due to their simplicity, and specific parameters are easily obtained.

2. EXPERIMENTAL PROCEDURE

The sample preparation and test set up as described in the British and American standard were adopted [9, 7]. A well-known consolidation test was carried out using the Oedometer apparatus. This test model the one-dimensional consolidation of subsoil when pressed by structures only in a vertical direction (the lateral strains are considered to be negligible). The test was run over a period of 24 hours at certain load increments under double drainage condition under equal strain (where a porous stone was used for uniform strain loading). Data were obtained using a computerised system connected to each Oedometer test with an accuracy of $\pm$0.1%. The system was able to read data from six channels simultaneously.

At each D/H ratio, the initial moisture content and void ratio were calculated from the tested fine-grained soil. The properties of the soils used (Table 1) were obtained as per the standard methods [4, 9, 3, 7, 6]. In order to get the undrained shear strength (c_u), the soil was compacted at a mould 100mm diameter and the vane size of 25mm was used. The vane was inserted at a depth of 10mm within the compacted soil with moisture content ranging from 35% to 61% with c_u of 0.28 to 12kN/m^2 respectively. The mineralogy analysis of the soil was obtained using X-ray diffraction (XRD), and the principal clay minerals present in the clay is shown in Table1.

Table 1: Soil Properties summary

	LL	**PL**	**PI**	**G_s**	**c_u (kN/m^2)**	**Mineralogy**
Kaolin clay	63 %	32.4 %	30.6 %	2.6	0.28 – 12	Kaolinite and Quartz

Where; LL = liquid limit, PL = plastic limit, PI = plasticity index, G_s is specific gravity and c_u = undrained shear strength (kN/m^2)

2.1 Diameter to Height Scale Range

A series of Oedometer tests were conducted at different D/H ratio as shown in Table 2. For valid comparison in the D/H ratio, the average of the initial moisture content was considered

Table-2: Scale range Oedometer tests

D/H	0.5	1	1.5	2 (a)	2 (b)	3	4	5	6.5	11
Diameter (mm)	100	150	250	150	250	250	100	150	150	250
Height (mm)	200	130	200	80	130	80	23	30	23	23
Test	T1	T2	T3	T4	T5	T6	T7	T8	T9	T10
Initial moisture content	91%	74%	55%	65%	55%	55%	80%	60%	59%	64%
Average moisture content	66%									

2.2 Frictionless Boundary Condition

Previous researchers have shown that in order to reduce friction, Telfon grease is applied to the wall of the Oedometer apparatus by D/H ratio of 6. Kolay and Bhattacharya [17] used Telfon with kaolin soil and found that friction has an effect on c_v. Moreover due to this side friction in the consolidation test apparatus, the vertical pressure on any horizontal plane will decrease with sample depth. As side friction is a function of pressure, it is expected that the e-log p graph becomes flatter with an increase in side friction, thereby producing a small value of the compression index [16]. In the current study, Vaseline was applied to the Oedometer wall to reduce friction. Figure 1 shows the e – log p curve obtained in this study that contradicts the statement by Healy and Ramanjaneya [16] thus side friction was eliminated in this study. The e – log p curve in Figure 1 shows a gradual decrease in void ratio with an increase in pressure except at D/H 0.5 and 5 where the graph becomes flatter. Thus, taking Healy's statement, friction was not reduced at D/H 0.5 and5.

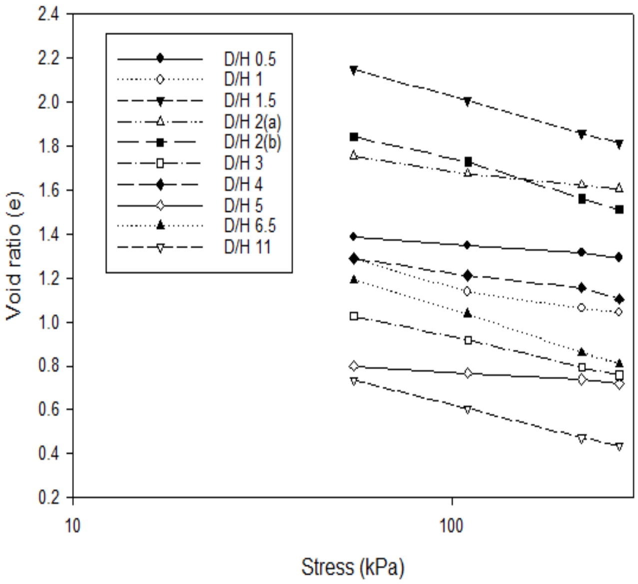


Fig-1: e-log p curve at different D/H ratios

2.3 Frictional Stress

Many studies have been performed to investigate the effect of sample thickness on both primary and secondary compression. Testing soil sample in the laboratory with considerable great thickness will involve friction problem. Sivrikaya and Togrol [25] revealed that the frictional stress does not remain constant during the test and slightly increases with a decrease in time for soil thickness of 60mm and diameter 75mm (D/H 1.25). Friction was found to be most significant at low stresses where the clay soil is still over-consolidated [25]. In the current study, the clay is term normally consolidated, and Figure 1 shows friction was eliminated with reference to Healy and Ramanjaneya [16] statement. Sivrikaya and Togrol [25] derived an expression to obtain the frictional stress (τ) as shown in equation 1.

$$\tau = \frac{T}{\pi DH} \tag{1}$$

Where, D is soil diameter (mm); H is soil height (mm), and T is the load transmitted to the ring (N).

Figure 2 shows the application of equation 1 in the current study. It was observed that at a pressure less than 150kPa, there is no significance difference in frictional stress. The observation was at all scales except at D/H 2(b), 6.5 and 5, where D/H 2(b) being greatly influenced. Figure 2 is contradictory to Sivrikaya and Togrol [25], where it is observed that the friction stress is most significant the high stresses for normally consolidated soils under primary consolidation. The difference in findings is due to D/H 1.25 used by Sivrikaya [25] on overconsolidated soils and 10 D/H ratios utilised in this study on normally consolidated soils. Hence, the present study at D/H 1 and 1.5 normally consolidated can directly be compared with D/H 1.25 over-consolidated which shows the difference in findings is due to the fine-grained soil state.

