Construction and Building Materials 23 (2009) 1249–1264

Contents lists available at ScienceDirect

Construction and Building Materials

journal homepage: www.elsevier.com/locate/conbuildmat

Experimental study of the effect of fines content on dynamic compaction grouting in completely decomposed granite of Hong Kong

Shanyong Wang [a,*], Dave Chan [b,1], Ka Chi Lam [c,2]

[a] Centre for Geotechnical and Materials Modelling, University of Newcastle, University Drive, Callaghan, NSW 2308, Australia
[b] Department of Civil and Environmental Engineering, University of Alberta, Canada T6G 2W2
[c] Department of Building and Construction, City University of Hong Kong, Tat Chee Avenue, Hong Kong

ARTICLE INFO

Article history:
Received 1 May 2008
Received in revised form 4 August 2008
Accepted 4 August 2008
Available online 13 September 2008

Keywords:
Dynamic compact grouting
Fines content
Shear strength
Optimum water content
Soil improvement

ABSTRACT

This paper addresses the effects of fines contents on the mechanical behavior of CDG (completely decomposed granite) during dynamic compaction grouting. The first set of tests involved adding different amounts of kaolin clay into CDG to study the effects of fines contents on the compaction characteristics, permeability, consolidation behavior and shear strength of soils. It is found that, as the fines content increases, the dry density decreases, however the optimum water content increases. In addition, the permeability decreases due to reduction of void ratio, and the coefficient of consolidation increases with higher fines content. Furthermore, with increasing fines contents, the shear stress tends to be constant after it reaches the peak strength. However, when reaching the peak value, the shear stress reduces with further adding of fines content. In the second set of tests, the dynamic compaction grouting tests are carried out to study the effect of fines content on the compaction efficiency of Hong Kong CDG. It is found that, the compaction efficiency increases with increasing fines content, and reaches the peak compaction efficiency when the fines content reaches 6%, and then the compaction efficiency drops abruptly from a peak value of 0.55–0.25 when the fines content increases from 6% to 8%. However, when the fines content increases from 8% to 41%, the compaction efficiency decreases very slowly.

1. Introduction

Completely decomposed granite soil has been widely used as the subgrade material in Hong Kong. The mechanical behavior of the soil changes depending on its fines content. Due to particle crushing during loading, unloading and reloading, the soil could behave as either cohesive or cohesionless. In addition, cohesionless soils with high fine content are found to be highly susceptible to liquefaction due to their low permeability. It has long been recognized that fine content (passing through the No. 200 sieve or its diameter smaller than 0.075 mm) could affect the dynamic response of soils significantly. During dynamic compact loading, dynamic forces disrupt the soil skeleton and force the particles to compact into a denser arrangement. Obviously, the fines contents play an important role on the mechanical response of soils, especially when the soils are subjected loading.

In general, by increasing clay content, the plasticity and the coefficient of secondary consolidation (creep) increase, while the friction angle and permeability decrease [1]. Chang et al. [2] suggested that the liquefaction resistance increases with the increase in fine content. Nagaraj et al. [3] investigated the effect of plastic fines on the permeability of soil by mixed sand with medium plastic kaolin clay. Thevanayagam et al. studied the effect of fines content and confining stress on undrained shear strength of silty sands. The experimental data indicated that the relative density at which this transition occurs, increased with fines content. For silty sand with fines content in the range of 20–30%, this relative density for transition occurring could be as high as 80–90% [4]. Yin [5] investigated the stress–strain characteristics of a marine soil with clay contents of up to 27.5% and concluded that for the same confining stress, by decreasing the clay content, the deviator stress increases and the effective stress path shifts to the right.

In addition, Chien and Oh [6] used the resonant column tests to study the influence of fines content and initial shear stress on dynamic properties of hydraulic reclaimed soil. The experimental results showed that the maximum shear modulus decreases as the fines content increases. Furthermore, Chien et al. [7] investigated the effects of fines content on liquefaction strength and dynamic settlement of reclaimed soil. They concluded that the liquefaction strength of reclaimed soil decreased as the fines content increased under constant relative density. Shapiro and Yamamuro [8] also studied the effects of silt on three-dimensional stress–strain

* Corresponding author. Tel./fax: +61 (0)2 49216991.
E-mail addresses: bcwangsy@126.com (S. Wang), dave.chan@ualberta.ca (D. Chan), bckclam@cityu.edu.hk (K.C. Lam).
[1] Tel.: +1 780 492 4725; fax: +1 780 492 0249/8198.
[2] Tel./fax: +852 27887238.

0950-0618/$ - see front matter Published by Elsevier Ltd.
doi:10.1016/j.conbuildmat.2008.08.002

behavior of loose sand. Naeini and Baziar [9] studied the effect of fines content on steady-state strength of sand by a series of undrained monotonic and dynamic triaxial compression tests. It was found that, as the silt content increased up to 35%, the steady-state line moved downward and the shear strength decreased.

Up to this point, however, there were very few studies on the relationship between fines content and on the soil improvement by dynamic compaction grouting. Therefore, there is a need to investigate the effects of fines in the soil behavior due to dynamic compaction grouting. In this paper, the results from an experimental study are presented to address the roles of fines content on the dynamic drained behavior of CDG (completely decomposed granite) of Hong Kong. The first set of tests involved adding different amounts of kaolin to CDG to study the effect of fines content on the permeability, consolidation and shear strength of the soil. In the second set of tests, dynamic compaction grouting tests are carried out to study the fines content effect on the compaction grouting. The experimental study includes 45 dynamic compaction grouting tests on samples with fines content from 1% to 41%. The experimental results are examined in terms of the effect on void ratio, relative density, and shear strength.

2. Laboratory experimental studies on the effect of fines contents on the mechanical behavior of soils

In this section, in order to evaluate the effects of fine aggregate content on the mechanical properties of the compacted decomposed granite soils, the Standard Proctor test, falling-head test, consolidation test, direct shear test and triaxial shear test as well as some fundamental property tests are carried out by using samples with the different fines contents. The fines content is controlled by mixing CDG with different percent of kaolin clay. The basic properties of CDG are listed in Table 1. The liquid limit and plastic limit of kaolin are 38% and 19%, respectively.

2.1. Relationship between of fines content and compaction of soil

The Standard Proctor test is adopted to study the effect of fines content on the compressibility of the soil. Fig. 1 shows the compaction curves for soil samples with different fines contents of 1%, 4%, 8%, 11%, 13%, 16%, 21%, and 26% by weight. It is found that as the fines content increases, the dry density decreases. However, the optimum water content increases with increase in fines content which is illustrated in Fig. 2.

A possible explanation for the increase in the optimum water content is due increase in the void ratio in the sample due to higher fines content in the "dry of optimum" stage. Examining the microstructure of soil, the fine grained particles move randomly in pores space and can intrude into the inter-particles space of larger particles and separate the soil particles apart [9]. Furthermore, referring to cluster hypothesis by Barden and Sides [10], the clay particles would form cluster with soil particles and water. These two effects separate the soil particles, formed intra-aggregate, and inter aggregate pores within the soil sample. As a result, the dry density will decrease. In addition, the absorption of water reduces the viscosity of the pore water. Thus, it increases the lubricating effect for rearrangement of soil particles. Accordingly, as the percentage of clay in the sample increases, more clay particles exist in the void of the soil. It tends to absorb water and more water must be added to reach the optimum water content.

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%

2.2. Relationship between of fines content and permeability

Falling-Head tests were carried out to determine the permeability of soils with different fines content. It is shown in Fig. 3 that the permeability declines with increase in the clay content. It is suggested that this relationship is highly non-linear which shows an exponential relationship. Fig. 4 shows the plot of fines content against $\log K$ as a linear relationship. The permeability decreased gradually with fines content increased could be explained by the fact that clay particles are being suspended in the pores water and they moved along the flow path. This results in an increase in the viscosity of the pores water thus reducing the flow rate of the fluid.

2.3. Relationship between fines content and compression characteristics

Fig. 5 presents the relationship of the coefficient of consolidation (C_v) and different fines content. The coefficient of consolidation shows an increasing trend from 1% to 8%, and reaches a peak value at 8% and then begins to decrease gradually until a fines content of 26%. Owing to the vertical pressure in the consolidation process, the soil particles re-arranged to form a new packing. This leads to a reducing of void space between the particles, consequently, the total volume of specimen is decreased. As one-dimensional consolidation test is adopted, the reduction of volume is reflected by a decrease in the sample depth. Microscopically, the particle size of the clay is much smaller than other soil particles. Moreover, in the fully saturated state, the clay particles could flow randomly in the pores of the soil. Hence, the clay particles might fill up the void of the soil, thus increasing the bulk density of soil and allowing the soil sample to have a higher degree of volume reduction.

2.4. Relationship between of direct shear strength and fines content

Fig. 6 shows the corresponding internal friction angle (φ) of different fines contents in the soil. It is clear that at 1% fine content, the internal friction angel is high, about 43.2°. The value of φ drops to 41.3° at a fines content of 4%, and then progressive rise to the 45.6° at the fines content of 8%. It is noted that the 8% has the largest friction angle among all other fines contents tested here. After reaching the peak value at 8%, the friction angle begins to decrease and drops to 39.1° at fines content of 13% and then becomes almost the same until a fines content of 26%. Fig. 7a–c shows the differences of measured particle size distribution before and after direct shear tests for the samples with the fines contents of 4%, 8% and 13%, respectively. The result shows that the variation of the particles distribution after shearing is increased. This is due to soil particles crushing during shearing which contributes to producing finer particles. Moreover, the change of particles distribution is smaller with the fines content increased from 4%, 8% to 13%. This is because the lower fine content of soil, the more and bigger particles can be crushed to smaller particles. As a result, the change of particle distribution is more distinct.

