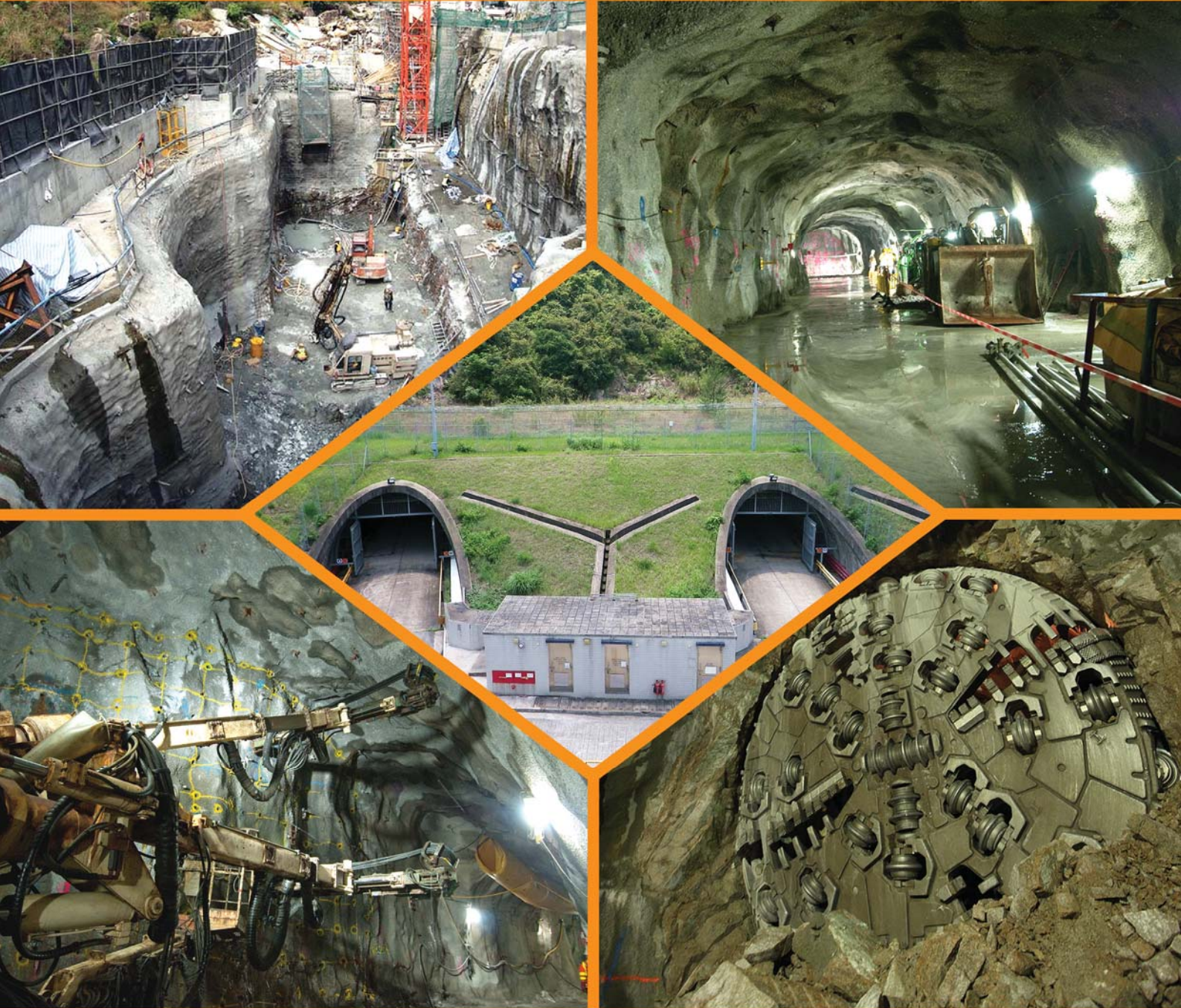


Geotechnical Aspects of Tunnelling for Infrastructure Development

25 May 2012
Hong Kong



**Proceedings of the 32nd Annual Seminar
Geotechnical Division, The Hong Kong Institution of Engineers**

Geotechnical Aspects of Tunnelling for Infrastructure Development

**25 May 2012
Hong Kong**

**Jointly organised by:
*Geotechnical Division, The Hong Kong Institution of Engineers
Hong Kong Geotechnical Society***

Captions of Figures on the Front Cover

Centre: Government Explosives Depot at Kau Shat Wan
(Courtesy of Civil Engineering and Development Department, Government of the Hong Kong SAR)

Top-left: Deep excavation of intake structure for Tsuen Wan Drainage Tunnel
Top-right: Temporary tunnel support for Harbour Area Treatment Scheme
Bottom-left: Drilling works at Harbour Area Treatment Scheme
Bottom-right: Breakthrough of Lai Chi Kok Drainage Tunnel
(Courtesy of Drainage Services Department, Government of the Hong Kong SAR)

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Published by:

Geotechnical Division

The Hong Kong Institution of Engineers

9/F., Island Beverley, 1 Great George Street, Causeway Bay, Hong Kong

Tel : 2895 4446 Fax : 2577 7791

Printed in Hong Kong

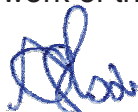
Foreword

The HKIE Geotechnical Division Annual Seminar provides a platform for geotechnical engineers and researchers to exchange their knowledge on hot geotechnical topics. No doubt tunnel and underground construction have been a hot topic amongst geotechnical engineers in recent years. Many ongoing mega infrastructure projects and mega projects on the drawing board including many of the Chief Executive's ten major infrastructure projects, are associated with tunnel and underground constructions. These projects call for substantial geotechnical input, often to overcome constraints and difficulties due to complex ground conditions, to protect the existing developments, and to interface with other projects. The geotechnical profession in Hong Kong has gained valuable experience in tunnel and underground constructions. The 2012 HKIE-GD Annual Seminar will serve as a platform for the profession to consolidate our experience and geotechnical expertise in tunnel and underground constructions, and to equip ourselves with the knowledge to meet the challenges from underground developments in the years ahead. This is in line with the Government initiative to house Government facilities underground and vacant lands for other use.