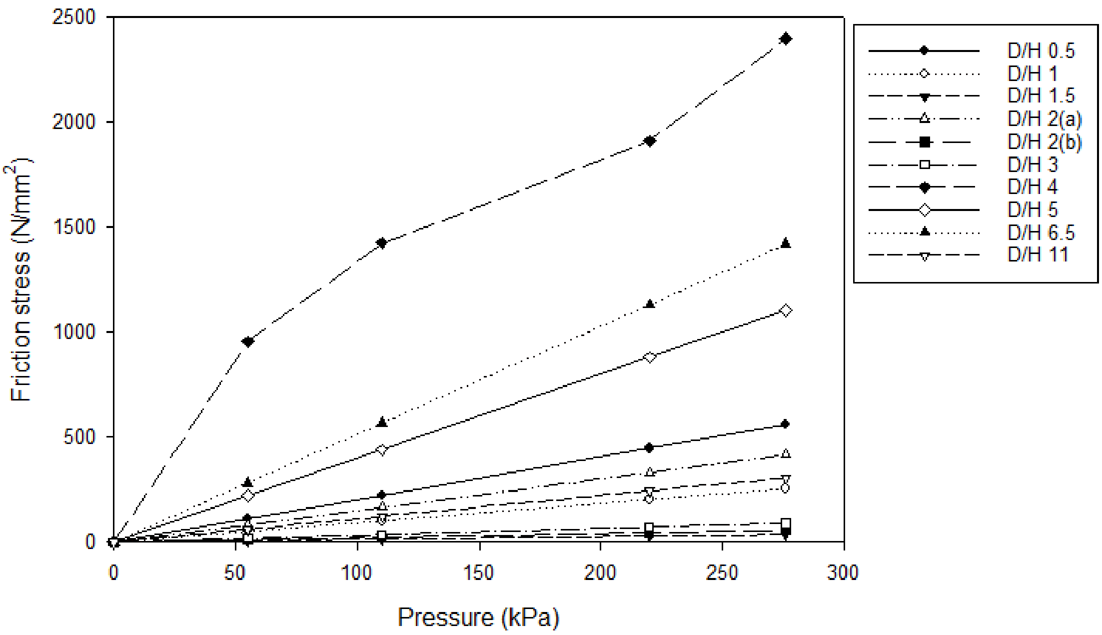


Fig-2: Frictional stresses at various D/H ratios

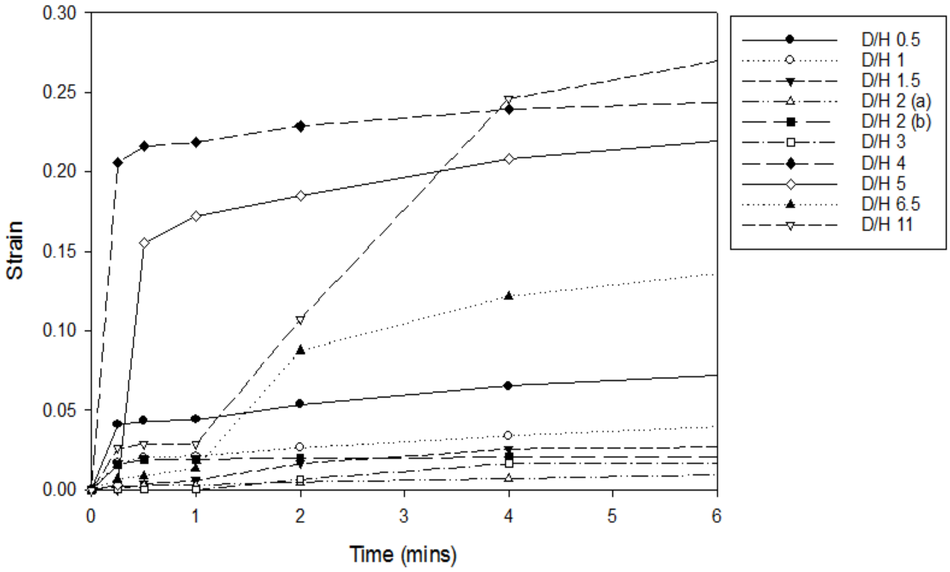
3. RESULTS

In this section, tests arise from the laboratory model, and numerical analyses are shown. A discussion emphasised in the response of fine-grained soils at different D/H ratio scale. Furthermore, its effect on compressibility parameters is presented.

