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A study on the improvements of geotechnical properties of in-situ soils by grouting

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Abstract. This paper discusses improvements of compressibility, permeability, static and liquefaction strengths of in-situ soils by grouting. Both field testing and laboratory evaluation of the on-site samples were conducted. The improvement of soils was influenced by two main factors, i.e., the grout materials and the injection mechanisms introduced by the field grouting. On-site grout mapping revealed the major mechanism was fracturing accompanied with some permeation at deeper zones of sandy soils, where long-gel time suspension grout and solution grout were applied. The study found the compressibility and swelling potential of *CL* soils at a 0.5 m distance to grout hole could be reduced by 25% and 50%, respectively, due to the grouting. The effect on hydraulic conductivity of the *CL* soils appeared insignificant. The grouting slightly improved the cohesion of the *CL* soils by 10~15 kPa, and the friction angle appeared unaffected. The grouting had also improved the cohesion of the on-site *SM* soils by 10~90 kPa, while influences on the friction angle of soils were uncertain. Liquefaction resistances could be enhanced for the sandy soils within a 2~3 m extent to the grout hole. Average improvements of 40% and 20% on the liquefaction resistance were achievable for the sandy soils for earthquake magnitudes of 6 and $\geq$7.5, respectively, by the grouting.

Keywords: ground improvement; geotechnical properties; soil grouting; laboratory testing; field testing

1. Introduction

The technique of grouting has been used in the petroleum industry and rock engineering for oil or gas extraction and for in-situ stress measurements (Mitchell and Van Court 1992). In environmental engineering the grouting technique has been adopted to form a barrier for contaminant migration or to enhance landfill gas recovery (Wong and Alfaro 2001). In recent years, the grouting technique with special grout solutions has been considered as a tool for fixing deteriorated historic structures (Yang and Cheng 2013).

For geotechnical engineering applications, the grouting has been adopted for two main purposes: strengthening the ground, and stoppage of groundwater flow. A number of case histories have shown the grouting was of critical importance for a successful excavation of underground structures (Tseng *et al*. 2001).

In terms of the pressure and material for injection, the grouting can be broadly categorized as

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the jet grouting with an injection pressure usually greater than 100 bars, the compaction grouting with a thick grout of less than 1-inch (2.5 cm) slump, and other types of grouting, such as penetration or fracturing grouting, which normally involve a relatively low injection pressure ($\lesssim$ 30 bars) and a thin grout (Hausmann 1990).

Disregarding the jet grouting due to its significantly high injection pressures and unique mixing method, injection mechanisms of compaction grouting and other types of low pressure grouting would generally include permeation, compaction and fracturing (Hausmann 1990). A permeation mechanism can be assumed for a very fine-grained grout or solution injected under a very low pressure and flow rate (Mori *et al*. 1992, Axelsson *et al*. 2009). In such case, the grain structure of the grouted soil would not be altered, and improvements of the geotechnical properties (e.g., compressibility, permeability, or shear strength) of the grouted soils would be due mainly to the adhesion and solidification of the grouts. Alternatively, the compaction or fracturing mechanism would be resulted by relatively coarse-grained grouts or suspensions and/or injection with somewhat higher injection pressures or rates (Mori *et al*. 1992, Axelsson *et al*. 2009). In contrast to the permeation mechanism which commonly forms a more uniform but smaller size grouted mass, the compaction and fracturing mechanisms would improve the ground through densification of the soils adjacent to the grout (for compaction mechanism), as well as the reinforcement of soil mass by the grouted fractures (for fracturing mechanism). Due to random nature in fracture distribution, the influenced zone by the fracture grouting would tend to be wider in size, but not as uniformly improved, as for the permeation grouting.

Obviously, the improvement of geotechnical properties of in-situ soils would depend not only on the grouts adopted, but also the injection mechanism that might introduce densification or reinforcement of the soil mass. Existing literatures on the improvement of soil grouting are mostly related to the experimental study of mechanical properties of grouts or grout-mixed soils. Relatively few deal with in-situ grouted soils with influences of injection mechanism due to grouting. It is therefore the aim of this study to investigate the improvement in geotechnical properties of in-situ soils by a field grouting. Both on-site and laboratory testing were conducted for the evaluation of improvements and the results are discussed in the following sections.

2. Previous studies on engineering properties of grouted soils

The laboratory approach is often adopted in the literature for evaluating the improvement of geotechnical properties of soils by grouting (Liao and Liu 1996, Soga *et al*. 2004, Anagnostopoulos *et al*. 2011, Mutman and Kavak 2011, Pantazopoulos and Atmatzidis 2012, Fattah *et al*. 2015). Relatively few studies, however, show field investigation or testing of the improvement of in-situ grouted soils (Kikuchi *et al*. 1997).

Both neat grouts and grouted soils are two main subjects in the literature for laboratory evaluation. For the neat grout studies, Rosquoet *et al*. (2003) and Eriksson *et al*. (2004) conducted experiments on the rheology and penetrability of the cement-based grouts. Schwarz and Krizek (1992) and Liao *et al*. (1992) considered microfine cement grouts, the grouts with a mean particle size of approximately 1/10 to 1/5 of that for the ordinary Type I Portland cement, and examined their physical and mechanical properties. Other types of grout, such as silica sol, slag cement, microbial mortar, additives of superplasticisers, etc., have been reported and tested in the literature (Axelsson 2006, Lee *et al*. 2008, Yang and Cheng 2013, Anagnostopoulos 2014).

For the grouted soil studies, Krizek *et al*. (1992), Maalej *et al*. (2007), and Mutman and Kavak

(2011) performed laboratory testing on the geotechnical properties of sands injected with ordinary or microfine cements. Dynamic properties of sands with cement-based or other types of grouts were studied through laboratory testing by Delfosse-Ribay *et al.* (2004) and Pantazopoulos and Atmatzidis (2012). Vipulanandan and Shenoy (1992) conducted a series of experiments for sands with additives of sodium silicate, bentonite, calcium chloride, silica fume, and fly ash. Soils injected with special types of grout, such as acrylic resin, urease, and calcium phosphate compound with calcium carbonate (CPC-CC), have also been studied in the literature (Anagnostopoulos 2005, Yasuhara *et al*. 2012, Akiyama and Kawasaki 2012).

Four engineering properties including compressibility, permeability, shear resistance and liquefaction resistance are considered for the grouted soils, and relevant studies are reviewed. In a study of two types of cement-based grouts (European vs. Japanese), Yang *et al*. (2009) found the increase in injection volume would decrease the compressibility of the grouted sand. For Ottawa sand with an initial relative density (D_r) of 30%, an increase in the injection volume of European type of grout from 0 to 20% would decrease the compression index (C_c) of the sand from 0.12 to 0.03; about 25% of the original ungrouted value.

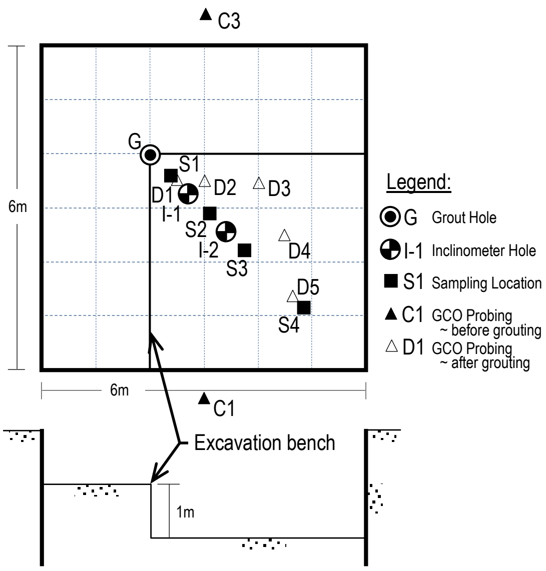
Reduction in the hydraulic conductivity (K_h) is often a design target for grouting in granular materials. Generally, grouting can reduce the hydraulic conductivity of sandy soils. However, experiences show a hydraulic conductivity of less than 10^{-4}~10^{-5} cm/sec would be difficult to achieve in the field for grouting in sandy layers. Some studies have indicated the hydraulic conductivity of the grouted sands with microfine cement or chemical solutions could be reduced by 1~2 orders of magnitude as compared with the untreated sands (Krizek and Helal 1992, Anagnostopoulos *et al*. 2011, Yasuhara *et al*. 2012). Based on model grouting results, Yang *et al*. (2009) found the hydraulic conductivity of loose Ottawa sand decreased from 2.5×10^{-3} cm/sec to 1.25×10^{-3} cm/sec, a 50% reduction, for a cement-base grout with the injection volume varied from 0 to 20%.