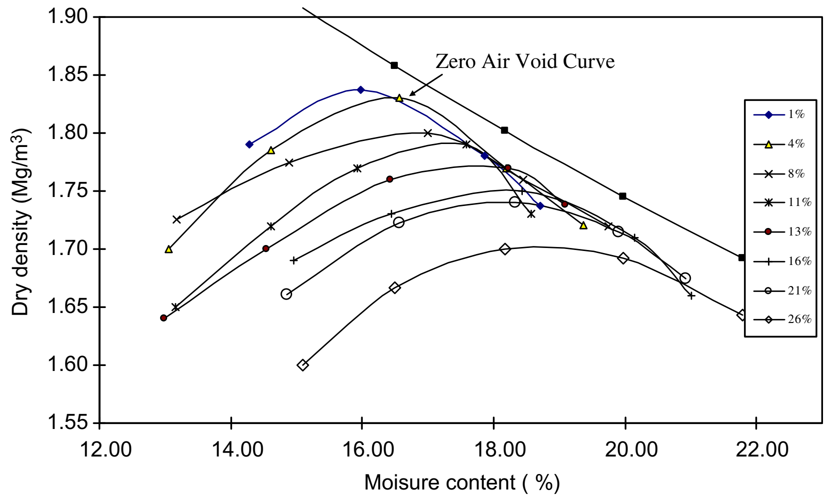


Fig. 1. Relationship between fines content and dry density.

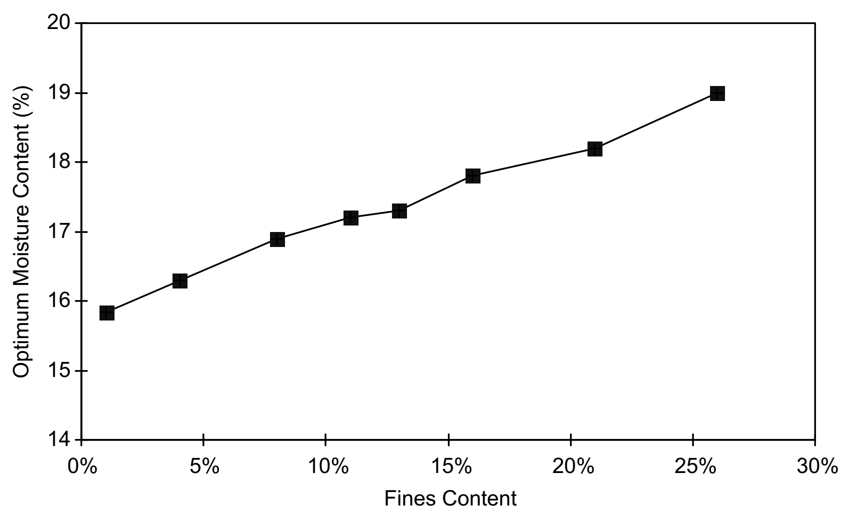


Fig. 2. Relationship of fine content and optimum moisture content of compaction.

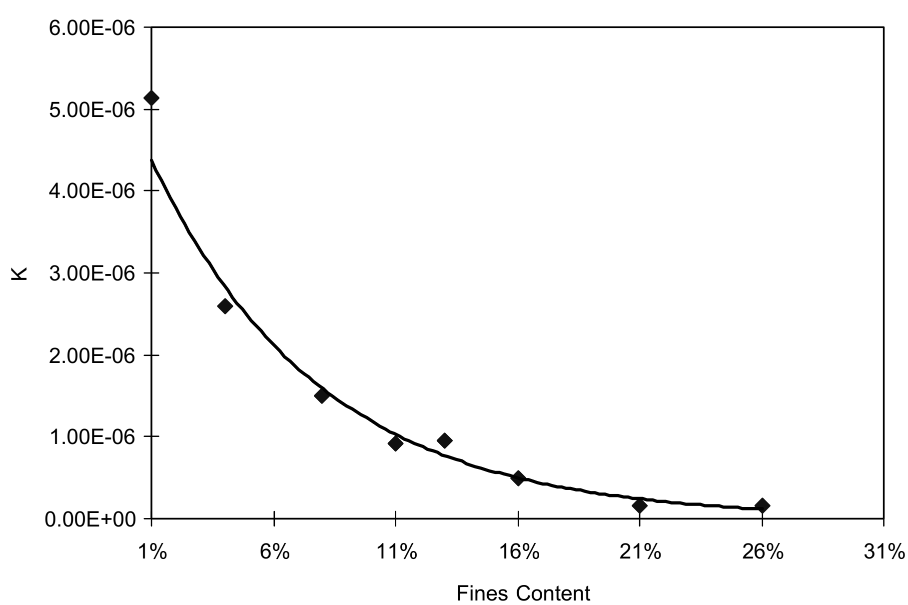


Fig. 3. Relationship between fine content and coefficient of permeability (K).

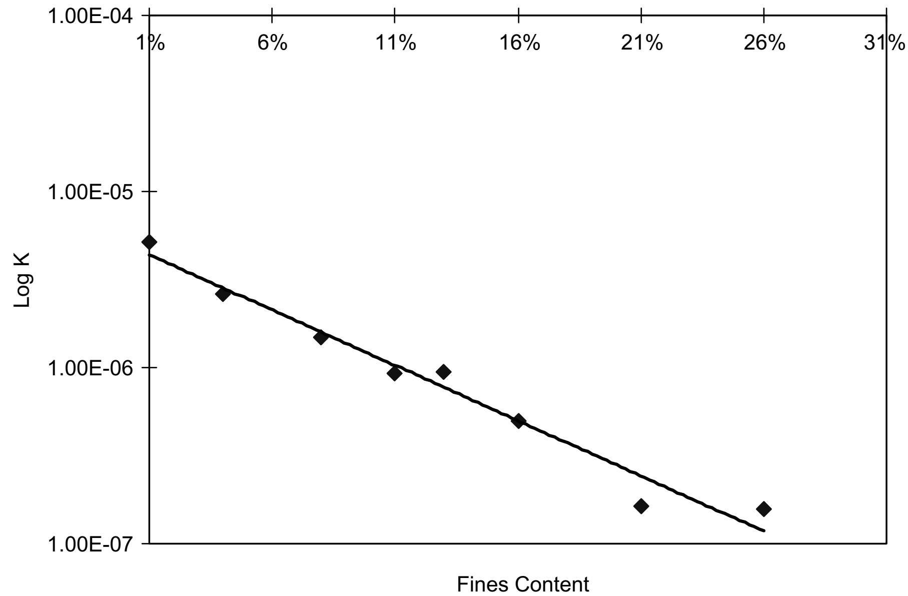


Fig. 4. Relationship between fine content and log$K$.

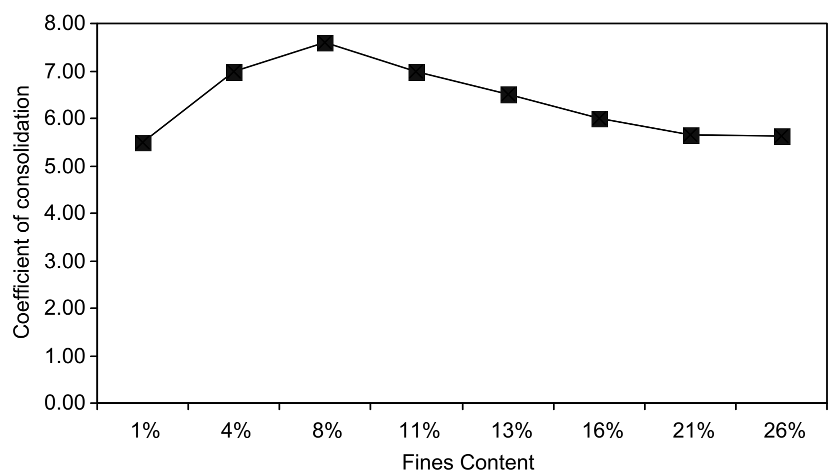


Fig. 5. Relationship between fine content and coefficient of consolidation (C_v).

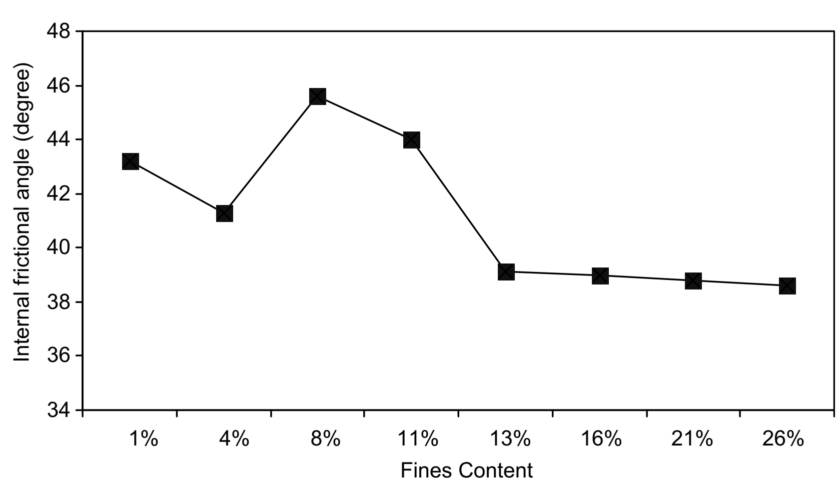


Fig. 6. Relationship between of direct shear strength and fines content.

