Investigation on mechanism of grouting and engineering characteristics of in-situ grouted soils

Recherche sur le mécanisme de jointoyer et de machiner des caractéristiques des sols scellés au ciment in-situ

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ABSTRACT
Soil grouting is generally adopted to enhance engineering properties of soil deposits by introducing cohesive agents into the ground. The effectiveness of the technique, however, is influenced by various factors, and relies upon a substantial portion of on-site experience and engineering judgment. Major uncertainties in grouting appear to be the actual mechanism of injection and the performance of the grouted ground. Although several attempts have been made in the past, results of related studies on the injection mechanism are generally not conclusive and the improvement of grouting on engineering properties of soils can not be practically quantified. The study herein discusses an examination on the injection mechanism, through an in-situ grouting program and the subsequent excavation. Various types of grout were introduced into different soil layers of the ground, with controlled injection pressure and volume. Field mapping and laboratory testing of the grouted soils were carried out. A numerical simulation of the grouting process was also performed and discussed for clarification of the observed mechanism.

RÉSUMÉ
Le jointoiement de sol est généralement adopté pour augmenter des propriétés de technologie des dépôts de sol en présentant les agents cohésifs dans le sol. L'efficacité de la technique, cependant, est influencée par de divers facteurs, et compte sur une partie substantielle de jugement sur place d'expérience et de technologie. Les incertitudes importantes dans le jointoiement semblent être le mécanisme réel de l'injection et l'exécution de la terre scellée au ciment. Bien que plusieurs tentatives aient été faites dans le passé, les résultats des études relatives sur le mécanisme d'injection ne sont généralement pas concluants et l'amélioration du jointoiement sur des propriétés de technologie des sols ne peut pas être pratiquement mesurée. L'étude ci-dessus discute un examen sur le mécanisme d'injection, par un programme de jointoiement in-situ et l'excavation suivante. De divers types de coulis ont été présentés dans différentes couches de sol de la terre, avec de la pression d'injection et le volume commandés. Le champ traçant et essai en laboratoire des sols scellés au ciment ont été effectués. Une simulation numérique du processus de jointoiement a été également effectuée et discutée pour la clarification du mécanisme observé.

Keywords : soil grouting, injection mechanism, ground improvement, field mapping, laboratory testing

1 INTRODUCTION

Soil grouting with low injection pressures (<2000kPa) has commonly been adopted for treatments of grounds that are susceptible to liquefaction or settlement in Taiwan (TCRI 1984, Woo et al. 1999, TFEC 2000). The effectiveness of the technique, however, is influenced by various factors and relies upon a substantial portion of on-site experience and engineering judgment. Major uncertainties in grouting appear to be the actual mechanism of injection and the performance of the grouted ground. Although several attempts have been made in the past, results of related studies on the injection mechanism are generally not conclusive and the improvement of grouting on engineering properties of soils can not be practically quantified (Hou & Bai 1991, Uchida et al. 1996, Chang et al. 2004). In accordance, the aim of current study is to re-examine the mechanism and the improvement of grouting, through in-situ mapping, laboratory testing and numerical simulations.

2 FIELD GROUTING

2.1 *Site condition*

An in-situ grouting program was carried out in an alluvial deposit (Tzuo-swei river alluvial fan) in mid-west Taiwan. The deposit consists of material layers, including (in descending order): silty clay (CL; $\omega_n \fallingdotseq$ 30%; GL:-1~-5m), silty sand (SM; D_{50}=0.23mm; FC=22%; GL:-5~-6m), silty sand to sandy silt (SM-ML; D_{50}=0.13mm; FC=30-35%; GL:-6~-9.5m), and silty clay (CL; $\omega_n \fallingdotseq$25%; GL:-9.5~-15m). According to studies in the alluvial fan after the 1999 Chi-chi Earthquake (Ueng et al. 2000, Chang, et al. 2003), the sandy layers located at the depth of 5~9.5m are susceptible to liquefaction during strong earthquakes.

Table 1. Grouts adopted in the study and mechanism of grouting.

Type	Component*	Gel time	Grouted depth	Soil type	Injection** mechanism
GCB	S+C+B+W	1min	4m~5m	CL	**F**, C
			5m~6m	SM	**F**, C
CB	C+B+W	16hrs	6m~7m	SM-ML	**P**, C, F
SA-40	S+A+W	4min	7m~8m	SM-ML	**P**, F

Note: * S=sodium silicate; C=cement; B=bentonite; W=water; A=SA-40.
** F=hydro-fracturing; P=permeation; C=compaction.