Mitchell (1976) indicated the unconfined compressive strength of cement stabilized soils would increase linearly with cement content. Liao and Liu (1996) conducted a laboratory compaction grouting and found the undrained strength of clay in the neighborhood of injection hole was increased by 200% for the grout volume up to 12%. Generally, the friction angle of sands would not be changed significantly due to the grouting; however, the cohesion of the soils could be increased with the content of cement-based grouts (Hsiao 1996, Maalej *et al*. 2007). Krizek *et al*. (1992) indicated an increase in the cohesion from 142 kPa to 456 kPa for the grouted Ottawa 20-30 sand with the water/cement ratio ranged from 5:1 to 2:1.

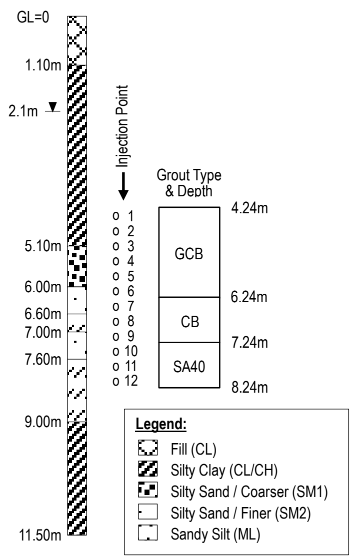
As found in the literature, the tensile strength would be approximately 1/10 to 1/8 of the compressive strength for the sands injected with cement-based or sodium silicate grouts (Krizek *et al*. 1992, Vipulanandan and Shenoy 1992).

Grouting is commonly adopted to mitigate the liquefaction potential of sandy soils in seismic prone areas. Hsiao (1996) indicated the liquefaction resistance of Li-Gang sand with D_r = 35% and a cement content of 1% would be equivalent to that of the clean sand with D_r = 50%. Similarly, the liquefaction resistance of the sand with D_r = 35% and a cement content of 4% would be about the same as that of the clean sand with D_r = 70%. Uchida *et al*. (1996) conducted a laboratory study on the improvement of liquefaction resistance of sands by inclusion of cement mortar sheets, simulating a situation for fracture grouting. They found the liquefaction resistance of sands with D_r = 70% would be increased by 20~50% due to a 4% inclusion (by volume) of the mortar sheets. They also found a delay response of the excess pore pressure built-ups for the sands with mortar sheets during the cyclic loading. Chang *et al*. (2004) performed a cyclic triaxial testing on the liquefaction resistance of Tzuo-Swei sand (D_r = 30%) injected with a cement-based grout. They showed a 10~25% increase in the liquefaction resistance could be obtained with a grout volume of 7~21%, in a condition that allowed the test specimen to expand during model grouting. An additional increase of 20~25% in liquefaction resistance was found in the test specimen with restriction to change volume during model grouting, indicating an influence of compaction by the grouting.

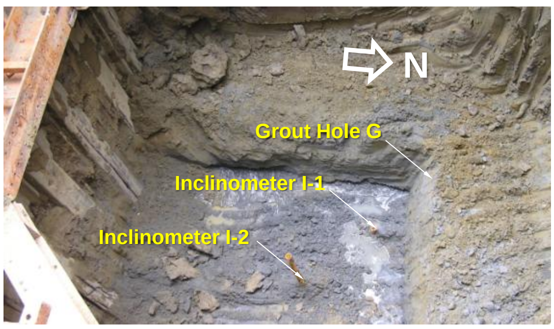
Cyclic shear modulus and damping ratio were studied by Delfosse-Ribay *et al*. (2004) and results showed the grouting would improve the shear modulus of the test sands at small strains. For the large strains, however, bonds between the grout and sand grains were altered and the moduli of the grouted sands became the same as for the clean sands. Pantazopoulos and Atmatzidis (2012) performed torsional resonant column and bender element tests for the cement-based grouted sands, with a grout volume equal two times the void space of the sands. They found the dynamic shear and initial Young’s moduli were improved by a factor of 4~25 for the grouted sands.



(a) Site plan, testing and sampling locations



(b) Soil profile and grouting depths



(c) In-situ excavated bench for mapping

Fig. 1 Field grouting site plan and soil profile

3. Field grouting and observations

This paper discusses the improvement of geotechnical properties of in-situ soils by grouting. Both field testing and laboratory evaluation of the on-site samples were performed. As shown in Fig. 1(a), the field grouting program included a 10 cm-dia. grout hole (*G*) drilled to a depth of 9 m. A 5 cm-dia. sleeve tube (*tube à manchette*; *TAM*) was then placed in the center of the grout hole. The injection holes with sleeve covers spaced 33.3 cm along the *TAM* were designed with a grouting depth ranged 4.2~8.2 m. Prior to the grouting, the annular space between grout hole and *TAM* was filled with a cement-bentonite (*CB*) grout and cured for 24 hours for protection purposes. Fig. 1(b) shows the soil layers encountered at the site. Fig. 1(c) shows a photo of the excavated bench.

Three types of grout were adopted for the grouting. As indicated in Table 1, *GCB* grout consists of a cement bentonite mixture (Grout A) and a sodium silicate solution (Grout B), which were prepared in separate grout tanks and then mixed and injected into the ground. The gel time of *GCB* grout was controlled at 50~60 seconds. The *CB* grout, as mentioned previously, was designed with a gel time of 16~24 hours. The *SA*40 grout is a sodium silicate solution prepared with a gel time of 3~5 minutes. The depths applied and the soils encountered for each of the grout types are shown in Table 1 as well as in Fig. 1(b). Due to generally acceptable groutability limits ($G_R = D_{15}^{soil} / D_{85}^{grout} \geq 15$) for a permeation grouting (Mitchell 1982, Axelsson *et al*. 2009), the suspension grouts (*GCB* and *CB*) adopted in this study would be expected to produce a more fracturing or compaction type of injection mechanism rather than a permeation type of mechanism.

After the grouting, the site was excavated in benches as shown in Fig. 1(a). Within the depths of grouting, the on-site mapping of grout distributions was conducted at each excavated bench and results are summarized in Fig. 2. As illustrated, fracturing appeared to be the primary injection mechanism for the grouting, implying improvements of the ground would be related to the compression or reinforcement of the grouted fractures. Some permeation types of mechanism were also found, however, at the excavation depths of 6.5 m and 7.5 m where the *CB* and *SA*40 grouts were applied. Due to long gel time of the *CB* grout and a solution type of the *SA*40 grout, we expected some bleeding or permeation of the grouts into the on-site sandy soils would likely occur, as mentioned by Mori *et al*. (1992).