In view of this interesting and popular topic, the response from local and overseas geotechnical engineers and researchers has been overwhelming. A total of about 50 papers will be published in the proceedings, a record number as compared with all previous GD Annual Seminars. The number of participants is expected to be over 600 which would be another record.

In addition to local speakers, we have invited overseas speakers to share their experience in tunnel and underground construction. Two international experts, Professor Raymond Sterling and Ir Nick Shirlaw will deliver keynote lectures in the areas of underground developments in rock caverns and soft ground tunnelling.

On behalf of the Geotechnical Division, I would like to thank the Hong Kong Geotechnical Society for jointly organising this seminar. I would also like to thank our Guest-of Honour, Ms Grace Lui, the Keynote Speakers, the speakers, and the authors of the papers for their support. The contributions from the sponsors are gratefully acknowledged. In particular, I am most grateful to the Organising Committee, under the leadership of Ir Terence C F Chan, for their excellent and dedicated work in making this seminar a great success. The hard work of the Technical Sub-committee is also appreciated.



Ir Edwin Chung

Chairman, Geotechnical Division (2011/12 Session)

The Hong Kong Institution of Engineers

May 2012

Acknowledgements

The Organising Committee would like to express sincere thanks to the following sponsors for their generous support of the Seminar:

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Ground Improvement for a Large Jacked Box Tunnel

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ABSTRACT

The Airport Link road tunnel at Toombul comprises two large reinforced concrete box structures, successfully installed by jacking techniques beneath Queensland's busiest railway in Brisbane, Australia. Limiting track movements and maintaining the integrity of the railway during jacking were key design issues. Challenging ground conditions required the use of novel ground improvement techniques to facilitate jacking. The ground improvement consisted of jet grout blocks, grout wall and an innovative use of horizontal "geonails". The geonails consist of TAM grouting tubes combined with either GFRP rods or a steel TAM tubes to form a combination of ordinary soil nails and fracture grouting in clay strata to both improve the weak soil properties and improve the soil nail pull out resistance. The design of the geonails required the development of a new design method which was verified, with modifications, on site by the testing of trial geonails. The use of the geonails, combined with the other soil improvements, facilitated the jacking of two adjacent large tunnels beneath six tracks of suburban and freight railway without disruption to railway operations.

1 INTRODUCTION

1.1 The project

The Brisbane Airport Link is a AUD \$4.8 billion dollar Design and Construct project funded as a Public – Private – Partnership. The project involves approximately 15 km of tunnels with at-grade and elevated access roads to relieve traffic congestion in Brisbane, Australia. The Airport Link contract was awarded to the consortium 'BrisConnections', and constructed by Thiess John Holland Joint Venture supported by key specialist subcontractors.

Much of the tunneling was constructed by either TBM or road header. As part of the project, the tender design envisaged a 50 m length of jacked box tunnel section, with very low cover, under a railway embankment carrying 6 tracks of urban and mainline (heavy freight) railway.

Thiess John Holland JV (TJH) employed a permanent works designer for the project, but designs requiring specific expertise, or with a high temporary works component, were carried out by specialist sub-consultants and sub-contractors. Benaïm, now part of the URS Corporation, were employed by TJH for the design of the temporary works, installation methods and ground improvement associated with the jacked boxes.

1.2 The jacked box tunnel

Use of the jacked box construction method for the section under the railway minimizes disruption to the busy live railway and is seen as being instrumental in the winning of the contract.

The tunnel for this section consists of two concrete boxes of overall dimensions 21.4 m wide x 12.5 m deep and 16.7 m wide x 12.5 m deep, both approximately 65 m long and immediately adjacent to each other. The tunnel roof is just below existing ground level outside of the embankment. The jacked box section is linked to two cut and cover sections of tunnel on either side of the railway embankment. One cut and cover section

forms the jacking pit for the jacked boxes and the other forms the receiving pit. An overall view of the site is shown in Plate 1 below.



Plate 1: Overview of site

The soils through which the tunnel passes are variable in both vertical and horizontal regimes. To enable the successful jacking of the tunnel, significant ground improvement was required for the soft clay soils prevalent over 60% of the tunnel face. Other soil improvement techniques were used to facilitate the jacking works. These included jet grout columns to form a grout block mass retaining wall and a grout wall formed using diaphragm wall techniques to provide nail anchorage.

2 THE SITE AND GEOTECHNICAL CONDITIONS

2.1 Overall description

The tunnels formed by the jacked boxes are to the north east of the Brisbane CBD in the Nundah suburb and pass below the six railway tracks of the Queensland Rail (QR) Brisbane suburban line, the QR mainline heavy freight line from Brisbane to the north and the Airtrain line from the CBD to Brisbane Airport.

The railway embankment is approximately three to four metres above the surrounding flood plain. The tunnel alignment requires the box base slabs to be approximately 13 m below grade, with a constant fall of 3.5% to the west. Two tracks of the Airtrain diverge from the mainline tracks just north of the jacking site. The alignment of the tunnels is approximately 30 degrees skew to the alignment of the railway, creating a wider face and complicating the box, shield and jacking arrangements. Plate 1 shows the layout of the boxes.

4.5 Geotechnical regime

The geological regime along the line of the jacked box tunnels is very variable. The site is adjacent to the Kedron Brook, which is a significant watercourse. The soils are alluvial deposits overlying the residual siltstone rock. The railway embankment, above the general grade, is an engineered embankment consisting of

generally firm to stiff silty clay with a rock separation layer at pre-existing ground level. The underlying alluvial soils consist of soft to firm clay, soft silty clay, overlying a stiff residual clay above the rockhead. Within this mix of different strength clay strata there are a number of significant bodies of medium dense sand lenses which were fully saturated with water and likely interconnected over the wider area. The clay layers are normally to very slightly overconsolidated. The underlying rock is a soft siltstone with some interbedded very stiff clay layers and coal seams.

