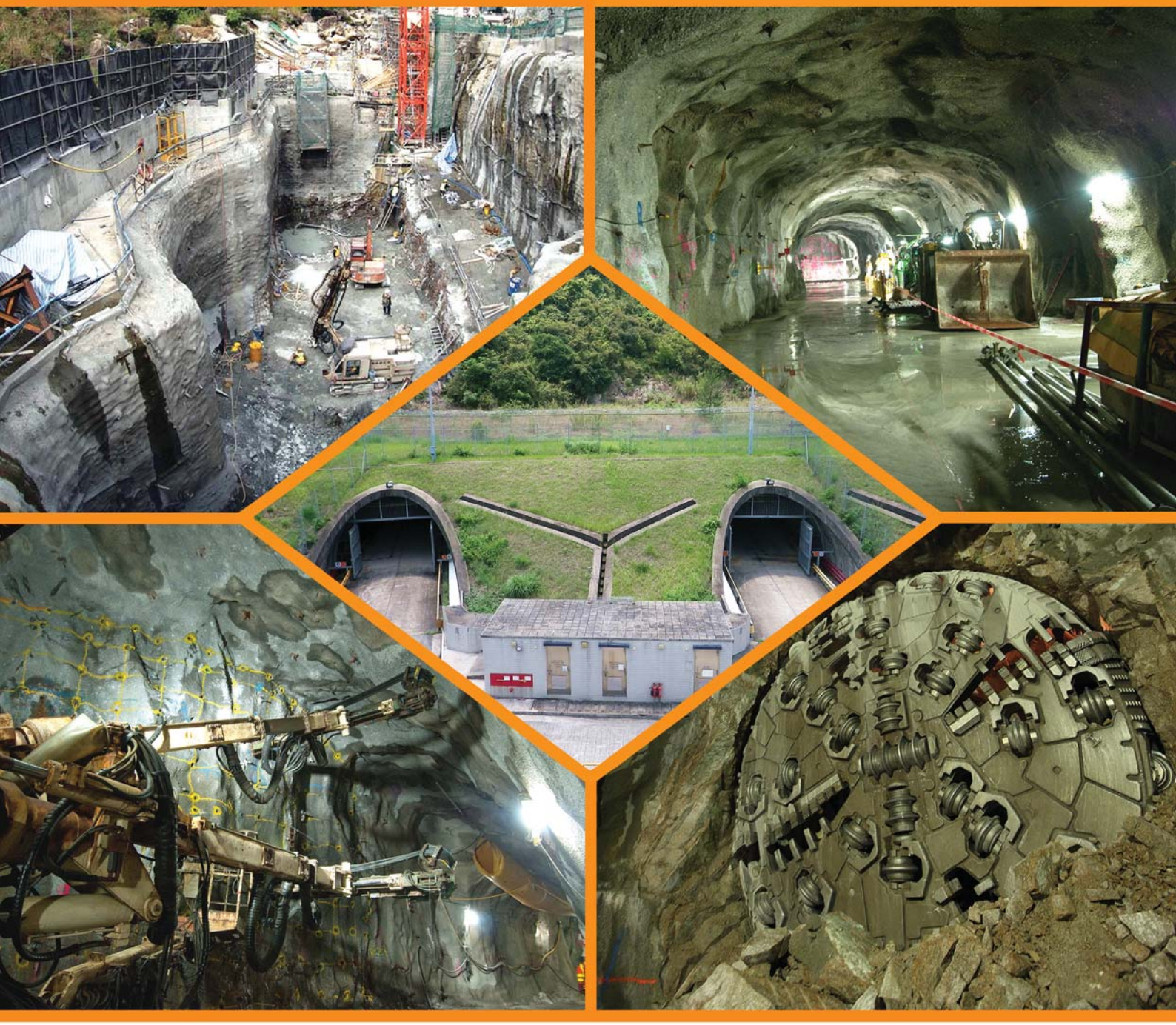
Geotechnical Aspects of Tunnelling for Infrastructure Development

25 May 2012
Hong Kong



HKIE THE HONG KONG INSTITUTION OF ENGINEERS 香港工程師學會

GEOTECHNICAL DIVISION 岩土分部

HKGES 香港岩土工程學會

Since 2003

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 additional 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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Understanding the Sustainability and Resiliency Implications of Underground Space Use

R.L. Sterling
Louisiana Tech University, Ruston, Louisiana USA

ABSTRACT

As more cities worldwide are pushed towards a greater use of underground space, there is also underway a reevaluation of how the way that we approach the planning and design of communities affects their long-term future prospects. Issues such as climate change, sustainability and resiliency have become important topics of discussion and are beginning to impact the way in which facilities are designed and communities planned. The extent to which underground space use and underground facilities promote or detract from sustainability and resiliency are examined in this paper.

1 INTRODUCTION

In the past half century, the world has increasingly turned its attention to understanding how human development can continue to fit into the environment of the planet as well as the potential impact of global environmental changes on the viability of current modes of development. A parallel interest has been to work to understand how civilian populations as well as military capabilities can be protected from modern weapons and how to protect infrastructure and facilities from the growth of terrorist actions aimed at maximum impact on people and economic activities.

Over this period, the foci of interest have shifted according to current events and current areas of public concern. In the 1960s, protection of the natural environment and ecological issues were much discussed. To these concerns were added energy supply issues in the 1970s and a more general discussion on the limits posed to human development by natural resource issues. In the 1980s, such concerns seemed to be pushed into the background as energy prices dropped, the cold war dissipated and many economies around the world boomed. The 1990s saw increasing concern about climate change and its future impact and the role that human development is playing in accelerating such change. In the 2000s, all of the above concerns have taken on greater and greater urgency due to a series of major natural and manmade disasters, the rapid rise in world population, the migration to urban areas and the economic concerns posed by increasing prices of natural resources.

The scope of these issues is extremely broad and touches all aspect of human endeavor from technological development to living patterns and social systems. However, two general terms “sustainability” and “resiliency” have emerged to describe the characteristics of human settlements that will provide a more secure long-term future for human development.

For the underground engineering community, there is an important need to develop a better understanding of the role that underground construction can play in creating or nurturing sustainable and resilient communities. This paper is intended to discuss some of the key issues in this regard.

2 SUSTAINABILITY

2.1 General issues

The definition of sustainable development created by the World Commission on Environment and Development (Brundtland, 1987) states that “sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”

In the long term, sustainability implies that the world’s population must have sufficient water, food and other natural resources for its communities to continue to exist. This, in turn, implies a balance of the needs of

the population with the ability of the natural environment to continue to supply those needs for an indefinite period of time. As these issues are examined in more detail and are applied to current living conditions, however, these relatively simple concepts become far more complicated. Without trying to be comprehensive in this short paper, some issues that must be considered in the long term are:

- Over what time scale is sustainability to be considered? Any consumption of non-renewable resources will eventually lead to exhaustion of those supplies at feasible costs.
- How can the current world population growth be reconciled with global sustainability?
- How can regional imbalances in needs and resource availability be solved? Is it feasible to support current living patterns if regions become critically short of resources or if environmental changes increase the magnitude and frequency of natural hazards? Will people need to move to safer or more resource rich areas? Will this be feasible politically?
- The rapid urbanization of the world's population requires that urban communities must provide a viable mode of living for the foreseeable future.

In his book Collapse: How Societies Choose to Fail or Succeed, Jared Diamond (2005) examines past civilizations such as the Anasazi, Maya, Viking settlements on Greenland and Pacific Island communities to understand why some past civilizations went from successful communities to extinction in a relatively short period of time. He postulates some common elements in these collapses that can be examined in terms of a five-point framework:

- The extent and reversibility of environmental damage (which in turn relates to the fragility/resiliency of the local environment)
- Climate change and its impact on food supplies, etc. (in his examinations of past societies, this included drought cycles, volcano-induced climate cooling, etc.)
- The presence of hostile neighbors ready to take advantage of a society in a weakened condition
- Decreased support from friendly neighbors that interrupts essential trading for supplies not available locally
- Whether and/or how a society recognizes and responds to the developing crisis.

The basic three pillars of sustainability are generally recognized to be *environment*, *economy* and *society*, although it is not uncommon to add a fourth pillar to reflect *local conditions and priorities*. Two other notable additions often made to the pillar list and of relevance to this paper are *natural resources* and *governance*.

It can be inferred from the above discussion that, while preserving a viable natural environment is a necessary condition for sustainability, it is not a sufficient condition to maintain the rapidly growing urban populations in livable communities.

For urban areas to be sustainable, they must be able to:

- Obtain sufficient water supplies that remain in balance with the natural recharge of these supplies
- Have a sustainable source of adequate food supplies
- Have a viable economy to provide jobs and wealth sufficient to pay for their needs
- Enable livable and equitable communities that will remain socially stable
- Have the necessary infrastructure to deliver supplies, remove wastes and support the mobility of the population that is necessary for a good economy.

2.2 Relationship of underground space use to sustainability

Table 1 provides a summary table for the sustainability-related implications of underground space.

Table 1: Sustainability implications of underground space

Issue	Advantages or mitigations	Disadvantages or limitations
Compactness	New facilities/services with low surface impact Land use efficiency and promotes compactness while maintaining livability 3-dimensional planning freedom	Local geology must be accommodated Poor knowledge of geology and existing underground structures Irreversibility considerations Support, span and access limitations
Isolation	Protects from climate, storms, fire, earthquake Protects from noise, vibration, explosion, fallout, industrial accident Provides high security	Flood protection required Psychological concerns Fire safety and personal safety concerns
Preservation	Low visual impact Preserves natural landscape, ecology Isolates hazardous materials and processes Low material degradation	Skilful design required for best effect Environmental degradation from underground construction and use itself
Life cycle cost	Land cost savings Potential sale of excavated materials Savings in specialized features Savings in maintenance, insurance and energy use for some facilities High longevity of facilities	Often high initial construction cost Higher degree of cost uncertainty Often high embodied energy Limited access may affect operating cost

At one end of the sustainability spectrum are communities where it would be possible to harvest local produce, to use only renewal energy sources and to conserve/recycle other material resources to the maximum extent possible. For such sustainability (typically in low-density population areas), the impact of underground space on sustainability could be useful but is probably not critical. Examples of contributions could include the thermal benefits of underground or semi-underground buildings in favorable climates, the use of underground food and energy storage to cushion the variability of the supply and the use of the underground for aesthetic and protection reasons (for utilities and shelters). While this is conceivable for many rural areas around the world, the geographic trends of population growth, population migration and urban area growth show that the world's population will increasingly live in urban areas and that urban areas will continue to grow in size and population (Bobylev, 2009). Thus, the more pressing issue is to understand how urban areas can be made as sustainable as possible.

For sustainability in major urban areas, underground space use does have an important impact and Parriaux et al (2007) have identified the four basic elements that constitute the underground environment as a resource as: space, materials, water, and energy. The key issues for sustainability with relation to each element are summarized below:

- Space in an urban area becomes an increasingly valuable commodity as a city grows. This leads increasingly to the placement underground of service facilities and other facilities that do not require a surface presence.
- Materials in the underground environment include the soil/rock fabric; any useful resources/minerals that can be extracted; and any hazardous materials that need to be isolated.
- Groundwater is an important natural resource of the underground that is connected to the local and global hydrological cycle.
- Energy is an underground resource including geothermal resources as well as energy conservation possibilities for earth-contact facilities.

In simple terms, underground facilities can be thought of as providing the ultimate “green roof” (Sterling et al, 2012). Facilities placed fully underground (once constructed) do not impact the surface aesthetic and can provide natural ground surfaces and flora that maintain the natural ecological exchanges of thermal radiation, convection and moisture exchange.

In greater detail, underground infrastructure contributes to sustainability of its environment in many ways: saving natural resources, including land, water, and biodiversity; reducing air pollution (mainly in the transport sector) and unnecessary visual intrusion; creating opportunities for less energy use and waste generation (compact city); creating structures less impacted by earthquakes and many other catastrophic events; and enhancing of overall landscape and environmental quality. Underground infrastructure allows a reduction of land area covered by manmade structures and creates an independent spatial layer of communication and services, including critical facilities that enhance a city’s coherence and resilience.

It is argued that underground space has been undervalued by society leading in turn to the lack of planning for the use of the resource (Bobylev, 2009). Bobylev also divides the consideration of underground space assets into: renewable versus non-renewable assets; passive versus active use of resources (e.g. vibration isolation vs. mineral extraction); and the degree of “rivalness” and/or excludability of the use of the resources (can multiple functions co-exist or does the use of underground space for one preclude other possible uses).

In summary, underground space use can be an important tool in creating and/or reshaping urban areas to promote sustainability in concert with economic viability and livability.

3 RESILIENCY

3.1 General issues

Resiliency in the context of this paper is considered as the ability of a community or some aspect of that community to withstand a catastrophic event or, if such an event cannot be withstood, to return the community to effective functioning as quickly as possible after the event. While earthquakes, hurricanes, tornados, tsunamis, floods, terrorist attacks, etc. are key events in the analysis of resiliency, the concept also applies to disruptive changes that may occur over longer periods of time such as those that may derive from climate change, sea level rise, and other environmental changes. In the longer term context, resiliency concerns start to merge with sustainability concerns because it would be hard for a community to be sustainable if it could not cope with expected irreversible changes in its environment.

Studies that look at the history of civil engineering failures and their prevention provide useful insights into how and why some types of failures occur. Delatte (2006) for example looked at well documented failures of individual structures and suggested that common elements in many of the failures were:

- Pushing the envelope of existing knowledge and practice
- Not paying attention to early signs of failure
- Site supervision problems during construction
- Lack of redundancy and robustness in design
- Maintenance (and inspection) problems.

He also indicated that the typical overall design approach to avoid failure is to:

- Figure out everything that can possibly go wrong.
- Make sure that everything that can possibly go wrong doesn't happen.

When looking at the resiliency of communities or regions, however, and extending resiliency concerns to longer term issues such as climate change, the complexities multiply rapidly and the will to make massive investments against poorly understood threats is often lacking. In this broader context, resiliency evaluations must consider the social functioning of a community as well as the physical resistance of structures and infrastructure systems. The difficulty of reestablishing social systems after major catastrophes can be seen in a number of recent disasters (New Orleans in the aftermath of Hurricane Katrina and Japan in the aftermath of the earthquake/tsunami and nuclear plant failure). In New Orleans, where the author was involved in a study of the damage to underground utility systems, it was clear that the disruption to key utility services prevented or discouraged the return of residents and businesses and that the lack of presence of the community members greatly hampered both the cleanup and the restoration of normal functioning of the community. Evaluation of resilience must also consider the resilience of the local or regional environment in which the community exists

– exemplified also in Louisiana in the loss of barrier islands that protect the coastal ecosystem. Finally, a key issue that has emerged in recent years from the examination of cascading failures of infrastructure systems is that the issue of fragility of individual systems and interdependencies among all infrastructure systems is an increasing problem as systems become more highly interdependent and automated (Nelson & Sterling, 2012).

For coastal and low lying regions, key resiliency concerns are the potential effects of hurricanes, tsunamis, the effect of exceptional rainfall events and the extent to which these will be exacerbated by climate change. The flooding in Bangkok and the surrounding regions in 2011 highlighted the interaction between urban growth and changes in vulnerability to extreme weather events. Because of the urbanization and channeling of water flow that has occurred in recent decades, the flooding was intensified in areas that were less well protected – changing the perception of damage from that of a natural uncontrollable event to that of a result of conscious decisions to protect some areas and allow damage to others.

3.2 Relationship of underground space use to resiliency

Many types of underground space use can impact resiliency because of the isolation provided by the covering soil or rock from the catastrophic events that occur on the surface. Underground structures typically provide an excellent resistance to events such as earthquakes, hurricanes, tornados, external fires, external blasts, radiation and other terroristic threats (Parker, 2008). Earthquake resistance has been demonstrated in many earthquakes including the Loma Prieta earthquake in San Francisco where the transit system was inspected and put back into service in less than half a day whereas much of the city was immobilized for many months. However, underground structures are not immune to damage from catastrophic events and some of the principal issues both positive and negative are outlined in Table 2 (in general terms only and certainly with caveats). For example, shallow underground utility systems, despite their protected location, can be damaged in a variety of ways by major natural catastrophes – leading to the community disruptions discussed in the previous section. For a discussion of the impact of hurricanes and flooding on buried utility systems, for example, see Allouche et al (2006) and Chisolm (2007).

Table 2: General advantages and disadvantages of underground facilities with respect to catastrophic events

Type of Event	Advantages or mitigations	Disadvantages or limitations
Earthquake	Ground motions reduce rapidly below surface	Fault displacements must be accommodated
	Structures move with the soil	Instability in weak materials or poor configurations
Hurricane, Tornado	Wind loadings have minimal impact on fully buried structures	Damage to shallow utilities from toppling of surface structures such as trees and power lines
Flood, Tsunami	Ground provides protection from surge and debris flow	Extensive restoration time and cost if the structure is flooded
External fire, blast	Ground provides effective protection	Entrances and exposed surfaces are weaknesses
External radiation, chemical/biological exposure	Ground provides additional protection	Appropriate ventilation system protections required
Internal fire, blast	Limited extent of damage with appropriate compartmentalization	Confined space increases internal damage and personnel risk
Internal radiation, chemical/biological releases	Limited extent of damage with appropriate compartmentalization	Confined space increases internal damage and personnel risk

For most types of underground structures, however, even in the case of hurricanes and floods, provided the entrances are well protected and/or sealed, the loadings on the underground structures are well understood and easily managed compared to the surge, wind and impact loadings for aboveground structures. However, the impact of internal fire or explosion is typically more serious in an underground structure than in a surface structure.

3 CONCLUSIONS

Today, new demands are being placed on future city plans – a city should be as sustainable as possible (nurturing a viable economy while saving the environment, minimizing energy consumption and preserving options for future generations) and as resilient as possible (able to withstand and/or quickly recover from a variety of natural and man-made disasters). With proper planning and design, underground space use can contribute to both the sustainability and resiliency of urban areas.

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Setting Operating Pressures for TBM tunnelling

J.N. Shirlaw
Golder Associates (Singapore) Pte. Ltd., Singapore

ABSTRACT

Pressurised face TBMs, such as slurry and Earth Pressure Balance (EPB) machines, are now widely used. These machines have the ability to control a wide variety of potentially unstable ground conditions. The control of ground movement that can be achieved is particularly valuable in urban tunnelling, to minimise the risk of damage to buildings and utilities. In recent years the maximum size of TBMs has increased rapidly, requiring ever increasing degrees of control to minimise the volume loss and thus the settlement due to tunnelling. In order to achieve the required control, it is essential to carry out geotechnical calculations to establish target operating pressures. The site investigation should establish the variation in the ground and groundwater conditions along the alignment. Calculations based on the investigation data can then be used to establish how the TBM operating pressures should be adjusted to cater for those variations proactively. The sensitivity of the results to the input parameters and the limitations of current methods are discussed, in the context of the increasingly stringent settlement criteria required. Current practice includes a relatively limited theoretical basis for predicting volume loss due to tunnelling in coarse grained soils, and the magnitude of consolidation settlements due to tunnelling. It is also common to significantly underestimate the risks associated with pressurised TBM tunnelling.

1 INTRODUCTION

Pressurised face Tunnel Boring Machines (TBMs), such as slurry and Earth Pressure Balance (EPB) machines, are widely used for tunnelling. These TBMs have the capability to control ground movements in a wide variety of potentially unstable ground conditions. The control of ground movement that can be achieved with these types of TBM is particularly valuable in urban tunnelling, to minimise the risk of damage to buildings and utilities. In recent years the maximum size of TBMs has increased rapidly, requiring ever increasing degrees of control to minimise the volume loss and thus settlement due to tunnelling. A requirement to maintain volume loss to below 1% is now common, and maximum values of 0.75% or even 0.5% have been specified on some recent projects. Such specifications can, in principle, be met in most ground conditions. However, to meet such targets consistently requires informed selection of the TBM, careful planning of the work, and consistently excellent tunnelling.

The very tight control over settlement represented by current specifications can be illustrated by comparing the expected volume losses outlined above, with those measured over pressurised face TBMs twenty to thirty years ago. Particularly large volume losses were recorded over EPB TBMs working in soft, near normally consolidated clay. As summarised in Shirlaw (1994), the assessed values of volume loss on four documented projects ranged from 0.5% to 16%, based on surface settlements that ranged from 5 mm to 210 mm. The measured volume loss was commonly greater than 3%. Careful control of all of the potential sources of volume loss is necessary to achieve the much lower values for volume loss that are now commonly required.

There are three primary areas of ground movement towards a pressurised TBM: at the face, along the shield skin and at the tail void. The gaps around the shield skin and at the tail void are illustrated in Plate 1. Even if the face is fully supported, there is a potential for significant volume loss at the shield skin and tail void gaps. Examples of the potential volume loss if the shield skin and tail void gaps close fully are given in Table 1. These examples come from TBMs in tender proposals.

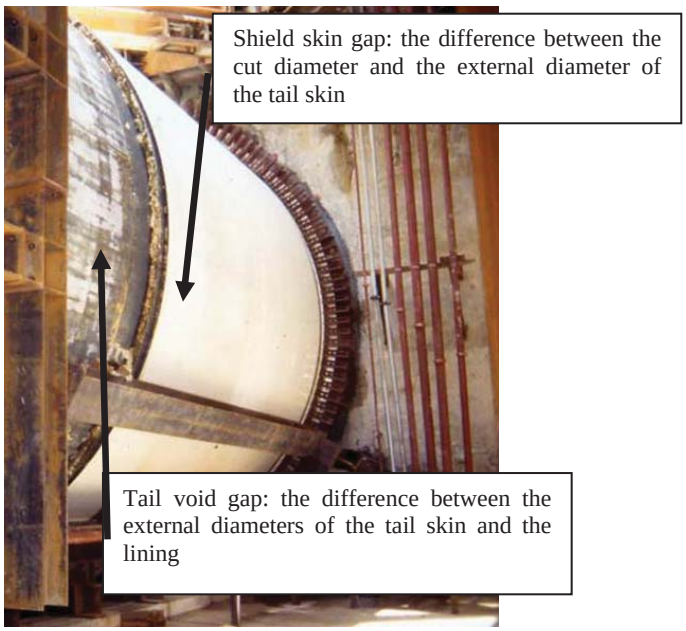


Plate 1: The shield skin and tail void gaps

Table 1: Examples of potential volume loss at shield skin and tail void gaps

	TBM cut diameter (m)	Tail skin diameter (m)	Lining outside diameter (m)	Shield skin gap, potential volume loss (%)	Tail void gap, potential volume loss (%)
TBM 1	2.952	2.914	2.8	2.625	7.671
TBM 2	6.68	6.60	6.35	2.439	7.432
TBM 3	16.82	16.74	16.4	0.958	4.021

It is evident that the larger of the two gaps is the tail void gap, but that the tail void and shield skin gaps are both of concern when compared with the typical targets for volume loss discussed above. The degree of closure of these gaps will depend on the nature of the soil, but in weak or soft soils complete closure is possible unless a support pressure is maintained.

Although pressurised TBMs can provide a supporting pressure at the face of the tunnel, it has been found that conventional grouting, through the rings, of the gap at the tail void is often of limited effectiveness. Much of the measured settlement in the examples listed in Shirlaw (1994) was the result of partial or complete closure of the tail void. The introduction of simultaneous tail void grouting has greatly improved the effectiveness of the tail void grouting. 'Simultaneous' grouting is grouting that is carried out simultaneously with the advance of the TBM, using grout pipes that are laid along the shield skin and below the tail seals (Plate 2). The effectiveness of this method of grouting can be seen in a uniform annulus of grout, as shown in Plates 3 and 4.

For slurry shields the slurry generally flows from the face to the shield skin gap, transmitting some of the support pressure provided at the face. This support pressure is augmented, towards the tail of the shield, by the pressures due to grouting (Bezuijen & Bakker, 2008). For EPB TBMs the transmission of pressure from the face to around the shield skin is uncertain, due to the nature of EPB spoil. However, it is common for modern EPB TBMs are equipped with a bentonite injection system, so that thick bentonite slurry can be injected into the shield skin void. This is done to ensure that the support pressure can be maintained around the shield skin.

The provision of an adequate face pressure cannot be taken for granted. For a slurry shield this depends on the quality of the bentonite slurry; additives may be required, particularly in coarse grained soil. For an EPB TBM the screw conveyor has to be of adequate length to provide the required pressure difference between the face and the discharge gate, and the spoil has to be conditioned to form a suitable EPB paste.

A properly specified pressurised TBM can provide support to the ground at the face, along the shield skin and at the tail void, and has the potential to keep the settlement over the tunnel to a reasonable minimum. However, to fully realise this potential, the correct operating pressures have to be used throughout the tunnel drive. By calculating the required operating pressures along the alignment, the need to change pressure in

response to changing geotechnical and hydrogeological conditions can be predicted; the calculated pressures can then be fine-tuned by considering the results of settlement and other monitoring during tunnelling, using the observational method.

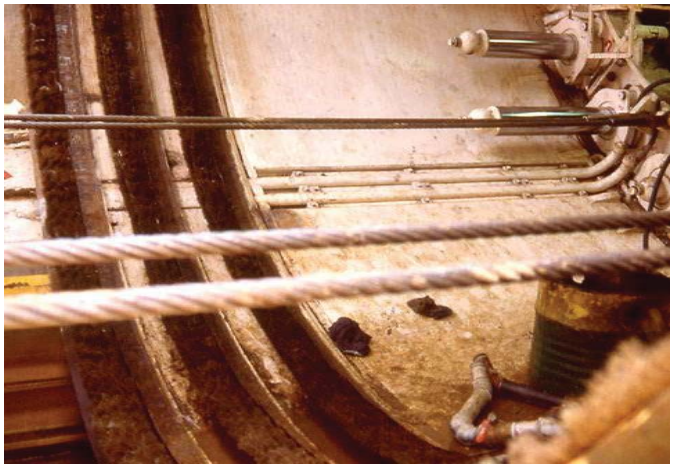


Plate 2: Pipes laid along the shield skin



Plate 3: Grout around a segment in weathered rock

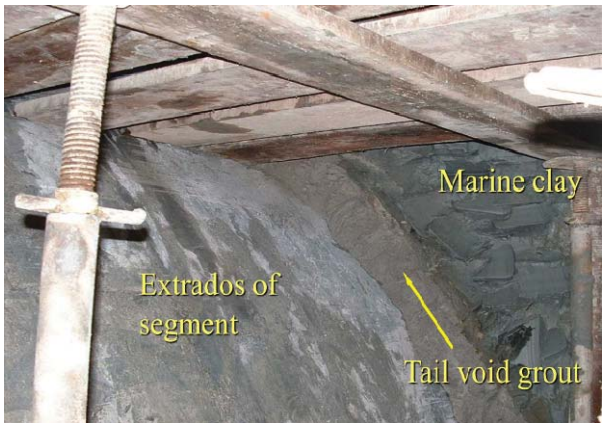


Plate 4: Grout around a segment in soft marine clay

The potential problems that can result from not calculating and planning face pressures can be seen from some of the examples of sinkholes over EPB driven tunnels in Singapore given in Shirlaw et al (2003). Several of the sinkholes were related to interfaces between hard and soft soils; these interfaces had been identified prior to tunnelling, but the necessary adjustment to the face pressure were not been made until it was too late to avoid a large increase in settlement or a sinkhole.

In addition to normal EPB or slurry mode operation it is also necessary to consider the support pressure needed for intervention into the excavation chamber of the TBM, to change cutting tools or carry out other maintenance. It is common to use compressed air to provide a support pressure during interventions in soft or water bearing ground. The distribution of pressure over the face is different for compressed air than for EPB or slurry pressure, so a separate set of calculations is required for compressed air interventions.

2 BASIC METHODS FOR ASSESSING OPERATING PRESSURES

2.1 Limit states for pressurised TBM tunnelling

Following modern geotechnical practice, the limit states for tunnelling can be divided into ultimate limit states, associated with failure, and serviceability limit states, associated with unacceptable settlement, heave, lateral movement or other unacceptable effects on third parties. There are a number of potential mechanisms that need to be considered in the analysis, as illustrated in Figures 1 and 2.

In Figure 2, SLS 3 and 4, for lateral movement, are generally of concern only when tunnelling adjacent to piles, tunnels, or other underground structures.

The loss of slurry, foam or compressed air up an existing borehole or well is considered an SLS (SLS 5), as the initial effect of the loss is to cause inconvenience at the surface; the loss may be followed by collapse (ULS) due to loss of pressure, but this would then fall under ULS 1.

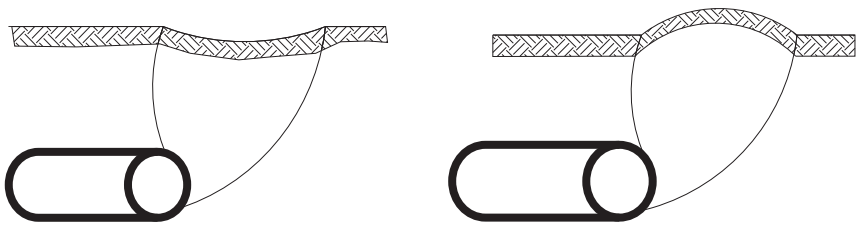
No separate ULS or SLS is identified for tail void grouting, as inadequate or excessive tail void grouting could be a factor in all of the ULS or SLS mechanisms outlined in Figures 1 and 2. Inadequate or excessive tail void grouting could also result in failure to satisfy ULS for the tunnel lining; ULS and SLS for the tunnel lining are not included in Figures 1 and 2.

In order to demonstrate that the ULS and SLS mechanisms shown in Figures 1 and 2 are satisfied, it is typically necessary to define the operating pressures listed in Table 2.

Table 2: Operating pressures for pressurised TBMs

Pressure to be defined
Minimum face pressure, Slurry or EPB mode
Maximum face pressure, Slurry or EPB mode
Compressed air pressure for intervention, chamber partially empty
Compressed air pressure for intervention, chamber completely empty
Injection pressure around TBM skin (typically EPB TBMs only)
Minimum tail void grouting pressure
Maximum tail void grouting pressure

The minimum and maximum face pressure in slurry or EPB mode can also be defined by setting a target pressure and an acceptable range of variation, +/-v, from the target pressure.



ULS1- Loss of ground into the tunnel creating a sinkhole or area of very large settlement.

ULS2- Rupture of the overburden, leading to a very large heave and/or major loss of slurry, EPB spoil or compressed air.

Figure 1: Examples of Ultimate limit states for pressurised TBM tunnelling

Note: The limit states apply to slurry or EPB tunnelling, and interventions, but the limit states for the tunnel lining are not included.

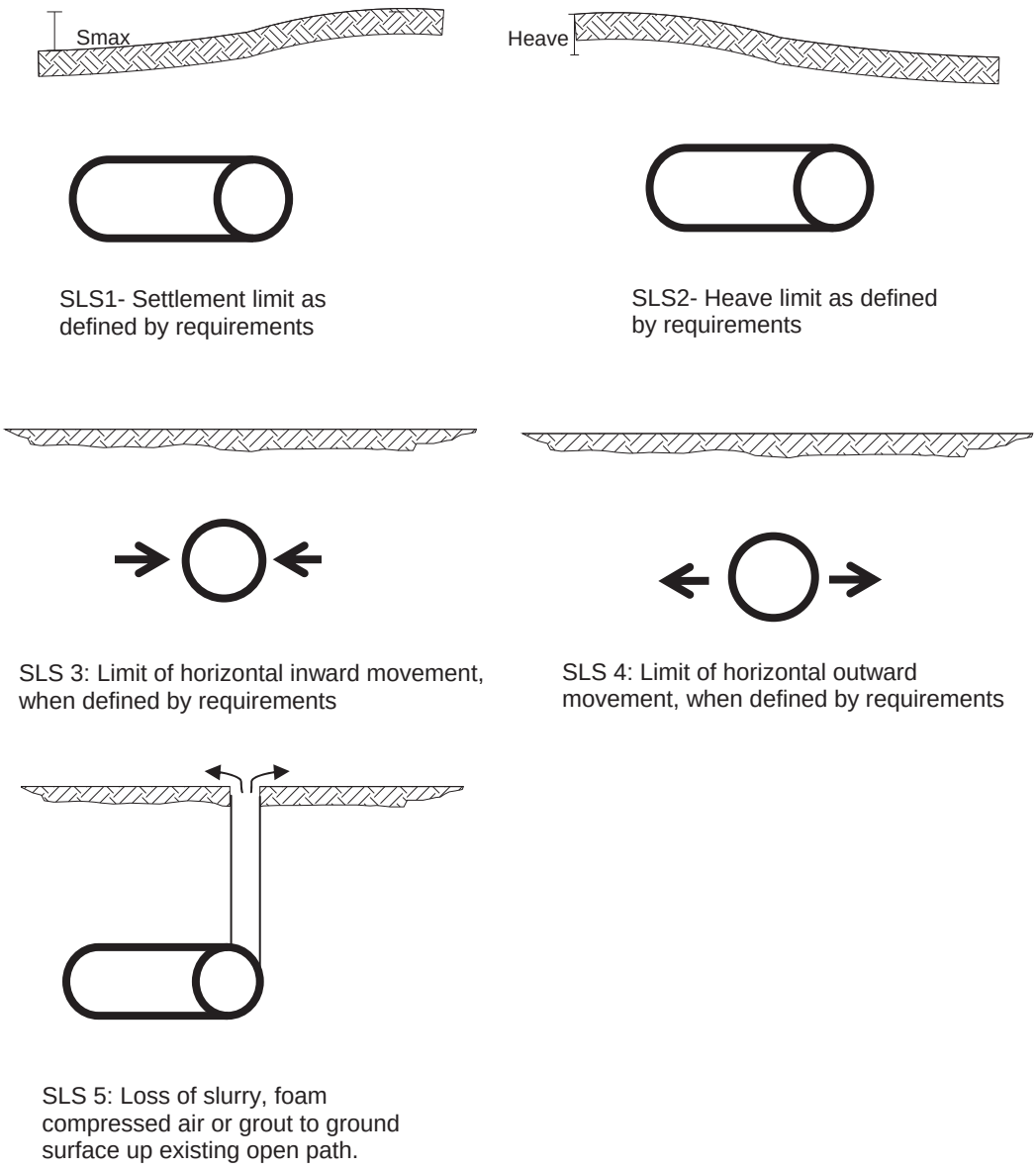


Figure 2: Examples of serviceability limit states for tunnelling
Note: The limit states shown apply to slurry or EPB tunnelling, and interventions, but the limit states for the tunnel lining are not included.

2.2 Operating pressure calculations

It is necessary to define the operating pressures for every ring of tunnel advance; however, it is not usually necessary to carry out separate calculations for every ring. The change in the required pressure between adjacent rings is generally very small, much smaller than the typical +/- 0.1 to 0.3 bar fluctuation in face pressure that is commonly assumed for pressurised face tunnelling. It is usually adequate to provide calculations such that the change in pressure between adjacent calculations is 0.1 to 0.2 bar. This typically results in 20 to 50 sets of calculations per kilometre of tunnelling, with a greater intensity of calculations in areas of major changes in ground conditions.

Although the planning for the tunnelling can include designated locations for interventions (‘planned interventions’) most interventions occur at short notice (‘unplanned interventions), and depend on the assessed condition of the cutting tools and other parts of the TBM. It is common to calculate the required compressed air pressure at each of the points where the slurry/EPB pressure is calculated, to provide guidance in case of an intervention at short notice. Interventions may be carried out with the excavation chamber partially or fully

emptied, depending on the work required, and the compressed air pressure required is typically calculated for both scenarios.

Methods based on the results of limit equilibrium methods, or on the results of the testing of model tunnels, can readily be set up to provide the necessary pressures to satisfy the various ultimate and serviceability limit states, using a spread sheet. GEO Report 249 (GEO, 2009) was primarily based on such methods, although it was envisaged that more detailed calculations, such as the use of numerical analysis, can be required in special situations. The use of the simple spread sheet methods for calculation and numerical analysis should not be seen as competing alternatives, but as complementary.

The assessment of operating pressures using published charts derived from limit equilibrium calculations and the results of the testing of model tunnels is outlined in GEO Report 249. It is not intended to duplicate GEO Report 249 here, but to provide some complementary commentary based on experience in the use of these methods. Examples of face pressure calculation are provided below, to demonstrate some of the issues involved in the determination of the target pressures for tunnelling.

2.2.1 Effective stress calculations

The effective stress methods outlined in GEO Report 249 are based on the charts in Anagnostou & Kovari (1996). For the ULS analysis, if the face pressure is greater than the insitu water pressure in the ground, the target face pressure at tunnel axis level, P_t, can be evaluated from:

$$\text{Target } P_t = \text{Pressure due to water at crown} + F_O \gamma' D + F_1 c' + \gamma_{SL} (D/2) + T_\gamma q + v \quad (1)$$

Where:
F_O and F_1 are dimensionless coefficients obtained from charts in Anagnostou & Kovari (1996), after the application of the appropriate partial factor
γ' is the submerged unit weight of the soil
D is the diameter of the TBM
γ_{SL} is the unit weight of the slurry, for a slurry TBM, or EPB spoil, for an EPB TBM
T_γ is a dimensionless coefficient based on Atkinson & Mair (1981)
q is the average factored surcharge pressure at ground surface
v is the maximum allowable fluctuation of the slurry pressure from the target pressure

The pressure to meet an SLS of approximately 1% volume loss can be evaluated from:

$$\text{Target } P_t = \text{Pressure due to water at crown} + F\gamma' D + \gamma_{SL} (D/2) + T_\gamma q + v \quad (2)$$

Where: F is a dimensionless coefficient used for the design of rigid tunnel linings, and depends on the relative density of the soil.

The charts from Anagnostou & Kovari (1996) are not suitable for use in fine grained soils, as the charts are based on the failure surface proposed by Horn (1961). The failure surface in fine grained soils (see Mair & Taylor, 1997) is quite different to that proposed by Horn, so no reliance can be placed on the charts in fine grained soils.

2.2.2 Total stress calculations

The total stress methods outlined in GEO Report 249 are based on the results of the testing of model tunnels in a geotechnical centrifuge, as reported by Kimura & Mair (1981). For the ULS analysis, the target face pressure, P_t, can be evaluated from:

$$P_t = \text{Total overburden pressure at tunnel axis level} + \text{surcharge} - (S_u N_C) + v \quad (3)$$

Where:
N_C is the stability number at collapse
S_U is the factored undrained shear strength.

N_C can be assessed from Kimura & Mair (1981) (reproduced in O'Reilly (1988)). The value for N_C varies with the ratios C/D and P/D, P being the length of the unsupported heading as defined in Kimura & Mair.

The pressure to meet an SLS can be evaluated from:

$$P_t = \text{Total overburden pressure at tunnel axis level} + \text{surcharge} - (S_u N_C LF_t) \quad (4)$$

Where:
S_U is the unfactored undrained shear strength
q is the unfactored average surcharge at ground surface
LF_t is the target load factor. The target load factor can be assessed from a chart in Kimura & Mair (1981), based on the target volume loss, after allowing for the volume loss at the tail void gap.

It should be noted that for the examples of total stress calculations at SLS given below, the variation in the face pressure (v) is not included in the equation. In this respect the example analyses are different to the recommendations in GEO Report 249. Provided that the load factor is low, the variation in the face pressure will have little effect on the settlement over the tunnel, which will be governed by the average face pressure applied. This is different to the SLS case in effective stress, as the methods used to determine a pressure to satisfy the SLS cases in effective stress are semi-empirical, and so the variation has to be included for consistency.

2.3 Examples of pressure calculations – general

In order to illustrate the sensitivity of the face pressure calculations to various input parameters, sample calculations are summarised below. These calculations are for a typical EPB driven subway tunnel. The shield and lining dimensions are shown in Figure 3. The depth from ground surface to tunnel axis level was taken as 20 m, and the length of the shield as 8.5 m.

Calculations were carried out for tunnelling in a range of clays, from soft to stiff, and sands, from loose to dense. The parameters for the soils are provided in Table 3. The highest likely water table is set at 1m below ground surface, the general surcharge is taken as 15 kPa, and the allowable variation in the face pressure as +/- 0.2 bar. The unit weight of the EPB spoil is taken as 14 kN/m^3 throughout. The reduced unit weight compared with the natural unit weight of the soil is to allow for the addition of conditioning agents. In actual calculations this should be assessed based on the planned nature and volume of conditioning agents, and can be checked based on the distribution of pressure from the top to the bottom of the excavation chamber.

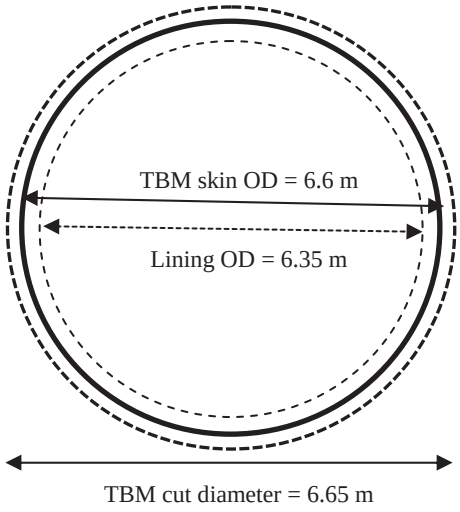


Figure 3: Dimensions for example tunnel

Target volume losses of 1%, 2% and 3% are used in the total stress SLS examples. An allowance is made for a constant 0.5% volume loss at the tail void gap in those calculations. For the effective stress SLS example, the target volume loss is 1%.

Table 3: Soil parameters used for example calculations

Soil type	Bulk unit weight (kN/m^3)	ϕ' (Degrees)	c' (kPa)	S_u (kPa)
Soft clay	16			30
Firm clay	18			55
Stiff clay	20			80
Loose sand	17	30	0	
Medium sand	18	33	0	
Dense sand	19	36	0	

Before undertaking the detailed analysis, it is a useful preliminary step to assess the various potential sources of settlement, and compare the potential movement with those that are allowable.

The difference between the cut diameter, of 6.65 m, and the external diameter of the tail shield, 6.6 m, is termed the 'shield skin gap'. The difference between the external diameter of the tail shield, 6.6 m, and the external diameter of the lining, 6.35 m, is termed the 'tail void gap'. The volume of the example gaps is given in Table 4, as is the potential volume loss if these gaps close fully.

Table 4: Shield skin and tail void gaps for example TBM

Gap	Potential volume (m^3/m) of tunnel	Potential volume loss if gap closes fully (%)
Shield skin gap	0.52	1.52
Tail void gap	2.54	7.43
Total minimum gap	3.06	8.95

The gaps given in Table 4 are for illustration only. The total gap can be larger in practice, as it could increase due to one or more of the following factors:

- If extendable over-cutters are used to increase the overcut; this may be done to aid steering in variable ground conditions
- When tunnelling around curves
- When tunnelling with overhang or look-up

It can be seen from Table 4 that, even excluding these additional factors, the potential volume loss at the shield skin and tail void gaps is large in the context of the typical requirements for volume loss that are now commonly required for urban tunnelling. In order to achieve those requirements it is generally necessary to ensure that an effective support pressure is provided, at all times, around the shield skin and at the tail void, as well as at the face.

For the examples of total stress calculations given below, the ULS calculations have been based on P/D = 0, i.e. that failure is considered only at the face of the TBM. The reason for this is that settlement is not a concern at ULS. Therefore, the shield skin gap can be allowed to close, so that the ground around the periphery of the TBM is supported by the shield skin, and failure can only occur at the tunnel face.

For the total stress calculations at SLS, the calculations have been based on P/D = length of shield/shield diameter. It is assumed that the requirements for the control of settlement are sufficiently tight that it would be unacceptable to allow total closure of the shield skin gap. This is the case up to about 3% volume loss, for the example TBM in Table 4. If the allowable settlement was much higher, say 5%, then the shield skin gap could be allowed to close. However, it is unlikely that such a value for volume loss would be considered 'allowable' for urban tunnelling. In order to maintain the shield skin gap open is necessary to ensure that a support pressure is provided in the gap. For the example EPB TBM it would be necessary to inject bentonite slurry around the shield skin to ensure a controlled pressure in this gap, unless the ground is very competent.

The residual volume loss at the tail void gap, after allowing for the beneficial effects of grouting, is assumed rather than calculated. There is relatively little data on what might be an appropriate value to use for this portion of the volume loss. Gens et al (2011) report measured values of up to 0.8% for the volume loss at

the tail void gap for EPB tunnelling in soft deltaic deposits. The actual volume loss at the tail void gap will depend on the type and properties of the grout, the number of grouting ports, the grouting pressure, the consistency of the grouting and the type of ground. For stiffer clays and for sands, the reported total volume loss due to pressurised TBM tunnelling is often very small, typically 0.5% or less, suggesting that the grouting is generally significantly more effective in controlling settlement in these soils than in soft clay.

For the example calculations for clay that are summarised below, a volume loss at the tail void of 0.5% is assumed for all cases. This is added to the calculated volume loss at the face and along the tail skin.

2.3.1 Examples of effective stress calculations

The sample calculations are based on a TBM with the dimensions shown in Table 4 and Figure 3. The calculations follow the recommendations of GEO Report 249. The ULS calculations are based on the charts in Anagostou & Kovari (1996), while the SLS calculations are carried out using the empirical 'Proctor and White' method. The pressures given are the target pressures at the axis level of the tunnel.

The calculated target pressures at tunnel axis level that were obtained from the example calculations are summarised in Table 5. The target pressures are given as absolute values, and, in parenthesis, as a percentage of the total overburden pressure (excluding surcharge).

From the results in Table 5, it is evident that there is only a small difference between the pressure required to satisfy ULS and SLS in sand.

Table 5: Calculated target face pressure, in bar, for the example tunnel in sand.

Soil type	ULS	SLS, Vl of 1%
Loose Sand	2.4 (74.9%)	2.5 (78.1%)
Medium Dense Sand	2.39 (66.3%)	2.45 (68.2%)
Dense Sand	2.38 (59.4%)	2.39 (59.7%)

Note: Values in parenthesis are the target pressure as a percentage of the total overburden pressure, excluding surcharge.

Table 6: Components of the calculated face pressure to satisfy ULS, for the example tunnel in loose sand

Component of required pressure	Pressure to meet ULS at tunnel axis (bar)	Percentage of ULS target pressure (%)
Water pressure at axis	1.9	79
Soil pressure	0.15	6.3
Pressure due to surcharge	0.02	1
Difference between pressure due to water and spoil between crown and axis	0.13	5.4
Allowance for variation in pressure	0.2	8.3
Total	2.4	100

The various components of the target face pressures for the example in loose sand are summarised in Table 6. It can be seen that the water pressure is the dominant factor in the target face pressure, requiring 79% of the total face pressure to balance the water pressure. The pressure required to support the effective stress in the soil, including the effect of the surcharge pressure, is relatively small, at just 7.3% of the target face pressure. It is for this reason that the variation in the face pressure is included in both ULS and SLS calculations, for the effective stress calculations. If the variation were not included, the fluctuation in the applied face pressure could result in the pressure applied at the tunnel crown being regularly lower than the insitu water pressure. This would induce transient seepage towards the face and risk loss of ground.

2.3.2 Examples of total stress calculations

The results of the example calculations are summarised in Table 7. In addition to the absolute values of the calculated target face pressure, the target pressure is given as percentage of the total overburden pressure (excluding surcharge), in parentheses. The values in Table 7 are also shown graphically in Figure 4.

Table 7: Calculated target face pressure, in bar, for the example tunnel in clay. Values in parenthesis are the target pressure as a percentage of the total overburden pressure, excluding surcharge

Soil type	ULS	Vl of 3%	Vl of 2%	Vl of 1%
Soft Clay	1.86 (58%)	2.38 (74%)	2.5 (78%)	2.82 (88%)
Medium Clay	0.86 (22%)	1.97 (55%)	2.19 (61%)	2.78 (77%)
Stiff Clay	0	1.55 (39%)	1.88 (47%)	2.74 (68%)

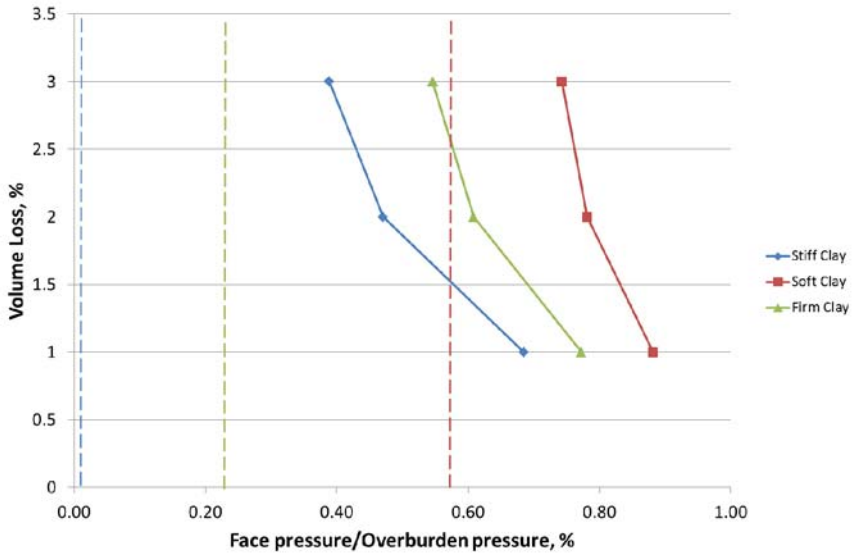


Figure 4: Results of the calculations for the example tunnel in clay
Note: The solid lines are the relationship between face pressure and total overburden pressure, while the dashed lines are the pressure required to satisfy ULS.

It can be seen that very high face pressures, relative to total overburden pressure, are required to minimise the volume loss in soft clay. Such pressures would be reasonably consistent with the data provided in Shirlaw et al (2003) for EPB tunnelling in marine clay in Singapore, after allowing for the more stringent control of ground movements around the shield skin and at the tail gap that are assumed here, compared with the Singapore data.

The calculated target pressure for 1% volume loss in the soft clay example would exceed the in-situ horizontal stress: for a Ko of 0.63, the total horizontal pressure would be approximately 85% of the total overburden pressure. In these conditions it is possible to have settlement at the ground surface, but outward horizontal ground movement at tunnel level. This reflects the difference in the insitu ground pressures, which are anisotropic, and the fluid pressures exerted during tunnelling.

With the assumed allowable variation in face pressure of +/- 0.2 bar, the use of the target pressures for soft clay from Table 7 would imply minimum and maximum planned face pressures as shown in Table 8. As previously, the values in parenthesis are the minimum and maximum planned pressures expressed as a percentage of the total overburden pressure, excluding surcharge.

Table 8: Minimum and maximum planned face pressures, based on the target pressures in Table 7

	ULS	Vl of 3%	Vl of 2%	Vl of 1%
Minimum planned face pressure	1.66 (52%)	2.18 (68%)	2.3 (72%)	2.62 (82%)
Maximum planned face pressure	2.06 (64%)	2.58 (81%)	2.7 (84%)	3.02 (94%)

Note: Values in parenthesis are the target pressure as a percentage of the total overburden pressure, excluding surcharge.

In this particular example, the maximum planned face pressure is (just) lower than total overburden pressure even for the target volume loss of 1%. Therefore ULS 2 and SLS 2 (for heave), as shown in Figures 1 and 2 are satisfied, as heave would not occur if the face pressure is less than the total overburden pressure. However, shallower or larger tunnel than the one used in the example would be more sensitive to heave.

Comparing Tables 5 and 7, it can be seen that the face pressure required to satisfy ULS in sand with a high water table is consistently higher than the pressure required to satisfy ULS in the fine grained soils. However,

the highest face pressures are required to satisfy an SLS requirement of 1% volume loss in soft clay. The calculated target face pressure of 88% of total overburden pressure is reasonably consistent with the report by Osborne et al (2008) of EPB tunnelling in marine clay in Singapore. At one location there was an old building, in poor condition, on shallow foundations. Osborne et al (2008) reported that the volume loss was controlled to 1% by the use of a face pressure of between 80% and 100% of the total overburden pressure.

2.4 Sensitivity to errors in key parameters

2.4.1 Effective stress calculations

For the effective stress calculations, if the water table is high, as it generally is in Hong Kong and Singapore, then the calculated face pressure is largely determined by the groundwater pressure. In the example above, 79% of the target pressure required to satisfy ULS 1 was to balance the water pressure. The calculated component from the earth and surcharge pressures was 7.3% for the ULS calculations and 10.7% for the SLS calculations. In the ULS calculations the margin provided by the factor of safety on the effective strength of the soil is very small, so it is essential that the design water pressure reflects the 'highest likely' pressure, and that there is confidence that this is the case throughout the tunnel alignment. For the example, if the water pressure was actually 2m (for ULS) or 3m (at SLS) of head higher than that used in the calculations, the face pressure applied at crown would regularly drop below the insitu water pressure, inducing seepage towards the tunnel face.

Where the water table is close to the ground surface level, the water pressure is the dominant factor in the face pressure calculations. However, it is common for significantly less effort to be devoted to establishing the water pressure in investigations and interpretation than is spent on the soil parameters.

2.4.2 Total stress calculations

For the total stress calculations, the example for soft clay was used to assess the sensitivity of the calculated volume loss to errors in two of the key input parameters: the unit weight and the undrained shear strength of the soil. The results of the assessment for the example of 3% volume loss in soft clay are summarised in Table 9.

Table 9: Sensitivity of the SLS calculations for tunnelling in soft clay to a 2% error in the unit weight of the soil or a 7% error in the undrained shear strength

	Minimum value	Change in calculated volume loss	Maximum value	Change in calculated volume loss
Unit weight	15.68 kN/m^3	-11%	16.32 kN/m^3	+13%
Undrained shear strength	27.9 kPa	+15%	32.1 kPa	-11%

The calculations summarised in Table 9 included a constant volume loss of 0.5% at the tail void. The percentage change in the volume loss would have been significantly higher if only the volume loss at the face and along the shield skin gap had been considered.

Overall, it can be seen that even a small margin of error in the unit weight and the undrained shear strength of the soil could lead to the settlement being +/- 30% of the target value. This margin only allows for practical limitations in deriving the input parameters for the calculations. In practice the actual volume loss will also be affected by varying levels of workmanship in the tunnelling. The calculations for the target pressure should be understood in this context: the calculations will not give the exact value of face pressure required to give a particular volume loss. In practice there is always a significant scatter of measured values for volume loss, even when tunnelling through apparently homogeneous ground conditions, using the same TBM and tunnel crews. The calculations always need to be combined with observation during construction, and to be fine-tuned based on the observed settlements.

2.4.3 Limitations in the ground model

The significant differences between the target face pressures for the limited range of soils considered in the example calculations is evident by comparing the Tables 5 and 7. Any major differences between the assumed ground conditions and the actual conditions, even over a very short length of the tunnel, could result in a large

increase in the surface settlement or a sinkhole over the tunnel. Where there are interfaces between different soils, or between soil and rock, the ground model should be critically reassessed as part of the design process. Where the exact location of an interface is critical to the determination of the operating pressures, then either the interface has to be identified with additional investigation, or very conservative assumptions made. Geotechnical Baseline Reports commonly include long sections with interpreted subsurface conditions for contractual purposes, but the long sections should not be used uncritically when assessing operating pressures.

2.5 Other operating pressures

The example calculations summarised above are for the target face pressure, and the checks required to ensure that ULS and SLS are satisfied if the target face pressure is applied, considering both settlement and heave. In addition to the face pressure, there are a number of other operating pressures that also need to be assessed. These are:

- The minimum and maximum tail void grouting pressures
- For EPB TBMs with the facility for injection around the shield skin, the maximum pressure of injection
- The compressed air pressure (if any) required for interventions into the excavation chamber

2.5.1 Minimum and maximum tail void grouting pressures

The tail void grout has a number of functions. As outlined in Shirlaw et al (2004), these include:

- Ensuring even contact between the ground and the lining
- Minimising the settlement by filling the tail gap void before the ground can move significantly
- Holding the ring in place against flotation forces
- Carrying the load from the TBM back-up
- Reducing seepage and potential for loss of fines if the gaskets are damaged or not in contact

Consistent filling of the tail void, simultaneous with tunnel advance, is necessary to achieve these objectives. This is typically done by injecting grout through 2 to 6 grout pipes installed along the inside of the tail skin. The grout may have to travel 5m to 10m around the lining from the point of injection. In order to achieve this, it is an empirical rule of thumb that the grout injection pressure needs to be up to 1 to 2 bars higher than the face pressure, giving the maximum pressure for the grouting. Where the specified volume loss is very low, the ground is soft or loose, and/or the tunnel is shallow this is likely to mean that the maximum grouting pressure will exceed the total overburden pressure. If this is the case, the pressure can be checked against the pressure required to initiate fracturing of the soil or cavity expansion. With very shallow tunnels, it may be necessary to balance the risk of ineffective grouting of the tail void with the risk of the grout breaking out from the tunnel annulus.

It is common to require a minimum injection volume of 115% to 120% of the theoretical volume of the tail void. Injection typically continues beyond this minimum volume if the injection pressure is low, or if there is evidence of loss of ground at the face during preceding rings.

2.5.2 Injection pressure around skin

The injection of bentonite slurry around the shield skin is a relatively recent development, and there is no consensus on the pressure to be used. The pressure should be close to the target face pressure, to be consistent with the calculations. However, it is reasonable to apply a pressure slightly lower than the target face pressure, to minimise the volume of bentonite slurry lost into the face.

2.5.3 Compressed air pressure

The compressed air pressure provided has to be sufficient to ensure that there is the same degree of support as during slurry or EPB tunnelling. However, there are significant differences between support using compressed air and that provided by pressurised slurry or spoil.

Compressed air pressure is constant over the height of the exposed face. This difference in the distribution of pressure, compared with slurry or spoil, has to be considered in the calculations.

If the compressed air is applied directly to the soil, the compressed air will penetrate into the pores in the soil. The excess pressure over the initial pore water pressure will raise the pore pressure, and will not provide support to the soil skeleton. The soil will also be dried out by the compressed air, and will then run or ravel. Ensuring that there is a thick, low permeability, filter cake on the face will help to avoid these problems, by providing a membrane on the face.

2.6 Application at the Melbourne Main Sewer tunnel

The calculation methods outlined in GEO Report 249 were used for the calculation of operating pressures for the tunnelling for the Melbourne Main Sewer Replacement project. The project is used as an example because the tunnelling was carried out through a wide range of ground conditions using an EPB TBM methods, and has been successfully completed. The project has been documented in a number of papers (Dixon, 2009; Dixon, 2011; and Clark et al, 2011), so only a summary of the information is provided here.

The project included approximately 2 km of 3m internal diameter tunnel driven by an EPB machine, manufactured by Lovat. John Holland was the contractor for the project. Golder Associates Pty Ltd provided operating pressure calculations for both the main tunnel and the pipe-jacking; these calculations followed the recommendations of GEO Report 249. The calculations were carried out in a large spreadsheet using the same methods as the example calculations given above.

The main tunnel was driven through a wide variety of ground conditions, under a residential area of Port Melbourne. The ground conditions encountered in the tunnel face included:

- Coode Island Silt: soft, silty clay
- Port Melbourne Sand: loose fine to medium grained sand
- Fisherman's Bend Silt: Mainly stiff to very stiff silty clay, with a limited bed of sand and minor gravels
- Basalt and weathered basalt

The range of materials encountered, ranging from strong rock to loose sand and soft silty clay, made this a good test of the calculation methods given in GEO Report 249.

As discussed by Dixon (2009), the construction team had assessed anticipated and worst case volume losses of 1% and 4% respectively. The face pressure calculations were based on achieving 1% volume loss; in the soft soils this required relatively high face pressures and a high standard of tunnelling practice. There was particular concern over the tunnelling in the Coode Island silt. A 'test' section of instruments was installed to give detailed information on the response of the ground to the EPB tunnelling. The results of this test section, with a measured settlement trough showing 0.73% volume loss, are described by Dixon (2009) as 'comforting and surprising'. The results can also be seen as the result of careful calculation and excellent tunnelling performance. Low values for volume loss were recorded over the EPB drives, showing that the simple methods for calculating operating pressures outlined in GEO Report 249 are effective in a wide range of ground conditions, when combined with good tunnelling performance.

While there was settlement over the tunnel at the test section, the ground moved horizontally away from the tunnel, as measured by inclinometers installed on either side of the tunnel (Clark et al, 2011). This is consistent with the example calculations for soft clay summarised above: it was necessary to apply relatively high pressures to control volume loss to the low value required. These pressures exceeded the insitu horizontal stresses in the ground.

3 LIMITATIONS IN THE BASIC METHODS

3.1 Simplifications required

Where the tunnel is to be driven through ground consisting of interbedded sands and clays, it is not possible, using the simple charts, to account for the combination of the different types of soil. Use of the Horn model, or its derivatives, is not appropriate where there are substantial thicknesses of clay, as the failure surface in clay does not correspond to that used for the Horn model.

This limitation can be partially overcome. Separate analyses can be carried out, for the beds of clay or sand in and over the face of the tunnel. The highest calculated face pressure can then be adopted as the target pressure. This approach should be conservative, in relation to the actual conditions. Using it, a variety of possible ground conditions are allowed for, giving a significant degree of robustness to the calculations.

3.2 Limitation, effective stress methods

The semi-empirical 'Proctor and White' method of obtaining a face pressure to satisfy SLS in granular soils appears to provide reasonable results where the water table is high, as evidenced by the tunnelling in the Port Melbourne Sand for the Melbourne Main Sewer project. However, the method does not provide a real basis for the relationship between pressure and settlement for tunnelling in granular soils. It is possible that the method underestimates the settlement where the water table is relatively low. Testing of model tunnels in sand, in the geotechnical centrifuge, would likely provide the information required to improve the current methods. There has been some published data from such testing, for example Plekkenpol et al (2006). However, the data that has been published so far is fragmented, and does not yet provide a comprehensive basis for the assessment of volume loss over tunnels in sand.

Generally, there has been little study of heave mechanisms over tunnels, compared with those for settlement. It is normally assumed that the heave mechanism is simply the reverse of that for settlement. Where possible, the resistance to heave is ensured by maintaining the maximum operating pressure below the total overburden pressure. However, for large, shallow tunnels with stringent settlement criteria this may not always be possible. It is likely that more detailed assessment of the limit states for heave will become more critical in the future.

