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EFFECT OF LATERAL EARTH PRESSURE COEFFICIENT ON PRESSURE CONTROLLED COMPACTION GROUTING IN TRIAXIAL CONDITION

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ABSTRACT

The design, fabrication, and assembly of a new laboratory apparatus for the investigation of the behavior of compaction grouting in triaxial condition are presented in this paper. Using this laboratory apparatus, pressure-controlled compaction grouting tests were carried out in specimens of completely decomposed granite (CDG) in Hong Kong. Precisely controlled injection water into a specially designed latex balloon in the specimen was to simulate a compaction grouting process. In these tests, the effective confining pressure, lateral earth pressure coefficient (K), excess pore water pressure, back pressure, void ratio change, and vertical deformation of the specimen were measured. The main focus was to investigate the development of injection pressure, void ratio, and excess pore water pressure due to compaction grouting and subsequent consolidation of soils. In addition, both the compaction efficiency and the average strength enhancement ratio are defined to evaluate the effect of compaction grouting.

Key words: compact grouting, completely decomposed granite, injection rate, laboratory tests (**IGC**: K14)

INTRODUCTION

The basic concept of compaction grouting is that of injecting an expanding bulb of highly viscous grout with high internal friction into a compressible soil, which could physically displace soil particles and move them into a denser packing, thus achieving controlled densification. The compaction grouting procedure and the basic principle have been described by Graf (1969). During the process of compaction grouting, excess pore water pressure is built up which subsequently dissipates by draining outward. The consolidation process that follows the compaction process is responsible for the decrease in void ratio for the surrounding soil and hence, increases the strength of the material.

Usually compaction grouting is used to generate upward displacement of ground surface to compensate the settlement induced by tunneling or excavation (Au, 2003; Soga et al., 2004; Wang, 2006). However in some cases, the primary objective of compaction grouting is to enhance the shear strength of the soil, while heaving of the ground surface is not necessary or even deleterious to the building founded above. In addition, although the injection volume can lead to expansion of soil (be responsible for the heave of ground), the shear strength enhancement is based on the consolidation of soil, which will result in the soil be contractive (be responsible for the settlement of ground). Therefore, it is important to know whether the balance between displacement of the ground surface and shear strength enhancement induced by compaction grouting.

In the present investigation, a new laboratory apparatus for the investigation of the behavior of compaction grouting are designed. Using this laboratory apparatus, pressure-controlled compaction grouting tests were carried out. The main focus was to investigate the development of injection pressure, void ratio, and excess pore water pressure due to compaction grouting and the subsequent consolidation of soils in triaxial conditions. In addition, both the compaction efficiency and average strength enhancement ratio were defined to evaluate the effect of compaction grouting.

EXPERIMENTAL SETUP

Schematic of Experimental Setup

Figure 1 shows the schematic triaxial apparatus for the compaction grouting tests. The diameter and height of the specimens are 100 mm and 200 mm, respectively. The soil is compacted in the triaxial apparatus to obtain the desired initial relative density. Confining pressure is applied to the specimen surrounded by a rubber membrane. Confining pressure and pore water pressure are measured using transducers 1 and 2, respectively. Water is injected

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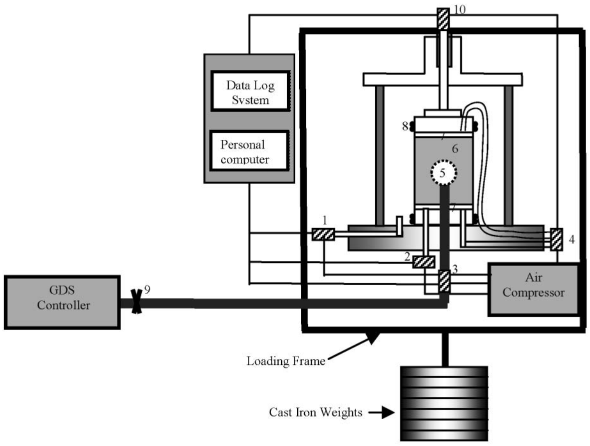

Notes:
1: Confining pressure transducer, 2: Pore water pressure transducer, 3: Injection pressure transducer, 4: Back pressure transducer, 5: Balloon, 6: Soil specimen, 7: Porous stone, 8: O ring, 9: Tap, 10: LVDT