All of the strata are variable in depth and thickness in both the longitudinal and transverse directions of the jacked box. However, only a limited soil investigation was possible in the railway reserve, due to the potential disruption to rail traffic and the obstruction formed by the rock separation layer. The strata levels and soil strengths were interpolated from soil investigations (boreholes and CPTs) carried out adjacent to the railway and along the line of the overall tunnel. Soil properties under the rail embankments were adjusted, from those estimated outside the embankments, based on an inferred consolidation from the embankment surcharge.

Within the perimeter of the jacked boxes there were two sets of existing drainage culverts, consisting of bank of 4 x 1650 mm diameter concrete pipes and a bank of 2 x 1800 mm diameter concrete pipes. Also within the ground was known to be the timber pile foundations and headstocks from an old timber trestle railway bridge, left in place during raising of the embankment many years prior to the Airtrain construction.

A transverse geological section is shown in Figure 1. This section, and others used in the designs, were derived from the investigation data and supplemented by logging during headwall pile installation. The initial design properties of the soil strata were estimated to be as those given in Table 1, below.

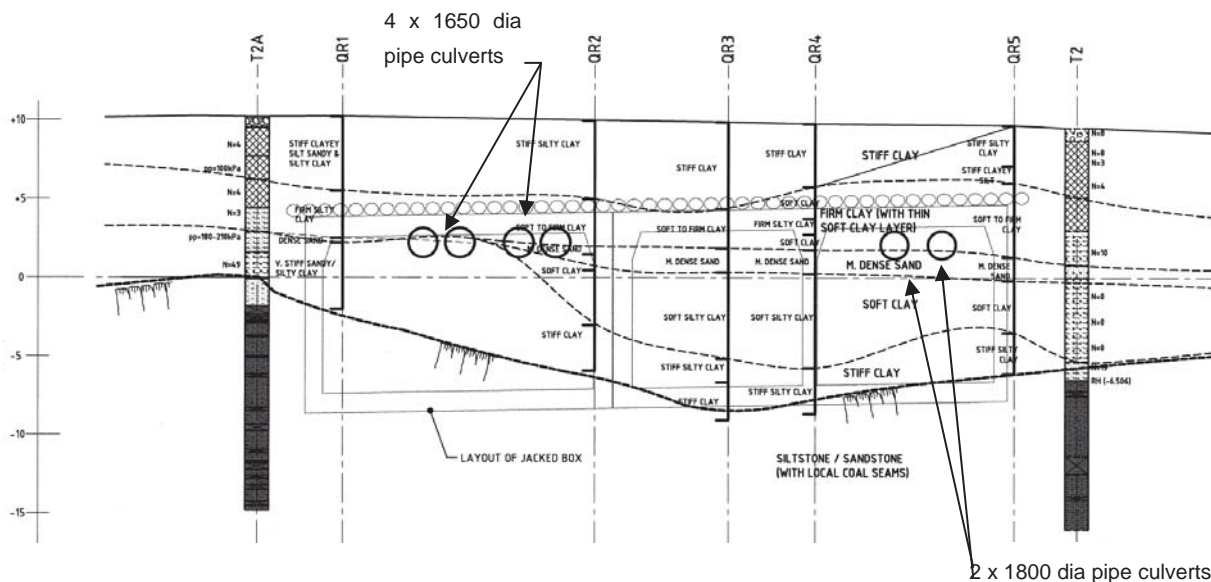


Figure 1: Transverse geological section

3 OVERALL DESCRIPTION OF TEMPORARY WORKS DESIGNS

4.5 Preliminary works

In order to construct the jacked box tunnel on the required alignment, preliminary works such as the removal of piled supports to the railway overhead line gantries, land clearance and railway slope re-grading was undertaken.

To facilitate the jacking of the boxes a jacking pit was constructed to the east of the railway embankment along the alignment of the eventual permanent works in this zone. The jacking pit headwall consists of 900 mm diameter bored, cast-in-place contiguous piles. The headwall was designed in conjunction with a trapezoidal jet grout block placed immediately behind the piles and acting as a gravity type wall, so that no soil anchors or other external supports were required. The jacking raft is a 1200 mm thick concrete, ground bearing raft, cast below the level of the permanent works tunnel slabs and on a vertical alignment to facilitate the required final gradient and position of the boxes. An interlocking canopy of steel tubes was installed immediately above the top slab of the boxes to provide separation between the jacking works and the railway.

The 760 mm internal diameter tubes were used to control soil settlements in advance of the excavation face during jacking, to distribute surcharge loading to the box front and to provide an anti-drag surface to prevent the boxes dragging the soil of the railway embankment sideways during jacking. The canopy was designed to maintain support for the railway under even the most extreme of the assumed design conditions.

Table 1: Unimproved design soil properties

Soil Strata		Approx. top and bottom levels of layer (mRL)	Bulk Density, γ (kN/m ³)	Poisson Ratio, ν	Undrained		Drained		
					c_u (kPa) Best Estimate	E_u (MPa)	c' (Kpa)	ϕ' (deg)	E' (Mpa)
Fill	Embankment Fill – Stiff Silty Clay	+9.5 to +5	20	0.3	90 (75-150)	36	5	27	31
	Embankment Fill – Silty Clay with Sand/Gravel	+9.5 to +5	22	0.3	-	-	5	36	20
Alluvium	Firm Clay outside Embankment	+5 to +1	18	0.3	30 (10-40)	7.5	3	26	6.5
	Firm Clay below Embankment	+5 to +1	18	0.3	40 (30-75)	10	3	26	8.7
	Soft Clay below Embankment	+0 to -5	17	0.3	25 (15-40)	6.5	5	23	5.6
	MD Sand	+1 to 0	19	0.3	-	-	0	32	10
Residual soil	Stiff-V. Stiff Clay	-3 to -13	19	0.3	90 (50-300)	22.5	-	-	19

Jacking of each box was effected by 750 tonne “pull” jacks supplemented by 1000 tonne “push” jacks. The jacks acted on the temporary jacking tail, of each box, which was demolished after completion of jacking. JB2 required 12 “pull” jacks and 14 “push” jacks whereas JB2 required 16 “pull” jacks and 14 “push” jacks.