3.3 Limitations, total stress methods

The centrifuge data published by Kimura & Mair in 1981 was for a limited range for the ratio of tunnel cover to diameter. The chart for N_C does not extend to C/D ratios below 1, and the volume loss against load factor chart is for only two values of C/D. There have been additional studies that extend these charts to a limited extent. However, with the relative increase in the need to consider large, shallow tunnels, or relatively deep tunnels, with very tight settlement criteria, more comprehensive charts would provide useful guidance for design.

4 CONSOLIDATION SETTLEMENT

Consolidation settlement is the result of changes in pore pressure, and thus effective stress. There are a number of mechanisms associated with consolidation settlement over and around tunnels, and these are discussed below. Generally, large magnitudes of consolidation settlement are due to pore pressure changes in soft, near normally consolidated clay, due to the high compressibility of such soils. However, significant consolidation settlements have also been measured due to tunnelling in stiff clay; in areas of particular sensitivity even the consolidation settlement associated with pore pressure changes in fractured rock may be sufficient to be of concern.

Consolidation settlements can be a large proportion of the total settlement over tunnels, as documented by Shirlaw et al (1996), however, there is much less published data on the pore pressure changes associated with tunnelling than on the volume losses over tunnels as they are driven.

4.1 Causes of consolidation settlement

There are a number of potential mechanisms that can lead to consolidation settlement over and around pressurised TBM tunnels. Shirlaw et al (1996) proposed three different patterns of pore pressure changes that could occur during tunnelling, and that would lead to consolidation settlement. These are:

(a) The generation of positive excess pore pressures by tunnelling at a pressure higher than the initial stress in soft clay
(b) The generation of positive excess pore pressures by tunnelling at a pressure lower than the initial stresses in the ground, when tunnelling in soft clay
(c) Due to seepage into the tunnel, either at the TBM or through the lining

The consolidation settlement would result in the change from the initial pattern of pore pressures that develop just behind the TBM, (a) or (b) above, to the long term pattern, (c) above. However, consolidation settlements could also occur if there was seepage into the TBM.

For completeness, two more potential mechanisms are proposed here:

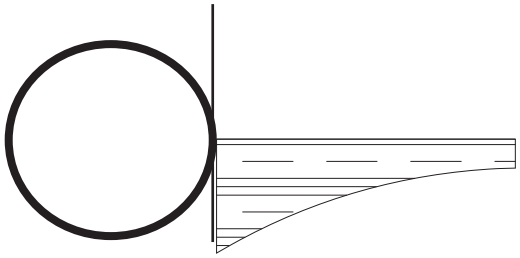
(d) Due to the consolidation of the tail void grout as the grout comes under ground loading
(e) Following a compressed air intervention; in granular strata the compressed air will act to push the pore water away from the tunnel. Once the compressed air pressure is removed, the groundwater will flow back into the zone dewatered by the compressed air. This process may lead to the drainage of more compressible overlying strata, as well as the strata at tunnel level



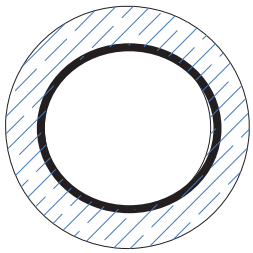
(a) Tunnelling with pressure > in-situ pressure in soft clay



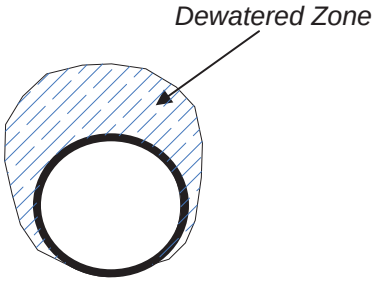
(b) Tunnelling with pressure < in-situ pressure in soft clay



(c) Effect of seepage into the tunnel, short term or long term.



(d) Consolidation (bleed) of tail void grout.



(e) Dewatering effect of compressed air.

Figure 5: Patterns of pore pressure change, and other effects, that can lead to consolidation settlement

These patterns are illustrated in Figure 5, and discussed further below. The shape of the settlement trough due to consolidation settlement will depend on which mechanism(s) are causing the settlement. For (a) and (d), the consolidation is concentrated around the tunnel, and the resulting trough will have the same shape and width as the volume loss settlement. For (b), (c) and (e), the resulting consolidation settlements are typically much more widespread than the volume loss settlement.

4.1.1 Tunnelling at a pressure higher than the initial stress in soft clay

Positive excess pore water pressures can be generated immediately around the tunnel as a result of the face or tail void grouting pressures, or a combination of both. This is illustrated in Yi et al (1993), Hwang et al (1996) and Shirlaw et al (1994); typically the excess pore pressures immediately after the TBM has passed are finally the result of the pressures exerted during tail void grouting.

4.1.2 Tunnelling at a pressure lower than the initial stress in soft clay

If the ground arches around the tunnel heading, ground loading will be transferred from the immediate area of the tunnel, resulting in increased stresses and pore pressures at some distance from the tunnel. This can be seen in the results of tests on a model tunnel in a geotechnical centrifuge, reported by Ong et al (2007). This mechanism was also postulated by Shirlaw et al (1994), and supported by field data published by Ng et al (1986).

4.1.3 Drainage into the tunnel

Drainage into the tunnel can occur either at the face, if the face pressure is less than the insitu water pressure, or through the tunnel lining.

Operating the TBM at low or no face pressure can result in consolidation settlements of very large magnitude, particularly if the tunnel is in a relatively permeable soil or rock under thick deposits of soft clay. For example, Kwong (2005) records that there was up to 750 mm of consolidation settlement at Tsung Kwan O town centre due to the construction, in rock, of tunnel C of the SSDS. This settlement occurred approximately 1.5 km from the tunnel. Although the tunnelling was not by pressurised TBM, the same mechanism will occur if a slurry or EPB TBM were to be operated in rock with a face pressure that was significantly lower than the original insitu groundwater pressure.

Very large consolidation settlements are usually associated with thick deposits of soft clay. However, in particularly settlement sensitive areas even the consolidation of rock can be of concern. In Singapore, open face tunnelling for the North East Line caused a large reduction in the groundwater pressure in weathered, highly fractured sedimentary rock underlying an operational railway tunnel. The consolidation of the weak rock resulted in up to 10mm settlement of the tunnel, although this reduced to about 7 mm as the groundwater pressures recovered.

The pore pressure changes associated with seepage into the tunnel, through linings with modern gaskets, should be relatively small in comparison with the potential changes due to tunnelling with no or low face pressure. However, when the tunnel is in soft clay, even relatively small pore pressure changes can result in significant long term settlement.

4.1.4 The consolidation of tail void grout

The potential for the tail void grout to consolidate with time, losing water when subjected to ground loads, is not generally considered. However, Komiya et al (2001) carried out some laboratory trials on a two component grout of a type that is very commonly used for tail void grouting. This type of grout involves mixing component A, an OPC cement grout, with component B, diluted sodium silicate. The mixed grout has a high W:C ratio, typically about 3.5:1. An initial set or gel occurs quickly, typically within 30 to 120 seconds, providing the minimal strength required to hold the ring in place against flotation pressures. Komiya et al (2001 & 2003) report that, in laboratory tests, this type of grout exhibited 30% consolidation under load, after the initial gel had occurred. With the addition of bentonite and chemical hardener to the mix, this reduced to 7%. Bezuijen & Talmon (2003) recorded a comparable 5% to 10% loss of volume due to consolidation in an unspecified grout.

For the example tunnel gap in Table 4, shrinkage of the tail void grout by 30% would result in an additional volume loss of 2.23%, while shrinkage by 7% would result in an additional volume loss of 0.52%. In the context of specifications for total volume loss for tunnels that are now commonly 1%, and sometimes 0.75% or 0.5%, the potential to exceed the specified volume loss just in the consolidation of the tail void grout is of concern. It is uncertain to what extent the consolidation of the tail void grout is being captured in published data for the volume loss over tunnels.

4.1.5 The dewatering effects of compressed air interventions

When compressed air is used for interventions, the compressed air will penetrate into the pores in the soil and create a zone of partially dewatered ground around the tunnel. The permeability of the ground is much higher for compressed air than for water. As a result the compressed air will penetrate much further into the ground than slurry or EPB spoil, or any bleed water from either. As a result, when the compressed air pressure is

replaced by slurry or EPB pressure, there is likely to remain a zone in the ground which has been partially dewatered by the compressed air. Migration of groundwater into this zone can lead to consolidation settlements. Examples of consolidation settlements over tunnels driven under compressed air in Hong Kong are provided in Cater et al (1984).

4.2 Controlling consolidation settlement

There is an incomplete understanding of the mechanisms causing consolidation settlement due to tunnelling, and limited data on the potential magnitudes of the pore pressure changes and consolidation settlements. The information that is available suggests that consolidation settlements can be minimised if:

- The permanent lining is as watertight as possible, to minimise long term seepage into the tunnel
- The tunnelling is conducted at a pressure equal to or higher than the static groundwater pressure, to avoid seepage during construction
- When tunnelling through soft clay, pore pressure changes are minimised by tunnelling with pressures close to the insitu earth pressures
- The tail void grout that is chosen does not exhibit excessive consolidation under load
- Consolidation settlements associated with interventions in compressed air are minimised by creating a membrane over the face, for example a bentonite filter cake, to minimise air losses into the ground

There are practical difficulties that mean that consolidation settlements cannot be entirely avoided, particularly when tunnelling through or under near normally consolidated clay. Even the best lining is likely to leak, even if only to a very limited extent, resulting in some long term reduction in pore water pressures close to the tunnel. When tunnelling through soft clay, the pressures exerted by the TBM are almost isotropic, whereas insitu ground pressures are not. This can be seen at the phenomenon noted in the Melbourne Main Sewer Replacement tunnel, where there was surface settlement over the tunnel, but the inclinometers in the same array showed outward ground movement. This, and the practical need to exert sufficient pressure on the tail void grout to ensure effective grouting, means that it is not possible to tunnel in soft clay without causing some positive excess pore pressure, relative to the initial pore pressures. The change from positive excess pressures generated during tunnelling to the long term seepage condition means that there will be some consolidation settlement. Osborne et al (2008) are quoted above in relation to the face pressure required to control the volume loss to 1% when tunnelling in marine clay in Singapore. Osborne et al. continue to record that the settlement subsequently doubled as a result of consolidation settlement.

Consolidation settlement may not have a significant adverse effect on buildings and utilities, as the type of widespread, relatively uniform, consolidation settlements typically associated with patterns 2, 3 and 5, in Figure 5, are unlikely to cause damage to uniformly founded buildings or utilities. However, there is a risk of damage occurring where buildings are on mixed foundations; consolidation settlements may also be critical where structures are particularly sensitive to settlement or where there is rigid adherence to a pre-defined maximum value for settlement. In contrast, pattern 1 and 4 consolidation causes settlements that have a similar trough to volume loss settlement, and would have a similar effect on buildings and utilities.

4.3 Limitations in knowledge

Generally, there has been very little study of consolidation settlement over tunnels, in comparison with the much more extensive study of the immediate, ground loss, settlement. Although a few relevant papers are referred to above, there is very limited published field data on the pore pressures generated by the process of pressurised TBM tunnelling, and even less on the long term changes in the pore pressures around modern linings.

5 LOCAL FEATURES IN GENERALLY STABLE GROUND

5.1 The nature of the problem

The examples of face pressure calculations given above are for soils that would be unstable during tunnelling, and/or where pressure is required to control the settlement due to tunnelling. The need for a face pressure is obvious in such soils, and the setting of the target pressure is a simple matter of carrying out the requisite

calculations and applying them during tunnelling. A much more difficult decision making process is involved when tunnelling through generally stable ground (such as rock), but where there are, or could be, local zones of potentially unstable ground. Examples include:

- Rock with structural features such as fault zones, or dykes that have weathered at different rate to the parent rock
- Glacial till that is predominantly boulder clay, but that contains tortuous beds or lenses of sand

Site investigation boreholes drilled from the surface may not identify all of the areas of potentially unstable ground, so it is likely that there would be a significant degree of uncertainty as to the location of such features before tunnelling commenced.

One possible approach to this problem is to set the face pressure for the most adverse possible conditions along the alignment; however, this will mean that most of the tunnelling will be carried out at a pressure much higher than is actually necessary. While the use of a high face pressure will reduce the risk of a major ground loss, if a local area of potentially unstable ground is suddenly encountered, the use of the high face pressure will itself cause some increased risk. Use of an unnecessarily high pressure will result in greater wear in abrasive ground, and increase the number of interventions required. Interventions have been identified as high risk activities for loss of ground (Shirlaw & Boone, 2005), so applying a high face pressure during normal tunnelling, irrespective of the actual ground conditions, may not be the lowest risk option.

5.2 Alternative approaches that can be adopted

A number of alternative approaches can be adopted for tunnelling in these circumstances. One, as outlined above, is to drive at the pressure required in the most onerous likely ground conditions.

A second approach is to identify the conditions requiring the higher pressure, by more detailed investigation, possibly supplemented by probing from within the tunnel.

A third possible approach is to use the TBM itself as a means of probing the ground. This has been done in Singapore using an EPB TBM for tunnelling in Old Alluvium. The unweathered Old Alluvium is generally of low permeability, but there are occasional beds of poorly graded sand, which are relatively permeable (see Knight Hassell et al, 2001), but difficult to identify from the site investigation. One approach that has been adopted in EPB tunnelling in the Old Alluvium is to drive with just sufficient pressure to ensure that the chamber is full of spoil and that there is a plug in the screw conveyor. The change in pressure is monitored during driving and during the ring build; if the pressure builds up during the ring build it indicates more permeable ground and the need to use a higher face pressure. The success of this approach depends on the tunnel crew responding quickly to the information obtained during tunnelling.

6 RISK

6.1 Perception

It is very common for all concerned, including the owner, engineers and contractors to perceive pressurised TBM tunnelling, using segmental concrete linings, as being of very low risk. This is both in terms of the tunnel lining collapsing, and in relation to the potential for large ground movements during tunnelling. However, the likelihood of such events is often underestimated.

A much more frequent, but arguably lower consequence, hazard is that of encountering an old, poorly backfilled borehole, well or subsurface instrument. This hazard is present in all urban tunnels, but is often not adequately recognised in risk assessments.

6.2 Reality

6.2.1 Risk of a segmentally lined tunnel collapsing during construction

Although tunnels lined with segmental concrete linings have a good record in use, there is some risk of a major collapse during construction. Examples of such collapses include:

- An EPB driven tunnel in Hull, UK, as recorded in Grose & Benton (2005)
- A slurry shield driven subway tunnel in Cairo, as recorded in Wallis (2009)

- (Possibly) an EPB driven tunnel in Okayama, Japan Wallis (2012). This incident is very recent, and not many details are yet known, but as discussed in Wallis (2012), what facts are known appear to indicate either a collapse of the tunnel lining close behind the shield, or separation of the TBM and the lining had occurred.

Each of these incidents resulted in a massive sinkhole at the ground surface, and flooding/partial infilling of the tunnel. Recovery of the tunnel from this type of incident is extremely expensive and the delays to the completion of the tunnel are likely to run to years rather than months.

As a proportion of the number of tunnels driven using these methods in the last 10 years, the number of incidents of this type is extremely small. However, the consequences are invariably disastrous.

6.2.2 Risk of large ground movements

Large ground movements can appear either as a sinkhole (Plate 5) or as a typical settlement trough with a large value for the maximum settlement (Plate 6). For the assessment of the settlements over the tunnels for the North East Line in Singapore, Shirlaw et al (2003) used a settlement of 150mm or greater to define this type of large, exceptional settlement.

The frequency of localised very large settlements or sinkholes is very much higher than is often recognised or even officially recorded. Shirlaw & Boone (2005) identified the number of such incidents on seven major projects using EPB TBMs built between 1984 and 2005, in Singapore and Canada. Typically, for those seven projects there was one incident of a very large settlement or sinkhole for every 500 m to 2,000 m of tunnelling. This did, however, depend on the ground conditions, with few incidents for tunnels built entirely in soft soils, and a much higher frequency of incidents when tunnelling in weathered rocks and in glacial till.



Plate 5: Sinkhole over a slurry shield in mixed grades of weathered granite



Plate 6: General settlement over an EPB shield in near normally consolidated clay

It is a characteristic of pressurised TBM tunnelling that the measured surface settlements are generally very low, but with very occasional, localised, but very large, settlements or sinkholes. Occasional large settlements or sinkholes are not necessarily in the public documentation of a project. In a statement in a project case study that 'the surface settlements were generally well controlled', the 'generally' may be used to cover several, or even dozens, of sinkhole incidents. There has, however, been growing recognition of the risk of a large local settlement or sinkhole, and of the measures required to reduce the risk of a large settlement/sinkhole occurring. Some of these measures are discussed in GEO Report 249; a key risk reduction measure is the detailed calculation of the required target pressures, discussed above.

Although there is now increased understanding of the causes of sinkholes over pressurised TBMs, and of the measures required to reduce this risk, the consequences of a sinkhole occurring has tended to increase with time. Twelve years ago, a sinkhole in a road in Singapore would typically be backfilled within 6 to 12 hours. Although some grouting would usually be carried out to recompact the disturbed ground, tunnelling would commonly restart within one week of the incident. Bakker & Bezuijen (2008) report that there were a number of major incidents of loss of face stability during the construction of the 2nd Heinenoord, Sophia Rail and

Green Hart tunnels, but that these incidents did not cause significant delay to the tunnelling. In contrast, in Singapore it now takes typically 2 to 4 months to obtain permission to recommence tunnelling after a major settlement/sinkhole incident. This delay is for incidents that do not cause any injury, or damage to buildings; if there was any injury, say to a road user, or major building damage, it is likely that the delay would be significantly longer. Based on the scales given in Eskensen et al (2004), a three month delay to a pressurised TBM tunnel would, on the basis of either the delay to the work or the cost of the delay, be considered a 'serious to severe' event in a risk assessment. In the last twelve years the consequence of the same incident, in Singapore, has changed from being classified as 'insignificant to considerable' to 'serious to severe'. This is not because the incident has changed, but because the response to the incident has. This change in the consequence of a sinkhole occurring has not always been recognised in the risk assessments carried out for tunnelling projects in Singapore.

A significant proportion of large settlements/sinkholes occur over intervention locations (Shirlaw & Boone, 2005). Even if there is not a sinkhole or very large local settlement, it is likely that the settlement over the intervention location will be significantly higher than during normal tunnelling. An example is given in Cham (2009), who records that there was 78 mm settlement over an intervention in weathered granite, when the settlement over the twin running tunnels was typically less than 30 mm.

The relatively high likelihood of a large settlement or a sinkhole over an intervention may be for a number of reasons, including:

- Reluctance of the contractor to use sufficient compressed air pressure, due to the reduced working time in higher pressures
- Due to granular soils drying out and starting to ravel or run, particularly if a thick filter cake is not first formed on the face, and if the intervention is a lengthy one
- The creep of fine grained soils during the intervention
- Consolidation settlements, either due to drainage into the tunnel, if the insitu water pressure is not balanced, or due to the dewatering effect of compressed air that is discussed above

6.2.3 Risk of encountering boreholes/wells/instrumentation

Encountering a poorly backfilled borehole, well, or instrument such as a standpipe piezometer in a pressurised TBM drive can result in:

- Loss of slurry, foam and/or grout to the surface
- A sinkhole at surface: loss of pressure up the hole, or deliberate reduction in the pressure to control loss of material to the surface, is often a factor in the formation of sinkholes at the surface
- Poor tail void grouting, particularly with sub-aqueous tunnels: if the tail void grout is lost to the surface, this can result in an ungrouted, water filled tail void. There can then be segregation of the tail void grout around subsequent rings, as the grout is injected into a water filled cavity. This can lead to continuing problems with the effectiveness of the grouting.

Where the ground water table is high, there is usually little that can be done to mitigate these risks by altering the target operating pressures for the TBM, due to the requirement to satisfy the other limit states. The risk of encountering boreholes or other open zones is commonly mitigated by grouting all of those that can be identified, prior to tunnelling, and having contingency measures in place to mitigate encountering any remaining unidentified open holes.

7 CONCLUSIONS

The setting of target operating pressures for pressurised TBM tunnelling requires a first stage of calculation. In this paper, the relatively simple methods of calculation outlined in GEO Report 249 have been used to illustrate general relationships between different soil types and varying SLS requirements. Although more complex methods of analysing the problem are available, the effectiveness of the basic methods in a broad range of ground condition is demonstrated by the excellent results obtained for the Melbourne Main Sewer Replacement tunnel drives. However, the use of the basic methods is limited by the extent of the published information. In particular, an improved understanding of the relationship between face pressure and volume loss in granular soils of different relative density would be a valuable guide for future calculations. Expansion of the existing curves that are used to establish N_C and the relationships between volume loss and load factor,

to cover lower and higher C/D ratios than currently available, would also be useful. The potential magnitude and mechanisms leading to consolidation settlements are not well understood. In particular, there has been relatively little study of the long term pore pressure changes around modern segmental tunnel linings.

Although the calculations are an essential part in the development of suitable operating pressures, a significant degree of engineering judgement needs to be applied both before and after the calculations. Before carrying out the calculations the ground and groundwater model has to be developed, and the sections for analysis chosen, so that the risk of encountering more adverse (requiring significantly higher pressure) ground conditions than expected is minimised. Interfaces between relatively stronger and relatively weaker ground conditions need particular care, as the higher pressure required by the weaker ground has to be applied before the weaker ground is actually encountered in the tunnel. Simple interpolations between boreholes are not adequate to minimise the risk of encountering more adverse than expected ground conditions. Because of the high consequence of such an encounter, a conservative interpretation needs to be adopted, or the interface investigated in greater detail with further boreholes.

A particular feature of assessing the target pressures for tunnelling is the need to balance the consequence of using too low a pressure against that of using too high a pressure. There can be a very small margin (operating window) between an unacceptably low pressure and an unacceptably high pressure. This margin is typically small where any one of the following applies:

- The tunnelling is in soft or loose soil
- The groundwater level is high, close to or above the level of the ground surface
- The tunnel has a low C/D ratio
- The tunnel has a large diameter tunnel
- Very tight controls are given for settlement or horizontal movement

It is often necessary to use judgement to balance competing requirements in the assessment of the operating pressures.

There is a constant process of ‘pushing the envelope’, with bigger, shallower (or deeper) tunnels, and ever more stringent criteria for settlement and lateral movement. There are a number of issues, discussed above, where further research would assist in the calculation of suitable target pressures for some of the more extreme examples. Detailed case studies of completed tunnels, including the problems as well as the successes, could also assist in some of the issues identified above. This would help to provide the information necessary to make informed judgements of the risks associated with the tunnelling.

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26 km of Geotechnical Challenges

Alex C.W. Chan
Railway Development Office, Highways Department, the Government of the Hong Kong SAR
Augustine H.S. Li
MTR Corporation Limited

ABSTRACT

The Guangzhou-Shenzhen-Hong Kong Express Rail Link (XRL), with a total length of about 140 km, will connect Hong Kong with the 16,000km "four vertical and four horizontal corridors" national high-speed rail network of mainland China and will materialize the idea of one hour living circle between Hong Kong and the Pearl River Delta region. The Mainland section of the XRL starts at Guangzhou South Station, Shibi and enters Hong Kong via Huanggang in Shenzhen. The Hong Kong section of the XRL, which is entirely in tunnel, then runs through Mai Po, Tai Mo Shan, Kwai Chung and the urban area of Kowloon to the West Kowloon Terminus. Construction of the Hong Kong section of the XRL commenced in early 2010 and is expected to complete in 2015.

This paper will address the geotechnical challenges during the planning, site investigation, design and construction of this strategically important high speed railway tunnel.

1 INTRODUCTION

The Hong Kong Section (HKS) of the Guangzhou-Shenzhen-Hong Kong Express Rail Link (XRL) is one of the ten mega projects announced in the Chief Executive's 2007 Policy Address. It was recommended as one of the priority projects for implementation in the Railway Development Study 2000 (RDS2000). From concept to detailed design stage, the planning and development of the HKS of XRL has taken almost ten years.

The whole of XRL measures 140km in length, starting from Shibi (石壁) of Guangzhou and terminating at West Kowloon of Hong Kong, with intermediate stations at Humen (虎門) of Dongguan (東莞), Longhua (龍華) and Futian (福田) of Shenzhen (Figure 1). The HKS is a 26km long underground dedicated corridor, connecting with the Mainland section at the boundary in Huanggang and running south through Mai Po, Shek Kong, Tsuen Wan, Kwai Chung, Lai Chi Kok, Sham Shui Po and Mong Kok before terminating at the West Kowloon Terminus (WKT) (Figure 2). The HKS comprises the WKT, the tunnel and associated ventilation and access facilities, an Emergency Rescue Siding (ERS) and a Stabling Sidings with maintenance facilities at Shek Kong. The maximum operating speed of the HKS will be 200 km/h while the speed will reach 300 km/h after leaving Hong Kong and running in the Mainland.

2 GROUND INVESTIGATION

The designers were confronted with many geotechnical issues and challenges not encountered in other local railways during the design stage of this unique tunnel. The first challenge to the designer was that only limited geotechnical information was available as a significant portion of the tunnel route runs through previously undeveloped land such as Tai Mo Shan and the Mai Po Wetland. The essential ground investigation works were carried out in two stages. The aim of the first stage investigation was to provide sufficient information for the designer to complete the critical route selection and the preliminary tunnel design. The second stage work provided additional data for the design of the associated facilities like ventilation buildings and refinement of the tunnel designs. A total of 512 ground investigation drill holes were sunk specifically for the design of XRL tunnels. Substantial difficulties were encountered in implementing the ground investigation works. Many drill holes had to be sunk offset from the tunnel alignment due to their proximity to existing

buildings and problems with land and access rights. As a result more drill holes had to be sunk to allow a better assessment of the geology.

About 7.6 km of the XRL tunnels will be constructed under the Tai Mo Shan at a maximum depth of 500 m. In view of the high cost and practical difficulties drill holes were limited to not more than 200 m deep. It is however possible to obtain reasonably representative geological information in the deeper part of the tunnel via records from previous completed tunnelling projects such as the West Rail located 3 km west of XRL and the WSD Treated Water Tunnel between Tai Po and Butterfly Valley located 1km to 5 km away to the east.

At locations of major geological features like the Tolo Channel Fault a 570 m long horizontal hole was drilled to better identify the extent and conditions of the fault.

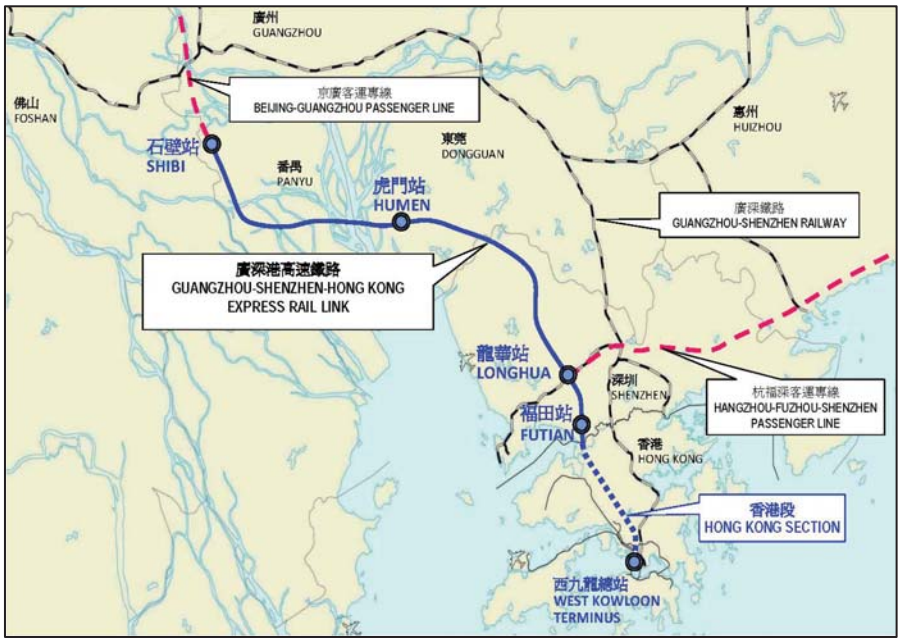


Figure 1: Guangzhou – Shenzhen – Hong Kong Express Rail Link

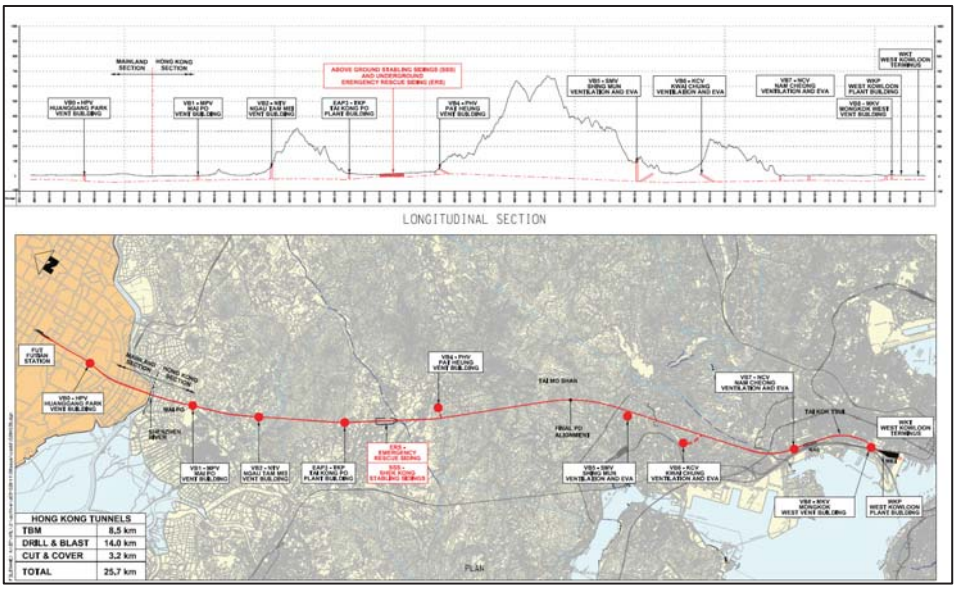


Figure 2: Alignment of the Hong Kong Section of the Guangzhou – Shenzhen – Hong Kong Express Rail Link

3 ALIGNMENT THROUGH URBAN KOWLOON

About 7 km of the HKS will pass through the congested Kowloon urban areas with the remaining 19 km to be constructed in northwest New Territories and the Tai Mo Shan areas. Choosing the alignment through the urban Kowloon area proved to be most difficult as in addition to satisfying technical and engineering requirements considerations had to be given to keeping any impacts to the local communities to the minimum.

Two different routes were considered for the tunnels' alignment through the congested Kowloon urban area at the early design stage, an easterly route via the Sham Shui Po area and a route which runs through the Tai Kok Tsui area after leaving the WKT. A third route was added for consideration in response to suggestions made during the consultation process - a westerly route which would run along the Lin Cheung Road. The major selection criteria include engineering feasibility; potential impact on the nearby communities, adjacent buildings and infrastructure; and operation flexibilities.

The option of running the tunnels through the Sham Shui Po areas was dropped for two main reasons - the poor geology in particular the low rock-head and the high density of old (many of pre second world war) buildings. A number of fault zones run across the proposed tunnel alignment where heavily weathered rock can be expected. Extensive ground treatment would have to be carried out during construction. This however would be difficult to implement due to the high density of old buildings in the area. If the tunnels were to be constructed in competent rock a long section would have to be driven at over 50 m below ground. TBM intervention would have to be carried out under compressed air of over 5 bars. This would increase the risk to the workers and nearby buildings during construction.

The proposed tunnel route along Lin Cheung Road is constrained by two existing structures, the elevated West Kowloon Corridor and the twin box section of the West Rail. It would only be possible to run the XRL tunnels along the narrow corridor between the foundations of these two critical structures and a staggered configuration would have to be adopted. The lower tunnel would have to be constructed at over 50 m depth. As in the case of the Sham Shui Po option TBM intervention would have to be carried out under unacceptably high pressure compressed air. Furthermore there exists a potential impact of the tunnel construction on other nearby important railway facilities including the Airport Express and the Tung Chung Line. This option was therefore not considered.

The route through the Tai Kok Tsui area was selected because of the lower construction risk and less implications on nearby residents and infrastructure facilities. The majority of this selected route (Figure 3) runs underneath Hoi Wang Road and Sham Mong Road. Any impact on residents and other users along the route is thus kept to the minimum. A short 500m section in the Tai Kok Tsui area joining the sections under Hoi Wang Road and Sham Mong Road will inevitably have to be constructed below 19 residential and combined residential/commercial buildings. The vertical profile was carefully selected such that this section of the XRL tunnels will be constructed with reasonable clearance from pile foundations of these 19 buildings and in reasonably good quality rock or CDG. A detailed assessment of the potential impact of the construction of the XRL tunnels on the 19 buildings in Tai Kok Tsui was carried out in the early stage of the design. The assessment concluded that any impact would be within acceptable limits.

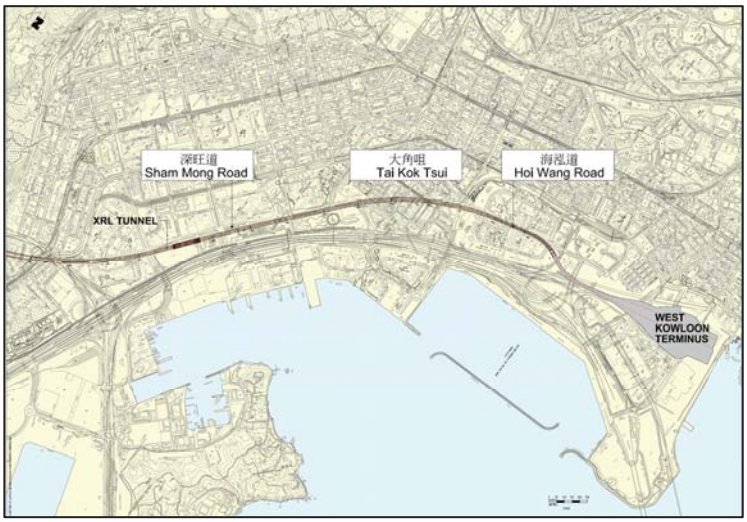


Figure 3: Alignment of HKS through urban Kowloon

4 TUNNELLING METHODS

The majority of the 26 km tunnels will be constructed either by the drill and blast method (14 km) or by tunnel boring machines (9 km). The approach tunnel to the West Kowloon Terminus together with a few short sections will be constructed by the cut and cover method (3 km). Drill and blast is a well-proved method for excavation of hard rock tunnels and have been used in the construction of many transportation tunnels in Hong Kong. Tunnel boring machines (TBMs) offer the potential for very high production rates and have been used successfully in a number of recently completed tunnels like the Lok Ma Chau railway tunnel, the Kowloon Southern Link (now part of the West Rail) tunnel and the West Rail Kwai Ching tunnel. The main criteria in the choice of the tunnelling methods are ground geology, construction access, potential impact on adjacent communities, and construction programme and cost.

Mixed ground TBMs have been adopted for the soft ground tunnels including the section in the urban Kowloon area and the section through the Ngau Tam Mei and Mai Po areas. Apart from the geological factor large sites are also available for the launching of the TBMs and accommodating the TBM supporting facilities. Furthermore these two contracts were programmed to be awarded early to allow sufficient lead time for the procurement and manufacture of the TBMs.

The drill and blast method has been selected for the construction of the sections of tunnels under the mountains of Golden Hill, Tai Mo Shan and Kai Kung Leng which will be in competent rock. Considerations were given to the use of hard rock TBM but this was dropped for programme and cost reasons. The lead times required for the supply of 9m diameter hard rock TBMs were expected to be significant (12 to 14 months). While it was estimated that the rate of production would compare favourably with the drill and blast method additional works associated with enlargements for the crossovers and the ventilation connections would impact on the programme. There was also concern on the capability of the tunnelling industry to produce the required number of mixed ground TBMs and hard rock TBMs along with orders from other major tunnelling projects. Given the higher costs, negligible programme benefits and the increased risk of large diameter hard rock TBMs it was concluded that there would be no benefit in constructing the rock tunnels by hard rock TBMs.

5 INTERFACES WITH OTHER TUNNELS

In the urban area the biggest challenge to the tunnel designer is in the Lai Chi Kok area where the XRL tunnels will be sandwiched between the existing MTR Tsuen Wan Line (TWL) tunnels and the Lai Chi Kok Transfer Scheme (LCKTS) drainage tunnels that are under construction (Figure 4).

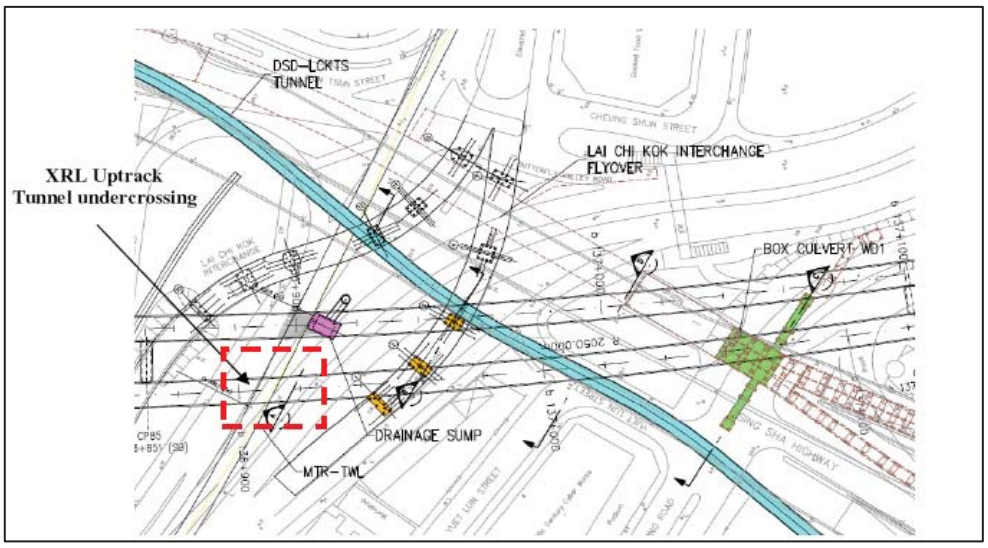


Figure 4: Interfacing of XRL Tunnels with MTR Tsuen Wan Line and LCKTS Tunnels

Both the XRL northbound and southbound tunnels will be driven across and above the LCKTS drainage tunnels with a separation of about 2 m. A number of drill holes were sunk under both the LCKTS and XRL projects in the vicinity of the tunnel crossings to facilitate a reliable assessment of the interface geology. Cross-hole seismic geophysical tests were carried out in order to determine soil and rock design parameters such as deformation modulus and Poisson's ratio to facilitate the impact assessment on the LCKTS tunnels. It was noted that the LCKTS tunnels would be fully embedded in Grade III or better competent rock with a cover of about 6m. The XRL tunnels were expected to be driven near the soil/rock interface. Such interpretation has been confirmed by geological information collected during the construction of the LCKTS in 2011. Consideration was given to fissure grouting at the tunnel crossings to eliminate the risk of damage to the completed LCKTS caused by any rock wedges moved by the advancement of the XRL tunnels. The drill holes did not reveal the presence of any boulders and grouting was subsequently considered not necessary.

As construction of the LCKTS project would start before that of the XRL project a detailed assessment was carried out by the XRL designer on the effect of the XRL tunnel construction on the LCKTS. The analysis by finite element programme PLAXIS covered a number of sections along the XRL tunnels at the crossings where the separations between the two tunnels vary. The results indicated that the impact on the LCKTS due to the XRL tunnel construction would be within the acceptable criteria. To safeguard the safety and integrity of the LCKTS tunnels, extensive geotechnical instrumentation has been proposed to monitor vibration, ground movement and the structural movement of the tunnels during the XRL tunnel driving. The monitoring system involves a remote monitoring system inside the LCKTS which can feed results back to the main monitoring station under both wet and dry conditions.

About 30 m north of the XRL / LCKTS tunnel crossing the XRL will have to cross underneath the MTR Tsuen Wan Line (TWL) tunnels. At this location, the existing TWL tunnels are in a stacked configuration with the up-track tunnel immediately above and running parallel with the down track tunnel. The vertical separation between the TWL down track tunnel and the XRL tunnels is approximately 2.8 m.

At the position of the XRL northbound tunnel under-crossing, the rock pillar separating the TWL and XRL tunnels is expected to be entirely within competent Grade II/III granite. Nonetheless, it is intended to excavate the XRL tunnel in closed TBM mode, in order to minimize the risk of movements affecting the TWL tunnel. The XRL southbound tunnel will pass underneath the TWL tunnel within a mixed face of completely decomposed granite potentially containing core stones and Grade II/III weathered granite. In order to reduce the risk of displacement of the TWL tunnels, two rows of horizontal pipe piles have been installed from within an existing TWL sump to form a canopy over the XRL tunnels to prevent possible face instability mechanism should the face/slurry pressure in the TBM be lost for any reason. The pipe pile canopy are installed in sections and in order to avoid welding in the sump (due to fire and ventilation requirements), whilst maintaining the required pipe stiffness, "threaded nipple coupling" has been adopted to join the pipes.

6 MAJOR GEOLOGIOCAL FEATURES

The XRL tunnels will cut across a number of major geological features including the Tolo Channel Fault in the Lai Chi Kok area, the Sham Tseng Fault in the Tai Mo Shan area, and the marble areas in Mai Po at the northern end of the Hong Kong section.

In broad terms the Tolo Channel Fault is anticipated to comprise a central fault zone of very poor rock mass condition flanked by an outer fault zone which comprises intermittent good and poor mass conditions. Beyond this is a broader zone in which discrete zones of closely spaced structure located in an otherwise competent rock mass will be encountered trending parallel to the main fault. The total fault zone width is about 260 m.

In 1990s, the DSD's Strategic Sewage Disposal Scheme Tunnel passed through similar geology when it was constructed in rock TBM below the harbour approximately 1.3 km to the west of the XRL alignment. The water inflows associated with the Tolo Fault zone were reported to be very high and significant additional grouting was undertaken ultimately in order to achieve the required permanent inflow to the tunnel. Although there are significant differences between the two tunnels in both hydrogeological setting and methods of construction, concern however remains that grouting ahead of the face may not be sufficient to meet the permanent groundwater inflow criteria in the wide zones of fractured rock and that significant quantities of grout will be required post-excavation with limited confidence that the permanent inflow criteria will be achieved. The drill-and-blast tunnels are in general designed as a drained structure allowing a limited amount

of groundwater getting into the tunnel and then discharged. In order to mitigate these concerns, an undrained lining has been specified for the section of tunnel crossing Tolo Channel Fault.

The Sham Tseng Fault is the only major extensive fault zone to cross through the Tai Mo Shan section of the XRL tunnels. The geological conditions in these areas can be expected to include zones of preferential deeper weathered zones, highly fractured rock which will potentially give rise to complex hydrogeological conditions. The West Rail Tai Lam Tunnel as built drawings provided useful information on the geology and rock mass conditions that can be expected at the Sham Tseng Fault. As mapped Q-Values ranged from 0.05 to over 100 in the northern section showing zones of more fractured rock than the southern section which was relatively sound with Q-Values ranging between 1 and 70.

Significant water inflows can be expected when the XRL tunnel is constructed through the Sham Tseng Fault, with the possibility of hydrothermal inflows (as recorded during the construction of the WSD treated water tunnel between Tai Po and Butterfly Valley). Probing in advance of the tunnel face can identify the presence of such inflow areas and, if encountered, forward grouting can be used to control the water ingress.

At the northern end of the HKS near Mai Po impure marble with relatively small size cavities has been identified. The marble encountered in the Mai Po area belongs to the Yuen Long Formation. The largest cavity encountered during the ground investigation within the study area was 5.4 m vertically as noted in one borehole and was believed to be partially infilled. The GI contractor noted the drill string dropped suddenly upon encountering the cavity and there appeared to be no increase in friction (torque required from the drill) for the first 0.8 m following the drop of the drill string. The GI contractor conducted SPT's at 2 m intervals thereafter, the first of which gave N=9 and the second N=40. Three less significant cavities with vertical extent ranging between 1.1 m and 1.6 m were intersected by other drill holes. Numerous smaller cavities, typically less than 0.5 m in vertical extent, were also intersected in numerous drill holes.

Most of the cavities are expected to be off the tunnel alignment. The possibility of cavities at or below the XRL tunnels invert level however could not be ruled out. To mitigate the risk of TBM instability caused by possible larger cavities from the marble outcrop, specific requirements have been included in the tunnel contract for continuous probing ahead from the TBM cutter head along the section where marble is likely to be present. As a minimum, the zone to be probed shall be two tunnel diameter in width and one and a half tunnel diameter in depth, measured from the TBM axis. A pattern of probe holes shall be drilled at intervals to ensure that all voids measuring more than 3 m in any direction are located. Where voids are identified the contractor is required to fill the voids with grout before tunnelling could proceed.

7 COMMISSIONING IN 2015

Construction of the HKS commenced in January 2010 and works are currently in full swing from West Kowloon to Mai Po. The HKS of the XRL is expected to be in operation in 2015 to connect with the fast expanding high-speed rail network in Mainland China.

Geotechnical Aspects of the Main Tunnel for Lai Chi Kok Drainage Tunnel

L.J. Endicott
AECOM Asia Co. Ltd., Hong Kong
W.C. Ip
Drainage Services Department, Government of the Hong Kong SAR
M. Plummer
Leighton-John Holland Joint Venture, Hong Kong

ABSTRACT

A new drainage scheme is being constructed at Lai Chi Kok. There are six intake structures along Ching Cheung Road and Tai Po Road feeding into the Branch Tunnel. The spillway from the 2.3 km long Branch Tunnel joins with the flow in Butterfly Valley and there is a drop shaft to the Main Tunnel. The Main Tunnel extends for 1.1 km under Lai Chi Kok to a riser shaft and outfall structure to discharge into the sea.

The Main Tunnel is located along the extension of Butterfly Valley in an area primarily formed by reclamation. It passes close to the foundations of highways viaducts and underneath the MTRC's Tsuen Wan Line tunnels.

During the planning and design stage there were many issues. The invert extends to some 42 metres below sea level and compressed air working would exceed the published tables for Hong Kong. The geology includes rock, mixed ground soil and rock, and soil. The alignment passes close to foundations, operating rail tunnels and a box culvert. There were concerns about the potential for creation of sinkholes at the surface and escape of bentonite slurry from the tunnel boring machine.

Mitigation measures were incorporated in the contract. Geological risk was shared by including a Geotechnical Baseline Report. Safe havens were incorporated in the reference design where the tunnel boring machine could be serviced without having to use compressed air.

During construction, with agreement from Commissioner of Labour, compressed air working was carried out up to 4.2 bar, which is a record for Hong Kong. There were no incidents of barotraumas. The Main Tunnel drive broke through in December 2011. There were no incidents of escape of bentonite slurry and no sinkholes at the surface. Ground movements were kept under control and there were no adverse effects on the nearby structures or facilities.

1 PURPOSE OF THE PROJECT

The Lai Chi Kok Drainage Tunnel project was developed on behalf of Drainages Services Department of the Hong Kong SAR Government, as part of the drainage improvement strategy for West Kowloon, incorporating a tunnel system to divert storm flows from upland catchments directly into the sea instead of allowing flows to pass through the downstream urbanised areas. The scheme is intended to relieve the capacity of the drainage system in the downstream urbanised areas and negate the need to upgrade the existing drainage system via conventional rehabilitation and replacement work. Such work would involve major disruption and inconvenience to the public and commercial activities over a sustained period during the improvement works. The public would also benefit from the future alleviation of flooding which in recent decades has caused significant impacts socio-economically.

2 DESCRIPTION OF THE PROJECT

A location plan of the project is provided in Figure 1. The Lai Chi Kok Drainage Tunnel consists of 3.4 km of 4.9 m diameter tunnel, together with six intake shafts located at existing stream paths at the north of the Sham Shui Po, Cheung Sha Wan and Lai Chi Kok urban areas. The intake shafts are located along Ching Cheung Road and are designed to intercept rainwater run-off during severe rainstorm events. The 2.3 km long Branch Tunnel is a gravity tunnel which collects the flow from the intakes and joins the watercourse of Butterfly Valley. At Butterfly Valley, there is a large intake structure and a drop shaft, some 48 metres deep. Water flow is conveyed beneath the developed area of Lai Chi Kok via the drop shaft and the 1.1 km long Main Tunnel, and then up through the rising shaft and via a cascade to discharge into the sea. The whole of the Main Tunnel operates as an inverted siphon.

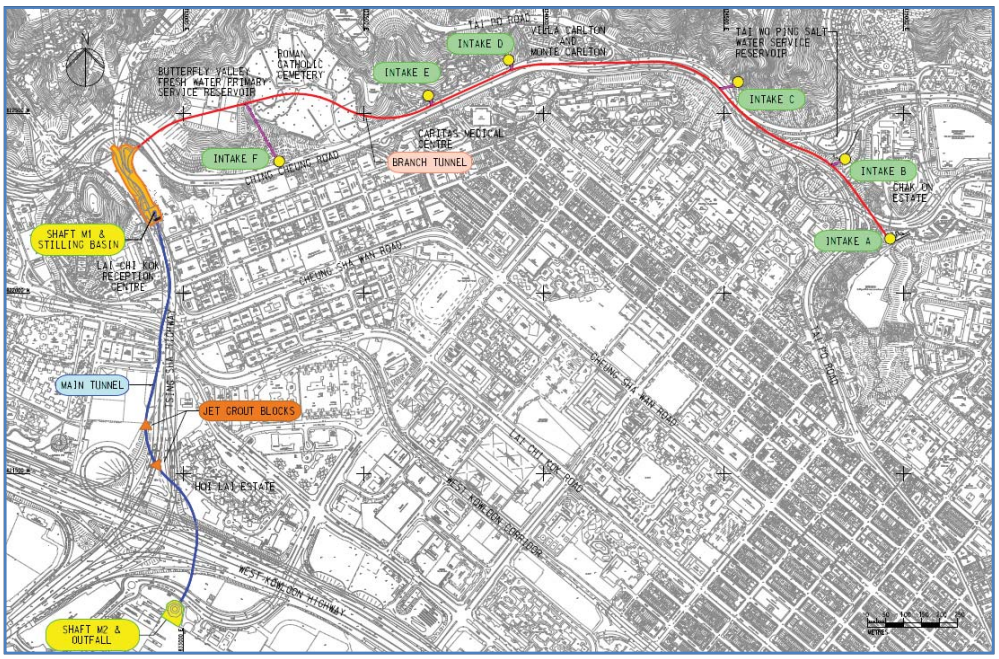


Figure 1: Site location

3 SITE CONSTRAINTS

Driving a new tunnel through a developed urban area presents many site constraints, see Figure 2. Owing to the land status of the highly urbanized Lai Chi Kok area, the alignment for the Main Tunnel was chosen to follow that of an existing main box culvert. However the tunnels for the MTRC's Tsuen Wan Line and the two proposed Express Rail Link tunnels cross the route requiring an alignment to be located at about 42 metres below sea level. The presence of underground obstructions due to piling for the highways viaducts and for the box culvert led to a double curved alignment in order to avoid the obstructions.



Figure 2: Site constraints

In opting for this tunnelling at-depth solution during the design development stage, a balance had to be struck between a number of key factors including hydraulic performance, construction risk, land issues, safety, disturbance to the public, cost, time and operation and maintenance. The option of constructing the Main Tunnel even lower at say, 80m depth, but in hard rock, was ruled out due to the increased construction risk and the increased operation and maintenance costs.

Location of intakes along Ching Chung Road provided very little works area for construction. Intakes structures were designed to be as compact as possible and the horizontal alignment was controlled partly by ownership of land.

4 GROUND CONDITIONS

Archived data and project specific ground investigation revealed that the site is located where the bedrock is granite. The granite is faulted in places and is weathered. The alignment of the Branch Tunnel is entirely in granite rock. The Main Tunnel is located partly in rock and partly in weathered rock, completely decomposed granite, CDG. The drop shaft and the rising shaft for the main tunnel are located in areas with filled ground overlying marine or alluvial soils resting on weathered granite. A simplified geological elevation is shown in Figure 3.

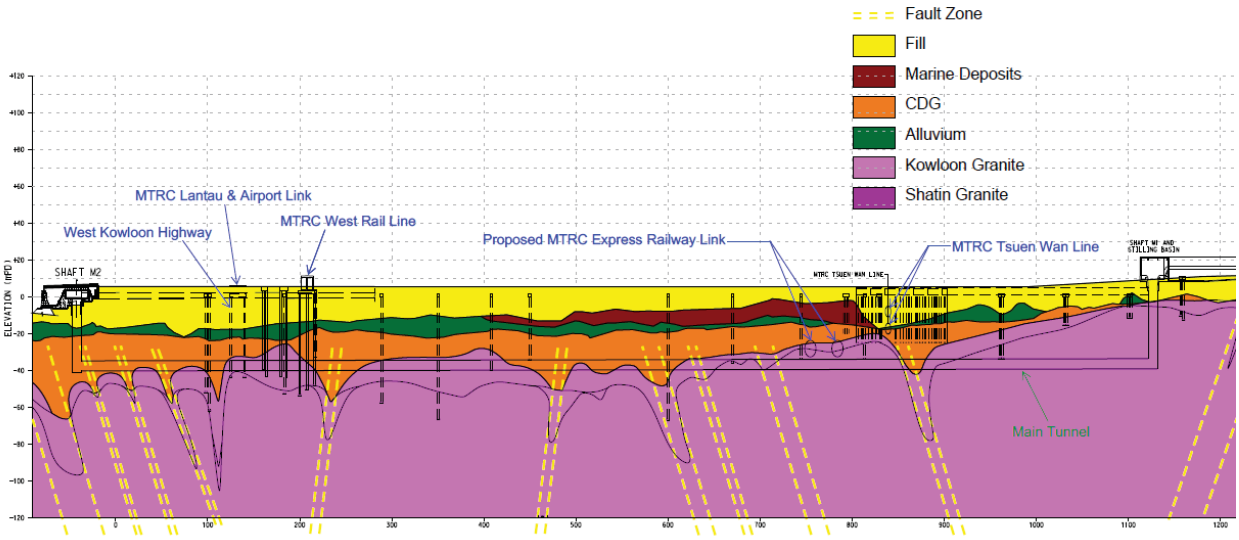


Figure 3: Simplified geological long section

At the time of the tender there were two major concerns. One concern was the requirement to operate at 42 m depth below sea level in the Main Tunnel. The other concern was the prospect of boring through mixed face ground conditions as the tunnel route transited from rock to CDG. Ground investigation data indicated that the transitional zones of mixed face conditions comprising soil and rock would amount to some 450metres of the length of the tunnel. It was anticipated that the rates of progress in mixed ground would be slow, that rates of wear of the cutters would be high necessitating frequent stops for inspection of the cutters and their replacement. Such interventions would require operating under excess compressed air pressure of 4.2 bars whereas Labour Department imposed published tables in Hong Kong for working under compressed air up to only 3.45 bar.

In addition there was some concern over the use of bentonite slurry regarding the possibility of excessive ground movements, due to either loss of ground or due to escape of bentonite slurry to the surface.

5 PRECAUTIONARY MEASURES

Several measures were adopted to facilitate the tunneling operations. Along with several other tunnel contracts that have commenced construction recently, this contract included a Geotechnical Baseline Report. The report includes baselines for geotechnical conditions. If the actual conditions that are encountered are significantly worse than the baseline then the changed ground conditions can be identified and remedy can be sought under the contract. By this means a lot of the geological risk is held by the Employer. However the contract is still in progress and it is premature to comment on the implementation of the GBR for this contract.

Tunneling through mixed ground conditions comprising soil and rock at the face can be very problematical. In order to mitigate the anticipated ground conditions, “safe havens” were established in the vicinity of where the tunneling was expected to transition between rock and mixed ground. At these locations it was intended to conduct long interventions to effect changes of cutters and servicing of the cutter head. Working in compressed air can be carried out only for short durations and extended working required lowered, or no, excess air pressure. In order to achieve safe working conditions, a block of ground was grouted to reduce its permeability such that compressed air was not necessary for men to carry out maintenance of the cutter head.

Control of the bentonite slurry to prevent loss of ground or escape to the surface was considered to be a workmanship issue under the control of the TBM operator.

6 ANTICIPATED TUNNELING CONDITIONS

Whilst the Branch Tunnel was primarily in granite rock and away from susceptible structures, the Main Tunnel was required to pass through a heavily congested urban area with numerous existing utilities, deep piles belonging to overhead highway structures and various other, charted and uncharted underground structures. The tunnel also had to pass beneath four operating, and extremely busy, rail lines, both

underground and surface, and an existing drainage culvert. The only possible way for the proposed tunnel to avoid these obstructions was for it to be aligned vertically at a depth of 40m. This placed it in very mixed geology as it coincided with the assumed rock head level, the rock/soft interface, where it was anticipated that the nature of the ground would change frequently and where there would be a large number of corestones of various shapes and sizes. Horizontally, it was necessary to introduce 200 m radius curves into the tunnel alignment to squeeze it through the existing obstructions, in places with clearances of only 2 m. Very careful accuracy of survey and control of alignment was required.

Tunnelling at such depth, in such ground conditions, required the use of a closed-face slurry Tunnel Boring Machine (TBM) to balance earth and hydrological pressures exceeding 4.2 bar. Whilst use of high pressure compressed air for TBM cutterhead maintenance was not uncommon practice in the industry overseas, tunnelling at these pressures had not previously been tried in Hong Kong and exceeded the limit (3.45 bar) imposed by existing Labour Department regulations.

7 PERFORMANCE

Although the reference design proposed two tunnel boring machines, the contractor proposed to use one tunnel boring machine for both tunnels. He selected to use a slurry type TBM built to order for this project by Herrenknecht. The slurry treatment plant was located in the works area at Butterfly Valley. From here the TBM was launched for the first drive that is for the Branch Tunnel, going slightly uphill. Plate 1 shows the TBM at the launch site. Plate 2 shows partial assembly of the TBM with man locks in place. The shaft at intake A was enlarged in order to receive the TBM at the end of the drive.



Plate 1: TBM assembly for the branch tunnel



Plate 2: Assembly with air locks for man-entry for hyperbaric operations

Recovery of the TBM provided an opportunity to thoroughly inspect all of the equipment and to service the machine and to make repairs. These included welding cracks in the cutter head and rebuilding the mounts for some of the cutters.The TBM was then re-assembled at the bottom of the drop shaft and it was driven towards the rising shaft.

The tunnel boring with simultaneous erection of reinforced segmental concrete lining took 313 days for the Branch Tunnel and 206 calendar days for the Main Tunnel. Overall average rates of progress were 7.4 m per day in the Branch Tunnel and 5.31 m per day in the Main Tunnel.

Two safe havens were adopted. Jet grouting was adopted from the ground surface requiring a works area which was difficult to locate. See Plate 3.



Plate 3: Jet grouting from the surface

No artificial obstructions were encountered by the TBM. There were no reports of escape of bentonite slurry to the surface and no reports of sinkholes at the surface. Monitoring of ground water and ground movements was carried out during the tunneling and there are no reports of adverse settlement or change in groundwater conditions. There are no reports of adverse deformation of MTRC's Tsuen Wan line operating tunnels.

8 COMPRESSED AIR WORKING

The use of compressed air was essential and compressed air locks had to be specified and built into the TBM right from the start. However, a major task was to establish a means of safe working under high working air pressures. Specialist advice was sought from local, French and German medical practitioners. Provisions that were made included getting permission for overseas medical specialist to work in conjunction with local doctors, for staff training in the proper use of compressed air and for dealing with problems should they occur when working under compressed air. Permission was obtained firstly for the use of French Tables for pressure of up to 4.2 bar and then for use of German Tables for working at pressure sup to 4.2 bar and for longer durations than allowed in the French Tables.

The performance in use of compressed air was fully successful. In total 90 numbers of hyperbaric interventions were carried out including working at 4.2 bar without any decompression illness.

9 CONCLUSIONS

The Contractor's choice of using one slurry TBM for both tunnels proved to be successful. During construction, with agreement from Commissioner of Labour, compressed air working was carried out mostly at 3.9 bar but going up to 4.2 bar, which is a record for Hong Kong. There were no incidents of barotraumas. The Main Tunnel drive broke through in December 2011. There were no incidents of escape of bentonite slurry nor of sinkholes at the surface. Ground movements were kept under control and there were no adverse effects on the nearby structures.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the Director of Drainage Services Department, the Government of the Hong Kong Special Administrative Region and AECOM Asia Company Limited for permission to publish this paper.

Hong Kong West Drainage Tunnel – Review of Key Geotechnical Aspects

Robert A. Evans & Louis C.T. Wong
Ove Arup & Partners Hong Kong Ltd

Calvin Cheung
Dragages Nishimatsu Joint Venture

Franky F.K. Pong
Drainage Services Department, Government of the Hong Kong SAR

Lawrence S.Y. Lee
Geotechnical Engineering Office, Civil Engineering & Development Department, Government of the Hong Kong SAR

ABSTRACT

Hong Kong West Drainage Tunnel (HKWDT) will intercept stormwater drainage between Tai Hang and Cyberport, effectively safeguarding the most densely populated areas of northern Hong Kong (HK) Island from the risk of flooding. In January 2011, two large diameter hard rock tunnel boring machines (TBM) completed breakthrough of the 10.7 km main tunnel, which is the longest drainage tunnel in HK. The whole drainage scheme comprises over 21 km of interconnected tunnels, adits and dropshafts, which have provided a wealth of information on structural geology, fault locations, ground and groundwater conditions. Eight major faults, numerous subsidiary faults, shear zones and contact zones were encountered. The geological structure encountered and particularly the groundwater conditions were the major factors controlling the progress of the TBM's. Major grouting works were required in the west of the HKWDT in the Telegraph Bay Fault and most notably in the Sandy Bay Fault where ground conditions were extremely poor. All information collected and practices adopted will be provided to the tunnel database maintained by the Geotechnical Engineering Office (GEO). The data provided will make a major contribution to existing knowledge on ground conditions and tunneling in HK.

1 PROJECT HISTORY

Rapid urbanization, change in land use and a substantial increase in the paved area of HK have resulted in a significant increase in surface run-off, affecting much of the lower catchment area of Northern HK Island. Despite local improvements to the 50 years old drainage system, flooding remains a problem in the summer months. During heavy rainstorms, high quantities of storm water run-off can flow from the steep upper hillside catchment down into the lower urban areas, causing flooding, traffic disruption damage to property and in extreme cases potential risk to life.