Fig. 1. Schematic layout of compaction grouting experimental tests

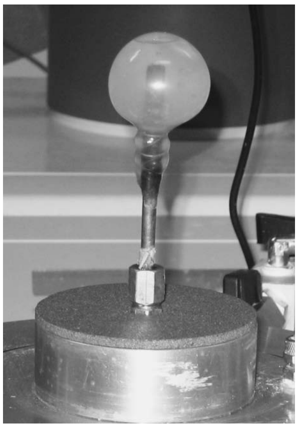

Fig. 2. The expanded needle into a balloon

into the specimen from the bottom of the triaxial cell to achieve full saturation. The injection tube is located in the center of the specimen, and water can enter to expand the membrane/balloon at the end of injection tube. The injection pressure is measured using transducer 3 (Fig. 1).

When the membrane is expanded by the injection water, it will first need to overcome the effect of the confining pressure applied on the soil. Further expansion will compact and densify the soil. Figure 2 shows the expanded needle into a balloon. Detailed design of the injection needle can be found in Au (2001) and Wang (2006).

Furthermore, in Fig. 1, due to increase in pressure, excess pore water pressure will increase and then dissipates with time. The amount of drained water can be measured using transducer 4. During the injection process, the injection pressure (p) and the injection rate (q) are measured. Water injections are carried out using a pressure/volume controller, Geotechnical Digital Systems (GDS), which would control the injection rate and volume. Vertical displacement of the specimen was measured using a Linear Variable Differential Transformer (LVDT). The volume change was measured using a volu-

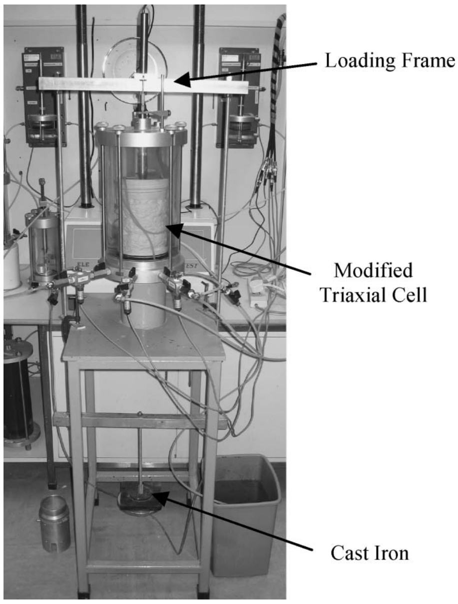

Fig. 3. The modified triaxial cell and the loading frame and cast iron weight to determine the *K* conditions-anisotropic consolidation

metric tank with movable piston. LVDT was used to measure the movement of the piston for calculation of the volume change.

In addition, anisotropic consolidation due to grouting can be carried out by a dead-weight hanger and a dial gauge for measuring axial deformation is clamped to the upper end of the piston as shown in Fig. 3. It is noted that the constant ratio of the horizontal and vertical principal stresses (σ_h/σ_v) is given in terms of total stress. Usually, the lateral earth pressure coefficient (K) is defined in terms of effective stresses. In the present tests, the pore pressure can be controlled at the beginning of the test. The condition of K is created by applying the same confining pressure with different vertical stresses. The dead weight loading can be calculated with reference to Head (1998). Finally, all the data from the transducers were recorded automatically in a data logging system with associated computer software.

It should be noted that although traditionally the grouts are cement or chemical based materials rather than water. Since grout is contained by a grouting bag, the grouts cannot enter in the surrounding soil. Therefore the type of grout is not a significant factor in this case. For simplicity, water was used rather than grouts. Effects of different grouts on compaction grouting will be discussed in the other paper. Nevertheless, the pressure loss between the pressure transducer and the grout injection point depends on the injection rate, travel length of water, diameter of injection needle, etc. Detailed description of the correction for pressure loss along the tubing and injection needle can be referred to Au (2001).

EXPERIMENTAL PROCEDURES

There were totally six stages for the experimental procedures of this compaction grouting tests.