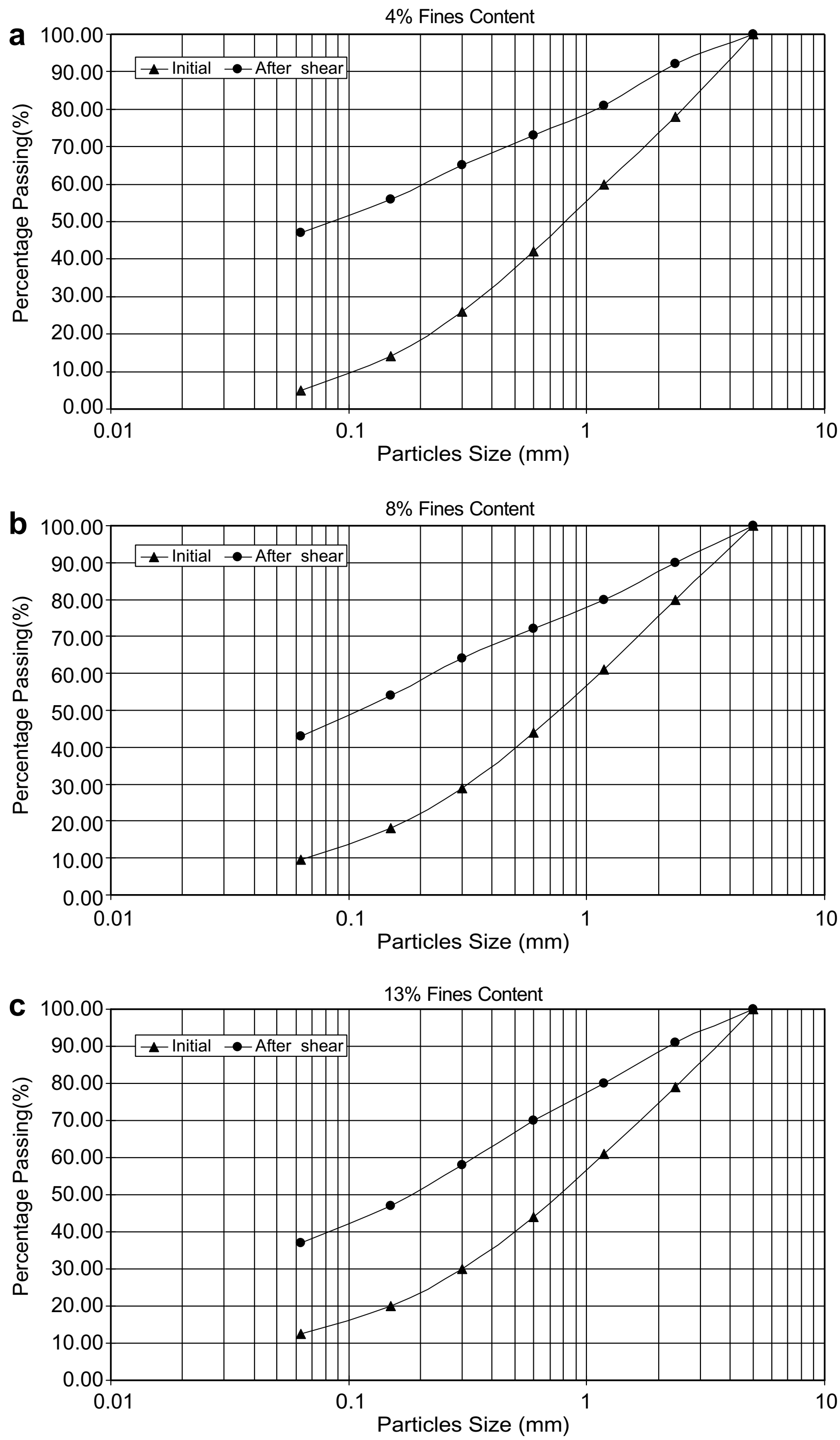


Fig. 7. (a) Particles distribution for 4% fine content, (b) particles distribution for 8% fine content and (c) particles distribution for 13% fine content.

2.5. Relationship between of shear strength and fines content

For the triaxial tests, a total of eleven triaxial CD (consolidation drained) tests were carried out with fines contents ranging from 1% to 41%. Fig. 8 shows the particle size distribution of these soils. Fig. 9a presents the stress–strain curves of the specimens, Fig. 9b shows the volumetric strain versus axial strain and Fig. 9c shows the effective principal stress ratio (σ_1'/σ_3') versus axial strain for the specimens. The results indicates that the peak shear strength decreases with increase in fines content up to 6%, whereas this trend is reversed when fines content reaches 36%. However when the fines content exceeds 36%, the peak shear strength increases again compared with the soil with fines content of 36%. One possible explanation for this behavior is that at low amounts of fines, most of the fines are occupying the voids between the sand grains. Since the applied loads are carried by the sand skeleton, these fines have little effect on the behavior of the mixtures, as they are just residing in largely empty void spaces. By increasing the fines content, some of the finer particles may also occupy locations near the contact points between the sand grains which separate the sand

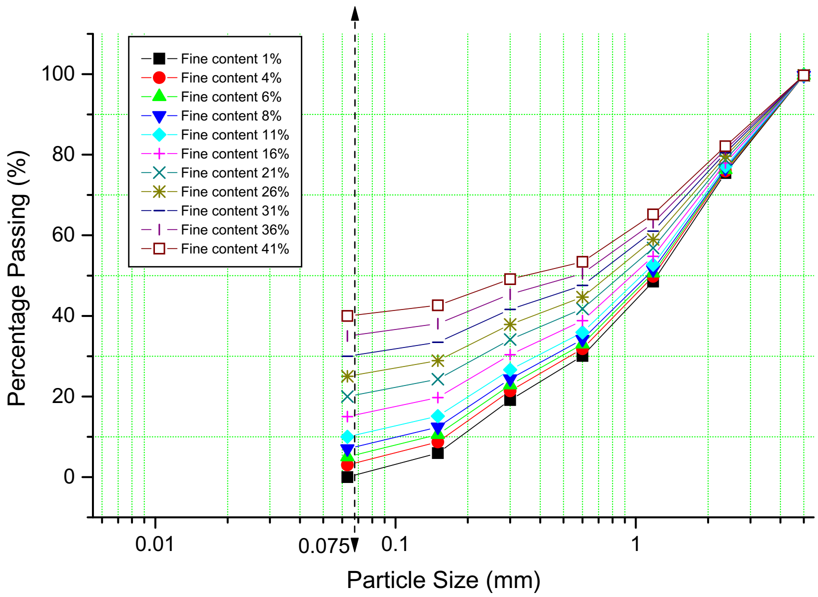


Fig. 8. Grain size distribution curves for different fine content.

grains slightly apart. While the fines located in the voids do not significantly affect the overall soil behaviors, the effects of fine particles near the contact points are much more significant. These fines can break the load bearing chain in the specimen which will lead to decrease of the peak shear strength. There is a fines content at which the contacts between the sand grains are almost vanished (36% in this case) and the mixture becomes clay with sand inclusions. It is noted that the threshold fines content depends on the characteristics of the base sand and the fine particles as discussed by Seed et al. [11].

3. Dynamic compaction grouting tests

The basic principle of dynamic compaction grouting is that vibrations are introduced to the grouting bag during the expansion/compaction process [12]. The vibration of the membrane of the grouting bag generates waves in the soil that move the surrounding soil particles to a denser state. This is assisted by the pressure that is generated due to the expansion of the grouting bag. That expansion fills the spaces which are lost due to the decrease in volume of the soil as a result of the densification process.

3.1. Description of experiments

This experimental study is aimed at investigating the effects of fines content on the compaction efficiency for dynamic compaction grouting tests. The most distinguished feature of this laboratory apparatus is that it could simulate the triaxial condition of dynamic compact grouting in the field. In the tests, the effective confining pressure, the lateral pressure coefficiency, excess pore water pressure, back pressure, void ratio change, vertical and lateral deformation of the specimen were measured. In addition, the dynamic compact grouting pressure, dynamic compact grouting frequency, and dynamic compact grouting period were also controlled.

Fig. 10 shows the triaxial apparatus for the injection tests. The diameter and height of the specimens are 100 mm and 200 mm, respectively. The soil was compacted in the triaxial apparatus to obtain the desired initial relative density. Confining pressure was applied to the specimen that was surrounded by a rubber membrane. Confining pressure and pore water pressure were measured using transducers 1 and 2, respectively. Water was injected into the specimen from the bottom of the triaxial cell to achieve full saturation of specimens. The injection tube is located in the center of the specimen, and injection water can enter to expand the membrane/balloon at the end of injection tube. The injection pressure was measured using transducer 3 .When the balloon was expanded by the injection water, it first needed to overcome the effect of the confining pressure applied on the soil. Further expansion compacted and densified the soil. Impulsive waves were produced by an impulse wave generator. The wave led to the vibration of the membrane, which transferred the waves to the soil moving the surrounding soil particles. It should be noted that although traditionally the grouts are cement or chemical based materials rather than water, as there is a grouting bag that the grouts will be introduced inside. The grouts can not enter in the surrounding soil. The current research is to study how the inside vibrations densify the surrounding soils. For simplifying, the water was used rather than the other grouts. For the effect of different grouts on the dynamic compaction grouting will be discussed in the other paper.

Due to increase in pressure and wave propagation, excess pore water pressure increased and then dissipated with time. The amount of drained water was measured by transducer 4. During the injection process, the injection pressure (p) and the injection rate (q) were measured. Water injections were carried out using a pressure/volume controller, GDS (geotechnical digital systems), which controlled the injection rate and volume. Vertical displacement of the specimen was measured using an LVDT. In addition, the condition for different lateral compressive coefficient (K) was obtained using the dead weight loading mechanism. All the data from the transducers were recorded automatically in a data log system with associated computer software. Soil samples for the compact grouting tests should be prepared in such a way that they are reproducible. The sample should achieve a predetermined dry density and should be homogeneous. More details about the sample preparation could be found in Wang' PhD thesis [12].