In response to this and to meet the community's increased expectations for higher flood protection standards, the Drainage Services Department (DSD) commissioned a drainage master plan study for Northern HK Island to assess the existing drainage systems in the area. The drainage master plan study recommended three key components to be implemented, one of which is HKWDT.

2 PROJECT DETAILS

Ove Arup and Partners HK Ltd (Arup) were commissioned by DSD in March 2006 to undertake the design, tender and construction supervision of the HKWDT. The whole system is designed to discharge a peak flow of 135 m^3/s to the outfall at Cyberport equivalent to a 200-year storm event. The construction contract was

awarded to the Dragages–Nishimatsu Joint Venture in November 2007 and is now nearing completion in 2012.

The main tunnel is the longest drainage tunnel in HK running from Tai Hang in the east to Cyberport in the west with a distance of 10.7 km. The main tunnel was excavated by two hard rock double-shield TBM with excavation diameters of 7.2 m for the eastern TBM tunnel and 8.2 m for the western TBM tunnel. The main tunnel branches into over 8km of approximate 3m diameter horseshoe shaped horizontal adits, which were excavated by drill-and-blast method. Adits connect to vertical dropshafts up to 170 m deep, that were excavated by a variety of methods, the most common being raise boring (23 nos.), other shafts being constructed by hand dug caisson, mechanical excavation and reverse circulation drilling. This is the first project in HK to systematically rely on the use of raise boring for vertical shaft excavation, a major and potentially problematic component of the project. At the top of the dropshafts, intake structures intercept the existing stream courses and culverts. The intake sites are scattered throughout the urban fringe including the Mid-Levels Scheduled Area (MLSA). Each intake is a mini construction site with deep excavation and lateral support (ELS) works generally formed by pipe pile cofferdams. Stability and surface settlement have been of paramount concerns necessitating much instrumentation which has been collated into a Tunnel Data Management System for close monitoring by the site supervision team.



Figure 1: Overall scheme of HKWDT

3 DESIGN CRITERIA & PROCESS CONTROL

HKWDT is designed mainly as a drained tunnel dependent on pre-excavation grouting to reduce water inflow during construction to a set of acceptable limits defined in the construction contract:

(a) 0.2 litre/minute (L/min) per metre of probe hole ahead of the excavation face and not more than 1 L/min from any 5m length of probe hole;
(b) 10 L/min over any 100m length of excavated tunnel, adit or dropshaft;
(c) 2 L/min through any excavation face; and
(d) 300 L/min at any portal.

Probing was carried out typically up to 30 m ahead of the TBM and water inflow was measured from the probes. If criteria (a) was exceeded then pre-excavation grouting was carried out by intruding micro-fine cement into the rock mass through the probe holes which passed through circular openings in the front of the rear shield. After completion of grouting, an additional probe hole was carried out to verify the performance of the grouting works. Further grouting would be performed if necessary. For zones of high groundwater inflow such as the Sandy Bay Fault up to 2 % micro-silica was added to the micro-fine cement to ensure better

penetration of the rock mass. It was fundamental that during the grouting operation skilled operatives closely monitored the actual site situation and reviewed the performance of the grouting works to select the most suitable grout materials, mixes and pressures compatible with the observed ground conditions.

4 KEY GEOTECHNICAL ASPECTS

With over 21 km of interconnected tunnels, adits and dropshafts, the project has provided a wealth of information on ground and groundwater conditions, fault locations and characteristics which will be extremely useful information for other underground projects in Hong Kong. The majority of current observations below are based on the TBM tunnels, with data from the adits and dropshafts still under review. When complete all data will be provided to the tunnel database maintained by the GEO for record and technical development, including the continuing update of geological maps by the Hong Kong Geological Survey of GEO. GEO have been heavily involved throughout the project providing useful comments on the construction contract and commenting on design submissions particularly those relating to the MLSA where seven Intakes and associated adits and dropshafts are located.

4.1 Regional geology

Based on the published geological data (Sewell et al, 2000a) and limited ground investigation information it was anticipated that the fine to medium grained granites of Cretaceous age would predominate in the eastern part of the tunnel drive and also be found more locally near the Western Portal. Volcanic rocks, of similar age (Sewell et al, 2000b) were anticipated in the central parts of the scheme between Pok Fu Lam and Mount Cameron.

The pre-construction ground models were proved to be quite accurate particularly in the case of the granites. Some changes in lithology were noted underlying Jardines Lookout, where the change from fine to medium grained granites, was displaced 500m further west than anticipated and similar changes and more minor differences west of Victoria Gap where the fine grained granites were absent completely, instead replaced by medium grained granites.

For the volcanic rocks there were significant differences on the western side of the project where the change from fine grained granites to fine ash vitric tuff was originally expected between Pok Fu Lam and High West. In reality the geological boundary and highly fractured contact zone was some 550 m to the east running up to the subsidiary geological structure associated with the Sandy Bay Fault.

Before construction, it was anticipated that there would be large differences in the unconfined compressive strength (UCS) of the rocks encountered. Based on the ground investigation and laboratory testing, average values for the granites were approximately 156 MPa and for the volcanics 205 MPa, although it had been found that for some samples of fine ash vitric tuff a maximum value of 339 MPa was obtained in the vicinity of Victoria Gap. These typical values were confirmed on site when the contractor undertook additional testing and confirmed a maximum UCS value of 353 MPa. The western TBM was slowed down to penetration rates of less than 0.2 m/hour and TBM cutter discs and the integral bearing units experienced very significant wear and had to be changed on a daily basis. Unfortunately the extremely strong vitric tuff had coincided with a rock mass that had very few joints making it difficult for the cutters to ‘pluck’ the rock from the face, instead progress could only be made by a slow grinding operation.

4.2 Structural geology & faults

Based on published geological data and pre-contract ground investigation, eight major faults (Figure 1 & Table 1) and approximately twenty two subsidiary faults and photolineaments were identified, intersecting the alignment of the main tunnel. The three major faults identified in the granites of the eastern TBM drive were the Tai Tam Fault, Wong Nai Chung Gap Fault and Middle Gap Fault. All three faults were characterized by local shearing, minor fault breccia and occasional fault gauge over quite limited extents. Maximum groundwater inflow in the eastern faults was less than 2 L/min/m measured in probe holes and more typically 1 L/min/m and was therefore easily managed. Maximum grouting volume per 30m round of probes was 40 m^3 in the Tai Tam Fault and less than 20 m^3 in the Wong Nai Chung and Middle Gap Faults. The Wong Nai Chung Gap Fault was found to have major influence immediately to the east of the anticipated location as it was found in a dropshaft adjacent to the French International School on Blue Pool Road.

Table 1: List of faults anticipated in pre-construction stage

Fault	Anticipated chainage (m)	Actual chainage encountered (m)	Observed ground condition	Q Value	Infiltration rate in probing (L/min/m)
Tai Tam Fault	Ch 645	Ch 635 - Ch 655	Highly fractured rock mass	0.1 - 1	1.5 - 2
Wong Nai Chung Gap Fault	Ch 2130	Ch 2095 - Ch 2135	Highly fractured rock mass	0.1 - 10	0 - 0.2
Middle Gap Fault	Ch 3270	Not obviously identified	No adverse ground conditions observed.	-	0 - 0.2
Wanchai Gap Fault	Ch 4540	Ch 4050 - Ch 4500	Intermittent fractured rock mass	1 - 4	0 - 0.2
Magazine Gap Fault	Ch 5080	Ch 4800 - Ch 5270	Intermittent fractured rock mass	1 - 4	0.5 - 2
Victoria Gap Fault	Ch 6570	Ch 6510 - Ch 6535	Highly fractured rock mass	0.1 - 1	0.5 - 1
Sandy Bay Fault	Ch 8360	Ch 8305 - Ch 8335	Highly fractured rock mass	0.1 - 1	0 - 0.5
Sandy Bay Fault (Subsidiary)	Ch 8960	Ch 8985 - Ch 9065	Severely fractured rock mass	0.03 - 1	0.2 - 1.5
Telegraph Bay Fault	Ch 10160	Ch 9455 - Ch 9950	Severely to highly fractured rock mass	0.03 - 1	0 - 3.5

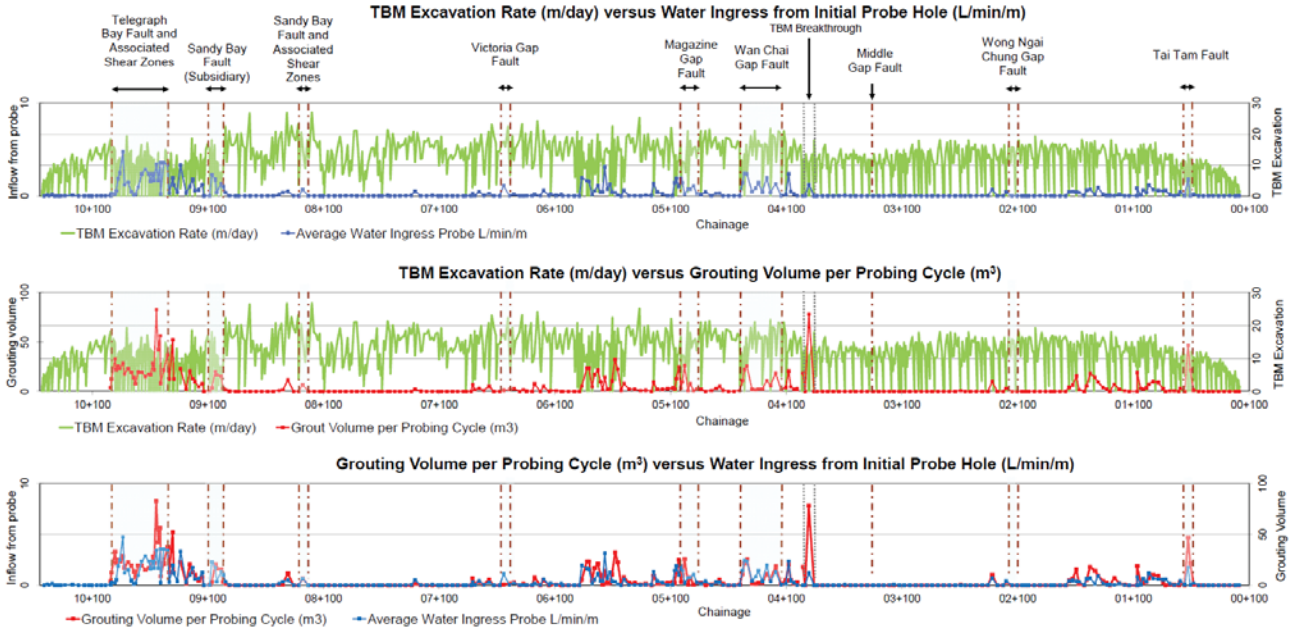


Figure 2: Summary of water inflow, grout quantities & TBM excavation rate

The western TBM experienced little difficulty with the Wanchai Gap Fault, Magazine Gap Fault and Victoria Gap Fault where groundwater inflows measured in probe holes were less than 2 L/min/m and grout volumes less than 30 m^3. 600 m to the west of the Magazine Gap Fault, further faulted ground was encountered with similar water inflows and grout takes as the main fault.

The Sandy Bay Fault and Telegraph Bay Fault at the western end of the project were identified at the project planning stage as significant risks to progress of the western TBM. A major concern was that the tunnel alignment ran parallel or sub-parallel with both faults which is a well documented high risk situation (Barton N., 2006), which has led to TBM on other projects having become stuck and even abandoned. Parallel fault zones tend to destroy much of the tangential stress necessary to ensure stability of the arch and crown of

the tunnel. In addition, it was known that both the extent and severity of the ground and groundwater conditions could be quite extreme. Options for tunnel re-alignment of the tunnel near the Telegraph Bay Fault were quite limited owing to the fixed location of the outfall at nearby Cyberport, however, re-alignment was possible near the Sandy Bay Fault. Under the original alignment it was possible that the alignment which is in a 600 m radius curve at this location could even pass through the fault twice. After further ground investigation in 2006 and review of the most likely incidence of the fault at tunnel level, the tunnel alignment was moved 50 m to the south of the main fault (Figure 3). Ultimately this re-alignment proved vital to the success of the project, as extremely poor ground conditions were encountered in the re-aligned section peripheral to the main fault. Had the hard rock TBM attempted to pass through the main part of the fault then the impact to the project could have been severe.

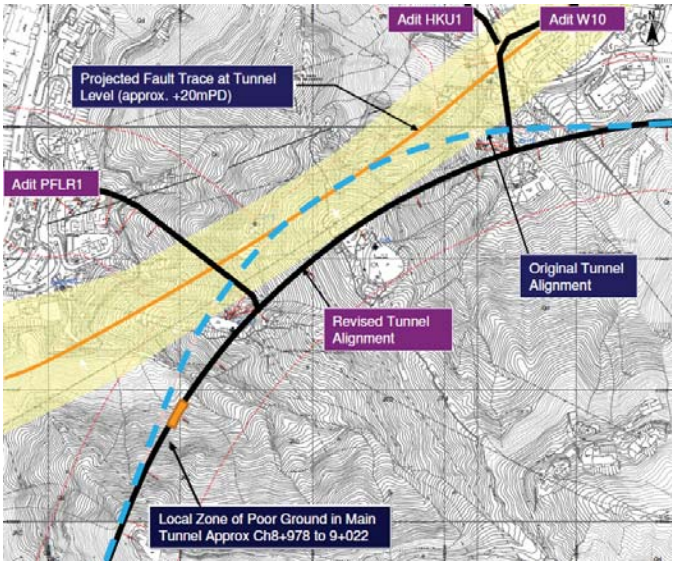


Figure 3: Re-alignment for Sandy Bay Fault

A major geological structure associated with the Sandy Bay Fault was encountered underlying Lung Fu Shan Country Park to the south west of the main fault location. In an approximate 44 m wide zone, Q values dropped to 0.03, in a highly fractured, very blocky, fault breccia with initial groundwater inflows into the tunnel in the region of 100 L/min. Due to the low strength of the rockmass, the tunnel face frequently failed and collapsed into the cutterhead with resultant overbreak around the TBM. The ground squeezed around the TBM which could only be advanced in single shield mode advancing the TBM by pushing off the tunnel lining, as the grippers were experiencing bearing capacity failure in the faulted ground. Planned emergency remedial works were agreed and implemented, as cavity grouting and secondary grouting were implemented. Convergence monitoring was installed confirming the stability of the tunnel lining.

The Telegraph Bay Fault also ran parallel to the tunnel alignment. Although the ground conditions were not as severe as the Sandy Bay Fault, water inflow from probe holes frequently attained over 3.5 L/min/m over an extensive length of tunnel 400 m in length. This necessitated major pre-excavation grouting which commonly attained 20 m^3 to 40 m^3 of grout per 40 m length of grouting cycle and on several occasions between 50 m^3 and 80 m^3 per grouting cycle. In such ground the time spent grouting a full round of grout holes would typically take up to 48 hours which adversely impacted the overall progress of the TBM.

5 GROUND CONDITIONS & TBM PROGRESS

Comparisons have been made on the relative performance of the TBM: the eastern TBM driving in the granites and the western TBM predominantly in volcanic tuff (Table 2). The relatively easier passage of the eastern TBM is highlighted by the fact that only 15 % of the operational time was spent probing ahead and associated grouting, as compared to 26 % for the western TBM.

Table 2: Comparison of production rates (m/day) for eastern and western TBM drives

	Average production rate (Fault zones)	Average production rate (Non-Fault Zones)	Overall production rate
Eastern TBM*	9.24	13.90	13.61
Western TBM**	7.27	10.97	9.86

*Eastern TBM production rates adjusted from 16 hours 6 days per week production cycle.
** Western TBM rates are for 24 hours 7 days per week production cycle.

In total, 703m (18.7 %) of the eastern TBM drive required ground treatment for water ingress compared to 1,800m (27.6 %) for the western TBM. The ground treatment for the respective tunnels are summarized in Table 3.

Table 3: Ground treatment summary for eastern and western TBM drives

	Number of probe cycles	Length of probes (m)	Number of grout holes	Length of grout holes (m)	Total volume of grout (m^3)
Eastern TBM	103	5,591	101	4,617	331
Western TBM	210	12,280	472	24,258	1,438

6 CONCLUSIONS

The HKWDT will meet a major social and business need by alleviating flooding problems in the densely urbanized residential and commercial areas of northern HK Island. The entire drainage tunnel system comprises over 21 km of interconnected tunnels, adits and dropshafts. The main tunnel was excavated by two hard rock double-shield TBM's with excavation diameters of 7.2 m for the eastern TBM tunnel and 8.2 m for the western TBM tunnel. The eastern TBM excavation was largely in granites and passed through two major faults, viz the Tai Tam Fault and Wong Nai Chung Gap Fault. Although fault breccia and fractured ground was encountered, water inflows into the tunnel were not severe and therefore managed through the normal pre-treatment grouting cycle. The western TBM excavation in volcanic tuffs and to a lesser extent granites, experienced more difficult ground conditions, both extremely strong vitric tuff with unconfined compressive strength up to 353 MPa and also severely faulted ground with significant groundwater inflows, associated with the parallel alignments of the Telegraph Bay Fault and Sandy Bay Fault systems. Systematic pre-treatment of the ground up to 40 m in advance of the excavation face was required using combinations of micro-fine cement grout and 2 % micro silica. Comparison is provided for the east and west TBM's in terms of cutting time, probing and grouting time relating to the ground conditions.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Director of Drainage Services, the Head of the Geotechnical Engineering Office, and the Director of Civil Engineering and Development, of the Government of the Hong Kong Special Administrative Region. The authors sincerely acknowledge the help from Mr CHEUNG Wah Fung Harry and Mr WAI Hung Leung David (Arup) for data preparation and collation.

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Knowledge Management and Development of Technical Guidance for Geotechnical Control and Risk Management of Tunnel Works in Hong Kong

H.W. Sun, Patrick K.S. Chau, Thomas S.K. Lam & H.M. Tsui
Geotechnical Engineering Office, Civil Engineering and Development Department, Government of the Hong Kong SAR

ABSTRACT

Tunnel works have been very common in infrastructure development projects in Hong Kong. As of 2011, more than 440 km of tunnels have been constructed, forming an integral part of Hong Kong's infrastructure. More tunnels and other underground facilities are being built or being planned to support the continuing population growth and economic development.

Tunnel works, especially those in the densely developed urban areas, could pose considerable risk to the public as well as the existing facilities. A regulatory system is in place to manage and control the associated geotechnical risk. This paper presents the background and gives an outline of the framework of the geotechnical control system for tunnel works in Hong Kong. The development of a tunnel database, risk management system and technical guidance as a means to facilitate risk management is summarised. Areas that warrant special attention by practitioners as identified from the recent tunnel projects are also discussed.

1 INTRODUCTION

1.1 Background

Hong Kong has a small land area of about 1,100 km^2 but with a population of some 7 million people. About 75% of the total land area is steeper than 15°, and over 30% is steeper than 30°. The steep topography and hillsides pose severe constraints to development and land is a scare resource. However, the strong granitic and volcanic rocks that form 80% of the land area of Hong Kong offer a great opportunity for placing transportation and other facilities underground. Tunnel works have long been part of the infrastructure development of Hong Kong. Pang & Woodrow (2009) and Pang (2011) provide comprehensive coverage of the past tunnel projects and the development of tunnel engineering in Hong Kong. The length of the tunnels constructed in Hong Kong since 1887 is shown in Figure 1. As of 2011, more than 440 km of tunnels have been constructed in Hong Kong, forming an integral part of Hong Kong's infrastructure. More tunnels and other underground facilities are being built or being planned to support the continuing population growth and economic development. By 2020, it is expected that approximately another 170 km of tunnels will have been constructed.

1.2 Tunnel construction

Tunnels in Hong Kong are built using different construction methods for the different ground conditions. Over the years, due to the need for constructing tunnels in highly complex and sensitive urban environment, the demand for advanced tunnel construction technology has increased, which has been met by technological development locally and overseas. As shown in Figure 1, there has been a drastic change in the construction methods for tunnelling. During the period of 1982 to 1991, the majority of the tunnels in Hong Kong were constructed by drill and blast method, with a small percentage constructed by slurry Tunnel Boring Machine (TBM) and open-face TBM with compressed air. However, from 2002 to 2011, the percentage of tunnels constructed by drill and blast method was reduced, and more tunnels were constructed by large-diameter

slurry, Earth Pressure Balance (EPB) and hard rock TBM. Innovative methods, such as ground freezing and horizontal directional drilling were introduced.

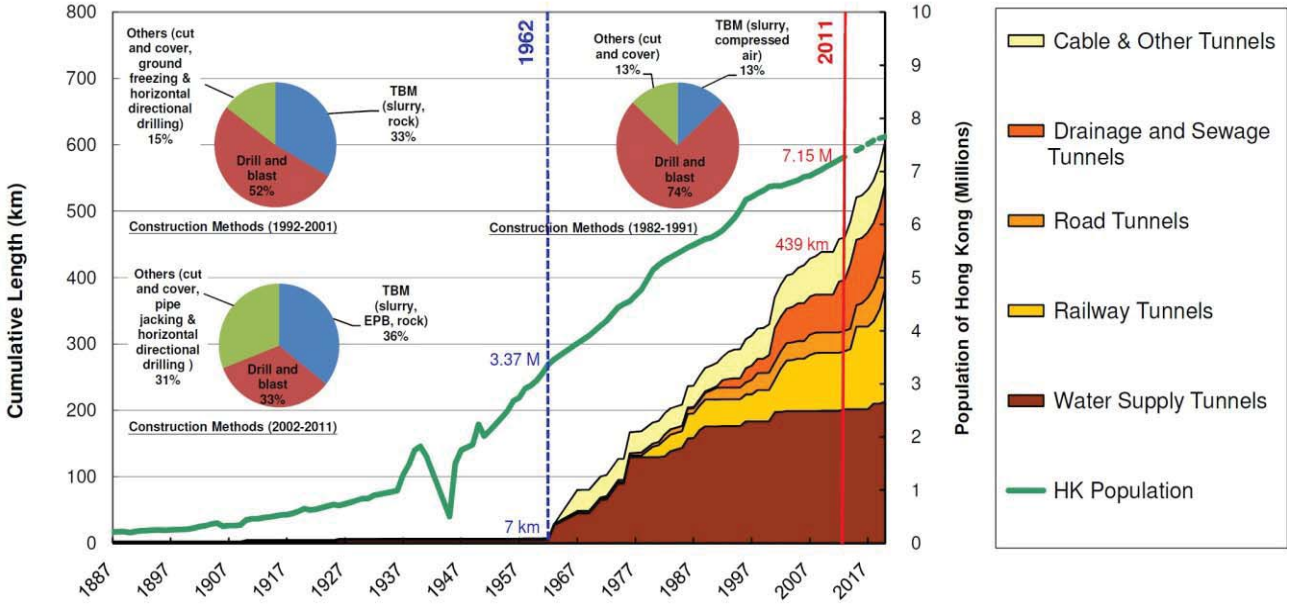


Figure 1: Tunnel construction in Hong Kong

2 GEOTECHNICAL CONTROL AND RISK MANAGEMENT

Being Government's in-house geotechnical specialists, GEO is responsible for geotechnical control of tunnel projects for protection of public safety. Control on private tunnel projects is exercised under the Buildings Ordinance. That for public works projects is exercised administratively under Environmental Transport and Works Bureau Technical Circular (Works) No 15/2005 (ETWB, 2005). Further details of the control regime are given in Pang & Woodrow (2009) and Pang (2011).

Depending on the funding arrangement, railway development undertaken by the MTR Corporation Ltd. (MTRCL) may be subject to geotechnical control under the Buildings Ordinance. However, Section 54(2) of the Mass Transit Railway Ordinance empowers the Buildings Authority to exempt the works from any provision of the Buildings Ordinance, having regard to the exceptional nature of building or other works connected with the operation. An Instrument of Exemption can be issued by the Building Authority to MTRCL subject to certain conditions, which would relief it of some of the procedural constraints under the Buildings Ordinance.

3 KNOWLEDGE MANAGEMENT AND DEVELOPMENT OF TECHNICAL GUIDANCE

To facilitate geotechnical control of tunnel works, it is important to consolidate expert knowledge, to develop geotechnical assessments and risk management framework for planning and construction of tunnels and to manage risk. Quantification tools exist to provide an open framework for exercising and deliberating judgements. GEO publishes guidelines on the standard of good practice and use of quantification tools, e.g., GEO Technical Guidance Notes (TGN) No 24 on site investigation (GEO, 2005a) and No 25 on geotechnical risk management (GEO, 2005b). GEO also issued a report that guides the specification and control of face support pressure in slurry TBM to minimise ground movement (GEO, 2009).

Recognising the fact that design and construction of tunnel works involves expert knowledge and experience as well as the rapid development of the technologies available, GEO has retained tunnel expert services since 2007 for calibration and benchmarking of the technical standards adopted in its technical audits with the international best practice.

Information is frequently collected during tunnelling, either as part of the risk control measures, or to provide record for future asset management. ETWB (2005) empowers GEO to collect such information from Government tunnel projects, for reference of future project proponents, as a means of facilitating risk management. Along this line, a centralised archive of Hong Kong tunnel logging records has been established by the Hong Kong Geological Survey in GEO to collect all factual geological and hydrogeological data from tunnelling works (Sewell et al, 2009).

GEO published a Catalogue of Hong Kong Tunnels and a Catalogue of Tunnel Failure Case Histories which can be accessed via CEDD website: http://www.cedd.gov.hk/eng/publications/geo/index.htm. The Catalogue of Hong Kong Tunnels disseminates information and references on major tunnels constructed in Hong Kong and serves as a source of reference for planning of new tunnel projects. The Catalogue of Tunnel Failure Case Histories was compiled from published information on 36 overseas and 6 local cases of ground collapse and excessive deformation between 1964 and 2008. This is to promote awareness of the profession on tunnel failures and lessons learnt. These catalogues are updated from time to time.

Proponents of Government tunnel projects may obtain from GEO geotechnical advice in respect of geology, engineering geology, ground investigation, laboratory testing, geotechnical risk management, and blasting control.

4 CHALLENGES OF RECENT TUNNEL PROJECTS AND TECHNICAL DEVELOPMENT

Driven by the demand of infrastructure development in an intensively urbanised environment, and armed with fast developing technology, tunnel engineers in Hong Kong are venturing into ever more taxing ground. With proper technology and due care and diligence, the difficulties could be overcome. Among them, we believe that the following are noteworthy.

4.1 Tunnelling adjacent to existing buildings and risk management measures

For TBM tunnelling underneath or close to existing foundations, a staged assessment approach proposed by Burland et al (2001) has been widely adopted to assess the tunnelling impacts on the existing buildings. In general, Stage 1 assessment will be carried out using a design volume loss of 1 to 2% to estimate the induced ground movements and settlements of the buildings along the tunnel alignment. If the maximum building settlements is less than 10 mm and the differential settlements less than 1:500, the degree of damage to the buildings will be considered 'very slight' and Stage 2 assessment is not needed for evaluating the maximum tensile strain induced onto the buildings based on the assessed ground movement profiles in the Stage 1 assessment. If the maximum tensile strain is less than 0.15%, the degree of damage to the buildings will generally be considered 'slight' and no further detailed assessment is required. Alternatively, Stage 3 assessment will be pursued, taking into account the stiffness of the buildings in the assessment of the tunnel induced ground movements and building deflections. For piled structures in close proximity to the tunnel extrados, additional checks on the adequacy of geotechnical capacity and structural capacity of the piles are required. Where needed, mitigation measures, including ground treatment, are required to minimise ground loss and to protect the buildings affected.

For assessment of tunnelling induced risk to adjacent piled structures, the current approach used by practitioners involves assessment of ground loss due to the tunnelling works (e.g. TBM) and the associated subsurface ground displacements. Performance of the piles due to the subsurface ground displacement is then analysed by adopting a pile deflection profile equal to the ground displacement profile corresponding to the specified ground loss or, perhaps more realistically using boundary element or finite element models of the pile group by imposing the predicted tunnel induced ground displacements to the groundmass (e.g. Loganathan, 2011). Assessment is then made to verify the adequacy of geotechnical capacity and structural capacity of the piles.

It has been recognised that analysis of performance of the existing pile foundations affected involves soil-structure interaction between the tunnel works, the pile foundation, the superstructure and the ground. For example, a better understanding on subsurface displacements as a result of various stiffening effects of existing piles may allow a more reliable risk assessment. As further experience and knowledge is being gained from academic research and the implementation of tunnel projects under such conditions, further

understanding of the problem is expected, which will lead to improvement in design and robustness of mitigation measures for future projects.

It is recommended by GEO (2009) that the TBM face support pressures should be checked for both Ultimate Limit State and Serviceability Limit State. For sensitive areas where there are overlying buildings, detailed assessment using numerical models may be required to justify the adequacy of the face support pressures. Other than monitoring of the ground and building movements for Alert, Action and Alarm, the instrumentation strategy and scheme to verify the design assumptions of the ground model (e.g. change in piezometric levels), as well as volume loss due to tunnel excavation may also be needed for sensitive buildings/facilities affected.

Compressed air interventions (CAI) for TBM tunnelling are high risk activities. The compressed air pressures adopted are generally based on the water head at the tunnel face and the drawn-down slurry level in the excavation chamber and should be controlled to avoid blow-out from the crown and heavy water inflow into the tunnel base. There are restrictions in Hong Kong on the use of compressed air pressures in excess of 3.45 bars, beyond which permission from the Labour Department must be obtained. Given the experience gained from the recent successful completion of the main TBM drive for Lai Chi Kok drainage scheme with CAI carried out up to 4.2 bars and with the publication of the ITA (2012) guidelines for good working practice in high pressure compressed air, the profession is better equipped in the use of CAI. For overall management/mitigation of geotechnical risks, all interventions for TBM tunnelling, especially tunnelling through sensitive areas, should be carefully planned at design stage.

4.2 TBM in marble ground

Marble is found under the Northwest New Territories, Ma On Shan and Northshore Lantau. The marble usually has a karstic upper surface with solution features. The main concerns for TBM tunnelling in an area of karst terrain are the very variable level of the rockhead and the possible presence of large cavities, either with or without cavity-infill deposits. The major risk would be the collapse of an undetected open cavity just under the tunnel due to the massive weight of the passing TBM. In cases where undetected open cavities are encountered by the TBM, the loss of face pressure could lead to face collapse and excessive ground settlement. Cavities in close vicinity of the tunnel may also have adverse effects on long-term performance of the tunnel lining.

In the ground investigation stage, geophysical methods such as microgravity and resistivity survey may be useful to provide a better resolution of the rockhead profile and extent of the cavities. Most of the ground improvement works, such as pattern drilling and grouting can be carried out from the ground surface using cementitious grouts to fill up the cavities. However, if surface access is not available, probing and grouting from within the TBM should be carried out. It is important for the TBM to be equipped with adequate probing and drilling capability to probe and treat any voids identified. In order to minimise the risk of undue movements of the tunnel lining and to fill up sizeable voids within the close vicinity of the tunnel extrados, a full 360° fan of proof-drilling and grouting should be carried out following tunnel lining installation and additional robustness and conservatism should be built in the detailed design of the tunnel lining. As a precautionary measure, slurry plants should have the capacity to store sufficient bentonite to fill voids or cavities unexpectedly intersected by the TBM. For a 10 km drainage cum road tunnel in Kuala Lumpur that mostly traversed a limestone formation, the TBM was supported with a slurry plant that had a storage capacity to provide 1,000 m^3 of fresh bentonite and 1,000 m^3 of used bentonite as contingency measures to prevent loss of face support pressure and ground collapse, in the event of unforeseen cavities and rock fissures (Yeoh & Klados, 2006). It is expected that local experience in TBM tunnel in marble ground will be gained from the design and implementation of the Express Rail Link currently under construction.

4.3 Break-in / break-out operations

Break-in and break-out zones of TBM drive are particularly vulnerable to ground collapse and it is common to undertake ground treatment outside the launching/receiving chambers. The length of the ground treatment zone using either jet-grouting or ground freezing method should be sufficient (e.g. at least one or two rings of lining engaged into the treatment zone) for break-in or break-out operations unless other measures, such as the steel bell installed within the launching chambers to house the TBM shield for break-in operations in the

MTRCL Express Rail Link Project (Contract 820), are in place to ensure water-tightness and effective building-up of adequate face support pressure. It is also of interest to note that ground freezing has been used successfully in the MTRCL West Island Line the Harbour Area Treatment Scheme (HATS Stage 2A) projects at TBM breakthroughs (Polycarpe et al, Leung et al, 2012 and Tsang et al, 2012).

4.4 Temporary rock support

The traditional approach for estimating the temporary tunnel support requirements based on the Q system has been widely adopted in Hong Kong for many rock tunnel projects. The Q system classifies the rock mass based on six basic parameters characterising the size of rock blocks, the shear strength along joint planes and the stress conditions around the excavation. Temporary supports/reinforcements are determined based on the classified rock mass quality, the tunnel size, and the sensitivity of the tunnel structure, as well as from mapping and assessment of local potential unstable blocks. Adequacy of site control to deal with unexpected ground conditions is of prime importance to avoid potential instabilities and possibly serious consequences. Unsupported, or partially supported rock faces should be re-inspected after each blast to identify signs of deterioration or potential instabilities. Supported faces should also be re-inspected regularly to identify signs of deterioration or damage. Based on experience gained from recent projects, several areas that require special attention of the practitioners are summarised below.

(a) Provision of Temporary Support. It is a common practice to have a temporary support schedule summarising different types of support for different rock classes expected for the tunnel excavation. It is also very important to present the maximum temporary unsupported lengths allowable for different rock classes in the schedule. It is important to identify through pre- and post-blast mappings potential kinematic instabilities of the ground and timely provision of supports. The need for pre-excavation reinforcement should also be specified in the temporary support in difficult ground, or where sensitive facilities are affected.

(b) Multiple Tunnel Excavation. Interaction and stress redistribution due to the excavation of adjacent tunnels should be assessed. Unavoidable slender rock pillars should be reinforced prior to the adjacent excavation. Bolt lengths of the supported tunnel may need to be extended to cater for the stress shadowing effects induced by an adjacent excavation. However, care should be taken to avoid penetrating through pre-grouted zones to minimise water inflow.

(c) Shallow Rock Cover. Geotechnical engineers should be aware of the basis and limitation of the Q system. Under shallow rock cover conditions, e.g. ratio of thickness of rock cover to tunnel span is below unity, the tunnel supports derived from the Q system should be further assessed using analytical models considering stability of the jointed rockmass as well as the effects of surface/foundation loads.

4.5 Hydrogeological assessment and inflow control

It has been documented that excessive groundwater inflow during the excavation of a deep rock tunnel may cause widespread ground settlement. The amount of groundwater inflow during tunnel excavation must therefore be controlled. Useful experience has now been gained from the successful application of high-pressure pre-grouting for deep shafts and tunnels in the HATS Stage 2A project (Garshol et al, 2012) and use of ground freezing in the MTRCL West Island Line to control water inflow into a gallery excavation (Polycarpe et al, 2012). Hydrogeological assessment taking into account rock mass permeability, orientations and locations of fault zones, transient nature of the groundwater flow and consolidation settlement in compressible ground as well as potential uncertainties of ground conditions are useful to assess sensitivity to adverse hydrogeological features/conditions (e.g. fault/fracture zones connecting to water source at shallow depths), to justify inflow control criteria during and after construction, and to establish triggering criteria for pre-grouting. Endicott et al (2012) provides useful references on hydrological assessments for tunnels in the HATS Stage 2A project.

5 CONCLUSIONS

Tunnel works have become an integral part of Hong Kong's infrastructure and GEO plays a key role in the geotechnical control and management of geotechnical risk posed by such works. GEO is responsible for geotechnical control of private tunnel works under the Buildings Ordinance, in partnership with Buildings Department; and for geotechnical control and risk management of public tunnel works by providing

comments/advice to Government project departments on the geotechnical aspects of the works. GEO is continuing to consolidate knowledge of good practice from recent tunnel projects and identifying areas requiring attention by the practitioners that warrant technical development. As part of its knowledge management effort, GEO has published technical guidance on management of geotechnical risk and efforts on this will be continued.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, Government of the Hong Kong SAR.

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Tunnelling in Difficult Ground: How the Geotechnical Baseline Report Helps

R. Perlo, M. Swales & T. Kane
Mott MacDonald Hong Kong Limited, Hong Kong
H.C.K. Louie & F.H.T. Poon
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

The design of tunnels and their subsequent construction can involve a high level of risk, especially with respect to unforeseen ground conditions or constructability issues. The management of such risks is essential and critical, and for the Tsuen Wan Drainage Tunnel (TWDT) was implemented by the inclusion of the Geotechnical Baseline Report (GBR) into Tender Documentation. The GBR included characterization of the geotechnical parameters and the geological conditions, in such a manner so as to allow the definition of an appropriate construction methodology, and optimal excavation and support techniques. This paper describes how the ground condition risk was managed on this Project utilising a custom-built Double Shield Tunnel Boring Machine (TBM), highlighting the use of detailed geotechnical characterisation as the primary driver for offsetting risk from unforeseen or changing ground conditions during tunnelling.

1 INTRODUCTION

1.1 The use of the geotechnical characterization in offsetting risk

The design of underground works has traditionally followed a deterministic approach, based on indirect management of the potential risks through a series of predetermined project decisions. In reality however both the design and construction phases are imbued with a certain degree of uncertainty, particularly with respect to the nature of the characteristics and their spatial variation, the behaviour of the rock mass and the level of knowledge of these factors acquired during the different stages of the Project.

Geotechnical characterisation for mechanized TBM tunnelling is of paramount importance, due to the low adaptability of such methods to changing ground conditions that are often inherent in linear underground excavations. Although TBMs are widely used, such is not a risk-free technology. Encountering geological and hydrogeological conditions different from those foreseen at the design stage may result in problems and risk, which may have significant effects on the program and safety, if such are not properly managed.

The basic concept of the GBR is to establish a realistic, common basis for all the contractors to use in preparing their bids and subsequently a basis for evaluating any contractor claims for differing site conditions that developed during construction (USNCTT, 1974). Therefore, the GBR shall be considered the baseline to define and evaluate the risk tolerance and the vulnerability of the Project, and to develop a managing scheme to address the foreseen risks (ITA, 2002). Furthermore, the GBR shall be considered complimentary to the allocation of responsibilities for managing and mitigating such risks, including the residual risk, and to increase the probability to be sheltered from economic lost and damages (Kovary, 2002).

Given the above factors, promoting the use of a detailed geotechnical and geological characterisation, as one of the primary drivers for offsetting risk from unforeseen or changing ground conditions, during the tunnelling operations, is considered essential to ensure the proper and efficient management of the risk.

1.2 Targets and residual risk

A main task of geotechnical design is to achieve the economic optimisation of the construction, taking into account the ground conditions, in particular the safety, the long term stability and environmental requirements. The variability of the geological conditions, including the structural geology, ground parameters and stress and ground water conditions requires that a consistent and specific procedure be used in the design process. The key influences governing the geotechnical design are the ground conditions and ground behaviour.

Thus, a certain degree of uncertainty and therefore a level of risk are not completely avoidable despite the experience, the time, and the costs incurred (Carol, 1992). In fact, the level of knowledge of geological parameters, which may well constitute the principal source of project risks, may be limited by the nature of the characteristics of the medium and the spatial variation and behavioural pattern of the rock mass (i.e. the geological asset). Furthermore, limitations in site investigation during the construction phase, must also be considered, especially when such could interrupt the tunnel advance.

1.3 The GBR

Detailed geological, geotechnical and hydrogeological characterisation, along the length of the TWDT alignment was undertaken, in the form of a GBR and a Geotechnical Data Report (GDR).This established the baseline ground conditions for the Contractor to take into account during the Tender and Contractor Design process, and these reports represented a contractual statement of the subsurface conditions be anticipated to be encountered during the construction of the tunnel.

The provision of such a detailed characterisation from the onset of project enables the Contractor to have a deeper understanding of the project, to be better able to optimise construction methods and techniques and to formulate a more detailed and informed Risk Analysis. The economic benefits, to both the Contractor and the Employer, are also baselined (Kovari, 2002).

1.4 Application of the GBR

The GBR describes the anticipated ground conditions with which the tunnel and associated structures will be constructed, and based on such the Contractor, and his Designer, can produce an effective design, formulate his construction methods and techniques and formulate an informed Risk Analysis and develop a comprehensive Risk Management Plan. The GBR details and describes the geotechnical and geological rock mass and material characteristics including rock mass and material strength, abrasivity, drillability and cuttability. It also details and describes the anticipated hydrogeology, in order that the control of groundwater, both in terms of drawdown, and the effects of such on existing buildings and structures (EBS) and features, and water inflow into the tunnel, can be ascertained. This is particularly pertinent for those areas of particular concern highlighted e.g. in the fault zones and areas of low rock cover. All of the above are relevant to the choice of the excavation technique, the type of TBM to be used and, the management of the design and construction, particularly with reference to efficiency and productivity which could be achieved.

2 THE TSUEN WAN DRAINAGE TUNNEL

2.1 Project description

The existing drainage facilities in the Tsuen Wan and Kwai Chung areas were constructed over 30 years ago. Those do not now have sufficient spare capacity to handle the additional stormwater run-off arising from the continuing urbanization of these and surrounding areas, resulting in greater risk of damaging flooding, particularly during severe weather conditions. The TWDT will operate by intercepting excess stormwater from the upland catchment areas and conveying such to direct discharge to the sea.

2.2 Project characteristics and geology

The TWDT has been designed as a 6.5 m internal diameter, circular profile tunnel, with a 7 m excavation diameter, and a length of approximately 5.1 km. It is concrete-lined with a 3,500 m undrained length.

The tunnel was driven through predominantly extremely strong and abrasive, fresh rock (tuffs and granodiorites of the Tsuen Wan Volcanic Group) with a variety of cross-cutting dykes (including basalt, rhyolite, fine-grained granite, and quartz). The tunnel alignment also traversed a number of faults and fault zones, generally consisting of blocky and highly fractured rock, and with a variety of mixed soil and rock conditions (e.g., from highly to completely decomposed rock, to mixed soils with corestones and residual soil). The effects of these ground conditions were exacerbated by the close proximity of the tunnel to existing major infrastructure (existing Water and Railway Tunnels) and locations with shallow overburden.

2.3 Project requirements

Severe constraints and specific requirements were imposed in order to minimise the impact of the construction to the public and on the environment, such as the adoption of a ground treatment programme for the stabilization of the tunnel excavation (it was expected that mechanical excavation may have to be carried out together with progressive installation of initial supports, in areas of poor ground, to best suit the extant ground conditions) and to mitigate against potential ground movement and water draw-down. In particular, pre-excavation grouting was carried out at specific locations, including the Fault Zones as identified in the GBR, and at the locations of sensitive ground level installations, such as existing Water Supplies Department facilities. Criteria for the control of groundwater inflow were specified in the Contract Documents.

2.4 Excavation methodology and choice of TBM

The anticipated geotechnical conditions, in conjunction with the course of the route and the tunnel gradient, represented the decisive pre-requisites for the selection of the tunnelling method. By taking into account the tunnel shape and cross-section, its length and the geotechnical conditions, and the available technology, the most suitable tunnelling machine could be procured. When selecting appropriate tunnelling technology, the environmental compatibility of the tunnelling methods must also be taken into consideration. A project-related analysis was however essential and represented the main basis for the approach (ITA-AITES, 2000).

Among the various environmentally-related hazards (natural and anthropic types) and construction equipment related hazards, the principal TWDT construction hazards were identified and distinguished into two main categories - geological hazards and induced geotechnical hazards related to the excavation behaviour of the rock mass. A similar approach was followed in listing the construction equipment-related hazards, and the main mitigation measures were determined in order to contain the construction risk to an acceptable level.

The objective of this evaluation was to ensure an appropriate machine choice, considering the possible hazards involved. A brief example evaluation is presented in Table 1, where a rate value of applicability (o = difficult application, oo = applicable, ooo = ideal field of application) has been assessed for each TBM type considered suitable for this Project.

Table 1: Evaluation of the Influence of Tunnelling Hazards in the TBM-type Selection

Tunnelling Hazards	TBM Type		
	Open	Single Shielded	Double Shielded
Abrasive/Hard rock	ooo	o	oo
Rockfall, rock wedge instability (overbreak)	o	oo	oo
Caving ground	o	oo	oo
Water inrush	ooo	oo	oo

The outcome of the evaluation indicated the adoption of a Double Shield TBM (DSTBM) and such requirement was included in the Contract. In fact, in lieu of an average lower production, double shield TBM has the advantage of the provision of radial grippers, and longitudinal thrust rams pushing off the tunnel lining (depending on the geological condition encountered), and therefore was bettered-suited for the heterogeneous ground conditions present.

The added advantage of the DSTBM is that boring can proceed utilising the grippers, whilst the lining ring is being erected.

The circular profile was chosen to facilitate mechanized tunnelling and to improve the efficiency, productivity and cost-effectiveness of the construction. The adoption of segmental concreted lining improved the hydraulic performance of the tunnel increasing its capacity, with the precast segments augmented by the mechanical efficiency of the TBM facilities

3 THE GBR OUTLINE

3.1 Site investigation and laboratory tests

An extensive ground investigation programme was undertaken for the TWDT, comprising ground investigation field work by drilling and trial pit excavation, with associated field and laboratory testing, and installation of piezometers, for the purpose of assessing the ground conditions (BS 5930:1999). The methodology, scope and layout of the ground investigation works were fully detailed in the GDR. A brief summary of the site investigation and laboratory tests is presented in Table 2.

Table 2: Summary of Boreholes, Site Investigation and Laboratory Testing

Boreholes	Vertical (m)	Inclined (m)	Total
No. boreholes	56	23	79
Borehole length	3505	2356	5861
Rock core obtained	3093	2079	5172
Lugeon Tests (rock mass permeability)			228
Core bulk density			46
Cerchar Abrasivity Index (standard fracture surface)			60
Vp			38
Tensile strength			55
Point Load Test			394
UCS			204
E_{Young}			94

3.2 Fault zones

The GBR detailed anticipated fault zones on the tunnel alignment, giving indications as to their possible extent, and the ground conditions likely to be experienced. The faults detailed in the GBR, included those where very closely-jointed rock mass could be expected, and particularly focused on seams of soil-like material (fault gouge) which could potentially contain water under high pressure and therefore be prone to loosening, ravelling, flowing and collapse, in the absence of effective support and ground treatment.

Fifteen fault zones were identified, along with approx. 430m of tunnel where severe ground conditions could be expected to affect tunnel progress. Particular emphasis was placed on the encroaching of the fault zone at the eastern portal (Wo Yi Hop Intake) where an extensive fault, >120 m was expected, with associated adverse ground conditions, due to fractured, weakened and severe rock decomposition indicated.

3.3 Critical conditions and possible consequences

The following critical ground conditions were highlighted:

(1) Possible fracture zones with increased permeability in saturated conditions; which could imply high pressure water inflows with consequent instability of the tunnel face and/or difficulties in the operations in the tunnel. Occurrence of high pressure water inflow must be considered during excavation, with possible washing-out of the soil/rock matrix and crown instability;

(2) Occurrence of fault zones characterized by gouge bearing fault rocks; in the case of large fault zones, this could lead to difficulties during excavation, and to possible severe caving although “Squeezing” conditions were not considered likely;

(3) Occurrence of high strength rock mass at the decametre – hectometre scale; such could severely hamper the TBM performance or even require by-passing the TBM, to excavate by drill and blast (in case of significant length of the sector to be excavated within).

Based on the GBR, specific requirements were included in the Contract with respect to the TBM type (double shield TBM) and to pre- and post-grouting treatment, in order to address the stabilisation of the tunnel excavation and to mitigate against water draw-down and potential ground movement.

Abrasive and strong rock occurrence

Abrasive rock is a significant factor in tool and cutterhead wear, and to the design of the excavation and in the design of the TBM. In the case of hard rock, the compressive and tensile strength strongly influenced the applicability and productivity of the TBM. For all TBM types, the machine architecture, the installed power at the cutterhead, and the choice and design of the cutting tools and cutterhead, are conditional upon the strength of the ground, particularly for single shielded TBMs, which are sensitive to high strength because of the thrust reaction force through the rams, reacting against the tunnel lining.

The expected wear can be countered by the use of boring or extraction additives, and protection or reinforcement to sensitive TBM parts. In all cases it is essential that cutting tools can be quickly and safely removed, and replaced from within the excavation chamber.

Cutter wear prediction can be made using the "Rock Mass Excavability Index" (RME) (Bieniawski, 2006). As well as the basic purpose of evaluating rock mass excavability, in terms of TBM performance, RME can be correlated to cutter consumption per excavated cubic metre; assumption valid for rock with UCS values of intact rock >45 MPa (Bieniawski, 2009). The results are presented considering three levels of variation of CAI (CERCHAR Abrasivity Index) (i.e. CAI>3, 1.5<CAI>3, and CAI<1.5).

From the GBR Data and analysis, CAI>3 should be chosen in order to evaluate cutter consumption during construction; e.g. for location, Chainage 3150-2380, where 65<RME<80 was expected, the changed cutters/excavated m^3 was expected to vary from 0.005 to 0.001, with such corresponding to 540-1400 changed cutters for this location. Substituting as-built records into the proposed formula, the results show a better performance in the maximum cutter consumption (0.01138), significantly improved from the prdicted (0.0139) which was confirmed by the total average cutter replacement (586) which is close to the predictable lower limit (544) for the same tunnel section (Bieniawski, 2009).

Whilst this exercise indicates an appropriate choice of cutter wear, the shape and dimension was made by the contractor, whilst its reliability confirms the accuracy of the GBR data.

Caving ground and water table drawdown control by ground pre-treatment and probing ahead

The GBR anticipated caving ground and water-table drawndown hazard at fault and shear zones, where ground conditions indicated the possibility of very low self-support. The nature of this hazard strongly influenced the choice of TBM and its characteristics (e.g. precluding the application of an open TBM). In fact, the project's requirements regarding the ground pre-treatment ahead of the excavation face, where fault occurrence was expected, demanded that systematic probe-drilling be carried out, which had a significant effect on TBM progress. Therefore the adoption of a double shield TBM was determined, despite its average lower progress, in lieu of an open TBM, with the added advantage that a it also ensured safer working conditions, and more flexibility in adapting to changing ground conditions.

Probing ahead was a fundamental operation enabling the Contractor, and the Designer, to locate potential critical conditions ahead of the tunnel face. During probe-drilling the main drilling parameters (advance speed, torque and thrust at least) were recorded and correlated with the expected geology and with the TBM performance parameters. Normal practice required the installation of high-performance drilling equipment, positioned within the TBM shield allowing advance drilling around the cutterhead (within 4º degree minimum to the tunnel wall, after Garshol K.F., 1997) and through the cutterhead (see Figure 1).

If probe-drilling did not reveal detrimental conditions, normal TBM procedures were applied. However, if detrimental conditions were detected, appropriate ground treatment (pre-grouting and pre-drainage) was carried out.

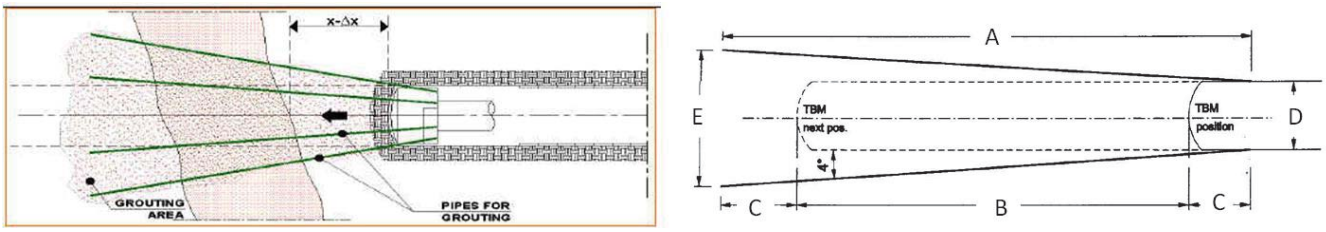


Figure 1: Illustration of grouting/drainage in advance of TBM excavation

Ground treatments typologies and criteria of application in TWDT

The efficiency and effectiveness of the ground treatment was strictly related to the geotechnical characteristics, and to the geometry of the ground to be treated. Therefore, the GBR was essential for proper design of the mix design, the injection pattern and grouting technique.

Ground treatment is normally carried out by pressure grouting, comprising drilled boreholes of suitable diameter, length and direction, aided by packers and grouting pipes, into which pre-prepared grout, at variable pressure, is pumped, with a view to achieving a consolidation of the ground ahead, and improving the long-term self-support of poor ground and rock conditions, and to reduce rock mass permeability. Ground treatment relies on the injection of cement grouts, with stabilizers or admixtures (common cement, microfine cement) as necessary or chemical grouts (resins and gels) and/or combinations of them both. Such grouting is additional to standard tunnel support measures, such as ground reinforcement, fiber glass dowels, self-drilling anchors and steel pipes canopies and ribs.

Due to the limited available work-space, ground treatment from within the tunnel is often a critical activity owing to high cost (e.g. average cost of tunnel face time is worth normally more than over 2,000 USD$/hr). Grouting is time-consuming and therefore whenever possible, advanced ground pre-treatment, from the surface, where such is feasible and cost-effective can be considered.

In-situ permeability tests are normally performed to assess the most suitable mix design, and detailed site investigation is carried out to determine such.. Nevertheless cement-based grouts still remain the most effective, and affordable, and a wide range of microfine cements are now available on the market, if a high penetration rate is required. However chemical grouts may still be required to deal with those joints where cement grout cannot reach. The penetration distance for a given volume, depends on the viscosity and pressure used and therefore the effectiveness of chemical grouts, consisting of only liquid components, is higher because they have ‘viscosity’ but not cohesion, and minimal friction.

The effectiveness of grouting in tunnelling can be enhanced by implementing stages of the treatment, and grouting behind the tunnel face (post-grouting) can be considered a supplement to pre-grouting. However the use of post-grouting only is far less effective and efficient and the total cost to achieve targets can be 2-10 times higher than if associated with pre-grouting (Garshol, 2003).

Permeability tests were used to define the cut-off criteria to apply during grouting (e.g. grouting pressure, volume and grout-setting time) and to define the most suitable equipment and material.

In this project stringent requirements were introduced for the stabilisation of the tunnel excavation and as mitigation against potential ground movement and water drawn down, particularly in the fault zones, where pre-excavation grouting was required for ground stabilisation and groundwater exclusion from the tunnel ahead of the excavation face. Additional criteria were also set for groundwater control into the tunnel, both in the short and the long terms, and to avoid any adverse impact on existing structures and features, with such criteria assessed based on the water inflow measured into the probe hole (ph) and the excavated sections of the tunnel (tu), considering three levels of variation of water inflow (i) lt/min/25m ph>10, (ii) lt/min/25 m tu>10, (iii) lt/min/100 m tu>35.

3.4 Monitoring

In order to record the influence of the tunnelling works, a specific monitoring system is necessary, and such should include procedures for prompt data collection and interpretation, as well as communication, to control in-ground deformation and surface movement, to validate proposed consolidation and support measures and to control the tunnel alignment. Alert, Alarm and Action (AAA) limits should therefore defined for the overall set of monitored parameters, to cover critical scenarios and in order to control residual risks.

Certain monitoring requirements were identified and specified in the Employer's Requirements indicating the minimum zone of influence, and highlighting particular existing structures and features, which had to be included in any monitoring system, and specific AAA limits and vibration values were given. The monitoring included the performance of the TBM to form the tunnel to the required shape and alignment, with tolerance limits specified.

Based on the GBR hydrogeological and piezometric data, specific criteria were introduced to control underground water drawdown in order to avoid any adverse impact on existing structures, resulting from water inflow to the tunnel and specified the action to be taken

A brief summary of the monitoring values and water draw-down criteria, are given in Tables 3 and 4, respectively.

Table 3: Summary of monitoring limits for EBS structures during the execution of the TWDT

EBS Type	Ground Movement Monitoring (mm)			Vibration Monitoring (Prolonged Vibration)	
	Alert	Action	Alarm	PPV (mm/s)	Amplitude (mm)
Buildings	8	12	15	15	0.2
Water retains structures	3	4	5	13	0.1
Existing Tunnels	3	4	5	13	0.1
All other EBS including Buried Utilities	25	40	50	25	0.2
TWDT Tunnel (Segmental lining)	Ovalisation (%)	Deviation (horizontal) (mm)	Deviation (vertical) (mm)	PPV (mm/s)	Amplitude (mm)
	1^	75	75	50	0.6*

Note: ^ BTS (2000); * BS7385 - Maximum displacement allowable = 0.6mm (frequency range lower than 4Hz)

Table 4: Summary of the water drawdown control criteria during the execution of the TWDT

Criteria	Description	Total water discharge (lt/min/m)
1	Discharge greater than, after completion of 25m length probe hole	10
2	General inflow greater than, for the excavated section within25m of the current face	20
3	Inflows of more than, over any 100m length (shorter excavated section to be calculated on pro-rata basis), or a concentrated inflow at any particular location	35 2

Should any of the "criteria" be exceeded pre-excavation and/or post-excavation grouting was required and no further excavation, at the particular developed face, could proceed until the grouting had achieved the criteria.

Regardless of inflows met the first criteria in the proximity of the existing waterworks facilities, pre-grouting was required.

The Project also required the implementation of a Tunnel Data Management System (TDMS), which was web-based, a digital database of real-time construction information including relevant construction information, as well as ground movement and instrumentation records.

4 CONCLUSION

In order to mitigate the risks inherent in the construction of TWDT, a detailed GBR was implemented, and the detailed geological and geotechnical data was considered the primary driver for offsetting such risks from unforeseen or changing ground conditions during the construction phase.

Furthermore, additional necessary precautions and relevant to the Construction Phase were included into the Tender Documentation, including:

- To adopt the proper method of excavation, which serves as a primary countermeasure for limiting instability and collapse.
- To set up a strict control of the secondary countermeasures for limiting the eventual instability.
- To set up an alarm system that is activated when threshold values are exceeded or not met.

- To elaborate the method statements to ensure the correct use of the construction methodology, and such shall also include the actions and the information procedures to face anomalous events.
- To create follow-up team (composed of the Construction Management and Representatives of the Contractor) to verify the systematic interface of the key parameters and the process of design, construction, monitoring, and design modifications.
- To develop project interfaces at the levels of design, monitoring, and analyses. Real time access to the system was be given to all the involved Parties.

ACKNOWLEDGEMENTS

The authors would like to express their sincere thanks to the Drainage Services Department, the Government of the Hong Kong Special Administrative Region, for their kind permission to publish this paper.

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Multi-stages Ground Investigation for the Alignment Selection of TKO-LT Tunnel

J.K.W. Tam, G.C.Y. Nip and B.P.T. Sum
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

Ground investigation (GI) planning for the preliminary design stage of the proposed Tseung Kwan O – Lam Tin Tunnel (TKO-LT Tunnel) has demonstrated the importance and merits of multi-stages GI strategy throughout the planning and selection of the alignment. The implementation of the multi-stages GI programme in the Project has allowed the preferred alignment to be assessed and selected at the end of preliminary design stage by minimizing the risk due to the inherent problematic ground conditions. Then, further GI works for the detailed design on the selected alignment could be carried out at later stage in order to better utilize and focus the GI works for a specific alignment and depth.

A comprehensive GI programme is important to identify potentially problematic ground and groundwater conditions along the proposed development. At different stages of a tunnel project, different GI requirements might be adopted possibly due to the public concerns or changes in alignment and design requirements during project planning or design. The multi-stages GI strategy can allow the ground model to be refined and updated at various stages of the project as new information is obtained. Such reviews can reduce the possibility of misinterpretations and uncertainties of the ground condition. Hence, more confident geological model can be obtained.

1 INTRODUCTION

In March 2009, AECOM Asia Co. Ltd. was appointed by Civil Engineering and Development Department to carry out preliminary design for the TKO-LT Tunnel Project. The main components of the Project comprise an approximately 4.8 km long, dual two-lane highway (of which about 3 km of the highway is in the form of Tunnel) and the Lam Tin Interchange. The proposed developments mainly include tunnel, viaduct, at grade roads, reclamation, the associated building, landscaping and environmental protection works. This paper describes the preliminary and detail design GI works for the TKO-LT Tunnel that were completed or are currently in progress. Apart from the geotechnical consideration, this paper will also touch base on the planning and selection of the alignment in response to public concerns and environmental issues.



Figure 1: Project layout plan showing the Route 6 and the two alignment options

According to the traffic impact assessment of the Tseung Kwan O (TKO) Study, the existing TKO Tunnel would experience serious congestion as a result of the progressive development of TKO New Town. It recommended the provision of a new highway in order to alleviate the anticipated transport need. The proposed TKO-LT Tunnel together with the proposed Trunk Road T2 in Kai Tak Development and Central Kowloon Route will form the Route 6 in the strategic road network (Figure 1).

2 CONSIDERATIONS FOR THE DEVELOPMENT OF TKO-LT TUNNEL ALIGNMENT

The alignment development of the proposed TKO-LT Tunnel took several stages. In short, the alignment was formerly planned as the Western Coast Road that changed to the current tunnel form over the past ten years, mainly due to the public concern, environment impact and potential geohazards. In response to the alignment changes in different stages, relevant design and GI requirements have to be incorporated at different stages. Many factors were taken into consideration before the Recommended Scheme could be proposed upon the comparison of several alignment options. The key consideration factors included the geotechnical conditions, engineering issues, costs, social impacts and environmental impact to the natural landscape.

2.1 Geotechnical issues

It is always favorable that the alignment can avoid, as much as possible, intersecting with adverse ground condition such as faults. However, known or unknown geological features are commonly encountered in underground construction. Therefore, it is important to minimize the ground uncertainty by investigating the identified geological features that would influence the proposed development. GI will be necessary to confirm the presence and extent of the identified features, and also to allow determination of the critical engineering properties of the features that would affect the final decision on alignment and construction method.

According to the available GI, the proposed TKO-LT Tunnel alignment will likely intercept several fault zones, hydrothermally altered rocks and the granite/tuff contact metamorphosed zone. Some of the key geotechnical issues have included the followings.

