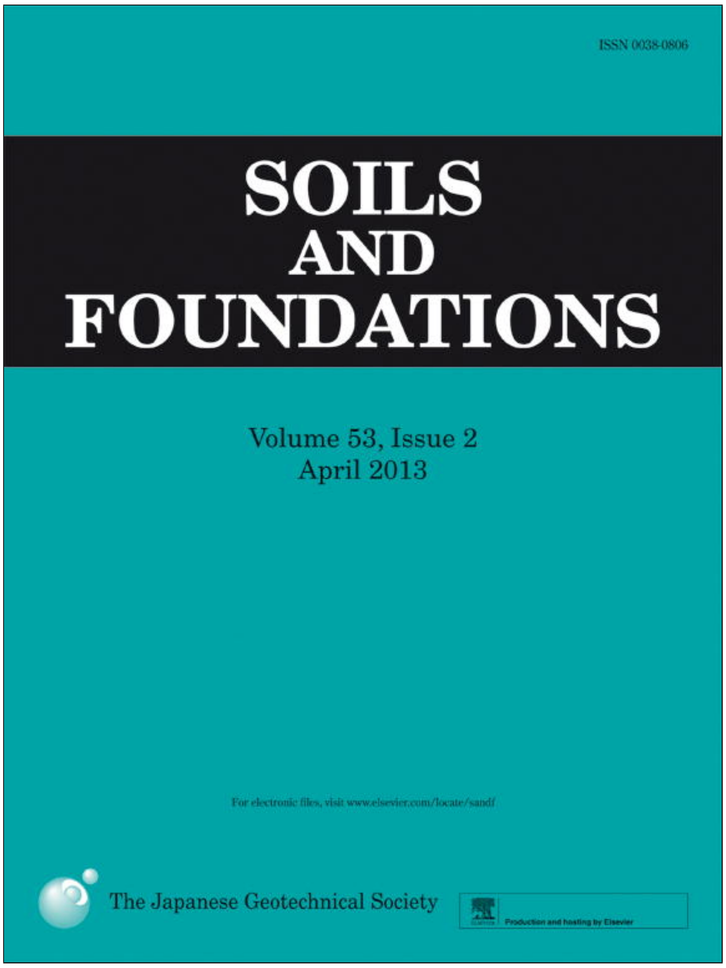
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An innovative Geonail system for soft ground stabilization

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Abstract

Soil nails are widely used in stabilizing and retaining the ground during constructions, with the high yield steel bar the most commonly used soil nail material at present. The classical method of soil nail construction is, however, not effective in soft clay as it is difficult to establish a good bond strength and global soil improvement. An innovative soil nail installation method has been developed for the Airport link in Australia, which combines the applications of fracture grouting techniques and composite GFRP soil nails to stabilize the ground soil as well as to compensate for the settlement of ground. Extensive laboratory and in-situ tests have been carried out to verify the mass soil properties methods and the performance of the Geonail system for the local and global stabilization of the soft ground.

Keywords: Geonail; Soft ground; Stabilization; Fracture grouting

1. Introduction

The concept of soil nailing is to increase the global shear strength of the soil by closely spaced nails. A soil nail is a passive in-situ reinforcement, which responds to movement by mobilizing the nail force. It has been used extensively to retain excavations and stabilize slopes in various countries. The soil nails are usually installed across or behind the potential failure surface to stabilize the global mass. The stabilization forces are provided between the cement grout surface of the soil nails and the soil. In general, the nailing system has the advantages of lower cost, quicker construction procedure and less impact on adjacent ground when compared to other traditional stabilization methods like retaining walls. Currently, the high yield steel bar is the most commonly used material because it is relatively cheap and is simple to install in most cases.

In general, soil nails are not adopted in soft clay because of various uncertainties in their performance in this soil condition. Firstly, the low cohesion of the soft clay equates to small bond strength between the ground and the soil nail, which results in a low pull-out resistance of the soil nails. Meanwhile, it is usually not cost effective to improve the pull-out resistance by increasing the length of the soil nails due to the limitations at the construction site. Secondly, the strength of the soft clay and the friction between the ground and soil nails are small so that it is difficult to stabilize a global soil mass. The function of the soil nail would thus be highly localized unless the soil nails are densely installed, which would be expensive and time-consuming (and a longer construction time means more ground settlement). Thirdly, the displacement can be great because of the creep of the soft clay, which can lead to the

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instability of the soil nail system. Soil nails must be installed as soon as possible after exposure of the ground surface in the soft ground. In addition, the underground water can cause seepage and piping during the excavation of the soil, which can influence the strength of soil nail greatly if global stabilization is not carried out as soon as possible. Since there are so many problems to overcome, soil nails are generally not recommended in soft clay, though there have been some reports for their successful use.

Fracture grouting has been successfully adopted in different projects for settlement compensation for more than 30 years. Based on the recent research on displacement grouting by Soga et al. (2001) and Au et al. (2007), subsurface cavity expansion in clay induced by fracture grouting is not only able to generate upward displacement of clay, but there is also an increase in the effective stress leading to consolidation which results in settlement compensation and shear strength enhancement in normally consolidated clay. As a result, fracture grouting can be a cost effective technique for ground improvement in soft ground condition.

Fracture grouting combined with the use of composite Glass Fibre Reinforced Polymer (GFRP) soil nails for maintaining the tunneling face stability was first adopted in the Airport Link tunnel project in Brisbane Australia. The Airport Link is a tunneled motorway grade road in the northern suburbs of Brisbane, Australia. It connects the Brisbane central business district and the Clem Jones Tunnel to the East–West Arterial Road which leads to the Brisbane Airport. It was built in conjunction with the Windsor to Kedron section of the Northern Bus way in approximately the same corridor. The Airport Link and bus way project involved 15 km of tunneling including the road (5.7 km of twin tunnels), bus way tunnels and the connecting ramps as well as 25 bridges and result in over 7 km of new road. The tunnel section under the QR railway embankment at Toombul was installed by the box jacking technique. The construction of the launch box requires 85,000 m^3 of soil to be excavated under the railway embankment, and had posed a major challenge to the construction with the poor soil condition. The ground is mostly soft clay, which is very prone to subsidence. As a requirement of this project, the railway had to be maintained in operation during the whole construction to ensure the transportation. It was extremely important to control the settlement of the embankment, which greatly increased the difficulty of the construction. The alluvial soil comprises of layers of soft, firm to stiff sandy clay, and the geological profile and the property of the ground soil are shown in Table 1 and Fig. 1. The SPT value of the soft clay is less than 10 while the friction ratio from CPT test ranges between 2% and 4% with a mean pore pressure of about 0.12 MPa. For the firm clay, the SPT value of the soft clay is about 20 while the friction ratio from CPT test ranges between 4% and 8% with a mean pore pressure of about 0.38 MPa. The soil properties for the soft clay in Table 1 have clearly illustrated the difficulty in maintaining stability and reducing settlement during construction.

Table 1
Averaged properties of ground soil (Young's modulus determined from dilatometer, vane shear and CPT tests) (value in bracket represent the range of the values).

Soil	Shear strength (kPa)	Young's modulus (MPa)	Water content (%)	Plasticity index
Soft clay	20 (15–27)	6 (4–8.5)	57 (52–59)	25 (23–27)
Firm clay	37 (30–47)	20 (17–29)	46 (44–49)	45 (41–49)

In order to minimize the settlement, headwalls, canopy tubes, soil nails and sidewall steel tubes were constructed to retain the railway embankment for the excavation of box jacking shafts. The stabilization measures and the finite element mesh used for the modeling of the jacking process are shown in Fig. 2.

Ground improvement works underneath the QR railway embankment were required to facilitate the box jacking stages. Large volumes of grout were injected into the ground to stabilize it prior to excavation. In order to optimize the ground improvement, Geonails were introduced into the project. The geonails used in this project, which are shown in Fig. 3, are essentially a combination of soil nails and Tube a Manchette (TAM) grouting. For the main face nails, it was formed from GFRP rods placed around the circumference of the TAM sleeve. The GFRP rods were developed and tested for this use so that they could be easily broken out as part of the excavation by mechanical plant. By adopting Geonails, the physical properties and pull-out strength of the soil nails in soft clay were improved by consolidation through the introduction of grout finger networks. The soil nails provided positive reinforcement for the excavation face slope and improved the soil strengths due to consolidation effects and grout replacement. The combination of soil improvement and soil reinforcement had maintained the stability of the face and limited the settlements of the railway and enabled the installation of the jacked box beneath the embankment.