2.2 *Grouts*

Soil grouting was conducted at the depth of 4~8m, using *tube a manchette* (TAM) method (DGE 2004, Hausmann 1990). As shown in Table 1, three types of grout were adopted in this study, in which GCB and CB grouts are of suspension type and SA-40 is a solution grout. The GCB grout with the shortest gel

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time was first injected into the ground to form a cap for the following lower injection layers (CB and SA-40 grouts). The grouts were injected with a controlled pressure ranged from 150kPa to 500kPa, and a rate of 5~10 l/min.

2.3 *Field observations*

After grouting, four stages of excavation were carried out in the depth interval from 4m to 8m. Figure 1 shows a schematic illustration of an excavation stage with locations of sampling and testing during the stages. Several observations or testing were conducted, including: surface heaving/lateral deformation of the ground, penetration resistance through GCO (Geotechnical Control Office, Hong Kong) probing, grout mapping, etc.

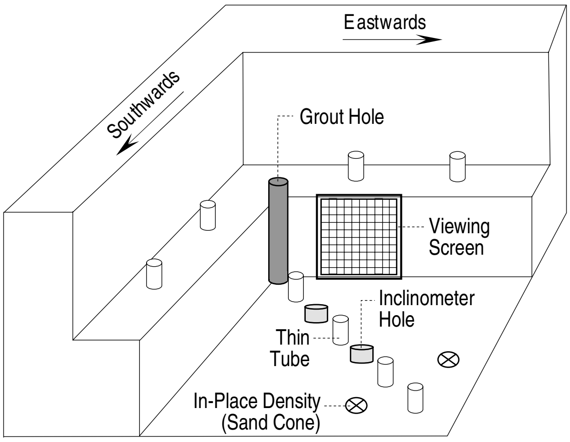


Figure 1. Illustration of excavated site & sampling locations.

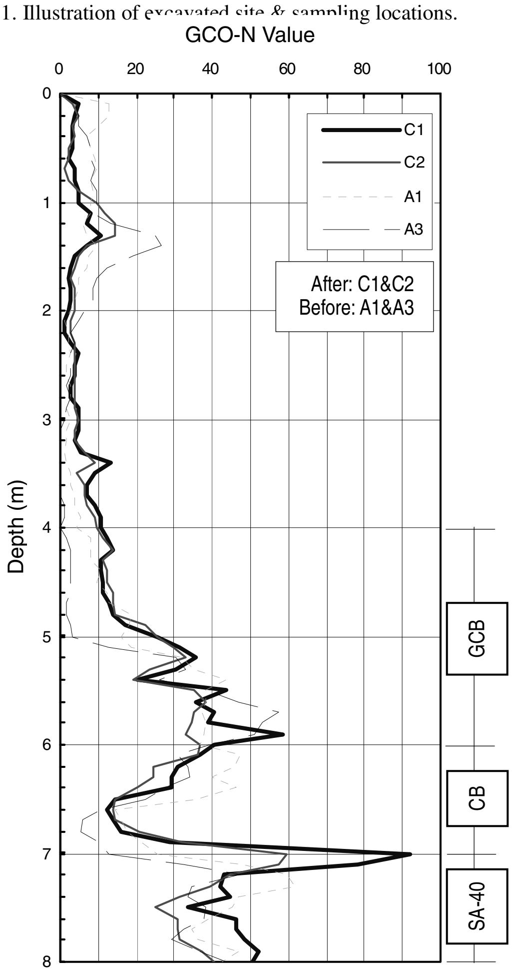


Figure 2. Improved depth intervals of penetration resistance.

The GCO probe is a device commonly used in Hong Kong for assessing the depth and degree of compaction of buried fill (GEO 1996). The device comprises of a sectional rod fitted at the end with a cone and is driven into the ground by a constant mass falling through a fixed distance. The number of blows per 100mm penetration (GCO-N value) is normally reported for the tests. Correlations between GCO probing resistance and the SPT-N value are scarce. Limited information showed the SPT value of 10 (blows per 300mm) could be equivalent to a GCO value of 5-20 (blows per 100mm) (Phillipson 1989). Another study showed the ratios of GCO-N value to SPT-N value were about 0-3 and 1-8, for clayey and sandy soils, respectively (Mao et al. 2004).