- High groundwater inflow and poor rock quality may be encountered when tunnelling through the inferred fault influence zone. Relatively more intensive grouting, temporary support and robust contamination protection are likely required. For example the Rennie's Mill fault, it is expected that the adverse tunnelling condition is likely to be encountered within the influence zone of the inferred fault and aplite dyke. Our knowledge from tunneling through these features for the Tunnel C of HATS stage 1 project indicates that it may pose problems for tunnel construction.
- The eastern section of the alignment might encounter the hydrothermal alteration zone near the Chiu Keng Wan Shan. According to GEO Publication No. 1/2007, a zone of hydrothermally altered granite could be traced across several of the MTRC Black Hill tunnels at a depth of about 200 m. The weak material in the zone was encountered unexpectedly and caused a collapse and some delay during construction. The ground investigation also indicated a depression in rockhead in the veins area shown on the geological map and the accompanying memoir indicates that greisenisation is extensive in the area. In conclusion, the hydrothermally altered and mineralized zone will likely result in reduced crystal bonding of the granite, associated reduction in material strength and if encountered, cause adverse ground conditions for tunnelling such as weak, unstable ground and increased ground water inflow.
- Natural Terrain Hazards that may affect the proposed development was also considered. The natural and disturbed terrain above the proposed eastern portal will require site formation works involving cut back, soil nailing, retaining wall construction or other natural terrain mitigation works.

2.2 Environmental consideration

One of the key objectives was to undertake an Environmental Impact Assessment (EIA) for the developed alignment options and to assist CEDD to obtain timely approval of the EIA Report of the Project. Therefore, a separated GI contract and EIA study were carried out at early stage and just before the preliminary design. The keys for obtaining approval of the EIA Report have included incorporating environmental criteria in the evaluation of various alignment alternatives, and to select the most cost-effective and environmentally acceptable and sustainable option. Also, to present the selected alignment option complying with environmental standards and without adverse residual environmental impacts by incorporation of appropriate

environmental mitigation measures in the design and construction methods. Any adverse ecological impact to these ecological resources should be minimized with the consideration of mitigation approaches including maintaining the flow connection between natural stream and marine habitat by engineering design, fish translocation and flora transplantation prior the construction works. Some key environmental considerations in both construction and operation stages included the followings.

- The key terrestrial ecological concern was focused on the potential impact to woodland, grassland/shrubland, natural stream habitats and the associated fauna/flora of conservation interest (e.g. Philippine Neon Goby and Small Persimmon) resulting from the construction and operation of the TKO-LT Tunnel alignment and reclamation at the western Chiu Keng Wan.
- Marine ecological concern for the 35 species of corals (with 3 locally uncommon hard corals species Favia helianthoides, Montipora mollis and Coscinaraea sp.) that were found growing healthily in Chiu Keng Wan in the 2009 survey. So, the select option should have minimal impact on marine ecology.
- Majority of the natural shoreline in Chiu Keng Wan were already removed by reclamation and the western coast would be the only section of natural shoreline remaining. As Chiu Keng Wan faces the open Sea to the southeast direction, the western coast is subject to the strongest wind and wave erosion. Rocky shoreline, intertidal rocky platform and pool, sandy shore and muddy seabed provide the various habitats that nourish rich variety of life. Therefore, preservation of this natural shoreline is important.
- Environmental performance of alignment options should be evaluated in terms of their lengths and routes. The proposed tunnel should be favorable for minimizing the excavated materials during construction and production of vehicle exhausted fume in operational phase. Comparing with a straight alignment, a curved tunnel requests a wider tunnel span as sightline reserve and thus, more rock would be excavated and disposed. The option with short and direct route was considered more desirable.
- Part of the TKO-LT Tunnel alignment is located close to Sai Tso Wan Landfill, environmental risk on landfill gas and leachate hazards should be evaluated among alignment options.

2.3 Public engagement issues

Public engagement for planning and development projects has become a pre-requisite in Hong Kong. It helps to minimize unnecessary conflicts through gathering consensus and understandings among various stakeholders. In the development of TKO-LT Tunnel alignment, alternatives options were presented to the public at early stage to facilitate an open public engagement exercise and explain all the consideration to the public. Moreover, the latest development of alignment options would be delivered to and discussed with the public soonest the possible. Public engagement of this Project gathered feedbacks from the public, including local residents, district councilors, professional bodies and also green groups. The received opinions have facilitated the development of the Recommended Scheme that would benefit the most to the society.

3 PLANNING AND IMPLEMENTATION OF MULTI-STAGES GI

Desk top study was carried out before the proposal of further GI works at the early stage of the Project. The relevant geotechnical information, site investigation data and laboratory testing data of the previous studies were assessed. After reviewing the existing information, it was concluded that the geological information, hydrogeological information and engineering parameters are insufficient for the designs of the proposed TKO-LT Tunnel developments. In particular, most of the archival boreholes did not investigate the proposed tunnel level. Therefore, further GI works were proposed to obtain sufficient data for forming the geological and hydrogeological models of the proposed development area.

Referring to TGN24 (GEO, 2009), site investigation for projects involving tunnel works should be phased. This approach is required because at different stages of a project, different GI requirements might be adopted possibly due to the public concerns or changes in alignment and design requirements during project planning or design. The GI programme of TKO-LT Tunnel has been implemented into 3 stages and a separated GI contract particularly for the EIA study. TGN24 has also recommended that the termination depth of boreholes should be approximately 2.5 times tunnel diameter below the invert level. The study areas of the multi-stages of GI are shown in Figure 2 and the aims of each GI stage are summarized as below.

- Stage 1 GI was completed in two-month time in mid-2009. It provided ground information on the critical issues that potentially affect the optimum project alignment location and associated key project interfaces

especially at the tunnel portal areas around the Cha Kwo Ling and Chiu Keng Wan Shan (at the proposed eastern portal area of S-curve alignment). Also, the Marine Archeology Investigation (MAI) of the proposed reclamation area at Chiu Keng Wan was carried out.

- The GI contract of Marine Environmental Investigation was carried out from Oct. 09 to Mar. 10, before Stage 2 GI. The result of the marine GI works was used for the environmental assessment of the reclamation works at Chiu Keng Wan.
- Stage 2 GI was carried out from Mar. 10 to Mar. 11 and comprised the investigations mainly proposed for the alignment options development for preliminary design and land environmental GI works for Sai Tso Wan Landfill. These GI works were proposed for the engineering and land environmental assessments of the overall proposed TKO-LT Tunnel developments. This GI implementation can allow the preferred alignment to be assessed and confirmed at the earliest stage possible.
- Stage 3 GI started in late 2011 and the works are still in progress at the time of writing. It comprised the investigations for detailed design, horizontal directional coring, conventional land and marine GI. The main purpose of the works is for the detailed design of the straight alignment option.

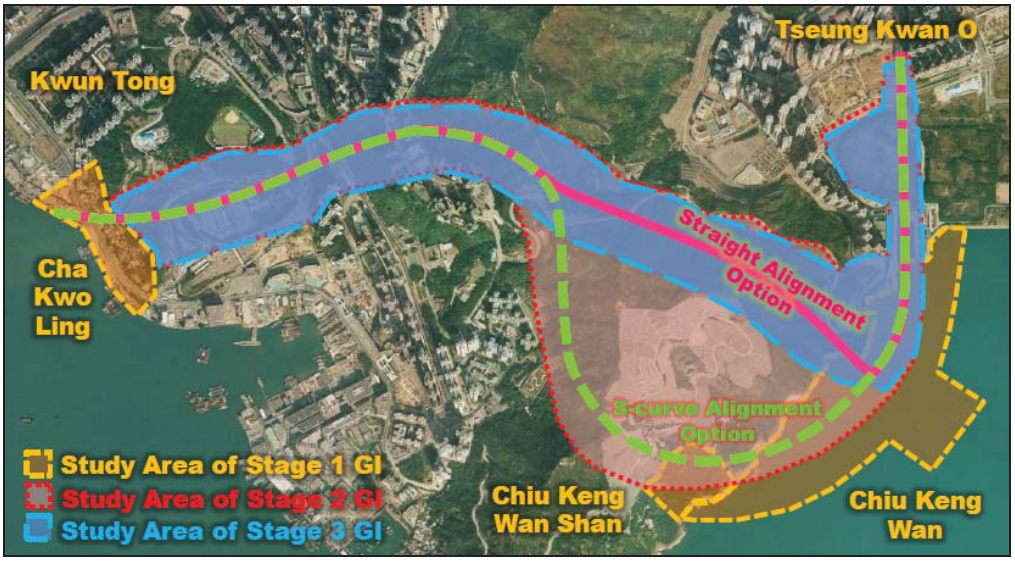


Figure 2: Study area of Multi-Stages GI for TKO-LT Tunnel

4 MULTI-STAGES GI FOR ENGINEERING AND ENVIRONMENTAL STUDY

The project specific land and marine GI works are aimed to obtain adequate information for engineering and environmental study, preliminary design and detailed design of the Project. The investigation information is being used to establish geological and hydrogeological models of the proposed development.

4.1 Stage 1 GI

Stage 1 GI was scheduled at the early stage of public consultation before developing the schematic alignment options. The GI works comprised 20 land boreholes at Cha Kwo Ling Area and 6 land boreholes at TKO eastern portal of S-curve alignment near Chiu Keng Wan. These boreholes mainly investigated the ground conditions below the Cha Kwo Ling Area, particularly targeting the relatively deep weathering zone inferred near the coastline and to assess the feasibility of rock tunnel alignment through the area. Moreover, the geology of the TKO eastern portal that is located in the vicinity of the greisenised granite with mineral veins was investigated in this prior stage. The Marine Archeology Investigation (MAI) of the proposed reclamation area at Chiu Keng Wan was also conducted. This MAI investigation comprised three types of geophysical surveys, including the echo sounding survey, seismic profiling survey and multi-beam survey.

The findings of Stage 1 GI are important for the selections of alignment options and the geotechnical assessments on the two interface portal constructions at Cha Kwo Ling Area and Chiu Keng Wan. The results of these geotechnical assessments were presented in public engagement activities for early stage discussion

and evaluation. Moreover, the findings of the MAI were used for the feasibility assessment of the reclamation options and methods.

4.2 Marine environmental investigation for EIA

It is anticipated that the development of TKO-LT Tunnel will involve reclamation and dredging of marine sediment will likely be required. Based on the preliminary review of the sediment quality around the proposed reclamation area, 26 nos. of sediment sampling locations using vibrocore were carried out offshore of southeastern Chiu Ken Wan.

4.3 Stage 2 GI

Based on the available ground information and addressing the public concerns, two schematic alignment options, namely the S-curve alignment and straight alignment (Figure. 1), have been developed. Therefore, a comprehensive GI for the preliminary design stage (Stage 2 GI) was conducted, which aimed to collect data for the geotechnical and environmental assessments of the area covering the two alignment options.

The Stage 2 GI works comprised 64 boreholes and 13 trial pits for engineering study and 17 boreholes for environmental study. These GI stations located at the proposed Lam Tin Interchange, along Lei Yue Mun Road, over Chiu Keng Wan Shan and Black Hill, and at TKO Town Centre South. Boulder survey in nine selected natural terrain areas along the proposed tunnel was carried out. Stage 2 GI works and purposes are summarized in Table 1. The works targeted to obtain sufficient geological information of alignment options to facilitate technical assessment and selection of the Recommended Scheme. The locations of potential geohazards such as faults and hydrothermal alteration zone were estimated. In addition, environmental boreholes and sampling were carried out to establish the environmental baseline condition for EIA.

Table 1: Summary of Stage 2 GI

Study Area	GI Works	Purposes of Investigation
Proposed Lam Tin Interchange	Land Boreholes	The ground condition and engineering parameters at site formation level.
	Environmental Boreholes	To assess the potential leachate from Sai Tso Wan Landfill and to supplement the landfill gas hazard assessment, waste management implications, and water quality impact assessment of the EIA Study
Chiu Keng Wan Shan	Land Boreholes	The ground condition and engineering parameters within the envelope of alignment options for tunnelling.
	Boulder Survey	Natural Terrain Hazards Study, to assess the boulders condition in natural terrain and the risk of boulder fall induced by tunnel blasting.
	Trial Pits	Natural Terrain Hazards Study, to assess the compatibility of soil in natural terrains and the risk of landslide or debris flow.
Chiu Keng Wan	Marine Boreholes	To collect samples of Marine Deposit for chemical and biological testing for EIA study.
TKO Town Centre South	Land Boreholes	The ground condition and engineering parameters along the proposed alignment for road works.

4.4 Stage 3 GI

The straight alignment option was considered as the Recommended Scheme after series of public engagement activities, discussions with various Government Departments and consideration of environmental issues. Therefore, the Stage 3 GI was proposed to investigate the straight alignment. These GI works comprised the investigation for detailed design, horizontal directional coring conventional land and marine GI. The GI works are in progress at the time of writing.

After reviewing the available GI information (including Stage 1 & 2 GI data), it was inferred that there are some local geological features that potentially pose a risk in terms of cost or delay to the proposed development, and this uncertainty should be reduced by specific additional GI. Also in general, it was considered that there are still some gaps in the geological information, hydrogeological information and

engineering parameters which need to be addressed for the detailed designs of the proposed TKO-LT Tunnel developments. Therefore, Stage 3 GI was proposed to obtain sufficient data for forming the geological and hydrogeological models of the straight alignment option. It was also aiming to obtain engineering parameters for the design of foundation of tunnel associated facilities, slope modification, tunnel lining, blasting works, reclamation, piling of via-duct. There are a total of 15 trial pits, 73 land boreholes, 27 marine boreholes, 5 Continuous Piezocone Penetration Tests, 3 slope stripping, 1 HDC, rock joint mapping and boulder survey proposed for this Detailed Design Stage GI programme. Stage 3 GI works and purposes are summarized in Table 2.

Table 2: Summary of Stage 3 GI

Study Area	GI Works	Purposes of Investigation
Proposed Lam Tin Interchange	Land Boreholes, Trial Pits and Slope Stripping	Detailed design of the proposed Lam Tin Interchange, including tunnel, open cut section, surface works slope features modifications and foundation works of proposed tunnel facilities.
	Geological and Rock Joint Mapping	To collect discontinuity data and carry out stability assessment of existing rock slopes for modification works.
Chiu Keng Wan Shan	Horizontal Directional Coring	To obtain continuous rock sample and conduct water inflow test along the tunnel horizon. Targeting to investigate the characteristic of the identified hydrothermal alteration zone.
	Land Boreholes	Reduce uncertainty of potential adverse geological features inferred from existing SI and detailed design of main tunnel and slip roads of the straight alignment option.
	Boulder Survey	Natural Terrain Hazards Assessments for natural slopes that may affect the proposed TKO Eastern Portal. To assess the boulders condition in natural terrain and the risk of boulder fall induced by tunnel blasting.
Chiu Keng Wan	Marine Boreholes	Detailed design of the proposed Tseung Kwan O Interchange and reclamation area. To determine the rockhead level in the area for piling design of structures
TKO Town Centre South	Land Boreholes and Trial Pits	Connection roads, depressed road, footbridge and surface works at the south of TKO town centre.

5 CONCLUSIONS

The multi-stages GI strategy can allow the ground model to be refined and updated at different stages of the project as new information is obtained. The GI works at different stages are planned according to the latest alignment development with respect to the public concerns collected in public engagement activities and most updated ground model of latest technical assessments. Such reviews can reduce the possibility of misinterpretations and uncertainties of the ground condition. Hence, more confident geological model can be obtained. Having comprehensive and reliable geological information is essential for developing a realistic programme. As a result, time, cost and resource allocation can be effectively assigned. The TKO-LT Tunnel project has demonstrated the importance and merits of multi-stages GI strategy throughout the planning and selection of the alignment.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Director of Civil Engineering and Development Department for permission to publish this paper.

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Horizontal Directional Coring (HDC) and Groundwater Inflow Testing for Deep Subsea Tunnels

B. Cunningham, J.K.W. Tam & J.W. Tattersall
AECOM Asia Co. Ltd., Hong Kong
R.K.F. Seit
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

A comprehensive ground investigation (GI) plan is important to identify problematic ground and the groundwater conditions along a proposed tunnel alignment. Continuous geological and engineering information is difficult to get on land, but even more so in the marine environment. However, Horizontal Directional Coring (HDC) can provide continuous core along the tunnel alignment and enable groundwater inflow testing over long lengths parallel to the tunnel axis. This enables the risk of unforeseen tunnelling conditions to be reduced when compared to using only isolated vertical and inclined drillholes.

This paper highlights the benefits of the use of HDC and groundwater inflow testing during the ground investigations for the deep tunnels of the Harbour Area Treatment Scheme (HATS) Stage 2A in Hong Kong. HDC holes were carried out along the tunnel alignments to reduce geological and hydrogeological uncertainty where major faulting was suspected and along the subsea tunnel from Hong Kong to Stonecutter's Island. The HDCs were continuously cored and located just above the tunnel crown for distances up to 1200 m. Inflow tests over 50-100 m lengths were carried out to supplement relatively isolated packer test data to provide additional insight into variations in potential rates of inflow at the tunnel scale.

1 INTRODUCTION

For tunnelling at great depth beneath a submarine environment, it is important that the rock mass quality and groundwater inflow prognosis is as close to the actual as possible. This is for the benefit of the development of construction programme and cost estimation. Six numbers of HDC were drilled for the deep subsea sewage tunnel (up to 160 m below ground) of HATS Stage 2A. The HDCs were carried out along the proposed tunnel alignment primarily where major faulting was suspected and in areas with difficult marine access for conventional drilling. The primary aim was to reduce geological and hydrogeological uncertainty to a level that would not be achievable with a conventional investigation programme using isolated vertical or inclined drillholes.

1.1 What is HDC?

HDC is a ground investigation technique developed in Norway in the 80's, and the wireline version was subsequently launched in 2001. The key specialist service provider is a Norwegian registered company that has more than 20 years of worldwide experience in directional coring.

The directional coring method has been used in petroleum and mineral explorations, as well as tunnel projects. One of the typical uses of directional coring is "Side-tracking" drilling for investigating the extent of the target ore body. The concept is to create multiple branches of drillholes extending out from a single primary hole drilled from one position. Directional coring is also commonly used in "Steerable" drilling along a planned trajectory, such as the HDC along a proposed tunnel alignment.

1.2 The Reasons for using HDC for tunnel project

HDC can provide a continuous core sample and more reliably identify the extent of problematic rock along the tunnel alignment. Hence, the risk of unforeseen tunnelling conditions can be minimized compared to using only conventional vertical and inclined drillholes (Figures 1 and 2). The HDC launching point can be positioned on land for core sampling seawards and under water.

Continuous coring and field testing in a drillhole steered parallel to the tunnel axis can provide good data for geotechnical assessments. Water inflow tests can be carried out over long, continuous segments to facilitate more realistic prognoses of potential groundwater inflows than can be obtained from conventional packer testing in isolated holes over short segments which are not aligned parallel to the tunnel axis. However, supplementary packer testing within the longer inflow test lengths also helps to gain a better appreciation of the spatial variability dictated by the distribution and condition of the water-bearing jointing systems. The information obtained provides a much better basis for tendering than in sections of the alignment where only isolated, vertical or inclined drillholes have been carried out. In consequence, the geological and hydrogeological assessments can be greatly enhanced and unforeseen geological and construction risks can be reduced.

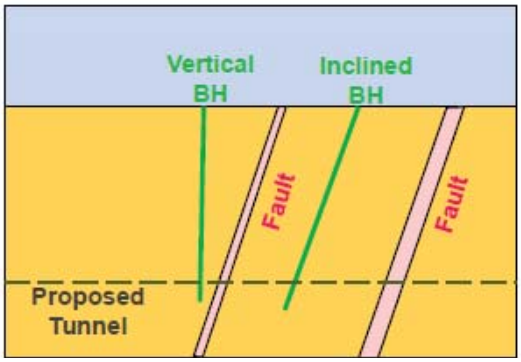


Figure 1: Fault cannot be encountered by the vertical or inclined drillholes.

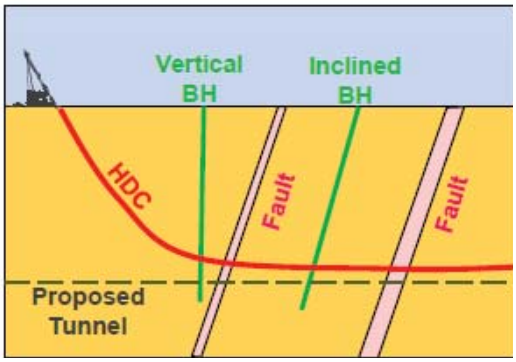


Figure 2: HDC encountered the faults and can estimate their extent.

2 HDC IN HATS STAGE 2A

The HATS Stage 2A project in Hong Kong includes the construction of a deep sewage conveyance system (SCS) with 13 vertical shafts. Approximately 20 km of tunnels will be driven at depths ranging from 70 m to 160 m below sea level. The main GI contracts for HATS Stage 2A were spread over a period of two years before the construction of SCS tunnels and shafts commenced in July 2009.

During the detailed design stage, six HDCs were drilled along the proposed tunnel alignment where major faulting had been previously inferred in the geological model (Figure 3). The HDCs provided continuous core (reaching 160 m below sea level) with the longest drillhole extending 1250 m into Victoria Harbour (HD01). Another ‘first’ was the use of groundwater inflow tests carried out in 50 m to 120 m long segments to supplement conventional packer test data to give greater insight into the transmissivity of the rock mass at the tunnel horizon. The characteristics of the key geological features encountered in the HDCs have been summarized in Table 1. Estimates of groundwater inflow rates in the tunnels are for the hypothetical condition assuming no pre-grouting is carried out.

Table 1: Characteristics of the key geological features encountered in the HDCs

HDC Nos.	Rock Core Characteristics
HD01	Grade III/II, chloritized granite with very few and minor zones of shearing, quartz veining and Grade IV/III. The results of the inflow tests conducted in HD01 are abnormally high (up to an equivalent Lugeon value (Lu) of 66 over a length of about 100 m), yet very few signs of faulting are present in the rock core. Part of the core trajectory spans the zone where the 'Sulphur Channel fault' extrapolated from the published geological map might have been expected.
HD02	Grade III/II granite with feldsparphyric rhyolite and some zones of fault gouge and increased weathering. The inflow results indicate equivalent Lugeon values which gradually increase from about 0.2 Lu in the middle part of the HDC to about 6 Lu at the most westerly fault belonging to the Sandy Bay fault zone. These results are equivalent to untreated inflows between 30 and 900 litres/minute/100 m at the depth of the tunnels.
HD03	Grade III/II granite, with locally grade V/IV, chloritized granite, basalt dykes. Locally intensely fractured. Good conditions confirmed within the Causeway Bay-Kellet Island palaeoridge, with poorer conditions on either side due to proximity of the Wan Chai Gap and Tai Tam faults. Inflow tests through the palaeoridge indicated relatively low equivalent Lugeon values of between 0.02 and 0.21 Lu.
HD03a	Grade III/II, highly altered and chloritized granite with many shear zones and micro-fracturing. A mafic dyke associated with the Tai Tam Fault with an apparent thickness of 120 m was also encountered. HDC03A provided a continuous record of the rock mass quality and a record of nearly continuous groundwater inflow tests through the Tai Tam fault zone. The groundwater inflow test results for lengths of about 100 m indicated equivalent Lugeon values of about 30 Lu, steadily diminishing westwards to about 5 Lu beyond the fault-affected rock mass. These results are equivalent to between 4,500 and 750 litres/minute/100 m at the depth of the tunnels.
HD04	Grade III/II granite, with 11 No. weakness zones up to 7.5 m thick comprising Grade IV-V altered, chloritised granite.
HD05	Grade III/II, metamorphosed and greisenized Tuff with multiple fault and shear zones, and local pegmatite veins, brecciation, basalt dykes and calcite veins. 18 No. zones of 'no core recovery' up to 1 m thick associated with the Telegraph Bay Fault.

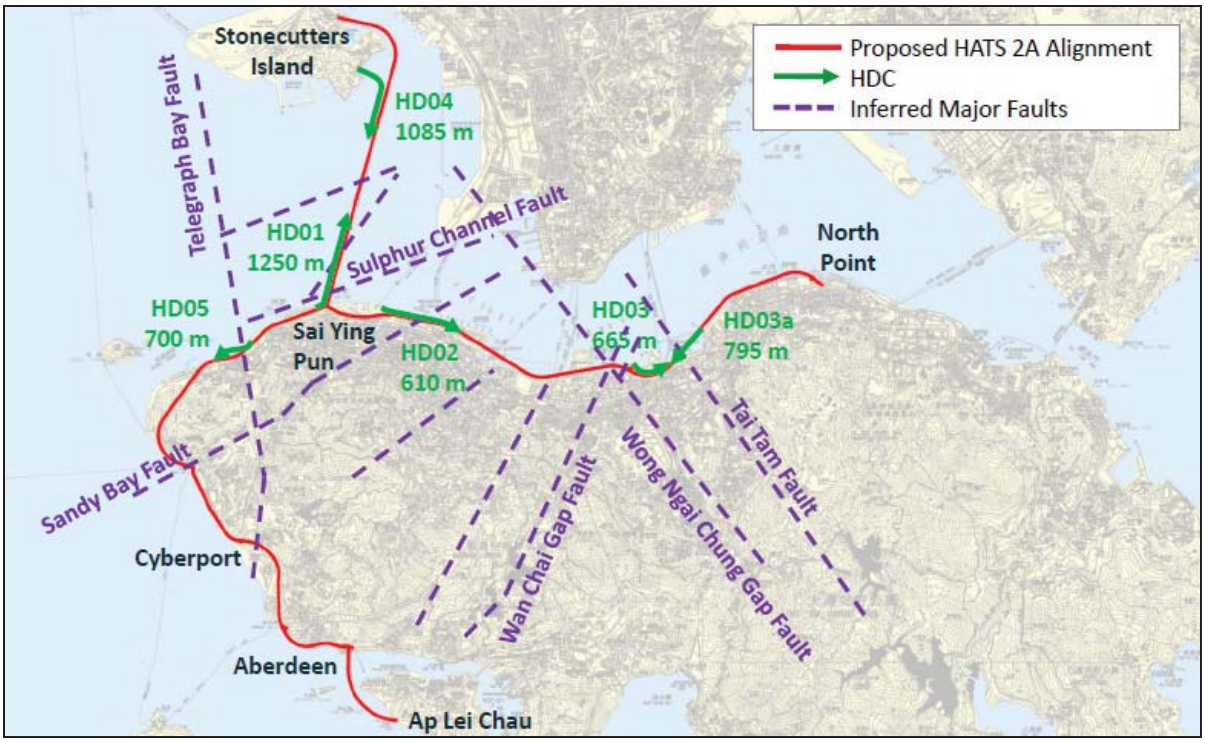


Figure 3: Layout plan showing the proposed HATS 2A tunnel, the completed HDC, and the inferred major geological features

3 FUNDAMENTAL PRINCIPLES OF DIRECTION CORING

The working principles of directional coring include three key components: planning, steerable drilling and coring orientation surveying as summarized below:

- Windows software package that has been developed by the drilling specialist is used for the planning and plotting of the drillhole trajectory. The designer should provide the coring trajectory with control points (i.e. coordinates and elevation of the proposed coring) and tolerance envelope of drilling. Then the drilling specialist will plan the drilling route with preset bending and roll angles.
- Steerable drilling is carried out using a steerable core barrel with wireline operating system. The drilling trajectory is navigated by the "toolface angle" (i.e. roll angle) that controls the drilling direction and the "dogleg angle" (i.e. bending angle) that controls the curvature of the trajectory. The straight section of the coring will be drilled by conventional wireline system, and the deviated section will be drilling by the steerable wireline system. Figure 4 illustrates the key components of the steerable core barrel.
- Core surveying is carried out using a miniature electronic multishot (EMS) instrument with timing interval specified by the drilling specialist. The EMS records the azimuth and inclination of drillhole for the specific point at different depths. The as-built drillhole trajectory will be compared with the proposed trajectory after each coring survey, in order to ensure the coring is advancing within the tolerance envelope of the proposed trajectory.

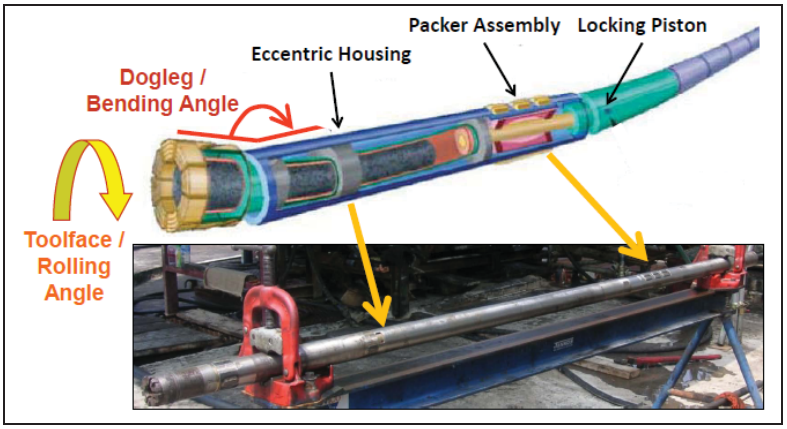


Figure 4: Illustration diagram of the directional core barrel

4 WATER INFLOW TEST IN HATS STAGE 2A

In some areas where major faulting was suspected, HDCs were carried out with the primary aim of reducing the level of geological and hydrogeological uncertainty to a degree which would be difficult to attain with a conventional programme of relatively isolated, vertical and inclined drillholes. The provision of a continuously sampled cored hole just above the tunnel crown for distances up to 1200 m can provide good data for tendering purposes and insight into potential rates of groundwater inflow in the tunnel if pre-grouting is not carried out.

The inflow test measures flow into a bore, simulating flow into a tunnel whereas a packer test normally measures flow out of a bore under a higher pressure than the ambient conditions. There are concerns about different hydraulic response in that pressurized flow out of a bore could open joints whereas flow into a bore could close them. Also the longer test length in an HDC orientated parallel to the tunnel axis is more analogous to the section of tunnel under consideration and is less subject to spatial variations in the jointing systems than in the case of Lugeon tests carried out in isolated drillholes with a different orientation to the tunnel. In addition to measuring rates of inflow into HDCs, the opportunity was taken to conduct packer tests at selected locations within the test lengths to gain further insight.

4.1 Testing method of water inflow test in HDC

Groundwater inflow tests were carried out using the "Pump Down Packer System (PDPS)" and "Shut-Off Packer System (SOP)" to measure the sectional inflow continuously along the proposed tunnel alignment. The PDPS comprises the double packer, memory gauges and filter. The SOP consists of flexible hose with a single packer, a down hole sensor, a pump and shut-in valve. The typical length of testing section was 100 m and ranged from 50 m to 120 m. Figure 5 shows the general setup of the water inflow test in the HDC.

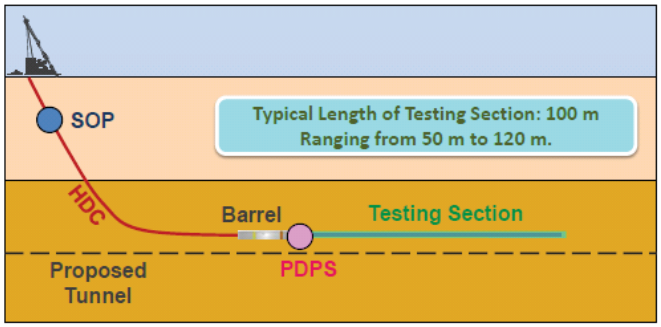


Figure 5: General setup of the water inflow test in HDC

Groundwater inflow measurements under atmospheric condition give the natural inflow rate into the cored hole by pumping out water and measuring the resulting range of drop in pressure and also the rate at which the system regains equilibrium. The testing method more closely simulates the effect of tunnel construction than packer tests which rely on forcing water into the surrounding rock mass which can induce dilation of the rock joints. The inflow measurements can be used to gain better insight into the hydrogeological regime and the potential effects of tunnel construction.

After the new testing section of cored hole is drilled using the wireline coring system, the drill string is pulled to form the testing interval between the drill bit and end of cored hole. The testing procedures can be summarized as follows:

- PDPS is pumped down through the core tube extensions to the coring barrel. The external packer passes through the coring barrel and is placed in the testing section. The internal packer is placed inside the core barrel (Figure 6).
- Packer inflation with increasing pressure until opening of last valve. The static formation pressure of the test interval can be estimated from the memory gauges shortly before opening of the last valve.
- Installation of SOP-system.
- Pumping of water with the pump integrated in the SOP-system, thus lower pressure in the test interval (constant head withdrawal test).
- Stop the pump and monitor pressure recovery.
- Retrieve SOP-System.
- Deflate the packers of the PDPS by pulling on the core tube.
- Pump overshot tool using the wireline system to retrieve the PDPS.

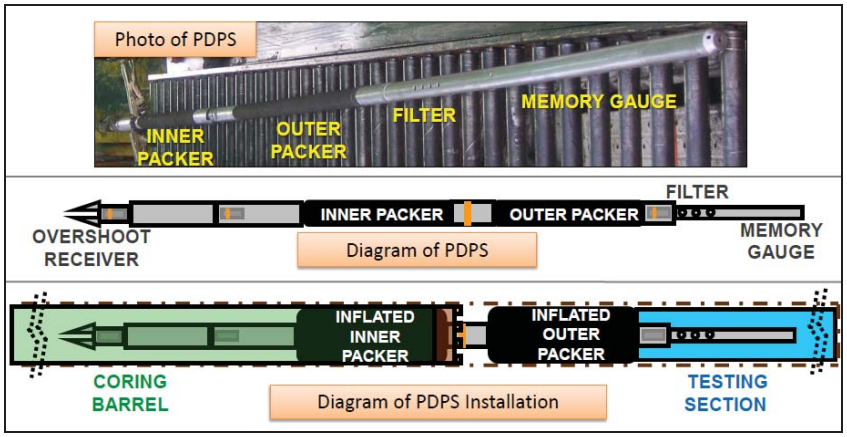


Figure 6: Photo of PDPS (top), diagrams of PDPS (middle) and PDPS installation (bottom)

4.2 Data interpretation of water inflow test

Groundwater flow in igneous rocks is via discontinuities which vary in location, intensity, orientation, conductivity and inter-connectivity. Such ground is not uniformly permeable (as in the case of clean sand) and it is therefore highly problematic to estimate rates of inflow in rock tunnels on the basis of tests conducted over short lengths in drillholes. Inflow tests over relatively long lengths help to minimize the potential difficulties but they do not completely remove them. They also give little indication of variability within the test length which can be important for prognoses of grouting requirements. In order to gain insight on both mass permeability and local variability, Lugeon tests were carried out on short sections within the longer segments subjected to inflow tests. In this case, it is convenient to express the inflow test results in the same units as the traditional Lugeon test to facilitate comparisons between the two types of test.

The measured rates of inflow in HDC were converted to Lugeon units by dividing the flow rates by the test lengths to obtain rates of flow per metre (Equation 1 & 2) and by multiplication to scale the results to a pressure of 10 bars (Equation 3). The results of the inflow tests ranged from 0.02 Lu to 92 Lu. Packer tests were also carried out within the inflow test lengths and the Lugeon values were plotted by location, for direct comparison with the inflow test data. For HDC03 the test results are scattered indicating how the variability of inflow is dictated by the conductivity of individual fissures. With packer tests giving Lugeon values well above and well below the inflow test value, it is apparent that selection of a packer test result to represent inflows for lengths in a bore of the order of 100m long can be wide open. By contrast, for HDC01, HDC02 and HDC03A, the inflow results plot in a range which is an order of magnitude higher than the packer test results conducted within the same sections of drillhole but over much shorter lengths. In these cases all of the packer tests would seriously underestimate the flow into the longer bores.

Conversion of inflow rate to lugeon units by:

$$Qa = Q \times \left(\frac{P}{Pd} \right) \tag{1}$$

Where Qa = Inflow rate at atmospheric pressure (litre/min/section), Q = Measured inflow rate per section (L/min/section), P = Measured static formation pressure (kPa) and Pd = Pressure difference (kPa).

$$Qm = \left(\frac{Qa}{L} \right) \tag{2}$$

where Qm = Rate of flow per metre (litre/min/m) and L = Length of testing section (m).

$$Lu = Qm \times \left(\frac{10}{P} \right) \quad (3)$$

where Lu = Lugeon value (l/min/m at 10 bar).

5 CONCLUSIONS

The advantages of HDC for tunnel works include:

- Areas of concern can be investigated from a remote coring entry point where access from directly above the alignment is severely restricted by buildings, infrastructure or the marine environment.
- More realistic groundwater inflow measurements can be carried out to help define the hydrogeological regime, potential groundwater inflows and provide a better basis for estimating grouting requirements. A suite of inflow tests over long lengths and Lugeon tests on shorter segments within the inflow test lengths help to gain insight on the effects of spatial variability within the jointed rock mass.
- Continuous core sampling along the tunnel alignment can greatly reduce the risk of unforeseen ground conditions when compared to a conventional investigation of isolated vertical or inclined drillholes where the conditions between each drillhole need to be inferred. In general, it can help to reduce the often high geotechnical risk associated with tunnelling.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Director of Drainage Services Department, the Government of the Hong Kong Special Administrative Region for permission to publish this paper.

Hydrogeological Assessment for Tunnels in the Harbour Area Treatment Scheme Stage 2A Sewage Conveyance System

L.J. Endicott & A.K.L. Ng
AECOM Asia Co. Ltd., Hong Kong
H.K.M. Chau
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

The Harbour Area Treatment Scheme Stage 2A (HATS 2A) Sewage Conveyance System, commissioned by the Drainage Services Department, is one of the major steps in improving the water quality of Victoria Habour. The system is to collect and convey the pre-treated sewage from existing treatment facilities located along the northern and south-western shoreline of Hong Kong Island, to the Stonecutter Island Sewage Treatment Works for treatment before final disposal into the western harbour.

The over 20 km long tunnel will be mostly constructed using drill and blast method and the rock face exposed within the tunnel during construction will allow water ingress. The subsequent reduction in water pressure around the tunnel will inevitably lead to ground settlement. Given the tunnel can be as deep as 160 m, the zone of influence can be extensive, covering highly developed areas in Hong Kong Island. A hydrogeological assessment was conducted to estimate the impact to the groundwater regime due to the tunnelling and allow prediction on the associated ground settlement to be made. This paper presents the methodology and some typical results of the assessment and the challenges it had faced.

1 INTRODUCTION

1.1 The project

The HATS Stage 2A Sewage Conveyance System is required to collect and convey the pre-treated sewage from eight existing Preliminary Treatment Works located along the northern and south-western shoreline of Hong Kong Island, to the Stonecutters Island Sewage Treatment Works for treatment before final disposal into the western harbour via an existing submarine outfall. The sewage conveyance system comprises the following elements:

- Construction of a deep sewage conveyance system to collect and convey the sewage from the northern and south-western areas of Hong Kong Island to Stonecutters Island Sewage Treatment Works (SCISTW) for centralized treatment;
- Upgrading of the eight existing preliminary treatment works at North Point, Wan Chai East, Central, Sandy Bay, Cyber Port, Aberdeen, Wan Fu and Ap Lei Chau;
  Construction of a main pumping station at SCISTW to extract the flows from the Stage 2A Sewage Conveyance System and expansion of the existing SCISTW to provide centralized CEPT and disinfection for all the sewage collected from the entire HATS catchment; and
- Provision of disinfection facilities to the HATS Stage 1 effluent before discharging into the harbour as the advanced works under the HATS Stage 2A.

The tunnel to collect and convey the sewage will be more than 20 km long, which will be in rock mostly by drill and blast method. Figure 1 shows the horizontal alignment of the tunnel which is divided into segments; Tunnels J, K, L, M, N, P and Q.

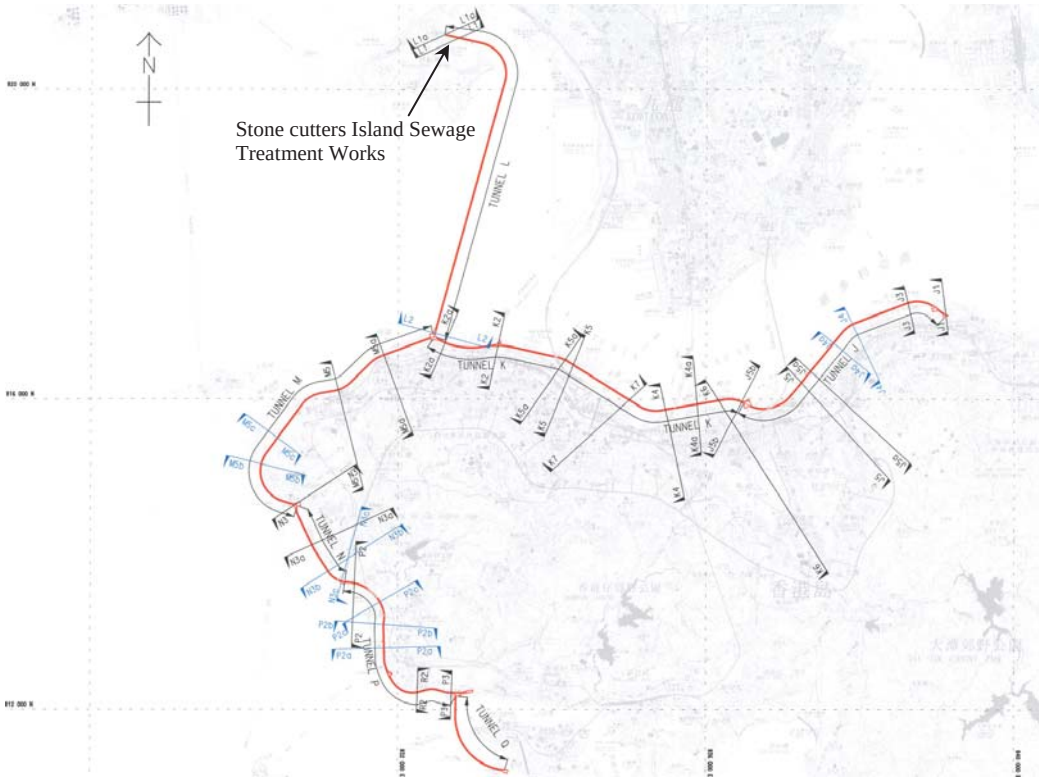


Figure 1: Tunnel layout plan

1.2 Solid and superficial geology

The depth of the tunnel varies considerably, with Tunnel Q at Ap Lei Chau having the shallowest depth at about -80 mPD and Tunnel J between North Point to Causeway Bay the greatest depth at -160 mPD. The tunnels will mainly go through rock in three geological units; Kowloon granite, volcanic coarse ash crystal tuff of Mount Davis Formation and volcanic fine ash vitric tuff of Ap Lei Chau Formation. The rock cover to the tunnels varies between 30 m and 130 m.

The superficial geology along and near the tunnel alignments comprises saprolite derived from in situ weathering, colluviums sporadically developed in sheets and depression on the hillsides above and sea level, alluvial deposits in the near-shore and offshore areas, marine deposits and reclamation fills of different ages.

1.3 Ground settlement during tunnelling

When tunnelling in rock, ground settlement directly due to the stress release on the tunnel face is expected to be negligible. However, the rock face will be temporarily exposed after drill and blast not until the permanent lining of the sewage tunnel is in place. During the time, there will be water inflows into the tunnel. The subsequent reduction in water pressure around and above the tunnel will inevitably lead to ground settlement. The area that will be affected can also be extensive due to the significant depth of the tunnel. Therefore it is important that the hydrogeological conditions during construction are assessed and the associated ground settlement predicted. Measures such as pre-grouting around the tunnel will be taken as the first measure to control the ground settlement.

2 METHODOLOGY OF HYDROGEOLOGICAL ASSESSMENT

2.1 Overall approach of hydrogeological assessment

With the tunnel of over 20 km length, going through regions of different solid and superficial geology, there is a risk that highly localized effect along the tunnel alignment is not studied in full. It is also an engineering challenge to model the problem which is highly complex, partly because of the transient response of the

groundwater regime and the time-dependent consolidation behaviour of the soft deposits. With these factors in mind, it is considered more practical to adopt an Observational Approach where the prediction from the assessment will be compared constantly against those monitored on site during construction, allowing adjustment in measures to be taken to account for localized and unexpected change in geology, soil and rock properties and construction issues. It means that a carefully designed monitoring programme and specification would have to be in place in order to monitor and trigger the necessary actions at the right time.

With the Observational Approach, the analyses in the hydrogeological assessment are conducted for the key locations, including the highly developed areas and areas with rapid change in geological setting. 30 cross-sections along the tunnel alignment are assessed and 20 of them are selected for seepage analysis (see Figure 1 for locations). The hydorgeological assessment is conducted with conservative assumptions and global parameters, which are determined with reasonable conservatism built-in. The computer program SEEP/W is utilized. Figure 2 shows a typical example of the SEEP/W numerical model and the groundwater pressures that the model generates. Prior to the analyses for the cross-sections, pilot basic models involving a tunnel in a homogeneous rock mass were also set up and tested to study the boundary effect, recharging, the sensitivity of the rock mass permeability and effect of grouting around the tunnel.

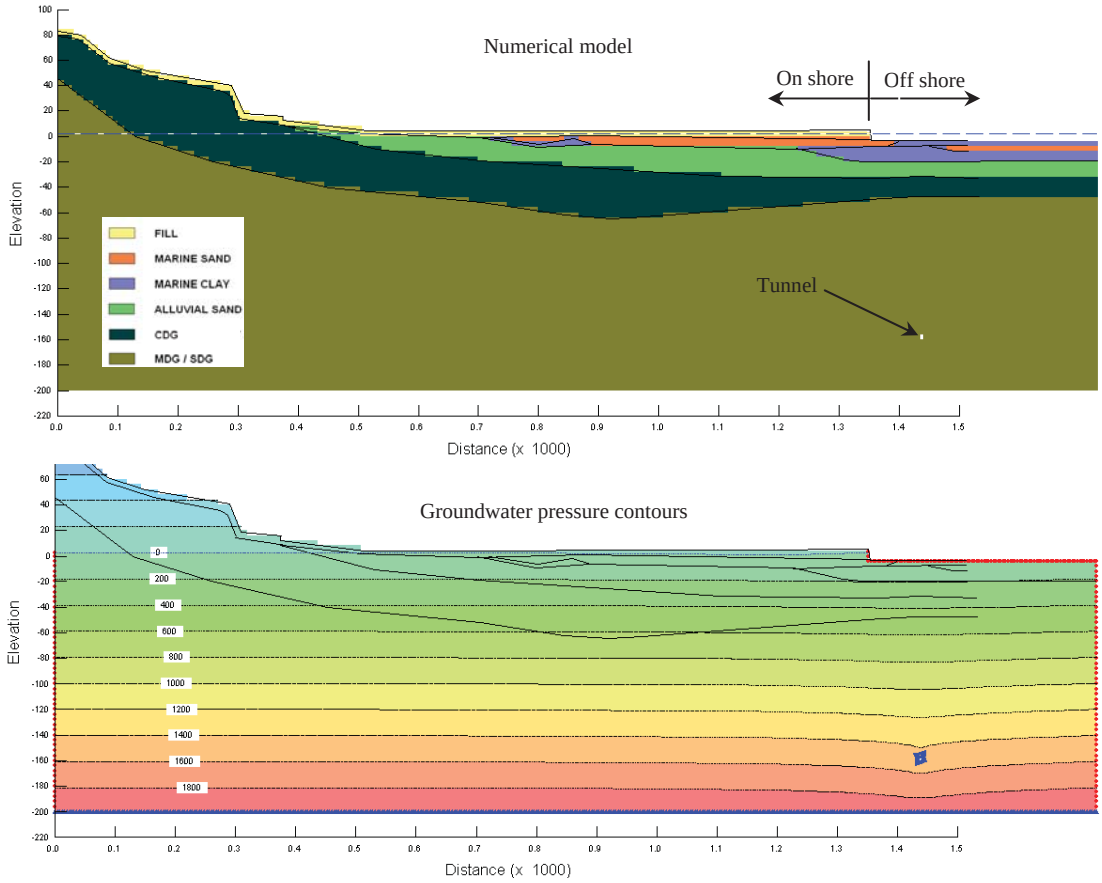


Figure 2: Typical SEEP/W model (cross-section J5)

2.2 Permeability of rock

One of the key parameters in the assessment is the permeability of the rock mass. Rock material is in effect impermeable, flow takes place on conductive fissures and faults. The flow also depends the connectivity of the fissures to potential sources of recharge. A rock mass permeability can be adopted to approximate to the overall flow through many conductive fissures throughout a mass of rock provided that the mass of rock is sufficiently large in relation to the spacing of the fissures. Considering the magnitude of inflow in zones of different rock type conditions and the results of over 600 packer tests expressed as Lugeon values, it was estimated that the rock mass permeability was in the range $1x10^{-8}$ to $1x10^{-7}$ m/s.

In the hydrogeological assessment, a rock mass permeability of $1x10^{-7}$ m/s is adopted. The subsequent parametric study shows that the predicted ground settlement becomes less sensitive to the rock mass permeability when there is a grouted annulus around the tunnel, which is of much lower permeability.

2.3 Ground settlement prediction

During tunnelling, there will be no significant changes to the total vertical stress in the soil and rock mass. It is assumed that the increase in the vertical effective stresses in soil and rock mass will be equal to the reduction in the groundwater pressures. With this assumption, the reduction in the groundwater pressures within different soil and rock strata can be extracted from the results of the seepage analyses and used for prediction of ground settlement. Since the differential ground settlement associated with inflow in deep tunnel is expected to be small, it is also assumed that the ground movement will be predominantly vertical. Thus the ground settlement at any point on the ground can be reasonably approximated by looking at the change in vertical effective stress in different soil strata right below that point.

3 KEY OBSERVATIONS FROM THE ASSESSMENT

3.1 Steady state vs transient analyses

To investigate the transient nature of the problem, both steady state and transient seepage analyses for the cross-sections were carried out and the corresponding ground settlement using the above methodology was calculated. Given the construction period for the tunnel is in the order of 2 years, the transient seepage analyses were carried out for 1 year and 2 years intervals. In order to compare with the steady state analysis, a transient analysis for a very long period, say, 100 years is also conducted. As an example, the results from the cross-sections along Tunnel J are tabulated below for a hypothetical case where there is no measure to control the tunnel inflow:

Table 1: Extracts of results from steady state and transient seepage analyses

Cross-section	Steady state	Transient		
	Ground settlement (mm)	1 year ground settlement (mm)	2 years ground settlement (mm)	100 years ground settlement (mm)
J1	92	81	81	91
J3	31	26	27	31
J5	1073	393	495	934

Note: Ground settlement is calculated for the ground surface directly above the tunnel

It can be seen from Table 1 that the results from the steady state analyses and the 100 year transient analyses are consistent. For cross-section J5, the ground settlement predicted for 2 years is only about 46% of that by the steady state analysis. On the other hand, the results from the steady state and transient analyses (2 years) for cross-sections J1 and J3 are similar, implying the ground settlement at cross-sections J1 and J3 would occur more rapidly. The much larger settlement in cross-section J5 is mainly contributed by the consolidation of the thick marine deposit present below the reclamation at that location. This observation also helps to explain why the settlement at cross-section J5 is occurring over a longer period of time.

The tunnel inflow rates calculated for cross-sections J1, J3 and J5 are 103, 123 and 95 L/min/100 m of the tunnel length respectively (at end of 2 year). The predicted ground settlements in some of these cases are obviously excessive and measures to reduce the tunnel inflow rate would have to be taken.

3.2 Variation of ground settlement with tunnel inflow rate

It is expected that the ground settlement would increase with the tunnel inflow rate. To study the effect, further analyses were carried out assuming the inflow rates of 50 and 30 L/min/100 m can be achieved with pre-grouting around the tunnel. Figure 3 shows the results for cross-section J5 for a given period of 2 years. The ground settlement appears to be approximately proportional to tunnel inflow rate. Similar observations are made in other cross-sections.

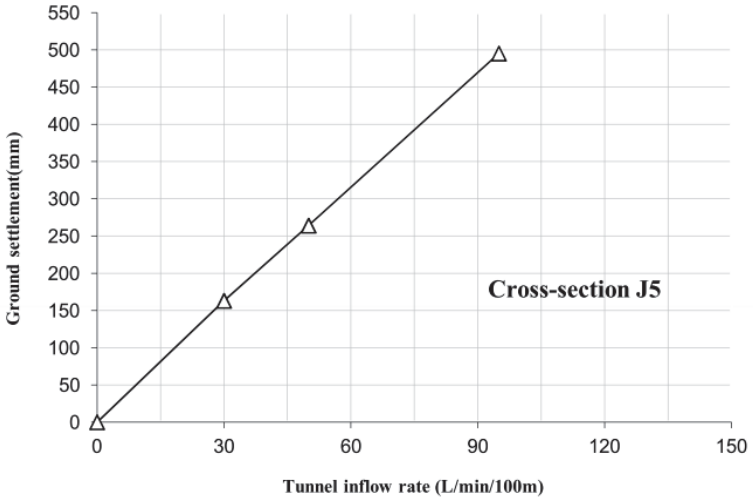


Figure 3: Ground settlement vs tunnel inflow rate (2 year period)

4 CONTROL OF GROUND SETTLEMENT

The amount of inflow rate to be reduced will depend heavily on the maximum allowable ground settlement. For this project, a global value of 50 mm is adopted. It should be noted that this refers to the green field settlement and for structure supported by deep foundation, the settlement of the structure will be much smaller.

Pre-grouting with cement grout around the tunnel is considered as the first measure to control the tunnel inflow rate and the associated ground settlement. However, there is a practical limit where the inflow rate can be reduced by grouting with cement grout. The consensus view is that the inflow rate of below 15 L/min/100 m may be difficult to achieve in local practice. Therefore, if the theoretical tunnel inflow rate has to be less than 15 L/min/100 m, some other types of measure such as first pass lining to reduce the exposure of the rock face will be needed.

Again, take cross-section J5 as an example. If the ground settlement for a 2 year period has to be reduced to 50 mm, then theoretically the tunnel inflow rate will have to be reduced to 7.5 L/min/100 m. Figure 4 shows the variation of ground settlement with time.

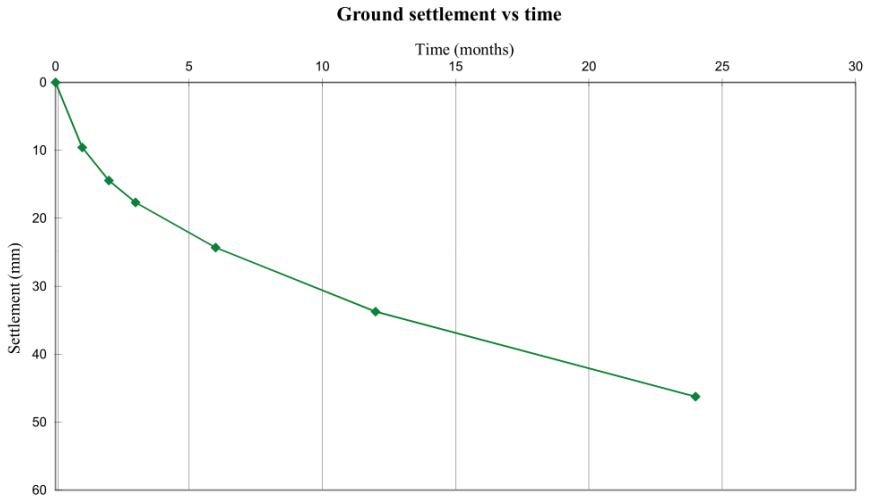


Figure 4: Ground settlement vs time (theoretical tunnel inflow rate = 7.5 L/min/100m)

In this case, the theoretical tunnel inflow rate may be difficult to achieve, an alternative measure using first pass lining can be adopted where temporary tunnel lining will be applied in order to reduce the exposure time of bare rock face. The result of a separate analysis shows that if the time of exposure can be reduced to about

10 months, the tunnel inflow rate can remain as 15 L/min/100m while the predicted ground settlement is also 50 mm.

The above procedure is also applied to other cross-section of the tunnels to determine the allowable tunnel inflow rate and whether first pass lining is needed. However, the grouting techniques to achieve the target tunnel inflow rate and programming for the first pass lining are separate subjects which are not covered in this paper.

5 CONCLUSIONS

Given the scale and complexity of the HATS Stage 2A Sewage Conveyance System project, prediction on the change in hydrogeology during tunneling and the potential settlement entailed has been a challenge. In this paper, the authors present a methodology for the hydrogeological assessment, the typical results of the assessment, key observations and describe how the necessary measures to control the tunnel inflow rate are determined. It has also been highlighted in the paper the need for adopting the observational approach and the transient nature of the problem as reflected by the analysis results.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Director of Drainage Services Department, the Government of the Hong Kong Special Administrative Region and AECOM Asia Company Limited for permission to publish this paper.

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Engineering Geological Approach for Assessment of Quantities and Programme for Deep Tunnels in Hong Kong

J.W. Tattersall, J.K.W. Tam & K.F. Garshol
AECOM Asia Co. Ltd., Hong Kong
K.C.K. Lau
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

Programming of tunnels in hard rocks and estimation of overall costs need to consider the sometimes great influence of the frequency and intensity of grouting that is required to limit groundwater inflow rates to pre-determined levels. This paper focuses on an engineering geological approach to assess the potential frequency of groundwater inflows and grouting effort based on a study of over 20 km of deep sewer tunnels in Hong Kong and Norwegian grouting experience. Many of the geological factors that appear to influence grouting frequency and effort are not considered or are not adequately quantified by existing rock mass classifications. Also, the geological conditions in the inevitable 'gaps' between drillholes can only be inferred based on indirect evidence from typical ground investigations and consideration of the overall geological setting. Notwithstanding these limitations, the range of impact that grouting may have on the programming of a project still needs to be estimated.

In an effort to meet this need for the design of the Harbour Area Treatment Scheme (HATS) Stage 2A tunnels, the engineering geological (EG) data and probing records from the HATS Stage 1 tunnels were examined. Trends in frequency and magnitude of groundwater inflows from probe holes were correlated with the engineering geological characteristics of the rock masses. This paper describes the approach adopted to facilitate estimation of costs and programme.

1 INTRODUCTION

A good geological model is essential for the geotechnical assessment of any underground construction project. The synthesized model requires expert interpretation of all relevant information including archival and project-specific investigation data. For deep tunnel construction in rock, geological and hydrogeological characterization of the ground is very important to enable realistic assessments of cost and programme and the recognition and management of geotechnical risk. Key issues for deep tunnel construction beneath intensive urban development built largely on reclamations and superficial deposits are the determination of the extent to which groundwater inflows and related groundwater draw down need to be controlled and the cost and programme implications of the mitigation works that will be required. This paper outlines the EG approach that was adopted to address these issues in connection with the design of HATS Stage 2A Sewage Conveyance System (SCS). The assessment was greatly facilitated by the lessons learnt from the construction of HATS Stage 1 and the availability of detailed records from this project on advance probing, groundwater inflows and geological conditions contained in an electronic 'Tunnel Database Management System' (TDMS). Detailed geological face logs were also examined to gain further insight into pertinent EG conditions that are not readily captured by rock mass classification systems originally intended for assessments of tunnel support only.

2 PROJECT BACKGROUND OF HATS STAGE 2A

HATS Stage 2A, commissioned by the Drainage Services Department (DSD), is aimed at further improving the quality of Hong Kong's inshore marine waters. The project includes the construction of 20 km of tunnels

and 13 vertical shafts. The tunnel alignment lies between 70 m and 160 m below sea level with long sections beneath urban areas built on reclamation. Construction of the tunnels could lead to unacceptable groundwater draw down and settlement if groundwater inflows are not sufficiently controlled. Groundwater control during construction is therefore a key priority for HATS Stage 2A which is reflected in the construction contracts by providing items for payment of grouting and tunnel support works on a re-measurement basis. This required realistic estimates to be made of quantity and impact of grouting on the construction programme.

The target levels of residual groundwater ingress during construction typically range from 5 to 30 L/min/100 m of tunnel under urban areas, and 2.5 L/min/100 m of tunnel for the most critical location. In order to satisfy these requirements, systematic probe drilling in front of the excavation face is mandatory and pre-excavation grouting (PEG) is required to reduce the residual groundwater inflows to acceptable levels.

The three contracts for construction of the SCS tunnels and shafts commenced July/August 2009. The works are anticipated to be completed in 2014.

3 EG APPROACH FOR ROCK MASS CHARACTERIZATION AND ASSESSMENT OF HATS STAGE 2A TUNNELS

3.1 Introduction to EG model and outline of overall approach

A snapshot of an EG plan of North Hong Kong Island and Kowloon is shown in Figure 1. This is based on interpretation of relevant available data from past projects, HATS Stage 2A investigations and archival data for more than 20,000 drillholes. EG interpretation, particularly the inferred rockhead surface and location and influence of faults needs to be consistent with all lines of evidence. The EG model also includes as-built data from past projects such as rock mass conditions, tunnel support requirements and groundwater inflows experienced during construction. As-built data from the HATS Stage 1 tunnels A/B, C, D and E which were constructed in Kowloon Granite (Klk) and volcanic rocks of the Repulse Bay Group are relevant to the HATS Stage 2A tunnels which are being constructed in the same or similar rock types.