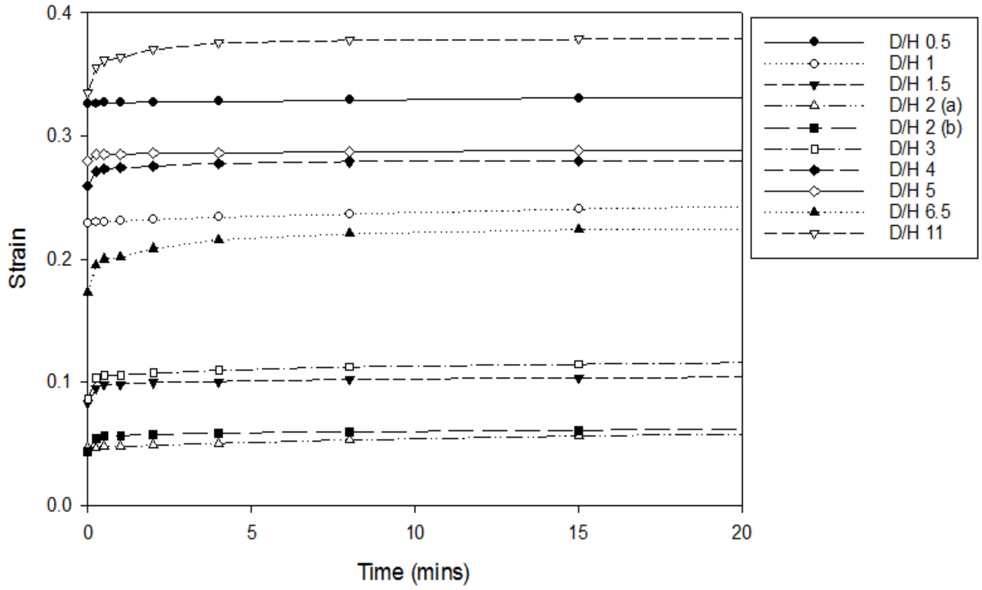
3.1 Experimental Results

3.1.1 Stress-Strain Distribution

The vertical stress distribution within the soil matrix derived from the finite element analysis corresponding to the maximum loading and was normalized by the applied pressure. Figure 3 shows a variation in strain at different D/H ratio under 55kPa and 110kPa. It displays the fluctuation in strain with D/H ratio observed at both loadings. The importance of D/H ratio, as stated in the American standard, is confirmed. At D/H ratio less than 2.5, the curve is difficult to construe, thus provide an uncertainty in the curve fitting procedure in obtaining the coefficient of consolidation (c_v).



a)

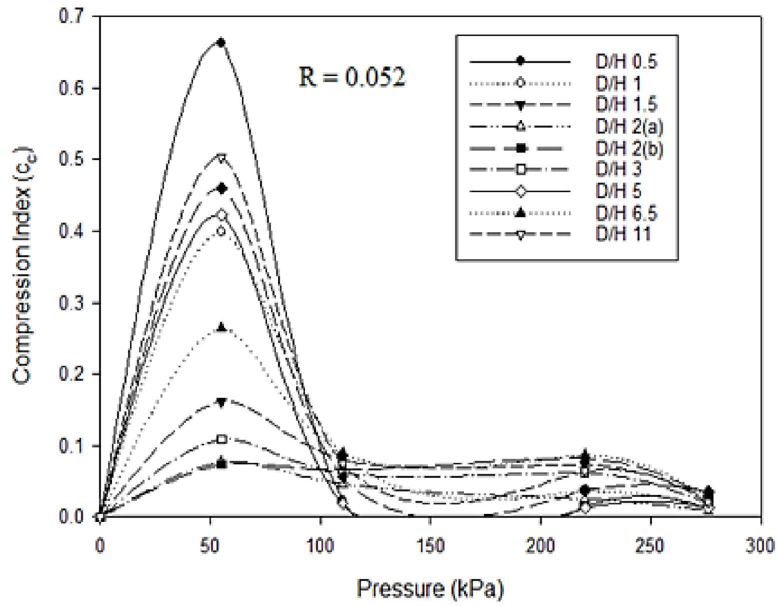


b)

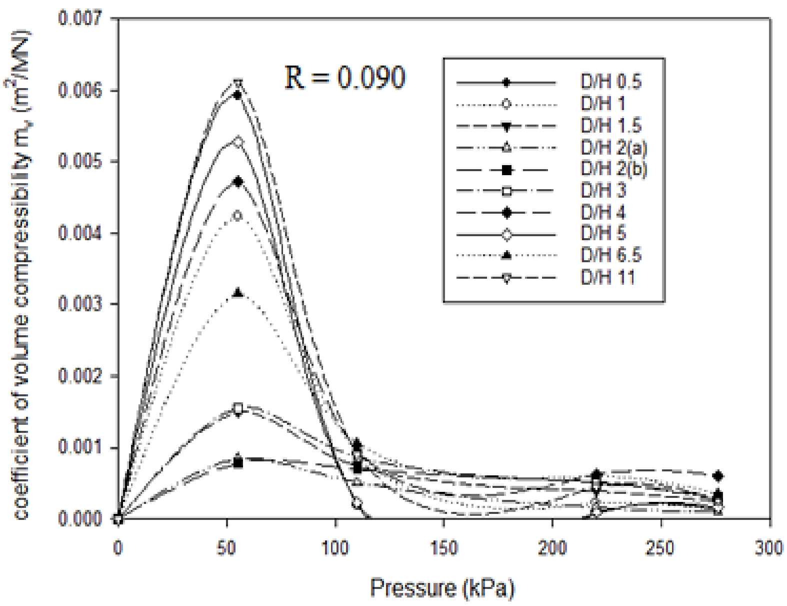
Fig-3: Time-deformation relationship at: a) 55kPa and b) 110kPa

3.1.2 Effect of D/H Ratio Scale on c_c and m_v

Compressibility parameters investigated were: coefficient of consolidation (c_v), compression index (c_c) and coefficient of volume compressibility (m_v). The compressibility parameters c_c and m_v are important in the calculation of settlement of structures. c_c is used to determine the primary consolidation settlement of the normally consolidated soil. Normally consolidated soils are a type of soil whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past [8]. Thus, the soil in this study can normally be termed consolidated. A high c_c value indicates greater compressibility and higher consolidation settlements (Figure 4a). A fluctuation in c_c value was observed with D/H scale, as the load increases, c_c reduces to a value less than 0.1 as depicted in Figure 4a. At 55kPa, there was a sharp rise in c_c and m_v with the maximum value observed at D/H 0.5 and D/H 11 respectively (Figure 4b). The lowest value is discerned at D/H 2(a & b) for c_c and m_v. It does show that D/H has an influence on the magnitude of settlement and kaolin clay compressibility with correlation factor 0.052 and 0.090 for c_c and m_v respectively.



a)