During box jacking, the mixed soft/stiff clay excavation face was maintained at approximately 60° to the horizontal (similar to the jacked box leading edge angle) by GFRP fracture grouted soil nails. No soil nail was proposed in the Siltstone strata with an in-situ strength greater than 1 MPa due to its coherent stability and strength. The soil nails were installed on a dense grid (i.e., at close spacing) through the gaps between the headwall piles during excavation of the jacking pit and extend across the entire length of the box jack. Sufficient anchoring force at the western end of the soil nails were developed by keying into a grouted groundmass on the western side of the railway. The GFRP soil nails provided sufficient tensile strength

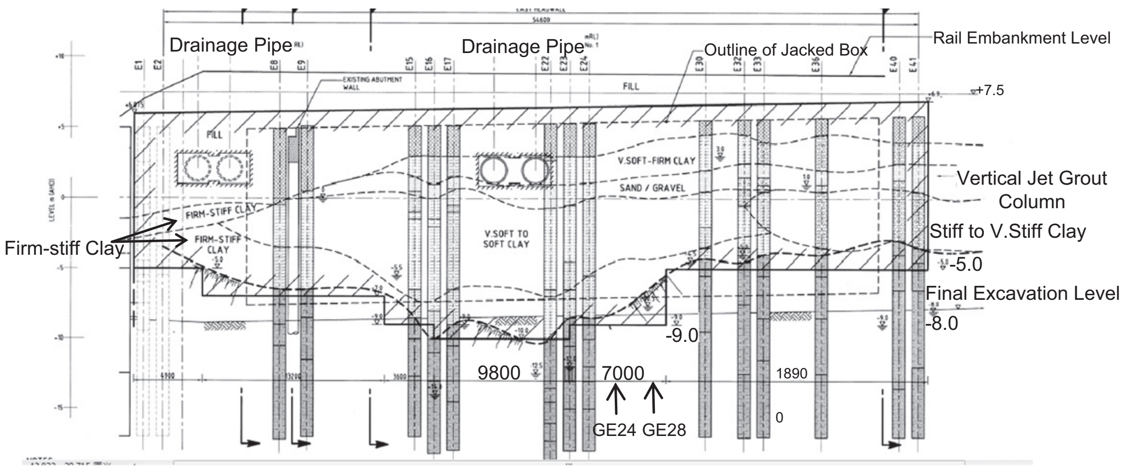

Fig. 1. Geological profile of ground soil.

without obstructing excavation (cut during excavation). The soil nails were post-grouted at high pressure (11–15 bars) by using 'Tube A Manchette' (TAM) injection pipes to hydro-fracture the initial grout column and the surrounding soil and to produce a wider grout column with 'fingers' of intruded grout material which extend into the surrounding soil. The grouted nail columns and grouted soil fractures provided increased pull-out resistance, resulting in improved soil strength properties, and the injected grout also improved the surrounding soil mass properties through the grout fingers and soil consolidation. TAM grouting could be repeated to achieve the required level of grout penetration if necessary. The use of fracture grouting by TAM is an innovation in soil nail construction, and was adopted for the first time in this project.

Since excavation slope stability was the key factor to the integrity of the railway embankment, a detailed survey of the installed nails were carried out prior to TAM grouting. Additional nails were installed prior to jacking to compensate for any out-of-tolerance areas and the facility to install further nails during the box drive was provided for an emergency situation.

Usually, no pressure was applied during the grouting of conventional soil nail (gravity flow of grout) because the application of pressure with a classical soil nail system was difficult to carry out. In this project, the grout was applied under high pressure and consolidation occurred in the soft clay. This was considered useful for the Geonail system, and provided a great amount of data which allowed the behavior and performance of the Geonail system to be explored. Large scale site trials were carried out adjacent to the location of the railway embankment, which included horizontal CPTs and pull-out tests carried out in both jacking zones to verify that the design mass soil improvement properties (using the ground settlement as the control to obtain the design values) and the pull-out strength (of the soil nails) that had been achieved prior to box jacking.

As the ground improvement technique was highly governed by the soil consolidation, it was important that the grouting pressure (during the fracture grouting process) was sufficient to ensure an effective stress state that would lead to/ promote consolidation. Prior to estimating the improved soil properties, a review of the grouting test results was required to ensure that the results satisfied the specified minimum fracture grouting pressure requirement.

The present project received the Fleming Award in 2011 and the Ground Engineering Award in Technical Excellence in 2012 for satisfactory performance under such difficult conditions. Herein, the Geonail system is introduced and the verification of this system as accomplished through trial tests and in-situ tests is explained in detail. Geonails serve dual purposes: stabilization and ground improvement. The present paper will concentrate more on the Geonail itself, and briefly refer to the ground improvement test results. The investigation into the design and evaluation of the ground improvement scheme and the design principle will be covered in a separate paper by Cheng et al. (In preparation).

2. Brief design theory

There have been various theories and techniques for ground improvement over the years. A detailed discussion of the historical development of the various techniques has been given by Kitazume and Okamura (2010). The typical ground improvement empirical relationship based on the volume replacement ratio was adopted for the estimation of the shear strength of the grouted soil. However, the ground improvement mechanism of fracture grouting was subsequently introduced, which made the original empirical relationship superfluous as it could not be directly applied to the fracture grouting techniques used in the ground improvement works. Accordingly, a new design method with a theoretical background developed on the basis of the works by Au et al. (2003, 2007) and Soga et al. (2004) was adopted for the design of the ground treatment works in the present project. Based on the revised design method, it was possible to estimate the 'improvement ratio' due to consolidation and overall mass improvement.

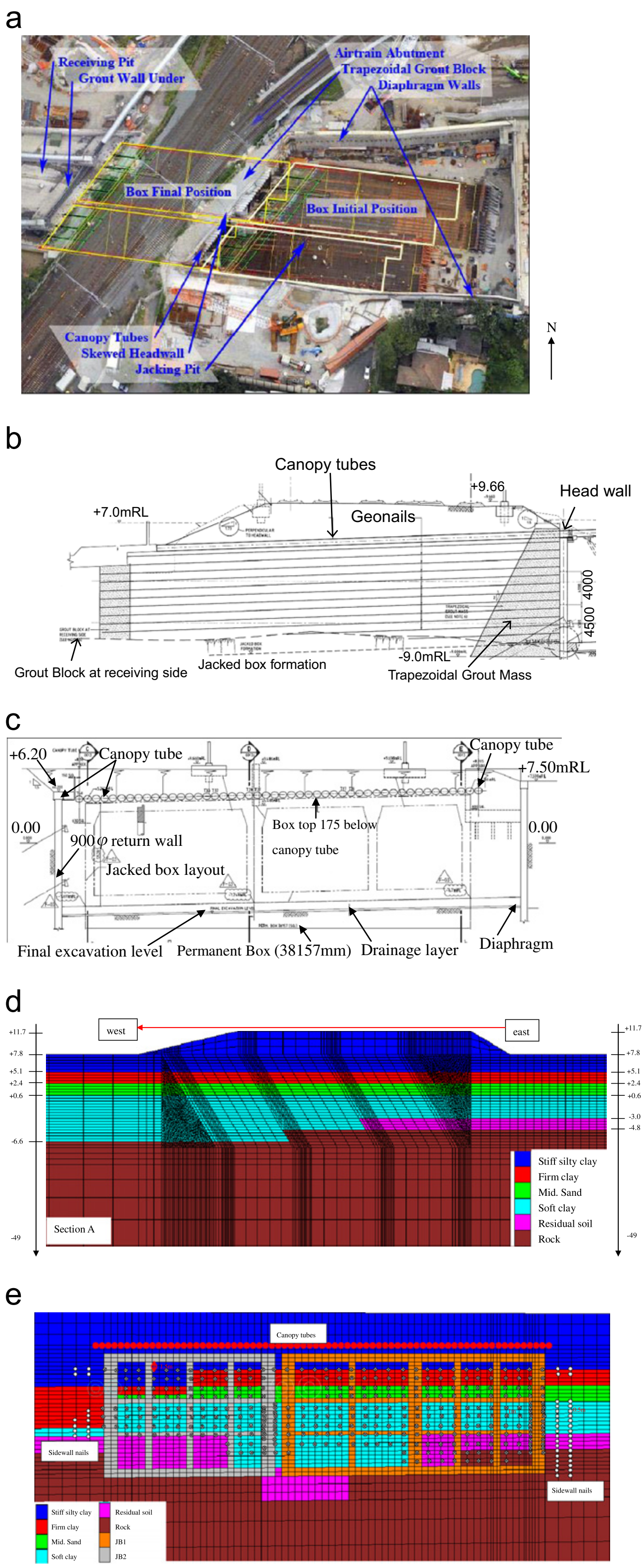

Fig. 2. Stabilization measures of the embankment and the finite element mesh for the analysis of the jacking process. (a) Plan view of the site, (b) section view along the tunnel (from left (west) to right (east) as shown in Fig. 2a) showing the use of canopy tubes and soil nails for stabilization of tunnel excavation, (c) the use of canopy tubes as the supports and the presence of foundations above the jacked tunnels (elevation at east headwall), (d) finite element mesh for the numerical analysis of the tunnel jacking (with more than 1 million elements and soil nails), (e) finite element mesh at a section of the jacked tunnel.