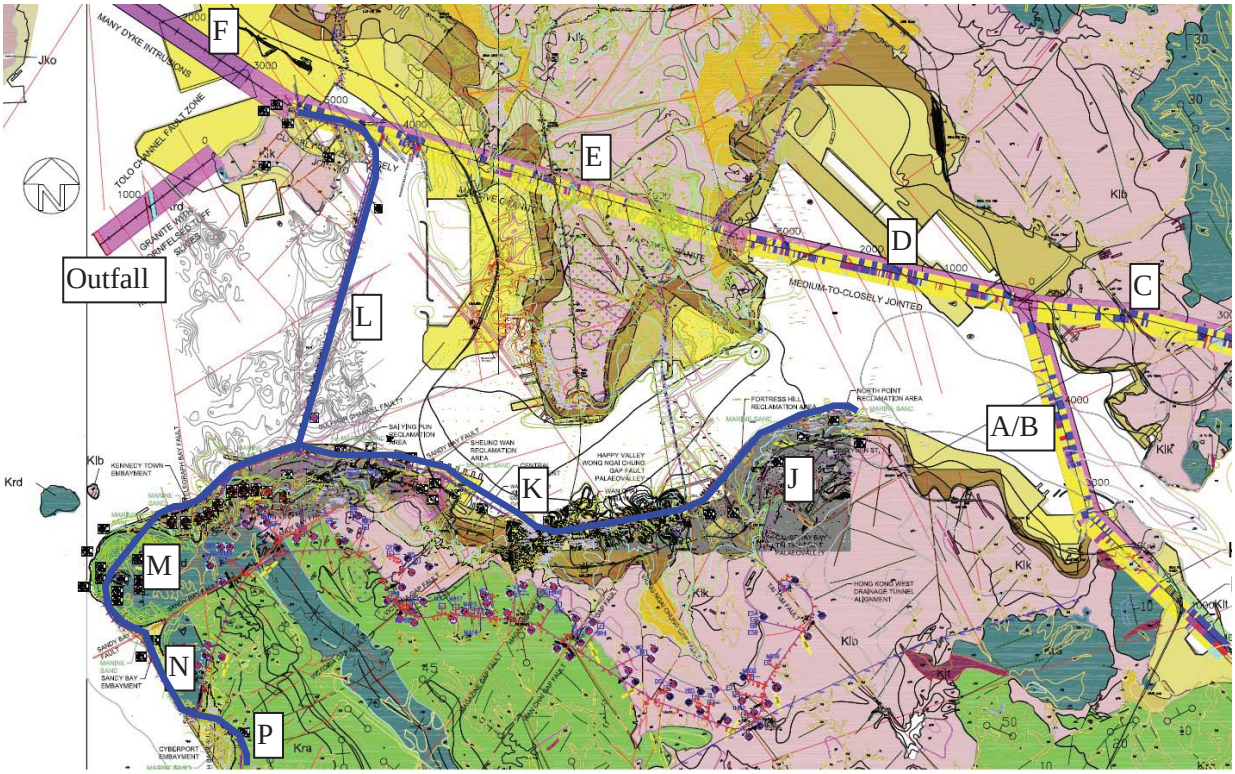


Figure 1: Snapshot of EG Plan of the Harbour Area Showing HATS Stage 1 Tunnels A/B, C, D, E & F and HATS Stage 2A Tunnels J, K, L, M, N & P.
Note: Plutonic rocks shaded pink. Volcanic rocks shaded green.

An outline of the overall EG approach to ground characterization and estimation of programme and quantities for the HATS Stage 2A tunnelling works is given in Figure 2 below.

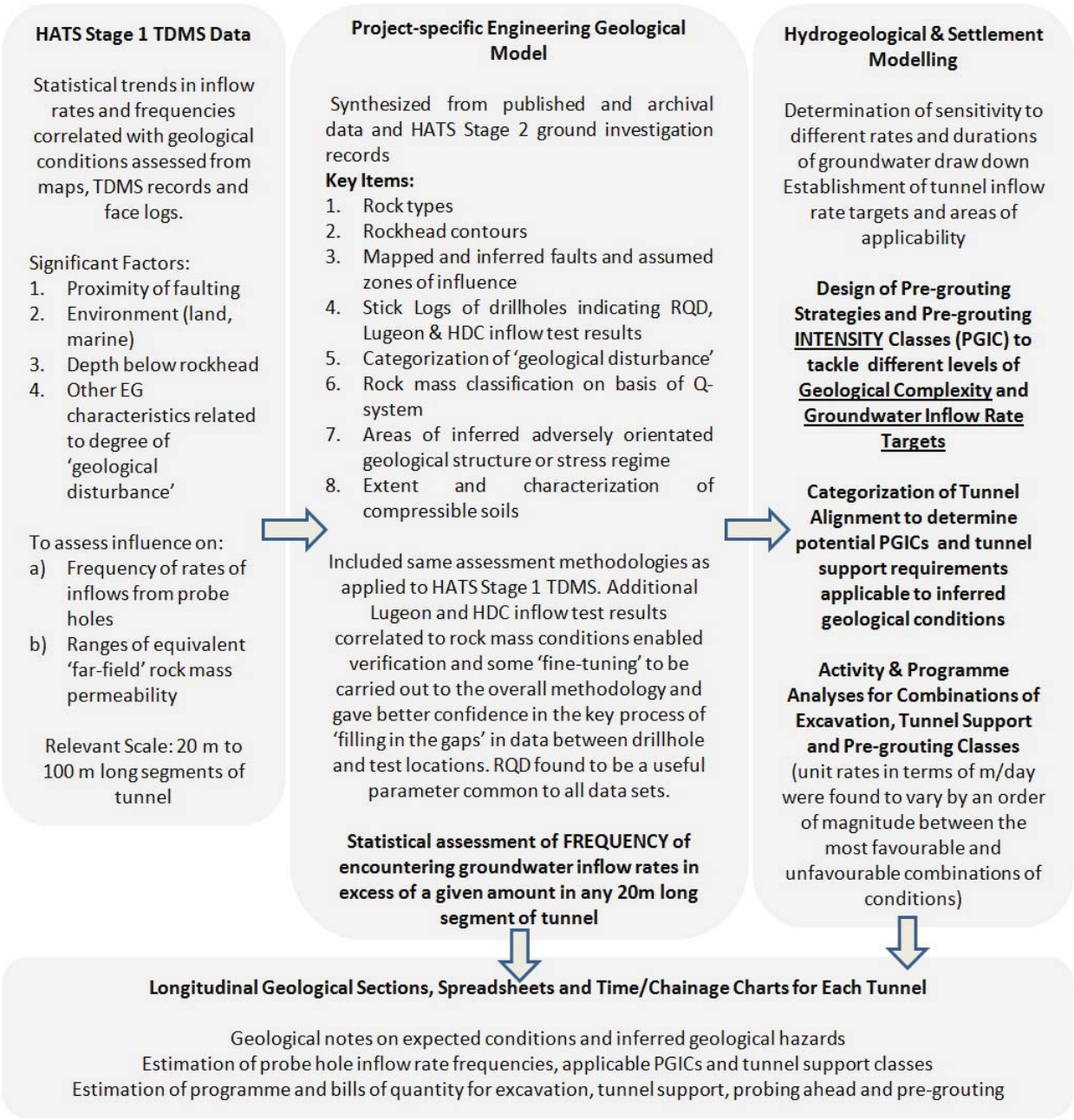


Figure 2: Overall approach to ground characterization and estimation of quantities and [rogramme for HATS Stage 2A tunnelling works

3.2 Significant geological factors relating to frequency and rates of groundwater inflows in probe holes

Previous experience and as-built records from HATS Stage 1 were assessed to identify significant geological factors that are statistically discriminating with respect to frequency of rates of groundwater inflows encountered in probe holes drilled ahead of the face (Endicott & Tattersall, 2009; Tattersall, 2010). The relationships established are summarized in Figure 3 and Tables 1 & 2 below. The key factors indirectly reflect the inherent block size of the parent material (i.e. widely-jointed to massive granite or closely to medium-jointed volcanics), the degree to which the rock mass becomes more highly fractured and disturbed by brittle tectonic disturbance (proximity of faulting) or stress relief (i.e. relatively thick rock cover below

land and thinner rock cover under marine conditions) and frequency of persistent or inter-connecting features of higher hydraulic conductivity (e.g. closely jointed dykes and veins or joint infills of hard, crystalline materials such as pegmatite, quartz or calcite which frequently contain voids).

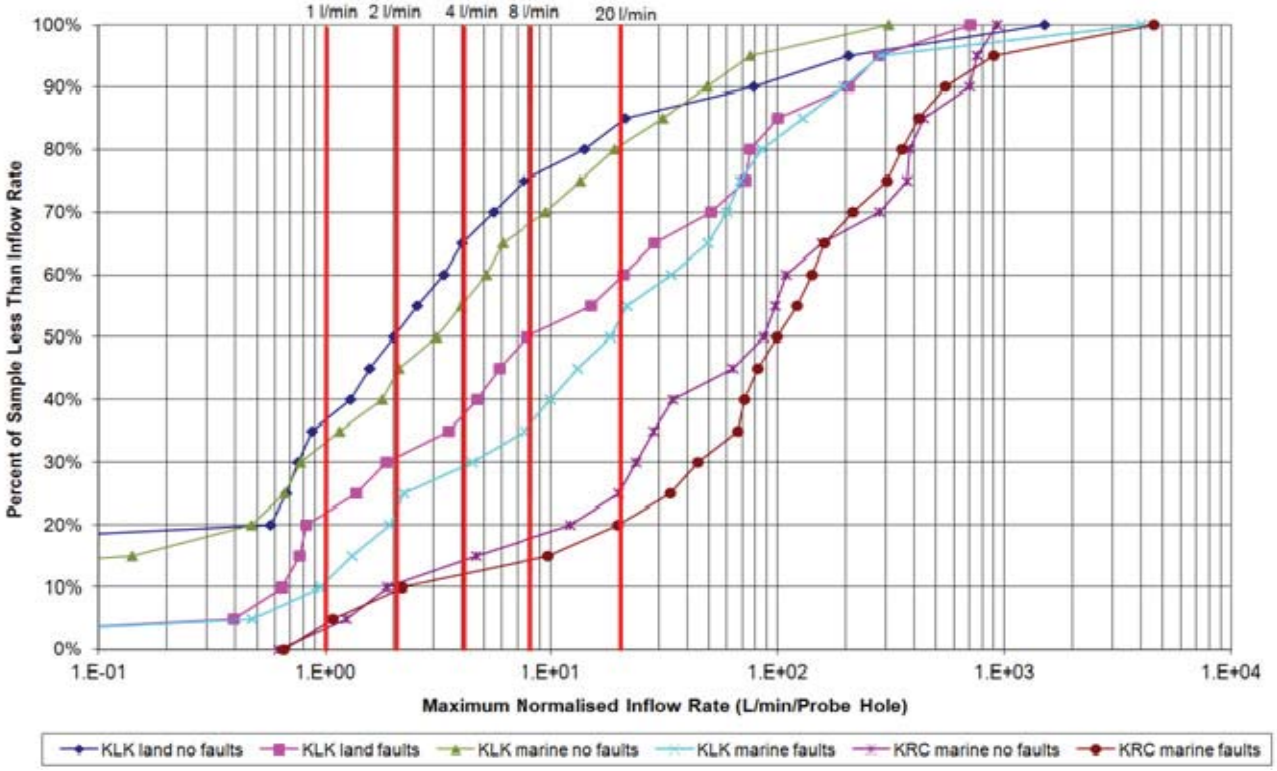


Figure 3: Variation in probe hole inflow distributions with rock type, environment and proximity to faults
Note: KLK: Kowloon granite. KRC: closely-to medium-jointed, volcanic Che Kwu Shan Formation, 'Faults': within 25 m of a minor fault or within 75 m of a major fault.

Table 1: Classes of geological disturbance for probe hole or drillhole assessments

Class	Condition
E	Fault
D	Dykes and absence of Class E conditions
C	Frequent crystalline veins and absence of Class D and E conditions
B	Frequent crystalline joint infills with RQD < 80 or no infills with RQD <40 and absence of Class C, D and E conditions
A	Absence of Classes B, C, D and E conditions

Table 2: Probe/grout fan intensities and inflow frequencies related to geological disturbance

	Klk Land		Klk Marine		Krc Marine	
Disturbance Class	Average No. of holes per fan	% fans with inflow >20 litres/min	Average No. of holes per fan	% fans with inflow >20 litres/min	Average No. of holes per fan	% fans with inflow >20 litres/min
E	5.1	29	7.8	65	7.6	90
D	2.8	0	5.3	37	7.7	83
C	2.9	40	4.3	31	3.6	70
B	3	30	5.3	58	2.5	76
A	2.7	16	3.7	18	2.6	37

As can be seen from Figure 3, proximity of faults has a larger effect on groundwater inflow distributions in the inherently less jointed granite than in the inherently more jointed and more frequently highly transmissive volcanic rock. The factors identified in Figure 3 are more readily amenable to spatial application within a GIS-based geological model. However, the classes of geological disturbance in Tables 1 & 2 can be applied to direct evidence from drillholes and can also be used to identify likely zones of high hydraulic conductivity where drillhole information exists.

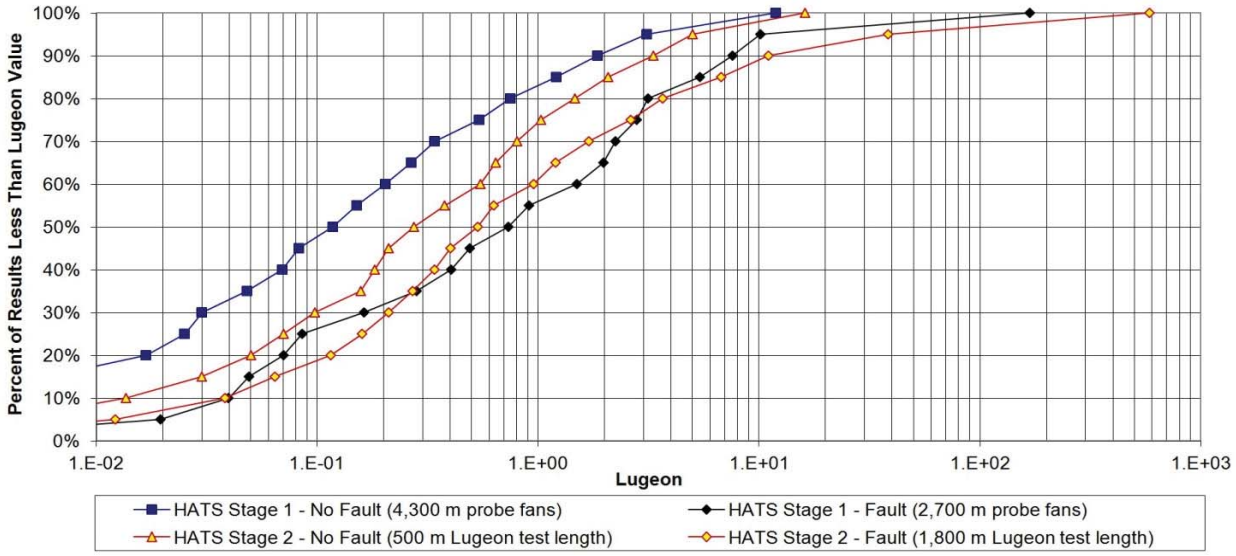


Figure 4: Hydraulic conductivity distributions for Klk (marine) from probe hole and Lugeon test data

Figure 4 shows the marine HATS Stage 1 data for Klk (probe hole inflows converted to equivalent Lugeon values) and the marine HATS Stage 2 Lugeon test data for Klk plotted on the same distribution chart. Very similar distributions are evident for the rock masses affected by faults. For Klk outside the influence of faults, the distribution from the HATS Stage 2 Lugeon testing appears more adverse than indicated by the HATS Stage 1 results. However, the HATS Stage 1 data represent about 4,300 m of near-continuous probe drilling (no selective bias), whereas the sample length for HATS Stage 2 is considerably smaller and is not continuous. Endicott & Tattersall (2009) discuss the limitations on applicability of isolated Lugeon testing and highlight the potential skewing that can occur when tests are targeted towards the more obviously jointed sections of a drillhole. In more closely fractured rocks, e.g. Klk affected by faults, there would appear to be less scope for selective sampling and the hydraulic conductivity distributions are similar. Comparisons were also made between the HATS Stage 1 probe hole data for closely to medium-jointed Che Kwu Shan Formation and the HATS Stage 2 Lugeon test data for the similarly jointed Ap Lei Chau Formation. The distributions for both 'Fault' and 'No Fault' categories were also found to be very similar.

3.3 Application to HATS Stage 2 EG model for assessment of tunnel support quantities and the need to carry out dedicated fans of PEG

Based on the similarities between the HATS Stage 1 data and HATS Stage 2 test data, the tunnels of HATS Stage 2 were categorized in terms of likely 'typical' hydraulic conductivity using the three criteria of rock type, proximity of faults and depth below rockhead in addition to direct, local evidence from drillholes. Data from both HATS Stages 1 and 2 and over 60 km of previous drillhole logging and as-built tunnel records for other projects also indicate good statistical correlations between the same three criteria and trends in RQD, block size and rock mass Q-values (Tattersall, 2010).

Application to the EG model for HATS Stage 2 helped to assess the hydrogeological characteristics and rock mass quality within the large 'gaps' in-between drillholes. The primary purpose was to aid assessment of the frequency of occurrence of different ranges of rock mass quality and hydraulic conductivity and hence facilitate estimation of tunnel support quantities and the frequency of the need to carry out PEG in dedicated

grouting fans to reduce residual groundwater inflows in the tunnels to less than or equal to the targets established on the basis of groundwater draw down and settlement modelling.

3.4 Estimations of grouting quantities and tunnel construction programme

Assessing the frequency of the need to grout is not sufficient on its own to derive realistic assessments of quantity and impact of grouting works on the tunnel construction programmes. This is because the quantities of grout holes, materials and time required are heavily dependent on the grouting difficulty and effort to achieve a given residual tunnel inflow target. In this respect, the hydraulic conductivity of the rock mass or measured magnitude of groundwater inflow from probe holes may not be a major factor in determining the required pre-grouting intensity since large inflows from a few joints of large aperture may be satisfactorily pre-grouted far more easily than smaller, well-distributed inflows from many joints of variable condition.

Scandinavian experience suggests that the required intensity of grouting is primarily dependent on the complexity of geological conditions, residual inflow rate targets and groundwater head. Table 3 below shows an outline of a matrix that was referenced when interpreting the overall EG model to provide crude estimates of 'Pre-grouting Intensity Class' (PGIC) to help determine Bills of Quantity and programme. The matrix is based on previous work by Scandinavian experts (Beitnes, 2009) adapted to Hong Kong conditions. The PGICs for estimating purposes range from 'I' for the least intensive to 'VI' for the most intensive. When considered in combination with tunnel support requirements, the range in estimated construction progress rate was found to vary by an order of magnitude between the best and the worst combinations of conditions.

Table 3: EG conditions affecting range of PGICs assumed for estimating purposes

Engineering Geological Conditions	**PGIC I**	**PGIC II**	**PGIC III**	**PGIC IV**	**PGIC V**	**PGIC VI**
Massive Rock with few joints or RQD = 100	⟷	⟷				
3 joint sets with variable apertures or RQD = 70 – 100		⟷	⟷			
Complex, multiple joint sets and joint conditions or RQD = 25 – 70			⟷	⟷		
Weakness Zones of Bedrock > 2.5 m thick with RQD <25			⟷	⟷	⟷	
Mixed Ground or Soil > 2.5 m thick					⟷	⟷
Hydrostatic Pressure <5 Bar: Reduce grouting pressure as necessary. Assume upgrading of PGIC I-V by one class.						
Highly anisotropic transmissivity favouring discontinuities sub-parallel to tunnel axis: May require multiple injections of different grouts and grouting pressure may need to be reduced to limit excessive 'travel'. Assume upgrading of PGIC I-V by one class.						

Note: Arrows indicate approx. influence of range of applicable residual inflow targets (left to right: least stringent to most stringent)

4 CONCLUSIONS

The statistical EG/hydrogeological relationships found from examination of HATS Stage 1 data helped to gain greater insight into key indicators that can be practicably applied to assess rock mass hydraulic conductivity on a statistical basis. The relationships were tested against project-specific Lugeon test data and were found to be discriminating. However, it should not be expected that relationships assessed for one geological unit can be directly applied to other geological units subjected to different geological conditions over time – although general principles and overall trends are likely to be similar.

The same key indicators used to assess frequency of the need for dedicated fans of PEG can also be referenced to obtain estimates of tunnel support requirements based on past experience in Hong Kong.

Prognoses of grouting intensity which are necessary to give crude, preliminary estimates of quantity and programme are notoriously fraught with uncertainty due to local variations in the combined influence of a large number of factors that are almost impossible to adequately assess using current ground investigation practice. Much EG interpretation, judgment and reliance on previous experience are necessary. Consideration and extrapolation of only the EG conditions listed in Table 3 and their implicit connotations is very much a simplified but pragmatic approach.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Director of Drainage Services Department, the Government of the Hong Kong Special Administrative Region for permission to publish this paper.

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Structural Geological Input for a Potential Cavern Project in Hong Kong

C.D. Jack, S. Parry & J.R. Hart
GeoRisk Solutions Limited, Hong Kong

ABSTRACT

Rock engineering requires an in-depth understanding of structural geology to assist with successful project outcomes. This paper examines how structural geological information can be incorporated effectively into a conceptual engineering geological model with reference to the proposed Sha Tin (Shek Mun) Strategic Cavern Area. It also examines the potential rock engineering implications and engineering geological uncertainties for the proposed Sha Tin Sewage Treatment Works Cavern site.

1 INTRODUCTION

Rock engineering requires an in-depth understanding of structural geology, as discontinuities govern the behaviour of many rock masses at the scale of most rock engineering projects. Consequently, the development of a conceptual engineering geological model by an experienced engineering geologist should be an early product for any rock engineering project. All too often the structural geological information for such projects is superficially evaluated and interpreted in isolation, without reference to an overall hypothesis or model. Given the variable and complicated nature of structural geological data, the lack of such an interpretative framework can result in misleading or incorrect interpretations.

The preferred approach (Baynes et al, 2005), is to develop a series of evolving engineering geological models (conceptual, observational and analytical), which explicitly and systematically include structural geological data. This paper demonstrates the use of conceptual engineering geological models in rock engineering, using as a case study the proposed Sha Tin Strategic (Shek Mun) Cavern Area in Hong Kong. The case study demonstrates how structural geological data from desk studies can be quickly, effectively and inexpensively incorporated into a conceptual engineering geological model. This in turn allows the establishment of a register of uncertainties and risks and demonstrates how key information can be quickly gathered in the early stages of a project, allowing better decision making. The paper also examines the required ‘transforms’ and appropriate methods of communication for conveying effectively the important aspects of these models and their associated uncertainties and risks to non-engineering geologists e.g. the planners and engineers involved in the design and construction. This approach ensures that the key points are understood by the project team and the model is re-evaluated during the later phases of the project. Finally, the paper discusses how the conceptual model can be used to develop the observational and analytical engineering geological models.

2 THE PROPOSED SHA TIN (SHEK MUN) STRATEGIC CAVERN AREA

The proposed Sha Tin (Shek Mun) Strategic Cavern Area (the 'cavern area') is one of five such areas identified in a recent report (GEO, 2011) and the location is shown in Figure 1. It is located under Nui Po Shan (Turret Hill), which lies to the south and east of the Shing Mun River Channel. As described by GEO (2011), the Sha Tin Sewage Treatment works, located on the opposite side of the Shing Mun River Channel (Figure 1), is considered to be a particularly suitable facility for relocation into caverns within the cavern area. As such, this facility has been selected as one of three preliminary feasibility studies (note that this paper is unrelated to these feasibility studies). Section 2 of this paper will discuss the geological structure of the cavern area in general, while Section 3 will focus on the engineering geological issues of the proposed sewage treatment works cavern site.

A desk study for the cavern area was carried out for the purpose of this paper, in which the relevant geological maps, memoirs, reference texts and published papers were consulted. This research was not as comprehensive as would be required for an actual feasibility study, as the aim of this paper is to demonstrate how basic structural geological information for Hong Kong can be relatively quickly and easily obtained and synthesized into a conceptual engineering geological model. The main limitations of this paper are: not all of the available reference material was consulted; a site specific aerial photograph interpretation (API) and site reconnaissance were not carried out; and, those features or aspects of features unrelated to structural geology, rock engineering and caverns were not considered. In particular, it is considered that a site specific API and reconnaissance (in particular a visit to Turret Hill Quarry) are critical to the development of a robust conceptual engineering geological model. Furthermore, it is recommended that a reinterpretation of the lineaments is made using high resolution LiDAR data and reference should be made to the records of the numerous water tunnels in the area. However, for the purposes of this paper the API and field mapping information from two unrelated studies at the cavern area were deemed to be sufficient to illustrate the concept. It should be noted that some geological observations are based on the 1986 edition of the 1:20,000 geological map, rather than the 2008 revised version. Finally, the cavern area extends to the south of the Ma On Shan fault, into volcanic rocks of the Repulse Bay Volcanic Group and Clearwater Bay formation. However, this area has not been considered for the purposes of this paper. A synthesis of the findings of the desk study is as follows.

2.1 Solid geology

To the north of the Ma On Shan Fault, the cavern area is situated in the Shui Chuen O Granite of the Cretaceous Cheung Chau Suite and the Sha Tin Granite of the Jurassic Kwai Chung Suite (underlying the upper, steeper slopes of Nui Po Shan). Some minor sections of the cavern area also fall within the Cretaceous Tei Tong Tsui Quartz Monzonite of the Lion Rock Suite. It is interpreted that the Needle Hill Granite was the first pluton intruded in the region (which also belongs to the Kwai Chung Suite). This in turn was intruded by the Sha Tin Granite. These units were subsequently intruded by the Shui Chuen O Granite during the Cretaceous. The final significant phase of intrusion in the area was the intrusion of the NE trending dykes of the Tei Tong Tsui Quartz Monzonite. It is clear that all of these intrusions are strongly controlled by a NE trend, with the intrusions being elongate in this direction, having been controlled by transtension (probably dominantly extensional) associated with deep crustal structures. Tectonic activity has continued after the intrusion of these rocks, which have subsequently been faulted, jointed and intruded by dykes, again following the NE orientation, which is the dominant structural trend in Hong Kong (the Lianhuashan Fault Zone) and which includes the Lai Chi Kok – Tolo Channel Fault (Sewell et al, 2000), discussed in the following section.

2.2 Structural geology

A consideration of faulting is of importance in determining the regional structural trends, which can help in interpreting joint trends. Figure 1 shows the locations of the faults and lineaments interpreted in the region of the cavern area. This interpretation is based on a combination of faults and photolineaments interpreted on the 1:20,000 and 1:100,000- scale geological maps, Lau & Kirk (2001) and a site specific interpretation of a 1:5,000-scale digital elevation model (DEM) by the authors. The main trends in general decreasing order of magnitude are NE-SW, NW-SE, ENE-WSW and WNW-ESE (the regional dyke swarms also follow these trends, with NE also being the dominant trend of the minor intrusions and dykes). As a result of the previous tectonic and igneous activity, structures will exist at all scales from microstructures, through joints and to faults of varying magnitude.

There is no indication from this basic desk study of major fault zones crossing the proposed cavern area and this is probably one of the reasons that this region has been selected for consideration. However, a number of lineaments, many of which probably represent moderate and minor faults are present (faults in this paper have been tentatively classified as *deep seated, major, moderate* and *minor* on the basis of Table IV in Burnett & Lai, 1985). Therefore, knowledge of the faulting pattern can be used to minimise the risks posed by unidentified faults to underground excavations in the cavern area and to reduce the risk of encountering unforeseen ground conditions. It also allows early optimisation of the cavern orientation with regards to the joint sets and potential instability.

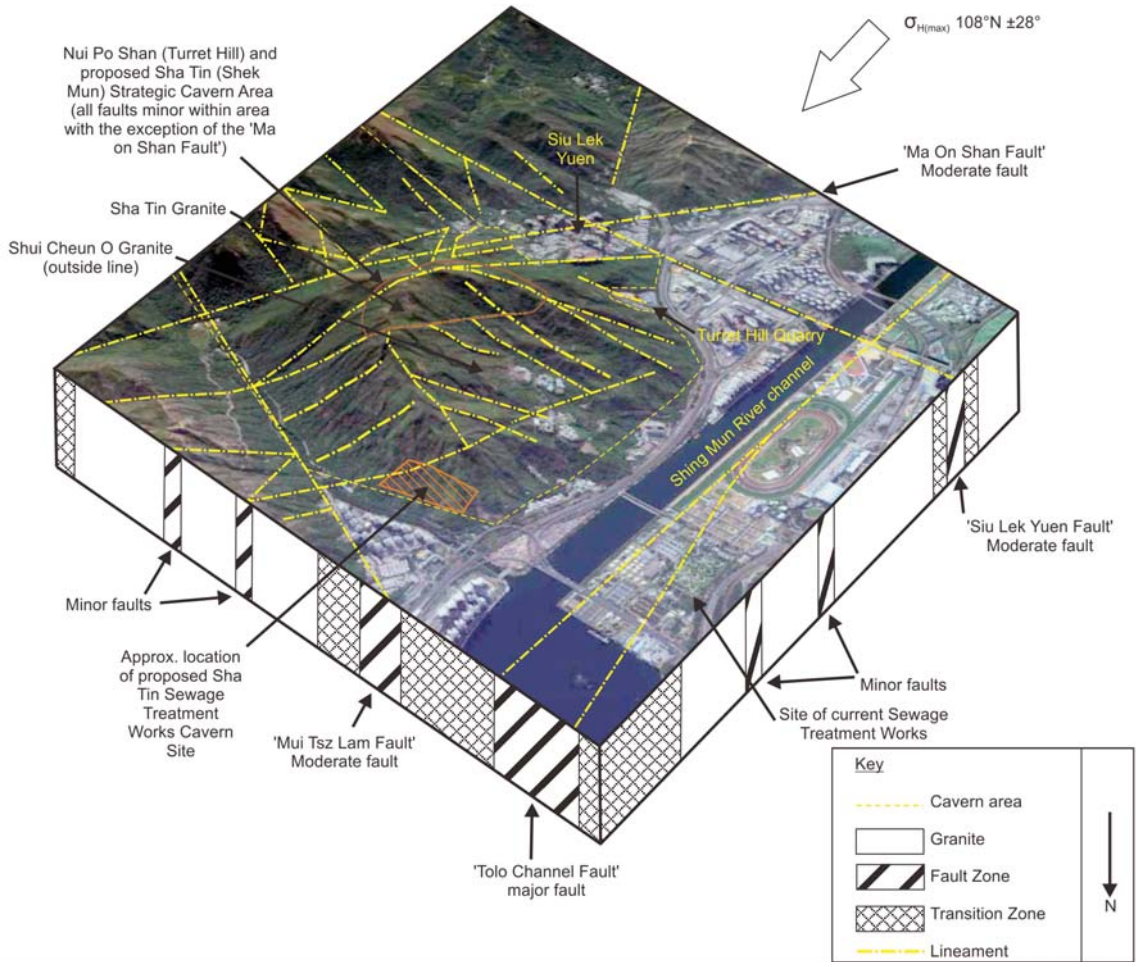


Figure 1: Proposed cavern area – location and regional conceptual engineering geological model (Image © 2011 Google, © GeoEye, ©2011 DigitalGlobe)

NE-SW trend: This is the main structural trend in Hong Kong, forming a component of the Lianhuashan Fault Zone. A NE trending major fault zone, the Lai Chi Kok - Tolo Channel Fault (Burnett & Lai, 1985, henceforth referred to as the Tolo Channel Fault) is located to the immediate north of the cavern area (and may affect the northernmost part of the cavern area), running through the Sha Tin urban area and parallel to the Shing Mun River. The fault zone has probably been periodically active since the late palaeozoic and may have continued to be active until 3-4 Ma. The NE trending moderate Ma On Shan fault is located in the southern part of the cavern area and is mapped as downthrown to the south. The Tolo Channel Fault and, to a lesser extent, the Ma On Shan Fault are considered to be subvertical, trending 050° (approx.) with crush zones of several to tens of metres wide (30 m width has been found on the Tolo Channel Fault), comprising, cataclasite, breccia, slickensides, sheared granite, fault gouge, closely jointed zones adjacent to the fault and significant kaolinization and chloritization of the host granite. There may also be mafic and quartz monzonite intrusions locally. Minor faults of this trend, if encountered in the cavern area, would be expected to have related albeit much less well developed characteristics. Adjacent faults of this trend appear to be 1.5 to 2 k m apart (Lau & Kirk, 2001).

NW-SE trend: This is the second most important structural trend in Hong Kong and these are considered to be moderate faults in accordance with Burnett & Lai (1985). In the region these faults dip steeply and trend between 310° to 340°. A NW trending fault is probably located to the immediate west of the cavern area running parallel with Siu Lek Yuen Road. Another, NW trending fault (interpreted to downthow to the NE), and possibly of greater magnitude as it has a greater topographic expression, is located to the immediate east of the proposed cavern running along the well-defined Mui Tsz Lam Valley. Both of these faults may affect the areas at the boundary of the cavern area. In a few locations mafic dykes are intruded along faults in the region with this trend. Some quartz veins with mineralisation are associated with faults with a similar trend to

the west. These faults are of compressive-shear or tensile-shear type, with a narrow crushed zone. They are thought to have been active since the late Mesozoic (possibly active up to 33,330 +/- 2,700 years BP according to Ding & Lai (1997). Present river systems, estuaries, channels and sections of coast are commonly influenced by faults of this orientation. A lineament of this trend has been identified in the NE part of the cavern area. These faults are often found to be cross-cut and offset by the NE-SW trending faults (Lau & Kirk, 2001).

WNW-ESE trend: Several WNW-ESE trending lineaments have been identified within the cavern area. The orientation of these features indicate a probable structural origin and they appear to be the least persistent lineaments. This might correspond with the findings of Lau & Kirk (2001) who indicate that most faults in the region with this trend are minor faults that are 0.5 to 1 km in length. Where faults of this orientation have been described in Lau & Kirk (2001), they vary in width (they can be many metres wide), vertical or inclined and may be weathered to considerable depths. Quartz mineralisation (hydrothermal) is commonly associated with the faults.

ENE-WSW trend: There is less published information on the nature of the ENE-WSW trending faults as these are typically minor faults. There is a group of faults of this orientation to the north of the Tolo Channel Fault that strike about 060° to 080°. These faults are intensely brecciated and are associated with sericitisation. Movements on these faults are relatively minor, the main effect being to form a minor graben on the northern side of the granite outcrops. Minor zones of mylonite and narrow shear zones occur striking 065° to 075° (Addison, 1986). Several lineaments of this trend have been identified crossing the cavern area.

The following information on the joints is mainly derived from information from two unrelated study reports which have been carried out within the cavern area. Note these two studies were located in the Shui Chuen O Granite and the joint patterns in the Sha Tin Granite may differ. The subvertical joints are likely to be tectonic joints, formed in relation to the faulting. However, the possibility that some of these joints might have originated as cooling joints cannot currently be discounted. The engineering properties of joints are controlled by their mode of formation and subsequent history. Therefore, establishing the mode of formation of the joints can significantly assist with rock mass characterisation and subsequent rock engineering analysis (Hencher et al, 2010).

Sheeting joints: Where recorded during the studies it was found that the sheeting joints dip 20° to 50° towards 260° to 305° (average 25°/285°), subparallel to the slope faces (note that the orientations of sheeting joints will vary across the cavern area and these values are unlikely to be applicable elsewhere). The joints are typically associated with increased weathering and typically have 1 m to 3 m spacing (reducing to as low as 0.5 m near the surface). These joints are best developed near the ground surface and spacing is expected to increase with depth, with sheeting joints probably confined to within 30m of the surface (Hencher et al, 2010). These joints have wavy, rough undulating surfaces, are occasionally dilated (<15 mm) and infilled with kaolin. Seepages along these joints have been reported.

NE-SW trending steeply dipping to subvertical joints: This set dips 55° to 90° towards 325° to 345° (pole concentration at 80°/150°). This joint set has a persistence >10 m length on some rock slopes and the joints are typically smooth planar to rough planar, and close to medium spaced, but can be widely spaced.

NW-SE trending steeply dipping to subvertical joints: This sets dips 75° to 90° towards 220° to 250° (average 75°/225°). These joints are occasionally slickensided (one measurement plunging 53°/347°), indicating that they may occasionally form minor faults, given the NW-SE trend and presence of slickensides and associated quartz veins. This joint set is tentatively considered to be widely spaced (considered to have an average spacing of 1 m, but ranging from 0.6 m to 2 m) and the joints are described as smooth planar to rough planar. A drainage line within the proposed cavern area also appears to be controlled by an extremely persistent discontinuity dipping 75° towards 230°.

The sheeting joints are considered to have been formed by stress relief close to the land surface, although they may have developed along pre-existing microstructures and joints (possibly related to cooling). The subvertical joints are most likely tectonic joints related to the faults. However, some of the steeply dipping or subvertical joints might also represent cooling joints roughly subparallel to the flanks of the Shui Chuen O Granite and Sha Tin Granite intrusions.

3 IMPLICATIONS FOR THE PROPOSED SHA TIN SEWAGE TREATMENT WORKS CAVERN

The preceding information is of importance to the rock engineering aspects of the proposed cavern, but its use is limited if the key points are not communicated effectively to the project team. One of the engineering geologist's roles is to take the kind of geological information presented in Section 2 and 'transform' this into a form that is of use to those who will use the data, typically, but not only, engineers (Baynes, 1999). This transformation requires a synthesis of the information into text, maps/plans, sections, block models, uncertainty and risk registers. In the case of this paper, the text of Section 2 along with Figure 1 constitutes a basic conceptual engineering geological model for the cavern area. Figure 2 provides a larger scale model for the proposed Sha Tin Sewage Treatment Works Cavern site, which is located within the NE corner of the cavern area as shown on Figure 1. In addition, Tables 1 and 2 are intended to communicate some key items of engineering significance and uncertainties for the proposed sewage treatment works cavern site. Note that a full conceptual engineering geological model would provide a much more comprehensive account.

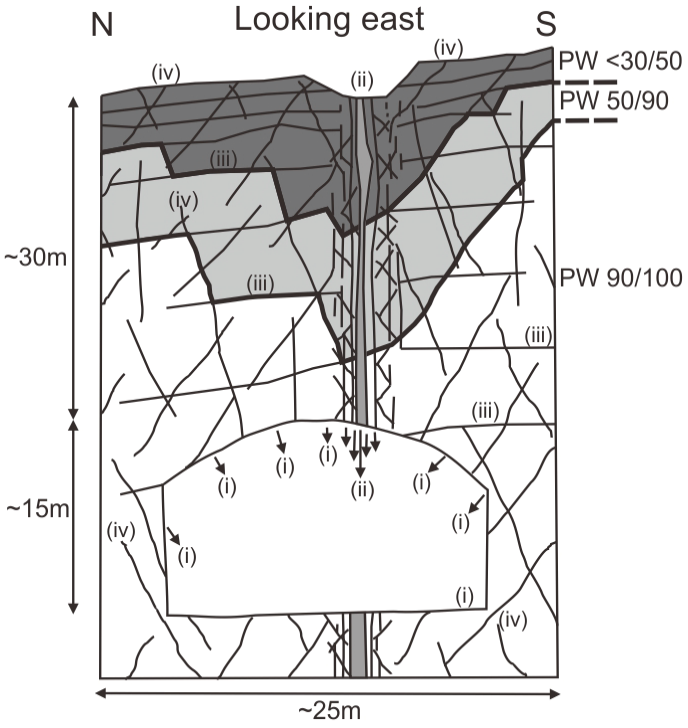


Figure 2: Proposed Sha Tin Sewage Treatment Works conceptual engineering geological model (Note: Roman numerals refer to Table 1)

On the basis of Section 2 and Table 1, a simple uncertainty register has been prepared (Table 2). Those uncertainties which have engineering significance to the project should be transferred to a risk register and proactively managed.

4 CONCLUSIONS

The following conclusions can be drawn from this basic assessment:

- The main fault trends in the region of the proposed cavern area, in general order of decreasing magnitude, are NE-SW, NW-SE, ENE-WSW and WNW-ESE.
- Minor faults or lineaments are interpreted to cross the southern end of the proposed sewage treatment works cavern site, although unidentified minor to moderate faults may be present (classification of faults in accordance with Burnett & Lai, 1985).
- There may be scope to optimise the orientation of the proposed sewage treatment works cavern site in relation to identified lineaments, primary joint trends and *in-situ* stress.
- The primary joint trends near the cavern site are NE-SW and NW-SE with sheeting joints likely to be present, predominantly within 30m of the surface, orientated subparallel to slopes.
- These are not the conditions that will be encountered during future investigations or construction, they are only a possible model of the conditions that may be encountered.

Clearly the uncertainties that remain and the risks associated with these for the proposed cavern area are not insignificant and hence considerable additional research, investigation and analysis is required. It is hoped that this brief paper, which constitutes a basic conceptual engineering geological model, has demonstrated the advantage of such models for reducing the risk of unforeseen ground conditions, along with facilitating early decision making (such as cavern location and orientation), cost estimates and planning the optimum development of the observational engineering geological model (as in providing key features, uncertainties and risks to be targeted by mapping and GI). It also anticipates and assists with the development of the analytical engineering geological model (providing assumptions and parameters which can be investigated,

tested, refined or discarded). Finally, it is hoped that this paper shows how easily structural geological data can be obtained and synthesized for Hong Kong, given the information available.

Table 1: The possible engineering significance of the structural geology – conceptual engineering geological model stage

Feature	Engineering significance and rock mass behaviour
General rock mass (i)	For the purposes of this paper it is assumed that the proposed sewage treatment works cavern site is typically situated at 30m depth or greater in moderately to slightly decomposed Shui Chuen O Granite (although weathering in Hong Kong can easily extend to these depths or greater and therefore the proposed caverns may need to be deeper). The orientation assumed is as shown in the GEO (2011) report. It is further assumed that the sheeting joints (see below) will be less well developed or absent below this depth (Hencher et al, 2010). Based on the approach suggested in Chapter 5 of Palmstrom & Stille (2010) the granite at the cavern depth might generally be classed, depending on the extent of weathering along joints, between *'jointed rocks or blocky materials – Class B – rocks intersected by joints and partings - jointed homogenous rocks'* and *'jointed rocks or block materials - Class C – jointed rocks intersected by seams or weak layers – prominent weathering along joints'*. Consequently, it is assessed that the main issues associated with the typical rock mass will be block falls and areas of water inflow. Based on an assessment of the information in Section 2 and the conceptual engineering geological model, the following parameters have been derived as estimates for initial consideration: Strength (100 – 250 Mpa); block dimensions might typically range from 0.1m to 2m, perhaps with 0.5m being an average; GSI (50 to 80, assessed typical value – 75); Q (best – 37.5 [RQD 100, Jn 4, Jr 1.5, Ja 1, Jw 1, SRF 1]; typical – 4.5 [RQD 90, Jn 6, Jr 1.5, Ja 2, Jw 1, SRF 2.5]; worst – 1.1 [RQD 75, Jn 9, Jr 1, Ja 2, Jw 0.6, SRF 2.5]. RMR (best/typical/worst – 77 (good rock)/ 67 (good rock) / 49 (fair rock). RMi (best/typical/worst 87 (very high) / 33 (very high) / 3 (high).
Faults (ii)	Descriptions and orientations as per Section 2.2. No major or moderate faults have been identified crossing the proposed sewage treatment works cavern site, although some lineaments, possibly representing minor faults, cross the proposed cavern site, particularly at the southern end. Possible materials in the minor fault zones include breccia and minor areas of fault gouge. Adjacent to the faults, the frequency of tectonic joints may increase markedly and the rock may be comminuted or very closely jointed with well-developed joints. The faults may also be partially (quartz lenses) or fully silicified and/or intruded by dykes. Unless replaced by secondary mineralisation the fault material will be 'weak'. Consequently, these features may result in increased overbreak, block falls, cave-in and water inflow. In addition, whilst more unlikely, the possibility of running ground, raveling and water inburst should be kept in mind. Depending on the nature of the material, fault zones can act as an aquiclude or an aquifer. Some types of mineralisation (e.g. sulphides) can cause problems with concrete. If the fault zone is silicified it may present some difficulties with tunneling, such as increased bit wear. Blocks along fault zones are likely to be small and crushed and may occur as clasts in a finer matrix. In any case block sizes will be smaller than the assessed range for unfaulted rock. Assessed parameters for consideration: GSI (10 – 30, assessed typical value - 20). Q, RMR and RMi can be assessed for faults, but it is recommended that these classification systems are used with great care in these circumstances and that it is better to assess faults in detail as individual features.
Sheeting joints (iii)	Orientations possibly as per Section 2.2, although orientations will vary with slope aspect, possibly towards the NE and E at the proposed cavern site. Unlikely to be encountered at the depth of the cavern (i.e. assumed >30m), or where encountered may be widely spaced and weakly developed (Hencher et al, 2010). For certain cavern depths, these may form surfaces to which block falls would fail back to, or sliding planes for sidewall blocks depending on cavern orientation. Sheeting joints may have a basic angle of friction of 40° (Hencher & Richards, 1982) to which a roughness angle of the surface *i* can be added (assumed to be 2° for initial assessment purposes), giving an effective friction angle of 42°. However, this could be much less if some sheeting joints are dilated, weathered and/or have significant kaolin or other infill. Joint Roughness Coefficient (JRC, Barton, 1973) values might range between JRC 10 and 20.
Steeply dipping to subvertical joints (iv)	Orientations as per Section 2.2. These primary joints will typically control the stability in the proposed cavern. It may be possible to optimise the cavern orientation with respect to the average orientation of the primary joint sets. In the absence of other information the angle of friction of these joints could be considered to be 40° (Hencher & Richards, 1982), although this could be much less where joints are slickensided or weathered. No surface roughness angle is applied due to lack of information and typical description as rough planar. JRC values might range between 5 and 10.

(Note: parameter values are conceptual estimates and are not intended for use in design.)

Table 2: Uncertainty register for structural geological features – conceptual engineering geological model stage

Uncertainty (related to structural geology)	Description and importance	Suggested actions
Location and properties of faults	Faults can be critical to the stability, safety and success of projects involving tunnels and caverns. Faults and related issues, such as significant water inflow, are the most likely 'unforeseen' (rather than unforeseeable) ground conditions to affect the proposed cavern site.	Carry out site specific API and LiDAR interpretation to identify all possible lineaments, carry out a site reconnaissance, followed by carefully planned and targeted field mapping and then GI, preferably involving several stages of investigation, with the observational model being constantly updated. Where encountered, faults should be described in detail using a scheme such as Chapter 5 of USBR (2001) combined with GeoGuide 3 terminology.
Properties of joints	Joints will control the most commonly encountered instability mechanisms underground in the vicinity of the proposed sewage treatment works cavern site.	The rock mass of the area should be split into engineering geological and structural domains, the joints divided into sets and if possible the formation mechanism of the joint sets should be established (sheeting, tectonic or cooling joints) as this will greatly assist with the description and characterisation of these joints (Hencher et al, 2010). Full description of joint sets should be carried out in accordance with GeoGuide 3. This will require several phases of mapping and GI (including the detailed discontinuity logging of orientated core and comparison with field mapping and televiewer data) along with field testing and laboratory testing. Mapping and orientated core should be the basis of the joint characterisation, not televiewer results. Note that the joint patterns may be different in the Shui Chuen O Granite and the Sha Tin Granite.
Optimum cavern location and orientation	There is an opportunity at the earliest stages of a cavern project (i.e. when the conceptual engineering geological model is developed) for engineering geology to have a significant input to the selection of the orientation of the caverns, potentially saving much cost and time with regards to rock support. However, later in the project this opportunity is typically not available as the cavern orientation will have been set by a myriad of other factors.	If possible the proposed sewage treatment works cavern site should not be located in an area crossed by faults or lineaments (particularly major or moderate faults), and it appears that this aspect has already been taken into consideration with the selection of the proposed site. The information on the primary joint trend could also be used to make an initial estimate of the best cavern orientations to reduce block fall and overbreak. However, this assessment is exceedingly coarse given the information available and much more data and analysis is required, although enough information is available to make initial considerations. There are many other factors that would need to be considered such as *in-situ* stress (approximately 108°-288° +/- 28° in Hong Kong, Free et al (2000)), access points, mucking out, operational requirements of the cavern etc.

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Engineering Geological Considerations for Computer Analyses for Tunnel and Cavern Stability Assessment

A.D. Mackay
Nishimatsu Construction Co. Ltd
N.R. Wightman
Snowy Mountain Engineering Corporation (SMEC) Asia

ABSTRACT

With the continual advancement of modern analytical software for subsurface excavation support, and the current trend to utilize underground space for a range of amenities, appropriate software application for the stability assessment is vital. Of particular importance is an appreciation of the variability of the ground and how this can be suitably represented through ground modeling for analysis. To achieve this, difficulties need to be overcome by effectively communicating findings from the site representatives to the design team, ensuring relevant details are incorporated into an updated analysis. Often the site team may provide a large quantity of data, which cannot be analysed efficiently; as a result skilled judgement is needed to identify information of most relevance to the stability. This paper provides an overview of the trends in sub-surface space development in the HK SAR and the relevance to suitable numerical analysis and software required for the excavation support and stability. Concerns are raised on the over-reliance of software for the design without the due consideration for the feedback and update from site data as it is gained. Examples of stability assessments from projects in the HK SAR and overseas, highlighting the relevance of important geological features, are provided.

1 INTRODUCTION

There have always been difficulties ensuring that the design analysis for sub-surface excavations account for relevant ground conditions that govern stability. Despite major advances in the analytical software, such as the use of 3 dimensional discrete element analyses capable of modeling and analysing rock block conditions, difficulties modeling and analyzing ground conditions representative to those encountered during excavation still remain. To this end it is vital that knowledge gained from past excavations is disseminated to ensure the appropriate excavation assessment is adopted. This paper summarises recent trends in the use of underground space in the Hong Kong SAR, the software applicable for the analyses, with examples provided from past projects both in Hong Kong and overseas. A particular emphasis is given to obtaining high quality field data, its appropriate usage in the excavation design analysis and its continual update as the design and construction continues.

2 MODERN TRENDS IN SUBSURFACE EXCAVATION

In 1988 the Hong Kong SAR government initiated a review of the potential uses of underground space (SPUN). These include underground space for Container Port back up facilities, oil and gas storage, sewage treatment facilities, refuse station, ware-housing, commercial government and institution and commercial spaces. The study concluded that the use of underground caverns is a viable alternative to construction above ground and would provide a significant environmental benefit Practice Notes for Authorised Persons, PNAP 177 (BA, 1995).

More recently the Hong Kong SAR government promoted initiatives to facilitate the development of underground space, in particular the enhanced use of rock caverns promoting sustainable development, in a Policy Agenda in 2009 to 2010. Following this the Geotechnical Engineering Office (GEO) of the Civil Engineering Development Department (CEDD) completed a study during March 2011 promoting the

implementation of rock cavern usage for the particular developments such as commercial, industrial, government and community institutions and public utilities.

In addition to these initiatives tunnel projects to accommodate rail, drainage schemes and highway infrastructure on behalf of the Mass Transit Railway corporation Limited (MTR), and the Hong Kong SAR government Drainage Services Department (DSD) and Highways Department (HyD) have commenced or about to commence. Refer to Table 1 for a project summary and Figure 1 for the rail and road locations:

Table 1: Summary of On-going Projects, adjusted from Thomas (2008) and Mackay (2009a)

Project	Construction Period	Project Summary
MTR West Island Line	2009 – 2014	Extension from Sheung Wan Station to Kennedy Town. Intermediate stations at Sai Ying Pun, University of HK and Kennedy town
MTR South Island Line (Central HK)	2011 – 2015	Extension from Admiralty Station to South Horizons, Ap Lei Chau. Intermediate stations at Lei Tung, Wong Chuk Hang and Ocean Park.
MTR South Island Line (West HK)	2014 – 2018	Extension from Kennedy Town to Ap Lei Chau. Intermediate stations at Wah Fu, Cyber-port and Aberdeen.
MTR Sha tin Central Link	2012 – 2019	Extension from Tai Wai to Central Station. Intermediate stations at Hung Hom, Diamond Hill, Kai Tak, To Kwa Wan, Ma Tau Wai, Ho Man tin and Wan Chai.
MTR Kwun Tong	2011 – 2015	Yau Ma Tei Station to Whampoa via the Shatin Central Link Ho Man Tin station.
MTR Express Rail Link	2011 - 2020	26km is in tunnel. The line will directly connect to the mainland China rail network.
HK West Drainage Tunnel	2008 – 2013	Tai Hang to Cyberport. The inside diameter ranges from 6.25 to 7.25m and has 8km of adits connecting to 32 dropshafts up to 170m depth , Tam (2012).
DSD Tsuen Wan	2009 – 2014	Tsuen Wan to Yau Kom Tau. The inside diameter is 6.5m with 3 connecting adits.
DSD Lai Chi Kok Transfer	2009 - 2011	Reservoir transfer tunnel between Kowloon Bye wash to Lower Shing Mun Reservoirs. 3m internal diameter.
DSD Lai Chi Kok Drainage	2010 - 2011	6 to 3m internal diameter with 6 connecting adits and dropshafts.
DSD Harbour Area Transfer Scheme	2009 - 2014	Depths up to 160m below sea level, internal diameter of 3m running along the north and west coast of Hk island from North Point to Ap Lei chau.
HyD Chek Lap Kok to Tuen Mun	2011 - 2016	Tunnel construction beneath the Urmstorm road-shipping channel.
HyD Central Kowloon route	2010 - 2016	Dual 3 lane carriageway running from Yau Ma Tei to Kai Tak
HyD Central – Wan Chai Bypass	2011 - 2016	Dual 3 lane carriageway running from Central to North Point



Figure 1: Location of the ongoing rail and road projects

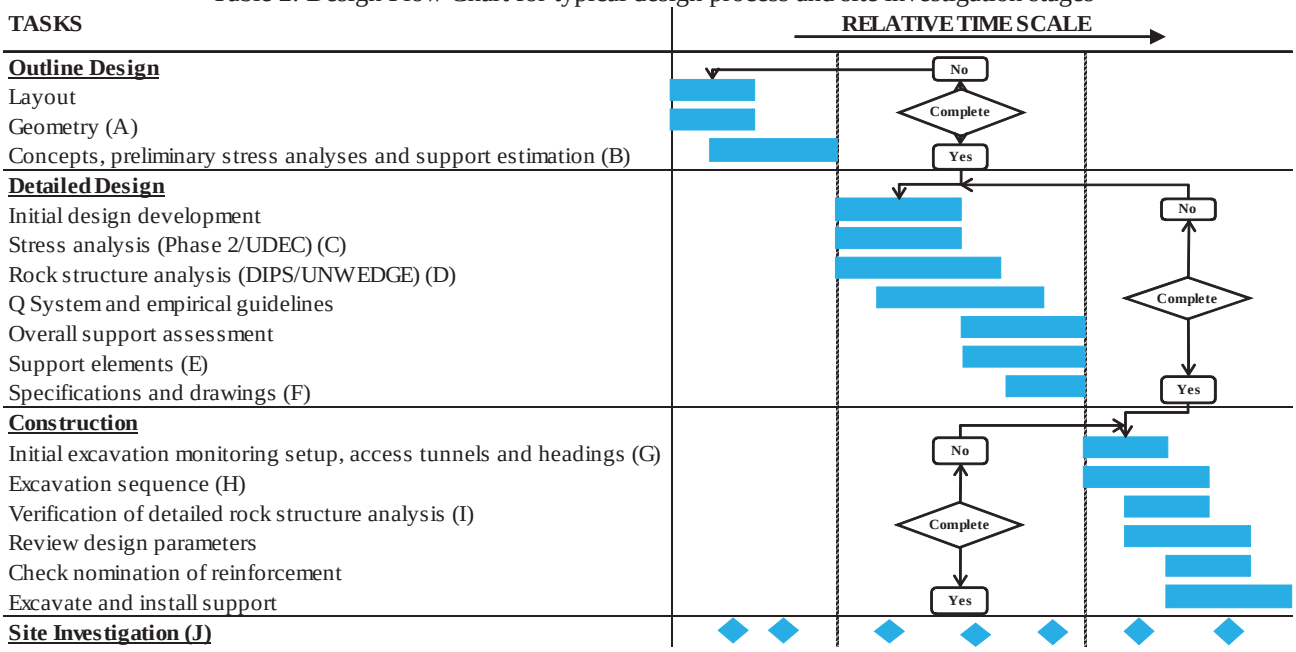
In addition, trends in sub-surface space acquisition taking place overseas include examples such as the construction of caverns to accommodate desalination plants and Hydro-Electric Power Schemes.

As a result a considerable on-going underground space acquisition, it is imperative that the knowledge gained is disseminated to other practitioners to ensure applicable design input and / or analyses are adopted.

3 PLANNING AND TRENDS TO DESIGN ANALYSIS

A typical design process for the development of underground space, in particular rock cavern development, is presented in Table 2 below, adapted from Swannell (1999):

Table 2: Design Flow Chart for typical design process and site investigation stages



Note:

(A) Includes space proofing, alignment, cavern spacing and arch profile

(B) Rock mass classification assessment, stress change estimate, compression zones to be maintained, design methodology;

(C) Stresses around excavation, possibility of intact rock failure, theoretical displacements, support requirements

(D) Failure mode identification, support required for stabilizing blocks

(E) Durability, styles and sizes, cost implications and installation time

(F) Rock support, shotcrete and installation

(G) Confirmation of the construction method, minimum support level, analytical methods and site Quality Assurance procedures. Demonstration of the significant rock structure and monitoring requirements

(H) check temporary conditions and programme

(I) Identification of potential local wedge failures, using UNWEDGE, and amendment of additional support requirements

(J) Desk study, including aerial photographic interpretation and past projects of a similar nature; site reconnaissance; ground investigation (sub-surface exploration), laboratory testing; rock nmass classification, detailed geological mapping, monitoring and ground update.

The site investigation that needs to be carried out in conjunction with the above design, following standard practice outlined in international standards such as British standard (BS) 5930 (1999) and GEO (2009). To ensure the design is updated to include relevant data as the design and construction progresses, stages need to be identified to ensure the ground models and analyses are updated at appropriate stages, see Table 2.

When planning the underground space for tunnel alignments, geological features, particularly geological anomalies, need consideration. These may include potentially unfavorable orientation of the proposed alignment with respect to the major planes of weakness, enabling large wedge failures in the roof and / or walls. Were possible cavern alignment normal to the strike of the major discontinuities is preferable. An example of this consideration is the Hong Kong West Drainage Tunnel (HKWDT), constructed by the

Dragages – Nishimatsu Joint Venture and presently approaching breakthrough, Tam (2012). A major criteria, used to optimize the excavation through the major fault zones intercepting the alignment, was to either avoid these zones or if unfeasible, to approach them perpendicular to their strike, Tam (2012). Refer to Table 1 for the HKWDT project summary and Figure 2 below for the alignment location relative to the major faults.

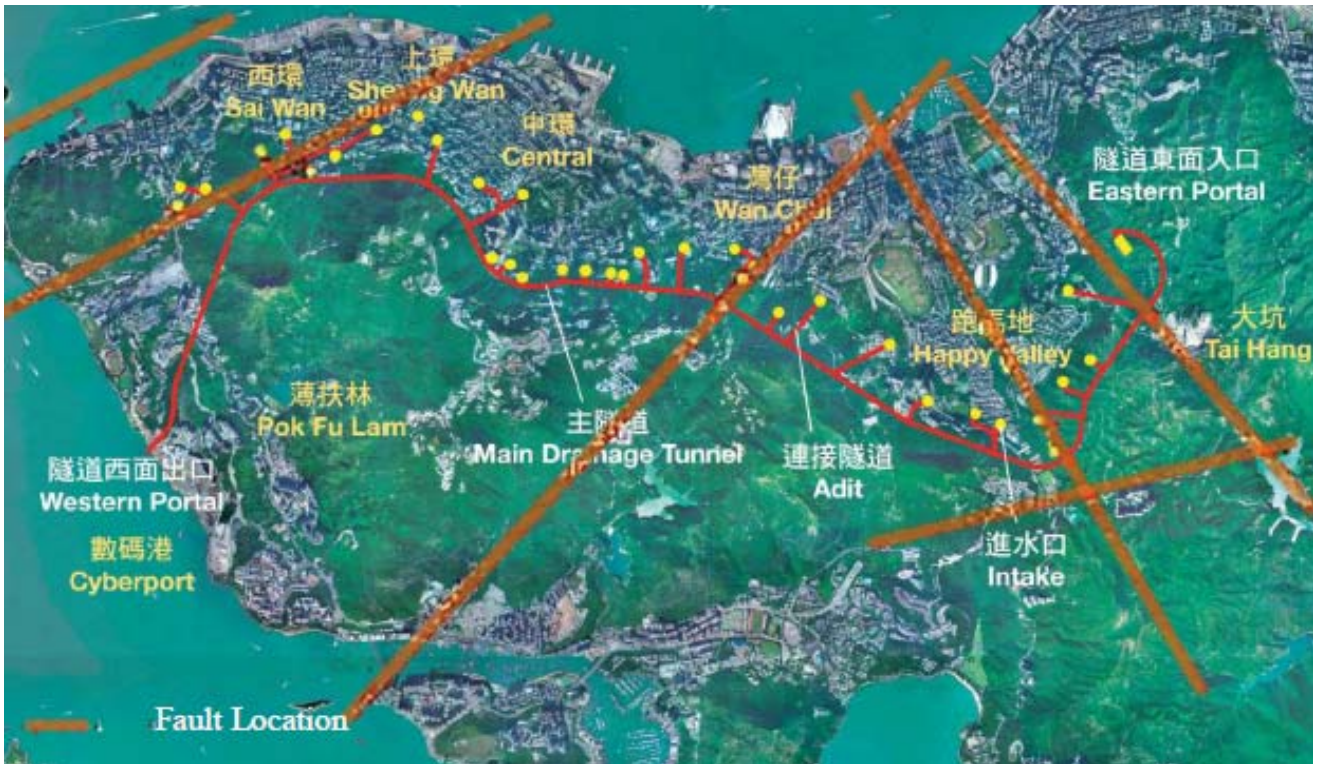


Figure 2: Alignment of the HKWDT, HK Island, Tam (2012)

4 SOFTWARE TYPES AND USAGE FOR SUB-SURFACE EXCAVATION SUPPORT

Analytical and numerical modeling is a typical design requirement to verify support requirements determined using empirical techniques, typically using Rock Mass Classification, such as the Norwegian Geotechnical Institution (NGI) Q Index value, Barton (1989) and the Rock Mass Rating system, Bieniawski (1976). In particular the numerical analyses used for sub-surface excavation can be divided into limit equilibrium, numerical continuous and numerical discontinuous categories, GEO (1992), as summarized in Table 3 below:

Table 3: Summary of Software Applications for Analysis (Swannel et al, 1999)

Type	Software	Supplier	Use
Limit equilibrium	UNWEDGE	Rocscience Inc.,	Requires a definition of kinematically feasible rock wedge release around an excavation opening, which can be interpreted using the software DIPS.
Numerical continuum	PLAXIS	Rocscience Inc.,	Finite difference software to analyse stresses / displacements. Discrete discontinuities may be included in the analysis
	PHASE 2	Rocscience Inc.,	2D finite element software for support in un-jointed / heavily jointed rock
Numerical discontinuum	UDEC / 3DEC	Itasca Consulting Group incorporated	For jointed rock masses / 3D analysis

4.1 Limit equilibrium

This provides a stability analysis for discrete blocks and wedges. Suitable software is UNWEDGE which identifies tetrahedral wedges which are free to fall, slide or rotate out of roofs and / or walls into the excavation. The input requires suitable assessment and judgment of the discontinuity set orientation retrieved from the site investigation and parameters to input to the program. The statistical assessment of the discontinuity data can be carried out using the software "DIPS". UNWEDGE includes the excavation cross excavation section and plunge; the rock unit weight; the orientation of the discontinuity sets and the hydrostatic pressure. For the hydrostatic pressure this is typically assumed to be free-draining through the discontinuities and the lining support.