3.2 Excavation process and mining shield

A number of Value Engineering workshops determined that a minimum acceptable excavation face angle would be 60 degrees, and that both a mining shield, embedded in the soil in advance of the box front face, and some form of soil improvement were required. This soil improvement was achieved by the use of the horizontally drilled fracture-grouted geonails, as described in Section 4 below. The 60 degree excavation face was designed to be supported by the soil nails, the soil improvement from the soil nails, a temporary concrete shield, at the inner walls, and steel shield, at roof level and outer walls, embedded in the slope in advance of the boxes. The shields were demolished after the box jacking was completed.

3.3 Instrumentation and monitoring

A comprehensive monitoring regime was installed for both the preparatory works, excavation of jacking pits and the box jacking phases. This monitoring included 24-hour, real time monitoring of the railway formation and tracks for a significant length above the work site. Also of note were horizontal inclinometers installed within the canopy tubes, to enable timely monitoring of soil movements in advance of the box drive.

4 SOIL IMPROVEMENTS TO FACILITATE JACKING

4.1 Components of the soil improvement

A trapezoidal jet grout block, constructed immediately behind the headwall, acted as a gravity type retaining wall to reduce earth pressures on the piled headwall. The grout block was also key to the initial wall breakthrough process, providing support to the excavation face as the headwall piles were demolished in stages to allow the boxes to be jacked through it. The width of trapezoidal grout zone is 3.0 m at top, linearly increasing with depth to formation level at 1 in 2 (horizontal to vertical). The trapezoidal mass grouting extends to 0.5 m below top level of the siltstone, to provide sufficient stability.

Another, smaller, jet grout block was provided at the north west of the final jacked box location and used as an anchorage to the northern sidewall nails. This type of anchorage was not required for the south sidewall as the siltstone rock was high and provided good anchorage for the sidewall nails.

A low strength grout wall was installed west of the railway to provide a water cut-off for the TBM launch box. The grout wall was also used in the jacking scheme design to provide adequate anchorage of the geonails at the receiving pit side, eliminating an approximate 10m length of nail to deliver significant time and cost savings. The resulting 'nail anchored' western grout wall could then also be used to maintain slope stability, enabling initial excavations in the cut and cover receiving pit to commence early. Figure 2 shows the arrangement of the ground improvement regime to facilitate jacking.

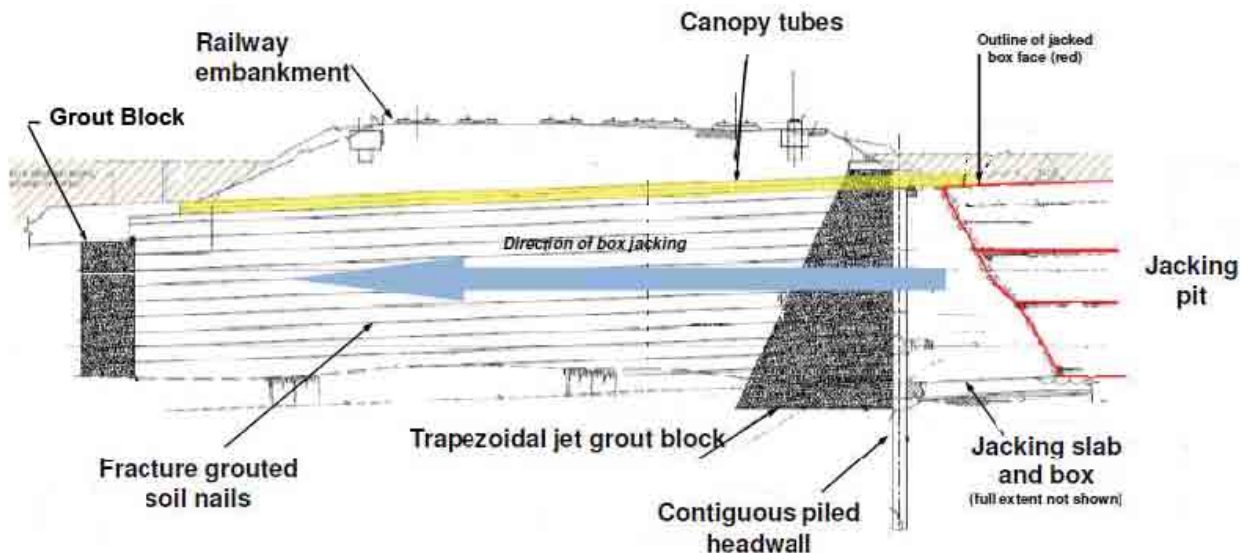


Figure 2: Arrangement of ground improvement regime to facilitate jacking

Fracture grouted geonails were the ideal ground improvement solution for the box jacking scheme. The geonails provided excavation face slope stability and minimised soil movements by the following means:

- (1) to provide a slope stabilisation force along the length of the nail by its pull-out resistance, similar to conventional soil nails;
- (2) to provide an enhanced pull-out resistance in the soft clays due to the interaction between nail, grout and soil;
- (3) to increase the stiffness and the shear strength of the surrounding soft clays by grout-pressure triggered consolidation and the physical presence of the grout materials penetrating into the surrounding soil mass;
- (4) to counteract any settlement of the railway due to bulk excavation by introducing grout volume, similar to compensation grouting techniques.

The geonails were used to facilitate permeation grouting in the granular sand layers. Fracture grouted geonails were installed in cohesive soils with a strength of less than 1 Mpa, in the soft and firm clay layers.

Secondary benefits of the geonails were that:

- (1) the nails assisted in stabilizing the headwall during excavation, the trapezoidal grout block during breakthrough and the west grout wall during early excavations, as described above;
- (2) the nail bodies acted as drains, reducing the length of water flow path and assisting consolidation;
- (3) Excavation and logging of the nails allowed a clear picture of the soils, the obstructions and any water bearing strata to be identified.