Prior to the excavation, penetration tests by GCO probing (GEO 1996; Fig. 3) were conducted at several locations within and outside the grouting area as shown in Fig. 1(a). In each of the

Table 1 Grouts adopted in this study

Grout	Type	Components*	Gel time	Soils encountered	Depths applied (m)	$\frac{D_{15}^{soil}}{D_{85}^{grout}}$
GCB	Suspension	S+C+B+W	50~60 seconds	CL	4.2~5.1	< 0.1
				*SM*1	5.1~6.0	0.6
				*SM*2	6.0~6.2	0.2
CB	Suspension	C+B+W	16~24 hours	*SM*2	6.2~6.6	0.2
				ML	6.6~7.0	No data
				*SM*2	7.0~7.2	0.2
SA40	Solution	S+W+A	3~5 minutes	*SM*2	7.2~7.6	N/A**
				ML	7.6~8.2	N/A**

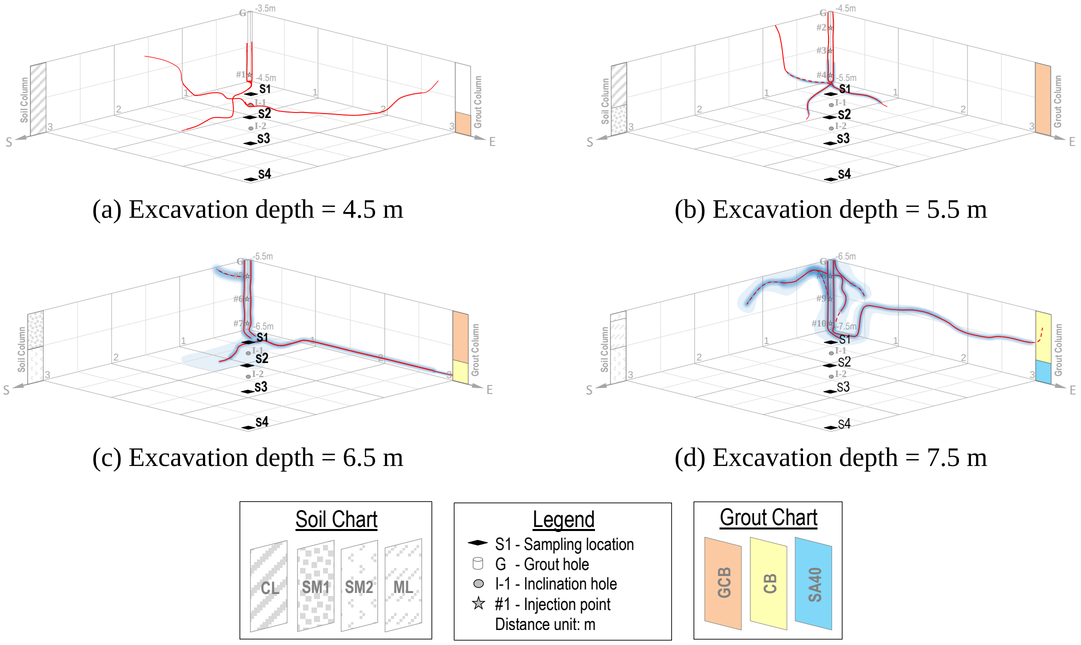


Fig. 2 Results of grout mapping at various excavation depths

Fig. 3 GCO probing

Fig. 4 Harvard miniature compactor

excavation steps, undisturbed thin-tube samples were retrieved at various distances (0.5~4 m) to the grout hole, with locations shown in Fig. 1(a). The tube samples were properly sealed, protected, and stored in a temperature controlled room for the subsequent laboratory testing.

4. Laboratory test preparation

Laboratory testing was conducted to evaluate the improvement of geotechnical properties of in-situ soils by the grouting. The geotechnical properties of concern had included the compressibility, permeability, shear strength, and liquefaction resistance. One-dimensional consolidation tests based on ASTM D2435 were performed for evaluating the compressibility and permeability of the clayey soils. Consolidated drained direct shear tests per ASTM D3080 were carried out for assessing shear strength characteristics of the soils. A cyclic triaxial apparatus was adopted for evaluating the dynamic behavior and liquefaction resistance of the sandy soils.

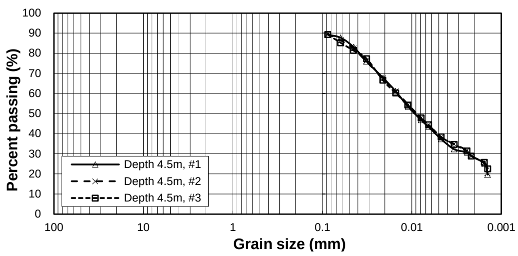
Undisturbed thin-tube samples collected at the grouted site were treated as the grouted samples. For the clayey soil at Depth 4.5 m, the tube samples were carefully extruded, trimmed, and then set up in the test apparatus for the testing. For the sandy soils at Depths 5.5~7.5 m, sampling technique was the same as adopted for the clayey soils. However, the tube samples had to be drained and frozen to obtain some sample rigidity before being extruded and trimmed for further testing.

Disturbed soils were also collected at the site where no apparent influence by the grouting was noticed. The disturbed soils were then used for the preparation of ungrouted samples. For the disturbed clayey soils at Depth 4.5 m, the test specimen were prepared by using a Harvard miniature compactor (Fig. 4) and compacted, per the procedure by Head (1994), to the density and moisture determined based upon the results of in-situ sand cone tests. For the disturbed sandy soils at Depths 5.5~7.5 m, the test specimen were prepared by the commonly adopted moist tamping method to the densities determined by the in-situ sand cone tests.

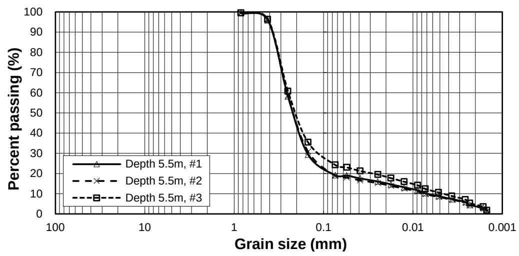
5. Results of laboratory testing

5.1 Physical properties of ungrouted soils

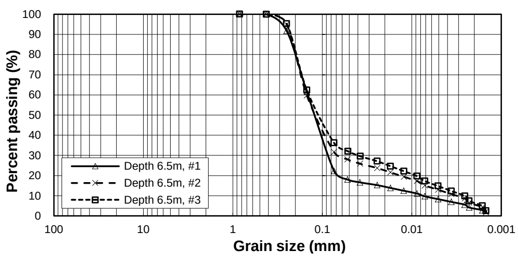
Grain size distribution curves and physical properties of the on-site soils are shown in Fig. 5 and Table 2.



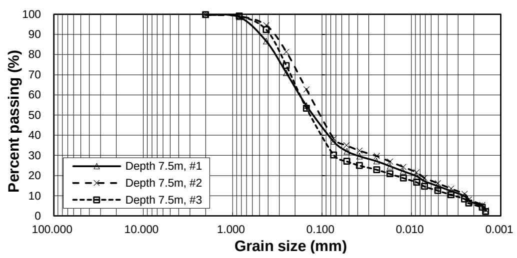
(a) Depth = 4.5 m (Soil: *CL*)



(b) Depth = 5.5 m (Soil: *SM*1)



(c) Depth = 6.5 m (Soil: *SM*2/*ML*)



(d) Depth = 7.5 m (Soil: *SM*2/*ML*)

Fig. 5 Grain size distributions of in-situ ungrouted soils

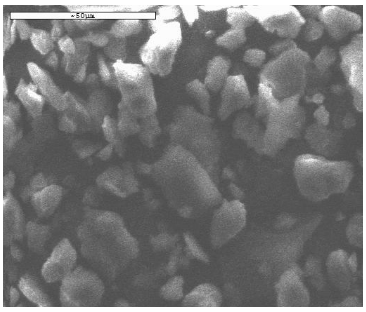
At Depth 4.5 m, the soil is classified as *CL* with a fines content of 88%. The liquid limit and plasticity index of the clayey soil are 39.7 and 20, respectively. At Depths 5.5, 6.5, and 7.5 m, all

Table 2 Physical properties of in-situ ungrouted soils

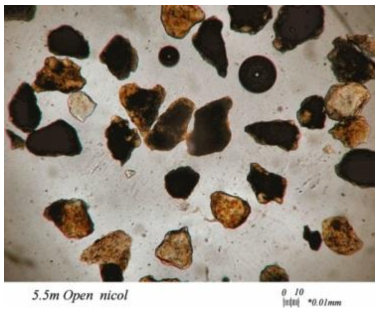
Property name	Property values at various sampling depths			
	4.5 m	5.5 m	6.5 m	7.5 m
Specific gravity, G_s	2.70	2.68	2.67	2.68
Effective grain size, D_{85} (mm)	0.054	0.350	0.200	0.300
Effective grain size, D_{50} (mm)	0.009	0.230	0.130	0.130
Effective grain size, D_{15} (mm)	0.0004	0.0200	0.0080	0.0060
Uniformity coefficient, C_u	-	34.7	37.5	53.3
Curvature coefficient, C_d	-	12.8	6.0	5.2
Fines content, FC	88%	~22%	~30%	~35%
Liquid limit, LL (%)	39.7	NP*	NP*	NP*
Plasticity index, PI (%)	20.0	NP*	NP*	NP*
USCS category	*CL*	*SM*1	*SM*2/*ML*	*SM*2/*ML*
Natural water content**, ω_n	27%	25%	14.5%	18.5%
Dry unit weight**, γ_d (kN/m^3)	17.5	15.0	15.5	15.9

