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Manuscript title: A comparative settlement response of soft clays reinforced with granular columns subjected to widespread loading

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Abstract

This paper examines the consolidation and settlement behaviour of soft clays deposits treated with granular columns (single and in groups). Two Rowe Cell loading chambers were used to conduct the investigations on samples of kaolin and a local Belfast natural estuarine alluvium called ‘sleech’. Tests were carried out on unreinforced samples and then reinforced samples with single and multiple column configurations. The test duration for each test was between 3 and 4 months, depending on the test material and the granular column configuration. The settlement reduction factors based on primary and secondary consolidation were examined. The study suggests that the effectiveness of granular columns at mitigating primary and/or secondary settlement is directly related to the loading intensity, the stress history and the creep characteristics of the subsoil. It was also found that the stress concentration ratio reduced with the stress level. Secondary consolidation also has some effects on the stress concentration ratio.

Keywords: Soft soils; creep; settlement; consolidation & stone column treatment

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INTRODUCTION

Alluvial deposits are globally widespread, and these deposits are typically soft saturated materials composed of clay with varying degrees of silt, and, and organic matters. Alluvial deposits are extremely variable in nature and are normally associated with low undrained shear strength and high compressibility. This combination of poor mechanical properties has contributed to significant construction problems for both short and long-term design situations (CIRIA C573, 2002). The situation is further complicated by the difficulties associated with recovering undisturbed samples of these soils and with their subsequent testing. With increasing pressure to develop marginal soft clay sites for infrastructure developments, there is growing demand on designers to utilise ground improvement techniques such as granular or stone column (also referred to as 'vibro columns') on these soils in particular if the undrained shear strength is not significantly lower than at least 15 kPa (Black *at al.,* 2011). In these soils, there will be significant primary and secondary consolidation settlement components.

The research of McCabe *et al.* (2009) concluded that the technique can be successfully applied on site for bearing capacity and settlement control purposes, however the settlement assessment is semi-empirical and does not address secondary or creep movements. The same authors suggested that there is no consensus on the impact of the technique on arresting post primary consolidation settlement. Thus, the stringent design serviceability limit state can be exceeded within the design life of the structure resulting in service failures (Pugh, 2017). Therefore, developing a better understanding of the mechanisms involved will better assist in deciding the applicability of stone columns (Sexton e*t al,* 2017) for ground improvement purposes. Specifically, this study seeks to explore the onset and magnitude of secondary consolidation settlement within the composite soil mass with respect to time.

Laboratory based model studies and full-scale investigations have been undertaken to understand the load carrying capacity and primary consolidation settlement of such foundations and summaries of selected investigations are given by Serridge (2016) and Pugh (2017). A classical contribution to the state of the art for this specific topic is based on laboratory investigations carried out by Hughes *et al.* (1974). This foundational work was expanded upon by Charles *et al.* (1983), Bachus *et al.* (1984), Narashima Rao *et al.* (1992), Hu (1995), Muir-Wood *et al.* (2000), McKelvey *et al.* (2004), Black *et al*. (2008), Black *et al.* (2011), Shahu and Reddy (2011), Cimentada *et al*. (2014) and Jeludin *et al*. (2016). These studies have been complemented with numerous field studies, looking into both bearing capacity and settlement performance (Hughes *et al.*, 1975; Greenwood, 1991; Bergado *et al.*, 1987; Cooper *et al*., 1999; Slocombe e*t al.* 2000; Watts *et al.*, 2000). Centrifuge testing was undertaken by Fragaszy *et al*. (1981), Almeida *et al.* (1985), Greenwood (1991), Al-Khafaji *et al*. (2000), Lee *et al.* (2004), Weber (2004) and Weber *et al.* (2010). Subsequently numerical and empirical studies have been undertaken to validate the observed performance of granular columns subject to wide (raft) and isolated (pad or strip) loading conditions and in a range of configurations (Priebe (1995); Castro *et al*. (2010); Sexton *et al*. (2017)). All the above references suggest that the methodology is fit for purpose and reliable, however, available information in relation to creep settlement is limited (here onwards the term "creep settlement" is adopted), and current design methods do not fully model the long-term changes within a composite soil mass, thus more work is needed in this area. A recent numerical study by Sexton *et al.* (2017), concluded that where granular columns are used in deposits prone to creep, the evolution of the creep settlement improvement factor is lower than otherwise obtained based on primary consolidation settlement alone and validated using experimental evidence reported by Moorhead, 2014.

Granular columns are essentially used for three different applications:

(a) Under embankment loading. In this case, the interface between the embankment and underlying soft deposit is flexible. Design for embankment loading is typically based on primary consolidation, which may, depending on the properties and thickness of the underlying soils, largely be completed before completion of any finished surfaces e.g. finished road surface and services (McCabe *et al.,* 2009). Most of the settlement reduction achieved at the end of construction is attributed to the rapid drainage and consolidation facilitated by the presence of the highly permeable granular column.

(b) Under ground-bearing floor slabs for industrial warehouse. In this case, the loading is not realised until after completion of the structural slab installation where the principal load contribution is typically from live loading and Nevertheless, with a lack of academic agreement on the influence of granular columns on the aspect of creep settlement it is important to assess whether the remaining post construction settlement magnitudes can be accommodated within the normal serviceability limits of the structure (McCabe *et al*., 2009 and Sexton *et al.*, 2017), and this forms the premise of this article.

Using a bespoke laboratory arrangement, this paper aims to investigate primary consolidation and creep settlement of single and multiple granular column arrangements in soft clay, under widespread loading. Due to difficulties, the boundary conditions adopted in the present investigation were rigid, thereby representing the conditions below, for example, typical floor-slabs used in industrial warehouses.

MATERIALS AND METHODS

The experimental programme involved the testing of two different fine-grained materials. The first material was pure kaolin, which is a non-active material, which exhibits marginal creep settlement. The second material was a Belfast post glacial estuarine deposit, locally known as 'sleech' (Lehane, 2002; Philips *et al.,* 2011). This material has been shown to be an active material, exhibiting considerable creep settlement (Glossop *et al.*, 1979; Davies *et al.*, 1981; Lehane, 2002). Within the remainder of this manuscript, this material will simply be designated 'sleech'.

The inert nature of the kaolin was chosen as a suitable comparison material against the non-inert clay (sleech) with a considerable creep tendency. Both materials were tested for the settlement performance of a foundation under identical stress conditions. Kaolin is commercially available and ideal for preparing large samples at pre-selected water contents with good repeatability (McKelvey *et al.,* 2004; Black *et al.,* 2006; Jeludin *et al.,* 2016). The relevant index properties of the kaolin are listed in Table 1. The post-estuarine clay or sleech was collected from a site located close to Queen's University Belfast (QUB), Northern Ireland. Sleech is a dark greyish clay, which often contains mixed silt, fine sand and sea-shells. This deposit underlies the banks of the River Lagan and most of central Belfast area (Philips *et al.,* 2011). Bulk samples of this material was transported in sealed containers to QUB for testing. The index properties of this material are also listed in Table 1. The amount of organic matter in the sleech from this test location was 8%.

Current design specifications (BRE BR391, 2010) stress the grading curve of the gravel should be appropriate for the creation of a dense interlocking column. The sizing range is a function of the method of installation with dry top feed using 40mm to 75mm sized stones. Conversely dry bottom feed and wet top feed use 20 to 50mm gravel. In all cases, fines shall not exceed 5%. To reflect a common practise and to allow for the variation in the installation methods, a prototype material with D_{10}, D_{30} and D_{60} equivalent to 30 mm, 37.5 mm and 45 mm was chosen. This is reflective of BR 391 (2010) good practice. Using the laws of similitude and adopting a 1/15th scale, the model gravel adopted was evenly graded crushed basalt. This material was washed and sieved to creating grading sizes D_{10}, D_{30} and D_{60} of 2.0 mm, 2.5 mm and 3 mm. In both cases the coefficient of uniformity ($C_u = D_{60}/D_{30}$) was 1.5 and the coefficient of curvature ($C_c = (D_{30})^2/(D_{30}*D_{60})$) was 1.04.