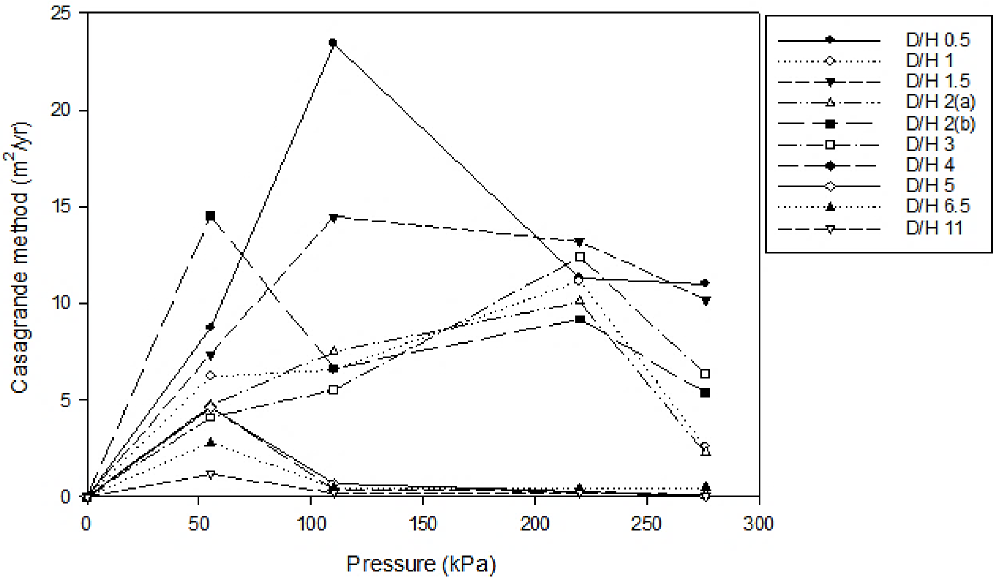
b)

Fig-4: Effect of D/H ratio on some compressibility parameters: a) c_c and b) m_v

Data obtained shows a gradual drop in m_v with pressure at all D/H ratios except at D/H 0.5 and 5. m_v was found to reduce at 220kPa and increase at 276kPa by 72% and 60% respectively (Figure 4b). The findings validate that by Retnamony et al. [22], where m_v was found to decrease with an increase in pressure for kaolinite soil except at D/H 0.5 and 5 at 220 – 276kPa. Thus, D/H scale has an effect on soil compressibility; nevertheless, the outcome is less significant with the overall m_v value being less than 0.006 m^2/MN exhibiting a very low compressibility.

3.1.3 Effect of D/H Ratio on the Ratio of [c_v ($\sqrt{t}$) / c_v (log t)]

The values of c_v were determined using Casagrande, Taylor, and Inflection methods. Statistical analysis was carried out using SPSS for a total of 120 c_v values at various D/H ratios. The analysis demonstrates the statistical significance of the value of c_v obtained using different methods. The outcome showed a normality in c_v values at all scale except at D/H 5. At D/H 5, the values of c_v were abnormal due to an outlier found, which is due to the experimental curve not matching the theoretical curve. There is no substantial difference in c_v value attained using the different method with a sizeable difference of 0.003. Thus, the value of c_v obtained using either test method is not significantly different. Hence, Taylor's method was found to be precisely more adequate to get c_v where scale is a concern (Figure 5b) [26]. As compared to Casagrande and Inflection method, c_v was affected by the curve fitting procedure [11, 13, 20]. The relationship in c_v with pressure obtained using the three method are depicted in Figure 5. The √t curve fitting process was a straightforward process as the primary consolidation part of the curve was easily identified. The issue with the remaining methods at D/H less than 2.5 especially at D/H 0.5 was that the secondary consolidation was not clearly observed rendering difficulty in deriving c_v from curve fitting procedure.



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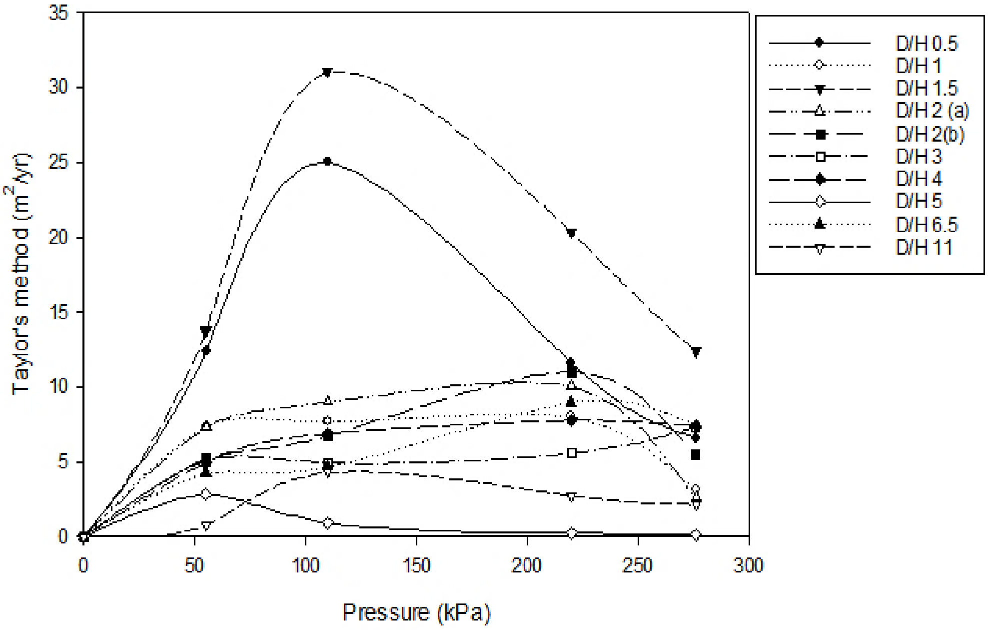