Stage-1: All the porous stones, filter papers and injection needle needed to be de-aired to make sure that there were no air bubbles being trapped.
Stage-2: Soil samples for the compact grouting tests should be prepared in such a way that they were reproducible. The sample should achieve a predetermined dry density and should be homogeneous. In addition, the soil sample was prepared in a split mould in twelve layers using the moist tamping method (Ladd, 1978). Every effort was made to prevent material segregation.
Stage-3: Saturation of the sample was necessary to provide reliable measurements of the volume change and the pore pressure response in drained tests. The degree of saturation was checked by measuring the pore pressure response parameter B (Skempton, 1954). The value of B was found to be 0.98–1.00, indicating that the samples were adequately saturated.
Stage-4: This stage was carried out in order to get the same effective confining pressure before injection stage.
Stage-5: After the consolidation stage of the test, injection of fluid and expansion of the membrane were carried out in the modified triaxial cell. Due to the water injected into the injection needle, the membrane of injection needle began to expand and compressed the surrounding soil, resulting in an increase in excess pore water pressure almost immediately. In order to investigate the compaction effect on the soil, the excess pore water pressure was allowed to dissipate by opening the back pressure valve and let the water in the soil drained.
Stage-6: The second consolidation started as soon as the first injection stage began. The change in volume of the water was recorded by the volume change transducer. After the first injection was finished, subsequent injections started. In order to compare conveniently the effect for different lateral pressure coefficient (K) on the grouting efficiency, the period of the second consolidation stage was set to be the same.

PROPERTIES OF HONG KONG CDG

The soil used in this study, around 2 m^3, which was excavated from a construction site at Beacon Hill, Kowloon Tong, Hong Kong. The fine content was about 8%. The permeability of CDG in Hong Kong was about 1.16×10^{-6} m/s. Table 1 summarized the physical properties of CDG in Hong Kong.

Table 1. Physical properties of Hong Kong CDG

Properties	Value
Natural water content	8%
Gravel	30%
Sand	64%
Fine particles	6%
D_{10}	0.18 mm
D_{30}	0.7 mm
D_{60}	1.8 mm
Coefficient of uniformity (D_{60}/D_{10})	10
Liquid limit	36%
Plastic limit	25%
Plasticity index	11%
Maximum dry density	1820 kg/m^3
Optimum moisture content	11%
e_{max}	1.096
e_{min}	0.493

DEFINITION OF COMPACTION EFFICIENCY

In this present research, it is assumed that the soil was totally saturated, the effect of compaction grouting can be expressed by the mean void ratio change of specimen ($\Delta\bar{e}$). As the maximum void ratio (e_{max}) and the minimum void ratio (e_{min}) can be measured beforehand as shown in Table 1, the compaction efficiency η^*, is defined as followed:

$$\eta^* = \frac{\Delta\bar{e}}{e_{max} - e_{min}}, \tag{1}$$

where $\bar{e}$ = average void ratio, and it is easy to obtain from $\bar{e} = e_0 - \Delta e$, e_0 was the initial void ratio, which can be controlled by the test. Δe is calculated by the volume of drained water in the test. In addition, based on the definition of the compaction grouting efficiency, the average strength enhancement ratio $\bar{\alpha}$ can be derived as followed:

$$\bar{\alpha} = \exp\left(\frac{(e_{max} - e_{min})\cdot\eta^*}{\lambda}\right) \tag{2}$$

where λ is a constant which can be measured by triaxial tests. The detailed description of these two definitions can be referred to Wang (2006).

EXPERIMENTAL RESULTS

Figure 4 shows the normalized void ratio e/e_0 versus time for different lateral earth pressure coefficients (K), with injection period equals to 0.3 minute, injection rate of 30 ml/min, injection volume of 8 ml in each cycle, and consolidation time per injection of 30 minutes. It is clear that void ratio changes in each injection cycle during the whole tests are decreasing gradually due to the increasing soil density for each injection cycle. It indicates that the soil becomes more and more difficult to be densified with increasing injection cycles, despite the injection volume each time is kept the same. The results also show that if the injection volume and injection period are controlled properly, certain injection cycles can achieve the most optimum soil densification.

Based on the definition of compaction efficiency and mean shear strength enhancement ratio in Eqs. (1) and (2), Fig. 5 shows that the compaction efficiency decreases from 0.16 to 0.12 for K values ranging from 0.6 to 1. In addition, Fig. 6 shows that the mean shear strength enhancement ratio increases from 1.9 to 2.6, when K decreases from 1 to 0.6.