Two Rowe Cell loading chambers were developed as illustrated in Figure 1. Each consolidation chamber had an internal diameter of 254 mm, a height of 150 mm and drainage was permitted only from the base of the chamber. The height of the chamber was limited to 150 mm, as a sample any longer than this would inherit a significant side friction between the clay and the chamber during consolidation which could potentially hinder the creep rates. Each system consisted of: (a) pressure transducers to monitor the pore water pressures along the length of the sample at two locations (see Figure 1); (b) three pressure cells (20 mm diameter) located on the base plate to measure the pressure under the column and surrounding clay; (c) displacement gauge to monitor the vertical settlement; (d) pneumatic pressure controllers to monitor and maintain a constant back pore water pressure and overburden pressure; and (e) a volume change unit to monitor the quantity of water that drained during testing.

Reconstituted kaolin was prepared for each test to a target water content of 55% by mixing 6 kg of dry kaolin powder with 3.3 kg of de-aired water using an electric mixer. In the case of sleech, the material was remoulded at its natural water content (38% derived from testing of 20 randomly selected specimens) in a rotating pan mixer. Once thoroughly mixed, the remoulded clay was removed from the mixing drum into heavy duty sample bags and secured with cable ties to ensure air-tight storage of the material. In order to assess sample uniformity, random samples were taken from the mixed material and found to be consistent. The approximate undrained shear strengths of kaolin and sleech (obtained using vane were 15 kPa and 18 kPa respectively.

The clays were hand kneaded into the one-dimensional consolidation chamber, while taking care to avoid air entrainment, thus ensuring a uniform homogeneous clay test bed. Once the chamber was filled, the clay was levelled to the required test height of 120 mm. A rigid piston plate combined with a rolling diaphragm was positioned and secured using the standard testing procedure. In the case of the unreinforced clay bed, normally consolidated samples of both materials were subjected to four loadings corresponding to effective overburden pressures of 25 kPa, 50 kPa, 100 kPa and 200 kPa at a constant back pore water pressure of 200 kPa. Tests were also carried out on overconsolidated samples of both soils. In these tests, the sample was initially subjected to 100 kPa of effective overburden pressure and upon completion of the consolidation phase, the pressure was reduced to 25 kPa. This was then followed by the loading increments specified above. Each loading increment was held constant for sufficient time to allow all three stages of settlement to develop fully. The required duration of each stage of loading was approximately 2 weeks and 4 weeks for the kaolin and sleech respectively. In the case of the reinforced samples, the granular columns (single or multiple) were installed after the samples had been initially consolidated to an effective overburden pressure of 25 kPa (i.e the overburden pressure was removed under undrained conditions and subsequently reapplied after the installation of the columns). An identical process was adopted in the case of overconsolidated samples, but in this case the materials were consolidated to 100 kPa and then unloaded to 25 kPa.

The granular columns were installed using the technique adopted by Black *et al.* (2011) and Jeludin *et al.,* (2016). A template was used to position the columns accurately thus ensuring that the bottom of the column aligned with the pressure cell located at the base. Thin-walled hollow tube was used to create holes in the clay bed depending on the diameter (discussed later). The tube was pushed into the clay in a uniform manner to minimise sample disturbance. Initially, a 3mm thick layer of sand was carefully placed at the bottom of the cavity, followed by a layer of filter paper. This methodology was adopted to protect the pressure cells located at the base of the testing chamber (Figure 1). The granular column was formed by uniform compaction and subsequent saturation of the crushed basalt in 5 layers until it was level with the surface of the clay bed. The level of compaction effort influences the overall performance of the composite material (Jeludin at al. 2016: Miranda at al., 2015). The compaction effort used in the present application was moderate as any intense compaction would deform the column laterally. For that reason, the relative density of the columns erected was approximately 60%. This is only an approximate value as it was not possible to measure the diameter of the columns after installation, and it was assumed that it was equal in diameter to the pre-formed cavity. The testing schedule for the investigations required the construction of a full depth column, approximately 120 mm long (depending on the clay thickness at the end of initial consolidation) of compacted crushed basalt. Single column (100mm diameter) and multiple columns (7 columns each having 37 mm diameter) arrangements were investigated. Both arrangements correspond to an area replacement ratio (A_s) of 15%. Note that Figure 1 only shows the column arrangements for multiple columns.

For the above testing configurations, the ratios between the height of the column H, to effective diameter d_e (H/d_e) is approximately 0.94 and 3.2 respectively for single and multiple column arrangements. Based on these ratios, the single column arrangement is somewhat unrepresentative of the full-scale application, but the multiple column arrangement may represent a near-field scenario where arrays of short columns are located under floor-slabs. A much higher H/d_e ratio could have been examined by increasing the chamber height, however that could have seriously impeded the purpose, as the side friction between the chamber and the clay would have likely reduced any potential creep settlement.

Method of analysis and interpretation

There are many hypotheses detailed in research literature in relation to creep settlement. One widely held view is that creep settlement commences after the dissipation of excess pore water pressure. An opposing argument is that the creep settlement begins as soon as the loading regime has changed (Yin and Graham 1990; Yin e*t al.*, 1994). However, this approach posed difficulties in determining the consolidation settlement in the present investigations and thus for simplicity, it was assumed that the creep settlement becomes more pronounced towards the end of consolidation stage.

The consolidation and creep settlements were examined when the effective overburden pressures were increased from 25 kPa to 50 kPa, 50 kPa to 100 kPa and 100 kPa to 200 kPa. The overburden pressures were initially applied under undrained conditions and the system was permitted to equalise for at least 8 hour period before drainage was permitted. Figure 2 shows the equalization of pore water pressure within the clay beds for both kaolin and sleech, which confirms the time allowed for the equalization of pore water pressure was sufficient before the commencement of drainage provisions. The observed performances are interpreted in the following manner:

Consolidation settlement $\Delta s_{c(NCo)} or\ \Delta s_{c(c)}$ Equ. 1

Settlement reduction factor $n = \Delta s_{c(NCo)}/\Delta s_{c(c)}$ Equ. 2

Creep rate $c_{\alpha(NCo)}\ or\ c_{\alpha(c)} = \Delta\varepsilon_{\alpha}/\Delta\log(t)$ Equ. 3

Coefficient of consolidation $c_{v(NCo)}$ Equ. 4

Creep reduction factor $n_{\alpha} = \Delta s_{\alpha(NCo)}/\Delta s_{\alpha(c)}$ Equ .5

Stress ratio $n_{\sigma'_v} = \sigma'_{v(c)}/\sigma'_{v(NCo)}$ Equ. 6

The subscript "NCo" refers to clay without columns and "c" to clay with columns. The additional terms are: creep strain $\Delta\varepsilon_{\alpha}$; time t; vertical stress σ'_v. The stress ratio $n_{\sigma'_v}$ is analysed based on effective stresses as drained conditions prevailed at the bottom boundary where the pressure cells were located. In addition to the above analyses, a limited analysis was carried out to predict the reduction in consolidation time factor b as defined below:

Consolidation time reduction factor $\beta = t_{90(NCo)}/t_{90(c)}$ Equ. 7

where $t_{90(c)}$ and $t_{90(NCo)}$ are time required for 90% consolidation for soil bed with and without granular columns. Three-dimensional consolidation theory was used to predict the $t_{90(c)}$ and the following formulations are used.

Overall degree of consolidation U: $(1-U) = (1-U_v)(1-U_r)$ Equ 8

Coefficient of consolidation c_v (vertical) $c_v = \frac{T_v}{t} h^2$ Equ 9

Coefficient of consolidation c_r (radial) $c_r = \frac{T_r}{t} d_e^2$ Equ 10

where U, U_v, and U_r are degree of consolidation overall, vertical and lateral respectively. h, d_e c_v , c_r T_v and T_r are: the maximum flow length in the vertical direction, effective diameter (for single and multiple column configuration), coefficient of consolidations in vertical radial directions and time factors in vertical and radial directions respectively. The chart reported by Han and Ye, 2001 was used to determine the relevant values for U_v and U_r for given time factors T_v and T_r respectively.