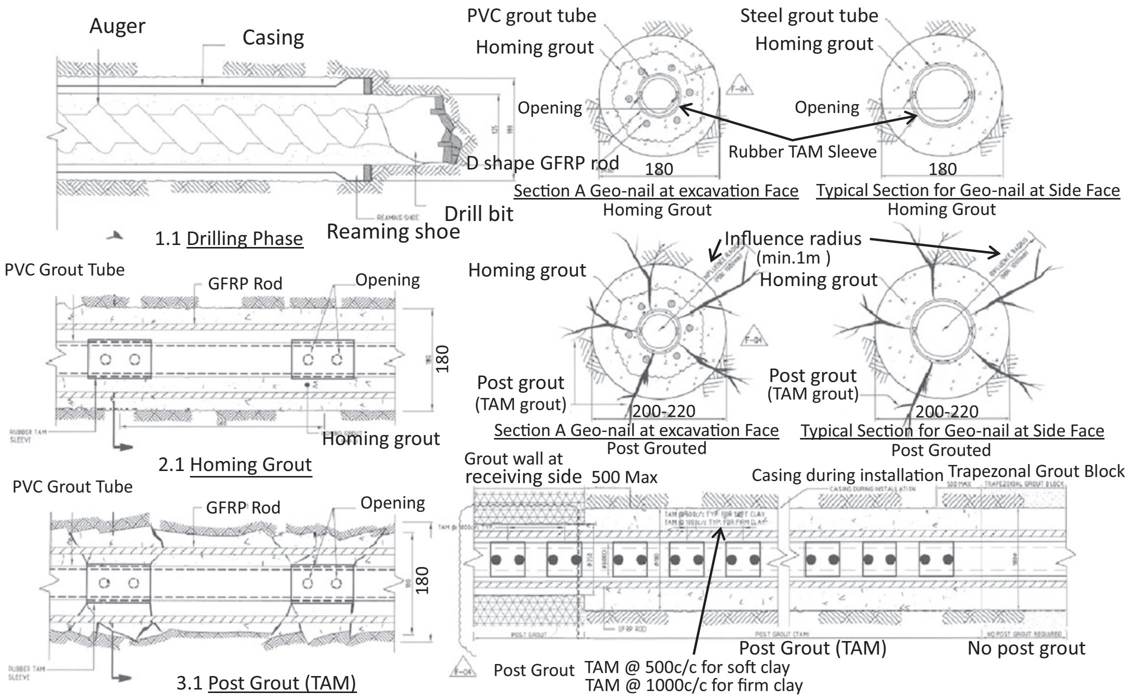

Fig. 3. Details of geonail used for stabilization (pvc grout tube 60 mm outside diameter and 50 mm inside diameter, steel grout tube 114.3 mm outer diameter and 111.7 inner diameter, unconfined compressive strength of homing grout = 5 MPa).

The following are considered to be the two main contributions to the improvement in the soil properties:

A. Replacement grout from fracture
B. Strength gained due to soil consolidation (dissipation of excess pore water pressure in soil induced by grout injection)

The combination of the above two effects was considered as the mass improvement from the current Geonail grouting method. Wang et al. (2010) have carried out model tests on steel nails in cohesive soil under earthquake conditions and have discussed the stabilization effect of the nails when subject to an earthquake. Voottipruex et al. (2011) have studied the behavior of stiffened deep cement mixing piles under full scale loading with a three-dimensional finite element modeling. While the present work is similar in some respects to these earlier studies, this work also considers fracture/compensation grouting and grout fingers. Falk (1998) and Essler et al. (2000) suggested that with an injected volume of 5%–10% grout volume, an improvement a factor of 3–4 in stiffness could be expected. This enhancement in shear strength due to replacement grout fracture can be utilized in design. Since the fracture grout was not fully mixed with the soil, the traditional grout area replacement empirical relationship could not be directly adopted when estimating the mass improvement. In order to quantify the degree of improvement due to consolidation, the following terms were introduced:

$$I_a = \mathrm{Cu}_s/\mathrm{Cu}_0, \quad I_E = \mathrm{Eu}_s/\mathrm{Eu}_0 \tag{1}$$

where

Cu_s the improved undrained shear strength of the grouted soil due to consolidation improvement only
Cu_0 the undrained shear strength of the original soil
Eu_s the improved stiffness of the grouted soil due to consolidation improvement only
Eu_0 the stiffness of the original soil
I_α Shear Strength Improvement Ratio (average shear strength enhancement ratio) due to consolidation only.
I_E Stiffness Improvement Ratio due to consolidation only.

An overall improvement ratio with respect to strength and stiffness improvement, resulting from both the soil consolidation and material enhancement of the fracture grout, can be expressed as follows:

$$\mathrm{IRMCu} = \mathrm{Cu}_T/\mathrm{Cu}_0 \tag{2}$$

$$\mathrm{IRMEu} = \mathrm{Eu}_T/\mathrm{Eu}_0 \tag{3}$$

IRMCu is the mass shear strength improvement ratio, which is defined as the ratio of the mass improved shear strength (Cu_T) of the soil to the original soil strength (Cu_0).

IRMEu is the mass stiffness improvement ratio, which is defined as the ratio of the mass improved stiffness (Eu_T) of the soil to the original soil stiffness (Eu_s). For the original soil properties, they are derived either from CPT, VST or DMT tests. Since it is not possible to form a uniform grouted mass in soil improvement, the mass properties after soil improvement are determined by the plate load test, which tests a larger amount of soil mass. The mass strength improvement is then determined in accordance with the mass properties and the original soil properties.

The typical equation for the area replacement ratio and the strength of grouted soil relationship in ground improvement design were modified as below for the present project:

$$Cu_T = \alpha\xi_C Cu_g + (1-\alpha)I_\alpha Cu_0 \tag{4}$$

$$Eu_T = \alpha\xi_E Eu_g + (1-\alpha)I_\alpha Eu_0 \tag{5}$$

Cu_g and Eu_g were the undrained shear strength and stiffness of the injected grout. ξ_c and ξ_E were the grouting network strength contribution factors for the mass improved undrained shear strength and stiffness when the volume replaced by the injected grout was not large. This parameter was determined by the site trial. Though the factors vary with the grout deformation pattern (thickness, orientation) and initial stress condition, a range between $I_\alpha Cu_0/Cu_g$ or $I_E Eu_0/Eu_g < \xi_c$ or $\xi_E < 1$. α Was the percentage of volume replacement of the injected grout. It should be noted that the nail homing grout columns were ignored in the strength calculations, and only grout fracture was considered in the design.

For the different effects on shear strength enhancement, reference was made to Liao et al. (2006) and Au et al. (2003, 2007). Their study showed that shear strength is enhanced due to consolidation decreases in the radial distance from the grout injection point. Apart from the consolidation effect, the grout fractured along the nail also played an important role in soil improvement and naturally, a higher volume replacement ratio was obtained at a closer distance from the grout injection point and thus the nail body. Based on this theoretical basis, a method for estimating the mass improved soil strength was proposed and discussed in the project.

The fracture grouted Geonail improves the undrained shear strength in two ways:

(A) By consolidation alone through the grouting pressure in zone where the influence of the grout finger network was absent, C_{u2}
(B) By combination of the consolidation and the grout finger network with maximum shear strength C_{u1} which decreased to C_{u2} as indicated in Fig. 4.

The amount of excess pore water pressure generated and the degree of consolidation were highly governed by the grout pressure and waiting period between each injection. Apart from the strength enhancement contributed by the grout fracture, soil consolidation due to grout injection was another important factor to be considered. Subsurface

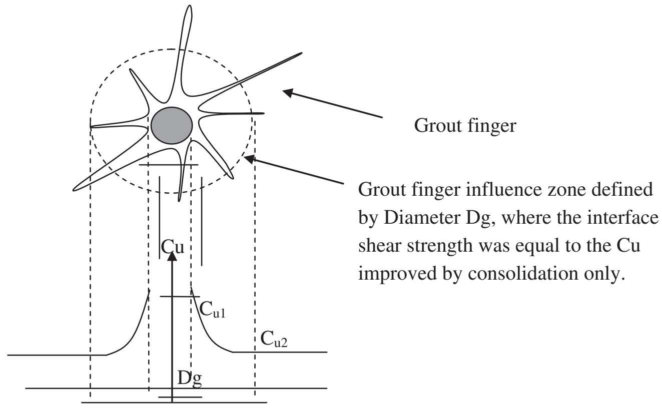

Fig. 4. Distribution of improved strength of grouted mass.

cavity expansion in clay induced by fracture grouting generated an upward displacement of clay and/or increase in effective stress leading to consolidation, resulting in settlement compensation and/or shear strength enhancement, respectively.

3. Site trial for verifying the design method

The large scale site trial tests included the CPTs, Dilatometer, Vane shear, Plate load test and Pull-out tests. The undrained shear strength, stiffness and pull-out strength in soft or firm clay were determined from these tests. Based on the results of the trial tests, the 'anticipated improvement ratio' due to 'consolidation only' were verified and modified.

After installation of the Geonails under the QR embankment and subsequent fracture grouting, extensive in-situ soil tests were undertaken, which included horizontal CPTs, hand vane shear tests and pull-out tests. These were carried out in both jacking zones to verify that the design mass soil improvement properties and pull-out resistance of the soil nails had been achieved prior to box jacking.

Trial nails (close to the QR embankment) were installed vertically on a grid pattern in the ground immediately adjacent to the railway embankment and in the area to be excavated for the jacking pit of the jacked box. Nails were fracture grouted in a number of patterns and grout volumes. Soil tests were carried out both before and after the nail installation and fracture grouting, to enable assessment of the soil improvement gained from fracture grouting. Pull-out tests were carried out on a limited number of nails to establish the design pull-out resistance values.

The trial area was located at the eastern side of the railway, approximately 25 m from the piled headwall, where soils representative of those beneath the embankment were expected. Approximately 3–4 m of fill had been placed on top of the existing ground to provide a working platform at a level of +7.5 m RL (RL is the reference level). The trial area was divided into three zones (Options) to assess the different grouting scenarios as shown in

Table 2
Design options for soil nail trials.

Trial option	Option 1	Option 2	Option 3
Longitudinal spacing of sleeve injection points	1 m c/c	1 m c/c	1 m c/c
No. of injection phases	Single	Double	Single
Fracture grout injection volume ratio	3%	(1st injection = 3%, 2nd injection = 3%) Total 6%	9%
Fracture grout volume (litres per grout sleeve) nail spacing 1.0 m	36	72	108
Fracture grout volume (litres per grout sleeve) nail spacing 0.5 m	18	36	54

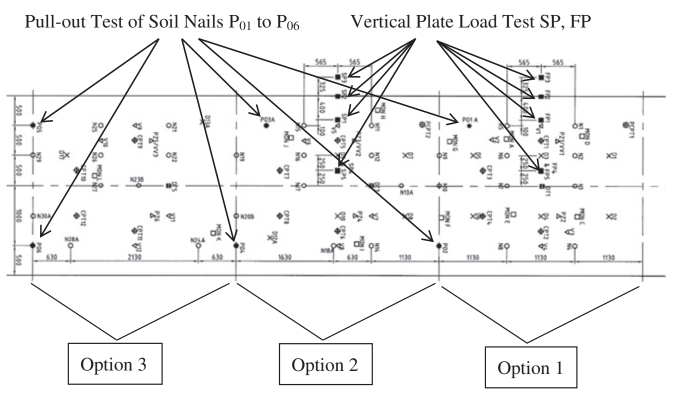

Fig. 5. Position of soil nails for pull-out tests for the three options out of the field nails.