Figure 7 shows injection pressures versus time with different K values. When K equals to 1, the peak injection pressure is about 400 kPa, however, when K equals to 0.6, the peak injection pressure increases to 540 kPa. In other words, higher injection pressure is necessary for

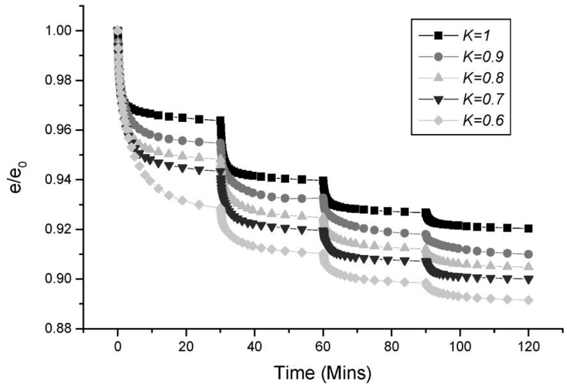

Fig. 4. **Normalized void ratio e/e_0 versus time for different lateral earth pressure coefficient (K) (Time is the whole time from the first injection to the end of the fourth consolidation)**

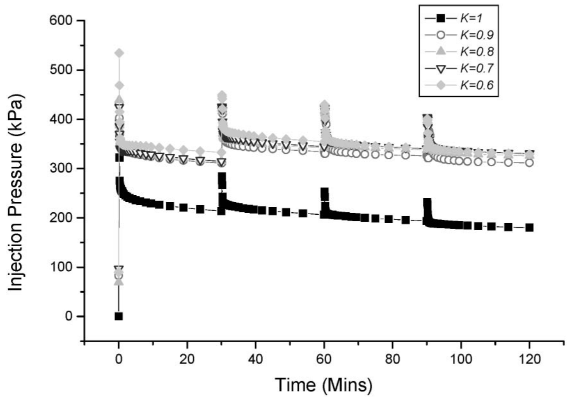

Fig. 7. **Injection pressure versus time for different lateral earth pressure coefficient (K)**

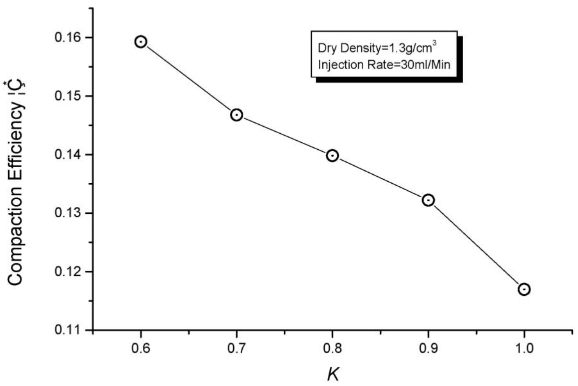

Fig. 5. **Compaction efficiency versus lateral earth pressure coefficient (K)**

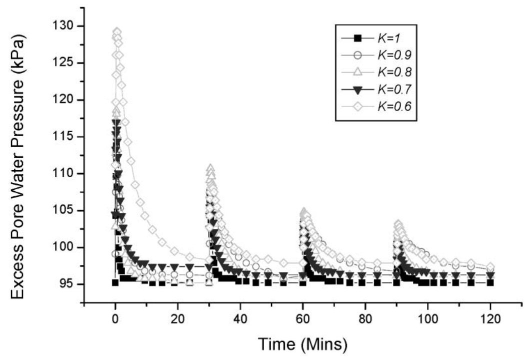

Fig. 8. **Excess pore water pressures versus time for different lateral earth pressure coefficient (K)**

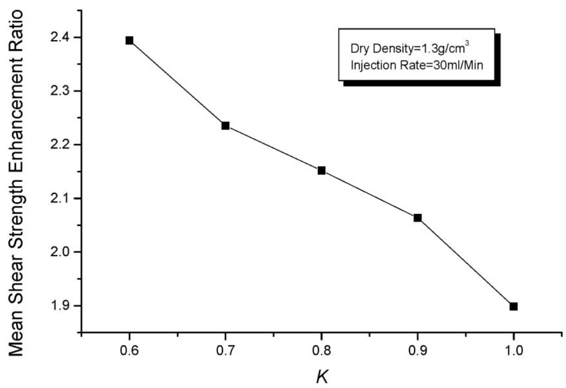

Fig. 6. **Mean shear strength enhancement ratio versus lateral earth pressure coefficient (K)**

decreasing K values. This can be explained that higher injection pressure is required to expand the membrane due to the increase in surrounding pressure with decreasing K values. It is interesting to note that injection pressures in the specimen decreases and becomes steady very soon after completion of each injection. However, the injection pressure is maintained at relatively high levels, which is crucial to sustain compression of soil and expedite the excess pore water dissipated.