RESULTS AND DISCUSSION

Settlement performance

Kaolin: Figure 3 (a & b) shows the settlement of the kaolin clay bed (untreated soil) and the dissipation of excess pore water pressure, measured at Location 1, plotted against log time for the three loading increments (i.e. 25-50 kPa, 50-100 kPa and 100-200 kPa). The classical Casagrande (1936) graphical construction was used to identify the time $t_{100(\Delta s)}$ to 100% consolidation settlement. The term (Δs) refers to consolidation time based on primary settlement only and is denoted with an arrow in Figure 3a. The $t_{100(\Delta s)}$ ranges between 12-20 hrs and this corresponds reasonably well with the time required for dissipation of excess pore water (Figure 3b), indicated by $t_{100(PWP)}$. In this instance PWP refers to consolidation time based on dissipation of pore water pressure. The corresponding estimated values of $c_{v(NCo)}$ are tabulated in Table 2. Also note that the magnitudes of the excess pore water pressures developed under undrained conditions were not the same as the change in overburden pressures. This may have been primarily due to slight flexibility of the systems (Northey and

Thomas, 1965), possible gas generation due to a prolonged testing period, and also condition at the "zero" time is not included in Figure 3b.

The coefficient of creep rate, (C_α) given by Equation 3 was used to examine the creep settlement response of the same clay bed. In Equation 3, $\Delta\varepsilon_\alpha$ refers to the vertical strain during the creep phase of movement. Figure 3a demarcates the vertical compression range of the clay bed under each loading. Strain calculations were based on the thickness of the clay bed prior to each loading stage. The relevant creep rates of the clay without granular columns, $c_{\alpha(NCo)}$, are 0.22, 0.20 and 0.18 corresponding to load increments of 25-50 kPa, 50-100 kPa and 100-200 kPa respectively (Table 3). These results which are broadly similar suggest that the creep rates are relatively constant for the plain kaolin test-bed (normally consolidated) and not influenced significantly by changes in overburden pressure. The estimation of the creep component to the overall settlement profile thus allowed the consolidation settlement, $\Delta s_{c(NCo)}$ (Equ. 1), to be determined for the relevant loading increments. The corresponding settlements $\Delta s_{c(NCo)}$ are tabulated in Table 3.

Figure 4 shows the settlement and excess pore water pressure dissipation of the clay bed (measured at location 1) containing a single 100 mm diameter granular column Similar graphical constructions to that used previously were used to determine the consolidation settlement and the required time for 100% settlement. In general, the duration for consolidation settlement, $t_{100(\Delta s)}$, was in the region of 5-10 hrs for the three stages of loading, which also corresponds reasonably well with the pore water pressure response obtained at Location 1 (Figure 4b). The relevant creep rates, $c_{\alpha(c)}$, are 0.14, 0.20 and 0.18 corresponding to loading stages of 25-50 kPa, 50-100 kPa and 100-200 kPa respectively. The relevant creep reduction factor, n_α, (Equ 5) ranges between 1 and 1.6 (Table 3) and it appears that the presence of granular columns did not significantly reduce the creep settlement, except at low overburden pressure. This is consistent with observations from the numerical simulation undertaken by Sexton *et al*., (2017).

The settlement improvement factor, n, (Equ 2), is the key design criterion considered when deciding the choice of ground improvement method (Pribe, 1995; McCabe *et al.*, 2009; Serridge, 2016; Pugh, 2017). The relevant settlement reduction factors, n, for the clay bed with a single column are 5.5, 1.3 and 1.3 for loading ranges 25-50 kPa, 50-100 kPa and 100-200 kPa respectively. In relation to the first load stage (i.e. under low applied stress), the settlement reduction factor, $n,$ and the creep reduction factor, $n_{\alpha},$ are significant. However, reductions in both n and n_{α} are of a similar order in magnitude (except at low overburden pressure), and the settlement reduction factor, including the influence of creep under high loading, is not considered to be significant (Table 3). These values (i.e. in relation to consolidation settlements) agree favourably with some of the published settlement reduction factors by Greenwood (1970); Vautrain (1977), Munfakh *et al*. (1983), Pugh, (2017) and Sivakumar *et al*. (2019).

Figure 5 (a & b) shows the settlement and excess pore water pressure dissipation of the kaolin clay bed containing multiple columns (7×37mm diameter), with an area replacement ratio of 15%. This percentage replacement is the same as the single column of 100 mm diameter. The improvement with respect to the settlement reduction factor, $n,$ and the creep rate reduction ratio, $n_{\alpha},$ are 2.4, 1.3 and 1.3 and 1.6, 1.3 and 1.1 respectively for the standard test loading ranges. Once more it is apparent that, under elevated loading, the granular columns (in a group) are also not as effective at reducing settlement both due to consolidation and creep.

Based on the above observations, it was evident that granular columns offered significant improvement in relation to consolidation settlement and creep under low loading (i.e. 25-50 kPa). Under this loading condition, it is possible that the soft clay surrounding the granular column exhibits an overconsolidated behaviour in terms of stress history (more on this will be later in this article). Thus, it was conjectured that such composite systems offer reasonable performance if the confining clay is still overconsolidated. Therefore, further testing was conducted using an overconsolidated clay bed. To ensure the test bed was overconsolidated, the clay bed was subjected to 100 kPa of initial consolidation and subsequently unloaded to 25 kPa before installing either the single or multiple granular columns. For comparison and as part of the testing quality control, an overconsolidated clay bed without granular columns was also studied. Due to lack of space, the observations pertaining to these tests are not reported graphically, but the readers can access this information in Moorhead (2013). The relevant settlement reduction factors n and creep reduction factors n_{α} are listed in Table 3. In general, the improvements in relation to n and n_{α} are significant during the first two loading increments, but tended to reduce at the highest loading. This confirms the earlier postulation that the settlement reduction factors are generally significant if the clay is in an overconsolidated state. The above observations are based on material that exhibits marginal creep behaviour. An opportunity to carry out the same investigation with clay that exhibits pronounced creep has further substantiated the hypothesis conjectured above and the relevant observations are reported later in this article.

The most beneficial effect of the granular column applications is that it leads to rapid dissipation of excess pore water pressure and therefore a significant completion of primary consolidation settlement during construction. Therefore, an attempt is made to assess the consolidation time reduction factor b using three-dimensional consolidation theory. In doing so, it was assumed that $c_{v(NCo)} = c_{r(NCo)}$ and the magnitude of it was determined using the consolidation characteristics observed on the clay bed without granular columns (Figure 3). The values of $c_{v(NCo)} = c_{r(NCo)}$ are listed in Table 2 together with the time required for 90% consolidation time $t_{90(NCo)}$. The predicted and measured $t_{90(p)}$ using the formulations given by Equations 8-10 are also listed in Table 2. The predicted values of b range from 2.0 to 4.8 and the values are comparable with the measured values as a first approximation for a single column application. However, differences are significant in the case of multiple columns where the measured consolidation time reduction factor b is significantly lower than the predicted values. The reason for this can be attributed to smearing effects caused during the installation of 7 individual columns.

Sleech: The observations referring to the normally consolidated clay bed with and without granular columns are reported here. Although tests on overconsolidated clay beds are reported, due to space considerations the associated figures could not be included in this paper. Figure 6 shows the settlement of the sleech clay bed and the corresponding dissipation of excess pore water pressure plotted against log time for the same loading conditions, for the non-treated clay bed. The required time for 100% consolidation, ($t_{100(\Delta s)}$), is 300 hours for the first loading increment (25-50 kPa) and slightly less for the other loading stages. These durations are in reasonable agreement with the estimated consolidation time based on the dissipation of pore water pressure ($t_{100(PWP)}$) . The relevant creep rates, ($c_{\alpha(nc)}$), are 0.96, 1.17 and 0.93 for loading stages 25-50 kPa, 50-100 kPa and 100-200 kPa respectively. These creep rates are 5 times greater than the corresponding creep rates estimated for the kaolin soil, thus validating the use of sleech as a suitable testing material for the present research. Furthermore, the results confirm that the creep rates are generally similar in all three stages of loading. Approximate consolidation settlements, ($\Delta s_{c(nc)}$), for the relevant loading increments are tabulated in Table 4.

Figure 7 shows the settlement and pore water pressure dissipation of the sleech clay bed with a single 100 mm diameter granular column, with an area replacement ratio of 15%. The relevant settlement reduction factors n, for the sleech with a single granular column are 3.7, 1.7 and 1.2 respectively for 25-50 kPa, 50-100 kPa and 100-200 kPa stress increments. The creep rate reduction factors, n_α, are approximately 1.8, 1.5 and 1.5. The improvement factors are significant under low loading ranges (25-50 kPa and 50-100 kPa) as was the settlement improvement. However, at high loading range, the improvement was marginal.