Table 2. The purpose of the three options suggested by the contractor was to establish an economical grouting option with adequate global soil mass improvement. Six pull-out soil nails fell into the three option areas, and the details are given in Fig. 5. For most of the soil nails in the present project, a double injection was required to achieve the designed improvements in soil properties. In the present project, the second phase of grouting was carried out either 4 weeks after the first phase of grouting or when the equilibrium pore pressure P_{eq} as defined by Eq. (6) was reached.

$$P_{eq} = P_0 + 0.1(P_{max} - P_0) \quad (6)$$

where P_{eq}, P_{max} and P_0 are the equilibrium pore pressure, baseline piezometric pore pressure before fracture grouting and the maximum piezometric pressure during fracture grouting, respectively.

The soil nails N01 to N30 were arranged in a grid pattern, at 1.13 m centers along the north–south direction and at 0.5 or 1.0 m centers along the east–west direction. The trial area was divided into three zones to assess the different grouting scenarios, as shown in Table 2. The nails were generally 12 m long, extending through firm and then soft clay strata, and terminating 0.5 m into stiff clay at an approximate level of −7.0 mRL. Grouting commenced at the original ground level, which was approximately at +4.5 mRL. The nail sleeves were grouted at 1 m intervals over the remaining depth according to the grouting options. A more closely spaced arrangement of the drains than the production scheme was used in the trial to speed up the consolidation process. Six drains were provided per grouting option, at a longitudinal spacing of 1.13 m. Vane shear, CPT and dilatometer test were carried out both before and after grouting to provide a basis for comparison. Soil tests, monitoring instruments and drains were installed in a regular pattern around the nail locations so as to provide consistent results for the analysis.

The installation and grouting of the trial soil nails generally conformed to the proposed production scheme nails, except for the orientation of the nails. Soil nails N01 to N30 were installed by cased rotary auger, with a 180 mm diameter drill hole. Installation progressed from north to south in a hit and miss pattern, to avoid damaging newly formed nails. Because their casing was withdrawn, the nail bodies were grouted from the bottom up on the same day that the soil nails were drilled.

The TAM pipes were filled with water during sleeve grouting and the sleeves were cracked with water within 1 or 2 days. Generally, no problems were encountered with blocked sleeves except in a limited number of cases at the very bottom of the nail. The cement grout had a low viscosity with 1:1 mix of water and cement, and 2% bentonite was added to control bleeding.

3.1. CPT, hand vane shear and dilatometer tests

Before grouting, a CPT, a vane shear test and a dilatometer test were carried out in the trial zone to determine the undrained shear strength of the ground.

3.1.1. CPT

Before grouting, cone penetrometer tests were carried out from the current ground level of approximately +7.5 mRL prior to any nailing work. CPTs were successfully completed in the Options 1 & 2 areas but not in Option 3 area due to obstructions. After grouting and nail installation, twelve post-grouted CPTs were carried out between 15th and 19th February 2010 from the current ground level of approximately +7.5 mRL. The tests were carried out at 60 and 34 days after phase 1 and phase 2 grouting, respectively. The details of the two phases of injection are presented in Table 2, option 2.

3.1.2. Vane shear test

Before grouting, boreholes were logged during installation of the six piezometers. Vane Shear tests and samples were carried out at three of these exploratory holes, one within each grouting option area. Typically, vane shear tests were carried out at 0.5 m depth intervals from the original ground level. After grouting, twelve exploratory holes for vane shear tests and samples were prepared between the 8th and 12th February 2010, and four tests per grout option area were provided. The tests were carried out at 53 and 27 days after phase 1 and phase 2 grouting, respectively. The capacity of the vane shear gauge was limited to 58 kPa. For firm-stiff soils where the vane could not be sheared, the results were recorded as the gauge limits. In stiff soils where the vane could not be pushed into the soil, the result was assumed to be 100 kPa. These limits effectively meant that the recorded data was not sufficient to determine the higher degree of improvement of the clay.

3.1.3. Dilatometer

Before grouting, a standard Marchetti Dilatometer approximately 96 mm wide and 15 mm thick, with a button shaped membrane of diameter 60 mm was used, and one test was carried out within each grouting option area. After grouting, one exploratory hole per grouting option was carried out, with two pressuremeter tests in each of the soft and firm clay strata at each location. The tests were carried out between 19th and 23rd February 2010, which were 65 and 39 days after phase 1 and phase 2 grouting, respectively.

Results of the three pre-grouting tests are presented in Table 1, and the test results after grouting are given in Tables 3–5. In order to optimize the ground improvement design, option 1 with 1 m c/c spacing and option 2 with 0.5 m c/c spacing were selected for injection for firm clay and soft clay respectively. Based on the test results, option 2 was finally chosen for the present project because of the higher improvement in the global soil properties.

3.2. Plate load test

Plate load tests using a 300 × 360 mm rectangular plate was used to ascertain the improved global soil mass properties (the improved soil shear strength and Young's modulus of the soil), which were pertinent to the stability and settlement conditions that would be present during box jacking operations. The use of plate load test gave the average properties of greater amount of soil mass compared to the traditional laboratory soil sample. This was particularly important for grouted soil mass as the soil properties were not very uniform in general. The locations of the plate load tests are presented in Fig. 5.

Based on the trial nail assessment report, the unimproved, undrained shear strength of the firm clay and soft clay were taken as 37 and 20 kPa respectively from Table 1. The undrained shear strength for grouted consolidated mass was taken from the plate load test as it was difficult to achieve a uniform grouting and the results from the CPT tests were widely scattered. The target overall mass improved the undrained shear strength ratios of firm clay (trial Option 1 Area) and soft clay (trial Option 2 Area) by 1.43 and 2.45, respectively. The target grouted soil mass stiffnesses for the soft clay and firm clay were 20 and 22 MPa, respectively. The undrained shear strength and Young's modulus before and after fracture grouting are given in Table 6.

The plate load tests showed an overall mass improvement ratio of 2.5 for firm clay shear strength which was 74% better than the design values. However, the overall mass stiffness improvement ratio was only 1.02 for firm clay stiffness which was 7% less than the design values. This result was not surprising from an engineering perspective, as it is generally more difficult to improve the stiffness than the shear strength in firm clay. The plate load tests showed an overall mass improvement ratio of 2.3 for

Table 3
Average improved undrained shear strength for option 1.

Soil type (nail spacing)	Option 1-Improved undrained shear strength C_u (kPa)		
	From vane shear	From CPTs	From dilatometer
Soft clay (0.5 m c/c)	33	30	28
Soft clay(1.0 m c/c)	32	28	28
Firm clay(0.5 m c/c)	46	45	NA
Firm clay(1.0 m c/c)	39	43	NA

Table 4
Average improved undrained shear strength for option 2.

Soil type (nail spacing)	Option 2-Improved undrained shear strength, C_u (kPa)		
	From vane shear	From CPTs	From dilatometer
Soft clay (0.5 m c/c)	38	37	39
Soft clay(1.0 m c/c)	32	28	39
Firm clay(0.5 m c/c)	50	52	54
Firm clay(1.0 m c/c)	48	50	54

Table 5
Average improved undrained shear strength for option 3.

Soil type (nail spacing)	Option 3-improved undrained shear strength, C_u (kPa)		
	From vane shear	From CPTs	From dilatometer
Soft clay (0.5 m c/c)	35	NA	36
Soft clay (1.0 m c/c)	29	32	36
Firm clay (0.5 m c/c)	43	NA	39
Firm clay (1.0 m c/c)	42	45	39

Note: 1. For very soft to soft clay, the results lie with 0–30 kPa.
2. For Firm clay, the results lie within 30–60 kPa.

the shear strength of soft clay, which was 6% less than the design prediction. However, the overall mass stiffness improvement ratio was 2.7 for firm clay stiffness, which was 35% more than the design prediction.

Although the results for the soft clay strength and stiffness varied from the design predictions, they gave quite consistent improvement ratio values. Given the limited plate width, it was expected that stiffness results would be heavily influenced by local grout fractures. The influence of the grout on stiffness was hence not considered in the design.

3.3. Pull-out test

The aim of the pull-out test was to examine the bond strength between the grout and soil interface of the Geo-nails. Totally, six soil nails were installed in either firm or soft soils. P_{01} to P_{06} were installed with a 3 m grouted bond length within specific soil strata, with two nails per grout option. Nails P_{01}, P_{03} and P_{05} were installed within the soft clay strata at approximately 11 m depth. Nails P_{02}, P_{04} and P_{06} were installed within the firm clay strata between 5 and 8 m in depth. A closely spaced arrangement of drains was used to speed up the consolidation process.