Figure 8 shows the dissipation of excess pore water pressures with time for different K values. The shapes of the excess pore water pressure plots are similar to the injection pressure plots shown in Fig. 7. This can be explained since higher injection pressure induces higher excess pore water pressure in the surrounding soil. While the excess pore pressure dissipation will in turn influence the injection pressure until the injection pressure inside the membrane reaches equilibrium with the effective stress in the surrounding soil. However, excess pore water pressure responses are affected by two factors. One is the internal grouting pressure and the other one is the confining pressure. The injection pressure and pore water pressure can also influence each other. From Figs. 7 and 8, both the peak injection pressure and peak excess pore water pressure decreases gradually with the injection cycles. It indicates that the volume of water drained from the specimens becomes smaller and smaller. This phenomenon also explains the reason for the void ratio changes in Fig. 4, the compaction efficiency in Fig. 5 and the mean shear strength enhancement ratio in Fig. 6 are all gradually decreasing with increasing K values.

Figure 9 shows the vertical displacement of the sample versus time for different K values. The vertical deformation is positive, heaving, at the beginning of the injection

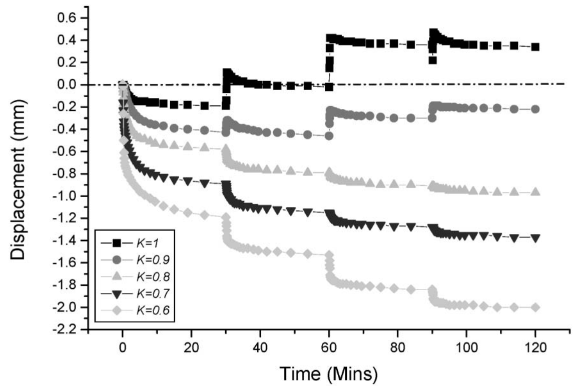

Fig. 9. Vertical displacement of sample versus time for different lateral earth pressure coefficient (*K*)

cycle, and then became negative, settling, with dissipation of excess pore water pressure for K values of 0.9 and 1.0. When K changes to 0.6, 0.7 and 0.8, the settlement process dominates. When K is equal to 0.9, the settlement and heaving process are about the same. The amount of vertical displacement depends on the time allowed for consolidation after injection and the number of injection as shown in Fig. 9. This is also dependent on the value of K. Therefore by controlling the consolidation time and the number of injections, the desired settlement/heave can be obtained. It is noted that, if the coefficient of earth pressure is not known for a site, trial compaction grouting test should be carried out with measurement of surface and sub-surface deformation. Small settlement or heave suggests higher values of K. The amount of settlement can be varied by controlling the time interval between injections and the number of times of injection.

CONCLUSIONS

Laboratory scaled pressure-controlled compaction grouting tests were designed and set up, including the detailed experimental procedures. In addition, the properties of soil (Hong Kong CDG) used in these tests were measured. Using this apparatus of compaction grouting, the effective confining pressure, lateral earth pressure coefficient (K), excess pore water pressure, back pressure, void ratio change, and vertical deformation of the specimen could be measured and controlled, in order to evaluate the compaction efficiency.

Experimental results show that both the compaction efficiency and the mean shear strength enhancement ratio decreased when the lateral earth pressure coefficient (K) increases from 0.6 to 1.4. In addition, higher injection pressure induces higher excess pore water pressure in the surrounding soil, and higher excess pore water pressure dissipation takes longer time to reach steady state. Meanwhile, the excess pore pressure dissipation would in turn influence the injection pressure until the injection pressure inside the membrane reached equilibrium with the effective stress in the surrounding soil. Besides, experimental results demonstrate that there is a balance between shear strength enhancements and settlement or heave that could be obtained by controlling the injection volume and injection times.

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