3.2. Experimental procedure

The testing stage is separated into five steps. The first step is the saturation of the sample, which are necessary to provide reliable measurements of the volume change in the drained tests. By pass-

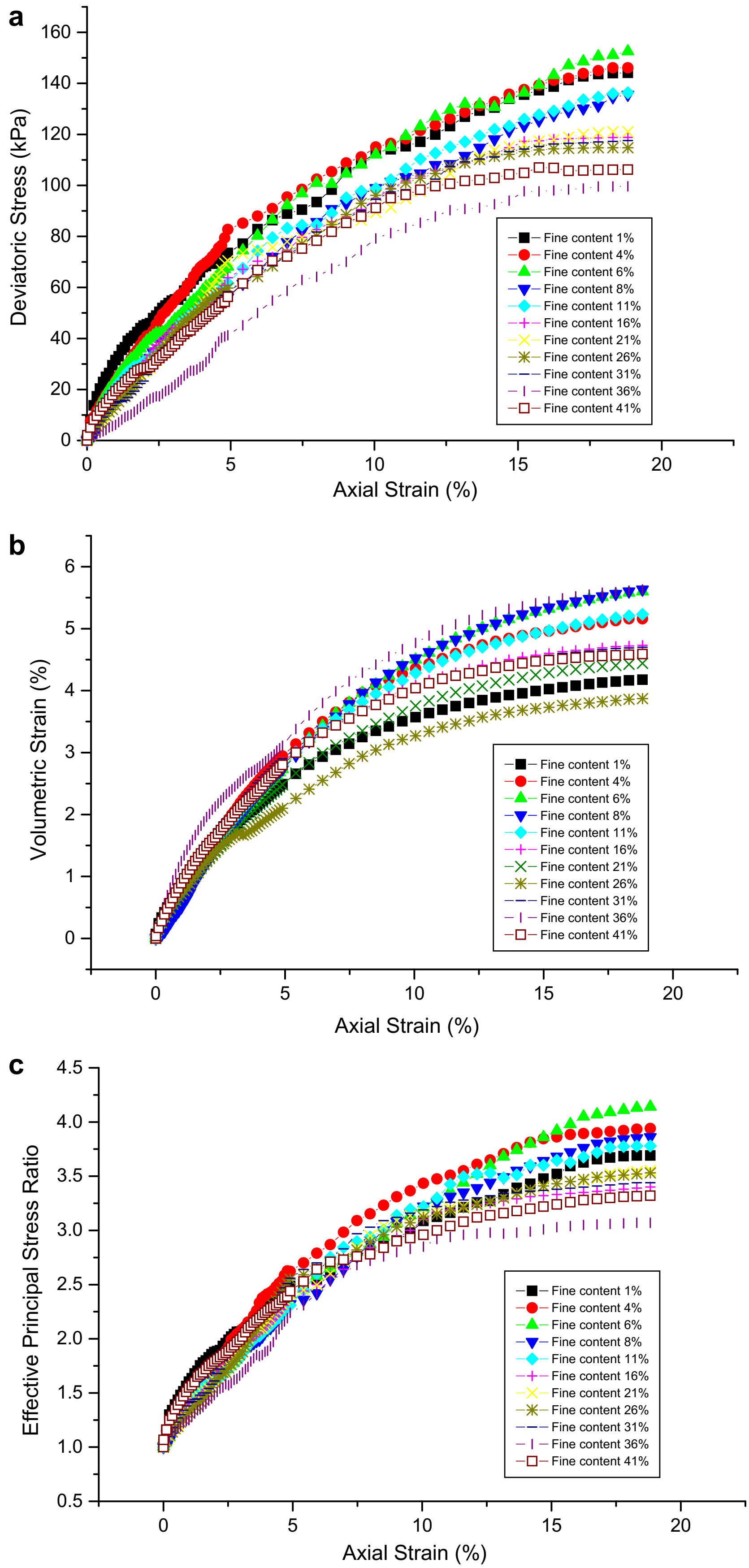
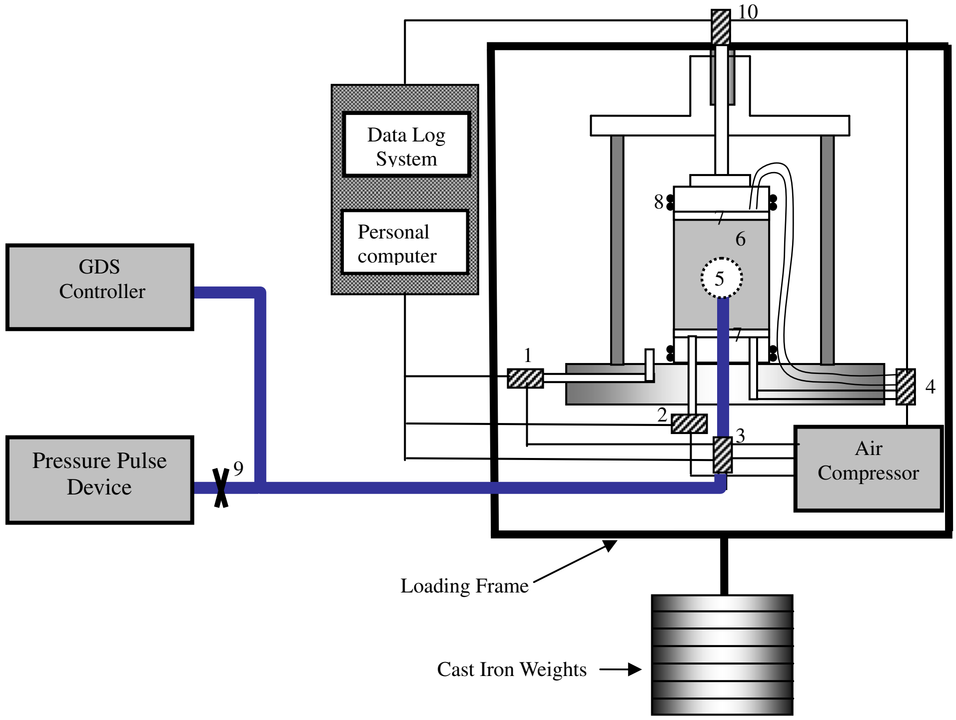


Fig. 9. (a) Deviatoric stress versus axial strain for the sample (dry density 1.3 Mg/cm^3), (b) volumetric strain versus axial strain for the sample and (c) effective principal stress ratio (σ'_1/σ'_3) versus axial strain for the sample.



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. 10. Schematic layout of static and dynamic compaction grouting experimental tests.

ing carbon dioxide before flushing the sample with water, the pore pressure parameter B was found to be 0.98–1.00, thus indicating that the samples were adequately saturated. The second step is to consolidate the specimen to control the initial void ratio. The third step is injection of water into the needle in the center of the specimen. The fourth step is to use the impulse wave generator to introduce vibration in the injection bag and the surrounding soil. The last step is to allow consolidation of the soil until the back pressure and pore pressure have reached equilibrium: i.e., consolidation is completed. It is necessary to note that the last step actually is included in the third and fourth steps in terms of timing.

The basic features of the soils for the dynamic compaction grouting tests are listed in Table 2. A total of 11 dynamic compaction grouting tests were carried out to study the effect of fines content on the compaction efficiency, which are listed in Table 3. In addition, for the optimum fines content based on the experimental results, another 24 dynamic compaction grouting tests were conducted to study the effect of dynamic frequency and effective confining pressure on the compaction efficiency. The basic parameters are listed in Table 4 and Table 5, respectively.

3.3. Definition of compaction efficiency

The mechanical compaction of soils can be defined as the packing of soil particles through a reduction of air voids through temporary application of loads. The reduction of air voids or densification results from the compression or expulsion of the gas phase in the soil's pore space. In this context, assuming that the sandy soil is totally saturated, the effect could 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 ($e_{\min}$) could be measured beforehand, the compaction efficiency η^*, is defined as followed:

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

where $\bar{e}$ = average void ratio, and it is easy to obtain from $\bar{e} = e_0 - \Delta e$, e_0 is the initial void ratio, which could 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

Table 2
Soil properties for experimental test with different fine contents

Fine content	1%	4%	6%	8%	11%	16%	21%	26%	31%	35%	41%
$m_{\max}$	326.5	339.8	340.2	342.9	346.7	339.3	337.8	340.3	338.6	339.7	340.1
$m_{\min}$	293.3	240.6	242.3	246.5	238.6	237.4	230.1	225.6	218.7	211.2	206.9
$\rho_{\max}$	1.691	1.760	1.761	1.775	1.795	1.756	1.749	1.762	1.753	1.759	1.761
$\rho_{\min}$	1.239	1.125	1.254	1.276	1.236	1.229	1.191	1.168	1.132	1.094	1.071
$e_{\max}$	1.122	1.111	1.096	1.060	1.129	1.139	1.207	1.251	1.322	1.405	1.455
$e_{\min}$	0.556	0.495	0.493	0.481	0.465	0.497	0.504	0.493	0.500	0.495	0.493
$e_{\max} - e_{\min}$	0.567	0.616	0.603	0.579	0.663	0.642	0.704	0.759	0.822	0.910	0.0961
D_r	0.445	0.391	0.374	0.328	0.389	0.419	0.479	0.502	0.550	0.587	0.608

Table 3
The parameters for the tests to study the effect of fine content of soil on the compaction efficiency

Test no.	Frequency (Hz)	Fine content (%)	Effective confining pressure (kPa)	Initial void ratio	Injection volume (ml)	Radius (mm)	Initial dry density (g/cm^3)
224	6	1	50	0.853	8	100	1.3
226	6	4	50	0.861	8	100	1.3
227	6	6	50	0.849	8	100	1.3
228	6	8	50	0.861	8	100	1.3
229	6	11	50	0.856	8	100	1.3
230	6	16	50	0.861	8	100	1.3
233	6	21	50	0.855	8	100	1.3
234	6	26	50	0.855	8	100	1.3
236	6	31	50	0.867	8	100	1.3
237	6	36	50	0.862	8	100	1.3
238	6	41	50	0.858	8	100	1.3