Figure 8 shows the settlement and excess pore water pressure dissipation of the clay bed with multiple columns (7×37mm diameter The improvements with respect to settlement reduction factor,n were 4.0, 2.6 and 1.3 respectively for the test loading ranges. The corresponding creep reduction factor, n_α , observed were 2.1, 1.5 and 1.5 for the same loading conditions. This part of the test programme confirms that granular columns are effective in reducing both consolidation and creep settlement under low to moderate loading, but less effective at elevated loading levels.

Investigations were also carried out on an overconsolidated sleech clay bed. The settlement reduction factor n and creep reduction factor n_α are listed in Table 4. The settlement and creep reduction factors are significant during all three stages of loading, although the creep reduction factor tends to unity under higher loading. This confirms the earlier observations that granular columns offer competent performance provided the surrounding material to the stone columns is in an overconsolidated stress state. This overall assessment, in particular the consolidation settlement reduction factors, was compared with the existing data reported by several researchers (Vautrain, 1977; Munfakh *et al*., 1983; Pugh, 2017; Sexton *et al*., 2017) which supported the current experimental findings.

As in the case of kaolin, the consolidation time reduction factors were predicted and measured based on 90% consolidation. The magnitude of $c_{v(NCo)}$ ($=c_{r(NCo)}$) together with t_{90} (m) are listed in Table 2. The predicted and measured $t_{90(p)}$ using the formulations given by Equations 8-10 are also listed in Table 2. The predicted values of b range from 4.6 to 9.3 and there appears to be no consistent pattern for comparison with measured b values. However, in the case of multiple columns, as observed in the case of kaolin, the predicted values of b are generally higher than the measured values.

Load Carrying mechanism

The stress concentration factor, $(n_{\sigma'_v})$, defined by Eq. 6 is an important parameter in the design of foundations supported on soft clay reinforced with granular columns (Pugh, 2017). Due to the stiffness differences between the granular column and the surrounding clay, it can be expected that the column will carry a larger proportion of the foundation loading, Najjar (2013). The load-carrying mechanism of the composite material (i.e. granular columns surrounded by soft clay) is complex; the load distribution cannot be assessed based on relative stiffnesses (modular ratio) alone. The research of Najjar (2013) observed, that the stress ratio is not constant and is affected by strain levels. Other factors that impact the column soil loading are the ratio of the working load and ultimate load carrying capacity, the consolidation time of the soil and the drainage conditions. Existing information is very limited on these factors and the observations made during this research yielded some useful data about these factors, including the influence of creep.

With the application of loading, the columns and the surrounding clay carry the applied stress in different proportions. However, the columns immediately reach fully drained conditions, whereas the surrounding clay is under undrained conditions. As the excess pore water pressure dissipates with time, the supporting clay becomes gradually fully drained. As the clay consolidates, it transfers some of the loading to the granular columns, but it also gains strength and stiffness. In addition, the consolidation of the clay also exerts negative skin friction or down drag on the columns (Sivakumar *et al.*, 2012). The combined effects of these contribute to a lateral expansion or bulging of the column, leading to a reduction in column loading, but an increase in clay loading (i.e. both the vertical and lateral stresses on the clay). This process continues until a condition of equilibrium is achieved, at the end of primary consolidation. However, in soils that exhibit significant creep, such as sleech, the equilibrium conditions may not be reached and therefore a constant stress ratio will never be achieved. In the present investigations this aspect was examined in both the kaolin and sleech clays.

Bulging capacity of columns: There is always a possibility that the column may fail on bulging on the application of the 1st loading, even though the present investigations were carried out in one-dimensional loading chamber, where the composite material is usually modelled as a “unit cell”. This is largely due to a significant effective stress variation within the clay bed surrounding the clay, as the excess pore water pressure was only allowed to dissipate from the base. The weakest case, in the present investigation, would be the kaolin clay bed with multiple column arrangements (low undrained shear strength compared to sleech). The bulging capacity of the column can be given by (Hughes and Withers, 1975):

$$\sigma'_{vco} = \frac{(1+sin\phi')}{(1-sin\phi')}\lfloor\sigma'_v + 4c_u\rfloor \quad \text{Equ 11}$$

where f' and c_u are angle of internal friction of gravel and undrained shear strength of clay. At the end of the equalization stage, prior to the 1st loading increment, the average vertical effective stress acting on the clay bed was 12 kPa. Using the geotechnical parameters listed in Table 1, the ultimate bulging capacity of the column calculated to be around 315 kPa. However, the pressures measured at the base of the columns were 177 kPa and 78 kPa respectively for outer and inner columns, confirming that columns did not fail by bulging.

Stress concentration ratio: Based on the unit-cell approach (as a first approximation) the stress ratio between the effective vertical pressures on column and clay can be approximated using the following relationship:

$$n_{\sigma'_v} = \frac{\sigma'_{v(c)}}{\sigma'_{v(NCo)}} = \frac{E_c}{E_s} \quad \text{Equ 12}$$

where E_c and E_s are Young's modulus of gravel and clay respectively. A conservative value of E_c for the column, with a relative density of 0.60, would be around 80 MPa. The Young's modulus for kaolin and sleech are listed in Table 1 based undrained shear strength and plasticity index (Jamiolkowski *et al*., 1991). On the basis of the values quoted, the stress ratios $n_{\sigma'_v}$ can be expected to be about 8.0 and 4.5 respectively for kaolin and sleech at the commencement of loading stages. Figure 9 shows the actual observations made in relation to the stress ratio $n_{\sigma'_v}$for kaolin and sleech and how it varied with strain and stress levels. Note that the observations pertain to the conditions at the base of the clay bed, and that the stress levels in the clay and along the length of the column can vary significantly as the conditions in the clay may be partially drained. It is promising to see that the calculated values of $n_{\sigma'_v}$ are in a reasonable agreement with the experimental observations at the commencement of the first loading, which is indicated by a circular data point in Figure 9 and also in agreement with the values reported by Najjar, 2013.

Both soils beds exhibited high stress ratio at low consolidation pressure (25-50 kPa) and this significantly reduced under higher consolidation pressures. As the excess pore water pressure dissipated, the stress ratio significantly reduced at low loading ranges for kaolin, and generally remained constant for sleech during the primary consolidation phase. However, there is a clear indication in the case of kaolin, that there is a consistent reduction in the stress ratio during the creep phase, that leads to a postulation that high creep rate, as in the case of sleech, may provide adequate support to the column to carry a greater share of the foundation loading. Direct evidence for this is shown in Figure 10 for multiple column arrangement for kaolin and sleech. This figure also suggests that the exterior columns carried more loading than the middle column. The reason for this can be attributed to a possible artefact of the testing configuration, where the boundary conditions for the exterior columns are not exactly homogenous in nature, in particular significant restrain may have been provided by the steel boundary of the testing chamber. Direct data for other testing configuration can be accessed by referring Moorhead (2013).

Both single and multiple columns were installed at a same area replacement ratio. Therefore, for a direct comparison, the stress ratio $n_{\sigma'_v}$ was calculated based on the average pressure on the columns (7 Nos) and that of the clay, and the results are shown in Figure 11. Perhaps surprisingly, the results are generally similar to those observed for a single columns.

CONCLUSION

Granular columns are an environmentally friendly ground improvement option, particularly under moderate loading conditions. There is a large quantity of literature that deals with design of granular columns, however little or no information is available in relation to the effectiveness of granular columns in arresting settlement, particularly the settlement caused by creep.

A comprehensive experimental programme was undertaken in order to examine primary and secondary consolidation on kaolin and sleech clays. The investigations were carried out in Rowe Cells (i.e. one-dimensional loading conditions). Due to the way the load were applied, the modelled testing conditions reflected the use of granular columns under floor slabs. The testing chambers were also instrumented with pressure cells in order to measure the load carrying mechanism of column and the clay and the dissipation of pore water pressure. The key conclusions from the research are:

a. The presence of column(s) has helped to reduce the consolidation time and this was significant in the case of the multiple column arrangement. The predicted values of consolidation time using three-dimensional consolidation theory generally agreed with the measured consolidation time, except in the case of multiple columns where the measured consolidation time was noticeably higher than predicted and a possible reason for this is attributed to intense smearing during the installation of columns.
b. Settlement reduction factors are significant under low-moderate loading, and are more pronounced in the case of sleech. Such an improvement is not apparent under higher loads for both kaolin and sleech. It is difficult draw any conclusion as the benefit of multiple columns, in this regard, as the improvement factors are somewhat lower than those of a single column. The general census from the research is that the settlement reduction factors are generally significant if the clay is in an overconsolidated state.
c. The primary objective this research was to assess if the granular columns can arrest the creep settlement. Based on the observations, it can be postulated that the primary and creep settlement reduction factors are similar, and design protocols can easily be extended when granular columns are to be used as a means of reducing settlement.

d. The stress concentration ratios are generally high in both types of soils investigated here and that reduced significantly with the magnitude of loading range. Interesting observations is that the stress concentration ratio changed marginally during creep phase in sleech, implying that hardening of the soil, due to creep, provided further lateral support to the granular columns.