Sidewall geonails were designed and installed in the embankment along the side of the jacked boxes to strengthen and stiffen the soils at these interfaces and minimize the settlement at rail level caused by a sideways shift of the soil into the disturbed region in front of the boxes during jacking. An important function of the sidewall nails was to redistribute the pressure on the side of the sloping excavated face back to the concrete box and forward to the unexcavated, undisturbed mass of soil in advance of the excavated face. The nails were designed to do this by a combination of bending and catenary action, and hence requiring positive end anchorage. The sidewall nails were made stiffer and stronger than the normal nails by the use of steel casings as the TAM tubes, and the nails were anchored at the headwall, grout block, or the siltstone rock.

4.5 Design of geonails

The geonails used in this project are essentially a combination of soil nails and Tube a Manchette (TAM) grouting. The original idea of the geonails was suggested by Keller Ground Engineering Pty Ltd, the specialist sub-contractor for the soil improvement works. For the main face nails, the structural part of the nail was formed from glass fibre reinforced plastic (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. The side wall nails minimize steel TAM sleeves to make them both stronger and stiffer to control deflection of the soil at the sides of the box jacking excavation.

The use of TAM tubes is a common form of grouting. For compaction grouting, the grout is injected at relatively high pressure and high flow rates to fracture the nail body and surrounding soil to achieve significant penetration of grout. Grouting is done at each sleeve in turn, with accurate control of injected volumes and may be done in several stages over a period of time.

The discrete sand layers required strengthening to achieve a stable 60 degree excavation face, but just as important was the need to minimize water flows, which had the potential to minimize the slope face. Permeation grouting was therefore used for these soils, with the aim of achieving a relatively homogeneous mass which did not allow any significant flow of water into the excavated face. The grouting of the sidewall nails provided a water cut off and avoided consequential wide spread settlements from water drawdown.

In the very soft and soft clay layers, fracture grouting was used with two aims. The first of these was to increase the strength of the soils by a consolidation process. This required that the clay was fractured by the pressure of the grout and along the fracture lines the grout "fingers" displacing the clay. The grout injection pressures exceed the shear strength of the soil, which fractures to form grout fingers penetrating the surrounding ground. The grout fractures compress the clay and cause an increase in pore water pressure. The dissipation of the pore water to drains causes consolidation and strengthening of the clay and is also accompanied by a settlement. This fracture grouting can be carried out in a number of stages to achieve the desired soil strengthening via consolidation and, importantly for settlements, enhanced stiffness. As well as the consolidation improvement of strength of the clay, the soil mass also contains the fingers of grout, which are much stronger than the clay, thus enhancing the average strength of the soil mass. The second aim of the fracture grouting in clay was to increase the bond between the geonails and the soil. With the increase in soil strength this bond is naturally increased. The bond is also increased by mechanical interlock of the grout fingers, embedded in the soil mass, and the TAM tubes. Figure 3 shows some of the geonail details.

4.3 Design of fracture grouted geonails and fracture grouting trials

A literature search revealed only a limited amount of data and experience in the improvement of the strength and stiffness of very soft and soft clays by fracture grouting. Au (2007), Cheng (2009) and Bjerrum (1973) together with the Hong Kong GEO publication "Report on Potential of Using Grouting to Stability Loose Fill Slopes", GEO (2007), were used for the initial design of the improvement and theoretical verification that the fracture grouting could achieve the desired results. Based on this approach a theoretical improvement was estimated for a number of practical fracture grouting scenarios, including variations of grout pressure and

grout volume, consolidation periods and number of grouting stages. This study proved the feasibility of the method, a design method was established and improved properties were selected for an initial design. A fundamental of the model is that the total improvement of the soil mass is a combination of consolidation improvement and physical presence of the relatively strong (and stiff) grout fingers in the soil mass.