4.2 Numerical continuum analyses

This is defined as finite element and boundary element analyses and are based on semi-empirical input to define the rock mass strength parameters, Hoek (1980). These parameters can be linked to rock mass classification schemes such as the Q system, Hoek (1995) which allows designers to readily update the finite element software from field data. Generalised constants for common rock types allowing estimates for peak rock mass strength (m, s); residual (post-yield) rock mass strength (mres, sres) and rock mass modulus and Poisons Ratio are available from Hoek (1995). These parameters assume a plastic or brittle plastic failure criterion. The basis for residual parameters requires judgement on the reduction in rock quality from effects such as blast damage. Dilation parameters also need consideration as this can have a significant effect on the convergence estimates, Swannell (1999).

Suitable software for finite element and boundary element analyses is PLAXIS and PHASE 2. An example of the application of the PLAXIS was used for the Hong Kong University Centennial Campus Cavern support design. The caverns were formed to provide space for salt water reservoirs located on a leveled platform at Pok Fu Lum, Figure 3, which freed space to allow the extension of the Hong Kong centennial Campus to be carried out west of the existing Hong Kong University Campus. The works were highlighted as an example of sustainable development associated with the provision of underground space in the recent study of enhanced use of rock caverns by the GEO completed March 2011.



Figure 3: Location of the Hong Kong University Salt Water Reservoir Caverns (Mackay, 2008 & 2010)

The cavern excavation comprised an arched tunnel of 7.8 m internal span connecting to 2 transition tunnels leading to 2 caverns, of 15 m internal span and 50m length. The geology comprised coarse ash crystal tuff with localized fine ash tuff, eutaxite, hornfels and sandstone inclusions. Due to the influence of localized intrusions and faults, the rock was metamorphism to varying degrees. Furthermore the bedding and foliation,

led to pronounced strength anisotropy, with the weak zones aligned parallel to the dominant fault orientation, trending north east to south west, with pronounced foliation (Figure 4). The physical properties for the strength anisotropy and other physical characteristics were considered when preparing the ground model and tunnel stability analysis.



Figure 4: Rock core showing pronounced foliation orientation in rock, Mackay (2008)

Due to the low rock head cover above the cavern crown, aligned parallel to the overlying natural terrain, weaker rock, described as highly to moderately decomposed tuff (H/MDT), was located immediately above the cavern crown (Figure 7). The ground model, based on extensive site investigation data, including rock face mapping as the excavation progressed, was analysed using PLAXIS as presented in Figure 5.

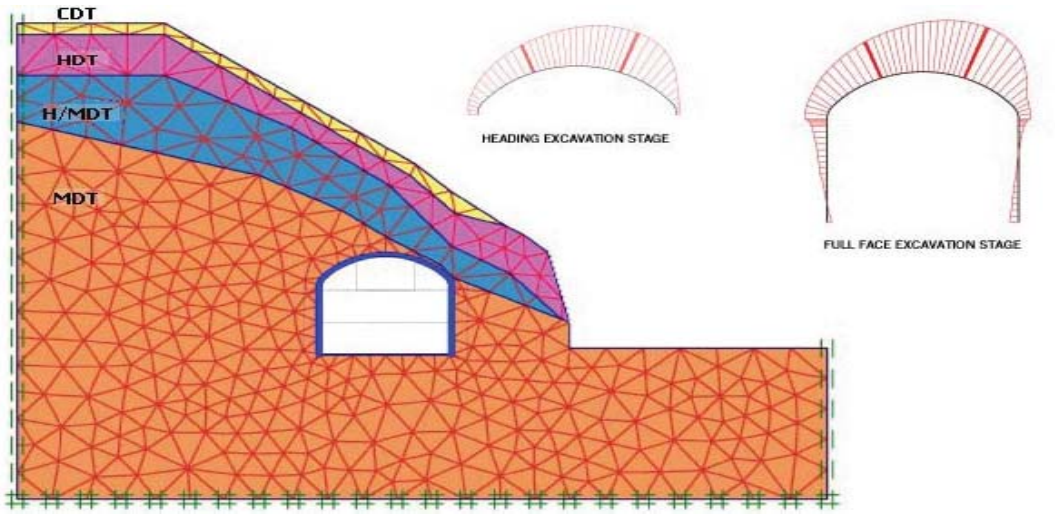


Figure 5: Findings of the analyses for the Salt Water Cavern (Mackay, 2008)

4.3 Numerical discontinuum analyses

This method models the ground as discrete blocks, separated into deformable zones, by joints. The software UDEC and 3DEC, which analyses the ground in 2 and 3 dimensions (2D and 3D) respectively, are suitable software for geological structure analysis with discontinuities modeled in sets with variable persistence, openness and permeability. A limitation is that the blocks are impermeable which may not represent the secondary permeability through these rock blocks.

An example of the application of the 3DEC input was for the cavern stability analysis for the Adelaide Desalination Plant. The Adelaide Desalination Plant was formed due to a water supply shortage from the Murray River to Adelaide (Mackay, 2010). The plant was located at Port Stanvac between Adelaide (Figure 6) and the coastline. Due to the precipitous coastline at Port Stanvac the plant for salt water retrieval, desalination processing and discharge, was accommodated within a cavern of dimensions 60 m by 15 m by 25 m; the main components of the Desalination Plant are presented in Figure 7.



Figure 6: Adelaide Desalination Plant location, South Australia

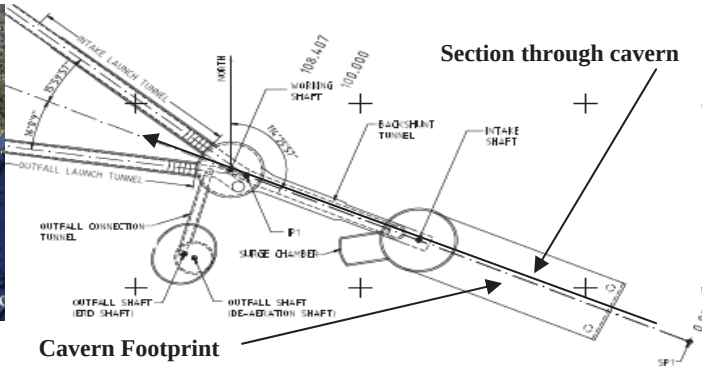


Figure 7: Adelaide Desalination configuration

The geology at the cavern location comprised the Brachima Formation, which exhibited variable physical properties influenced by the dominant north east to south west trending faults, the sub-vertical bedding planes trending parallel to the faults and the differential weathering penetrating to variable depths along the bedding planes. An important feature identified during the site investigation was the presence of shear zones extending to the cavern invert, which were weak relative to the surrounding rock, and were also aligned parallel to the bedding. The main shear zones are presented in Figure 8. The shear zones would allow large scale instability to occur if present in combination with other subordinate discontinuity sets. Each shear zone was therefore incorporated into the ground model for analysis using 3DEC, refer to Figure 9. The support requirements were assessed accordingly.

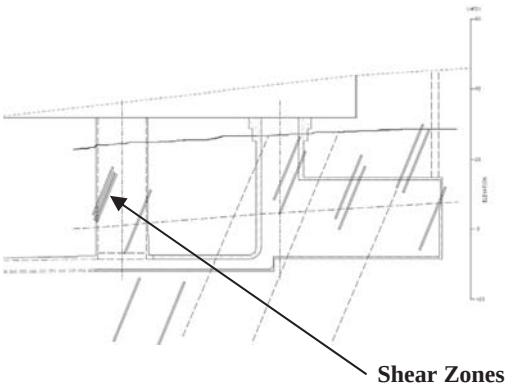


Figure 8: Cavern section presenting shear zones

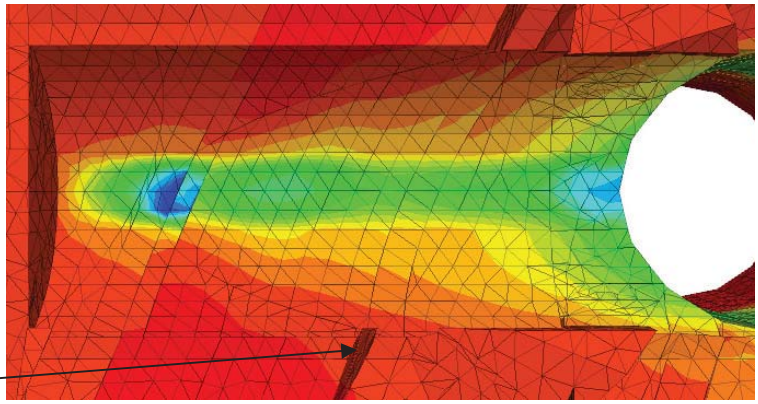


Figure 9: 3DEC analysis including shear zones

5 CONCLUSIONS

There is presently a demand for underground space internationally and in the Hong Kong SAR in particular. In the Hong Kong SAR initiatives for the use of underground space are being carried out by the GEO and ambitious tunneling infrastructure projects are being carried out by the MTRCL, HyD and the DSD. It is therefore of prime importance to ensure the stability of tunnels and caverns is assessed based on high quality site investigation data, including geological mapping during construction. To achieve these ambitions the experience gained locally through the on-going projects needs to be considered through the design and construction phases of the forthcoming projects to ensure effective measures are taken to provide support. The software development has enabled more complicated ground conditions to be modeled and analysed and necessary support procured. Notwithstanding this is only beneficial provided that the most important aspects of the ground controlling the stability, reviewed by experienced practicioners, are considered in the analysis.

ACKNOWLEDGEMENTS

The authors would like to thank Mr Bivek Gurung for his contribution to preparing this paper. The views expressed in this paper are those of the authors and not of any other parties.

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High Pressure Grouting for Groundwater Ingress Control in Rock Tunnels and Caverns

K.F. Garshol & J.K.W. Tam
AECOM Asia Co. Ltd., Hong Kong
H.K.M. Chau & K.C.K. Lau
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

In underground hard rock construction, ground treatment is normally limited to installation of support to provide stable ground and safe working conditions. In addition, groundwater ingress control is often necessary to prevent surface settlement and damage, or environmental impact to vegetation and groundwater resources. Worldwide there are many projects that demonstrate the potentially serious consequences of inadequate groundwater ingress control during underground construction. Therefore, for many projects it is necessary to implement groundwater control as an integral part of the underground construction process.

Pre-Excavation Grouting (PEG) offers effective restriction of groundwater ingress in advance of the excavation. PEG ground treatment can provide "dry" underground openings and as a side effect also improved ground stability. Modern PEG includes high-pressure injection of non-bleeding stable grout with low viscosity and mostly fixed water-cement ratio. Furthermore, suitable Microfine Cement and Colloidal Silica injected through proven grouting equipment have to be ensured. The maximum grouting pressure should be in the range 50 to 100 bar.

1 INTRODUCTION

In underground hard rock construction, ground treatment is normally limited to installation of support to provide stable ground and safe working conditions. In addition, groundwater ingress control is often necessary to prevent surface settlement and damage, or environmental impact to vegetation and groundwater resources. Worldwide there are many examples of serious consequences of inadequate groundwater ingress control during underground construction. Therefore, for many projects it is necessary to implement groundwater control as an integral part of the underground construction process. It should be noted that the ingress control measures installed as part of the final lining (typically sheet membrane), mostly will become effective far too late to prevent surface settlement and damage.

This paper describes the most important elements of high-pressure Pre-Excavation Grouting (PEG) necessary for the purpose of achieving targeted maximum residual groundwater ingress into tunnels and caverns in hard rock. Examples from relevant projects in Hong Kong and elsewhere are presented.

1.1 Reasons for the increased use of high pressure grouting

In the past 20 years, high-pressure grouting (PEG) ahead of the face in tunnels or caverns has become an important technique in modern underground construction. Garshol (2007a) provided some reasons for this:

- Limits on permitted ground water drainage into underground space are now frequently imposed by the authorities for environmental protection reasons or to avoid settlement above the underground space. Settlement may cause damage to infrastructure like buildings, roads, drainage pipes, supply lines, cables and ducts.
- The risk of major water inrush, or of unexpectedly running into extremely poor ground, can be virtually eliminated (due to systematic probe drilling ahead of the face being an integral part of PEG). It should be noted that if the excavation hits lots of water this would have to be sealed by post-grouting. This

process is not only time consuming and expensive, but is also far less effective than PEG. In difficult situations it can be close to impossible to successfully solve the problem.

- Poor and unstable ground ahead of the face can be substantially improved and stabilized before exposing it by excavation. This improves the face area stable stand-up-time, thus reducing the risk of uncontrolled collapse.
- Risk of pollution from tunnels transporting sewage, or other hazardous materials, can be avoided or limited. Ground treated by pre-injection becomes less permeable and such hazardous materials cannot freely egress from the tunnel.
- Sprayed concrete linings are increasingly being installed as the final and permanent lining in tunnels. The savings potential in construction cost and time is substantial, this being the main reasons for the increased interest and use. Such linings are difficult to install with satisfactory quality under wet (running water) conditions and ground water ingress control by pre-grouting can solve the problem.
- With modern drilling jumbos even very hard rock can be penetrated at a rate of 2.5 to 3.0 m/min. Therefore, the cost of probe drilling to guard against sudden catastrophic water inflows is now much lower than it used to be. A number of projects have experienced such catastrophic cases, typically being stopped for months and probe holes offer an inexpensive insurance.

2 PEG METHOD FOR UNDERGROUND CONSTRUCTION

PEG offers effective restriction of groundwater ingress in advance of the excavation resulting in “dry” underground openings and as a side effect also improved ground stability. For this to work out as planned the project must use the latest grouting technology and avoid shortcuts. This includes high-pressure injection of non-bleeding stable grout with low viscosity and mostly fixed water-cement ratio. Furthermore, suitable Microfine Cement (MC) and Colloidal Silica (CS) must be used. Proven grouting equipment is equally important. Two main properties that define a suitable MC are early set and high final strength. High pressure injection means that existing cracks and joints in the rock mass will dilate and allow grout penetration where MC would otherwise not permeate. Maximum grouting pressure from 50 to 100 bar is normal.

Where MC cannot penetrate sufficiently to satisfy very strict residual ingress limits, CS offers an excellent supplement. Even though CS is a suspension of particles, it behaves practically as a true liquid and will permeate the ground almost like water. The volcanic tuff in Hong Kong has typically more closely spaced joint sets than the granitic rock and CS injection is often required following MC injection. High rock conductivity contrast is dealt with by using MC first and CS next and by dual stop criteria. The dual stop criteria approach limits the grout material consumption and prevents unnecessary spread, while still achieving sufficient grout penetration and distribution. High-pressure grouting requires careful consideration of safety. Besides proper dimensioning of couplings and pressure lines, the packers or standpipes installed in the grout holes must be secured against blow-out.

The aim of PEG is to seal off joints and fissures in the rock mass by providing grout screens along the tunnel or cavern, which can stop or reduce water ingress during excavation. Figure 1 shows a typical illustration of systematic grout screens with overlap around an underground space. Note that of course the screen is also covering the invert.

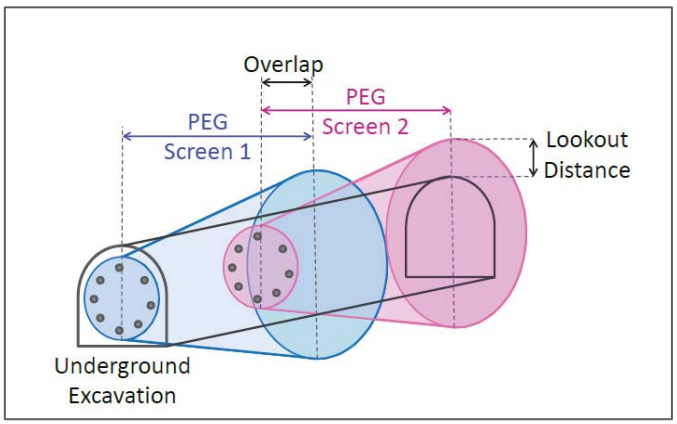


Figure 1: Typical systematic grout screens with overlap around an underground space

3 SPECIAL ISSUES OF PEG

3.1 Use of stable grout

Cement grout can only permeate into cracks and joints by applied pump pressure. If the grout is not pressure stable, the water in the grout will easily be squeezed out of the grout, leaving a dry plug behind and further grout penetration will stop. This process is particularly negative when the grout reaches narrow joints and channels in the ground.

Cement grout with high bleed has also typically very poor pressure stability and this is one reason why stable grouts perform better. However, pressure stability needs to be checked by measuring the pressure filtration coefficient (K_{pf}) according to the American Petroleum Institute recommended Practice 13. Good pressure stability would give $K_{pf} < 0.1$.

3.2 Maximum grout pumping pressure

The maximum allowed injection pressure is commonly discussed from two different viewpoints:

- The low-pressure approach where the focus is on not creating damage in the rock structure around the tunnel or anywhere in the surroundings of the project. It is normally linked to the use of cement and Bentonite and very high w/c-ratio (typically > 3.0). This requires grout-to-refusal technique to counteract the negative effects of the unstable and bleeding grout by squeezing out surplus water.
- The high-pressure approach where the focus is on getting the job done efficiently both regarding time, economy and quality of result. It is typically executed with stable, non-bleeding grout and individual boreholes are stopped either on specified maximum pressure or a maximum quantity, whichever is reached first. By limiting quantity per hole the potential lifting force created by pressurized grout is also limited and any damage is typically not done.

Grouting in real life is executed to control ground water flow and/or to improve stability of the rock formation before excavating into it. Both these motives for grouting exist because of cracks, joints, channels, low friction joint materials, clay, crushed shear zone material etc. and sometimes pretty high hydrostatic ground water head (e.g. > 20 bar). It should be quite easy to agree that the purpose of pre-injection in such cases can only be satisfied if the grout can be placed into those openings and discontinuities by the use of sufficient pumping pressure.

The maximum pressure specified for pumping of the grout is normally given as a net value in addition to the local hydrostatic head. However, when starting injection on a hole, there has normally been a lot of drainage from the drilling process before any packers can be installed, so the practical GW head will mostly be substantially lower than the original virgin ground water head.

The maximum injection pressure has to be evaluated on a running basis and especially it has to be checked against local conditions in the tunnel. Very poor rock conditions in the face area, high hydrostatic water head and existing backflow will be indicators that maximum pressure must be limited, even if the rock cover is hundreds of meters. Otherwise, 50 to 100 bar works very well.

3.3 Accelerators for MC or CS injection

There are situations where accelerated setting can be necessary. This will typically be in post-grouting cases for backflow cut-off, but also in pre-injection, backflow may happen through the face. If for any reason the grout is pumped into running water, or pressure or channel sizes are extreme, accelerated grout may become necessary. A non-return valve is needed for use with a dosage pump when adding accelerator to cement grout through a separate hose connected at the packer. When pumping accelerated CS, 2-component pumping should be considered rather than working with batches. Furthermore, 2-component PU can be used to block concentrated water leakage at the face.

3.4 Use of packers for high pressure injection

When a hole has been drilled into the rock formation for the purpose of injecting grout at high pressure, a tight connection (seal) between the pumping hose and the borehole is needed. The normal way of achieving this is to insert a packer and two typical types of packers include:

- Re-usable mechanical packers available in different standard lengths (pipe and expander assembly), typically from 1.0 m to 5.0 m in steps of 0.5 m. For very deep packer placements, it is normal to use connectors to join standard pipe lengths of e.g. 3.0 m length. At the outer end of the packer pipe it is normal to fit a ball valve or similar. When injection is completed, the ball valve can be closed and the pump hose disconnected. The valve must remain closed with the packer in place until the grout has set sufficiently to keep the ground water pressure without backflow. The packer may then be removed and cleaned for re-use in a different hole. If removed too late, packers will need to be discarded due to set cement.
- Disposable packers have the same working principle as the re-usable packers, but they are constructed so that when expanded, the expansion is automatically locked in place to allow removal of the inner- and outer pipes used to place the packer and expand it. The packer itself has a one-way valve to keep pressurized grout in place without backflow when releasing the pump pressure and removing the insertion pipes. It is possible to keep the non-return valve open to be able to detect connections from other boreholes being injected or to measure bore hole water ingress.

3.5 Use of standpipe or bag-packer techniques in unstable ground

In poor ground condition, packer placement can be very difficult and borehole stability may also be a problem. When fractured ground conditions are combined with high water ingress at high hydrostatic head, the combination may lead to loss of face stability and progressive collapse. In such cases, shallow packer placement must be avoided, because high water pressure will attack very close to the face conveyed through the drilled probe- or injection hole. Installation of standpipe or bag-packer may be adopted to mitigate such problems:

- Standpipes (Figure 2) are installed by drilling with an over-size drill bit of e.g. 76 mm diameter to a depth of say 3 to 4 m and inserting a steel pipe of suitable diameter (i.d. > 55 mm, o.d. < 66 mm) into this hole. The pipe must be grouted in place using a high quality shrinkage compensated cement grout. This is easy to do by placing a packer close to the inner end of the pipe and by pumping the grout into the annular space between pipe and rock, until it appears at the borehole collar.
- Bag-packer (Figure 3) is a quick and efficient alternative technique when grouting of the normal standpipe is difficult because of ground water encountered in the drilled oversize hole.



Figure 2: Standpipe



Figure 3: Bag-packer

4 EXAMPLES FROM RELEVANT PROJECTS

4.1 Harbour Area Treatment Scheme Stage 2A (HATS 2A), Hong Kong

The HATS 2A Project includes construction of 20 km deep seated tunnels and thirteen vertical shafts. The tunnel alignment runs at about 70 m and 160 m below sea level and mainly underneath urban areas with long sections located subsea. The HATS 2A tunnels with long sections located underneath settlement sensitive built-up reclaimed land, requires strict residual inflow limits. To achieve this, PEG is the only practical solution to the problem. The hard rock fissure grouting is executed by normal grout permeation, but is also greatly enhanced by pressure-widening of existing fissures. This use of high grouting pressure (up to 80 bar) greatly improves the grout penetration and sealing effect.

MC is the primary grouting material, supplemented by CS where the cement cannot penetrate and further sealing off is required. Standpipe technique and quick foaming polyurethane have been used to block running water through cracks and joints in the face and to avoid backflow of grout materials in locally highly fractured rock. Accelerator added at the packer when grouting with MC or CS, is also highly efficient for solving such problems.

The two main rock types that have been encountered in the tunnel excavation are volcanic tuff and plutonic granite. In volcanic tuff that typically has more closely spaced joint sets than the granitic rock, CS injection is often required after MC injection. To reduce the effect of high conductivity contrast, the HATS 2A Project has adopted dual stop criteria on pressure or volume. This approach limits the grout material consumption, while still achieving sufficient grout penetration and distribution.

4.2 Holen Hydropower Project, Øyestøl, Norway

At Holen hydropower project, access Øyestøl (52 m^2), the recorded ground water static head was up to 50 bar. Such pressure may cause quite dramatic effects in the tunnel. When drilling into water bearing zones, frequently water, sand and fines would punch through the drill rod all the way back into the drilling machine. Water supply hoses of normal quality on the drilling machine would blow. When withdrawing the drill rod, the water jet out of a 51-mm diameter hole would easily reach 25 m back from the face. The water yield from a contact at 10 m depth would typically be 2 to 3 m^3/min. A measurement made on a 45 mm diameter hole being 4.5 m long gave 4 to 5 m^3/min.

When high pressure ground water is expected, the drill jumbo must be equipped with hydraulic clamps for securing the drill string during coupling and de-coupling of rods. A last resort at extreme pressure, without such equipment, is to drive the drill jumbo from the face, until the drill string is free of the hole. Also, if necessary, when drilling more holes into the zone, drill all holes to almost full depth. Then couple the last one or two rods by moving the drill jumbo to «motor» the rods in and out.

When conditions allow, it is beneficial to drill a number of holes into contact with the water-bearing zone. The pressure will then normally drop somewhat due to drainage, making it easier to place packers in the holes. Such conditions will significantly benefit from fast-set and high strength grout since drilling of new holes too early may cause a rupture and flushing out of the injected material.

To place packers against static head of 50 bar, adaptations on the drill jumbo have to be made. The drill feeder and drill rod guides must allow handling of the packers by the hydraulic system. Even with such a solution, it is quite complicated to enter the borehole, due to the produced water spray and resulting lack of visibility.

4.3 Oset drinking water treatment plant, Oslo, Norway

Oset Drinking Water cleaning plant is situated in Maridalen, Oslo. The Plant is built in hard rock with 2 caverns (100,000 m^3) and a 500 m long tunnel. Total excavation amounts to 140,000 m^3 of rock. The treatment plant is designed for 390,000 m^3 water/day, and will deliver drinking water to about 500,000 people.

The plant is located in syenite rock of good quality. Average Q-value is 40, but also there were weak zones with Q-value < 1. The allowable water ingress was set at 100 L/min for the whole plant. During construction, water ingress was measured at up to 200 L/min in some of the probe holes.

The whole project, tunnel and caverns were systematically pre-injected with MC and supplemented with rapid hardening OPC. For some zones accelerated grout (MC + alkali free accelerator) was used. Length of

the injection holes was 21 m, with a hole spacing between 1.5 – 2 m. After the injection, 3 rounds of advance were made, making the injection overlap about 6 meters. Injection was carried out with a modern computerized injection rig. The rig had integrated accelerator dosage pump for injection of accelerated grout.

The materials consumption amounted to 1,510 tons of MC, 820 tons OPC and 38 tons of accelerated MC grout used for blocking backflow or limit materials spread. The final result is amazingly good. The total ingress into the whole plant (tunnel and caverns), is only 20 L / min (the requirement being 100 L/min).

5 CONCLUSIONS

High ground water static head, high ground water ingress, project access through shaft or decline, strict limitations on residual water ingress and other possible problem-enhancing features, require that the following set of measures must be considered for dealing with ground water issues:

- Probe drilling ahead of the face on a routine basis must be executed. The amount of pre-grouting must be balanced against the project specific consequences of not achieving the required target ingress rate.
- The reserve pump capacity must be at least 100 % more than the maximum expected water inrush.
- Back-up diesel generators are required to ensure supply of electricity to the dewatering pump system.
- It is a requirement that the grouting equipment has sufficient capacity regarding flow and pressure and the ability to pump particle size up to 5 mm.
- Post grouting is difficult and time consuming and may become impossible.
- Pre grouting, on the other hand, is simple and efficient, provided that a tight face area is maintained. A 5 m tight buffer zone is recommended in sound rock. In weak and poor rock more may be required.
- High static head requires care and special measures. Do not allow high-pressure water too close to the face, particularly in poor ground. It does not help to have 10 m of buffer zone if the packers are placed only 2 m into the borehole and face collapse could result.
- Finally: Keep the face area watertight and never blast the next round if in doubt.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Director of Drainage Services Department, the Government of the Hong Kong Special Administrative Region and AECOM Asia Company Limited for permission to publish this paper.

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Management & Mitigation of Groundwater within Deep Shaft Excavations – the HATS 2A Project Experience

A. Indelicato
Gammon Construction Limited, Hong Kong

ABSTRACT

Water infiltration into underground excavation sited in rock is a common challenge wherever shafts and tunnels are realized below the groundwater level. The Harbour Area Treatment Scheme (or HATS 2A), includes in its second stage, the excavation of deep conveyance tunnels under Contract DC/2007/23 from North Point, via Sai Ying Pun to Stonecutters Island totaling over 12 kilometers in length and up to 165 meters below sea level. Their alignments locate very close to highly urbanized areas along the northern shoreline of Hong Kong Island. In order to manage and mitigate the groundwater inflow into the tunnels, a series of methods have been adopted and applied. Deep diaphragm walls are used to protect the shaft during excavation through soft ground above the level of rock head. On commencement of rock excavation, pre- and (if necessary) post-excavation grouting is used to exclude groundwater by sealing discontinuities around the excavated area. To test the effectiveness of this work, water measurements are taken both inside the shaft and around the adjacent area via different methods. All those activities are considered fundamental to secure the entire working area not only for the project personnel but also for the public. The scope of this article is to describe those activities and the problematic encountered during the first part of the project.

1 INTRODUCTION

Hong Kong is situated on the southern coast of China at the mouth of the Pearl River on the southeastern coast of Guangdong Province. With more than 7 million inhabitants, Hong Kong can be considered one of the most densely populated cities in South East Asia. High population results in a considerable amount of sewage output with potentially high environmental issues, especially regarding the quality of the water in the Victoria Harbour. For this reason a new sewage conveyance system of deep tunnels was chosen to suit the local urban environment.

In 2001 the HATS Stage 1 (see Plate 1) was completed, which included 23.6 kilometers of deep tunnels with a capacity to collect and treat 1.7 million cubic meters a day of sewage produced around the Harbour (Tai et al, 2009).

HATS Stage 2A represents the next phase of the regional program to further improve the water quality. It is designed to collect the remaining sewage produced by the city and transfer it to the Stonecutter's Island Sewage Treatment Plant. The depth of the tunnel (which reaches 165 m below sea level), has been chosen to avoid underground utilities and any type of conflicts or disruptions with underground mass railway routes and harbour crossings, it will also avoid social disruption in an area very densely populated. The location and the depth of the tunnel of HATS 2 have posed few challenges especially linked to the groundwater inflow.

The risk of groundwater infiltration into the underground excavations is addressed with a series of actions to manage and mitigate such. The scope of this paper is to illustrate these methods, and provides some examples which occurred during the excavation of the deep shafts and the action undertaken to mitigate them.



Plate 1: Tunnel layout under Contract DC/2007/23

2 MAIN TECHNIQUES USED DURING SHAFT EXCAVATION

2.1 Diaphragm wall

Diaphragm walls (see Figure 1) are normally used during excavation of soft ground to prevent the collapse of soil and the water infiltration. They must withstand the high bending moments caused by the combined earth and hydrostatic pressures (Hajnal et al, 1984). Diaphragm walls serve as the primary structural elements for supporting excavations. Often they also become part of permanent structures. They have the structural advantage of being stiff which is beneficial in urban infrastructure projects requiring strict specifications with respect to ground movements generated by excavations (SEI/ASCE, 2000). For the HATS 2A project 15 liters/min per 100 m of shaft length has been used as the target ingress limit for the pre-excavation grouting design.

Sophisticated emplacement techniques, such as slurry trench techniques (Hunt, 2005), are used to install the diaphragm wall through the permeable superficial deposit material. This is a more expensive technique than sheet piling but may provide a much more effective barrier to groundwater movement as the excavation is progressing (Price, 1985).

Any deterioration or absence of concrete causing exposure of steel rebar exposes a risk of deterioration through presence of sulphuric salts in the water flow. Chemical agents normally aggressive to concrete include CO_2, Chlorides, Magnesium, Sulphate and Ammonia (Hunt, 2005). pH tests quantify the acidity of the water to identify such and mitigation can be achieved by spraying a layer of shotcrete over the area in question and continuing a regime of regular inspection. In the HATS 2A project a few damp areas were observed within diaphragm wall typically along the panel construction joints (see Plate 2). These left damp areas on the diaphragm wall caisson surfaces which were further quantified to be within acceptable criteria.

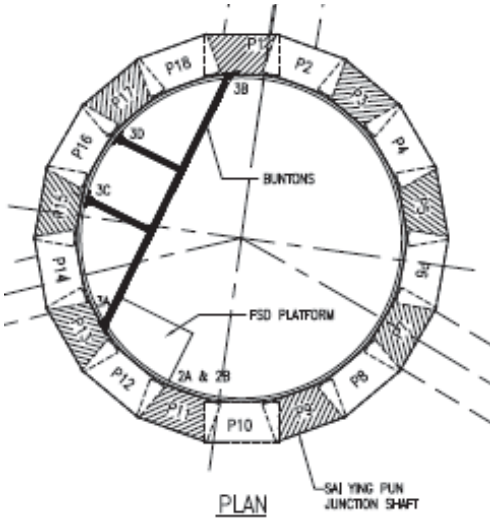


Figure 1: Plan of one of the HATS 2A diaphragm wall shaft section



Plate 2: Damp areas on the deep diaphragm wall

2.2 Probing and pre-excavation grouting

During the excavation of the shafts and tunnels of the HATS 2A project, probe and grouting holes are drilled as part of the Pre-excavation Grouting (PEG) process ahead of the face. This work is repeated every 20 m of advancement into the rock. These typically have a length of 25.2 m and have an inclination which is typically 8 degrees to the vertical (see Figure 2). The first holes to be drilled are probe holes which are located in four quadrants of the shaft bottom. Probe drilling is a technique employed to verify the predictions of geological conditions ahead of the face prior to excavation.

The purpose of this drilling is to detect potential geological or hydrogeological hazards such as zones of high permeability, broken weathered or faulted strata or clayey-sandy infillings (Voirin et al, 1996). During the drilling observation and recording by the geologist provides information on the type of chippings, water color, presence of soft and clay-rich zones and groundwater leakages.

After the probe holes are completed, a hydraulic inflatable packer is installed at 2-3 m from the hole collars and the water inflow rate is measured by means of stopwatch and calibrated container collecting water via a flexible hose which is connected to the packer shut-off valve. The average inflow is calculated over a time period of several minutes and care is taken that the water has reached a steady flow rate. In certain circumstances a staged series of water flow measurements may be undertaken at different levels within the probe or control holes to ascertain more precisely the location of the water bearing sections.

Subsequent to the probe holes, a series of grouting holes may be drilled around the entire circumference of the shaft bottom to provide a “grout fan” which, once pressure-grouted will seal water-bearing discontinuities within the rock mass around the shaft wall so as to minimize possible water infiltrations.

PEG is typically carried out with microfine cements (MFC) and/or colloidal silica (CS). The material selection is based on an inflow criteria measured from the probe or PEG holes, which, in turn is linked to permeability of the rock mass to be treated (Hunt, 2005). Grouting operations are normally is influenced by several factors such as knowledge of the in-situ rock mass and its properties, ground water and its movement and the observed takes of grouting material during the process of injection of the grout fan. Criteria for injection of MFC is typically based on water inflow of > 3 L/min, and colloidal silica for inflow normally < 3 L/min. Cementitous grouts, sometimes referred to as ‘particulate grouts’ (as they consist essentially of particles suspended in water) are typically restricted to a limit of ~0.2 mm to the size of pore or fissure into which they can be injected, because of the natural filtering action of the rock (Price, 1985). Mix design of the MFC grout was typically standardized within the shafts with dosage of super-plasticisers and hydration control additives optimizing gelling times during the high pressure grouting and enabling long-delivery distances in excess of 100 m to be achieved.

Colloidal silica also employed for grouting on the HATS 2A project is pure mineral grout. The gelling process takes place by a physical reaction between particles of silica (SiO_2). Both the grout and accelerator for

this new generation product are very user and environmentally friendly, minimizing health risks due to chemical reactivity or toxicity during the injection works.

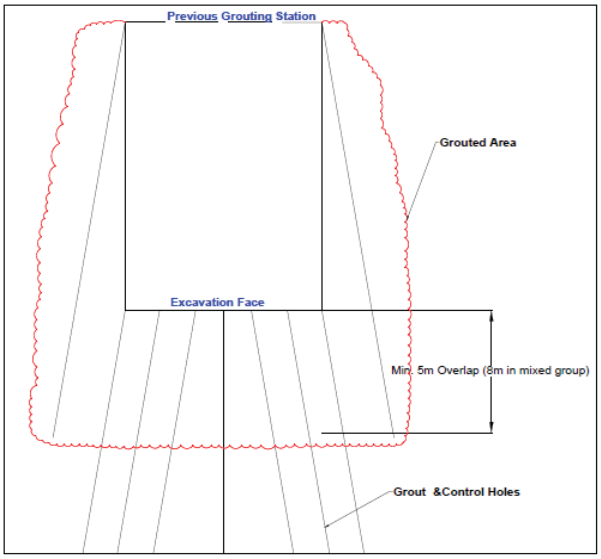


Figure 2: Section showing location of grouting fan

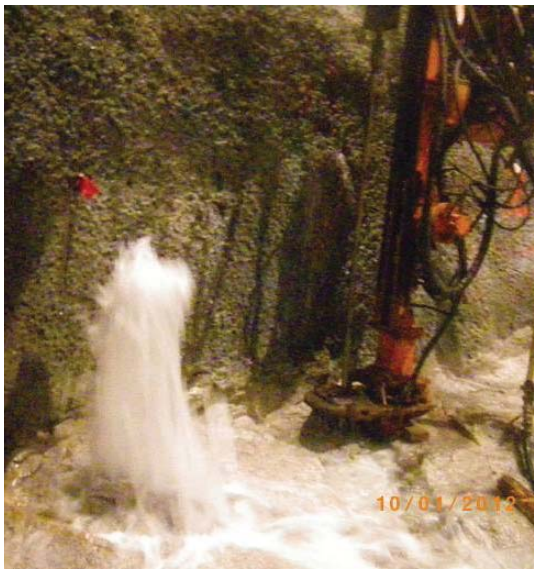


Plate 3: Water flow exceeding 50 L/min during probe drilling

The effectiveness of each PEG round is quantified by drilling control holes after every sequence of grouting/closure sequence and further mitigation to meet the target inflow criteria (where necessary), was achieved by secondary or tertiary injection operations.

Occasionally, grouting will achieve a complete cut-off but it is generally difficult to achieve, especially in rock fractures or where conditions of groundwater flow persist (Hunt, 2005).

A vital point in grouting is the injection pressure. Care is required in rock masses to obtain pressure greater than water pressure but less than that required to cause new fractures (Hunt, 2005). High pressure pre-injection of microcement at 5MPa to 10MPa excess pressure, will generally cause local opening of joints and probably local shear and dilation on inclined joint sets (Barton 2004).

One example of interception water during drilling 18 meters below the collar position occurred in one of the HATS 2A shaft where water interception and inflow measurement highlighted inflows > 80 L/min (see Plate 3). By stopping the drilling upon interception of the water and insertion of the hydraulic inflatable packer into the hole, water inflows were managed. Subsequent grout holes exhibited high takes and three successive stages of grouting were necessary to meet the inflow criteria in this case. The holes of these stages can either be drilled in a regular pattern or based on targeted locations (Goodfellow, 2011). In our case the secondary and tertiary grouting holes were drilled adjacent to the primary ones. Extra tertiary holes were also drilled in targeted locations to reduce further the water inflow.

2.3 *Large-scale water inflow measurements*

The water inflow calculated from the probe holes is not the only measurement of ground water taken during the excavation works at the HATS 2A project. To test the effectiveness of the diaphragm wall and the pre-excavation grouting, water ingress measurements are carried out frequently. Two methods used within the HATS project which are “the Kibble method” and “the Pumping method”, both have been used to calculate the amount of water inflow inside the shaft.

The ‘Kibble method’ uses a sealed bucket (kibble) to collect water from the shaft bottom (see Plate 4). When the water pump is switched off for long periods (e.g. at weekends) water accumulates within the bottom of the shaft. This water is subsequently pumped into the kibble and them carried out on the surface where it is measured the total volume and consequently the inflow/min/100 m inside the shaft.

If the in-situ water inflow was to exceed the design criteria, post-grouting works would be initiated in any areas of the shaft wall where water infiltration occurs.

Another system tested is the 'pumping method'. This method is carried out on the sinking stage inside the shaft where a container of known volume is used for this measurement (see Plate 5).

Basically the water pump is moved inside the container and it is calculated the time the pump employed to fill the container with 100 liters. This method has the advantage that can be carried out without switching off and on the water pump which may otherwise slow down the production cycle. Care has to be taken that any water used during grouting and drilling operations is closed off during any in-situ water ingress measurements.



Plate 4: Kibble method ingress testing



Plate 5: Pumping method test on the sinking stage

Another water inflow measurement system which has been developed employs the use of a catchment pipe forming a ring structure around the shaft walls which is embedded into the shotcrete layer. Any water leaking from the surface of the shaft profile can be collected within this structure and subsequently quantified (see Figure 3).

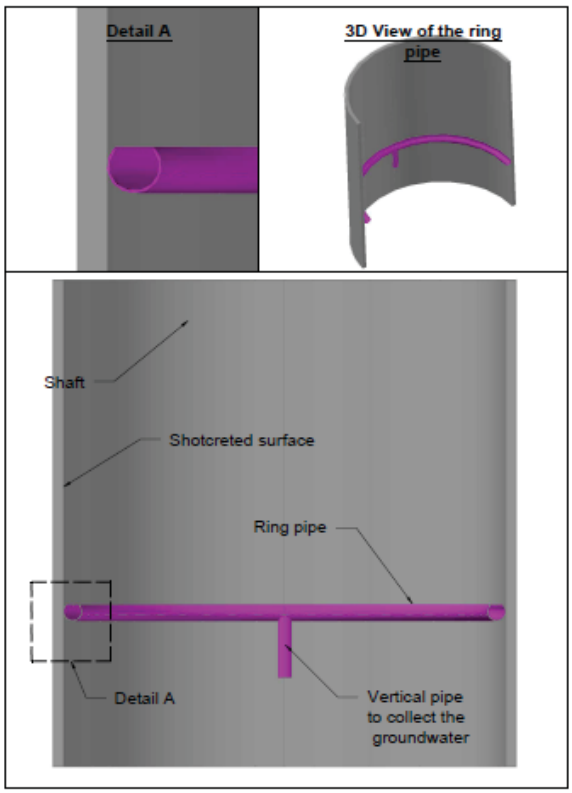


Figure 3: Pipe channel used for inflow collection and measurement

2.4 Post grouting

If the water inflow inside the shaft fails to reach the design criteria with the PEG, additional grouting is required. Sub-horizontal holes drilled into the walls of the shaft to envelope locations intersected by obvious water infiltration and subsequently grouted accordingly at lower injection pressures than the PEG. Further water measurements are subsequently obtained until the desired criteria are achieved. During the excavation of the shaft under the HATS 2A project, the water infiltration exceeded the desired limit. In this case post grouting works were carried out to reduce the water infiltration.

2.5 Piezometers and settlement markers

The impact of groundwater inflow management not only affects the stability of the underground workings, it also ultimately plays an important role in the potential effect of settlement of surface infrastructure around any excavation areas. As mentioned earlier, the HATS 2A project is developed along highly populated areas, risks of subsidence have to be addressed and mitigated properly via the water inflow management. Careful monitoring utilizing an extensive series of settlement and piezometric points which are regularly monitored identify any variations in level that may be caused by tunnelling works (see Figure 4).

The piezometers are used to monitor changes in piezometric head which indicate changes in the groundwater conditions (Hunt, 2005). Their great advantage is that they are small in scale and relatively cheap and easy to execute providing in the same time useful site information (Hiscock, 2005).

For the HATS 2A, three types of piezometers are employed: Manual, vibrating wire and Wireless Automatic Ground Water Monitoring Device (WAGMD). Hydraulic head is checked every 30 minutes and all information stored on a web-based geotechnical database system especially developed to manage and report geotechnical instrumentations and investigations data acquired on large construction or site investigation projects. This system also stores the information regarding the settlement markers levels.

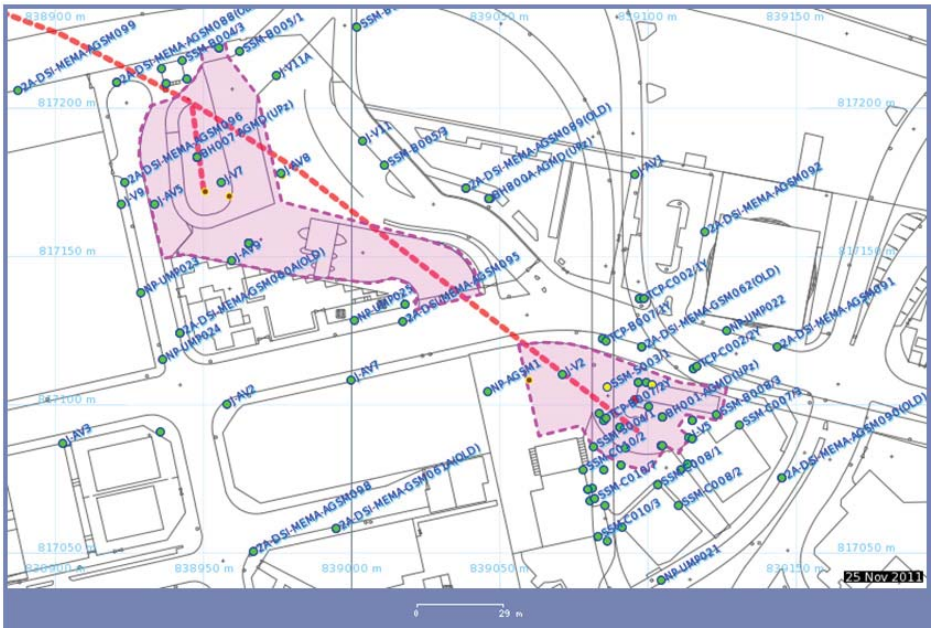


Figure 4: Location of settlement markers and piezometers around the construction site

Three trigger levels (Alert, Action and Alarm) activate every time one of the piezometers or settlement markers exceeds these agreed levels and automated notification is sent through email or SMS messaging to responsible personnel. Subsequent investigation work takes place to analyze the data and to identify the key cause(s) of the trigger and propose mitigation against further deterioration.

3 CONCLUSIONS

The scope of activities described in this paper encompasses a "holistic" approach adopted to monitor, manage and maintain ground water inflow into the underground excavations on HATS 2A, Contract DC/2007/23. A key challenge to achieve groundwater inflows to within acceptable criteria within the shafts excavations on the project have been carefully managed in this way.

As excavation proceeds, new challenges will occur and continued implementation of these methods will be necessary in order to maintain the ground water infiltration within the limits.

ACKNOWLEDGEMENTS

The author wishes to thank his works colleagues for their input, support and review. A special thank you also to all parties includes the Drainage Services Department, the Government of the Hong Kong Special Administrative Region, AECOM Asia Limited and Gammon Construction Limited for their kind permission to publish this paper.

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Artificial Ground Freezing for TBM Break-through – Design Considerations

R.K.Y. Leung & K.K.Y. Ko
Hyder Consulting Ltd., Hong Kong

H.B. Hu
China State – Shanghai Tunnel Joint Venture, Hong Kong

A.K.K. Cheung
Ove Arup & Partners Hong Kong Ltd., Hong Kong

W.L. Chan
Drainage Services Department, HKSAR Government

ABSTRACT

Artificial ground freezing (AGF) has been widely adopted in Shanghai, China, as the ground improvement method for break-through of tunnel boring machines (TBM) from their launching shafts and into their receiving shafts. In Hong Kong, AGF application has in the past been limited to the construction of mined adits and cross-passages between tunnel bores. In one of the construction contracts under the Harbour Area Treatment Scheme (HATS) Stage 2A, the contractor has initiated to adopt AGF for the first time in Hong Kong using brine for TBM break-through. Under the contract, a tunnel about 4m in diameter and 250m in length is to be constructed inside Stonecutters Island Sewage Treatment Works at 30m below ground in marine deposits, alluvium and decomposed granite by TBM to connect a new pumping station to the existing pumping station. This paper presents the design considerations for the application of AGF using brine for TBM break-through. It details the thermal and stress analyses required to confirm the viability of the construction method, and the laboratory testing required for derivation of the necessary thermal and geotechnical parameters of the soils.

1 INTRODUCTION

Contract No. DC/2009/05 is one of the HATS Stage 2A contracts being implemented by the Government of the Hong Kong Special Administrative Region to improve the water quality in the Victoria Harbour. This construction contract was awarded by the Drainage Services Department to a joint venture of China State Construction Engineering (Hong Kong) Limited and Shanghai Tunnel Engineering Company Limited (CSSTJV). Works under this contract consist of construction of an interconnection tunnel, and of a diaphragm-walled cofferdam for the main pumping station at Stonecutters Island Sewage Treatment Works. Hyder Consulting Ltd. was appointed by CSSTJV to carry out detailed design for the construction of the 4m diameter interconnection tunnel which comprises Part A Tunnel 236m in length excavated by TBM, and Part B Tunnel 14m in length excavated by hand-mining. AGF was employed as the ground improvement method to facilitate TBM break-through from the launching shaft. Figure 1 shows the site layout.

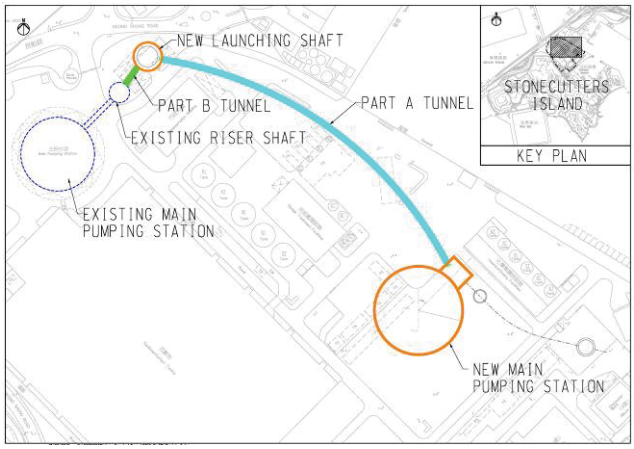


Figure 1: Site location plan

This paper presents the detailed design, covering both the thermal and stress analyses, and the required

laboratory testing for the application of AGF using brine for TBM break-through. Thermal analysis was carried out to estimate the freezing energy and time needed to achieve a frozen zone down to a designated temperature. As some of the vertical freezing pipes had to be lifted prior to TBM break-through, an assessment was also made of the temperature change with time after those pipes were removed. Stress analysis was carried out to confirm stability of the soil mass with the soft-eye cut out in the diaphragm-wall of the launching shaft. Assessment of the effects of frost heave and thaw consolidation with the help of numerical modeling is also discussed.

2 SITE GEOLOGY

The site is formed by reclamation with a ground level of about +5.5 mPD. The interconnection tunnel is situated at approximately 30 m below ground level and the encountered geology at TBM break-through from the launching shaft includes Marine Deposits and Alluvium, as shown in Figure 2 and described below:

Marine Deposits - firm to stiff, slightly sandy silty CLAY with occasional angular to subangular fine gravel sized rock and shell fragments;

Alluvium - medium dense to dense, clayey silty fine to coarse SAND or stiff to very stiff, sandy silty CLAY, with some subangular to subrounded fine to medium gravel sized rock fragments.

The design groundwater level is at 2 m below existing ground level.

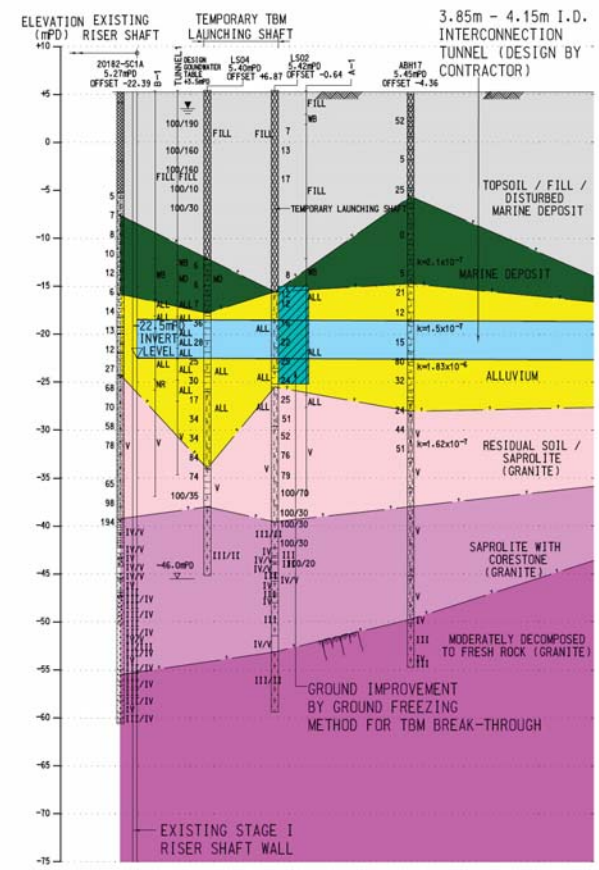


Figure 2: Geological profile

3 DESIGN CONSIDERATION

General design considerations for AGF are as follows:

Groundwater Level – Soil with sufficient moisture content is a pre-requisite of AGF. In this project, with the lowest groundwater level at +1.0 mPD and tunnel crown level at -18.0 mPD, the tunnel is completely submerged and hence the soil around it is saturated;

Soil Material – AGF is considered generally effective in improving the strength of silty, clayey and sandy type of soil materials but less effective for bouldery soil though cut-off effect would still be achieved. The soil materials encountered at the locations where AGF was applied vary from clayey, silty to sandy;

Salinity – Salinity affects the freezing point of water and hence the saturated soil materials. It also affects the mechanical properties of the frozen soil. Laboratory testing has been carried out on frozen in situ soil samples to determine the thermal and mechanical properties of the soils;

Groundwater Flow – Groundwater flow affects the shape and time for formation of the frozen soil. In significant groundwater flow, more energy will be required to create the design frozen soil block. Groundwater flow has been monitored through the reading of piezometers in this case and found insignificant;

Frost Heave and Thaw Consolidation – With the existence of some sensitive building structures along the tunnel alignment, frost heave and thaw consolidation needed to be considered. In view of this, laboratory testing to determine frost heave and thaw consolidation ratios has been carried out for the heave and settlement assessment.

4 LABORATORY TESTING

Laboratory tests were performed on soil samples retrieved from the site to obtain the mechanical properties of frozen soils and thermal properties of soils in both the non-frozen and frozen states. The tests were carried out in accordance with the Chinese national code GB/T50123-1999 Standard for Soil Test Method (土工试验方

法) and Chinese code for coal mining industry MT/T593-1999 Testing for Physical and Mechanical Properties of Artificially Frozen Soil (人工冻土物理力学性能试验). Table 1 gives a summary of the tests performed.

Table 1: Summary of laboratory testing

Test	Details
Unconfined compression	2 tests each at -10ºC, -15ºC and -20ºC for each soil type
Creep	For each soil type, one test each for each combination of temperature (-10ºC, -15ºC, -20ºC) and stress level (0.3q, 0.4q, 0.5q, 0.7q)*
Freezing temperature	2 tests for each soil type
Frost heave	2 tests for each soil type
Thaw settlement	2 tests for each soil type
Thermal conductivity	One test each at 20ºC, 0ºC, -5 ºC and -20ºC for each soil type
Specific heat capacity	One test each at 20ºCand -10ºC for each soil type

*q is the peak axial stress or axial stress at axial strain of 20% if there is no peak.

The unconfined compression tests were carried out with specimens cut from Mazier soil samples. Compression was applied at a rate of 1% strain/min. until an axial strain of 20%. Figure 3 shows a typical stress-strain curve of the test. Whilst the test standard defines the strength as the peak stress or the stress corresponding to 20% strain if no peak is experienced, a more conservative approach of taking the stress at which the specimen began to yield as the strength was adopted in arriving at the design values.

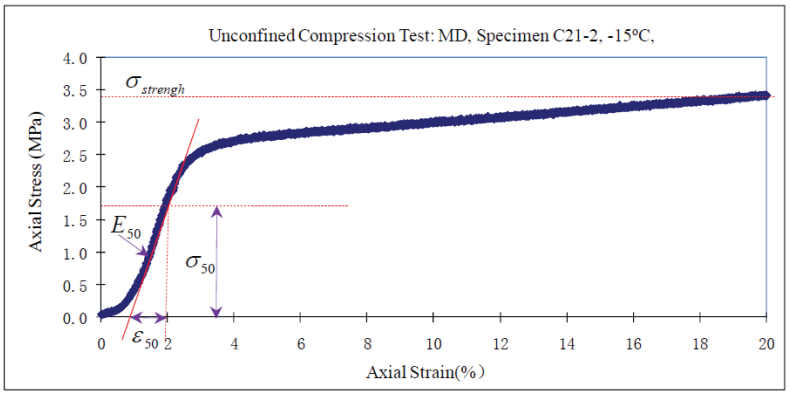
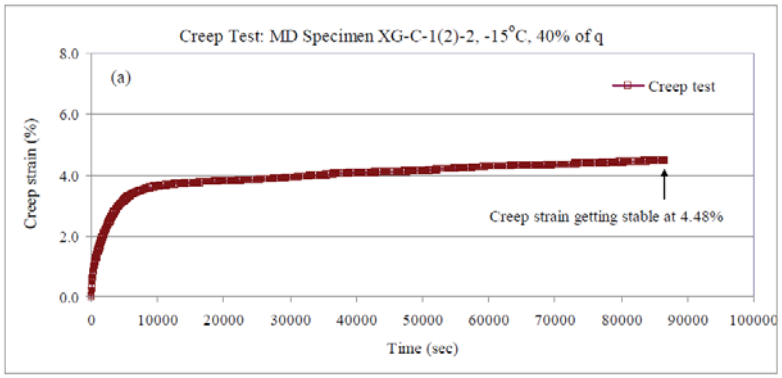


Figure 3: Typical stress-strain curve of unconfined compression test

In the creep test, compression was applied at a rate of 1% strain/min to the required stress level and then maintained for 24 hours. Figure 4 shows a typical plot of the creep strain and strain rate against time. The tests results showed that at a stress level of 0.5q or lower, the strain of the tested specimens consistently came to be stable with time. Hence, 50% the design compressive strength is taken as the design creep strength of the soils. The creep modulus corresponds to the ration of the applied stress to the strain when stable.



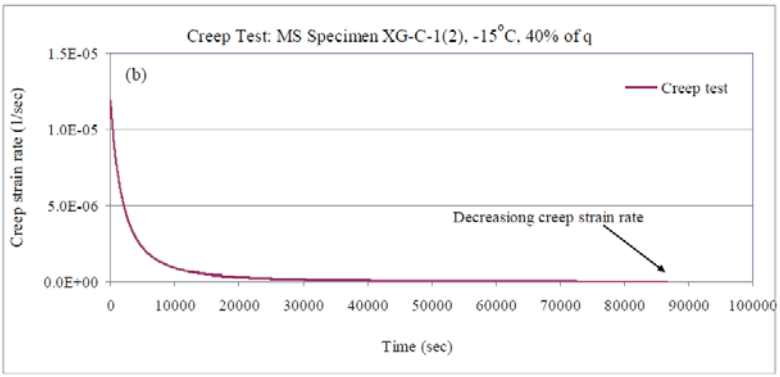


Figure 4: Typical strain-time & strain rate-time plots of creep test

In the tests for frost heave ratio, temperature of soil specimens were brought down to below their freezing temperature and the change in height of the specimens was measured. Likewise, in the test for thaw consolidation ratio, frozen soil specimens were allowed to thaw and the reduction in height of the specimens was measured.

Tables 2 and 3 summarize the values of the mechanical and thermal properties

adopted in the design.

Table 2: Design values of mechanical properties

Soil	UCS* (MPa)	Young's Modulus E_{50} (MPa)	Creep Strength (MPa)	Creep Modulus (MPa)	Frost Heave Ratio	Thaw Consolidation Ratio
Frozen Marine Deposits	2.0	160	1.0	21	1.18%	13.00%
Frozen Alluvium	3.5	315	1.75	36	5.10%	8.65%

* at temperature of -15˚C

Table 3: Design values of thermal properties

Soil	Freezing Temperature (˚C)	Specific heat capacity (kJ/kg ˚C)		Thermal Conductivity (W/mK)			
		In-Situ	Frozen	-20˚C	-5˚C	0˚C	20˚C
Marine Deposits	-1.71	2.14	1.28	1.966	1.628	1.485	1.367
Alluvium	-1.84	1.48	1.04	2.272	1.993	1.916	1.746

5 CONSTRUCTION ASPECTS AFFECTING THE DESIGN

Figure 5 shows the configuration of the ground freezing work for TBM launching. Key aspects affecting the design are discussed as follows.

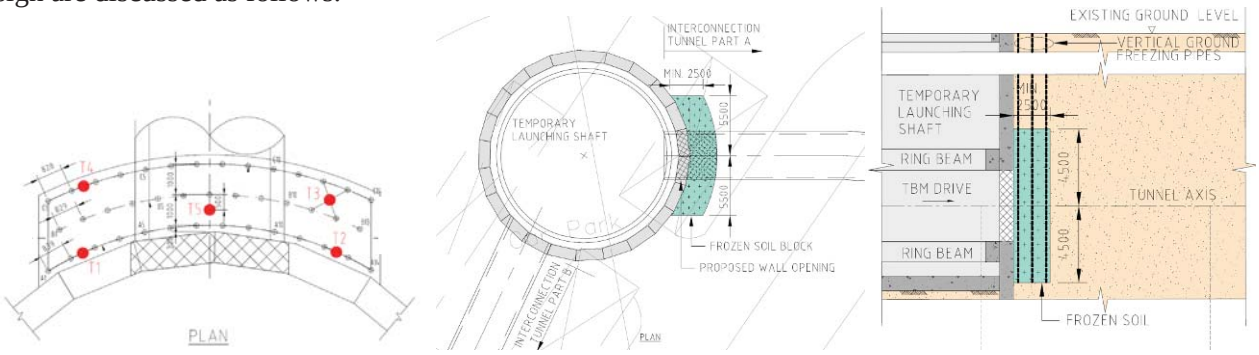


Figure 5: Configuration of the frozen soil wall for TBM launching

5.1 Use of brine as freezing agent

Two artificial ground freezing systems are available in the construction industry, using either liquid nitrogen or brine. In this project, a closed circuit freezing system using brine was adopted. Brine lowered to a temperature of -28°C using industrial refrigeration plant was circulated into freezing pipes installed in the ground in a regular pattern. The target was to form a 2.5 m thick frozen block with a temperature of -16°C in front of the TBM break-through.

5.2 Partial removal of diaphragm wall prior to TBM launching

It was intended to remove the part of the diaphragm wall of the launching shaft facing the TBM prior to TBM launching for facilitating the launching operation. The frozen soil block thus should have the capability of withstanding the soil and water pressure acting from the back of it.

5.3 Partial extraction of ground freezing pipes prior to TBM launching

Prior to TBM launching, the ground freezing pipes falling within the drive of the TBM had to be partially extracted to above the tunnel crown level in order not to obstruct with the TBM drive. After the pipe extraction, the frozen block had to remain frozen for at least 72 hours to allow the TBM launching operation.