Table 6
Average undrained shear strength and Young's modulus of soil

Soil type	Unimproved		Improved (mass properties)	
	Shear strength (kPa)	Young's modulus (MPa)	Shear strength (kPa)	Young's modulus (MPa)
Soft clay[1]	20 (15–27)	6 (4–8.5)	46 (43–52)	15.4 (13–18)
Firm clay[2]	37 (30–47)	20 (17–29)	93 (90–97)	20.4 (18–25)

Note: 1. Option 2 with 0.5 m c/c is adopted as shown in Table 2.
2. Option 1 with 1 m c/c is adopted as shown in Table 2.

The position and details of the six nails are shown in Figs. 5 and 6. The drains were approximately 13 m long within the Option 1 area and 15 m long within Option 2 and 3 areas. The full complement of six drains was installed in the Option 1 area upon completion of phase 1 grouting. The five drains within the Option 2 area were only installed after completion of phase 2 grouting. Only two drains were installed within the Option 3 area, but due to obstruction, only one drain was constructed for the trial test.

The test was considered successful if the displacement of the soil nail after three cycles did not exceed 0.1% of its length after a holding period of 15 min. The test load was two times the design working load, applied in three equal increments from an initial load of 20%.

The test criteria include

a) The grout should have a minimum strength of 20 MPa and be at least 4 days of age,
b) The test load should be two times the working load,
c) The test load should be measured with an accuracy of ± 1 kN,
d) A test frame was used to mount the testing jack so that the retained face was not supported at a localized point only,
e) Dial gauges used to record the displacement of the soil nail should be accurate to at least 0.01 mm,
f) The soil nail shall be loaded to 20% of the test load (to take up slack), and this point shall be taken as the datum for displacement measurements,
g) The remaining test load should be applied in three equal increments and displacement measurements should be recorded at each stage. The full test load should be maintained for 1 h.
h) Three complete cycles of the test load should be applied sequentially, and

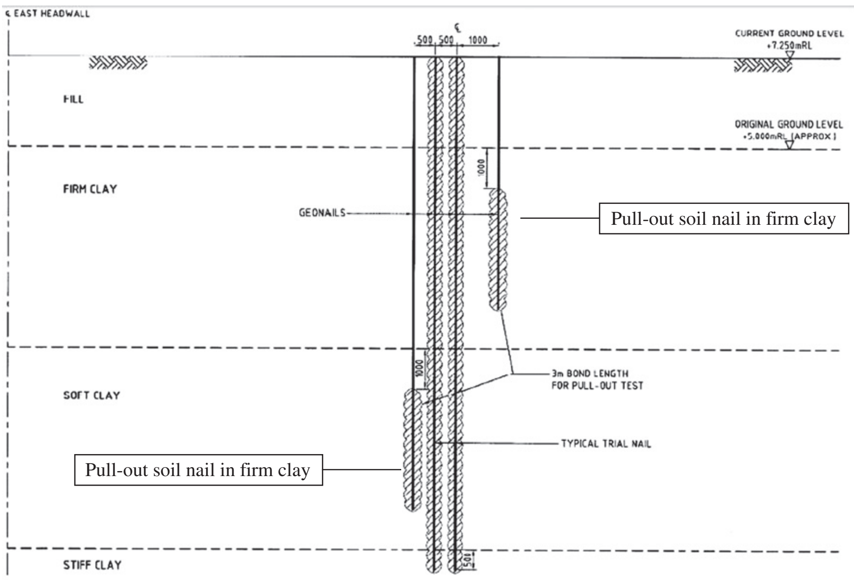

Fig. 6. Profile of pull-out soil nails.

i) The test should be considered successful if the displacement of the soil nail after three cycles did not exceed 0.1% of its length.

A review of the pull-out test results indicated that nails P_{01} to P_{04} satisfy the acceptance criteria which were stated above. P_{05} failed due to excessive displacement under the working load and the test for P_{06} was not successful due to the uneven loading of jacks in the later stages. Based on the trend of the first three cycles, it was estimated that P_{06} would have been able to pass the test if it had continued.

To facilitate testing, the complete pull-out nails used two pre-stress strands as the reinforcement instead of the standard GFRP reinforcement, as GFRP was slippery when held tight by a hydraulic jack, which was reported by Cheng et al. (2009). The strands were generally difficult to assemble and install. Nail P_{05} and P_{06} appeared to have been affected by varying degrees of strand pull-out or failure of the strand to grout bond. The results were presented in two parts as P_{01} and P_{03} were embedded into the soft clay while P_{02} and P_{04} were in the firm clay. The properties of the strand are specified in Table 7. In fact, Cheng et al. (2009) also reported various difficulties in carrying out pull-out for GFRP nails in sand.

4. Pull-out test in soft clay

Based on the Load-versus-Deformation curve for P_{01} pull-out test, some small positive nail head movements (in the same direction as the loading phase) were measured during the unloading phase in the final cycle. This anomaly could be explained by two possible mechanisms: friction loss in anchor free length and residual soil creep. The Load-versus-Deformation curve of P_{01} and P_{03}, which was embedded into soft clay, is shown in Figs. 7 and 8, respectively.

Friction might have developed in the free length of the trial nail anchored into soft clay due to its long installation length (greater than 10.5 m), i.e., the testing strands might have contact with the sheath along the free length and thus generate friction during the tests. Another possible explanation for the observed movement during the unloading phase was the existence of on-going residual soil creep. A conservative friction correction was applied to eliminate the possible friction developed in the free length. An estimation of the friction in the free length was made based on the following assumptions:

a) Friction was generated at the top of the nail.
b) Relatively small movement was required to mobilize the limiting friction generated.

The friction underwent reversal in direction from the loading to the unloading phase and the limiting friction values in the ‘‘down’’ and ‘‘up’’ directions were equal.

The linear elastic stiffness of the nail was obtained by drawing a line parallel to the loading curve of the last loading cycle for the P_{01} pull-out test. The frictional force experienced during loading and unloading was obtained by measuring the difference between the ultimate pull-out force and the force at the point where the strand began to contract, according to the estimated linear elastic stiffness. The difference in the force was equal to twice the frictional force due to its reversal in direction. The free length friction force was estimated to be 45 kN. The frictional correction relationship was given by Eq. (7) as follows:

$$\text{Friction}-\text{corrected ultimate Bond Resistance} = (P_{ultm} - \Delta \text{Fr})/L_b \tag{7}$$

Table 7
Properties of GFRP rod produced by pultrusion process and strand.

Materials	Yield strength (MPa)	Young’s modulus(GPa)
Strands	350	200
GFRP	850	300

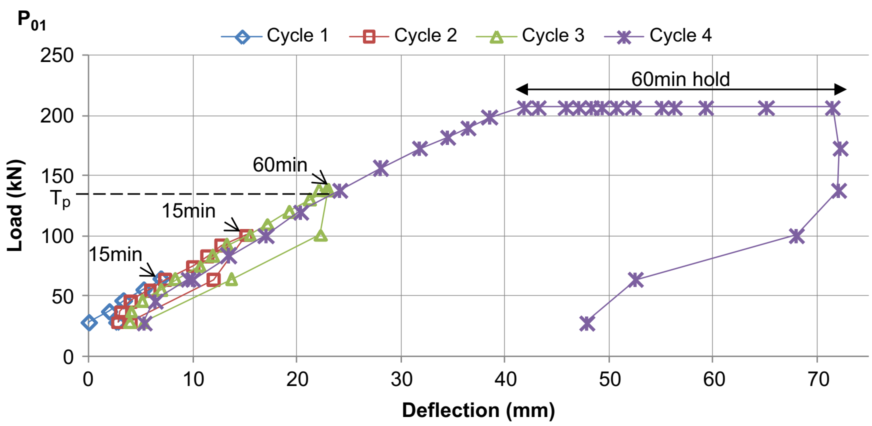

Fig. 7. Load vs. displacement of nail P_{01} (Tp is the pull-out resistance).

where

ΔFr was the frictional force against the loading and unloading phase

P_{ultm} was the measured ultimate pull-out force,

L_b was the bond length of the trial nail

Since the pull-out force for P_{02} and P_{04} were similar, same of amount of the friction loss was assumed in option 2.

5. Pull-out test in firm clay

The abovementioned anomaly in the measured nail head movement during the unloading cycles was not found in the Option 2 test nail P_{04}, which was anchored into firm clay. It was believed that since the test nails anchored in firm clay were in general shorter in length than those anchored in soft clay, the chance of having the test strands came into contact with the sheath within the free length was therefore smaller. As such, no correction was required for the pull-out test results in firm clay under the Option 2 improvement scheme. Deformation curves of P_{01} and P_{03}, which were embedded into firm clay, are shown in Figs. 9 and 10, respectively.

The ultimate pull-out force measured in P_{02} anchored in firm clay Option 1 was about 320 kN, which was much higher than that measured in Option 2. This excessively high pull-out resistance could have been explained by the substantial friction developed within the free length during the last cycle, with the ultimate pull-out resistance associated discarded completely. The results obtained in the third loading and unloading cycle with a similar friction correction discussed above were those used as the ultimate pull-out resistance in Option 1. The friction correction had been found to be 20 kN. The results of the pull-out tests are summarized in Table 8.

The actual measured bond resistance, as shown in Table 8, was higher than the design values in all cases, despite the fact that the improved soil strengths were generally less than that predicted. The greater improvement in the nail bond resistance was attributed to the contribution of the grout network. The results of the nail pull-out test indicated that most of the nails satisfied the acceptance criteria under the design ultimate load condition. Nail P_{05} was considered to have failed the tests due to excessive displacement under working load; however, it was noted that a test bond length of only 3 m was very susceptible to local soil or nail installation anomalies.