Figure 2 indicates variations in the GCO-N value of the ground in a lateral distance of 1.5m to the grout hole before and (1-day) after the grouting. It shows that the GCB grout approximately increased the penetration resistance of silty clay (CL; GL:-3~-5m) by 0~10 blows, while the increase in the silty sand to sandy silt (SM-ML; GL:-6.5~-8m) by CB and SA-40 grouts was varied and could up to about 40 blows.

Figure 3 shows observed injection mechanisms of the grouts. The injection mechanism appeared to be influenced by a number of factors, including soil type, in-situ stress state, grout type, injection pressure and rate, gel time of grout, etc. For GCB grout (suspension type, shorter gel time) in silty clay (CL, low permeability), hydro-fracturing mechanism was dominant. For SA-40 grout (solution type, longer gel time) in silty sand or sandy silt (SM-ML, high permeability), permeation mechanism was apparent.

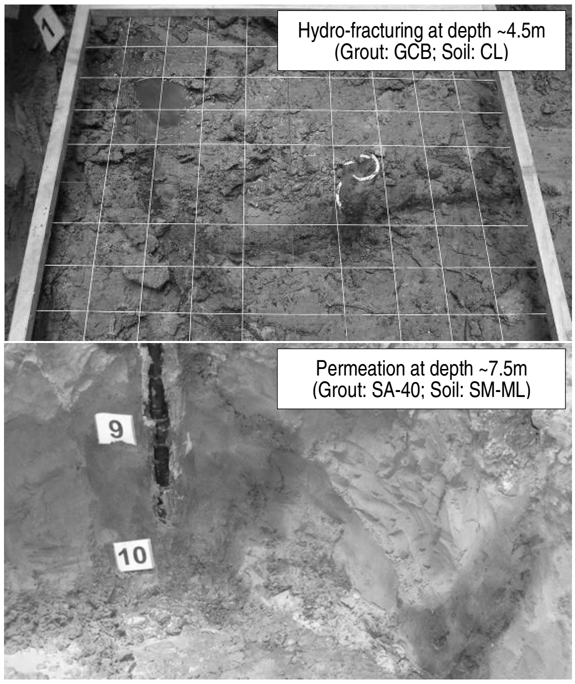


Figure 3. Observed mechanisms of grout injection.

3 LABORATORY TESTING

Thin tube samples were collected on site. Laboratory testing was conducted to evaluate the engineering properties of soils before and after the grouting. Results of the evaluation are discussed as follows.

3.1 *Compressibility*

For silty clay samples located at a depth of 4.5m and a lateral distance of 0.5m from the grout hole, Figure 4 shows results of consolidation tests, indicating the compressibility of grouted soil on the virgin curve is about 30% less than that for the un-grouted soil; and about 70% less for soils on the rebound curves.

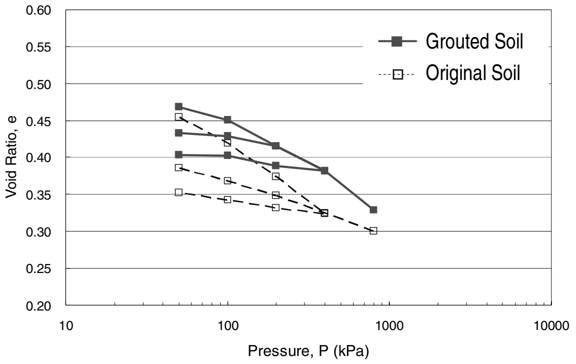


Figure 4. Compressibility of grouted silty clay (GL:-4.5m).

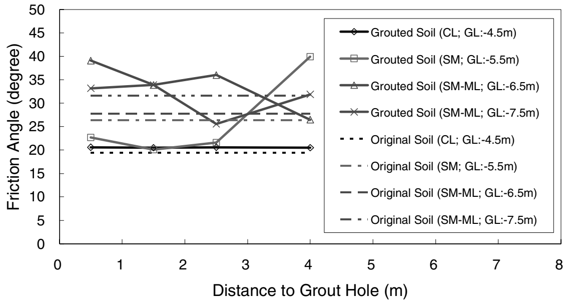


Figure 5. Friction angles of grouted soils.