Notation

$\Delta\varepsilon_\alpha$ creep strain
t time
σ'_v vertical stress
$n_{\sigma'_v}$ the stress ratio
U, U_v, and U_r are degree of consolidation overall, vertical and lateral respectively
h the maximum flow length in the vertical direction
d_e effective diameter
c_v, c_r coefficient of consolidations in vertical radial directions
T_v and T_r time factors in vertical and radial directions
$\left(n_{\sigma'_v}\right)$ the stress concentration factor
f' and c_u are angle of internal friction of gravel and undrained shear strength of clay
E_c and E_s are Young's modulus of gravel and clay respectively

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Table 1 Material Physical Properties

Descriptor	Kaolin	Belfast Sleech
Liquid Limit (%)	71	49
Plastic Limit (%)	32	21
Angle of internal friction (^{o})	22	31
Organic content (%)	0	8
BS EN ISO 17892-12:2018 Plasticity Designation	CH (High Plasticity CLAY)	CI (Intermediate Plasticity CLAY)
Undrained shear strength kPa	15	18
Preconsolidation pressure (approximate value) kPa	25	75
Initial void ratio e	1.45	0.98
Young’s modulus MPa (Clay)	9.7	18.0
Young’s modulus MPa (Gravel)	80	80

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Table 2 Consolidation characteristics. NCo: No column; m: measured; p: predicted

Soil Type	Kaolin			Sleech		
Loading	25-50 kPa	50-100 kPa	100-200 kPa	25-50 kPa	50-100 kPa	100-200 kPa
Compressibility m_v (m^2/MN)	1.10	0.68	0.38	1.73	0.87	0.40
$C_{v(NCo)}$ and $C_{r(NCo)}$ m^2/year	5.7	8.6	9.5	0.46	0.51	0.46
t_{90} (NCo) (hours)	11.0	14.0	8.0	230	130	140
Single column						
Compressibility m_v (m^2/MN)	0.20	0.52	0.29	0.47	0.51	0.33
C_v m^2/year	16.0	38.7	19.0	4.2	2.3	2.9
t_{90}(m) and t_{90}(p) (hours)	4.0/6.00	3.0/4.50	2.0/3.50	25.0/60.0	28/54.0	22/58.0
β (m)	2.8	4.5	2.0	9.2	4.6	6.3
Multiple columns						
Compressibility m_v (m^2/MN)	0.46	0.52	0.29	0.43	0.33	0.31
C_v m^2/year	37.0	92.0	108.0	11.8	6.6	4.9
t_{90}(m) and t_{90}(p) (hours)	1.7/0.30	1.3/0.25	0.7/0.25	9.0/4.5	10.0/4.5	13.0/4.5
β (m)	6.5	10.8	11.4	25.5	13.0	10.7

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Table 3 Summary of settlement and consolidation parameters for kaolin

Parameters	Normally consolidated			Overconsolidated		
	No Col	*1x 100mm ϕ*	*7x 37mm ϕ*	*No Col*	*1x 100mm ϕ*	*7x 37mm ϕ*
	25-50 kPa					
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	3.9	0.7	1.6	0.3	0.08	0.05
n		5.5	2.4		3.75	6
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	0.22	0.14	0.14	0.036	0.02	0.015
n_α		1.6	1.6		1.8	2.4
50-100 kPa						
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	3.3	2.6	2.5	0.7	0.41	0.38
n		1.3	1.3		1.7	1.8
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	0.20	0.20	0.15	0.11	0.12	0.11
n_α		1.0	1.3		0.92	1.0
100-200 kPa						
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	4.3	3.2	3.4	3.6	2.13	2.51
n		1.3	1.3		1.7	1.4
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	0.18	0.18	0.16	0.17	0.14	0.16
n_α		1.0	1.1		1.2	1.1

Table 4 Summary of settlement and consolidation parameters for sleech

Parameters	Normally consolidated			Overconsolidated		
	No Col (NC)	*1x 100mm ϕ*	*7x 37mm ϕ*	*No Col*	*1x 100mm ϕ*	*7x 37mm ϕ*
	25-50 kPa					
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	5.2	1.4	1.3	0.3	0.084	0.052
n		3.7	4		3.5	5.7
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	0.96	0.54	0.45	0.03	0.01	0.014
n_α		1.8	2.1		3	2.1
50-100 kPa						
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	4.85	2.9	1.9	0.5	0.13	0.56
n		1.7	2.6		3.8	0.9
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	1.17	0.78	0.79	0.27	0.1	0.23
n_α		1.5	1.5		2.7	1.2
100-200 kPa						
$\Delta s_{c(nc)} or\ \Delta s_{c(c)}$ (mm)	4.5	3.65	3.5	2.7	0.68	2.18
n		1.2	1.3		2.3	1.2
$c_{\alpha(nc)}\ or\ c_{\alpha(c)}$	0.93	0.60	0.63	0.62	0.27	0.52
n_α		1.5	1.5		1.2	1.1

Figure captions

Figure 1. Schematic of one dimensional consolidation chamber

Figure 2. Equalization of pore water pressure during external loading under undrained conditions

Figure 3. Settlement and pore water pressure versus logarithm of time for kaolin (no column)

Figure 4. Settlement and pore water pressure versus logarithm of time for kaolin with 1x100 mm ϕ column

Figure 5. Settlement and pore water pressure versus logarithm of time for kaolin with 7x 37mm ϕ columns

Figure 6. Settlement and pore water pressure versus logarithm of time for Sleech with no columns

Figure 7. Settlement and pore water pressure versus logarithm of time for sleech with 1x100 mm ϕ column

Figure 8. Settlement and pore water pressure versus logarithm of time for sleech with 7x 37mm ϕ columns

Figure 9. stress ratio for single column configuration

Figure 10. Vertical effective stress on columns and clay (multiple column arrangement)

Figure 11. stress ratio for multiple column configuration

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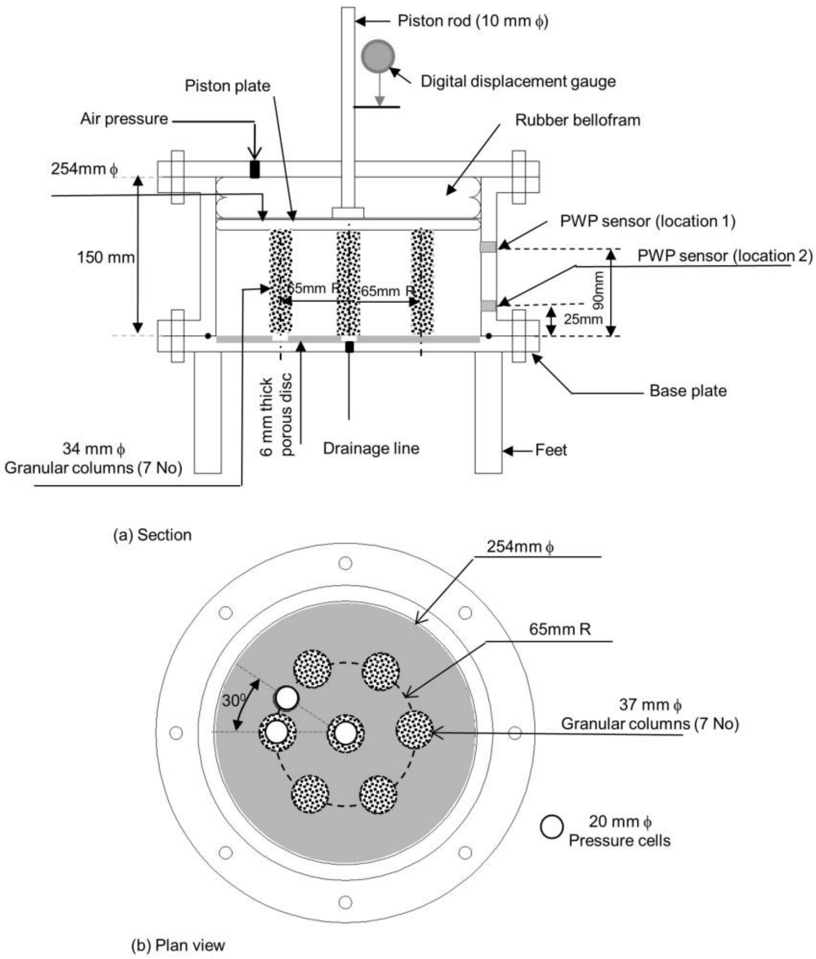