Table 4
The parameters for the tests to study the effect of dynamic frequency on the compaction efficiency for the samples with diameter of 75 mm

Test no.	Frequency (Hz)	Fine content (%)	Effective confining pressure (kPa)	Initial void ratio	Injection volume (ml)	Radius (mm)	Initial dry density (g/cm^3)
192	0	6	50	0.867	8	100	1.3
196	1	6	50	0.864	8	100	1.3
198	2	6	50	0.858	8	100	1.3
200	3	6	50	0.863	8	100	1.3
204	4	6	50	0.856	8	100	1.3
205	5	6	50	0.869	8	100	1.3
206	6	6	50	0.851	8	100	1.3
207	7	6	50	0.849	8	100	1.3
208	8	6	50	0.863	8	100	1.3
209	9	6	50	0.863	8	100	1.3
210	10	6	50	0.859	8	100	1.3
212	11	6	50	0.853	8	100	1.3
213	12	6	50	0.861	8	100	1.3
214	13	6	50	0.859	8	100	1.3
216	14	6	50	0.846	8	100	1.3
217	15	6	50	0.853	8	100	1.3
219	16	6	50	0.867	8	100	1.3
220	17	6	50	0.862	8	100	1.3
221	18	6	50	0.858	8	100	1.3

Table 5
The parameters for the tests to study the effect of effective confining pressure on the compaction efficiency

Test no.	Frequency (Hz)	Fine content (%)	Effective confining pressure (kPa)	Dynamic period (s)	Injection volume (ml)	Radius (mm)	Initial dry density (g/cm^3)
155	6	6	50	60	8	100	1.3
158	6	6	100	60	8	100	1.3
160	6	6	150	60	8	100	1.3
166	6	6	200	60	8	100	1.3
167	6	6	250	60	8	100	1.3

grouting efficiency, the average strength enhancement ratio $\bar{\alpha}$ can be derived as followed:

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

where λ is a constant which can be measured using the triaxial tests.

4. Results from the experiments

4.1. Effect of fines content of soil on compaction efficiency

Fig. 11 shows the normalized void ratio/initial void ratio (e/e_0) as a function of time for different fines content in the soil. From Fig. 11, the void ratio change is the largest at a fines content of 6%. Similarly, from Fig. 12 the compaction efficiency is increased when the fines content is increased, and reached the peak compaction efficiency at a fines content of 6%. After that the compaction efficiency decreases abruptly from a peak value of 0.55–0.25, when the fines content increases from 6% to 8%, however, when the fine content continues to increase from 8% to 41%, the compaction efficiency decreases very slowly. For example, the grouting efficiency is found to be 0.25 at a fines content of 8%, while the compaction efficiency is 0.15 when the fines content is 41%. In addition, with increasing fines content, shear stresses are not transferred effectively and the vibrations are severely damped. From Fig. 13, the peak mean shear strength enhancement ratio also occurs at a fines content of 6%, and dropped suddenly from 19.5 to 4.0 when the fines content changed from 6% to 8%. The whole trend of the plot is similar to that of the compaction efficiency.

Figs. 14 and 15 show injection pressure and pore water pressure versus time for different fines content, respectively. From Figs. 14 and 15, both the injection pressure and excess pore water pressure represented the fluctuations during the dynamic compaction

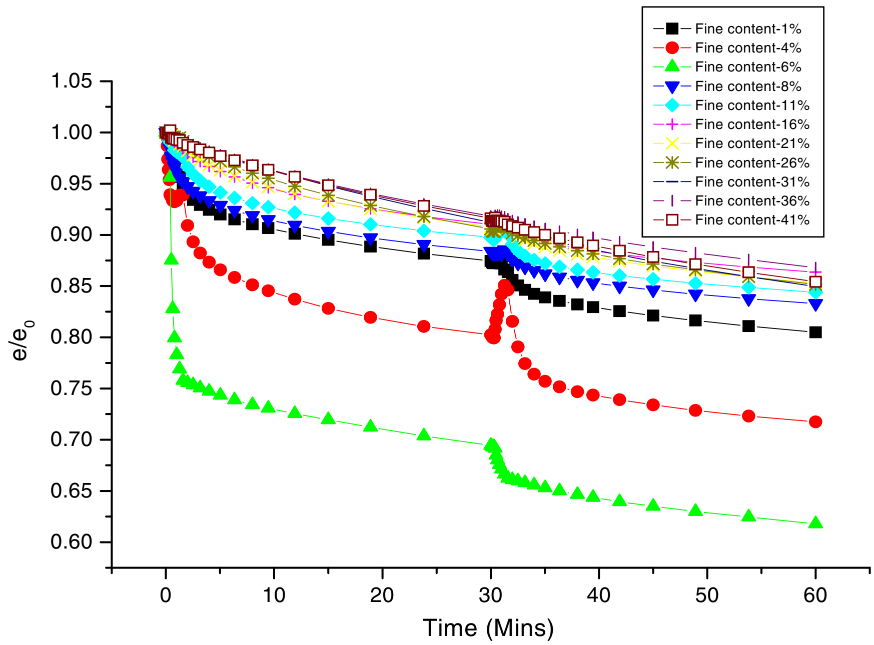


Fig. 11. Normalized void ratio/initial void ratio (e/e_0) versus time for different fine content of soil (Time is the whole time from the first injection to the end of the second consolidation).

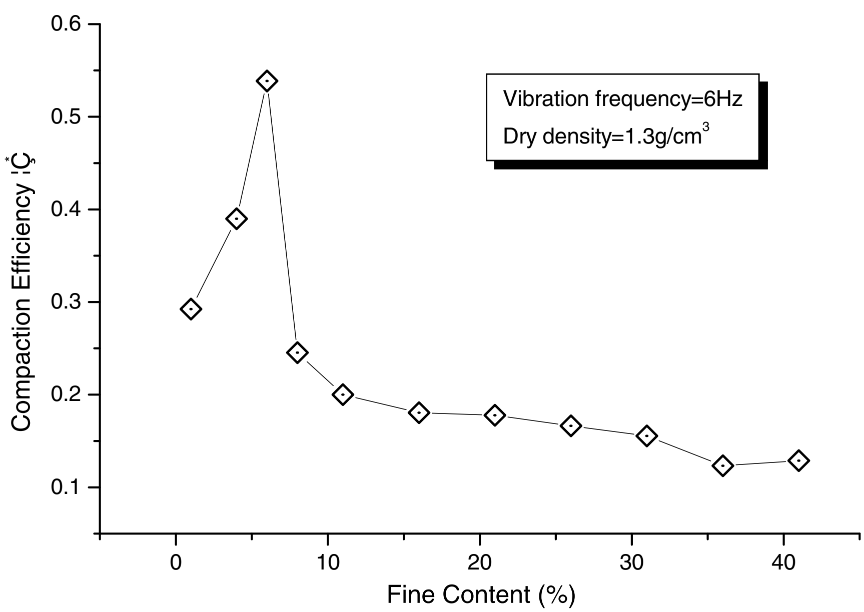


Fig. 12. Relationship between compaction efficiency and fine content of soil.

period, which begin to level off to a steady value, and the excess pore water pressure for fines content of 6% is lower than that for higher fines content tests. This phenomenon of the effect of fines content of soil on the compaction efficiency of dynamic compaction grouting can be explained as follows.

During the application of the pressure pulses, dynamic forces disturb the soil skeleton which tends to force the particles to compact into a denser arrangement. Owing to the sharp difference between the diameter of the finer portion (silt or clay) and the coarse particles (usually sands), most of the fine grains are occupying the voids between sand grains when the fines content is very low (such as 1%). Since the applied loads are carried by the sand skeleton, in other word, these fine grains have little effect on the behaviors of the mixture. It is difficult for the dynamic loads to densify the sand soil. Therefore, the compaction efficiency is not high before the peak value. By increasing the fines content to 6%, some of the finer particles might occupy locations near the contact points between the sand grains and hold the sand grains slightly apart. When this soil is subjected to dynamic loadings, the finer particles are forced to the void between the bigger particles, and the soil is densified subsequently. However, with the fines content increases, the contacts between the sand grains almost vanish and the mixture becomes clay with sand inclusions. In other words, the presence of high clay content distorts the granular structure and causes loosely packed groups of particles. During the application of cyclic loading, these loose particles collapse and cause an increase in pore

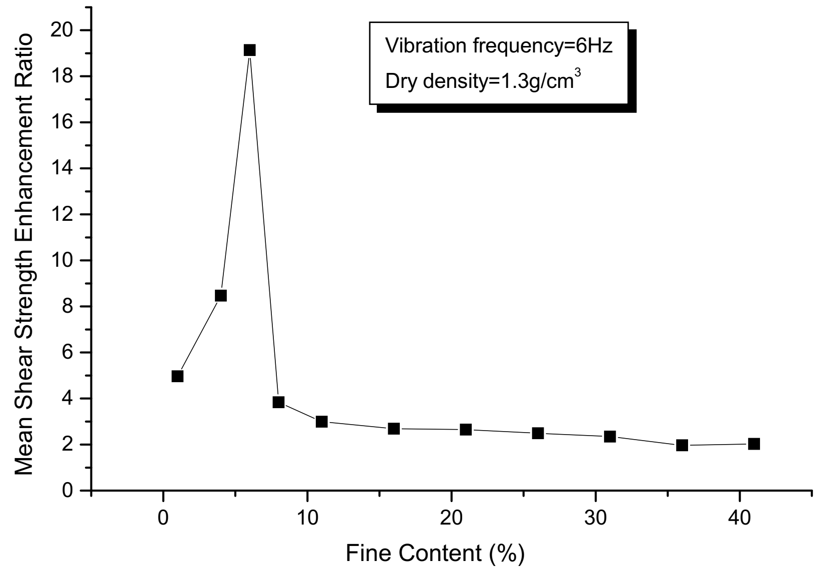


Fig. 13. Mean shear strength enhancement ratio versus fine content of soil.