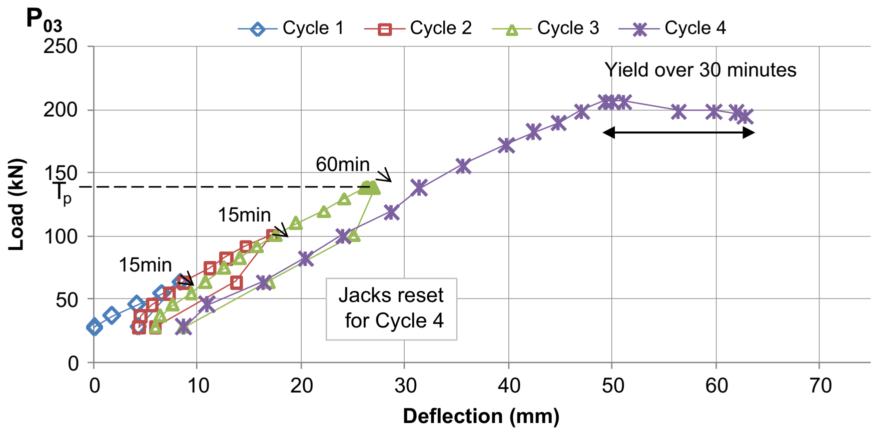

Fig. 8. Load vs. displacement of nail P_{03}.

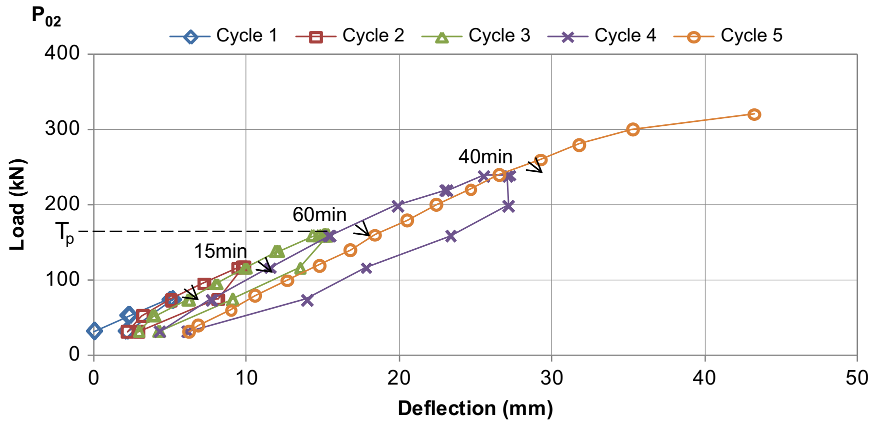

Fig. 9. Load vs. displacement of nail P_{02}.

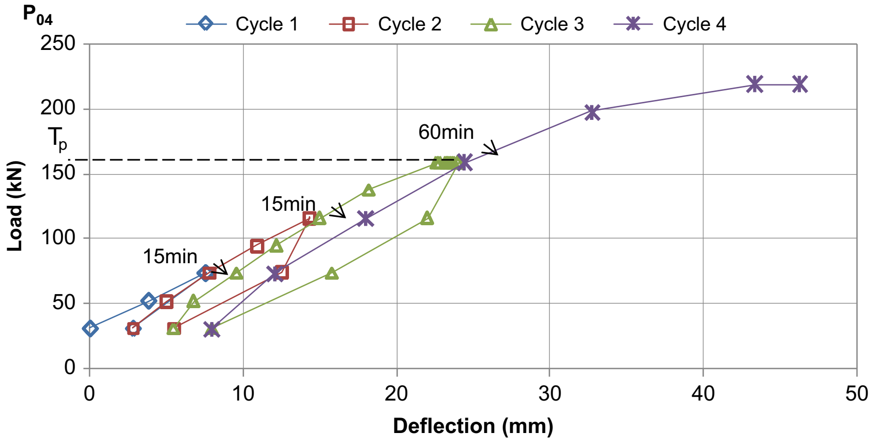

Fig. 10. Load vs. displacement of nail P_{04}.

Table 8
Summary of pull-out tests.

ULT bond resistance	Option 1	Option 3
Actual		
Firm Clay (1 m c/c) kN/m	106	NA
Soft Clay(0.5 m c/c) kN/m	70	NA
Friction corrected		
Firm Clay(1 m c/c) kN/m	73	NA
Soft Clay(0.5 m c/c)kN/m	55	NA

As such, this result was not considered representative of the group.

In general, the actual nail pull-out capacity was found to have exceeded predictions, and the increased pull-out resistance was able to be used to optimize the nail design. The advantage of using fracture grout in the Geonail was clearly demonstrated in the test results. For the same grout volumes, closer nail spacing gave a higher degree of soil improvement in soft clay. The bond resistance was highly increased in soft clay, at about 55 kN/m, while the original predicted resistance was about 46 kN/m. The results of the tests indicate a significant improvement in the innovative Geonail system, which was not possible with the classical soil nail installation method. In both soft and firm clay, the multiple phases grouting were found to be the most effective way to achieve strength enhancement, with the improvement in firm clay not as effective as that in soft clay. This was possibly due to the loosening of the soil structure for firm clay which was then compensated by the grout.

6. In-situ tests for verifying the design parameters

Horizontal CPTs, a hand vane shear test and pull-out tests were carried out at the headwall before the commencement of the box jacking operation. Accordingly, the in-situ strength of the Geonails and the grouted soils

Table 9
Result of HCPT for soft clay.

	GE28 (−3 m RL)	GE24 (−3.5 m RL)	Average	Design	% Different
Cu (con)	55	40	52		
Cu (mass)	63	46	60	61	−2

Table 10
Result of HCPT for firm clay.

	Average at +1.5 m RL (strength limited to 90 kPa)	Design	% Different
Cu (con)	78		
Cu (mass)	90	57	+57

within the embankment were obtained before the excavation started and prior stress relaxation.

6.1. Horizontal CPTs (HCPT)

Two HCPTs were carried at GE28 (at level −3 m RL) and GE24 (at level −3.5 m RL) for soft clay, and the locations of these two tests were shown in Fig. 1. The ratio between mass improved undrained shear strength and the improved undrained shear strength due to consolidation (IRMC = Cu (mass)/Cu (con)) was equal to 1.15. Test results in Table 9 indicated that the average of the improved undrained shear strength due to consolidation was about 52 kPa. Multiplying the shear strength of 52 kPa by IRMC (1.15), the mass improved undrained shear strength was found to have an average value about 60 kPa, which was 2% lower than the design requirement.

Due to obstruction, only one CPT was carried out in firm clay. IRMC = 1.3 was adopted for the prediction of the mass improved undrained shear strength of the fracture grouted firm clay. After testing, the undrained shear strength in the firm clay due to consolidation was found to be about 78 kPa. Multiplying IRMC with the improved undrained shear strength, the mass improved undrained shear strength was estimated to 101 kPa, which was higher than the undrained shear strength of stiff clay (90 kPa). It was important to note that this estimated '101 kPa' undrained shear strength was the same as the value (100 kPa) obtained in plate load test. In order to achieve a more robust design, it was decided to limit the design undrained shear strength to 90 kPa, which was still 57% higher than design requirement, as shown in Table 10.

6.2. Hand vane shear tests

During the box jacking operation, hand vane shear tests were carried out to examine the quality of the ground improvement. Based on Bjerrum (1973) the correction factors for undrained shear strength, i.e. the associated correction factors, were 1.0 and 0.8 respectively. In general, the undrained strength obtained from hand vane shear tests matched the design requirement. The undrained shear strength of soft clay (due to consolidation only) determined from the hand vane shear test was close to 50 kPa (the design improved undrained shear strength due to consolidation only). The average undrained shear strength of firm clay (due to consolidation only) was 44 kPa, which matched the design undrained shear strength (due to consolidation only).

6.3. Pull-out tests

Post installation of the Geonails under the QR embankment and a subsequent fracture grouting in-situ pull-out test was undertaken to examine the performance of the Geonail system. Although 17 pull-out tests were scheduled, only 12 pull-out tests were completed due to construction limitations. The positions of the pull-out soil nails under the embankment are as shown in Fig. 11.

There were two types of test condition which were

Type A—Pull-out tests carried out after homing grout (with two strands)
Type B—Pull-out tests carried out after homing grout and post grout (with two strands)

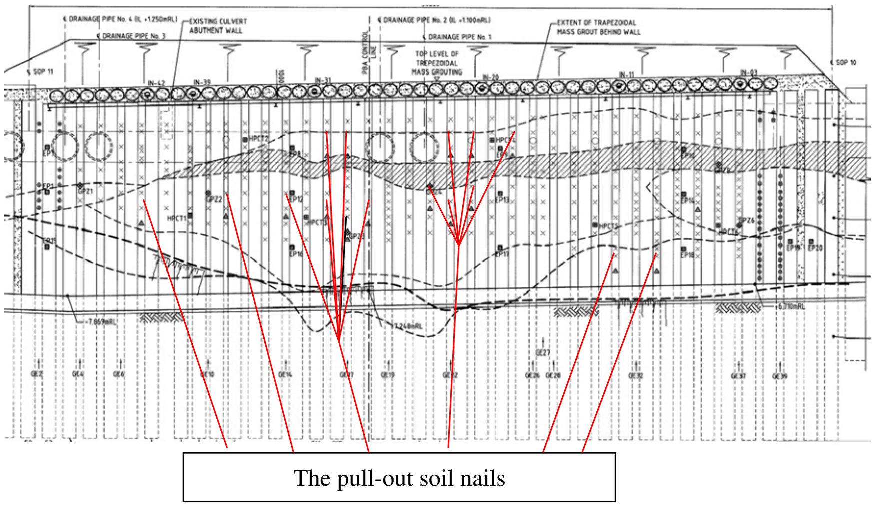

Fig. 11. Position of the pull-out soil nails under the embankment.