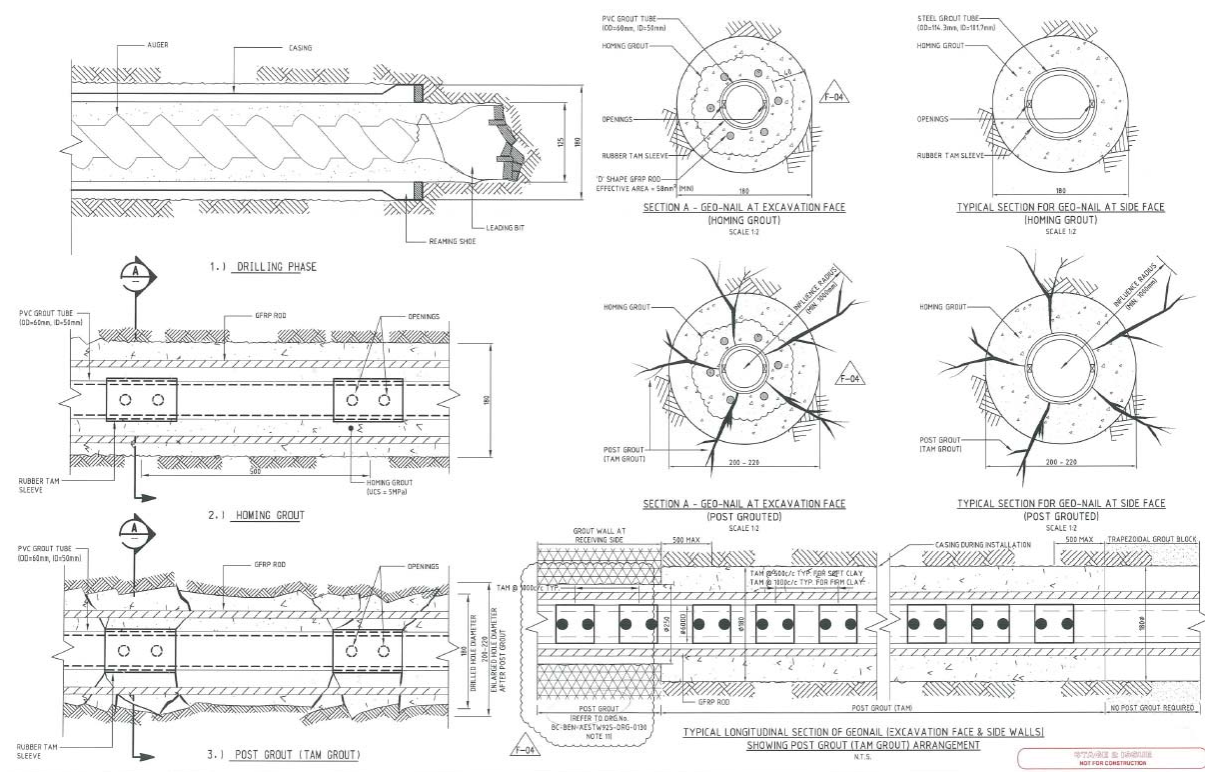


Figure 3: Detail of geonails

Nail trial

As the method and application was novel it was necessary to validate the degree of soil improvement and enhanced bond stress achieved. A trial was designed and implemented for the installation of a number of geonails with varying injection specifications and in both firm and soft clay target strata. This trial installation allowed testing of the unimproved and improved soil, as well as pull out testing to verify both the soil mass improvement ratio (defined as the ratio between improved undrained strength or stiffness to the original undrained shear strength or stiffness) and the improvement ratio for nail bond to soil mass (defined as the ratio between improved pull out resistance to the original pull out resistance). A series of in-situ tests were done before and after nail grouting, to quantify the strength & stiffness improvements. This was supplemented by monitoring during the installation process. The following tests were carried out:

- Cone Penetrometer Testing, (CPT)
- Dilatometer, (pressure meter testing)
- Piezometers
- Nail Pull-out tests
- Vane Shear (in borehole)
- Movement markers
- Visual inspection
- Plate testing

Exhumation of the nails was also carried out to verify fracture formation. Good extension of the fracture fingers was observed, with between 3-4 grout fingers per TAM sleeve extending for a significant distances from the nail body.

The elastic soil modulus improvement was primarily determined using plate load testing, so as to capture the mass effects of soil – grout interaction. Results were assessed based on Eurocode 7:

$$E_u = q_{net} b (1 - v_s^2) I_s / \delta_p \quad (1)$$

which is the slope of the Bearing Pressure vs Settlement curve. The predicted ultimate bond resistance of the geonails was originally estimated from the improved soil shear strength resulting from consolidation due to fracture grouting only. However, pull-out testing showed the actual bond resistance to be significantly higher

than predicted, in all cases trialled. The significant difference in the nail bond resistance was attributed to the contribution the interlock between grout fingers and consolidated soil. A finite element model was developed to verify the base assumptions and mechanism of pull-out failure. The design ultimate bond resistance for the soil nails used in the design was based on these test results.

An efficient design required recognition and quantification of the mass strength and stiffness behaviour; the interaction between improved soil and introduced grout. The assessment of pull-out strength and bond resistance was further used to define a mass soil strength prediction method for the improved soils. This was done by considering an influence zone around each nail with variations in soil strength based on the nail pull-out strength at one extreme and the consolidation improved soil and grout matrix at the other extreme. Figure 4 illustrates the assumed variation of strength at distance from the nail body based on the pull-out test results and CPT/Vane shear tests. This basic form of improvement was used to determine the value of the mass effect.

The design parameters for the soil improvement ratio and nail bond to soil were verified for several combinations of grout-soil parameters.

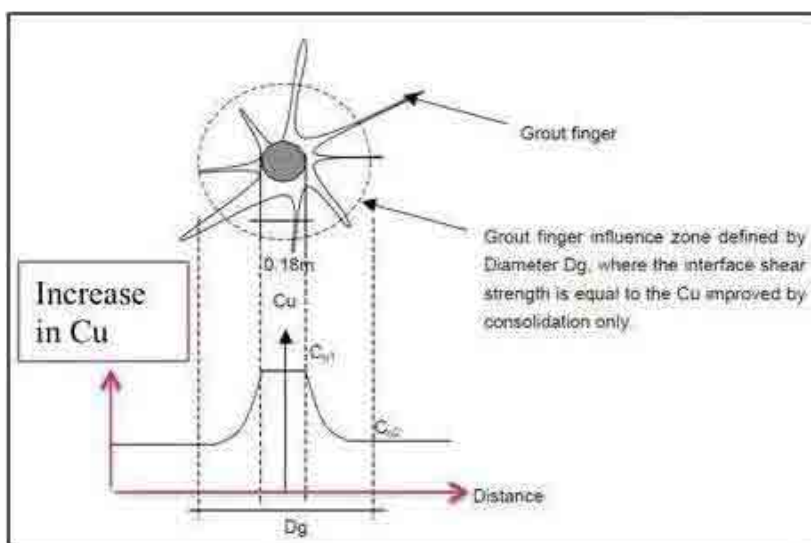


Figure 4: Effect of soil improvement with distance from the nail body

The main conclusions obtained from the trial nails were as follows:

- Observations during and post grouting provided confidence that the method used was effectively fracture grouting the soils.
- Grout injection pressures are a significant factor for consolidation improvement, especially in the soft clays
- The consolidation improvement of the soil was less than anticipated in both the soft and firm clays but nonetheless significant, and justified the modified design method as discussed below.
- Multi-phase injection was effective in increasing improvement in the soft clays but much less so in the firm clays.
- The improvement in soil stiffness followed the improvement in soil strength, and was less than originally estimated but still significant'
- The nail pull out resistance in both soft and firm clay was significantly greater than anticipated.

Based on the results of the geonail trials, the design method was modified so that conservative design improved soil parameters were calculated, compared to the test results. The design method was adjusted to account for the fact that more consolidation – and hence strength improvement – was taking place closer to the nails, but less consolidation further away from the nails, than was originally allowed for. The design also included for increased nail pullout strength and the effects on settlements of a less than anticipated stiffness improvement.

The improvement ratios for the soils, based on the trial nail data, and used in the designs are shown in Table 2, below. The nail trials, and especially the verified enhanced pullout resistance of the nails, allowed the

design of the geonails for face stability to be modified to significantly reduce the numbers of nails to be installed. Figure 5 exposed fractures from nail trial & CPT testing showing strength improvement.