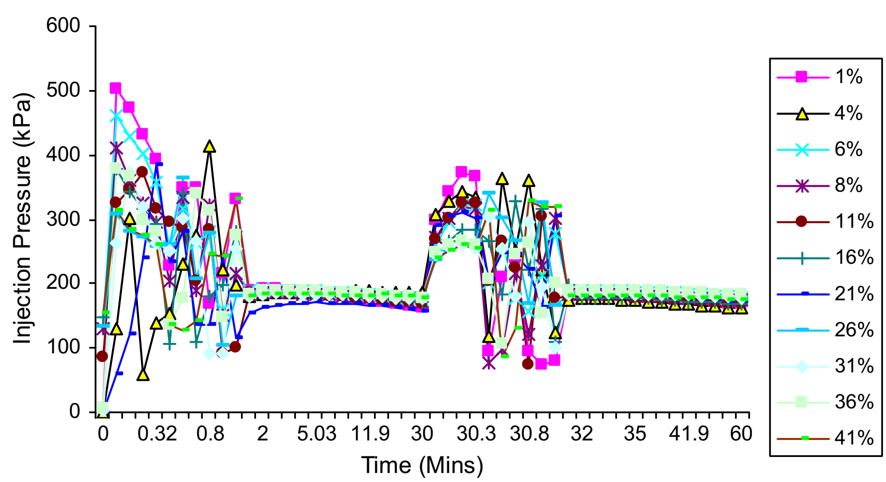


Fig. 14. Injection pressure versus time (time is the whole time from the first injection to the end of the second consolidation).

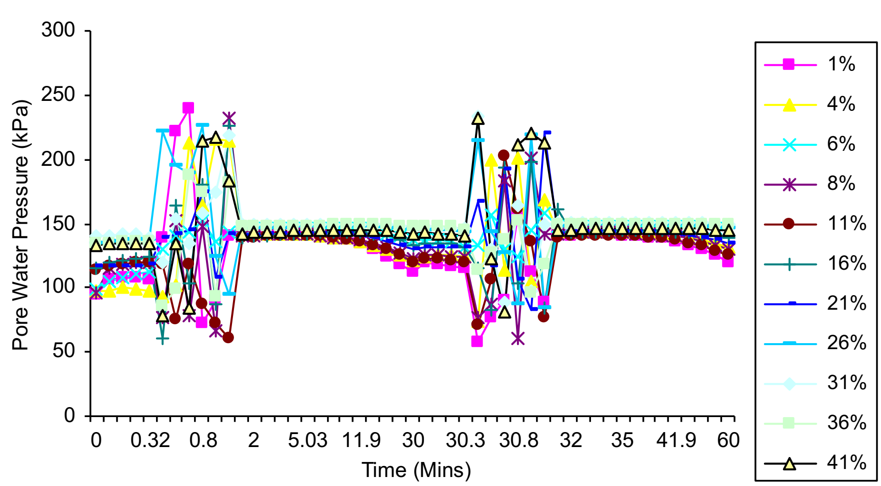


Fig. 15. Pore water pressure versus time (time is the whole time from the first injection to the end of the second consolidation).

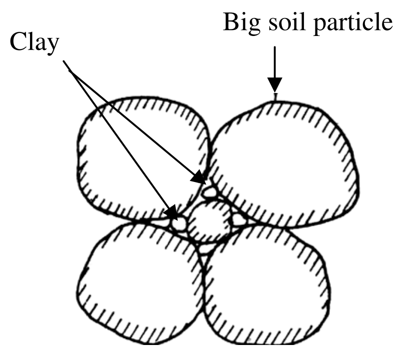


Fig. 16. Schematic particle size distributions.

pressure between grains. The elimination of local instabilities at the contact points produced a more stable structure, which results in an almost constant rate of increase in the excess pore water built up. Consequently, the compaction efficiency and the corresponding mean shear strength enhancement ratio is almost the same for fines contents changes from 16% to 41% in the tests.

In addition, ideal packing is achieved when smaller particles lodged themselves in the voids between larger particles (see Fig. 16). Silt is moisture sensitive with major changes in dry densities for small change in water content for a given compaction energy. Clay is energy sensitive, wherein a small change in compaction energy can produce large changes in dry densities. When the soil is subjected to repeated high-amplitude vibrations, pore water pressure is built up (see Fig. 15) and the effective stress

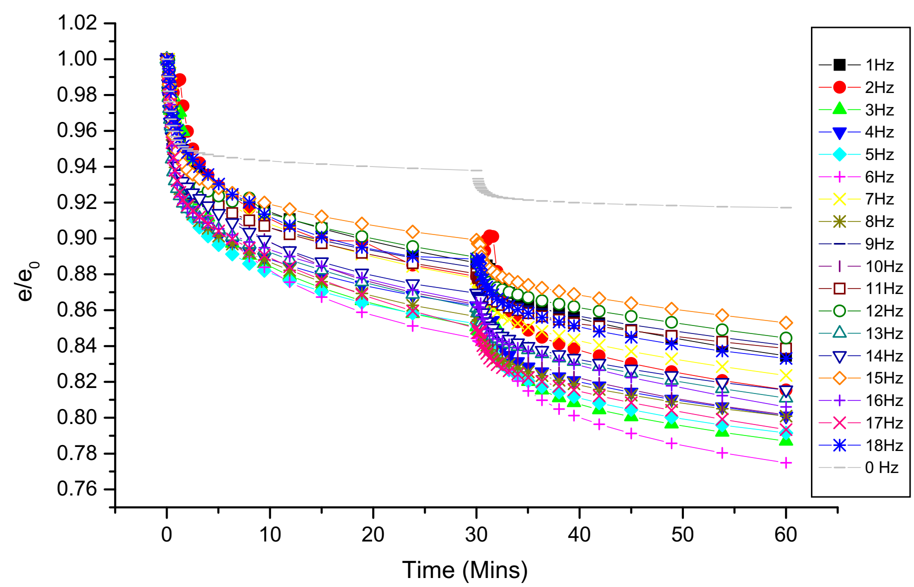


Fig. 17. Normalized void ratio e/e_0 versus time (time is the whole time from the firstinjection to the end of the second consolidation).

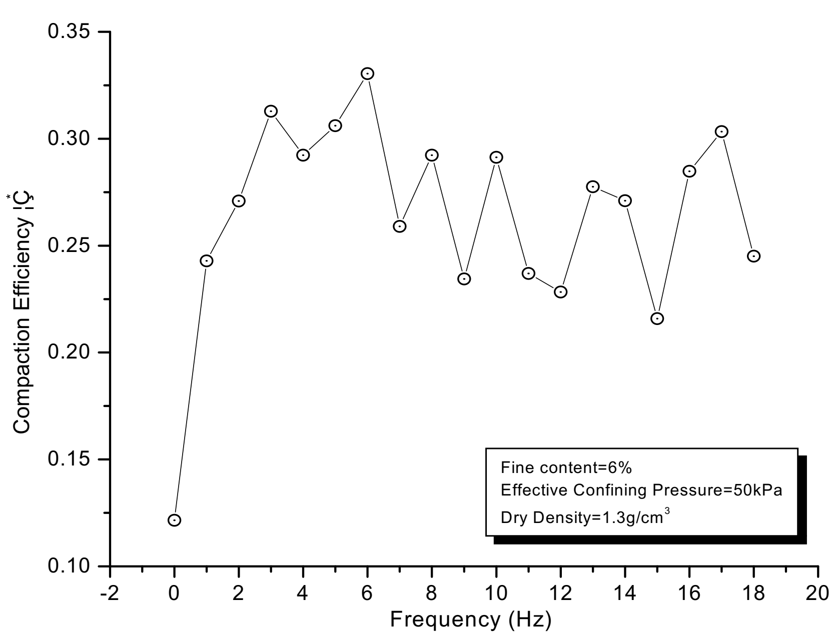


Fig. 18. Compaction efficiency versus dynamic frequency.

is reduced. During the initial phase of compaction, the soil in the vicinity of the compaction probe is likely to liquefy. Whether or not liquefaction occurs will depend on the intensity and duration of dynamic compaction and the rate of dissipation of the excess pore water pressure.

4.2. Effect of dynamic frequency on the compaction efficiency for special fines content of soil

Fig. 17 shows the normalized void ratio e/e_0 as a function of time for dynamic compaction grouting tests with dynamic frequency varies from 0 to 18 Hz. It is noted that the dynamic compaction grouting test for 0 Hz is in fact the steady compaction test. From Fig. 17 it is obvious that void ratio change for all the dynamic tests is much larger than that of the static tests which are denoted by zero frequency. Figs. 18 and 19 show the compaction efficiency and mean shear strength enhancement ratio versus time, respectively. From Fig. 18, the compaction efficiency is 2–3 times higher than the static test for all cases considered. Moreover, the compaction efficiency reaches a peak at around 6 Hz and there are fluctuations of the efficiency at other frequencies. For example, when frequencies are equal to 1 Hz and 2 Hz, the compaction efficiency is a little higher than that for frequencies of 3 Hz and 4 Hz. This is because both the dynamic amplitude and frequency will influence the compaction efficiency. The occurrence of peak efficiency when the dynamic frequency varied from 4 Hz to 18 Hz can be interpreted as the result of reaching the resonance frequency of the soil. Based on Eq. (2), the mean shear strength enhance ratio shown in Fig. 19 gives similar trend with the compaction efficiency.