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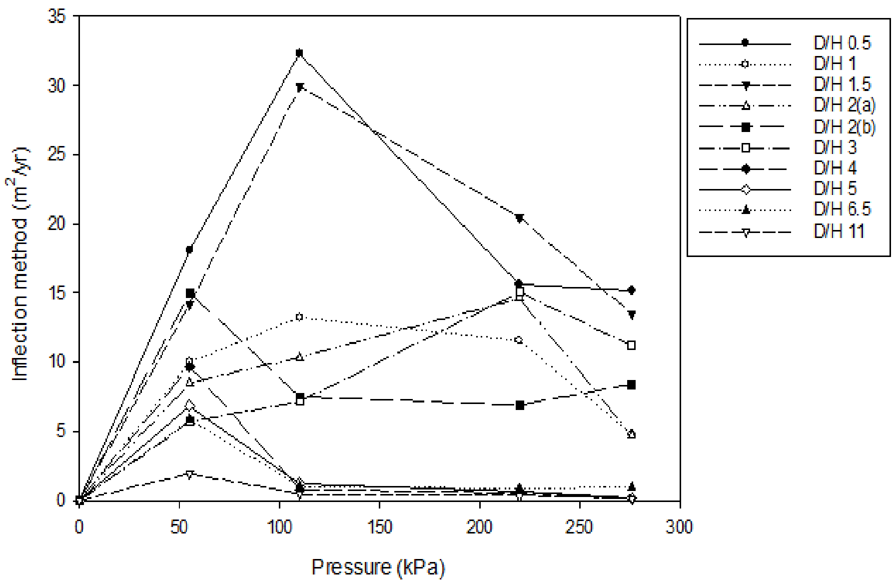
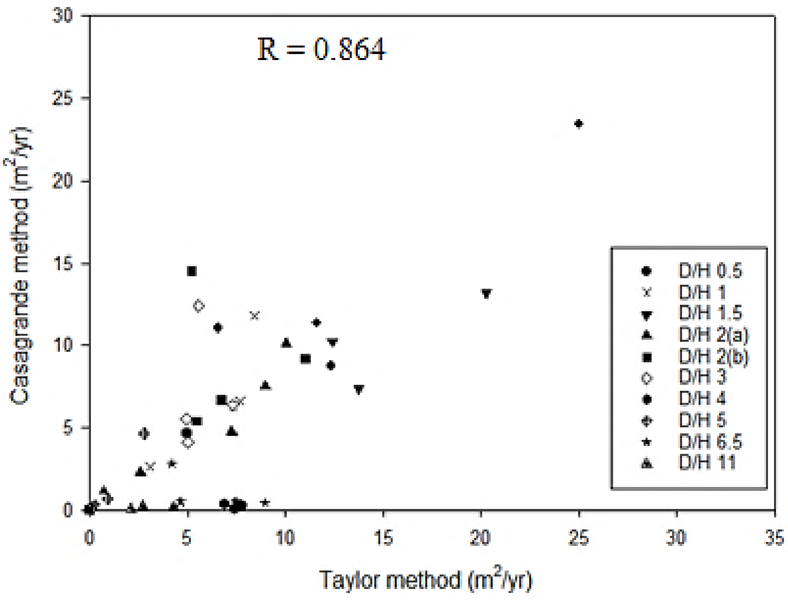


c)

Fig-5: Applied pressure against the coefficient of consolidation (c_v) obtained at a) Casagrande method, b) Taylor's method and c) Inflection method

Previous researcher's showed that c_v obtained using Taylor's method is higher than that attained using Casagrande method and Inflection method gives quite a similar result to that of Casagrande method [2, 12, 20, 23]. The reason being; Taylor's method is affected by the initial compression (leading to an increase in c_v) and in some cases secondary compression (decrease in c_v value). Casagrande and Inflection method are mostly dependent on a certain amount of secondary compression being observed on the strain-deformation curve. In the current study, the opposite was found where the highest value in c_v was attained using the Inflection method, and Taylor's method produced the lowest value. The cause of this is due to D/H scale at a particular scale (D/H < 2.5). Figure 6 shows the ratio of c_v $\sqrt{t}$ / log t_{50}, c_v $\sqrt{t}$ / log t_{60} and c_v log t_{50} / c_v log t_{60} with D/H ratio scale. The outliers in Figure 6 are observed at D/H less than 2.5 and are due to the experimental curve not matching the theoretical curve. The outliers present the uncertainty in the curve fitting process at D/H < 2.5 and the validity of the recommended value by the standard. Therefore, D/H > 2.5 is adopted as per the standard and values showed to be closely packed together. Despite the outliers, the inflection and Casagrande method was found to correlate well as deduced by previous researchers.



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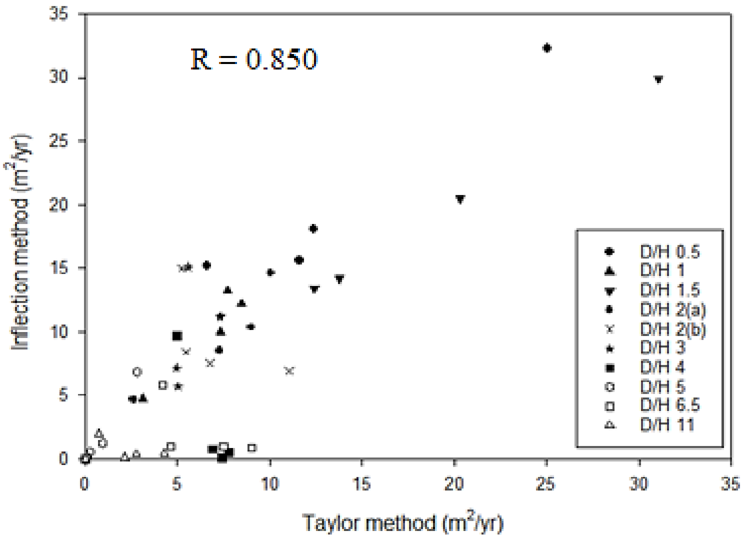