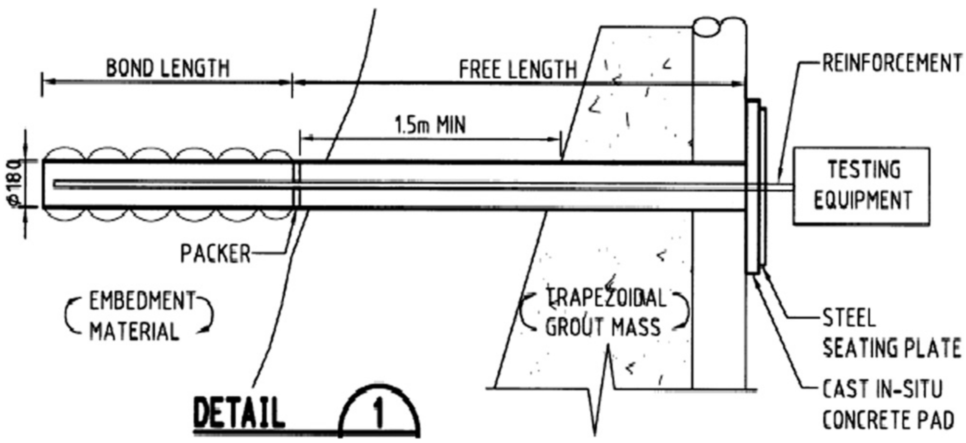

Fig. 12. Typical details for test nails.

The properties of the strand can be found in Table 7. The unconfined compressive strength (UCS) and Young's modulus of the homing grout and post grout were 5.0 and 1000 MPa, respectively.Fig. 12

The pull-out tests were carried out generally in accordance with Annex C of BS EN 14490:2010. The grouting specifications for the test nails are shown in Table 11.

Table 11
Grouting volumes and injection phases.

Soil layer	Temporary vertical spacing (m)	Longitudinal spacing (m)	Injection volume of 1st phase (L/m)	Injection volume of 2nd phase (L/m)
Firm clay	0.5	1	18	–
Soft clay	0.5	0.5	11	11
Firm clay (within +1.0 to +3.0)	0.5	1	24	–
Stiff clay	0.5	N/A	N/A	N/A

The test load was two times the design working load, applied in three equal increments from an initial load of 20%.

Out of the 12 pull-out tests, five tests which included GE11, GE16, GE18, GE22 and GE30 were successfully performed as shown in Table 12.

Based on the Load-versus-Deformation curves (Figs. 13–17) for the successful pull-out tests, some small positive nail head movements (in the same direction as the loading) were measured during the unloading phase in the final cycle. It was considered possible that friction may have developed in the free length of the trial nail anchored in soft clay due to its long installation length.

Although the pull resistance in firm clay was not obtained (due to the poor construction and installation), the pull-out resistance in soft clay and siltstone were obtained. The pull-out resistances of the four pull-out tests with fracture grouting in the soft clay were found to be higher than the design requirement. The average pull-out resistance of the Geonails was 85 kN/m. The test at GE22 was designed for determining the pull-out resistance

Table 12
Result of pull-out tests.

Test nail	Level (m RL)	Test type	Soil type	Designed Tp (kN)	Measured Tp (kN)	Friction corrected Tp (kN)	Corrected pull-out resistance (kN/m)	Pull resistance (kN/m)
GE30	−5.5	A	SS	303	294	269	90	90 (but not used in design)
GE18	−3	B	SC	207	237	219	73	85 (23% higher than the Tp)
GE11	−2.5	B	SC	207	352	322	107	
GE16	−2.5	B	SC	207	235	217	72	
GE22	−3	A	SC	42	200	184	61	61 (4 time greater than Tp)

Note: 1. Tp is the pull-out resistance.
2. SC and SS: Soft clay and siltstone respectively.

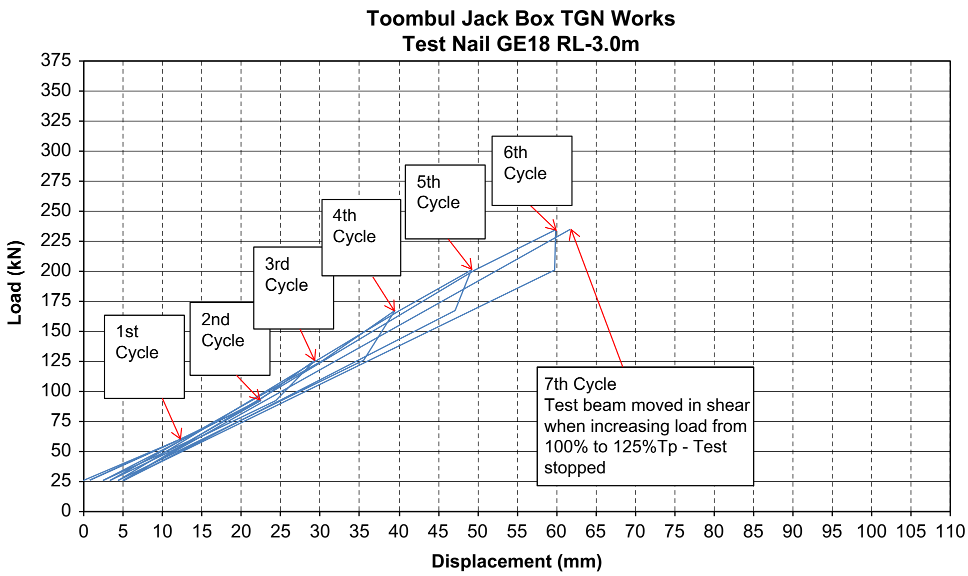

Fig. 13. Test nail GE18 at −3.0 m RL.

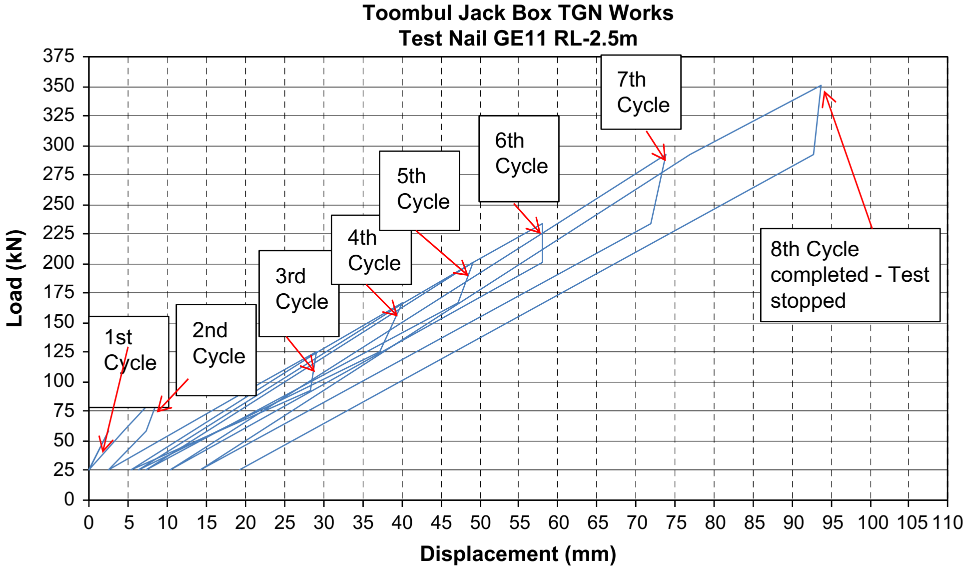

Fig. 14. Test nail GE11 at −2.5 m RL.

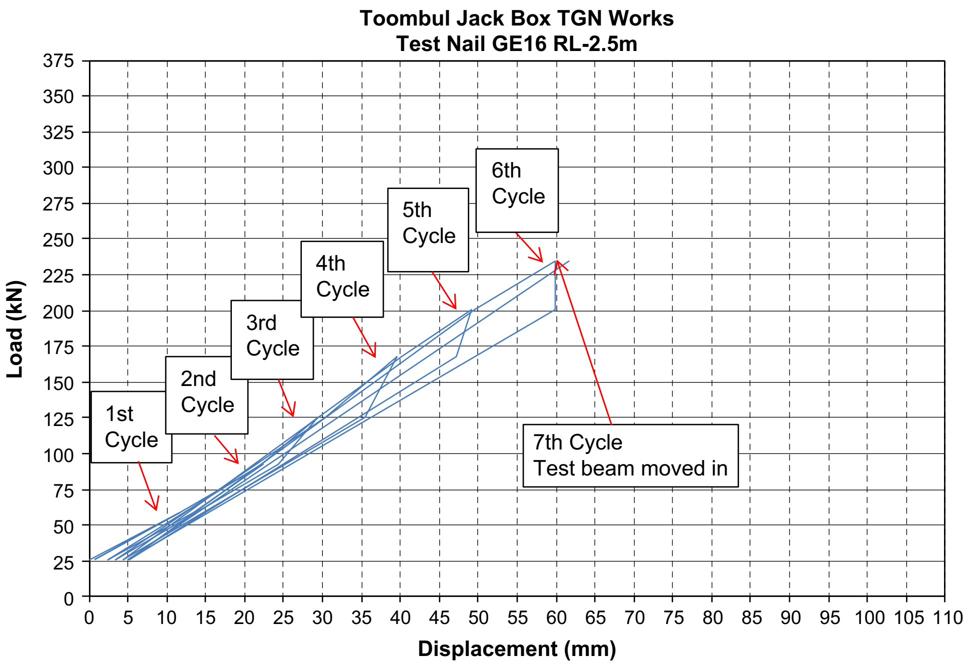

Fig. 15. Test nail GE16 at −2.5 m RL.

of the Geonail without fracture grouting. It was found that the pull-out resistance was 12% less than of the design pull-out resistance of the grouted Geonail. Even so, the pull-out strength was still four times the required design pull-out strength which was more than adequate.