Regimes for soil improvement in the soft and firm clays were chosen to achieve the desired soil strength and stiffness whilst minimizing installation cost and programme. The selected regimes were used for final verification of the stability of the excavation face and the side wall regions. These parameters included enhanced strength in the soft and firm clays, enhanced stiffness of the soft and firm clays and enhanced pull out strength of the soil nails but less consolidation further away from the nails.

Table 2: Comparison of unimproved and improved soil design parameters

Soil Type	Mass improvement ratio		
	Shear strength	Stiffness	Pull out resistance
Soft Clay	2.3	2.6	5.6
Firm Clay	1.4	1.1	2.7

- Notes: (1) Mass improvement ratio is define as the ratio of shear, stiffness and pull out resistance between the improved and unimproved of the soil.
 (2) The original soil properties can obtained from Table 1.

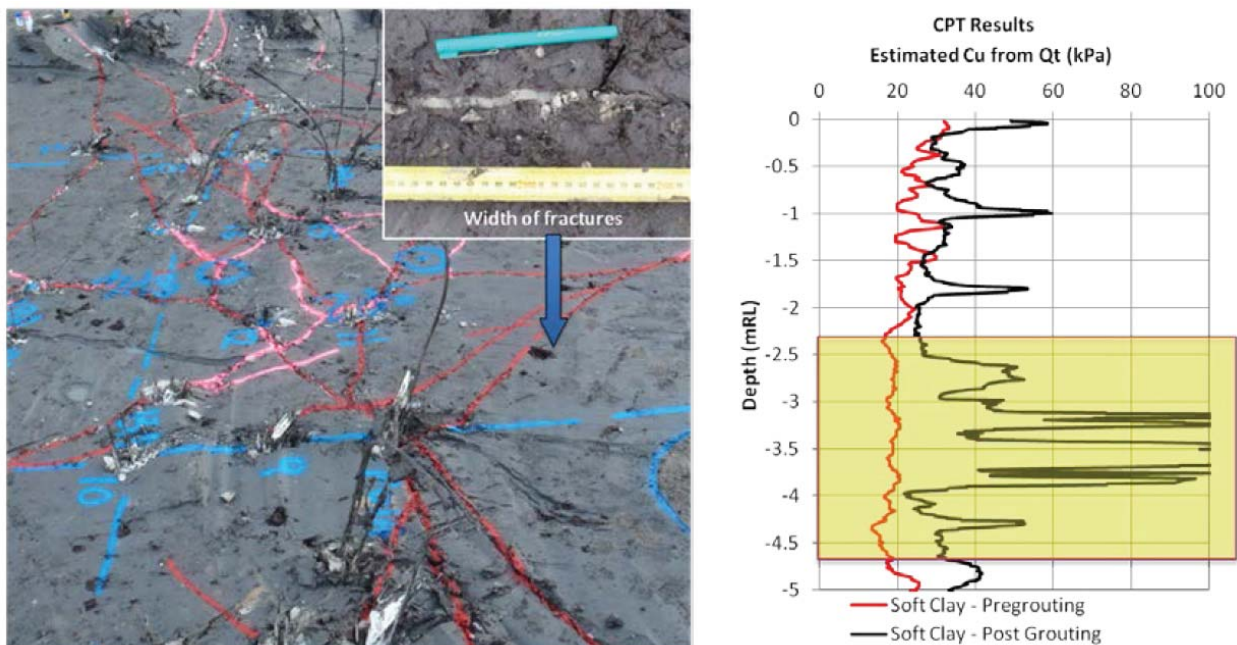


Figure 5: Exposed fractures from nail trial & CPT testing showing strength improvement

4.5 Excavated slope stability analyses and geonail layout

The excavated slope analyses were carried out in two dimensions (2D) using the computer program Slope2000. This program allows the definition of a maximum bond strength for embedded soil nails. A large number of excavation scenarios were assessed to ensure that all critical slopes within the jacked box excavation profile were assessed. The slope analyses took into account the railway surcharge and the skew effect of the slope relative to the nail orientation. A FLAC-3D numerical model was also constructed to verify the ground improvement, side face and excavated face stability design.

The geonail layout was highly constrained because of the obstructions, the constraints of ongoing construction and the requirement to drill the nails between the headwall piles. Some of the high level nails were drilled from the receiving pit end because of the obstruction from the existing pipe culverts. The nails at culvert level necessarily run parallel to the culverts, but the remainder of the nails run parallel to the direction of the box drive and at an angle to the skew excavation face. Nail and instrumentation layout is illustrated in Figure 6.