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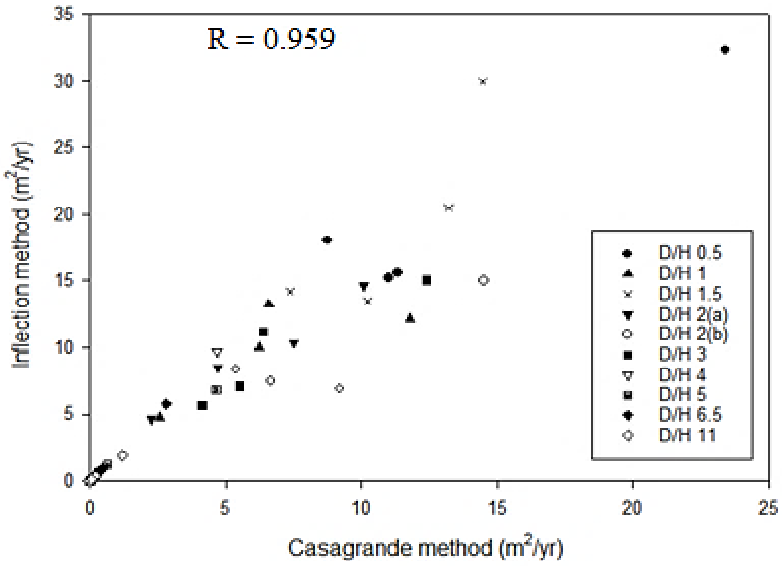


c)

Fig-6: c_v √t / log t ratio at various D/H ratio

3.2 Numerical Simulation

Finite element analyses were implemented to validate experimental results on the validity of D/H ratio scale on fine-grained soils. The D/H ratio model was completed using the Mohr-Coulomb model (MC). The finite element model (FEM) was firstly calibrated against the experimental data.

3.2.1 Finite Element Model

A geotechnical finite element program PLAXIS was used for the simulation of the fully saturated fine-grained soils at diverse D/H ratios. The one-dimensional model was axisymmetric and discretized using 16 nodes elements. The young's modulus was assumed to be 1000 kN/m^2, and a default coefficient of permeability (k) was employed in PLAXIS with value 5.5x10^{-7} m/s. The boundary conditions were set as permeable at both the top and bottom of the soil profile with the side being impermeable. In order to achieve comparative curve to the experimental data, the vertical load was applied in double increments.

3.2.2 Model Calibration

In order to establish the validity of the FEM, it was first compared with the experimental data under hydraulic loading at different D/H ratio scale with an average initial moisture content of 66%. Figure 7 shows the comparison between the experimental data and FEM, which shows that the FEM can simulate the stress-strain deformation of fine-grained soil with good accuracy with a maximum difference of 2%. The properties of the studied fine-grained soil used to calibrate the model as is as shown in Table 3.

Table-3: Fine-grained soil used in calibration of model

Scale	D/H 0.5	D/H 1	D/H 1.5	D/H 2(a)	D/H 2(b)	D/H 3	D/H 4	D/H 5	D/H 6.5	D/H 11
e	2.53	2.49	2.43	1.88	1.97	1.20	2.09	1.50	1.65	1.61
c_u	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00
γ_{sat}	14.33	15.00	14.40	15.20	15.40	17.00	17.30	15.90	15.70	15.8

Where; e is the void ratio, c_u is the undrained shear strength (kN/m^2), and γ_{sat} is the saturated unit weight (kN/m^3).