* "NP" = non-plastic

** Based on results of the in-situ sand cone testing.

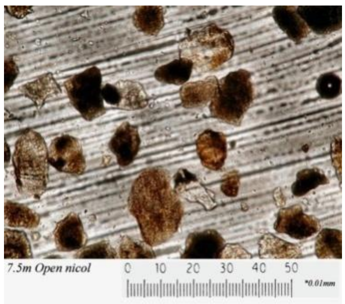


(a) Depth 4.5 m



(b) Depth 5.5 m

(c) Depth 6.5 m



(d) Depth 7.5 m

Fig. 6 Particle images of in-situ soils from scanning electron and optical microscopes soils are classified as *SM* with non-plastic fines. The sandy soil at Depth 5.5 m is coarser with the mean effective particle size (D_{50}) of 0.23 mm and a fines content of about 22%. The sandy soils at Depths 6.5 m and 7.5 m are finer, both with a D_{50} = 0.13 mm and a fines content of about 30~35%. Due to slight differences in particle size and fines content, the sandy soil at Depth 5.5 m are therefore termed as *SM*1 and the soils at Depths 6.5 and 7.5 m termed as *SM*2 in this study.

Fig. 6 shows the scanning electron and optical microscope images of soil particles at various depths of the site. As can be seen, the shapes of sandy particles at Depths 5.5, 6.5, and 7.5 m are sub-round to sub-angular.

Table 3 indicates results of X-ray diffraction (XRD) analysis of mineral composition of the in-situ soils. The *CL* soil at Depth 4.5 m has a significant portion of illite and chlorite minerals (60.0% and 27.8%) and relatively few quartz (< 5%). On the contrary, the *SM* soils at Depths 5.5, 6.5, and 7.5 m consist of less illite and chlorite (< 22% and < 14%) but with much more quartz (43~52%). Although three *SM* soils have similar proportions of mineral composition, the sandy soil (*SM*1) at Depth 4.5 m has relatively more slate, siltstone, sandstone and feldspar than those of the soils (*SM*2) at Depths 6.5 m and 7.5 m. The sandy soils at Depths 6.5 m and 7.5 m are alike in the mineral composition and can probably be treated as the same soil.

Table 3 Mineral compositions of in-situ ungrouted soils

Mineral or rock	Compositions at various depths (%)			
	4.5 m	5.5 m	6.5 m	7.5 m
Quartz	4.85	43.2	51.8	47.5
Polycrystalline quartz	0.0441	0.6	0	0
Feldspar	0.0	1.8	0.551	0.461
Chert	0.0441	0	0	0
Sandstone	0.221	3.6	0	0
Siltstone	0.353	9.6	2.2	2.3
Slate	0.485	18.0	13.8	14.3
Weathered slate	6.0	0.6	1.65	0.46
Hematite	0.0	0.3	0	0
Chlorite	27.8	9.48	14.6	13.8
Illite	60.0	12.5	15.4	21.2

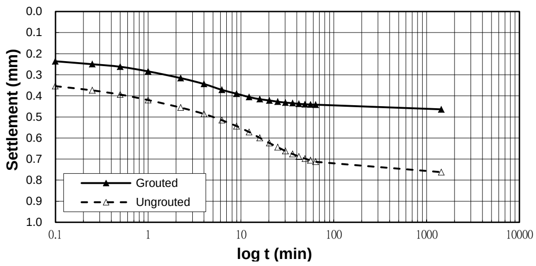


Fig. 7 Settlement vs. time curves for clays
(Depth = 4.5 m, Δ_h = 0.5 m, σ_N = 50 kPa)

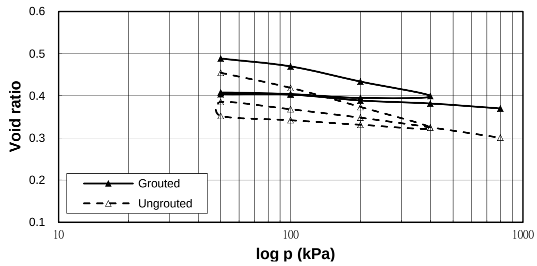


Fig. 8 Void ratio vs. load curves for clays
(Depth = 4.5 m, Δ_h = 0.5 m)

5.2 Geotechnical properties of grouted soils

5.2.1 Compressibility of clayey soils

The improvement of compressibility of grouted soil was examined for the clayey samples collected at the depth (D) of 4.5 m and a distance to grout hole (Δ_h) of 0.5 m. Fig. 7 shows typical settlement vs. time curves for the grouted and ungrouted clayey soils surcharged by a normal pressure (σ_N) of 50 kPa. As seen, the grouted soil exhibits lesser degrees of compressibility (initial compression, consolidation settlement, and secondary compression) than those of the ungrouted soil. Fig. 8 shows void ratio vs. load relationships of the grouted and ungrouted soils. It appears that the grouting decreased the compression and swelling potentials of the clayey soil. A compression index (C_c) of 0.118 and a swelling index (C_s) of 0.022 were computed for the grouted soil as compared with those (C_c = 0.158 and C_s = 0.050) for the ungrouted soil. Accordingly, the grouting had reduced the compression potential by 25% and the swelling potential by 55% for the clayey soils.

Yang *et al*. (2009) had reported a loose Ottawa sand (D_r = 30%) injected with a cement-based grout of 20% by volume would decrease its compressibility (C_c: 0.12 → 0.03) by about 75%, which is greater than the results obtained in this study for in-situ grouted clayey soils.

Fig. 9 shows the coefficients of consolidation (C_v) for the grouted and ungrouted soils at Depth 4.5 m and a distance to grout hole of 0.5 m. Both coefficients fall in an approximately range of the value for the low plasticity clays (Duncan and Buchignani 1976). The grouted soil, however, appears to provide a slightly higher C_v than that of the ungrouted soil. The C_v values for the grouted and ungrouted soils are noticed not to vary significantly with the surcharge loading.

The improvement on compression characteristics of in-situ clayey soils appears to be influenced by the type of grout and the injection mechanism associated with the location where the samples were collected. Since fracturing mechanism was found at the excavation depth of 4.5 m by the injection of GCB grout (gel time < 1 min), the improvement on compressibility of the in-situ clayey soils would be attributed to the compression and reinforcement of ground by the grouting.

5.2.2 Hydraulic conductivity of clayey soils

Fig. 10 shows relationships of permeability vs. loading for the grouted and ungrouted soils obtained at Depth 4.5 m and a distance to grout hole of 0.5 m. We notice that hydraulic conductivities of the grouted and ungrouted soils were similar and did not vary significantly with the magnitude of surcharge loading.