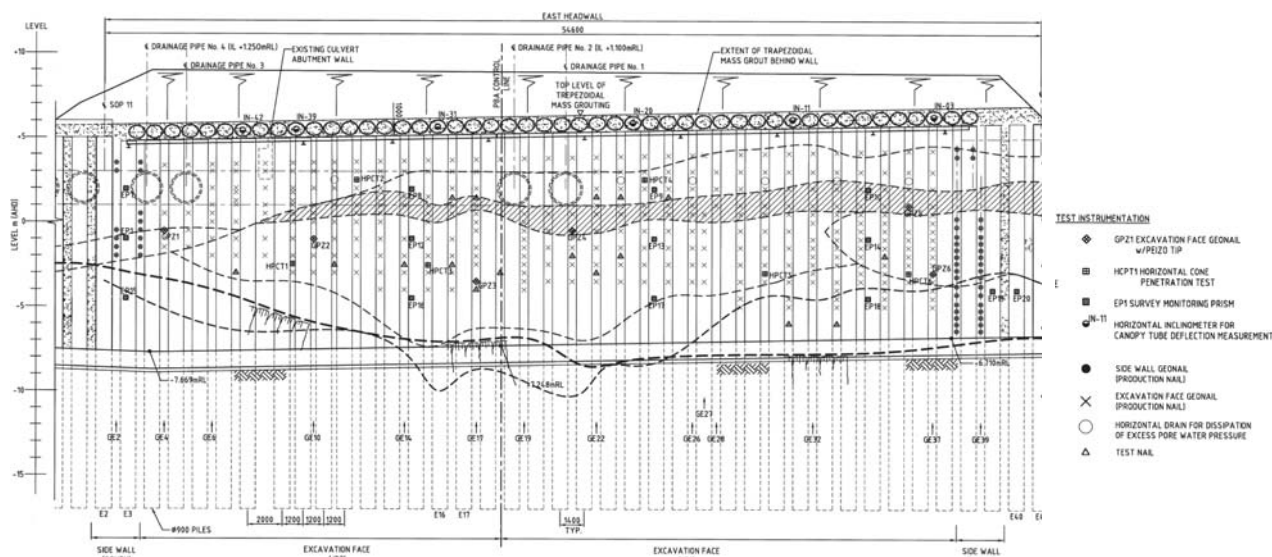


Figure 6: Elevation of headwall showing geonail & instrumentation layout

4.5 Settlement analysis & installation monitoring

Settlement analyses were carried out to take account of the installation of the piled headwall, trapezoidal grout block, canopy tubes, western grout wall, excavation of the jacking pit, partial excavation of the receiving pit and excavation during the two stage (one stage for each box) jacking. This analysis consisted of several interacting models built up to form a complete estimate of the settlement profiles due to all stages of construction. Included in the modeling was the canopy tube and concrete box interface, sidewall nail movements, excavations for the advancement of the boxes and the continuing settlements from the geonail induced ground consolidation. Settlement calculations for the box jacking were calculated from the integration of each stage of jacking as estimated in staged PLAXIS models of the jacking sequence.

The embankment stability design is based on assumed improved soil properties, limited direct ground investigation and little knowledge of the obstructions present, whilst allowing for natural variations in soil strata profiles & properties and a range of likely soil and structure stiffness. Monitoring was required to verify that the rail embankment and its complex restraint system behaved as anticipated in design and ensured that no issues which compromise safety were allowed to develop.

A risk based assessment approach was used to determine the monitoring requirements. Safety in Design processes and Construction Risk mitigation exercises carried out during design development generated a series of residual risks, largely due to the inherent variability of the embankment restraint system. Assessment of cause-effect-control scenarios led to the development of a series of key parameters requiring monitoring. The implemented monitoring may be divided into several basic categories:

- Instrumentation – the basis for confirmation of the jacking process;
- Construction ITP – procedures for excavation, formation preparation and dealing with obstructions;
- Jacking ITP – procedures for box advance, including monitoring of jacking force and rate of progress;
- Geotechnical assessment – visual inspection and categorisation of excavation face and formation. Identification of any issues and verification of design assumptions;
- Geotechnical inspection – inspection of soil slopes and surrounding works to supplement the monitoring data;
- Structural assessment – visual inspection of the critical elements of the jacking process.

The estimated settlements were used to derive trigger levels for the monitoring regime for jacking operations. For the jacked box the Green, (Alert), trigger levels were typically based on 50% of the expected in-service movements, as predicted by an analysis using worst credible parameters. The Amber, (Action), trigger levels were typically based on 80% of the expected in-service movements. The Red, (Alarm), trigger levels were based on the movement corresponding to the design Serviceability Limit State. The design for Ultimate Limit State conditions ensured that at these trigger levels, jacking remained structurally stable and the excavated face was not prone to collapse. Alarm trigger levels for general ground settlement were set at

50mm. The Alarm level movements of the Airtrain abutment and the railway overhead gantries were set at 10mm. Maximum tolerable movements, as defined by QR, were used for the assessment of the rail monitoring data and used to determine the necessity for rail retamping.

5 CONSTRUCTION OF THE SOIL IMPROVEMENT AND THE JACKED BOXES

Prior to jacking of the boxes pull out tests were carried out on sacrificial geonails to verify that the required soil nail pull out capacity could be achieved for the horizontal installed nails. Horizontal CPT tests were carried out to verify the actual, in-situ soil improvements achieved via the fracture grouting and permeation grouting. Although there was some variability of results, probably due to the influence of obstacles and the inherent variation of the original (unimproved) soil properties, the designed soil improvements were generally achieved. Box jacking therefore commenced as envisaged in the design.

The jacking installation of the first box, JB2, commenced on 22 April 2011 and the jacking of JB1 was completed using a continuous, uninterrupted, 24hour a day operation, on 26 June 2011. The best advance rate achieved was 2.5 m in a 24-hour period for the larger box. The settlements experienced were aligned with the tracks, due to the skew of the excavated face being similarly aligned. The Airtrain abutment movement due to jacking operations was insignificant. At no time did the preliminary works, the ground improvement works, or the box jacking works disrupt the railway services.

During jacking, a continuous design presence was maintained to deal with any issues that arose and to provide a regular review of the monitoring data. The jacking process was operated on a permit system, which required daily sign-off of the monitoring data and geological inspection findings. Hand vane shear tests were carried out to verify the soil properties as the excavation face advanced. The excavation face was mapped on a daily basis so that the actual soil strata could be compared to design assumptions. No significant deviations from the design assumptions were encountered.

6 FURTHER WORK

Based on the geonail trials undertaken as described above, and the success of the practical application of the geonails, further design work is being carried out with the aim of developing a reliable design method for use in determining the degree of soil improvement available from geonails of the type used in this project for a variety of sites and soils. The result of this work is expected to be the subject of another paper which will be published in due course.

7 CONCLUSIONS

The soil improvements, designed and installed to facilitate the jacking of two adjacent large tunnel boxes under the railway at Toombul in Brisbane, achieved their objectives of allowing the safe, stable and incident free process, with minimal effect on the operating railway above. The soil improvements were effective in allowing a practical excavation face and reducing settlements of the railway above. The soil improvement designs and required sequences were integrated with the required construction programme.

The success of the geonail method for soil improvement in cohesive soils has been verified by the success of this project, which was successfully completed without any disruption to rail services. A practical design method for estimating available soil improvement has been developed. The design methods will be further developed to enable their wider use.

ACKNOWLEDGEMENTS

The authors would like to express their appreciation to Thiess John Holland JV for their permission to publish this paper. They would also like to express their appreciation to all the members of the construction and design teams who were involved in the collaborative design workshops and design reviews which contributed to the success of the project.

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