6 THERMAL ANALYSIS

The finite element software ANSYS was employed for 2-D thermal analysis. This analysis helped determine the arrangement of the ground freezing pipes and estimate the time and energy required to form the frozen soil wall to the design temperature. The analysis algorithm is based on the heat balance equation from the principle of conservation of energy. Latent heat associated with phase change from non-frozen to frozen state was taken into consideration.

The input data for thermal analysis include geometry, thermal conductivity and enthalpy at different specific temperature, and boundary conditions. The initial temperature and temperature at boundary of the model were set at 25°C and the temperature at the locations of the freezing pipes was reduced progressively from 25°C to -28°C. According to the analysis results as illustrated in Figure 6, the frozen soil wall with temperature lower than -15°C could be achieved in 20 days of active freezing. The average heat transfer rate obtained from the analysis was 435 kJ/hour·m^2. The refrigeration system was required to have a heat exchange capacity of at least 435 kJ/hour·m^2 multiplied by total contact area and a factor of 1.3 to account for heat loss.

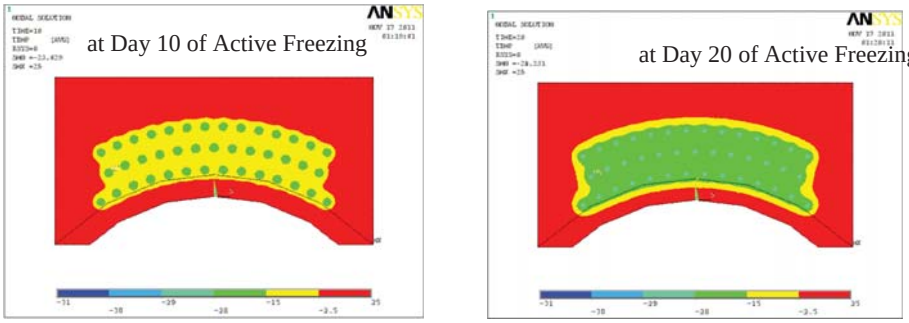


Figure 6: Temperature field of active freezing from ANSYS analysis

To simulate the effect of pipe extraction to the overall temperature of the frozen soil block, the action of circulating warm water to defrost the freezing pipe was modeled. Due to heat diffusion, the defrosted region was frozen again and returned to -15°C by the surrounding frozen soil as illustrated in below Figure 7.

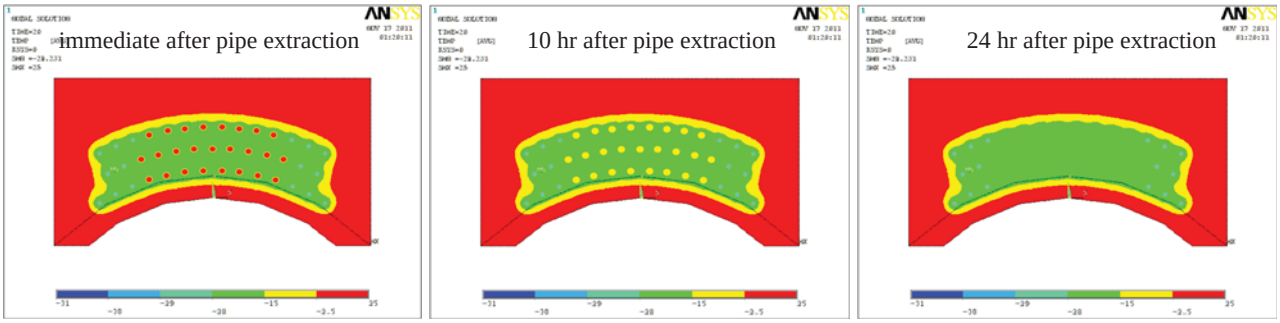


Figure 7: Temperature field of freezing pipe extraction from ANSYS analysis

7 STRESS ANALYSIS, FROST HEAVE AND THAW CONSOLIDATION CONSIDERATION

According to the TBM break-through sequence, a local 5 m diameter opening (soft eye zone) would be made on the diaphragm wall of the launching shaft. In advance, a 2.5 m thick frozen soil wall that had reached the design temperature and achieved the target supporting strength had to be formed immediately outside the opening. After the soft eye zone was broken off manually, the soil and hydrostatic pressure would be retained by the frozen soil wall spanning across the opening. Analysis was carried out using PLAXIS 2D. The frozen soil was modelled using Mohr Coulomb's failure criteria with undrained shear strength S_u taken as 0.5 x creep compressive strength. With a factor of safety of 2 applied, the undrained shear strength of frozen marine deposits and frozen alluvium were taken respectively as 250 kPa and 437.5 kPa. Two analysis cases using two different sets of Young's modulus had been carried out with one considering the short term loading effect using E_{50} and another one considering the long term loading effect using creep modulus. In view of the relative large frost heave and thaw consolidation ratios, the ground movement and associated impact to the

existing building structures including their foundations were also assessed using PLAXIS 2D. To consider the effect of frost heave, a positive volume strain was assigned to the frozen zone to model the expansion effect caused by formation of ice lenses. To mitigate excess settlement caused during the thawing process, permeation grouting through the grout ports pre-installed in the TBM tunnel lining was proposed. A negative volume strain was thus assigned to the frozen zone untreated by permeation grouting to simulate the thaw consolidation. Models for these analyses are shown in Figure 8.

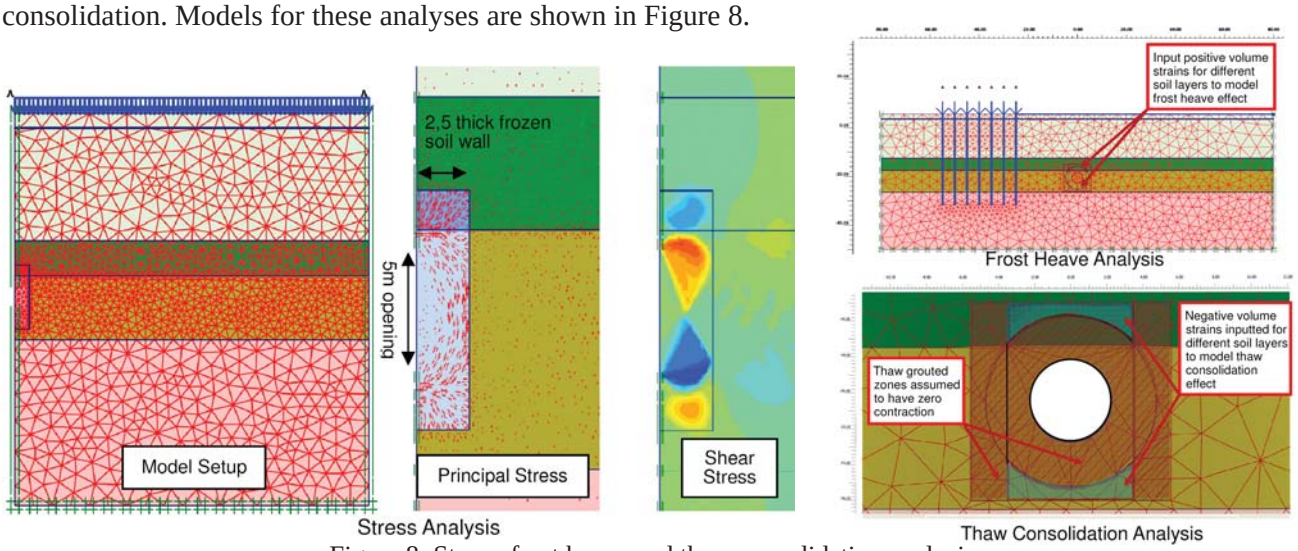


Figure 8: Stress, frost heave and thaw consolidation analysis

8 CONCLUSION

It was the first time in Hong Kong to employ AGF using brine as ground improvement method for TBM launching. The application in Contract No. DC/2009/05 has proven that AGF can be used to strengthen various types of soil provided that it is supported by careful planning, comprehensive laboratory testing and rigorous design. In the detailed design, thermal and mechanical properties of the soil including the frost heave and thaw consolidation were all considered. The TBM was launched successfully in January 2011 and this has set a benchmark of using AGF to improve the in-situ ground for TBM launching in Hong Kong.

ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Drainage Services Department, the Government of the Hong Kong Special Administrative Region.

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Storry, R.B., Kitzis, B., Martin, O., Harris, D. & Stenning, A. 2006. Ground freezing for cross passage construction beneath an environmentally sensitive area. *Proceedings of HKIE-GD 26th Annual Seminar*, 161-168.

Artificial Ground Freezing for TBM Break-through – Construction

L. Tsang & A. Cheung
Ove Arup & Partners Hong Kong Limited
C. Leung
China State – Shanghai Tunnel Joint Venture
W.L. Chan
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

This article presents a case of using Artificial Ground Freezing (AGF) as the soil improvement method in soft ground for Tunnel Boring Machine (TBM) launching break-through in the Harbour Area Treatment Scheme Stage 2A project. In Hong Kong, AGF technique is not a common soil improvement approach. There are only very few local cases and experiences on this technology. This article addresses the key construction considerations of using AGF for TBM launching. The performance of the frozen soil block is discussed. Difficulties encountered during the TBM launching operation and the solutions to the problems are also presented.

1 INTRODUCTION

The Harbour Area Treatment Scheme 2A comprises a 3.9 m internal diameter Interconnection Tunnel to connect the existing and the proposed pumping stations. The tunnel consists of two parts, with Part A (236 m long) to be built by an Earth Pressure Balanced type TBM and Part B (14 m long) to be built by hand mined method. The TBM was launched from the new Launching Shaft near the existing pumping station to the Inlet Chamber of the new pumping station (Figure 1). The tunnel invert is 28 m below ground level.

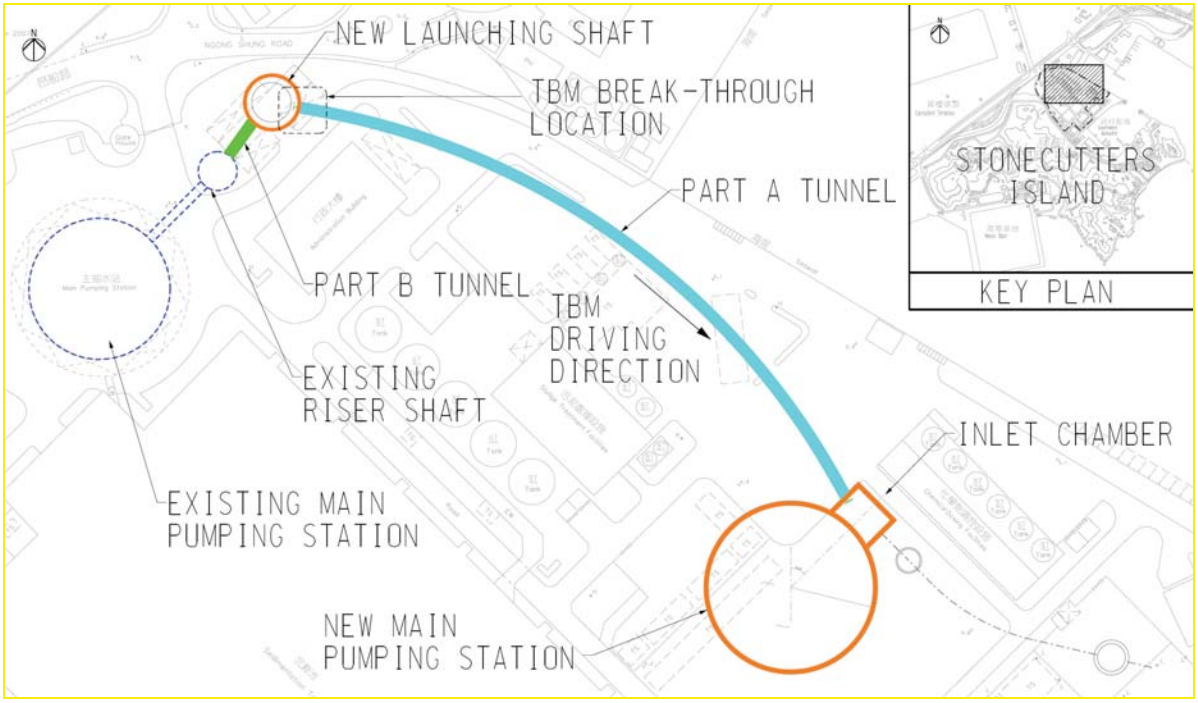


Figure 1: Tunnel layout plan

To avoid the wearing of the TBM cutters due to excavation of the diaphragm wall, the fibre glass reinforced concrete at the tunnel eye would be completely removed prior to the TBM launching. As the tunnel level is at water bearing sandy Alluvium strata, ground improvement at that level is required to avoid ingress of soil and groundwater into the shaft when the diaphragm wall is removed. The Contractor had selected the AGF as the ground improvement method because of their solid experience on this technology.

Apart from the aforementioned primary function, the frozen block can also reduce the water ingress during the TBM launching. When the TBM cutterhead passes through the frozen block, a gap would be created between the excavation profile and the TBM shield, forming a water leakage path into the shaft. The frozen soil block can seal off the gap by freezing the water seeping along the gap. This advantageous phenomenon however depends on the flow rate of water, which is difficult to be precisely quantified. As such, a rubber packer fixed on a one-way hinged steel plate and two rows of steel wire brush with grease injection ports were also installed, as shown in Plate 1. The frozen block, steel wire brushes and rubber packer forms a robust water-stop system during the TBM launching.



Plate 1: One-way rubber packer (left) and steel wire brushes (right)

2 GROUND CONDITION

The launching shaft is located at the reclaimed land of the northern side of Stonecutters Island. A summary of the typical ground condition is as follows:

Table 1: Summary of ground condition at launching shaft area

Soil Type	Thickness (m)	Description	SPT N Values
Fill	~ 17	Slightly silty fine to coarse SAND with bouldery fill locally	7 - 45 (avg 41)
Marine Deposits	~ 5	Firm to stiff, slightly sandy silty CLAY	7 - 17 (avg = 9)
Alluvium	~ 9	Clayey silty fine to coarse SAND or sandy silty CLAY	16 - 46 (avg = 28)
CDG (N<200)	> 20	Extremely weak to very weak, very sandy clayey SILT to slightly clayey silty SAND	16 - 100 (avg = 83)

With regard to the groundwater condition, the Marine Deposits and the clayey layer of the Alluvium separated the groundwater into upper and lower aquifers. The groundwater at the upper aquifer is mainly controlled by the sea. The lower aquifer is in the CDG and sandy layer of Alluvium, in which the piezometric head is at approximately +1 mPD. The water pressure at tunnel level is approximately 250 kPa.

3 DESIGN OF THE FROZEN BLOCK AND FREEZING PIPES

The frozen block formed adjacent to the tunnel eye was designed to withstand the soil and water pressure when the diaphragm wall at the tunnel eye was completely broken-off. The design temperature of the frozen block was -15^{o}C and the corresponding design UCS creep strength of Alluvium was 1.75 MPa. Calcium chlorite brine solution of specific gravity of 1.26 was used as the heat transfer coolant. The frozen soil block was formed by circulating -28^{o}C brine solution in three rows of freezing pipes in staggered pattern from ground level down to the bottom level of the frozen block. The spacing of the pipes was 0.8m to 1m. The freezing pipes layout was based on the Contractor's past experience and the PRC code of practice DG/TJ08-902-2006 (SUCCC, 2006). The layout was further verified by a thermal analysis. It was estimated that the active freezing period is 40 days. Figure 2 below shows the design of the frozen soil block.

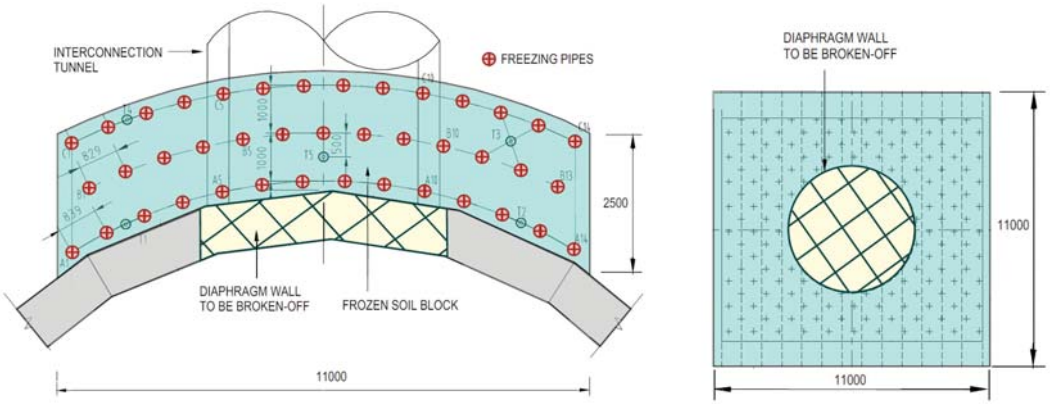


Figure 2: Frozen soil block plan (left) and elevation (right)

The design of the freezing pipes is shown in Figure 3. There were two inner tubes inside an outer tube. Brine solution was pumped into the longer inner tube and returned to ground level through the shorter inner tube. A capping plate was installed slightly higher than the short inner tube to keep the brine solution to come in contact with the outer tube at the freezing section only.

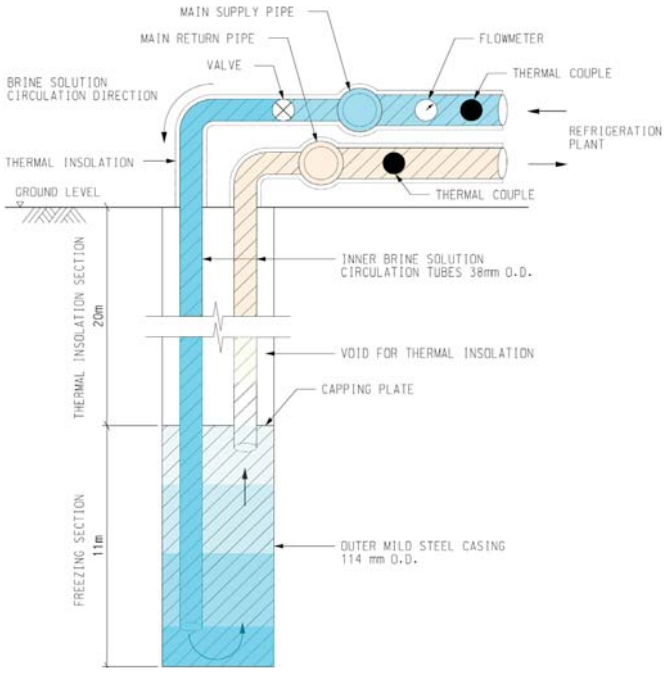


Figure 3: Freezing pipe design and brine circulation system

4 FREEZING PIPES INSTALLATION AND ACTIVE FREEZING

The installation of freezing pipes was carried out by HD 110 hydraulic crawler drilling rig using ODEX drill bit. After the hole was formed, the freezing pipe, which is made of mild steel, was then lowered into the pre-formed hole. Before the drilling casing was removed, bentonite slurry was used to fill the gap between the freezing pipe and the drilling casing such that heat in the ground can be extracted to the freezing pipes effectively.

The completed freezing pipes system is shown in Plate 2. The temperature of the soil was monitored by 26 thermal couples installed at different locations. The temperature was monitored daily and the readings during the active freezing period are presented in Figure 4. After 30 days of active freezing period, all the readings were below -15°C. The actual active freezing period is 10 days shorter than the estimated.



Plate 2: Completed freezing pipe system (left) and freezing pipe in operation (right)

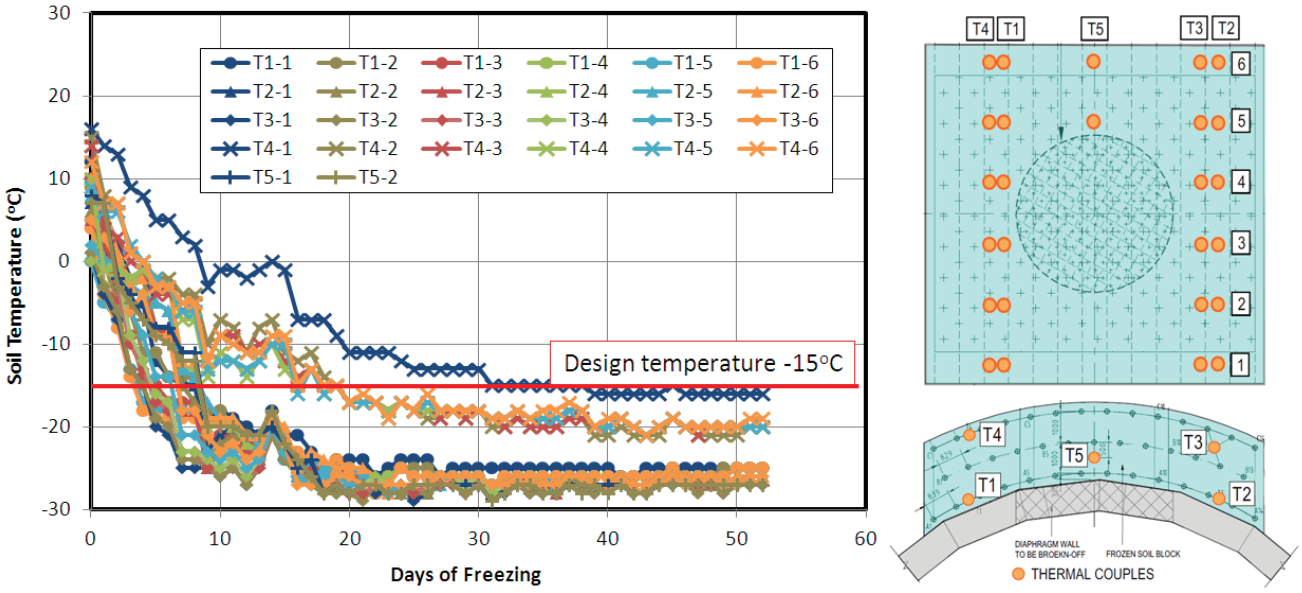


Figure 4: Temperature monitoring and monitoring layout

5 DIAPHRAGM WALL BREAK-OFF AND FREEZING PIPE LIFTING

After all the temperature monitoring readings reached the design temperature, water tightness of the frozen block was checked by a probe hole through the diaphragm wall. The diaphragm wall at the tunnel eye was then broken-off. The exposed frozen soil is shown in Plate 3. There was no sign of any water ingress after diaphragm wall break-off and the frozen block was considered to be completely water-tight at this stage.

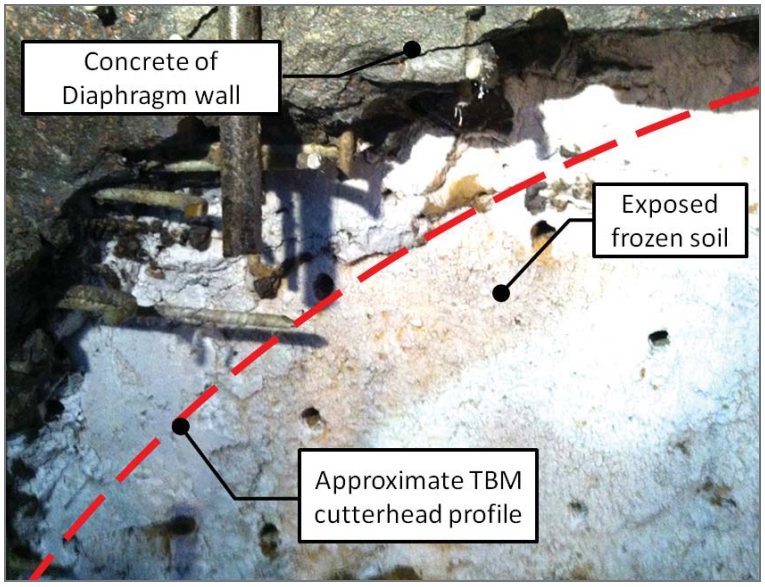


Plate 3: Exposed frozen soil during diaphragm wall break-off

The TBM was then pushed forward such that the TBM shield was encased by the rubber packer and the steel brushes to provide the water proofing function. The freezing pipes on the footprint of the TBM tunnel were temporary disconnected, lifted to a level above the tunnel crown and re-connected to the brine solution supply pipe one by one. Before lifting of each freezing pipe, warm water was circulated in it for a few hours such that the frozen soil in contact with the freezing pipes was thawed.

After a number of the freezing pipes were lifted, it was noted that there were some water leakage at the defect locations of the rubber packer. This indicated that the freezing pipes lifting operation altered the frozen block property and undermined the water-tightness of the frozen block. The steel wire brushes and the rubber packer started to carry water pressure at this stage. Groundwater seepage into the bulkhead was also noted. To ensure the face stability at this stage, the bulkhead was filled with bentonite PFA slurry before excavation. The bentonite PFA was designed to have a property of semi-solid material after set, which could fill up the bulkhead to avoid any soil ingress and also could be removed by the screw conveyor during launching. The defects at the rubber packer were rectified before the freezing pipes lifting operation continues.

During the period of rectifying the defects at rubber packer, it was found that the cutterhead was jammed. The possible reasons are that the bentonite PFA slurry itself was too strong or the slurry was stiffened by the AGF in front of it. The first solution to the problem was to remove the frozen slurry in the bulkhead by jetting water through the bulkhead access gate. This operation had cleaned up most of the frozen slurry within the chamber but the cutterhead was still jammed. During the bulkhead inspection, some frozen slurry in front of the cutterhead was seen, as shown in the Plate 4, and no water ingress through the frozen block was observed. To thaw those remaining frozen slurry, the bulkhead was filled up by warm water, followed by steam injection. The temperature of the warm water was controlled at approximately 40^{o}C.

6 TBM BREAK-THROUGH AND ASSOCIATED SOLUTIONS

The cutterhead was freed after 2 days of steaming and the launching operation then continued. All the freezing pipes within the footprint of the tunnel were lifted and the TBM then started to excavate into the frozen soil.



Plate 4: Frozen slurry in front of the cutterhead

The TBM excavated the frozen soil zone at a speed of 1m per day. The TBM was kept advancing during this period to avoid freezing up the shield. When the cutterhead passed the frozen block, slight dripping of water was noted at the packer once. It was considered that gap between the frozen block and the TBM shield was opened during the TBM advance but sealed up again afterwards. The steel wire brushes and the rubber packer were also able to resist the water pressure developed. The long term water-stop was achieved by welding a steel bracket connecting the steel collar at the packer location and the special fabricated ring. Cement grout and chemical grout were then injected to fill up the void inside the bracket. It was completely water-tight after the grouting operation.

7 CONCLUSION

AGF technique was demonstrated to be a viable solution for TBM launching. The frozen soil block could achieve the required strength and provide complete water cut-off performance after the diaphragm wall was removed. During the launching stage, the water-stop system using a combination of frozen block, rubber packer and steel brushes were also demonstrated to be effective.

The properties of frozen soil however are time dependent and as such time is a critical factor in planning the construction sequence. Any TBM stationary period should be avoided when it is close to the frozen block. Besides, freezing pipes lifting operation would inevitably undermine the frozen block properties in terms of strength and permeability. Some case had reported the use of aluminium to fabricate the freezing pipes at the tunnel eye section for the rock TBM to cutter through in order to avoid the need of lifting the freezing pipes (Sopko et al, 2011). However, soft ground TBM may not equip competent cutters to cut the aluminium freezing pipes. It could be a future research area to explore other suitable material, which is thermal conductive and high borability, for freezing pipes at the tunnel eye section.

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Mined Tunnel Construction using Artificial Ground Freezing Technique for HATS 2A Project

L. Tsang & A. Cheung
Ove Arup & Partners Hong Kong Limited
C. Leung
China State – Shanghai Tunnel Joint Venture
W.L. Chan
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

In Hong Kong, Artificial Ground Freezing (AGF) is not a common soil improvement approach. There are only very few local cases and experiences on this technology. There is also no local design guidance for AGF. This article presents a case of using AGF as the soil improvement method in soft ground for mined tunnel construction in Harbour Area Treatment Scheme Stage 2A (HATS 2A) project. The design and construction considerations of AGF are discussed. Laboratory test results of frozen soil and considerations of strength parameter selection are also presented. Based on the experience developed from this project, this article attempts to provide a reference to the design and construction for future AGF projects in Hong Kong.

1 INTRODUCTION

The Harbour Area Treatment Scheme Stage 2A (HATS 2A) comprises a 3.9m internal diameter and 28m deep Interconnection Tunnel to connect the existing and the proposed pumping stations. There is 14m of the entire tunnel to be built by hand mining method. This mined tunnel, named as Interconnection Tunnel – Part B, will connect the new Launching Shaft to the existing Riser Shaft (Figure 1). To facilitate the construction of it, AGF was selected as the soil improvement method to form a 2 m thick frozen soil ring in Alluvium and Marine Deposits as the temporary support for tunnel excavation.

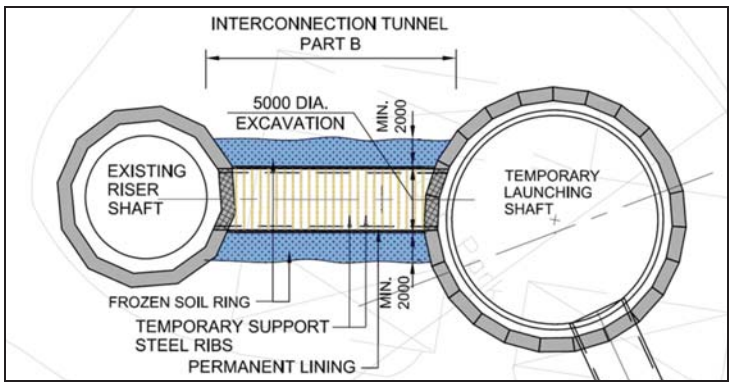


Figure 1: Tunnel layout plan

2 APPLICATION OF ARTIFICIAL GROUND FREEZING IN HONG KONG

There were only very few local cases reported the application of AGF in Hong Kong. The Kowloon Canton Railway Corporation Lok Ma Chau Spurline project adopted AGF for the construction of three cross passages (Storry et al, 2006). Horizontal ground freezing was carried out in Completely Decomposited Volcanic and intact volcanic rock using brine solution system.

AGF was also applied in Harbour Area Treatment Scheme Stage 1 for launching of a 1.8 m diameter pipe jacked tunnel in Kwun Tong (Pakianathan et al, 2002). The depth of the tunnel launching level is 22 m and a frozen block was formed in Marine Deposits by vertical freezing pipes. Liquid nitrogen was used as the coolant.

Mass Transit Railway Corporation West Island Land adopted ground freezing for the construction of a 26 m long tunnel for obstruction removal.

In HATS 2A, vertical ground freezing was also carried out in the same contract to form a 2.5 m thick frozen block for TBM launching break-through at the Launching Shaft.

3 REVIEW OF GEOTECHNICAL DESIGN CONSIDERATIONS

The formation of the ice block is a key consideration for the AGF technology in the feasibility assessment. The factors include water content in soil and groundwater flow rate at the freezing location. Laboratory test results by Enokido & Kameta (1987) for river sand indicated that UCS of 6 to 7MPa could be developed with water content at only 5% at -30^{o}C. In general, minimum water content of 10% is required to bond the soil particles (Harris 1995). The testing results on sand by Kuribayashi et al (1985), as shown in Figure 2, showed a strong relationship between water content in soil and strength of the frozen soil.

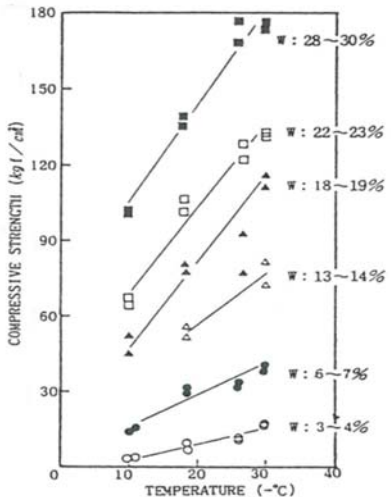


Figure 2: Relationship between strength, temperature and water content for sand (Kuribayashi et al, 1985)

With regard to groundwater flow rate, it is difficult to form a continuous frozen soil wall if the water flow velocity is larger than 1 to 2m/day for brine solution system (Andersland, 2004). PRC code of practice DG/TJ08-902-2006 (SUCCC, 2006) recommended that detail investigation should be carried out when groundwater flow rate is larger than 5m/day. However, liquid nitrogen system could tolerate a much larger water flow rate. Shuster (1972) reported that ice block is able to form where groundwater flow is as high as 50m/day for liquid nitrogen system.

Strength and deformation behaviours depend on the ice content, unfrozen water content, air content and original soil structure of the frozen soil. Temperature has direct effect on the strength of the frozen soil because of its influence on the amount of unfrozen water. Laboratory test from Bourbonnais & Ladanyi (1985a & b) indicated that strength of frozen sand increases sharply with decreasing temperature to about -40^{o}C but tends to level off at about -100^{o}C. Very different trend was observed for overconsolidated clay, on which the strength exponentially increases when temperature is below -60^{o}C. The strength of frozen clay could exceed frozen sand when the temperature is sufficiently low.

Because of the creep property of the ice, frozen soil also creeps under loading. When subject to loading, the frozen soil would keep deforming for a certain period of time. Eventually, strain of the soil may reach a constant value, if the stress level is low, or failure may occur, if the stress level is high. In general, the long-term strength of frozen soil is approximately 40% to 60% of the instantaneous strength (Schultz & Hass, 2011).

Application of AGF for the Interconnection Tunnel – Part B is considered feasible because the in-situ soils to be frozen are submerged and the water contents in which are all larger than 20%. The estimated groundwater flow at the site is only 0.12m/day and therefore brine solution system is applicable. Other design considerations include salinity of the groundwater, frost heave/thaw consolidation, thaw weakening, brittleness of failure, etc. The importance of which depends on the application and they are not discussed further in this article. The following discussion focuses on the testing and analysis on the strength of frozen soil.

4 TESTING OF FROZEN SOIL

The package of test carried out for frozen soil included UCS test, creep test, frost heave/thaw consolidation test, freezing temperature test, thermal conductivity test and heat capacity test. Determination of strength parameters for temporary support design relies on the results of UCS test and creep test. PRC standards MT/T 593.4 and MT/T 593.6 (MCI 1996a & b) were adopted as the testing standards for UCS tests and creep tests respectively.

For UCS test, as the design temperature is -15°C, the samples were tested at -10°C, -15°C and -20°C to establish the temperature and strength relationship. The samples were first frozen to the test temperature and then subject to increasing loading under a constant strain rate. The instantaneous UCS value is defined as maximum stress attained or stress at 20% axial strain, whichever is obtained first during the performance of a test. The adopted UCS value (q_i) was based on a moderately conservative fit line on the instantaneous UCS data as shown in Figure 3.

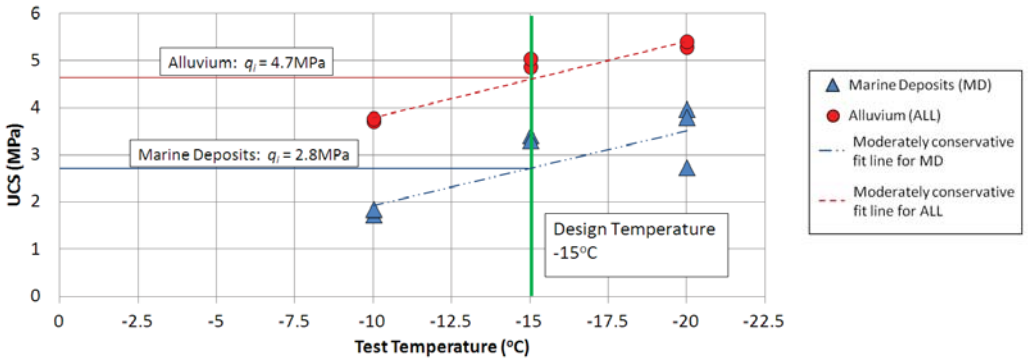


Figure 3: UCS test results

With regard to creep test, the frozen soil samples were subject to a constant loading at $0.7q_i$, $0.5q_i$, $0.4q_i$ and $0.3q_i$. A typical creep curve of a sample in the creep test consists of primary (decreasing strain rate), secondary (strain rate remains essentially constant) and tertiary (increasing strain rate) creep stage. For a sample which could sustain the load, the tertiary creep stage is absent. The termination criteria of the tests are failure of the frozen sample or 24 hours after the strain rate of the sample is less than 0.0005^{-h}. Where the latter criterion is met, the sample is considered to be able to sustain the corresponding load for sufficient long period of time without tertiary creep stage. Strain and time relationship for a frozen Marine Deposits sample failed under $0.7q_i$ is shown in Figure 4 below.

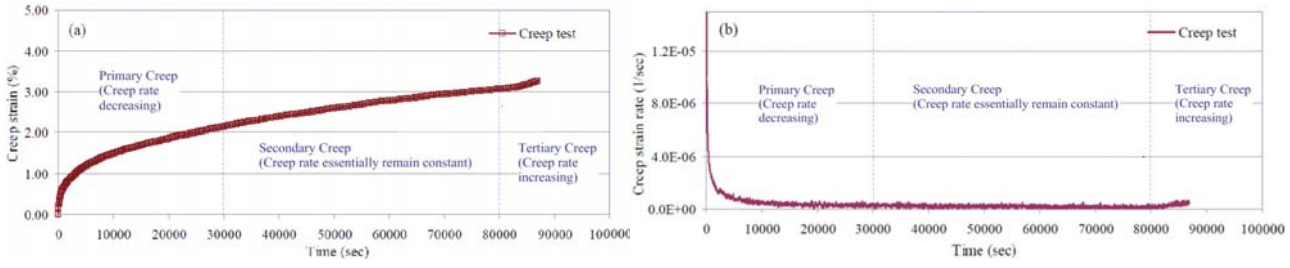


Figure 4: Relationship of creep strain rate and time under creep test at -15°C

5 SELECTION OF DESIGN STRENGTH PARAMETERS

In selecting the design parameters, the creep property is a key consideration. The instantaneous strength of frozen soil was considered not suitable for design. Based on some published results (Fish, 1991 and Sheng, 1997), approximately 50% of strength reduction from instantaneous strength would happen in less than 24 hours.

It was considered that strain rate is also a critical consideration for selecting design parameters. Typical stress-strain relationship for Marine Deposits is shown in Figure 5. It can be seen from the plot that although the stress level kept increasing, the strain level significantly increased when the stress is beyond 2MPa. It was felt that the sample was effectively yielded at this stress level and as such it was adopted as the yield strength.

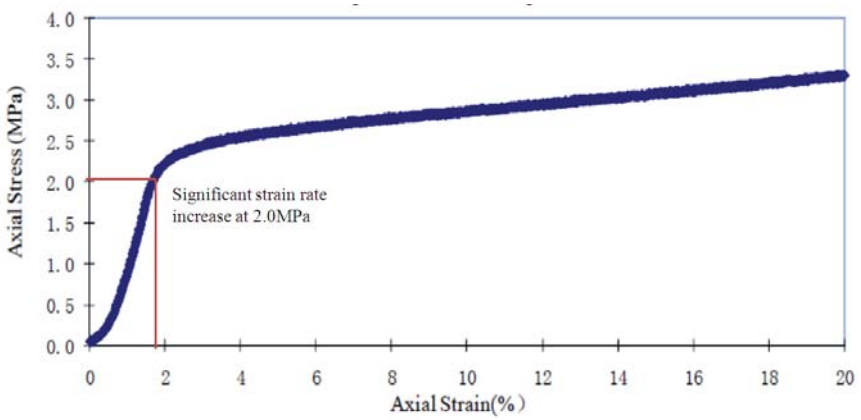


Figure 5: Typical stress-strain behaviour of Marine Deposits in UCS test

It was considered that the design strength adopted should address the concern of excessive deformation and at the same time fall within the reasonable range of strength reduction from q_i. Therefore, the design creep UCS, i.e. long-term UCS, was taken as half of the yield strength, which is approximately equivalent to 40% of q_i. The design strength parameters adopted are presented in Table 1.

Table 1: Summary of Testing Results and Adopted Design Strengths at -15°C

Soil type	UCS test q_i[1] (MPa)	Creep test at $0.5q_i$[2] (MPa)	Yield strength (MPa)	Unfactored creep UCS for design (MPa)
Marine Deposits	2.8	1.4 (no creep failure)	2	1
Alluvium	4.7	2.3 (no creep failure)	3.5	1.75

Note:

(1) The adopted UCS (q_i) was based on moderately conservative interpretation of instantaneous UCS data, as shown in Figure 3.

(2) All the frozen Marine Deposits and Alluvium samples at test temperature -10°C, -15°C and -20°C passed the creep tests at stress level of $0.5q_i$ but no sample could sustain the load of $0.7q_i$ in the creep tests.

The stability of the tunnel construction was assessed using finite element method. The conventional factoring approach for tunnel design would have an overall FOS contributed by load factor and material factor on structural members. However, this approach was considered not able to cater for the situation when the stability of the tunnel relies heavily or solely on the strength of frozen ring because there is no factor applied onto the frozen soil. A partial material factor of 2.0 was therefore applied onto the design creep UCS, together with the load factor and material factor on structural members. Face support was also considered in the analysis. To cater for the uncertainties related to the contribution of face support, the design could tolerate a relaxation factor of 0.3 to 0.7.

6 CONSTRUCTION CONSIDERATIONS

The temporary support of the tunnel comprises a 2 m thick frozen ring and steel rib installed every 600 mm. The frozen soil ring would be formed by 2 rows of freezing pipes surrounding the tunnel using brine solution system. The layout design of the freezing pipes was based on Contractor's experience and PRC code of practice (SUCCC 2006). Thermal analysis was carried out to further verify the freezing pipe layout design and estimate the active freezing period. The temporary support and horizontal freezing pipes layout are shown in Figure 6 below.

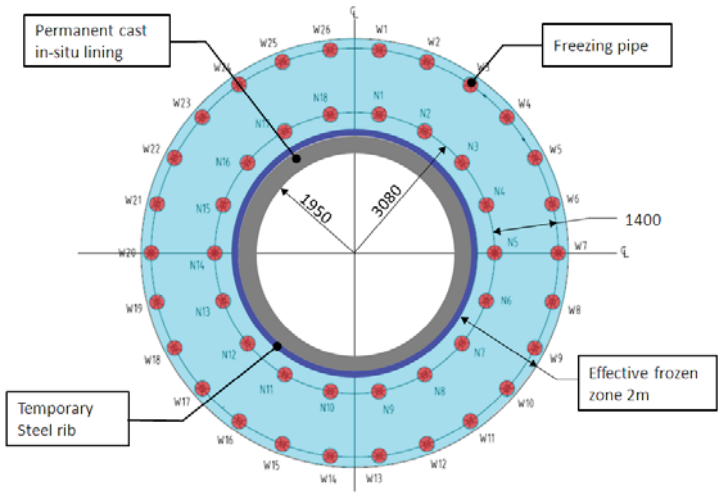


Figure 6: Tunnel temporary support and horizontal freezing pipe layout

Thermal couples would be installed to monitor the temperature of the frozen soil. The layout of the thermal couples should be carefully arranged such that the temperature gradient along the radial direction of the tunnel could be clearly revealed. More thermal couples should be placed at the shaft/tunnel junction location because the frozen soil near the shaft may be influenced by the atmospheric temperature in the shaft.

Generally, for horizontal freezing, the gap between the end tip of the horizontal freezing pipe and the outer surface of the shaft is the weakest location for water leakage. In the present design, two rows of the freezing pipes would be installed from opposite directions to provide a better water cut-off at the junction. As the accuracy of the position of the freezing pipe would affect the formation of the frozen ring, the allowable out of position for each pipe is 200mm. Where this limit is exceeded, additional freezing pipe is required. The design of the freezing pipe is shown in Figure 7. Brine solution will be brought to the tip of the freezing pipe by an inner tube and make contact with the outer casing along the full length of the freezing pipe.

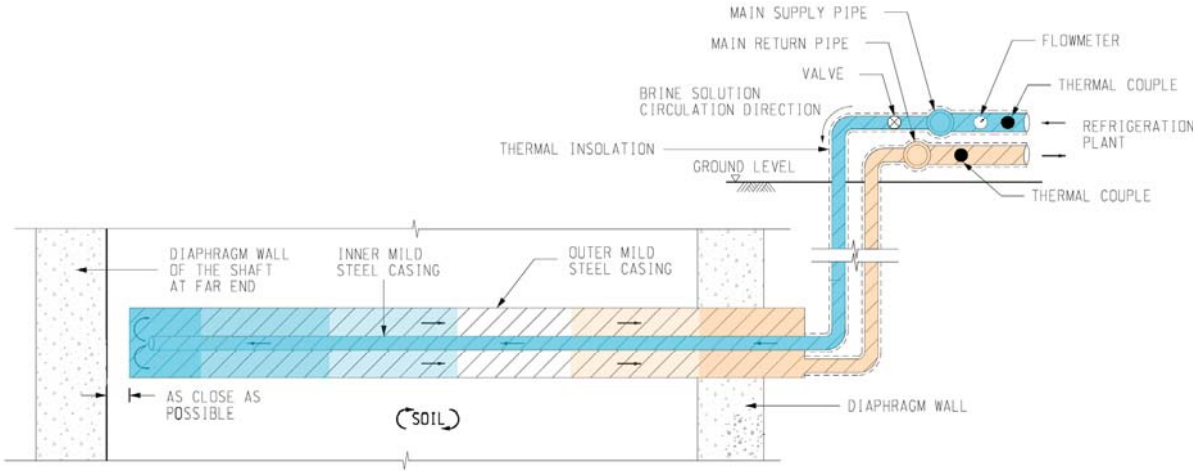


Figure 7: Horizontal freezing pipe design

The estimated active freezing duration is 60 days. Pressure relief holes would be installed to release the water pressure developed within the frozen ring and also to check the condition of ring closure. It can be inferred that a confined frozen ring is formed when the water pressure within the ring increases.

Excavation would commence when the readings at the thermal couples indicate that the temperature of the frozen ring reaches the design requirement. The advance length is 600 mm and steel rib would be installed for each advance length. Steel plate would be installed between steel rib and thermal insulator would be mounted onto the steel plate. During excavation, the soil temperature should be closely monitored using the thermal couples. Cast in-situ permanent lining would be installed after the tunnel is excavated through.

7 CONCLUSION

The AGF design presented in this article had gone through careful consideration on the fundamental mechanical properties of frozen soil. Some recognised good practice from overseas was incorporated in devising the construction method. It was felt that the parameters selection and factoring method adopted are a reasonably safe approach but further study is needed to rationalize these aspects in order to cater for the uncertainties related to AGF. As the AGF technology is becoming a more readily available soil improvement method in the Hong Kong, there is a need to develop a unified design approach and a sound construction practice for future AGF application in Hong Kong.

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Construction Risk Mitigation of the Tunnel to Station Connection Using Artificial Ground Freezing in the MTRCL West Island Line Contract 703

S. Polycarpe & P.L. Ng
Dragages-Maeda-BSG Joint Venture, Hong Kong
T.N.D.R. Barrett
MTR Corporation Limited, Hong Kong

ABSTRACT

During the design stage of the MTR Corporation Ltd (MTRCL) West Island Line (WIL) Project, it was highlighted that temporary rock supports may have been left in-situ during remedial works to the then Western Market development foundations completed under MTRC Contract 402. The presence of steel sets within the Tunnel Boring Machine (TBM) new up track tunnel excavation profile was confirmed early in the construction stage of WIL Contract 703. The sets posed an obstruction to the TBM, after evaluating the options to remove the steel sets it was agreed that they should be removed prior to the TBM excavation to minimize the risk to workers, third parties, the Island Line (ISL) and the construction program. Artificial Ground Freezing (AGF) was adopted to stabilize the 30 m gallery in mixed ground required to remove the sets. With detailed planning, implementation and assistance from MTRCL Operations Division steel grossing 700kg was successfully removed without impact to the operating railway or third parties. AGF also allowed localised excavations to survey the existing caisson piles confirming they were not within the excavation profile; moreover it made an AGF canopy for the TBM arrival and associated connection to the ISL possible further mitigating the construction risks.

1 INTRODUCTION

The WIL Project on Hong Kong Island extended the existing ISL by approximately 3.3 km of underground excavations from the existing Sheung Wan Station (SHW) to the new Kennedy Town (KET) Station via new stations at Sai Ying Pun (SYP) and University (UNV). WIL Contract No. 703 – SHW to SYP Tunnels constructed the works between SYP and SHW forming the connection between the new extension and the existing ISL Cross Over Box (COB) and Traction Current Building (TCB). The works comprised twin bore single track tunnels between the existing SHW COB and SYP with an access at Sai Woo Lane (SWL), see Figure 1. MTR Corporation Ltd MTRCL was the Client with the Main Contractor being a Joint Venture comprising Dragages, Maeda and Bachy Soletanche Group (DMBJV).

During the construction of the existing ISL in the 1980s a property development was constructed near SHW known as the Western Market Site (WMS). The site is to the west of the existing COB now known as Hongway Garden (HWG). During testing of the caisson piles it was highlighted that some remedial works were required, several galleries were excavated by hand from the newly constructed ISL overrun tunnel to investigate and perform remedial works. The as built records did not confirm whether all of the steel sets were removed but if left in-situ they would certainly impede the up track TBM drive. The records indicated that at least four steel sets could be present 20m from the COB with the excavation backfilled with concrete.

Core drilling to locate the steel sets was undertaken with the cores planned to intersect the steel set locations. After coring geophysical investigations were conducted including magnetic survey, electric cylinder, electric cross-hole tomography between adjacent boreholes and ground penetrating radar to detect the exact number and locations of the steel sets (Mogenier et al, 2011). Four steel sets and other metallic objects such as grout pipes and rock bolts were found from the core logs, the geophysical survey suggested that other objects were present with fixed lateral connection between the steel sets. . In order to allow the successful completion of the up track TBM drive the larger steel sets had to be removed from the excavation profile, this had to be completed with minimal risk to safety, the operating ISL and third parties.

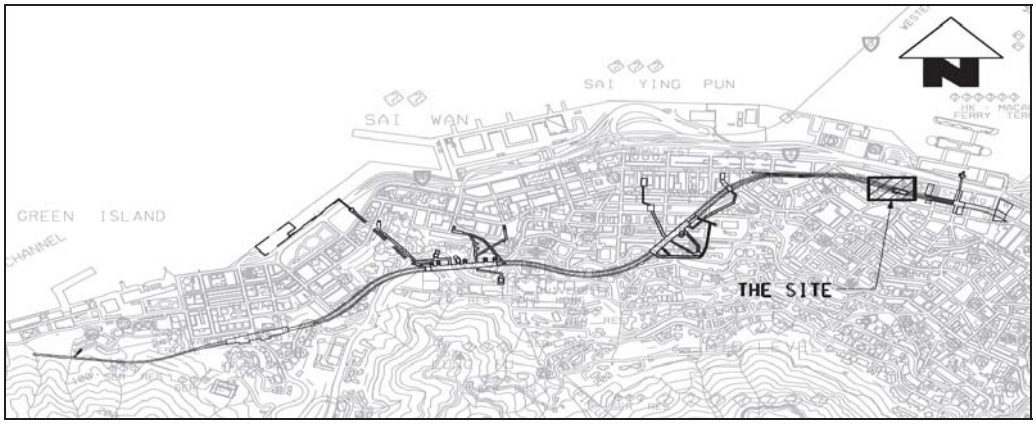


Figure 1: Location plan
Note: Western district of Hong Kong Island; The Site being the COB and gallery location.

2 OPTIONS TO REMOVE THE OBSTRUCTIONS

A number of options to remove the obstructions were considered including (1) ground improvement by grouting followed by intervention and removal from the TBM excavation chamber, (2) excavation of an access gallery from the COB under compressed air, (3) ground improvement by grouting followed by excavation of an access gallery from the COB in free air, and (4) ground improvement by grouting and AGF followed by excavation of an access gallery in free air. These options were evaluated for safety, risk, suitability to the geology, technical complexity, equipment availability, time and cost.

2.1 Intervention and removal from TBM

To overcome the complex geology along the tunnel alignment a slurry mix-shield TBM was required, the TBM was fitted with special disc cutters able to identify obstructions (the disc sensors are part of the Mobydic system developed by Bouygues Construction, parent company of Dragages Hong Kong Ltd.). To remove the obstructions from the TBM excavation chamber the TBM excavation speed would be reduced when in close proximity to the obstructions, as the Mobydic cutters encountered an obstruction the TBM would stop. The slurry level in the excavation chamber would be lowered and replaced with compressed air checking to ensure that no air loss or water ingress occurred. Then hyperbaric interventions would be carried out under the required confinement pressure, circa 3 bar, to first locate and assess and then remove the obstructions by cutting them into pieces to be extracted via the TBM material lock.

The TBM up track excavation had to be completed on time to ensure that the reinstatement of the degraded refuge siding was handed back to MTRC Operations Division on the agreed date and as such was one of the contracts program critical paths. Having to progress interventions from the TBM excavation chamber would have had a direct impact on the TBM construction program, further it could not be confirmed whether there were more steel sets causing further delay if overrunning the allowed time in the program, as such it made sense to remove the activity from the TBM drive which provided a program saving. When judged with the health and safety risks of prolonged hyperbaric works cutting steel, with any stoppage increasing the risk of settlement especially if longitudinal steel members were present, it was agreed that this option would not be pursued by MTRC and DMBJV as these risks could instigate significant escalation in duration and cost.

2.2 Excavation of gallery under compressed air

The as built construction records of the ISL Contract 402 confirmed that the underpinning excavations were completed under hyperbaric conditions of approximately 2.6 bar to provide temporary support to the excavations and avoid water/ground ingress. Using similar hyperbaric procedures as adopted on 402 could be replicated to remove the obstructions by a gallery from the COB, however again there would be similar risks associated with prolonged hyperbaric works as in the TBM option with a new risk associated with compressed air blow out or leakage to the operating railway. A further impact to the excavation program due to the requirements of safe hyperbaric works, the compression and decompression times required and short intervention durations would necessitate many hyperbaric interventions and prolongation. Again in a similar

manner to the TBM option it was agreed that this option would not be pursued by MTRC and DMBJV as more sophisticated methods were available that reduced the risks of excavating under compressed air.

2.3 Ground improvement by grouting followed by gallery excavation

Ground treatment by grouting to improve the mechanical properties and reduce the permeability of the strata was considered. However the reliability of grouting for full support at such pressures is impossible to confirm. The area above the gallery was occupied by the existing HWG and as such only 30m long inclined holes could be drilled from the adjacent New Market Street. Due to this the homogeneity of the ground treatment would be dependant on the accuracy of the drilling which is difficult to achieve. Grouting from the excavation face was also possible but would cause unacceptable delay to the program. Moreover the efficiency of the grouting would be limited given that the achievable permeability was in the order of 10^{-6} m/sec. Two key risks were highlighted with this option, damage to structures above the works due to settlement and the affects to the ISL associated with any material inrush. In view of these points it was agreed that this option would not be pursued by MTRC and DMBJV as more sophisticated methods were available that reduced the risks.

2.4 Grouting and artificial ground freezing followed by gallery excavation

AGF was a viable option and a proven technology having been first used in Germany during 1883 with many subsequent applications over the years for various underground construction works such as tunnelling, shaft sinking, and cross passages excavation (Harris, 1995; Andersland & Ladanyi, 2004). The frozen soil would effectively act as the tunnel support during excavation. Prior to ground freezing works, grouting would be performed to limit the groundwater flow, so as to minimize the heat loss between groundwater flows and the ice wall (Schultz et al, 2008). The ground freezing method offered an effective solution to the project constraints and challenges with improved construction risk management and control. The AGF option was selected by MTRC and DMBJV for the following reasons:

- Suitable for variable ground conditions;
- Offers reliable and robust structural support and groundwater cut-off for the excavation;
- Eliminates the health and safety risks associated with hyperbaric operations;
- Recognized technology in worldwide applications (Spiby, 2002; Arlet et al, 2005; Martin et al, 2007);
- Successful recent adoptions in Lok Ma Chau Spur Line project in Hong Kong (Storry et al, 2006);
- The costs associated with the AGF gallery where comparable to the other options with the benefit of greatly reduced risk;
- Reduced the planned TBM programme by 6 days and further reduced subsequent risk;
- Provided access to locate other steel obstructions and the adjacent caisson piles;
- The freezing plant could be reused to provide AGF stabilization for the TBM tunnel connection to the COB reducing risk at a saving compared to the original grouted pipe pile canopy design.

3 PLANNING AND DESIGN OF THE ARTIFICIAL GROUND FREEZING

3.1 Site geology

Prior to construction extensive ground investigation was undertaken in the form of vertical, inclined and horizontal drill holes, these revealed that the gallery would be excavated through partially weathered granite with completely decomposed granite (CDG) near the crown. Above the CDG was a band of Marine Deposits approximately 12 m thick, which in turn was overlain by 12 m of Fill material. Figure 2 shows the assumed longitudinal geological profile for the gallery with the steel sets situated under circa 2.5 bar hydrostatic pressure.

3.2 Laboratory soil testing

To obtain the mechanical and thermal properties of the frozen and unfrozen soil to inform the detailed design, a comprehensive laboratory test schedule of representative samples from the GI cores was completed. The testing of the frozen samples was conducted by a specialist laboratory in Germany in accordance with the

relevant German (DIN) Standards and American Society for Testing and Materials (ASTM) Specifications. The frozen soils creep properties were also tested and when compared with the design model showed that there was a factor of safety of approximately 2, the design took into account the duration over which the excavation would remain open and was concluded acceptable.

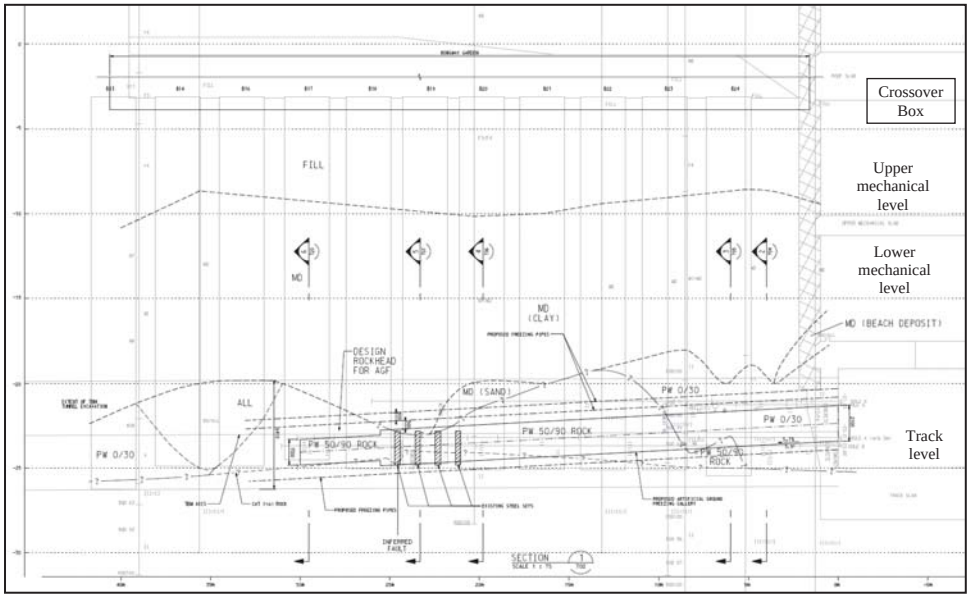


Figure 2: Longitudinal geological profile along the gallery

3.3 Computer analyses

The AGF was designed to freeze the strata locally around the excavation in order to form an annular ice wall. The ice wall required sufficient thickness so that during excavation it was able to resist the developing hoop stresses generated, thus maintaining the excavations stability. An ice plug 3m long was required at the end of the gallery to ensure face stability and water tightness, effectively isolating the excavation from the surrounding strata. Stability relied on the creep strength of the ice wall; a shotcrete lining was applied as a back up in case of freezing plant failure or excessive creep. The design assumed the ice wall and the subsequent shotcrete lining as independent structures to temporarily support the excavated gallery.

Mechanical, thermal, and hydrological analyses were completed using finite element and finite difference computer programmes, namely PLAXIS, FLAC, TEMP/W and SEEP/W. An account of the detailed design was reported by Wong et al (2012). The ice ring was conservatively defined as being enclosed by the -10°C isotherm. In the mechanical analysis, the structural integrity of the ice wall and the shotcrete lining were checked and found adequate with required minimum and maximum ice wall thicknesses. The thermal analysis provided an estimated time to achieve the required ice wall thickness, the hydrological analysis confirmed the groundwater flow would not dramatically impede ice wall formation. Impact assessments were conducted for the existing buildings, caisson piles and ORT to confirm there would be no detrimental effects from the freezing and thawing stages. The design was reviewed by the Buildings Department under the Instrument of Exemption consultation process.

3.4 Risk management

The gallery was adjacent to the operating ISL railway and underneath the urban residential area. To ensure satisfactory completion of works, a comprehensive risk assessment was conducted. The corresponding mitigation measures were developed and an Emergency Action Plan was formulated and enacted during the course of the works. Multiple parties were included during the design providing technical support to the contract team including AECOM as the JV Designer, GCG Asia as the Independent Checking Engineer and Bouygues Construction as the JV Expert and Steve Doran at the MTRCL Expert. The major risks and mitigations are noted below:

- Variability of design assumptions – a sensitivity study was completed;

- Health and safety of workers in low temperature environment – medical advice was obtained;
- Outbreak of Fire – fire alarm system integrated with the SHW station, fire fighting equipment;
- Breakdown of the freezing plant or power supply – 100% plant redundancy, ice wall thickness sufficient to last four days with no freezing plant;
- Damage to the freezing pipes and associated brine leakage – redundancy allowed in design;
- Target temperature and ice wall thickness not achieved – as-built alignment of freeze pipes recorded using Deviflex and 3D computer geometrical model run with additional pipes installed as required, rerun of freezing model;
- Collapse leading to major soil ingress – shotcrete lining with an inflatable rubber plug for contingency;
- Damage of nearby structures– extensive instrumentation and monitoring, impact assessments and modelling.