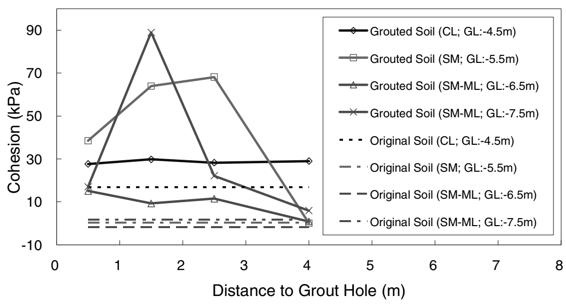


Figure 6. Cohesions of grouted soils.

3.2 *Shearing resistance*

A series of direct shear tests (shearing speed = 1.5mm/min) was performed to evaluate the improvement in shear resistance of the grouted soils, and results shown in Figures 5 & 6. For silty clay located at a depth of 4.5m, the friction angles for both grouted and un-grouted soils are about the same. The cohesions of the grouted soil, however, are approximately 15kPa higher than the un-grouted one.

For sandy layers located at depths of 5.5m~7.5m, the friction angle and the cohesion of grouted soils are generally increased by about 3° and 40kPa, respectively. Increases in the shear strength characteristics (ϕ:-6°~+14°; c:0~90kPa) of the sandy layers, however, vary with distance where sample was taken. It appeared that sample disturbance and in-homogeneity of the sandy materials could have been the major causes for the variation in the test results.

3.3 *Liquefaction resistance*

Liquefaction resistances of grouted and original sandy soils at depths of 5.5m (SM; FC<25%) and 7.5m (SM-ML; FC>30%) were evaluated through cyclic triaxial apparatus, and results shown in Figures 7 & 8. The liquefaction resistance was determined in terms of the cyclic resistance ratio (CRR; $\sigma'_{dp}/2\sigma'_c$) required to cause initial liquefaction in a certain number of stress cycles equivalent to a given magnitude of an earthquake (Ishihara 1993, Youd et al. 2001).

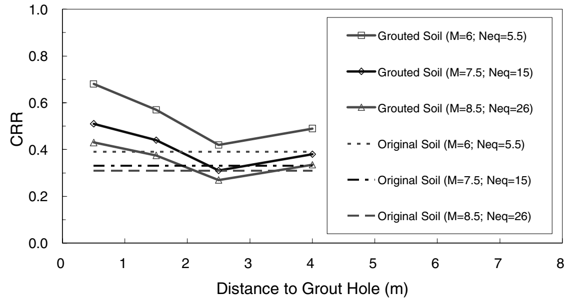


Figure 7. Liquefaction resistance of grouted sand at GL:-5.5m.

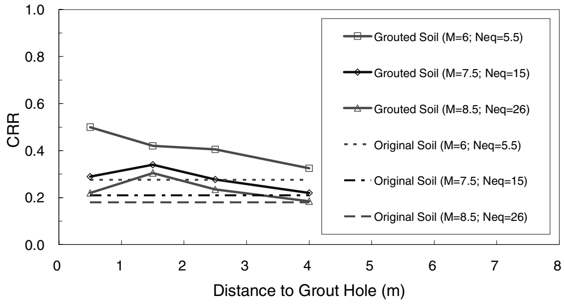


Figure 8. Liquefaction resistance of grouted sand at GL:-5.5m.

Results indicate a general increase in liquefaction resistance of 50-80% for M=6 EQs and 20-50% for M≧7.5 EQs, for the grouted sandy soils located within a 2m distance to the grout hole. For the sandy soils located further away (>2m) the grout hole, improvements in the liquefaction resistance would be less than about 30%. It is also noted that some grouted soils located at 3m to grout hole show slightly less liquefaction resistances than the original soils. It was suspected that sample disturbance and material in-homogeneity could have accounted for the results.