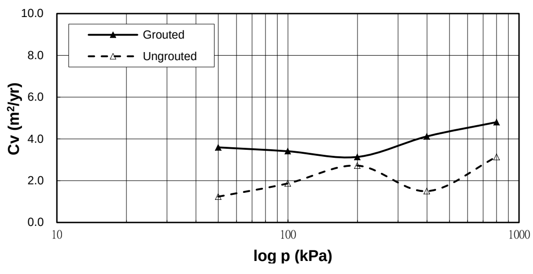


Fig. 9 Consolidation coefficient vs. load curves for clays (Depth = 4.5 m, Δ_h = 0.5 m)

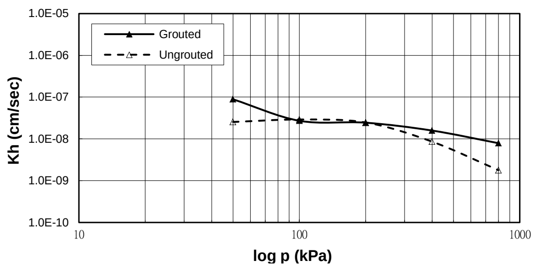


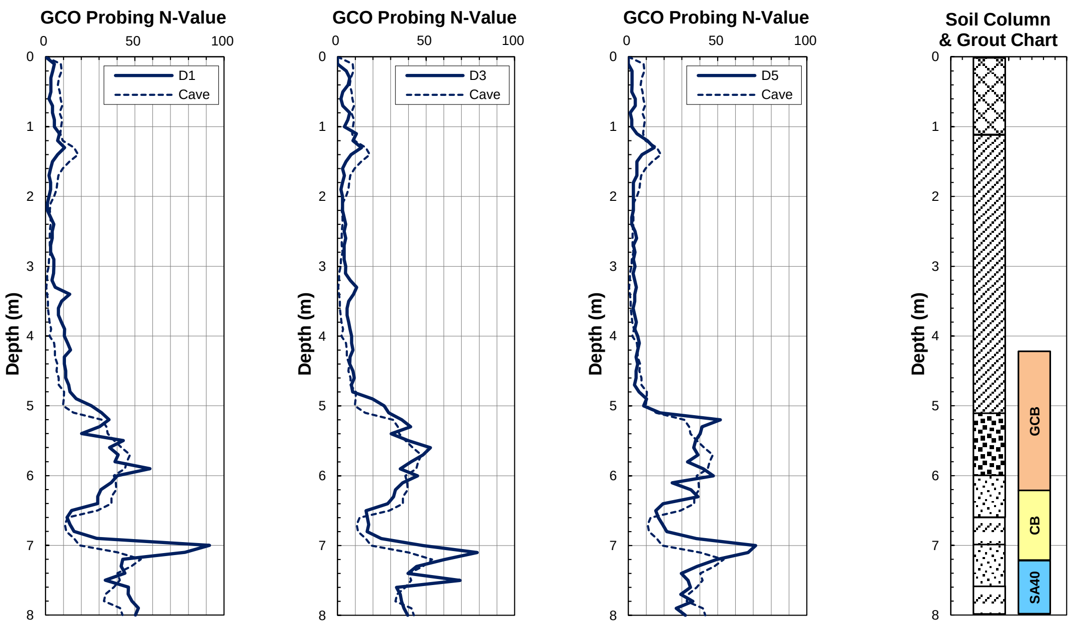
Fig. 10 Hydraulic conductivity vs. load curves for clays (Depth = 4.5 m, Δ_h = 0.5m)

Very limited data on the hydraulic conductivity of grouted clays is available in the literature. Most of the studies are related to sandy soils, and results often show obvious reductions in the hydraulic conductivity for the soils (Krizek and Helal 1992, Anagnostopoulos *et al*. 2011, Yasuhara *et al*. 2012), which is not the case as discussed herein.

5.2.3 Shear resistance of soils

Improvements on the shear resistance of grouted soils were evaluated through field GCO probing (GEO 1996) and laboratory direct shear testing. Fig. 11 shows results of the GCO probing blow counts (*N*-value; per 10 cm penetration) vs. depth, performed at *D*1, *D*3, and *D*5 with distances to the grout hole *G* of about 0.5 m, 2 m, and 4 m, respectively (see Fig. 1 for locations). The GCO probing was carried out approximately 1~2 days after the grouting operations. As can be seen, improvements in the GCO blow count were not significant. In the depth range of 3~5.2 m where the *CL* soil was encountered, increases in GCO penetration resistance were noticed in soils with a lateral distance to grout hole (Δ_h) of 0.5 m. However, the improvement appeared to be diminished with the distance Δ_h.

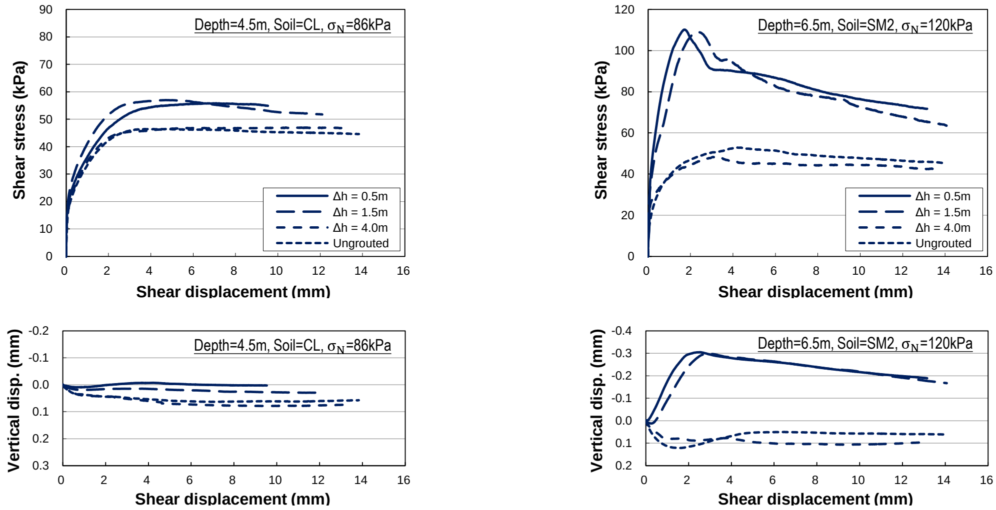
Some increases in GCO penetration resistance were also noticed in the depth range of 6.6~7.2 m with *ML* and *SM*2 soils, for various Δ_h. These soils were injected with a *CB* grout, a grout with a gel time of 16~24 hrs (Table 1). The on-site grout mapping had revealed a fracturing-permeation mechanism at this depth interval (Fig. 2(c)). It is therefore suspected that the grout and the grouting mechanism could have contributed to the increase in GCO penetration resistance of these soils.



(a) Probing at D1 (~0.5 m to grout hole)
(b) Probing at D3 (~2 m to grout hole)
(c) Probing at D5 (~4 m to grout hole)
(d) Soil column & grout chart

Note: C_{ave} is the average of C_1 and C_3 located at ungrouted ground; D-series are located in the grouted area.

Fig. 11 Results of GCO probing in the study area



(a) Soil *CL* (Depth = 4.5 m; σ_N = 86 kPa) (b) Soil *SM2* (Depth = 6.5 m; σ_N = 120 kPa)

Fig. 12 Typical results of direst shear testing for grouted and ungrouted soils at Depths 4.5 m and 6.5 m

Direct shear testing was carried out on the undisturbed tube samples collected on site. Fig. 12 shows typical results of the testing for the grouted and ungrouted soils at Depths 4.5 m and 6.5 m. For the *CL* soils at Depth 4.5 m, the results generally indicate "no peaking" in the stress displacement curves, and a contractive type of shear deformations. The grouting would slightly increase the shear strength of the clay. For the *SM*2 soils at Depth 6.5 m, a "no peaking" stress displacement relationship and a contractive type of deformation were also found for the ungrouted soil, or the grouted soil with a lateral distance of 4m to the grout hole. For grouted *SM2* samples closer to the grout hole (Δ_h= 0.5 m & 1.5 m), however, a "peaking" in stress and a dilation in deformation were obvious, indicating influence from the grouting.

Figs. 13-14 summarize friction angles and cohesions obtained from direct shear testing for the grouted and ungrouted samples collected at various depths and distances to the grout hole. As shown in Figs. 13(a)-14(a), the friction angle appeared unaffected by the grouting, while the cohesion of the grouted *CL* soils would be approximately 10~15 kPa more than that of the ungrouted soils at Depth 4.5 m.