Figure 1 Schematic of one dimensional consolidation chamber

Figure 1

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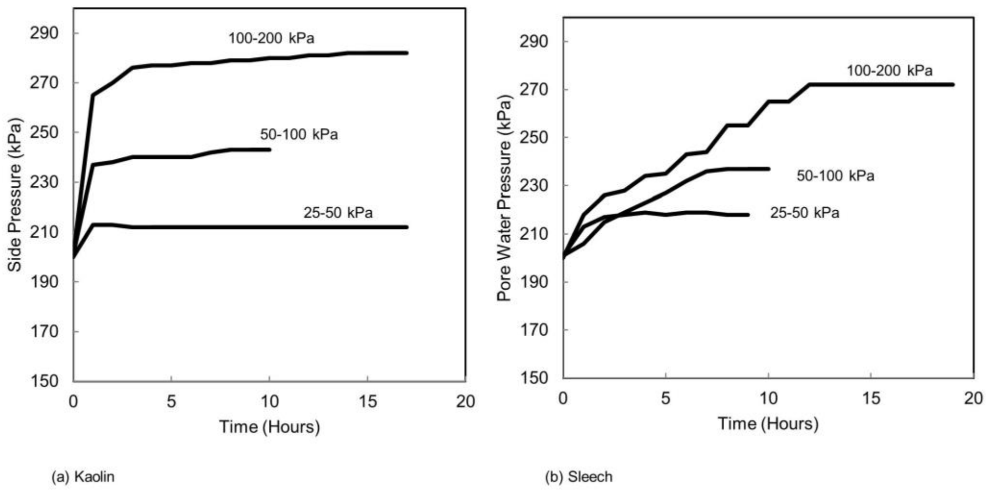

Figure 2 Equalization of pore water pressure during external loading under undrained conditions

Figure 2

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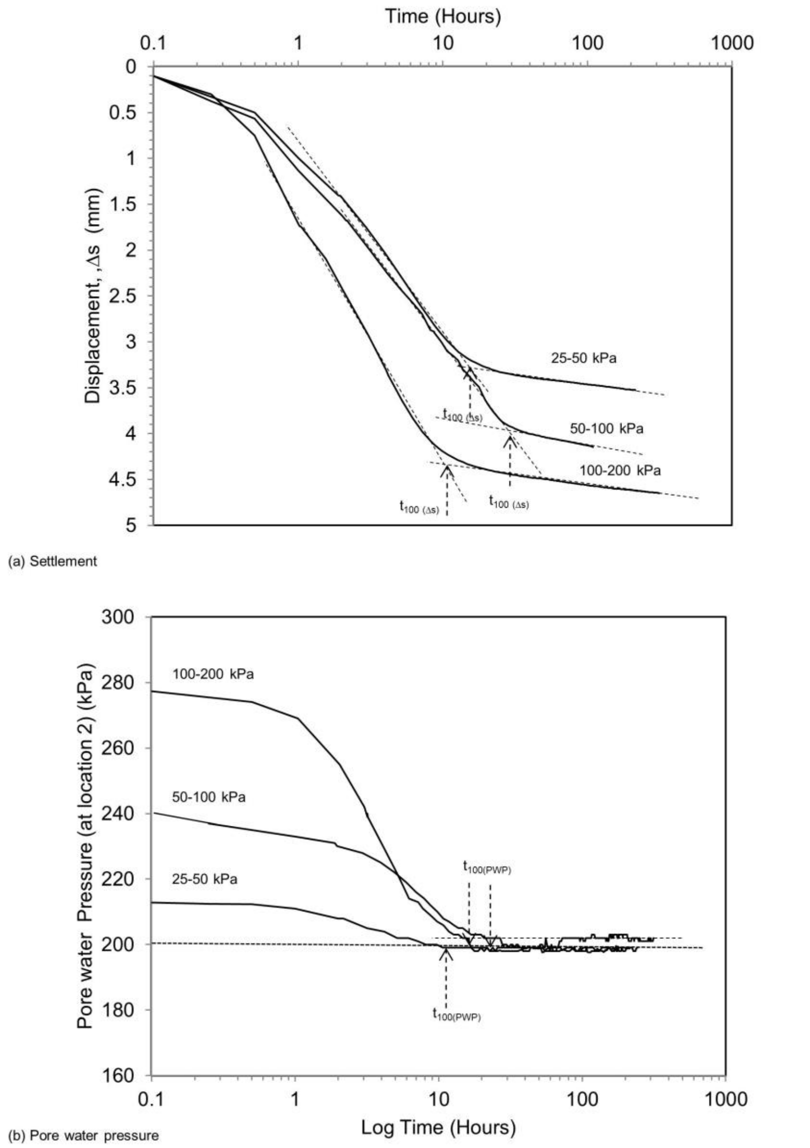

Figure 3. Settlement and pore water pressure versus logarithm of time for kaolin (no column)

Figure 3

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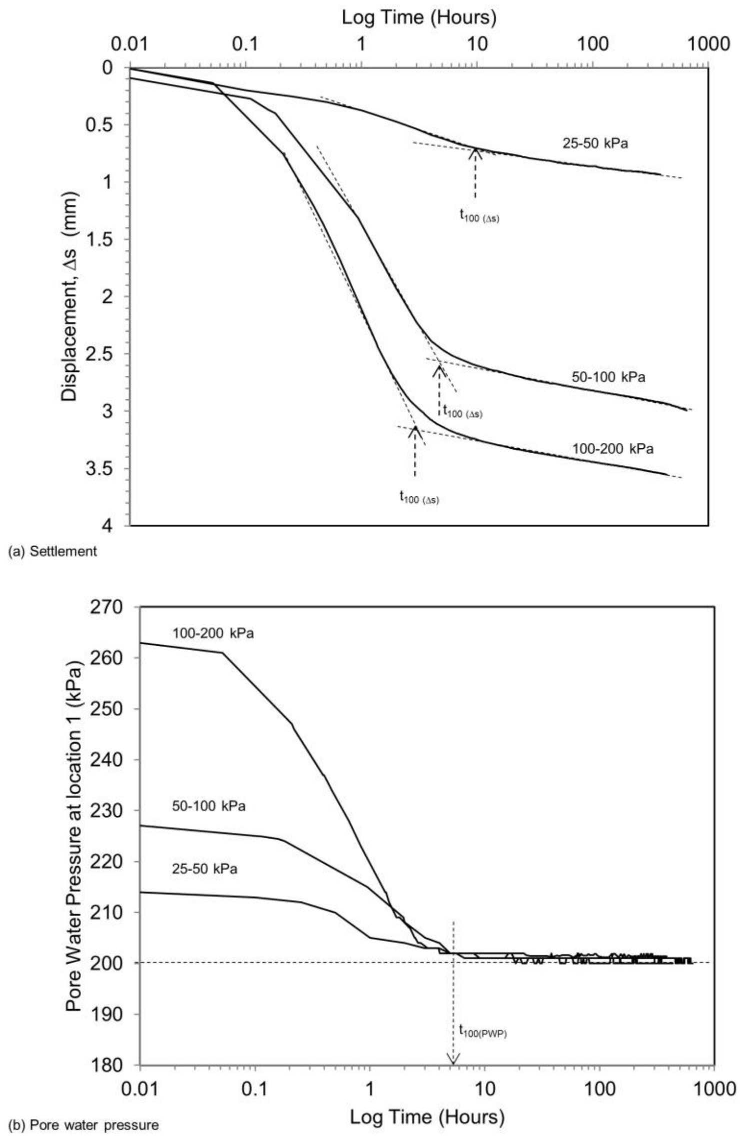

Figure 4. Settlement and pore water pressure versus logarithm of time for kaolin with 1x100 mm ϕ column