7. Discussion and conclusion

The present paper focuses on the newly developed Geonail system and the corresponding trial tests. Through extensive site trials and in-situ tests, the performance of the Geonail system in the improvement of shear strength of ground was investigated. An innovative Geonail system, which combines fracture grouting and soil nails, was shown to be very effective in soft clay. It can significantly improve the pull-out resistance of the soil nail, especially in soft clay. Rather than being based on the close spacing of soil nails, the ultimate bond strength of the soil nails is dramatically improved by fracture grouting, which forms a finger net in soil. This is a cost effective method which can be readily adopted for the purpose of soil improvement in soft clay.

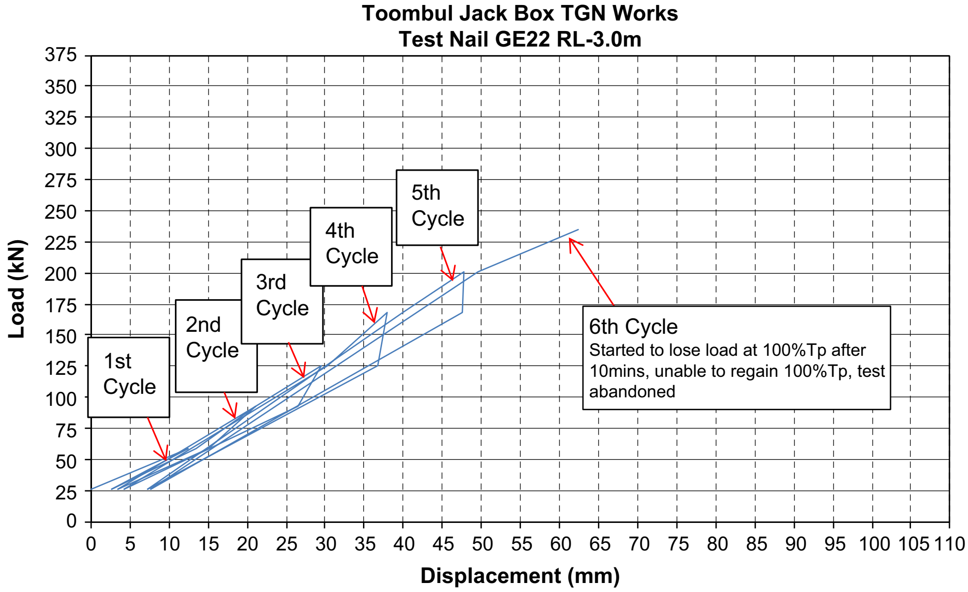

Fig. 16. Test nail GE22 at −3.0 m RL.

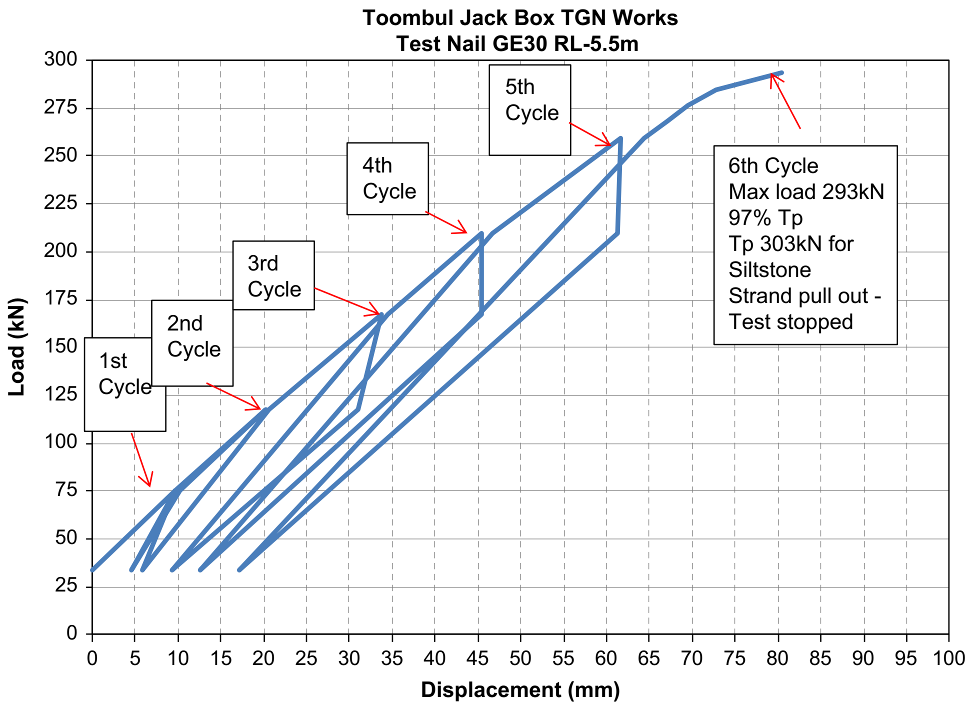

Fig. 17. Test nail GE30 at −5.5 m RL.

Also, the results of the vane shear, CPT and dilatometer test indicate that the global soil mass was improved. The shear strength of soft clay was found to improve from 20 to 46 kPa, while it improved from 37 to 93 kPa in firm clay. The gain in shear strength come from both consolidation and grout replacement, which is consistent with the design method. In addition, the plate load tests indicated that the stiffness of the soft clay is highly improved due to fracture grouting, while the improvement is less obvious in firm clay. To this degree, fracture grouting is very efficient for soft clay and can significantly improve the mass properties of soil. While the results from the CPT, vane shear test, DMT and plate load test were largely very similar, noticeable differences were also noted between the results from these tests. For design purposes, the global mass properties after soil improvement from the plate load test were used.

Based on the Geonail system and other stabilization measures, the construction of the airport link in soft ground proved to be successful. The railway remained in operation during the construction with limited settlement. The maximum measured settlement was about 40 mm which is considered small and acceptable for such a large scale construction in such a poor soft ground conditions. Much more settlement would have occurred in the soft clay had conventional methods of construction and stabilization been adopted. In conclusion, the Geonail system performs well when compared to the classical soil nail system. It has the advantage of effective local and global stabilization in soft ground and provides reliable performance. It can also effectively limit the ground movement and improve the global soil mass properties, neither of which is possible when using the classical soil nail system.

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References

Au, S.K.A., Yeung, A.T., Soga, K., Cheng, Y.M., 2007. Effect of subsurface cavity expansion in clays. Geotechnique 57 (10), 821–830.

Au, S.K.A., Soga, K., Jafari, M.R., Bolton, M.D., Komiya, K., 2003. Factors affecting log-term efficiency of compensation grouting in clays. Journal of Geotechnical and Geoenvironmental Engineering, ASCE 129 (3), 254–262.

Bjerrum L.,1973. Problems of soil mechanics and construction on soft clays. State-of-the-art report to Session IV. In: Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, 8, Moscow, Proceedings. vol. 3, pp. 11, 1–159.

Cheng, Y.M., Yeung, A.T., Tham, L.G., Au, S.K., So, T.C., Choi, Y.K., Chen, J., 2009. New soil nail material-pilot study of grouted GFRP pipe nails in Korea and Hong Kong. Journal of Civil Engineering Materials, ASCE 21 (3), 93–102.

Cheng Y.M., Au S.K., Pearson A.M., Yang Y., In preparation. Fracture grouting for soil improvement and geonails in soft soil tunneling. Journal of Geotechnical and Geoenvironmental Engineering, ASCE.

Essler R.D., Droof E., Falk E., 2000. Compensation grouting, concept, theory and practice. In: Advances in Grouting and Ground Modification Proceedings of the Sessions of GEO Denver 2000, August 2000. Geotech sp Publ no. 104, pp. 1–15.

Falk E., 1998. Bodenverbesserung durh Feststoffeinpressung mittels hydraulischer Energie. Ph.D. Thesis. Technical University, Vienna, Austria.

Kitazume, M., Okamura, 2010. Contributions to "Soils and Foundations": ground improvement. Soils and Foundations 50 (6), 965–975.

Liao, H.J., Su, S.F., Chen, W.L., 2006. Ground improvement piles induced shear strength increase in normally consolidated clay. Journal of the Chinese Institute of Engineers 29 (1), 13–21.

Soga, K., Komiya, K., Akagi, H., Jafari, M.R., Bolton, M.D., 2001. Soil consolidation associated with grouting during shield tunnelling in soft clayey ground. Geotechnique 51 (10), 835–846.

Soga, K., Au, S.K.A., Bolton, M.D., 2004. Laboratory investigation of multiple grout injections into clay. Geotechnique 54 (2), 81–90.

Voottipruex, P., Bergado, D.T., Suksawa, T., Jamsawang, P., Cheang, W., 2011. Behavior and simulation of deep dement mixing (DCM) and stiffened deep cement mixing (SDCM) piles under full scale loading. Soils and Foundations 51 (2), 307–320.

Wang, L., Zhang, G., Zhang, J.M., 2010. Nail reinforcement mechanism of cohesive soil slopes under earthquake conditions. Soils and Foundations 50 (4), 4459–4469.