The relative density D_r of a soil is an important measure to evaluate the degree of soil improvement. Fig. 20 shows the relative density as a function of frequency. The initial relative density for the soils are about 0.4, while the relative density increased to

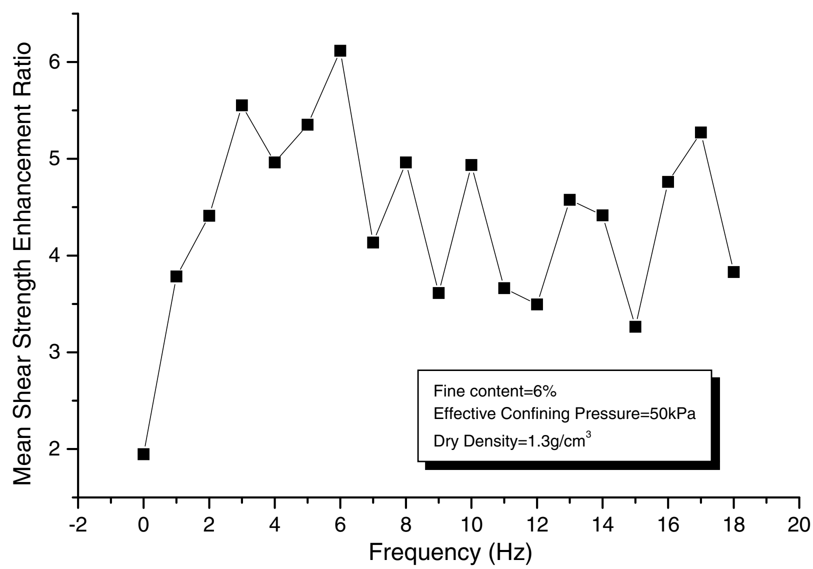


Fig. 19. Mean shear strength enhancement ratio versus dynamic frequency.

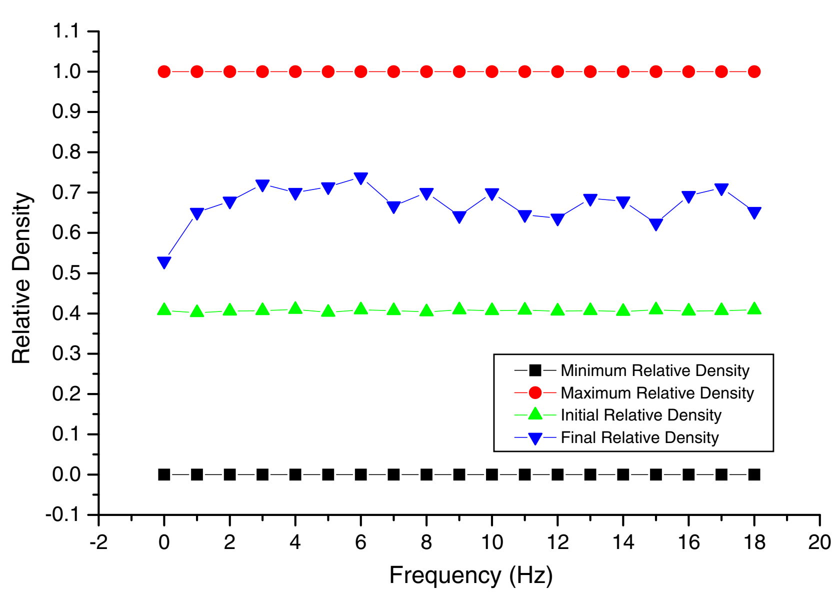


Fig. 20. Normalized relative densities versus frequency.

around 0.7, which indicates that the soil is densified from soft state to densified state. Furthermore, Fig. 20 indicates the amount of improvement in density that could be achieved with $D_r = 1.0$. For instance, for the test with frequency of 0, the initial D_r is 0.4, and the final D_r is only 0.53. In view of the maximum D_r is 1, the soil should be densified further. In comparison, for the test with dynamic frequency of 6, the initial D_r is also 0.4, but the final D_r is about 0.75, and there is less space for improvement.

In addition, the influenced zone around injection cavity is sketched in Fig. 21, where liquefaction occurs close to the injection cavity. Outside this zone is the plastic zone, and plastic zone is surrounded by the compaction zone. Outside compaction zone is the elastic zone, in which the density is not changed. It is obvious that the plastic zone contributes to most of the density enhancement, and the plastic zone is related to the fines content of soils. For this reason, it could be concluded that if the optimum fines content and dynamic compaction frequency are controlled, the best soil improvement effect can be achieved.

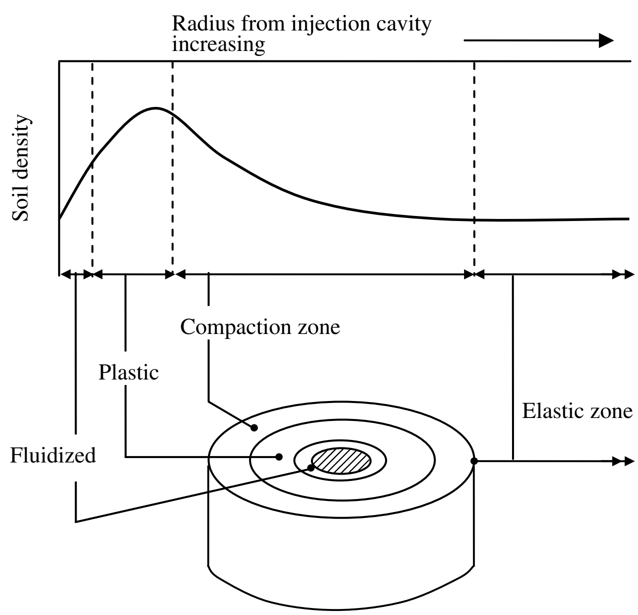


Fig. 21. Density change around injection cavity during dynamic compaction grouting.

4.3. Effect of effective confining pressure on the compaction efficiency for special fines content of soil

In this series of tests, the back pressure was kept at 100 kPa, the effective confining pressure was increased from 50 kPa to 200 kPa, and the corresponding confining pressure was increased from 150 kPa to 300 kPa. Fig. 22 shows the normalized void ratio e/e_0 versus time for different effective confining pressure for the special fines content of 6%. Figs. 23 and 24 show the compaction efficiency and mean shear strength enhance ratio versus time, respectively. From Figs. 23 and 24, both the compaction efficiency and shear strength enhancement ratio increase gradually with the effective confining pressure increasing from 50 kPa to 250 kPa. For instance, when the effective confining pressure increases from 50 kPa to 150 kPa, the compaction efficiency is increased gradually from 0.17 to 0.18. Similarly, the mean shear strength enhancement ratio also increases gently from 2.51 to 2.64. However, when the effective confining pressure increases from 150 kPa to 200 kPa, the compaction efficiency jumps from 0.17 to 0.23, and the corresponding mean shear strength enhancement ratio is increased from 2.62 to 3.58. With the increase of effective confining pressure from 200 kPa to 250 kPa, not only the compaction efficiency, but the mean shear strength enhancement ratio increase markedly. It should be noted that due to the increasing effective confining pressure, it becomes easier for the soil particle to be crushed and the fines content of soils may be changed. Accordingly, the optimum fines content mentioned above is only the best one under the same stress level. If the effective confining pressure is changed, the optimum fines content can also be changed.

Figs. 25 and 26 show the injection pressure and pore water pressure versus time, respectively. From Figs. 25 and 26, both the peak injection pressure and pore water pressure increases with increase of effective confining pressure from 50 kPa to 250 kPa. This is due to the fact that higher injection pressure is required to expand the membrane when it is subjected to higher confining pressure.

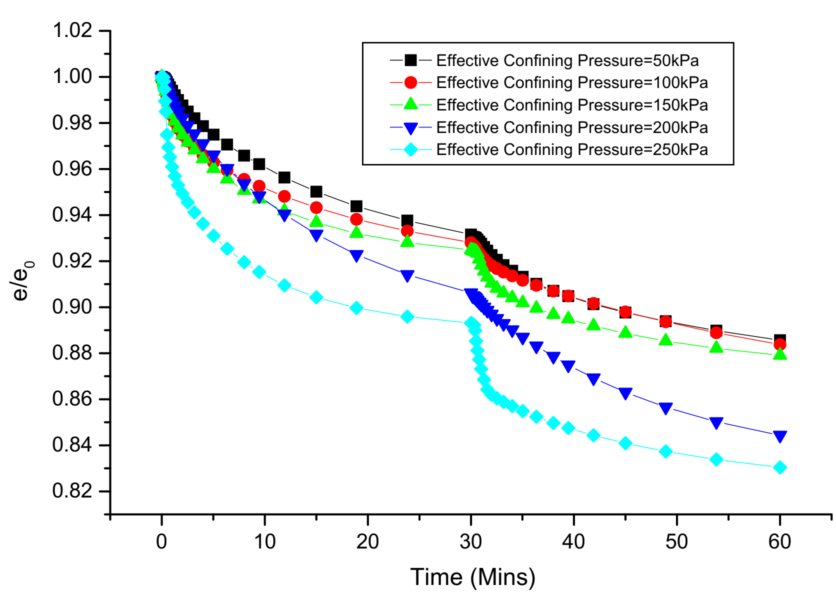


Fig. 22. Normalized void ratio e/e_0 versus time for different confining pressure.