Figure 4

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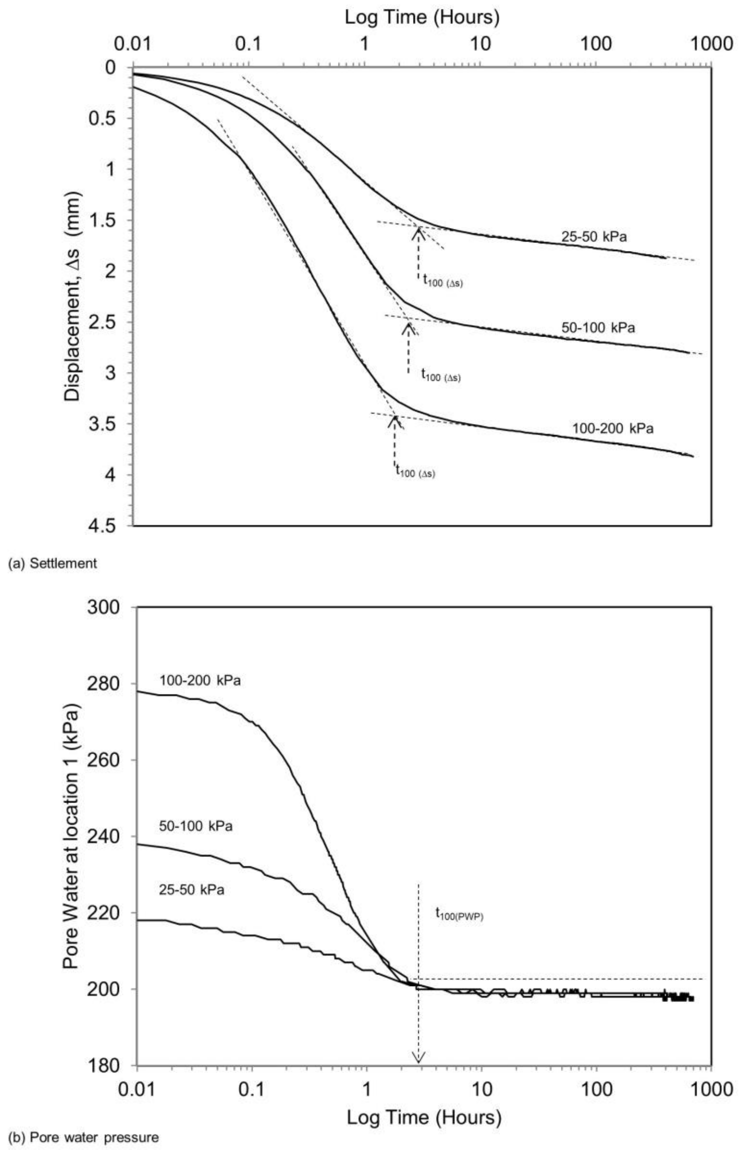

Figure 5 Settlement and pore water pressure versus logarithm of time for kaolin with 7x 37mm ϕ columns

Figure 5

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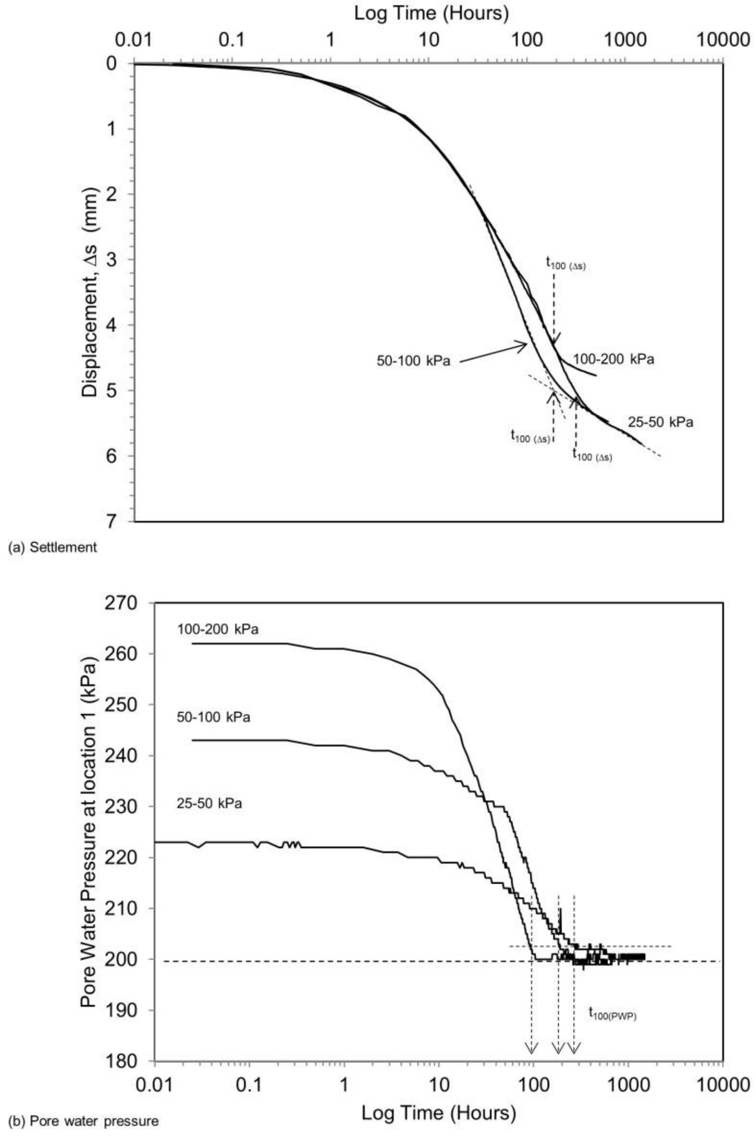

Figure 6 Settlement and pore water pressure versus logarithm of time for Sleech with no columns

Figure 6

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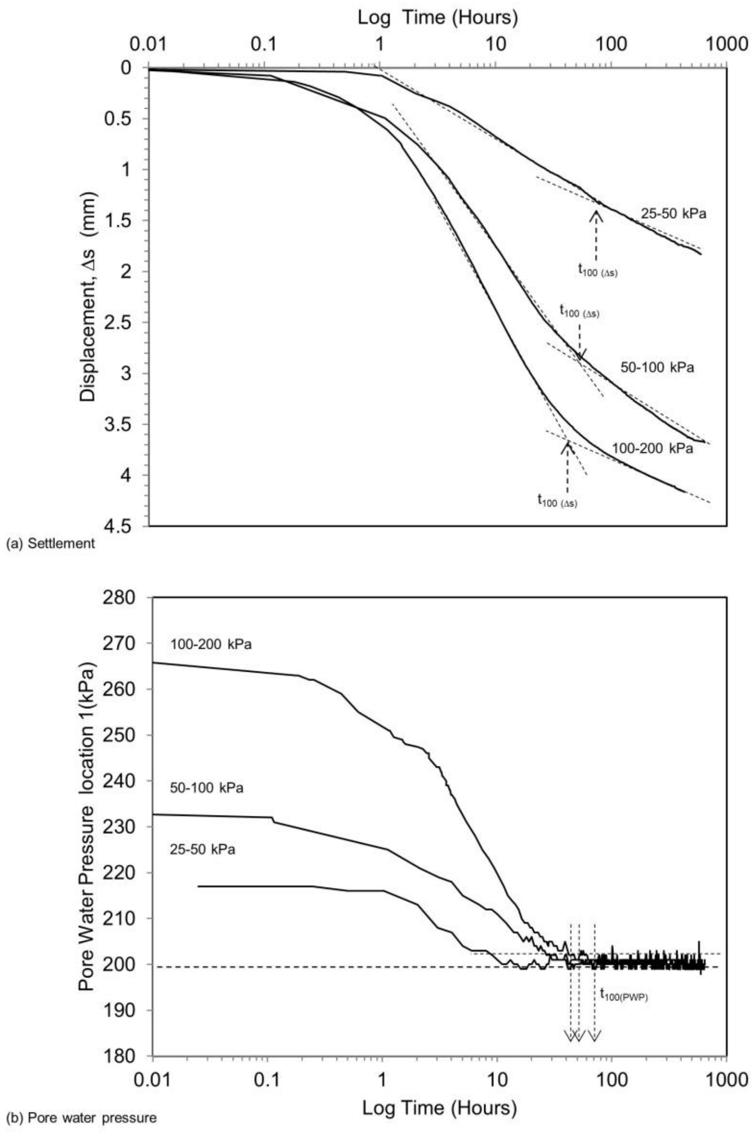