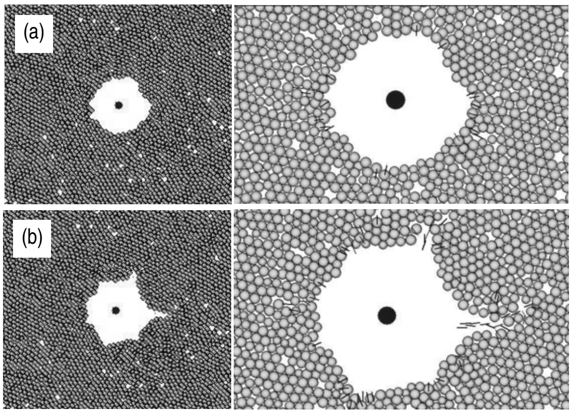
4 NUMERICAL SIMULATION

A distinct element code (PFC^{2D}; Itasca 1999) was employed in the numerical simulation of the grouting process. The code is capable of describing movements of particles in an assembly under the influence of pore fluid (grout) pressures and particle contact forces. The simulation procedure includes computation of resultant forces on each of the particles. The forces would cause movements of the particles and the rearrangement of the particle assembly. New pore fluid pressures and particle contact forces would then be updated as a result of the changes in pore distribution and particle contact conditions. The iterative process continues for a number of time steps defined by the user or until certain equilibrium criterion has been reached.

The aim of the simulation herein was to provide a quick check the responses of numerical analysis that would generally comply with the on-site observations in the grouting process. Without detailed investigations, a preliminary set of parameters (c=1e6Pa, *fric*=0.6, k_n=1e9N/m, and k_s=1e9N/m) was adopted for the analysis. Further studies are required in the future as to the suitability of parameters to be used in the analysis and the quantitative interpretations of the analysis.

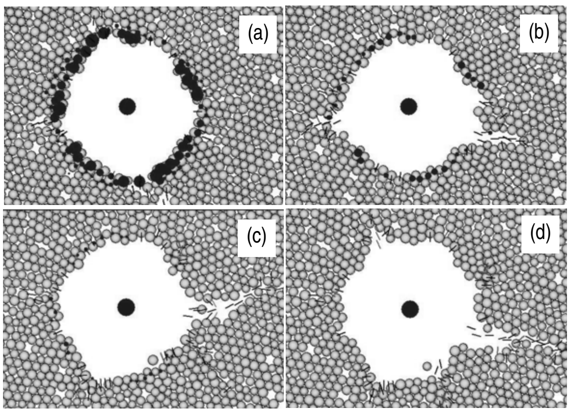
Figure 9 indicates the effect of grout pressure on the injection mechanism of grout into the ground, under a same set of material parameters (e.g., K_h=1e-7 m/s) and computation steps (TS=500). As shown, the increase in grout pressure generally would increase the tendency of hydro-fracturing in the soil surrounding the grout hole. It was also noticed that the increase in computation time would increase the tendency of hydro-fracturing as well.

Figure 10 shows the effect of permeability of the ground on the injection mechanism. With the same injection pressure (P= 1e8 Pa) and at the same time step (TS=600), the ground with a higher permeability (i.e., more sandy soils) will show a more permeation type of injection mechanism. However, the ground with a lower permeability (i.e., more clayey soils), the hydro-fracturing type of mechanism would be apparent.



Note: P = (a) 0.90e8 Pa; (b) 1.05e8 Pa.

Figure 9. Effect of injection pressure.



Note: K_h (m/s) = (a) 50e-6; (b) 10e-6; (c) 5e-6; (d) 1e-6.

Figure 10. Effect of hydraulic conductivity.

5 CONCLUSIONS

The study herein discusses an examination on the injection mechanism and the improvement of soil grouting, through a field mapping, laboratory testing, and numerical simulations. Major findings of the study are summarized in the following:

On-site mapping showed hydro-fracturing would be more significant in clayey soils when injected with suspension grouts. For solution grouts in sandy layers, permeation mechanism would be dominant.

Preliminary numerical analyses conducted herein appeared to be capable of simulating the field observations. The lower grouting pressure or more permeable of the ground, the more permeation mechanism will be observed. The higher grouting pressure or less permeable of the ground, the more hydro-fracturing mechanism will be expected.

The grouted clayey soils in the neighborhood (≦0.5m) of grout hole showed a 30% reduction in compressibility on the virgin compression line, and a 70% reduction on the rebound (or recompression) curves.

Soil grouting would increase the cohesion by about 15kPa and 40kPa, for the silty clay (CL; GL:-4.5m) and the silty sand-sandy silt (SM-ML; GL:5.5m~7.5m), respectively. The friction angle of the silty clay experienced no changes after the grouting.

An increase of 50~80% in the liquefaction resistance could be expected for the grouted sandy soils located within 2m distance to the grout hole during M≦6.0 earthquakes. For M≧7.5 earthquakes or for soils located at a distance of more than 2m to the grout hole, the increase in the liquefaction resistance would be 0~50%.

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