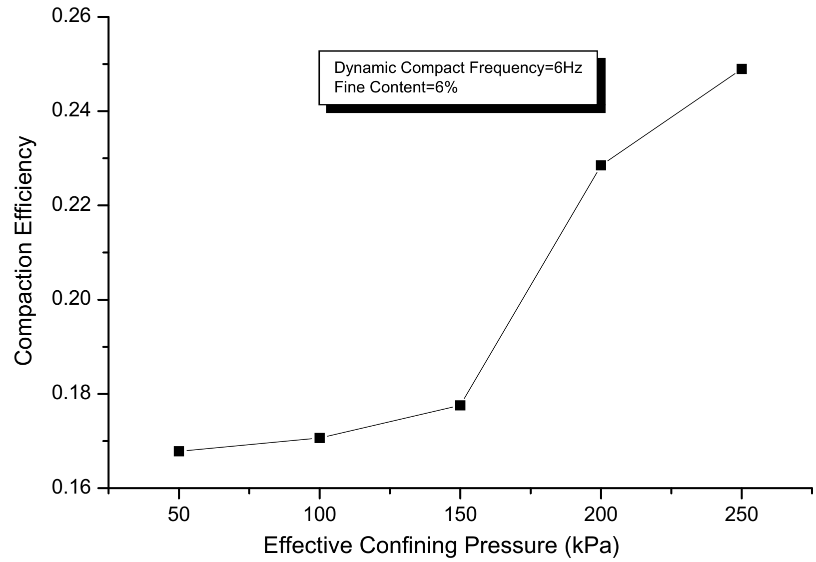


Fig. 23. Compaction efficiency versus confining pressure.

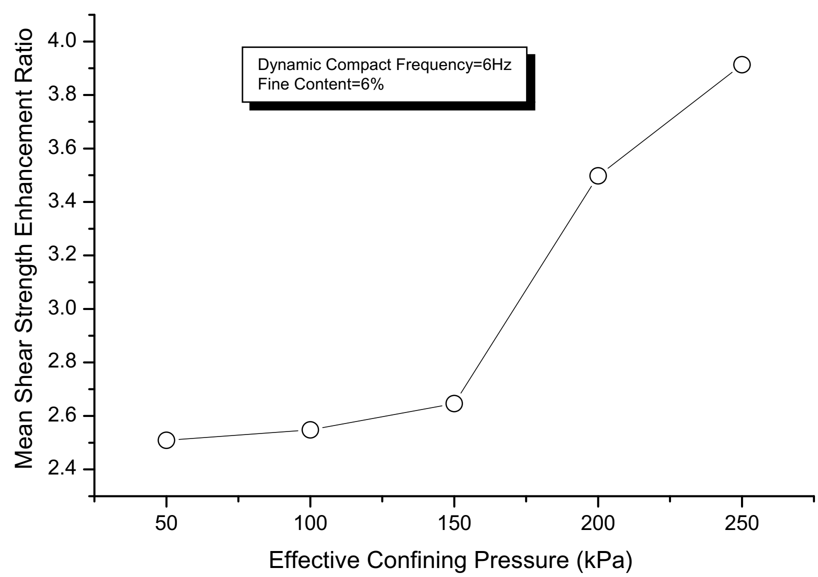


Fig. 24. Effective confining pressure versus confining pressure.

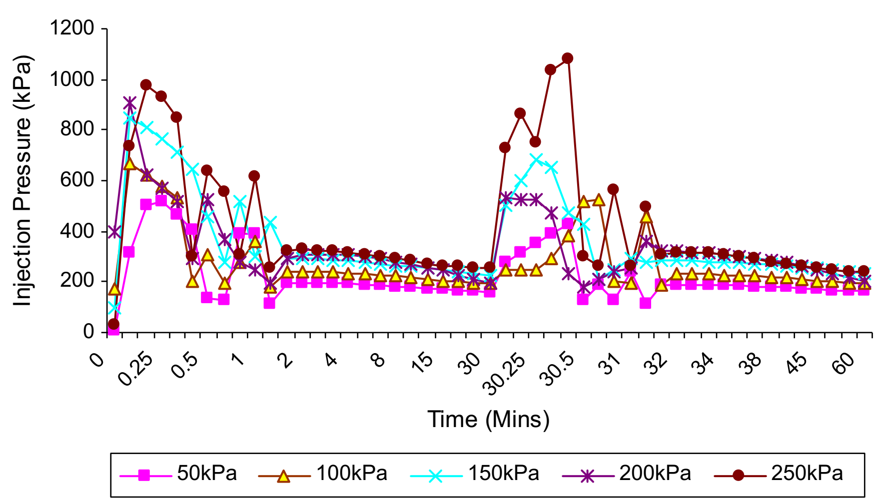


Fig. 25. Injection pressure versus time for different effective confining pressure.

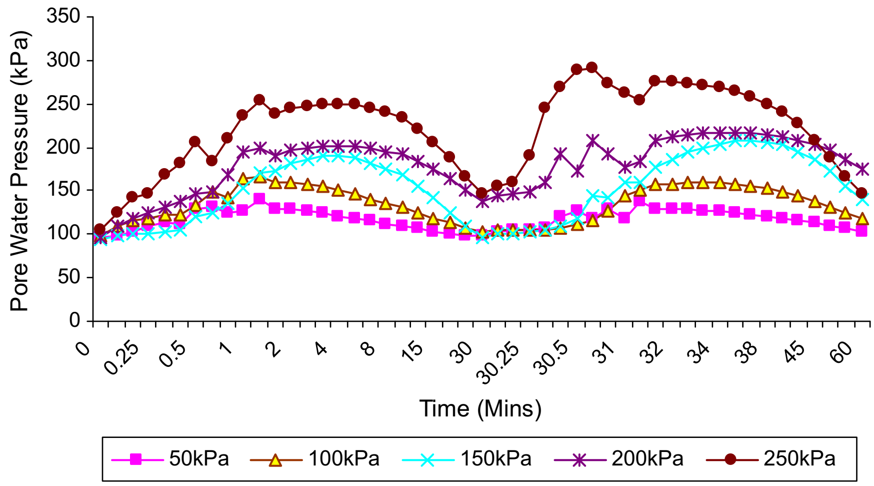


Fig. 26. Pore water pressure versus time for different effective confining pressure.

5. Conclusions

- It is found that, as the fines content increases, the dry density decreases, however the optimum water content increases. In addition, the permeability is decreased due to a reduction in void ratio, and the coefficient of consolidation increases with higher fines content. Furthermore, with an increase in fines content, the shear stress tends to be constant after it reaches the peak strength. However, after reaching the peak value, the shear stress is reduced with further increase in fines content.
- The compaction efficiency increases with the fines content increased, and reaches peak compaction efficiency at a fines content of 6% for the soil being tested. After that the compaction efficiency drops abruptly from a peak value of 0.55–0.25 when the fines content is increased from 6% to 8%. However, when the fines content continues to increase from 8% to 41%, the compaction efficiency decreases very slowly.
- For the CDG tested with fines content of 6%, the compaction efficiency reaches an optimum value when the dynamic frequency reaches 6 Hz. In addition, the optimum fines content is controlled by the effective confining pressure, as the higher effective confining pressure crushes the soil particles more easily.
- Experimental results show that the influenced zone around injection cavity can be divided into four zones: liquefied zone, plastic zone, compact zone and elastic zone. The plastic zone contributes the most to the densification of the soil.
- For the CDG with a fines content of 6%, the compaction efficiency decreases with increase in the effective confining pressure from 50 kPa to 250 kPa.

Acknowledgements

Financial support for this project was partly provided from the RGC Grants (Project No. 9040593 and CityU 1178/03E) of the Government of Hong Kong and the Australian Research Council grant DP0881238. The generous support in providing equipment needed for experimental portion of this research from the University of Hong Kong is gratefully acknowledged.

References

[1] Kim D, Sagong M, Lee Y. Effects of fine aggregate content on the mechanical properties of the compacted decomposed granitic soils. Construct Build Mater 2005;19:189–96.
[2] Chang N, Yeh S, Kaufman L. Liquefaction potential of clean and silty sands. In: Proceedings of third international earthquake microzonation conference, vol. 2. Seattle, USA; 1982. p. 1017–32.
[3] Nagaraj TS, Pandian NS, Narasimha Raju PSR. Stress state permeability relationships for fine grained soils. Geotechnique 1993;43(2):333–6.
[4] Thevanayagam S, Ravishankar K, Mohan S. Effects of fines on monotonic undrained shear strength of sandy soils. ASTM Geotech Test J 1997;20(4):394–406.
[5] Yin JH. Properties and behaviour of Hong Kong marine deposit with different clay contents. Canad Geotech J 1999;36:1085–95.
[6] Chien L-K, Oh Y-N. Influence of fines content and initial shear stress on dynamic properties of hydraulic reclaimed soil. Canad Geotech J 2002;39(1):242–53.
[7] Chien L-K, Oh Y-N, Chang C-H. Effects of fines content on liquefaction strength and dynamic settlement of reclaimed soil. Canad Geotech J 2002;39(1):254–65.
[8] Shapiro S, Yamamuro J. Effects of silt on three-dimensional stress–strain behavior of loose sand. J Geotech Geoenviron Eng ASCE 2003;129(1):1–11.
[9] Naeini SA, Baziar MH. Effect of fines content on steady-state strength of mixed and layered samples of a sand. Soil Dyn Earthquake Eng 2004;24(3):181–7.
[10] Barden L, Sides GR. Engineering behaviour and structure of compacted clay. J Soil Mech Foundat 1970;94(4):1171–5.
[11] Seed RB, Cetin KO, Moss RES, Kammerer AM, Wu J, Pestana JM, et al. Recent advances in soil liquefaction engineering: a unified and consistent framework, in keynote address, In: 26th annual geotechnical spring seminar, Los Angeles section of the GeoInstitue, American Society of Civil Engineers, H.M.S. Queen Mary, Long Beach, California; 2003.
[12] Wang SY. Fundamental study of static and dynamic compaction grouting in complete decomposed granite, PhD thesis. Department of Building and Construction, City University of Hong Kong, Hong Kong, China; 2006.