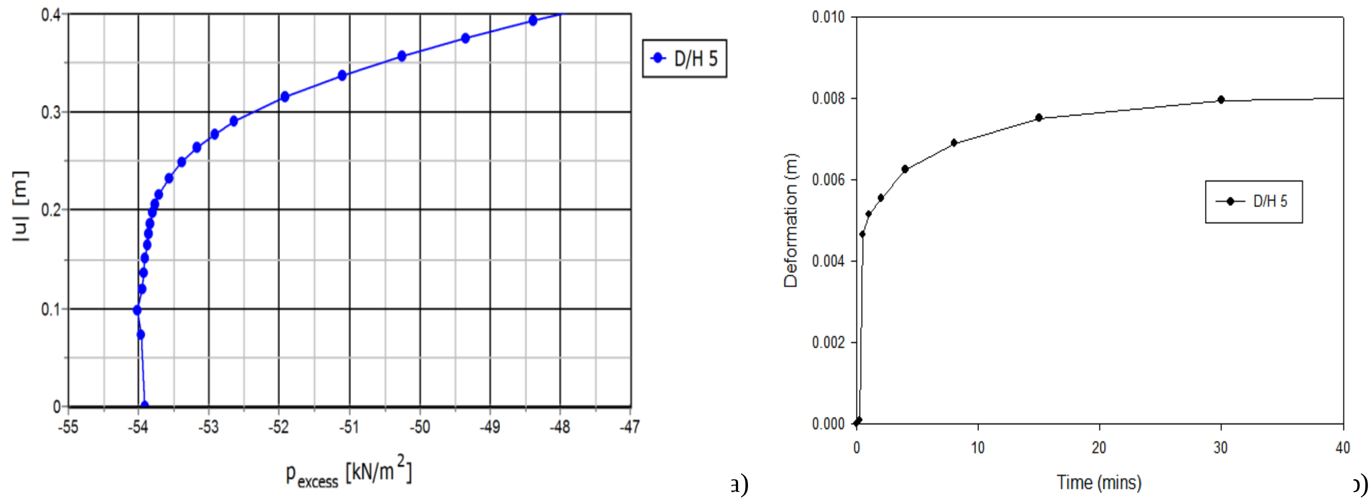
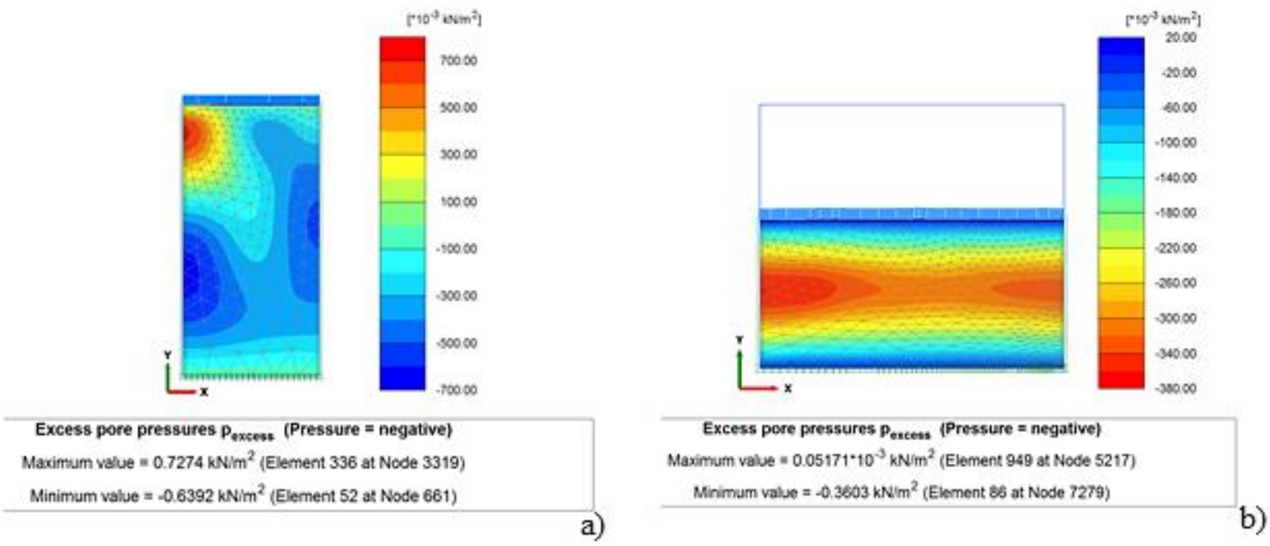
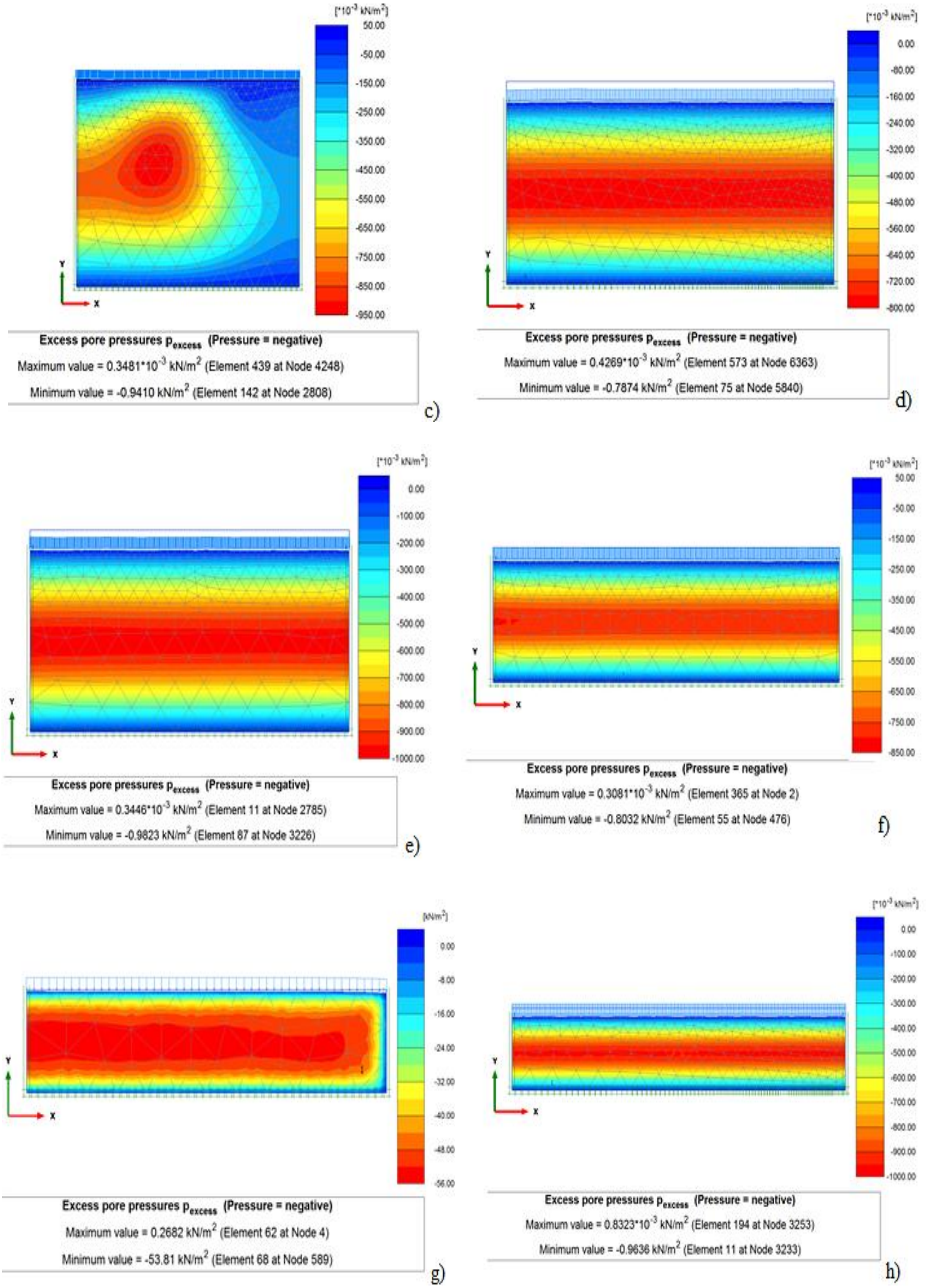


Fig-7: Comparison between experimental and numerical model at 110kPa for D/H 1; a) normalised FEM and b) experimental model. Where; p_{excess} is the excess pore pressure and |u| is the soil deformation

3.2.3 Excess Pore Pressure Distribution

Practically, during consolidation under double drainage, an excess pore water pressure (P_{excess}) is observed at the mid-height of the soil matrix. Figure 8 shows the difference in excess pore pressure at various D/H ratios ranging from 0.5 to 11. It clearly shows that, as the D/H increases, P_{excess} concentration in the mid-height of the soil matrix increases. At D/H 0.5, P_{excess} is observed at the top of the soil, which is mainly due to the scale and applied pressure proportion to the soil profile. The proportion relates to the ratio of applied pressure to scale on a fully saturated soil. Due to the pressure being 55kPa at D/H 0.5 under 24hours, the ratio of the pressure on the soil profile is trivial. However, as the pressure increase, P_{excess} was observed at the bottom of the soil. Therefore, at D/H less than 2 fluctuations in behaviour are found as compared to D/H greater than 2. Thus, the minor difference in the recommended standard method is not significant as the findings still validate that stated in the British and American standard.




Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.3481*10^-3 kN/m² (Element 439 at Node 4248)
Minimum value = -0.9410 kN/m² (Element 142 at Node 2808)
c)
Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.4269*10^-3 kN/m² (Element 573 at Node 6363)
Minimum value = -0.7874 kN/m² (Element 75 at Node 5840)
d)
Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.3446*10^-3 kN/m² (Element 11 at Node 2785)
Minimum value = -0.9823 kN/m² (Element 87 at Node 3226)
e)
Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.3081*10^-3 kN/m² (Element 365 at Node 2)
Minimum value = -0.8032 kN/m² (Element 55 at Node 476)
f)
Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.2682 kN/m² (Element 62 at Node 4)
Minimum value = -53.81 kN/m² (Element 68 at Node 589)
g)
Excess pore pressures p_{excess} (Pressure = negative)
Maximum value = 0.8323*10^-3 kN/m² (Element 194 at Node 3253)
Minimum value = -0.9636 kN/m² (Element 11 at Node 3233)
h)

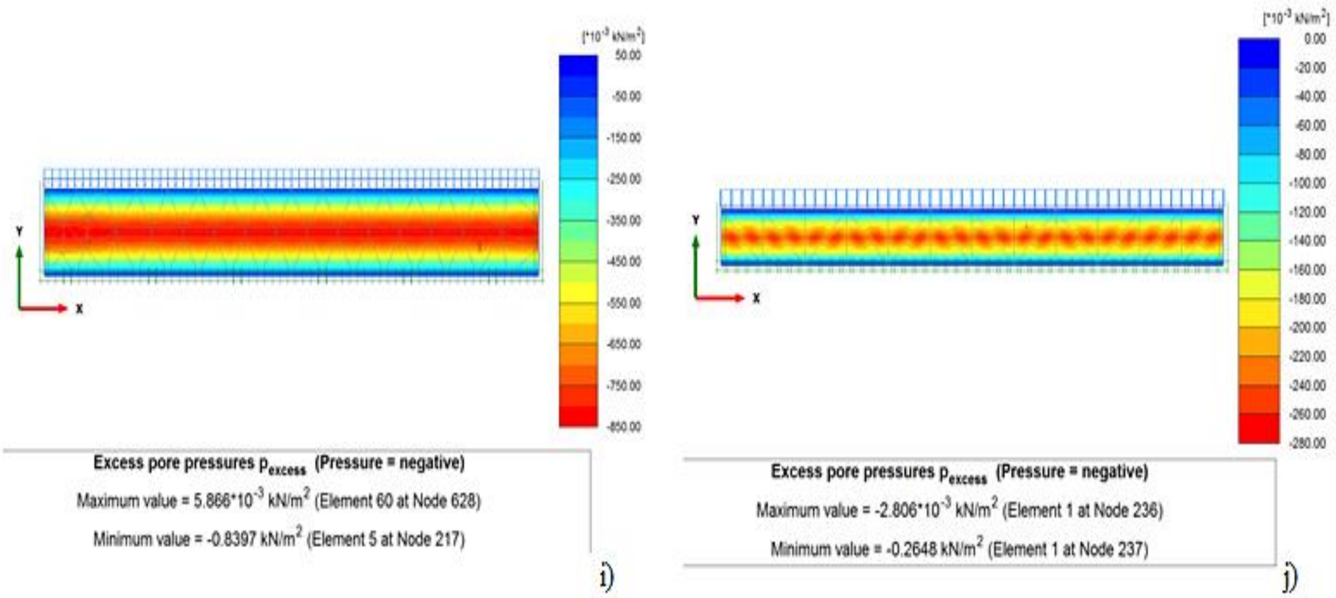


Fig-8: Excess pore water pressure distribution under double drainage at different D/H ratio scale at 55kPa: a) D/H 0.5, b) D/H 1, c) D/H 1.5, d) D/H 2(a), e) D/H 2(b), f) D/H 3, g) D/H 4, h) D/H 5, i) D/H 6.5 and j) D/H 11

In addition, from the trends discerned in Figure 8, D/H yielded the highest excess pore pressure with the lowest being observed at D/H 11. However, the value of D/H 0.5 can be deemed less accurate as the trend does not correspond to the remaining scales. The flaws in trend at D/H 0.5 was observed experimentally where the highest c_v and c_c values were noted at this magnitude. Nevertheless, D/H 1 and 2 (a&b) showed to produce adequate results that are relatively comparable to the remaining scale. The maximum values of P_{excess} at D/H 1.5 and D/H 2(a&b) are more or less analogous by 81% and 98% respectively.

4. CONCLUSION

A study of the influence of D/H ratio on compressibility parameters was investigated. The significant influence of D/H ratio was noticed on c_v but was insignificant in c_c and m_v. It was previously studied that scale has an impact on c_v but not taking D/H ratio into account. D/H ratio was previously investigated at D/H 3 and 6 and showed it greatly affect c_v and D/H greater than 6 recommended. The Standards suggested 2.5 and 4. The current study on Kaolin clay showed that, at D/H greater than 1, adequate result in the compressibility parameters are obtained. Previous recommended D/H ratio values were in different soil sample thus the difference D/H ratio values. D/H 0.5 showed to be most problematic in obtaining the compressibility parameters, and this was observed numerical where the highest excess pore pressure was noted. In addition, Casagrande and Inflection method were strenuous as the experimental curve did not match the theoretical curve. Therefore, in the absence of friction that is not usually obtained in routine testing, it was observed that in terms of D/H ratio, c_v is easily attained using Taylor's method.

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