Figure 7 Settlement and pore water pressure versus logarithm of time for sleech with 1x100 mm ϕ column

Figure 7

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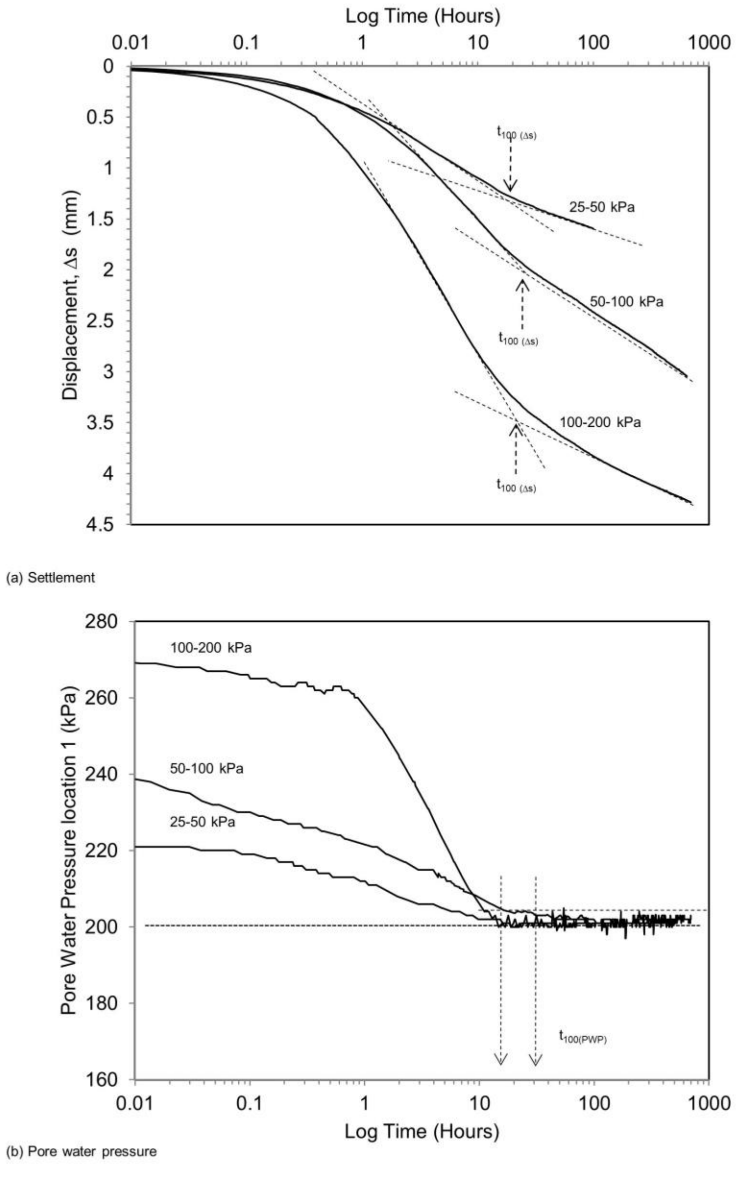

Figure 8 Settlement and pore water pressure versus logarithm of time for sleech with 7x 37mm ϕ columns

Figure 8

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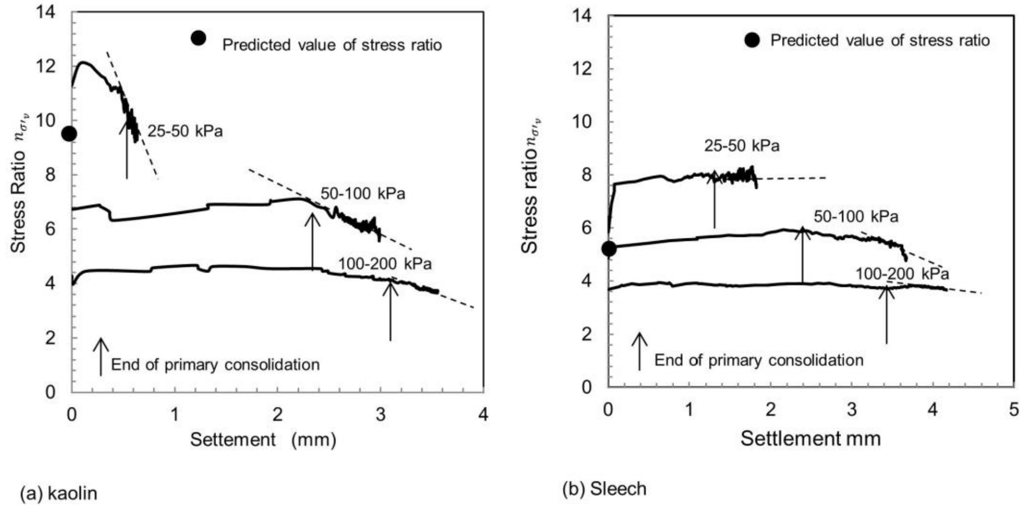

Figure 9 stress ratio for single column configuration

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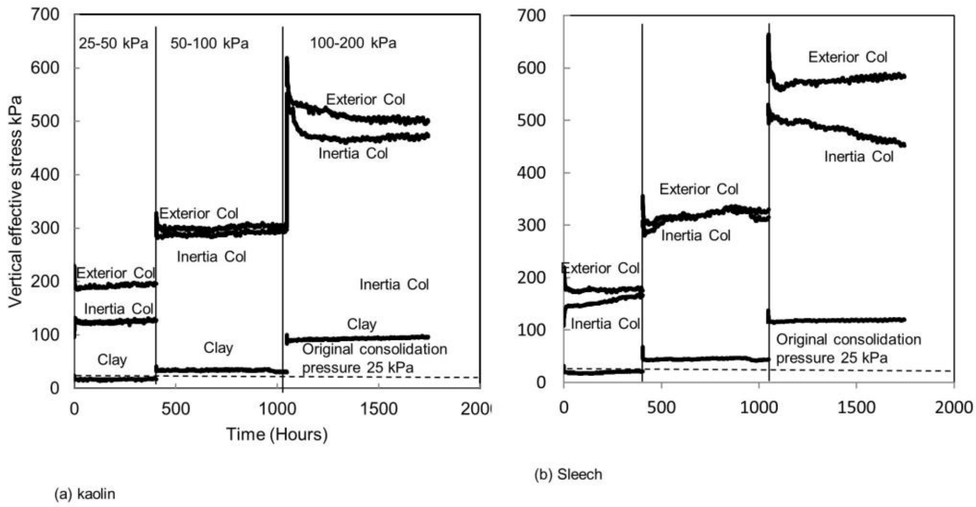

Figure 10. Vertical effective stress on columns and clay (multiple column arrangement)

Figure 10

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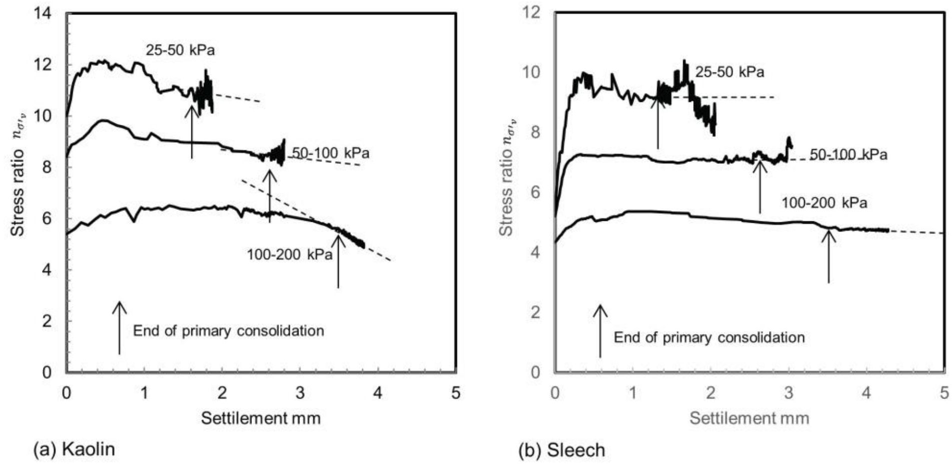

Figure 11 stress ratio for multiple column configuration

Figure 11