For the *SM* soils at Depths 5.5, 6.5, and 7.5 m, improvements in the cohesion were more pronounced than for the friction angle. The cohesion of the sandy soils could be increased by 10~90 kPa within a lateral distance of about 3 m to the grout hole. The friction angle of the sandy soils, however, showed no consistent trend but varied with depth and distance to grout hole. Two reasons may have contributed to this situation: (1) the sandy soils are structurally more fragile and easily disturbed than the clayey soils during sampling and preparation for testing; and (2) fracture grouting generally produces a less uniform improvement of ground than for the permeation grouting.

The increase in cohesion of in-situ soils appeared to be influenced by the grouts adopted and injection mechanisms introduced by the field grouting. The improved cohesions as found in this study are generally less than those stated in the literature, which were mainly based on laboratory model grouting and testing without properly considering the influence of injection mechanism in the field (Krizek *et al*. 1992, Hsiao 1996, Maalej *et al*. 2007).

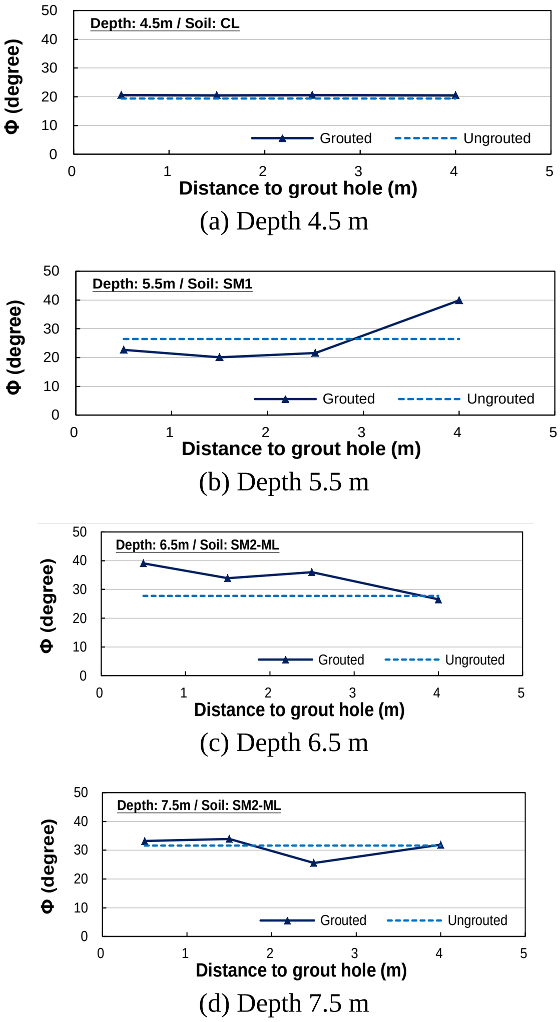


Fig. 13 Friction angle vs. lateral distance to grout hole for soils at various depths

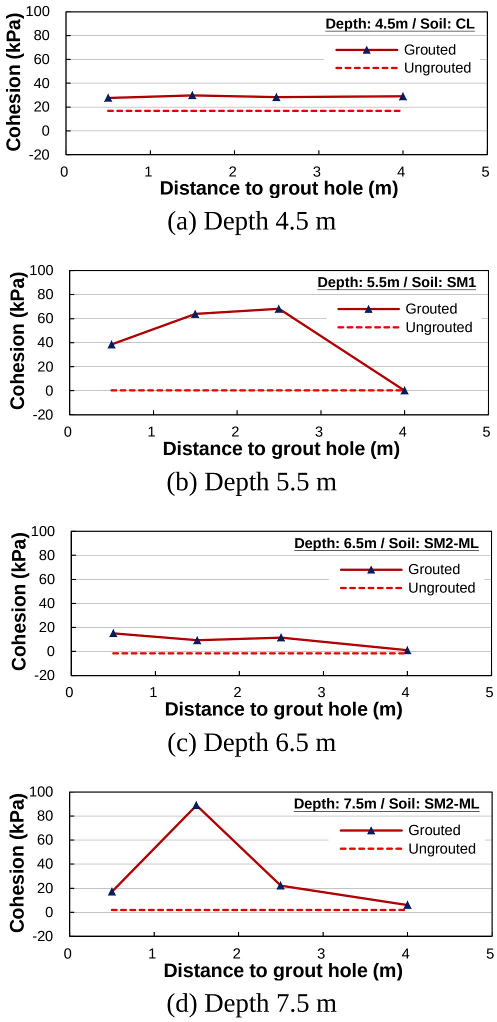
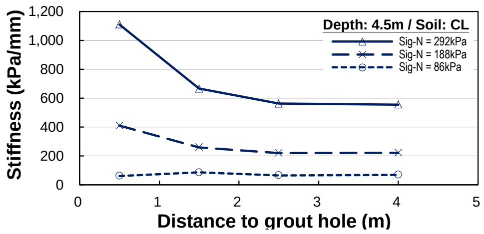


Fig. 14 Cohesion vs. lateral distance to grout hole for soils at various depths

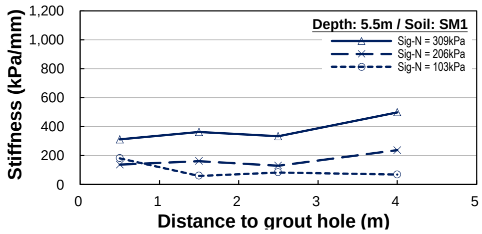
Fig. 15 shows the measured mean stiffness (E_{50}) of soils obtained from direct shear testing for the grouted and ungrouted samples collected at various depths and distances to the grout hole. For the *CL* soils at Depth 4.5 m, the grouting appears to have improved the stiffness of the soil. The degree of improvement decreases with the increase of distance to grout hole, but increases with the confining pressure. For the *SM* soils at Depths 5.5, 6.5, and 7.5 m, no consistent tend in the measured stiffness can be concluded with respect to the lateral distance to grout hole, a similar situation as discussed above for the friction angle. Both above stated reasons can be used to explain the situation observed herein for the measured mean stiffness.

5.2.4 Liquefaction resistance of sandy soils

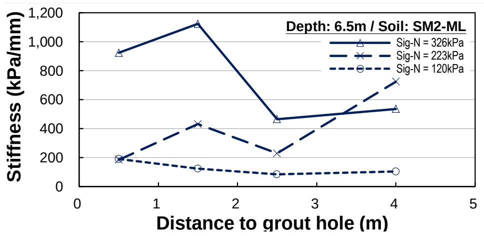
Grouting has been commonly adopted to improve the liquefaction resistance of sandy soils in seismic susceptible areas. A cyclic triaxial apparatus was employed in this study for evaluation of



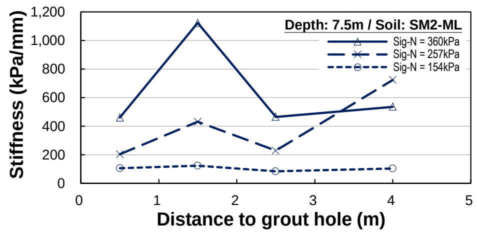
(a) Depth 4.5 m



(b) Depth 5.5 m



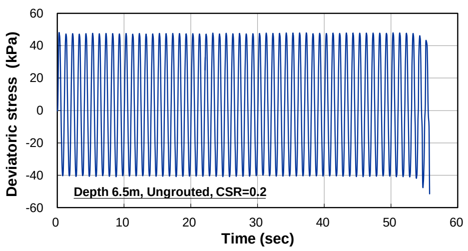
(c) Depth 6.5 m



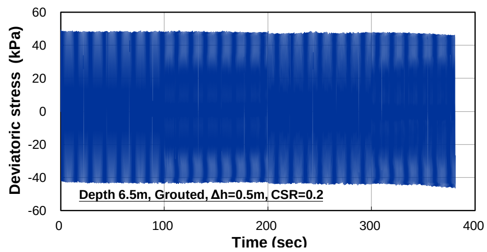
(d) Depth 7.5 m

Fig. 15 Stiffness vs. lateral distance to grout hole for soils at various depths

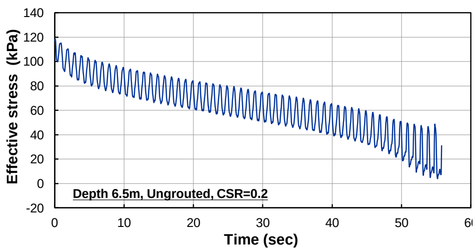
liquefaction potential of the grouted and ungrouted sandy soils at the site. Fig. 16 illustrates typical results of the cyclic triaxial testing for the grouted and ungrouted *SM*2 sands at the depth of 6.5 m. As shown in Fig. 16(a)-(d), a true liquefaction was found for the ungrouted sandy soil in which a dramatic increase in the cyclic strain was started at a cyclic number (N_l) of about 54, and



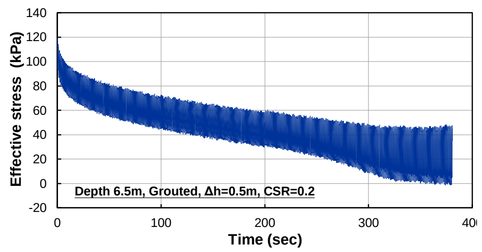
(a) Cyclic stress ~ time history
(ungrouted soil at Depth 6.5 m)



(e) Cyclic stress ~ time history
(grouted soil at Depth 6.5 m & Δ_h = 0.5 m)

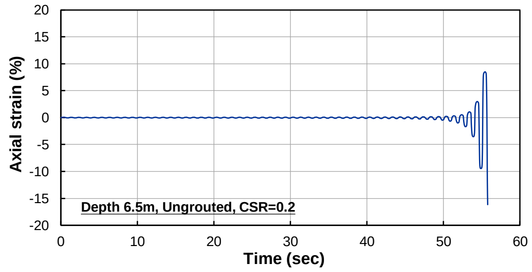


(b) Effective stress ~ time history
(ungrouted soil at depth 6.5 m)

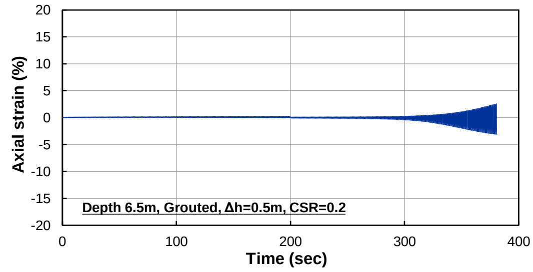


(f) Effective stress ~ time history
(grouted soil at Depth 6.5 m & Δ_h = 0.5 m)

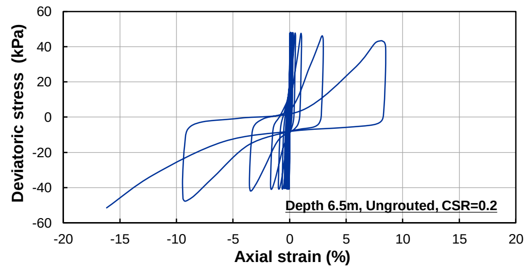
Fig. 16 Typical results of cyclic triaxial testing for soils at Depth 6.5 m



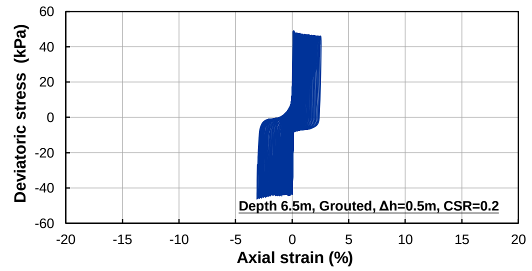
(c) Cyclic strain ~ time history
(ungrouted soil at Depth 6.5 m)



(g) Cyclic strain ~ time history
(grouted soil at Depth 6.5 m & Δ_h = 0.5 m)



(d) Strain hysteresis
(ungrouted soil at Depth 6.5 m)

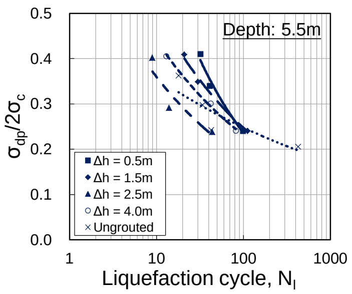


(h) Strain hysteresis
(grouted soil at Depth 6.5 m & Δ_h = 0.5 m)

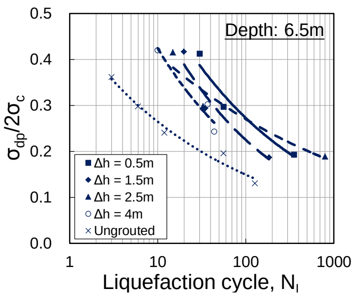
Fig. 16 Continued

subsequently the sample failed due to significant straining.

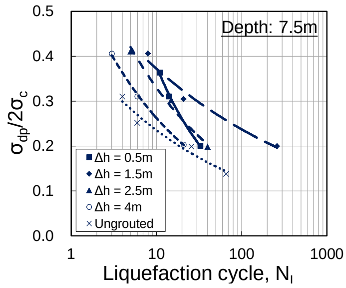
On the contrary, a cyclic mobility behavior was observed for the grouted sandy soil with the same test condition, as shown in Figs. 16(e)-(h). The test specimen was basically intact with no appreciable deformation until a number of cycles of about 300. After that, the specimen weakened and cyclic strains gradually increased. The test sample did not fail due to straining as the test ceased. For the purposes of failure definition, a 5% double-amplitude cyclic strain (Ishihara 1996) is generally acceptable in testing for determining the cycle required for "liquefaction" (N_l) for the soils with a cyclic mobility behavior. Accordingly, N_l would be approximately equal to 350 for the grouted soil discussed herein.



(a) Depth = 5.5 m; Soil = *SM*1

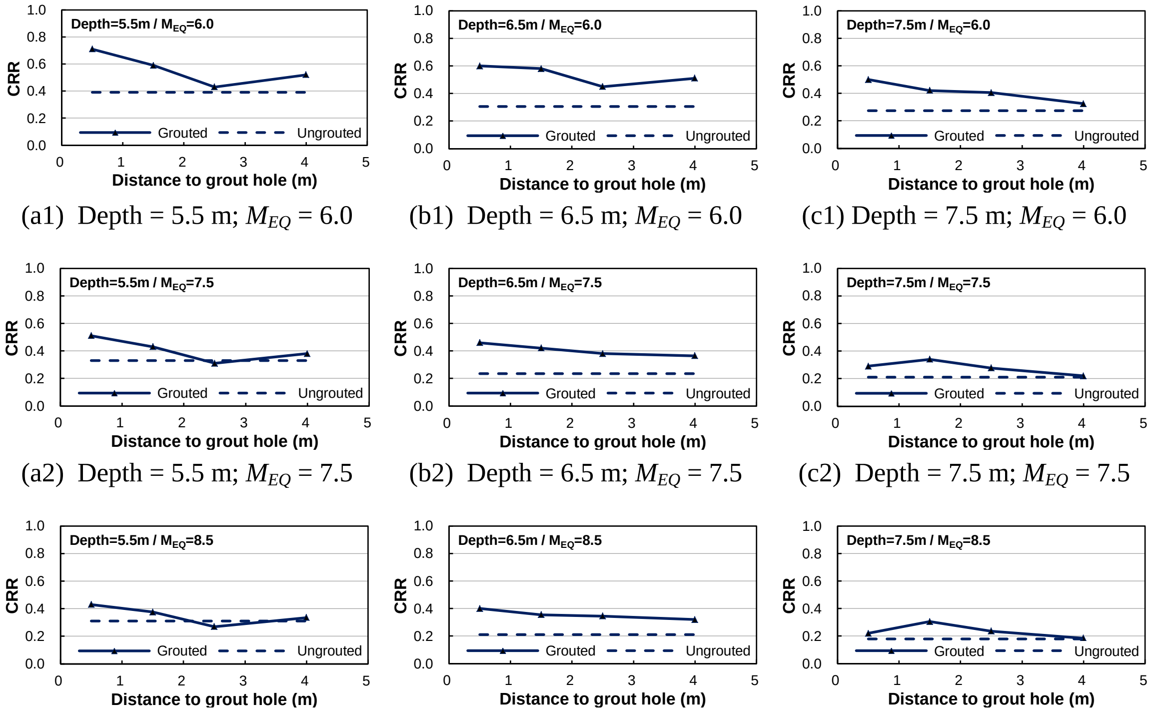


(b) Depth = 6.5 m; Soil = *SM*2



(c) Depth = 7.5 m; Soil = *SM*2

Fig. 17 Cyclic stress ratio ($\sigma_{dp}/2\sigma_c$) vs. liquefaction cycle (N_l) for grouted and ungrouted soils



(a1) Depth = 5.5 m; M_{EQ} = 6.0 (b1) Depth = 6.5 m; M_{EQ} = 6.0 (c1) Depth = 7.5 m; M_{EQ} = 6.0

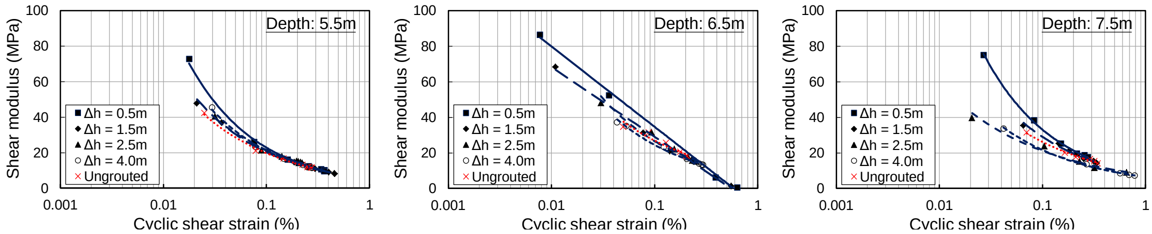
(a2) Depth = 5.5 m; M_{EQ} = 7.5 (b2) Depth = 6.5 m; M_{EQ} = 7.5 (c2) Depth = 7.5 m; M_{EQ} = 7.5

(a3) Depth = 5.5 m; M_{EQ} = 8.5 (b3) Depth = 6.5 m; M_{EQ} = 8.5 (c3) Depth = 7.5 m; M_{EQ} = 8.5

Fig. 18 Cyclic resistance ratio (*CRR*) vs. lateral distance to grout hole (Δ_h) for different earthquake magnitude (*M*) and soils at various depths

Results of cyclic triaxial testing for the grouted and ungrouted soils at various depths (5.5~7.5 m) and lateral distances (0.5~4 m) to grout hole are summarized in Fig. 17. The liquefaction resistance of sandy soils can be expressed in terms of the cyclic resistance ratio (*CRR*; $\sigma_{dv}/2\sigma_c$) for a given number of cycles causing liquefaction (N_l). In consideration of earthquake magnitudes (M_{EO}) of 6, 7.5, and 8.5, the equivalent numbers of cycles (N_{ea}) are 5.5, 15, and 26, respectively (Seed *et al*. 1985, Youd *et al*. 2001). The liquefaction resistance of the grouted and ungrouted soils for earthquake magnitudes of 6, 7.5, and 8.5 were then rearranged and compared in Fig. 18, for various depths and distances to grout hole.

Generally, liquefaction resistance of the soil (*SM*1) at Depth 5.5 m could be improved by the grouting within a lateral extent of about 2 m to the grout hole. Liquefaction resistance of the soils (*SM*2) at Depths 6.5 m and 7.5 m could also be improved with a wider extent of 3~4 m to the grout hole. For the soil at Depth 5.5 m, the average improvements of liquefaction resistance could reach about 40%, 25%, and 20%, for earthquake magnitudes of 6, 7.5, and 8.5, respectively. For the soil at Depth 6.5 m, the average improvements could reach about 70%, for all the earthquake magnitudes considered. For the soil at Depth 7.5 m, the average improvements were 50%, 30%, and 30%, for earthquake magnitudes of 6, 7.5, and 8.5, respectively. Generally, an average improvement of 40% in liquefaction resistance of soils could be reached by the grouting for an earthquake magnitude of 6, and an average improvement of 20% in liquefaction resistance of soils could be reached by the grouting for an earthquake magnitude of 7.5 or greater. The above results appear consistent with findings in the literature (Hsiao 1996, Uchida *et al*. 1996, Chang *et al*.



(a) Depth = 5.5 m; Soil = *SM*1 (b) Depth = 6.5 m; Soil = *SM*2 (c) Depth = 7.5 m; Soil = *SM*2

Fig. 19 Dynamic shear modulus (*G*) vs. cyclic shear strain (γ) for soils at various depths (*CSR* = 0.3)

2004).

Dynamic shear moduli of soils were determined based on hysteretic stress-strain relationships of cyclic loading at different strain amplitudes. Results of shear moduli at different cyclic strains are plotted in Fig. 19 for the grouted and ungrouted soils at various depths and distances to grout hole. Due to large strains by the cyclic triaxial testing, the bonding between grouts and soil grains would likely be altered, and differences in the modulus were found to be small for the grouted and ungrouted soils. This result appears consistent with the findings by Delfosse-Ribay *et al*. (2004).

6. Conclusions

This paper discusses the improvement of geotechnical properties of the in-situ soils by a field grouting. The geotechnical properties considered in this study include: compressibility, permeability, shear resistance, and liquefaction resistance. Both on-site and laboratory testing were conducted for the evaluation and major findings of this study are summarized as follows:

- The improvement of geotechnical properties of soils by grouting is affected by two major factors: the grout materials adopted, and the injection mechanism introduced by the grouting.
- Injection mechanisms of grouting in soils include: permeation, compaction, and fracturing.
- Permeation mechanism will occur in coarse-grained soils with fine grouts or solutions. The soil grain structure remains unchanged after grouting, and the degree of improvement depends primarily on the grout materials. The extent of improved zone is more uniformly distributed.
- Compaction and fracturing mechanisms will occur if the groutability requirements are not satisfied, or the injection pressure or rate is too high. Improvements of ground through these mechanisms are mainly due to the consolidation or densification of soils adjacent to the grouts and/or the reinforcement effect of the grouted fractures. The extent and degree of improvement by compaction or fracturing grouting are generally not uniformly distributed.
- Due to the grouts adopted and soils encountered, the injection mechanism of grouting in this study was primarily of fracturing type, which was confirmed by the on-site grout mapping. On-site mapping also indicated some permeation of grouts at the excavation depths of 6.5 m and 7.5 m where the *SM*2 soils were encountered and the CB and SA40 grouts were applied.
- The grouting has reduced the compressibility by about 25% and the swelling potential by 55% for the CL soil at Depth 4.5 m and a lateral distance to grout hole of 0.5 m.
- The hydraulic conductivity of the CL soil at Depth 4.5 m appeared not affected by the

grouting.

- On-site GCO probing has shown some improvements in the penetration blow counts at depth ranges of 3~5.2 m (CL soil; GCB grout), 6.6~7.2 m (ML/SM2 soils; CB grout), and 7.6~8 m (ML soil; SA40 grout).
- For CL soil at Depth 4.5 m, the friction angle appeared not affected by the grouting, while the cohesion was approximately increased by 10~15 kPa.
- For the *SM* soils at Depths 5.5, 6.5, and 7.5 m, the effect of grouting on the friction angle was not certain, while the effect on the cohesion of soils could be increased by 10~90 kPa.
- Liquefaction resistance of the *SM*1 soils at Depth 5.5 m could be improved by the grouting within a lateral extent of 2 m to the grout hole. The extents of improvement for the *SM*2 soils at Depths 6.5 and 7.5 m were about 3~4 m to the grout hole.
- An average improvement of 40% in liquefaction resistance of soils could be reached by the grouting for an earthquake magnitude of 6. An average improvement of 20% in liquefaction resistance could be obtained by the grouting for an earthquake magnitude of 7.5 or greater.
- Computed dynamic shear moduli by cyclic triaxial testing for the grouted and ungrouted soils were similar due to large-strain characteristics of the testing.

Acknowledgments

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