3.5 Design elements driven by constructability

During the detailed design it was initially envisaged that the gallery would be constructed in two separate stages. This was to limit the drilling length to 20 m in order to ensure the drill-hole deviation remained within the range of 1.5% to 2% when using conventional drilling methods. A review of available drilling methods was undertaken to see if the accuracy could be achieved over the full distance, as this would save time during drilling and excavation. It was found that the accuracy could be achieved with the down-the-hole water hammer method.

Both Liquid Nitrogen (LN) and Brine (concentrated calcium chloride) could be used as the cooling agent for ground freezing with LN offering faster ice wall formation. Brine was decided as the cooling medium for several key reasons, there was no LN storage available on Hong Kong Island, transportation of LN by road through one of the Victoria Harbour tunnels require a special permit and was logistically complex, the cost of LN is high compared to Brine and the excavation was underground in a confined space where any leakage of nitrogen would give rise to health and safety risk. Another key point was that the works programme could absorb the extended duration required to form the ice wall using Brine instead of LN.

It was agreed that a secondary lining was required to ensure a backup was in place should any failure of the freezing system or ice wall spalling occur. Steel was obviously not an option, wood was discounted leaving concrete. Due to the risks involved running a concrete supply line from ground surface through the ISL TCB and logistical issues transporting wet concrete from Chai Wan Depot dry batch shotcrete was selected.

Initially it was planned to backfill the gallery with grout however this again has similar logistical issues as concrete, further it was likely to adversely impact on the TBM confinement slurry during excavation. As such the solution that was used came in the form of concrete blocks, these were easy to transport and handle reducing the amount of grout required further it was thought that during TBM excavation that they would remain largely intact thus reducing the likelihood of deterioration to the TBM confinement slurry.

4 CONSTRUCTION

4.1 Preparation and site installation

The works area inside the COB was separated from the ISL behind a 4-hour fire-rated and flood-protected separation wall. Generally the utilities and services were independent from the MTRC operating systems to ensure the existing ISL service remained unaffected; however a low voltage supply and ventilation shaft were shared. Due to limited space and no access for large items at street level the ground freezing plant and most materials were transported from Chai Wan Depot to the COB using MTRC engineering trains in non-traffic hours. The freezing set up spanned three levels inside the COB, at track level the freezing pipes were installed and gallery excavated, above this in the lower mechanical plant room a segregated access and gantry was constructed, the freezing plant was located above this in the redundant upper mechanical plant room. With the exception of a water-cooling tower and an emergency generator installed at street level, the entire site installation was contained within the COB and not visible to the public, see Figure 2.

Prior to drilling the freezing tubes, the gallery extent was grouted with micro-fine cement from ground surface and inside the COB to reduce permeability and groundwater flow and to improve the horizontal

drilling accuracy. The limitation of groundwater flow rate was also a key factor to the success of ice wall formation (Schultz et al, 2008).

4.2 Drilling and installation of freeze tubes and instrumentation

A Wassara W100 down-the-hole water hammer system was used for the majority of the drilling, when soft ground or steel obstructions were encountered this was replaced with coring equipment. Blow out preventers were installed on the COB wall to mitigate the issue of uncontrolled water ingress from the ground.

Sub-horizontal and inclined freeze holes (22 nos.) and monitoring holes (5 nos.) of 33 m length were drilled through mixed ground with 1.5% targeted accuracy. Numerous parameters including the drill length, thrust pressure, torque, rotation speed, water flow and pressure were continuously recorded by specific software to gain further insight to the geology. The as-built alignment of drill holes was surveyed to an accuracy of 0.2 degrees using the Deviflex system. A 3D geometrical computer model was established based on the as built drill hole alignment, the spatial deviations indicated that six additional compensatory freeze holes were required in order to avoid prolongation of the ice wall formation.

Freeze tubes in the form of two concentric pipes were inserted into the drill holes. To avoid introducing obstruction to the subsequent TBM drive, the tubes were made from HDPE (high density polyethylene) as illustrated in Figure 3. The outer pipe of 90/73 mm diameter had a closed end while the inner pipe of 50/41 mm diameter had an open end and could be removed once the gallery was completed reducing further the obstructions in the path of the TBM, the outer pipe then being filled with weak grout. The use of HDPE in lieu of steel was modelled in the design, which showed it did not affect the freezing efficiency. The annular void around freeze tubes was grouted using bentonite-cement mix.

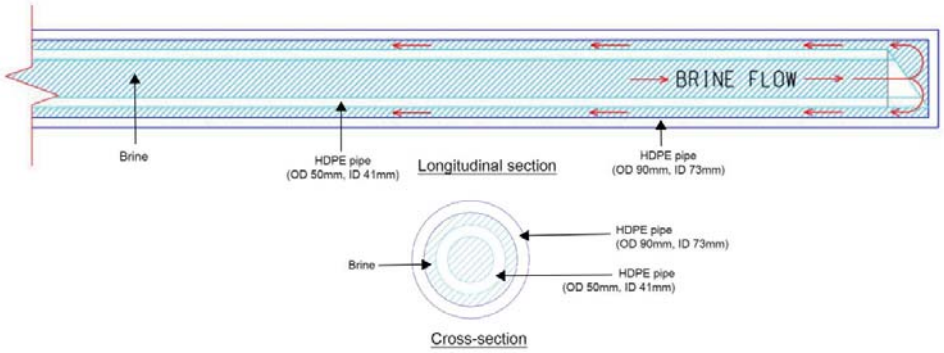


Figure 3: Freeze tube for ground freezing

Temperature sensors at regular 1 m spacing were installed along the monitoring drill holes at oblique angles across the ice wall at locations in accordance with the design. All monitoring data were automatically recorded 24-hours a day real-time which enabled the actual thickness of the ice wall to be monitored and assessed. If, during excavation, the temperature reached preset values, alarm messages would be triggered; the alert-action-alarm (AAA) values at the edge of the ice wall were set as -11.0°C, -10.5°C and -10.0°C.

During the freeze the pore-water pressure of ice ring would increase, especially once the end plug was closed. Daily monitoring of the pore-water pressure inside and outside the ice wall was conducted; in the event of excessive pressure difference the core drainpipe was opened to release the pressure.

An Automatic Deformation Monitoring System (ADMS) comprising strain gauges and prisms was installed in the ISL, surrounding buildings and utilities to monitor for deformation and movement. Direct coordination between the MTRC and DMBJV supervision teams was maintained and actions planned in case the respective AAA values were reached. During the works no significant movements were recorded or AAA values reached.

4.3 Establishment of ice wall

The freeze tubes were connected to the freeze manifold and pressure tested to 10 bar to complete the brine circuit and confirm there were no leakages, the system being close loop. The Brine solution, 400kg of calcium chloride per m^3 of solution, was circulated at temperatures between -25°C to -30°C. The brine was pumped to

the manifold from the freezing units and down the internal freezing pipes to the return apertures at the end of the pipes. On its way back through the annulus between inner and outer pipes, the brine absorbed the heat from the ground; the circulation was designed to flow from the inner to the outer pipes so that maximum cooling capacity was provided to the extremity of the ice plug and walls. The warmer brine returning from the freeze pipes was then re-cooled and re-circulated. During this cycle, heat was transferred from the Brine via the refrigeration unit heat exchanger to the surface cooling-tower using water cooling liquid, once cooled at surface the water was returned to the freezing units to repeat the cycle, the process being similar to the thermodynamic cycle used in refrigeration. It took roughly 8 weeks to achieve the designed ice wall thickness, which varied from 0.5 m for rock to 1.7 m for soil.

4.4 Excavation and obstructions removal

Decompression holes were drilled from the COB head wall through the excavation to facilitate rock splitting; these were backfilled with Bentonite cement to ensure soil and water ingress to the COB were minimised. The gallery was primarily excavated with an electric remote-controlled Brokk160 demolition robot as shown in Figure 4(a); a handheld Darda hydraulic rock splitter was used for delicate excavation. The gallery excavation was round the clock, 24-hour per day, with a day and night shift. The excavation cycle ran from Monday to Friday with the shotcrete lining application and maintenance occurring over the weekend.

The freeze tube manifold was designed to allow each tube to be disconnected from the freezing circuit whilst maintaining circulation in the remaining tubes; this was in case of rupture during excavation in the perimeter tubes. The central plug tubes through the gallery were disconnected and trimmed back to coincide with the excavation progress but were re-connected during the weekend to maintain the ice plug. The high early strength dry-mix shotcrete with 24-hour strength of 20 MPa formed the secondary 120 mm thick lining; this included a sacrificial layer of 20mm for degradation by the ice wall. The shotcreted surface was subsequently covered with thermal insulation to minimize heat transfer from the COB air to the ice wall (Figure 4(b)) thus reducing the load on the freezing system. There was no access for mucking out through the COB so all spoil was removed using specially designed muck skips, to ensure no contamination of the operating railway, these were transported by MTRC engineering trains and removed at Chai Wan Depot.

After 2 months of excavation the location of the steel sets was reached, four steel sets connected longitudinally were found confirming the findings of the geophysical survey. All the steel sets and a number of other steel items including rebar, steel concrete pipes and mesh were successfully removed from the TBM excavation path. Figure 4(c) shows a steel set to be removed from the gallery.



(a) Excavation by demolition robot



(b) View of gallery during construction



(c) Steel set in the gallery

Figure 4: Gallery excavation

The gallery by this point had been excavated to the last steel set, 27 m from the cross over box, however it was planned to excavate to 30m to ensure there were no surprises. The remaining 3 m were abandoned as the actual survey results confirmed the old 1980s gallery was now outside the TBM excavation profile.

During the drilling phase several tools and drill steels had been lost due to the complex geology and left in place steel items, although fishing recovery tools had been used to remove some of these items not all had

been recovered. The team did not want to leave any steel in front of the TBM so a review of the ice wall thickness and design was progressed to see if localised excavations could be made to try and recover the remaining items, this proved possible as the ice wall had continued to grow to sufficient thickness and all of the remaining items were recovered.

4.5 Existing pile survey Hongway Garden

The review of the ice wall thickness that had allowed localised excavation to recover the drilling tools gave rise to the possibility of ascertaining the exact location of the Hongway Garden building caissons utilising the same method. The as built records indicated there were 8 numbers of caisson piles in close proximity to the TBM excavation profile with 3 in particular that it was agreed to investigate as old records should be verified where possible. The gallery was locally extended to survey the critical piles; all three piles were located by the team. The exact locations were set into the survey control around the designed tunnel axis with the results conclusively proving that they were outside the TBM excavation profile giving greater security and further de-risking the works.

4.6 Backfilling and Bulkhead

Upon completion of excavation and obstructions removal from the gallery, it was backfilled with the precast concrete blocks, followed by void grouting with bentonite-cement mix to fill the gaps. The void grouting would ensure the reformation of an intact ground mass to avoid subsequent settlement upon de-freezing of the ice ring and effects on the TBM confinement. The de-freezing operation required shutting down the freezing plant, draining the brine circuit and allowing the ground to thaw naturally; this was confirmed with the continuation of temperature monitoring. A steel bulkhead was installed to ensure that the TBM arrival would not cause slurry ingress to the operating railway due to the confinement pressure required (Figure 7).

5 GROUND FREEZING FOR TBM TUNNEL TO ISLAND LINE CONNECTION

The existing structure of the COB has a diaphragm wall for temporary excavation support with an internal Reinforced Concrete (RC) permanent structure; previously a sacrificial RC panel had been built to accommodate the connection. Due to the tunnel alignment, depth and width of diaphragm wall along with two supporting corbels the TBM cutter head was not able to excavate up to the permanent structure (Figure 5). This meant that there was a span 2m long between the TBM shield and permanent structure that required support during the excavation and construction of the connection with the Island Line.

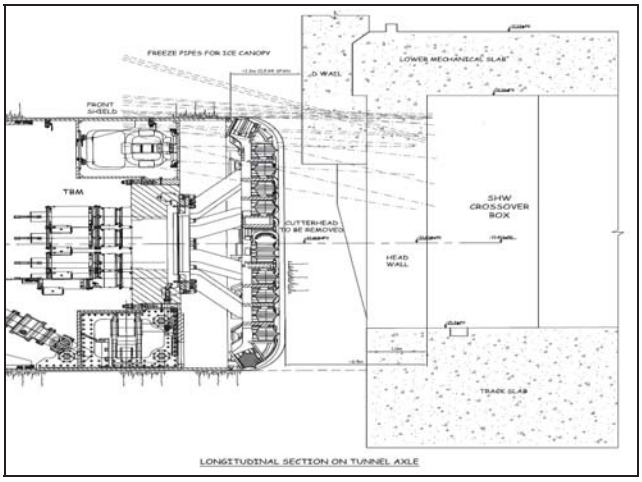


Figure 5: TBM stop location

The geology was slightly to moderately decomposed granite rock overlain by CDG and Marine Deposit at the tunnel crown posing significant risk, as shown in Figure 6. As noted in Section 2.4 a pipe pile canopy and grouting had been envisaged to support the soft and weathered materials at tunnel crown. After the success of

the ground freezing for the gallery excavation it was suggested to reuse the freezing plant to further reduce the construction risks of material ingress and associated settlement.

A detailed comparison of the two options focused on risk and budget was completed which showed that there was no additional cost to reuse the freezing plant and that the design solution was more robust. The team exploited the innovative solution of crown stabilization by an ice canopy for the TBM tunnel connection to the COB structure and ISL. The design required a total of 15 steel freeze pipes to form a pipe roof, however 16 were placed acting as the structural component taking up the soil and water load, as they were steel they had to fall outside the TBM excavation. The frozen soil formed an ice canopy to act consolidate the strata around steel pipes and prevent water ingress and associated ground destabilisation at the crown. Similar to the ground freezing design for steel set removal, finite element and finite difference computer programmes were employed for conducting the mechanical, thermal, and hydrological analyses using the same soil parameters. Figure 6 shows the layout of freeze pipes on-site.

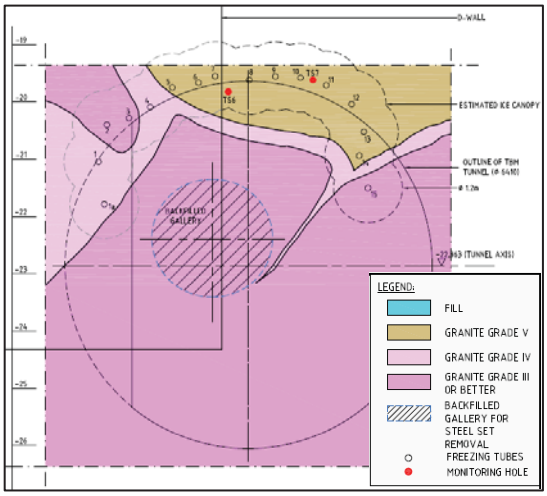


Figure 6: Geological section of tunnel break-out

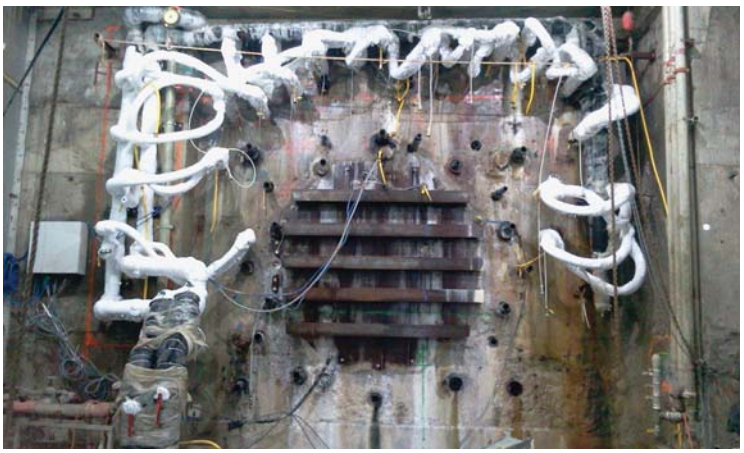


Figure 7: Freezing in COB for tunnel break-out and connection

The freeze pipes had to be drilled within a confined envelope to avoid the TBM excavation and RC structure. There was a minor risk of the TBM rupturing a pipe, so a similar principle was again adopted for the freezing manifold in that prior to TBM arrival the freezing circuit was turned off and the pipes isolated. Upon arrival the TBM stopped at the designated chainage 24 hour surveillance started inside the COB throughout the arrival with a phone to call the TBM Pilot in case of any slurry ingress. Then the confinement slurry was slowly removed from the excavation chamber and replaced with compressed air, this was completed during non-traffic hours to ensure no compressed air leaked into the COB as air is less dense than the slurry.

Next a face inspection was conducted to check the freezing pipes and to ascertain the ground quality, this confirmed 5 pipes were damaged which were then repaired. There was still a risk of ground water and material flowing along the TBM excavation annulus to the excavation chamber; this was mitigated by turning the freezing plant back on and using PU grout to plug the TBM annulus at the excavation chamber. The remaining annulus was grouted with a bentonite cement mix to ensure the TBM shield was properly sealed and to allow the ice wall to connect to it. The grout was allowed to set and then the confining pressure was systematically lowered in increments over a two day period to ensure the water ingress was controllable and no instability ground presented. This was completed without incident and joint approval to progress the connection was agreed, the sacrificial RC wall needed to be broken and opened from inside the COB, followed by excavation between cutterhead and COB wall and concurrent removal of cutterhead to achieve breakthrough. This was in fact facilitated by the gallery as the bulkhead could be removed with the grouted infill blocks easily effectively providing a pilot tunnel for the RC and cutterhead removal.

6 CONCLUSION

To effectively overcome the project constraints, secure the program and mitigate various construction risks, artificial ground freezing for excavation support using brine solution for cooling circulation was adopted on

the MTRCL West Island Line Contract No. 703 to excavate an access gallery for removing steel obstructions to the up track TBM drive, as well as to stabilize the ground for ensuing TBM tunnel connection to existing Sheung Wan Station COB structure. The planning, design and construction processes needed for the ground freezing works have been described in this paper. Comprehensive laboratory tests on frozen and unfrozen soils were carried out to establish the design soils parameters with thermal, mechanical and hydrological analyses conducted for detailed design. The drilling of freeze and monitoring holes, connection and testing of freezing plant, establishment of ice wall, gallery excavation, removal of obstructions totalling 700 kg of steel, survey of existing piles, backfilling of gallery, de-freeze and ground freezing for the TBM tunnel connection to the Island Line were completed predominantly within COB, this along with the majority of the logistics via the IL allowed minimal impact to the Public. Throughout the works, an extensive instrumentation and monitoring scheme was implemented to monitor the ice wall and existing structures and utilities.

The programme was secured with the gallery excavation completed seven weeks early due to detailed planning and shortening of the gallery by 3m, the TBM arrived in position a week earlier than the Project Master Program without incident or delay, this would not have been the case had the steel been left in place, bringing greater security to the hand over date to Operation Division for the new degraded refuge siding in case of train breakdown. Improving on the program has reduced the costs of the planned TBM works; the provisional sum for the gallery was under the planned budget by approximately 10% with the connection ice canopy cost neutral compared to the original grouted pipe pile canopy plan. All of this was achieved due to the active participation of both the Client MTRCL and Contractor DMBJV and the partnering sprit with which the problem was approached.

ACKNOWLEDGEMENT

The permission by MTR Corporation Ltd to publish this paper is grateful acknowledged.

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Confinement Pressure for Face Stability of Tunnel Boring Machine (TBM) Tunnel Excavation Under Hong Kong's Western District

A.C.M. Tsang & C.D. Salisbury
MTR Corporation Limited, Hong Kong
S.S.M. Yeung
Dragages Hong Kong Limited, Hong Kong

ABSTRACT

Confinement studies for the proposed tunnel excavation using a Slurry Mix Shield Tunnel Boring Machine (TBM) on the Mass Transit Railway Corporation (MTR) West Island Line extension project 703 were developed according to GEO Report no. 249 "Ground control for slurry TBM tunnelling" in the temporary works design stage. This paper presents the overall approach, development and implementing of the confinement studies with conclusions based on the comparison of the theoretically predicted values to measured data obtained on site. The proposed details were reviewed by Buildings Department (BD) and Geotechnical Engineering Office (GEO) based on the guidance provided in GEO Report No. 249. During implementation of the output from confinement studies, practical control measures were developed and applied to manage the process of excavation. Mitigations such as routine physical monitoring on the ground surface; technical monitoring on confinement parameters at the cutterhead including measurements of excavated and grouted volume; and quality control on slurry and grout were significant for the deliver of the successful completion of the TBM excavation. Comparison was made between the predicted values on settlement and ground loss to the measured settlement and values determined by back-analysis. It was concluded that the proposed approach to determining confinement parameters and measures for site control were adequate and could be used as a model for similar projects with TBM tunnel excavation in an urban environment.

1 INTRODUCTION

1.1 Project background

MTR WIL Contract 703 – Sheung Wan to Sai Ying Pun tunnel construction is the first part of the MTR West Island Line (WIL) Project to extend the existing Island Line in Hong Kong from the current Sheung Wan station (SHW) to Kennedy Town. It is located in the urban area of the Western District of the Hong Kong Island between Sheung Wan and Sai Ying Pun (see Figure 1).

A Slurry Mix Shield Tunnel Boring Machine (TBM) has been used by the main civil works contractor, Dragages – Maeda - BSG Joint Venture (DMBJV) for the excavation of a section of the tunnel to cater the mixed ground conditions. This paper describes the principles adopted in the technical assessment and the site control measures aiming to maintain the ground stability and minimize potential impact to the public.

1.2 Principles of Confinement Study and Site Monitoring

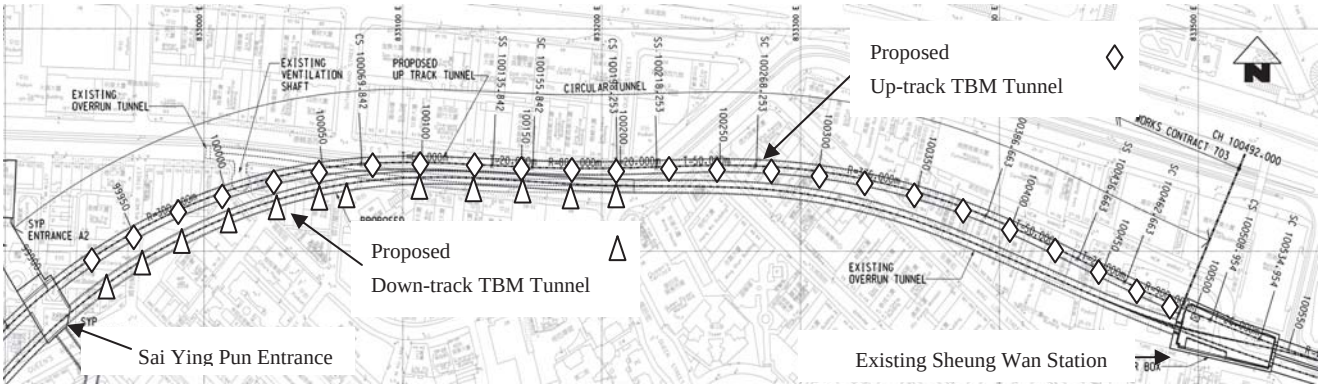
The internal diameter of the new tunnel is 5.45 m and lies between 28~38 m below ground level. It is located at the partially weathered zone with a combination of grade III granite and completely decomposed granite with some minor intrusions into overlaying marine deposits. The stability of the excavation face was maintained by the pressurized bentonite slurry at the TBM cutter chamber. Critical sections with adverse ground condition and high building surcharges along the tunnel alignment were selected for the detailed assessment.

The preliminary assessment was carried out by the in-house design team of the contractor and verified with parallel calculations in line with GEO Report No. 249 to estimate the minimum pressure to maintain the face stability. In addition, a settlement analysis has been done to predict the ground settlement and volume loss of the tunnel excavation. The estimated confinement pressures were further verified by the existing building impact assessment to ensure the influence to the existing buildings is within an acceptable level. The documents have firstly undergone consultation with reference to the empirical methods mentioned in GEO Report No. 249 to establish confirmation of compliance with this relatively new and largely untested GEO guidance report. Further to justifying the compliance to the defined local condition, the estimation on confinement pressures was re-assessed with the identical approach stated in the GEO guidance report.

The TBM tunnel excavation was closely instrumented, monitored and checked to confirm that the actual results were in line with predictions. The monitoring system comprised of various settlement, tilting and groundwater monitoring points in vicinity of the tunnel; Automatic Deformation Monitoring System (ADMS) for sensitive buildings and existing operating MTR Overrun Tunnel (ORT); and detailed instrumented parameters from the TBM and slurry treatment plant (STP) control systems. The measured values were reviewed by comparing to the predicted figures and volume loss worked out by back-analysis. Adjustments were made to cope with different scenarios encountered on site to improve the overall performance. The excavation of the 545m long up-track TBM tunnel was completed successfully between late October 2011 and early March 2012.



(a) Key plan



(b) Layout plan and details

Figure 1: MTR WIL project contract 703

2 CONFINEMENT STUDIES FOR TBM EXCAVATION

2.1 Background

The TBM face pressure confinement studies were prepared by Bouygues Travaux Publics, the parent company of Dragages Hong Kong, prior to the tunnel boring. These studies were designed to determine the range of recommended confining pressures for the slurry, compressed air and grout to be used during tunnelling to meet the criteria applicable to the project regarding to the local geological condition, surcharge

loading and recommendations stated in GEO Report No. 249. The key objectives of the confinement studies were to define parameters as follows:

(a) minimum required face pressure against face collapse mechanism
(b) maximum face pressure against heaving and blow-out mechanism
(c) minimum compressed air face pressure for hyperbaric intervention
(d) relationship between confining pressures, volume loss and settlement

The particular criteria for settlement and volume loss in this contract, i.e. general case: maximum ground settlement < 15mm; and volume loss < 1.5%, in order to minimise the impact to the nearby structures, buildings and utilities. Specific criteria with more stringent limits have been applied to particular building structures close to the TBM tunnel excavation in considering of the type of foundation and local geological conditions.

2.2 Selection of critical sections for the confinement studies

With regards to the predicted geological condition of full or mixed face of Grade III Granite, completely decomposed granite (CDG) and marine deposit (MD) in combination with the information of the estimated loading from the buildings in varies type of foundation. Critical sections along the tunnel alignment were selected for the confinement studies based on the following criteria:

(a) Geotechnical characteristics of the cross section:
- Type of soil and mechanical properties at the tunnel face
- Ground cover and hydrostatic pressure
- Soil stratigraphy

(b) Location of sensitive buildings and heavy building loads

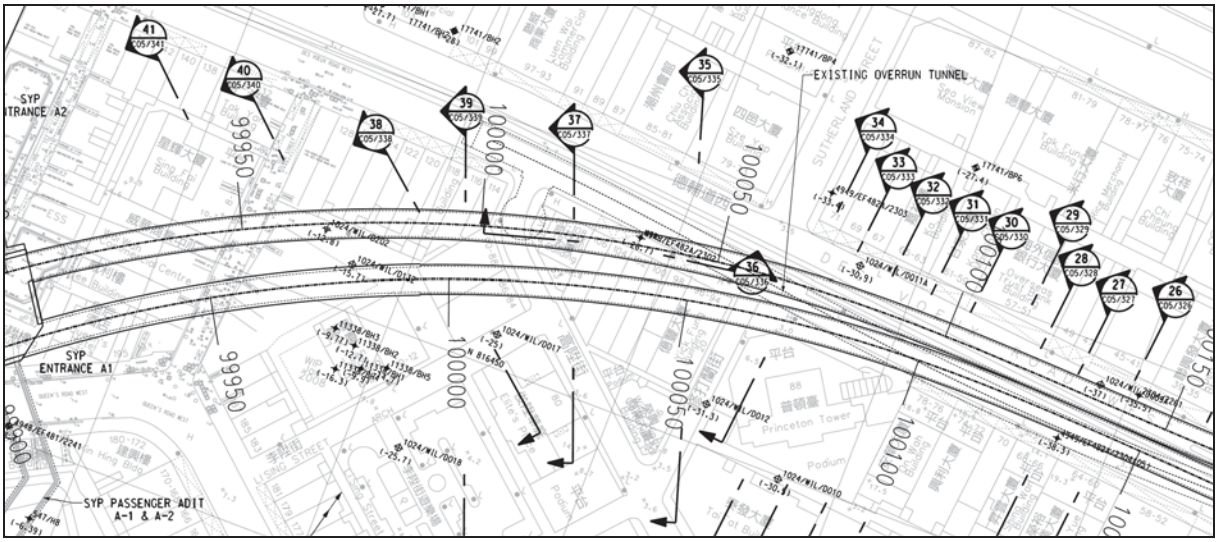


Figure 2: Layout plan of sections selected along the TBM tunnel alignment

The sensitivity of buildings had been determined during the preliminary design stage by a desk study of the existing building records held by the BD in combination with a “Worst Case” ground movement resulting from an effective 3% face loss.

In the confinement studies, 15 sections, as shown in Figure 2 and Figure 3, were selected and analysed along the 545 m long proposed tunnel alignment. The studies covered the cumulative effect of the first and second TBM drives as well as the dismantling of the existing tunnel.

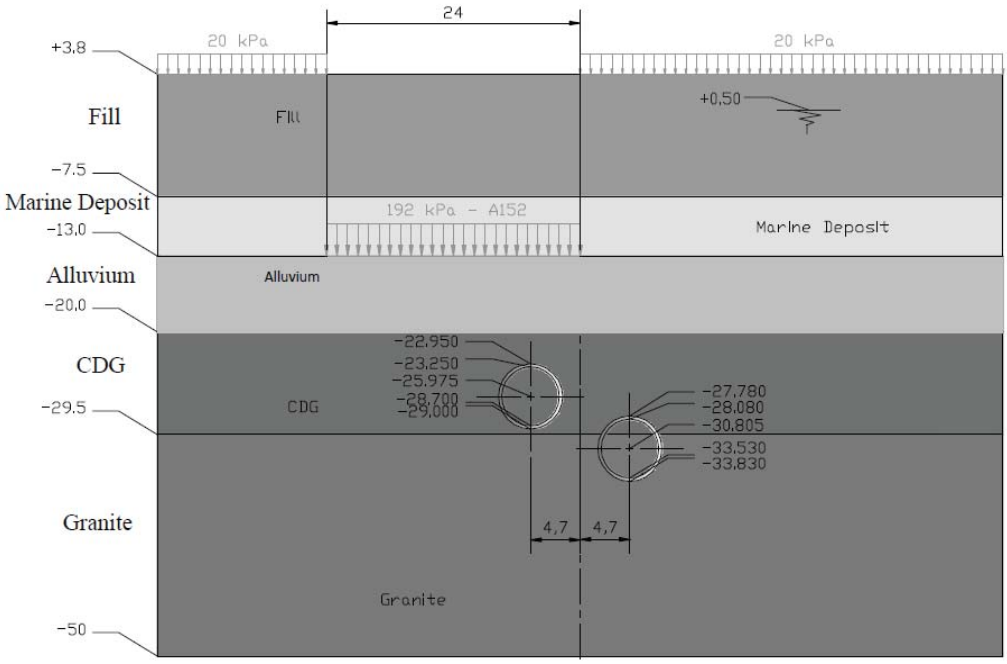


Figure 3: A selected section along the TBM tunnel alignment for the confinement study

2.3 Empirical method

Face pressure assessment for ultimate limit state (ULS)

To ensure the tunnel face stability and prevent major ground loss, the minimum face pressure was determined by an effective stress calculation for frictional soils. The ULS calculations were completed based on the method proposed by Anagnostou & Kovari (1994). This method adopts the silo theory by Janssen (1895) in assessing the failure mechanism which is illustrated in Figure 4. TBM face pressures were evaluated by achieving a limiting equilibrium of the prismatic body and the wedge. In the calculations, the minimum pressures were further divided into factors due to water pressure, soil pressure, surcharge and allowance for variations. It was found that as for similar projects, the water pressure was the dominant factor which contributes 90% of the face pressure.

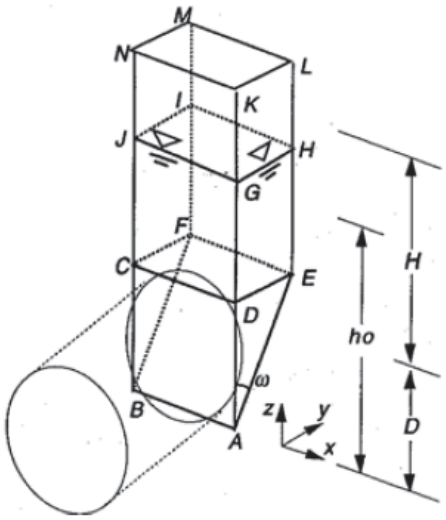


Figure 4: Diagram of 'Silo Pressure Theory' by Janssen (1895)

Face pressure assessment for serviceability limit state (SLS)
The tunnel is located at a densely populated area. Stringent control of ground movement is required to minimise the effect to the nearby buildings, structures and utilities. The face pressures were assessed for SLS based mainly on the method of Proctor & White (1977). The resulting pressures were in general higher than that of ULS. It is stated in the GEO report no. 249 that the volume loss expected using this calculation method would be in the region of 1%.

Summary of face pressure assessment
Table 1 below summarizes the characteristics of the performed calculations as mentioned in the above sections.

Table 1: Summary of face pressure assessment

	Ultimate Limit State	Serviceability Limit State
Effective stress calculation (Frictional soils, drained conditions) (CDG)	Method based on Anagnostou & Kovari, to evaluate the minimum face pressure to ensure the stability of the excavation face.	Method based on Proctor & White, to ensure that the volume loss is approximately 1% and thus to limit the surface settlements
Total stress calculation (cohesive soils, undrained conditions) (MD and alluvium)	Method based on Kimura & Mair, to evaluate the minimum face pressure to ensure the stability of the excavation face.	Method based on Kimura & Mair, to ensure that the volume loss is acceptable via the definition of a so-called “Load Factor”

2.4 Numerical analysis

FEM analysis for volume loss and settlement prediction
From the above calculations, it was found that the confinement pressures were governed by the results of SLS analysis. In addition, cross checking has been made using computer software on 2D Finite Element Method (FEM) based on the principle of Convergence – Confinement method. With the use of FEM analysis, a precise simulation to the ground condition was provided in the model considering of a variety of factors, such as multi soil layer properties, asymmetric surface surcharge and deep foundation loads. Also interaction between the confinement pressures, volume loss and settlement can be predicted with high accuracy. Based on the design assumptions on the maximum volume loss and settlement of 1.5% and 15mm respectively, in general, the computed confinement pressures were found relatively higher than the results of the mentioned ULS and SLS methods in the above sections. This indicated that a more conservative estimation from the FEM analysis was adopted.

For a number of critical buildings where the tunnel was passing underneath or between the piles, further 3D FEM computer software analysis were carried out to assess the effects and sensitivities due to different factors mentioned. The analysis aimed to determine an approach which would reduce the building impact by the ground treatment and TBM overpressure. The results of the study with an overpressure of 150 kPa showed that the volume loss could be reduced effectively to 0.4% by the overpressure while the effect of the soil stiffness was less significant. It was concluded that the TBM overpressure was the dominant factor in controlling the volume loss, induced settlement and impact to building.

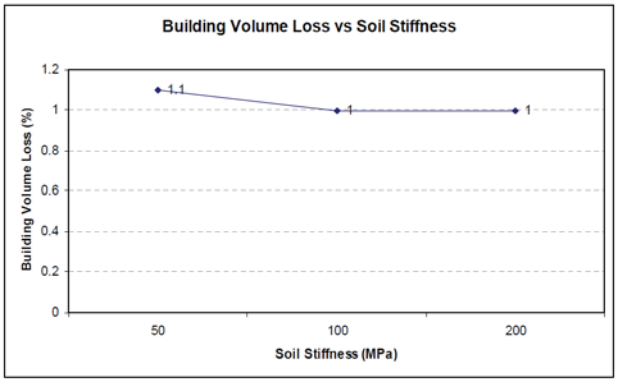


Figure 5: Building volume loss vs soil stiffness

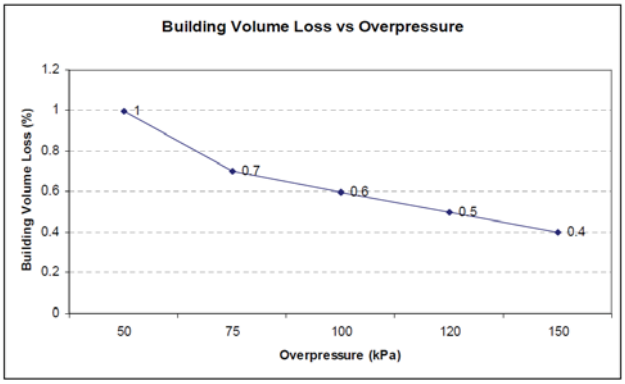


Figure 6: Building volume loss vs overpressure

FEM analysis for existing tunnel

Part of the TBM tunnel was excavated adjacent to the existing MTR Island Line tunnel with about 2.5 m clearance. Prior to the excavation, a FEM model was set up to simulate the deformation of the existing tunnel due to the effects of the new TBM tunnel to be constructed. The analysis with the using of FEM package was considered to be conservative with the assumptions as follows:

- the closest distance between two tunnels.
- the worst geological configuration
- beneficial effect of the ground treatment was not taken into account.

It was expected there would be a de-confinement of the soil around the newly bored tunnel leading to the assumption that part of the soil support to the existing tunnel would be lost in result of the ovalization to the profile of the existing tunnel. The checking was carried out in considering of the effect of the recommended slurry pressure applied at the excavation face, i.e. 40 kPa, and equivalent confinement overpressure applied on the ground, i.e.100 kPa. The output of the analysis indicated that the ovalization to the existing tunnel would be 0.15% or 4.3 mm in tunnel diameter which is within the required limit of 1% or 25mm.

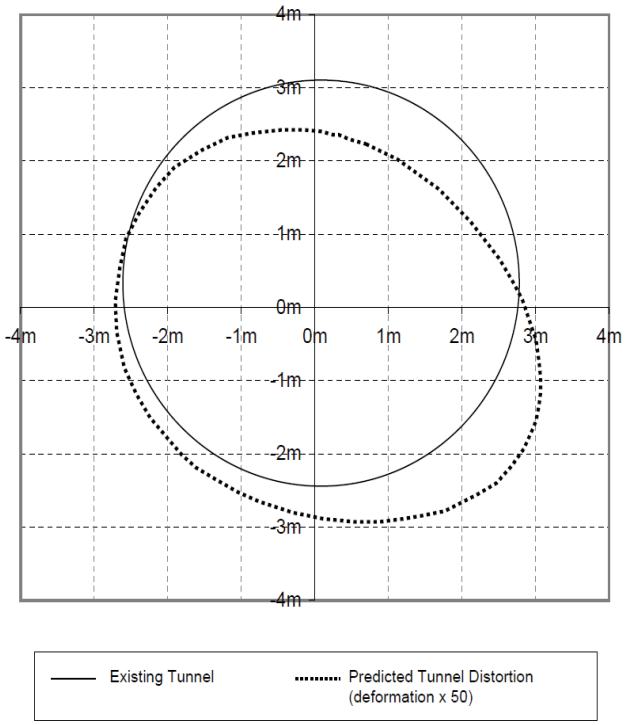


Figure 7: Estimated distortion of existing tunnel

2.5 Confinement profile and presentation of the results

Confinement pressures obtained from the above calculations were consolidated. The confinement pressures between the critical sections were referred to the confinement study with similar geological conditions and loadings. The remaining tunnel reaches between the critical sections were derived by interpolation. Therefore a full confinement profile was developed.

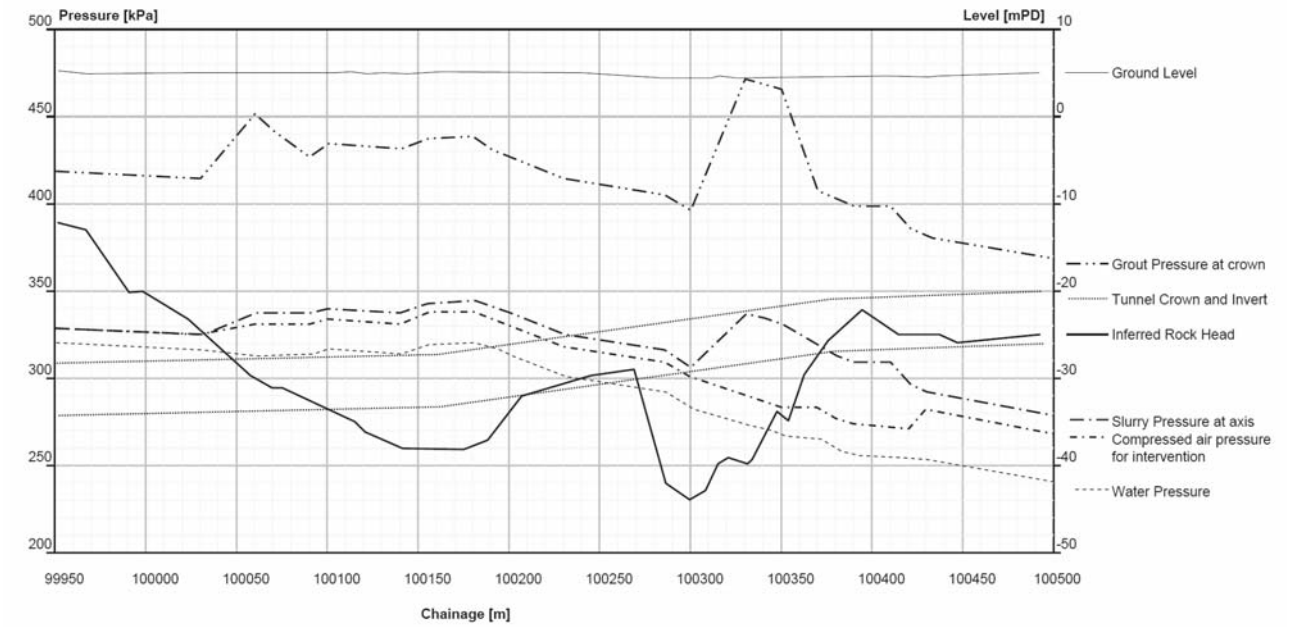


Figure 8: The confinement profile for up-track TBM tunnel

2.6 Equivalent Overpressure

The concept of equivalent overpressure introduced by Aristaghes & Autuori (2003) has been adopted in the 2D FEM analysis. It is based on the assumption that the ground deformation is a combined effect of excavation, lining erection and annulus void grouting. Aristaghes & Autuori have published in their paper that the minor variations in face pressure has almost no effect on settlement.

The key function of face pressure is to maintain the face stability for excavation. The direct impact on settlement is from the radial pressure (slurry pressure) around the shield and grout pressure around the lining. The equivalent overpressure is a single input parameter for 2D FEM analysis taking into account the effect of the slurry and grout pressures. It simplifies the communication between the designer and the works team in correlating the design target pressure and actual operating pressures to be applied for excavation.

3 IMPLEMENTATIONS OF CONFINEMENT STUDIES AND SITE CONTROL MEASURES

3.1 Monitoring and controlling of confining pressures during excavation

By combining the determined confining pressures from confinement studies with geological profile and foundation of the buildings along the tunnel alignment, the information was summarised and presented as the longitudinal confinement profile for the operation (see Figure 8).

The TBM tunnel excavation was monitored by both the production and technical teams on site throughout the entire operation. Major control measures on the operation of the TBM and Slurry Treatment Plant (STP) at ground level focused on 4 key activities as shown in Table 2 below.

Table 2 Control measures in TBM and STP on key activities of operation

Activities	Control measures in TBM	Control measures in STP
(i) Slurry confinement	Slurry pressure managed with compressed air bubble pressure and level.	Quantity & quality of slurry Volume and characteristics of excavated materials
(ii) Compressed air confinement during measured intervention for maintenance	Compressed air pressure during balancing and intervention.	N/A
(iii) Mortar grouting	Quantity & quality of grout Grout pressure	Volume and characteristics of excavated materials
(iv) Segments installation	Survey monitoring on navigation of TBM and ring built quality.	N/A

The parameters for the above control measures in TBM and STP were recorded in a database and available in the format of graphical output for real-time monitoring. The recorded data was also reviewed as the TBM progressed and used in the back-analysis.

For the particular confining pressures during excavation and intervention, those parameters were monitored and recorded in the real-time monitoring system of the TBM and STP, with the provided graphical information indicating the condition prevailing at the face of excavation. Over excavation and loss of confinement could be identified efficiently to avoid any consequences which could bring adverse effect to the surroundings or public.



Plate 1: Top view of slurry treatment plant at ground level in Sai Woo Lane site

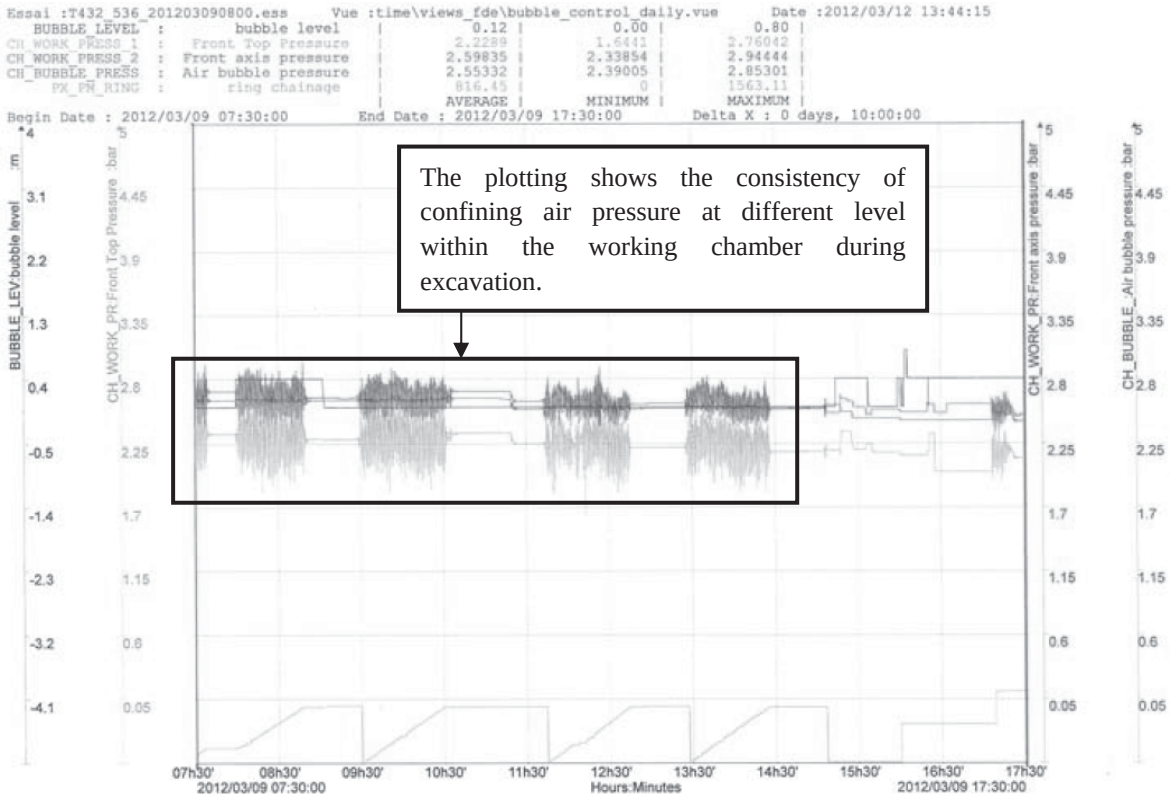


Figure 9: Typical graphical output of parameter showing the condition of confinement for real-time monitoring

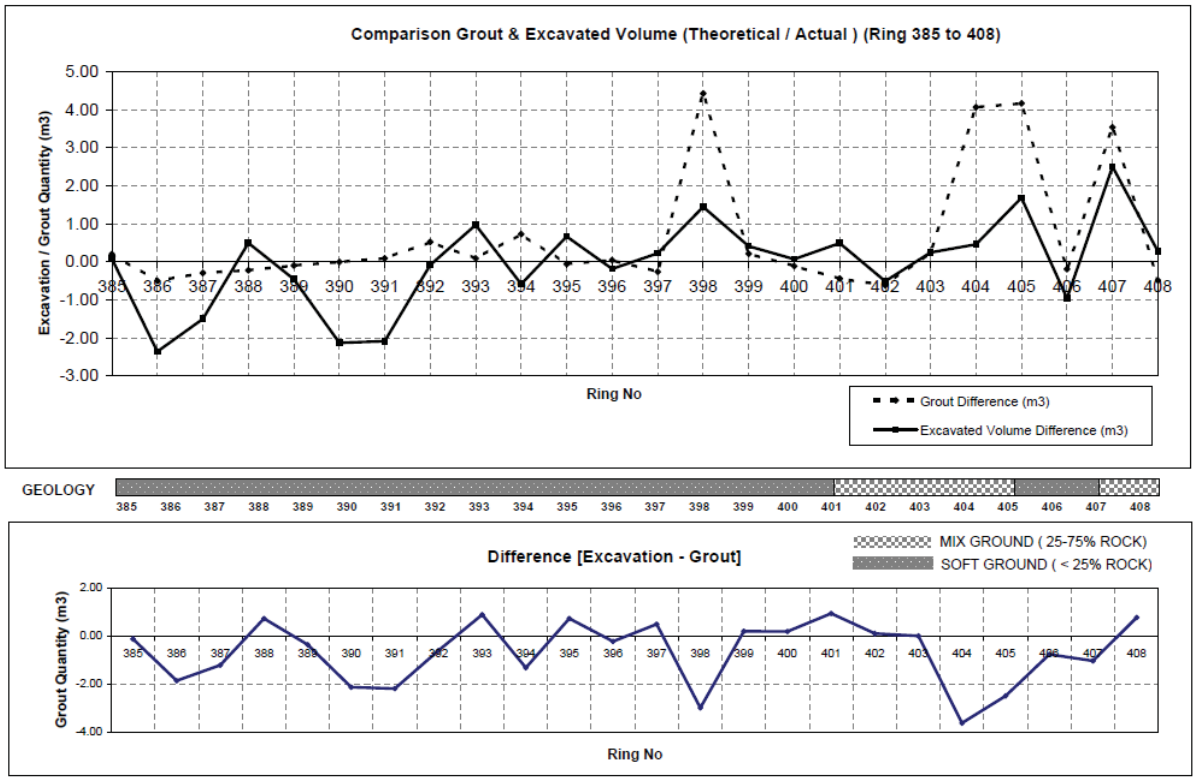


Figure 10: Graphical output showing the grouted volume is sufficient to excavated volume.

The volume of excavated materials was monitored and recorded by the Excavation Management Control (EMC) system. The system makes a comparison between the actual quantities of excavated material with the theoretical values. It also correlates the excavation volume with the back grouting volume to ensure that no voids remain behind the tunnel lining throughout the excavation.

3.2 Site Monitoring and risk management

Instrumentation monitoring

A total of approximately 2000 instrumentation monitoring points were installed along the alignment and about 1000 monitoring points along the TBM drive were measured on a daily basis for checking the effect of the TBM operation. Survey instrumentation including Automatic Deformation Monitoring System (ADMS) was installed along the proposed TBM alignment for monitoring of potential induced ground movement. The ADMS is an advanced monitoring system developed to suit the purpose of monitoring the movement at buildings. Total robotic stations and prisms were installed at specific building for 24-hour continuous monitoring for potential movement at the existing buildings. A separate ADMS system was installed at the specific section of the existing operating MTR ORT at an interval of every 5m to 10m. This was used to monitor the local deformation effect of TBM excavation at the location of less than 2.5m adjacent to the ORT.

TBM Tunnel Emergency Action Plan

As the TBM excavation operated underneath or closely adjacent to the foundations of existing buildings, a comprehensive risk review and emergency action plan for corresponding preventive measures and procedures were prepared with the main identified risks as follows:

(1) Collapse of tunnel leads to significant inflow of soils and water.
(2) Movement of existing building.
(3) Bentonite egress (egress into surface or ingress to the adjacent railway tunnel).
(4) Clashes to pile foundations of existing buildings.

4 COMPARISONS OF THEORETICAL RESULTS TO MEASUREMENTS OBTAINED FROM SITE

4.1 Settlement back-calculation analysis

Back-analysis was carried out to compare the predicted cumulative settlement and ground loss to actual values obtained during TBM up-track excavation. The predicted cumulative values were estimated assuming the TBM excavation was performed at both up-track and down-track tunnel.

Table 3: Summary of results of back analysis in comparing to predicted values

Locations (Section no.)	Predicted values (for both up-track and down-track tunnels excavation)		Actual values (calculated during up-track tunnel excavation)	
	Maximum settlement (mm)	Ground loss (%)	Measured settlement (mm)	Ground loss (%) (worked out from back analysis)
Sutherland Street (Section 34-34)	-14.9	1.07	-3.0	0.24
Up Track ch.100095 (Section 30-30)	-15.1	1.07	-6.0	0.44
Queen Street (Section 24-24)	-14.9	1.07	-4.0	0.29
Tung Loi Lane	-9.10	1.13	-2.0	0.25

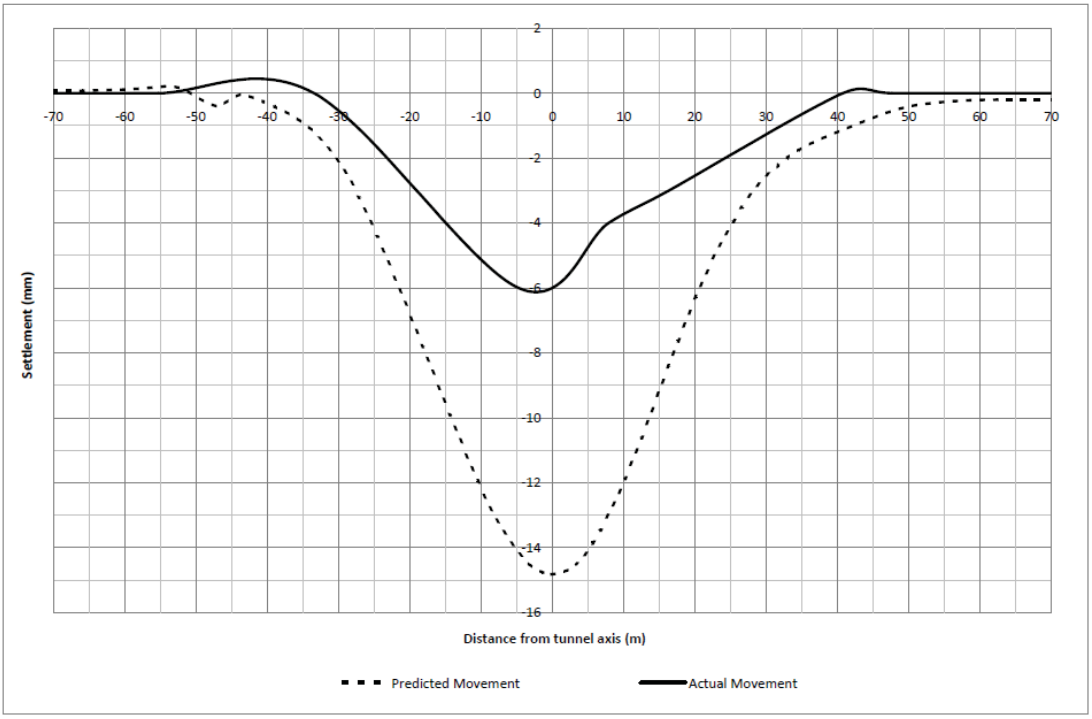


Figure 11: A graph for comparison of actual / predicted surface settlement

Table 4: Summary of predicted and actual ground loss and settlement values for Building A77

Values	Volume Loss		Maximum Settlement	
	Greenfield (%)	Building (%)	Greenfield S_{max} (mm)	Building S_{max} (mm)
Predicted Value	0.10	0.40	-1.0	-5.0
Measured Value	0.08	0.26	-1.0	-3.3

Note: TBM tunnel excavation performed adjacent to the pile foundation of the building in a distance of approx. 100mm.

From the results of particular analysis at 5 critical locations along the TBM alignment (Table 3 & 4), it showed that ground loss was under control with the pressure determined in confinement study. The differences observed from the comparison of results were considered to be initiated by the diversity of actual geological condition near surface adjacent structures and soil-lining interaction encountered on site. However, the analysis verified that the measured settlement and ground loss are within the design predictions as mentioned above.

4.2 Confining pressures

The settlement criteria could be achieved by a different combination of the slurry and grout pressure as explained in section 2.6. For the concept of equivalent overpressure, if the slurry pressure has been reduced, the grout pressure shall be increased in order to achieve the same effect on settlement. It allows the flexibility to the works team in controlling the confinement pressures. A very high slurry pressure is usually not desirable due to the risk of slurry egress to the ground surface through unexpected open paths at different type of interface within ground strata. Throughout the operation on site, a relatively lower slurry pressure and a higher grout pressure was applied for excavation.

5 CONCLUSIONS

For the recent successful completion of TBM excavation achieved in up-track tunnel, the settlement back-analysis proved that the applied confining pressure and controls were sufficient to be applied to the actual site condition.

The results of confinement studies verified that the analysis was carried out with appropriate assumptions, i.e. criteria on ground loss < 1.5% and maximum settlement < 15mm, which could be modified and optimized according to the actual site and geological condition encountered. Furthermore after the completion of the first drive, with the actual achieved ground loss of < 0.5%, it proved that the proposed control measures on confinement performed on site were practical and appropriate to be applied to projects with similar background.

The results of confinement also indicate that the applied analysis was more than sufficient compared to the obtained measurement considering that ground movement is the dominant factor for the operation. As observed, drilling and grouting could bring adverse effect to the control of confinement condition due to the creation of potential open-path to the ground surface. Therefore, it is suggested that extensive pre-grouting works proposed to be undertaken in the urban environment should be critically reviewed before implementation, particularly for achieving the most efficient ground treatment in providing stability and controlling ground movement for the tunnel excavation.

Moreover, the environmental impacts to the community of extensive pre-grouting works, often undertaken from existing carriageways, is obvious, and from the findings of this study are largely avoidable when it is not required to secure compressed air intervention. Proposed expenditure on pre-grouting works should be critically reviewed and reconsidered for reallocation to TBM specification improvements where appropriate. Other areas where the expenditure could be better utilized is in ground investigations to identify the geological condition along the alignment of TBM tunnel.

Based on the completed studies and tunnel excavation works in WIL contract 703, the results should allow contractors to process technical documents more efficiently with the government authorities in the forthcoming projects. The mentioned details in the development process and implementation could be used as an example for projects with TBM tunnels in urban environments in the future.

ACKNOWLEDGEMENTS

The authors would like to share our gratitude and thanks to all the colleagues in MTR and Dragages-Maeda-BSG Joint Venture who gave support and guidance on the preparation of this paper as well as appreciation to S. Minec and P. Autuori of Bouygues Travaux Publics and Geotechnical Consulting Group (Asia) Ltd for their inclusive technical support and 3-D Finite Element Modelling analysis used in the confinement studies.

Lastly, we offer our thanks and regards to the Highways Department Railway Development Office, Buildings Department and Geotechnical Engineering Office for their comments and support throughout the consultation process of the technical documents related to the TBM drives for WIL Contract 703.

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Risk Management and Construction of Drill and Blast Tunnel in Shallow Rock Cover

M. Baribault & M. Knight
Dragages Hong Kong Limited & Bachy Soletanche Group Limited
W.S. Chow
MTR Corporation limited, Hong Kong

ABSTRACT

During the excavation of the drill and blast tunnel section of the West Island Line Contract 703 from Sai Yin Pun (SYP) to Sheung Wan Station (SHW), the Contractor encountered a 66m-section with unexpectedly low rock cover. An extensive supplementary ground investigation was undertaken to assess the geological conditions. This investigation revealed the presence of completely decompose granite in the tunnel face under a 35 meter water head, bringing with it a high risk of groundwater draw down, settlement of the surrounding buildings and the possibility of collapse. A number of technical solutions were developed and jointly reviewed by the Contractor and the Engineer. By considering the risk, cost and planning impact a solution comprising of preliminary heavy ground treatment followed by mechanical excavation and partial face blasting was selected. The temporary support was a succession of canopy pipe piles supported by steel arch ribs. The risks were mitigated by permanent on-site technical supervision in the role of correspondent between the site team and design team. This allowed the design team to understand the exact ground conditions encountered as the work progressed, in order to adjust the design as necessary.

1 INTRODUCTION

The West Island Line project will extend the Island Line Service by approximately 3.3km, from the existing Sheung Wan Station to the future Kennedy Town Station via SYP and Hong Kong University. Contract 703 comprises of two 760m long single-track tunnels between SHW Station and SYP Station. The tunnels are being constructed by drill and blast method in the rock section and TBM in the soft and mixed ground from the King Georges Fifth Park (KGV) and Sai Woo lane (SWL) construction shaft.

During excavation of the drill and blast section of the uptrack tunnel, a zone of low rock cover was identified by probe drilling in advance of the excavation. Further investigation revealed this zone to extend 66m ahead, with completely decomposed granite in the excavation face and a 35m groundwater head above. The works were suspended to ensure the safety of the excavation prior to an alternative design being prepared and agreed.

This paper presents the geological conditions assumed during the initial and detailed design and compares them with the conditions revealed during construction and supplementary geotechnical investigation works. The alternative technical solutions are presented and the how these were judged according to risk, cost and programme is discussed. The eventual construction method is reviewed along with the manner in which the safety and effectiveness of the works was secured through full time on-site technical supervision.

2 DESIGN ASSUMPTIONS

2.1 Geotechnical investigation data available prior to contract award

During the preliminary and detailed design phases, the ground conditions in the 150m long uptrack drill and blast tunnel between SYP and SHW were determined using the data from 8-no drillholes, as-built bored pile records from 2-no buildings and photolineaments identified in the Mid-Levels Study (GCO, 1982). There were no drillholes within 10m of the tunnel alignment for a 130 m section of the tunnel.

2.2 Ground conditions assumed prior to contract award

Based upon the information described above the Contractor produced a Geotechnical Baseline Report (GBR) to state the geotechnical conditions anticipated when executing the underground works. According to the GBR, this section of the uptrack tunnel drill and blast tunnel would be excavated through granitic bedrock with more than 75% of the surface area formed by intact blocks. The remaining material (i.e. soil) is confined to discrete seams. Rock cover would be greater that 4m (0.66 x tunnel diameter; 0.66 D) and directly above the rockhead was expected to be mixed ground, having 25-75% boulders of intact rock. Excavation was envisioned to be drill and blast with temporary support provided by rock bolts and shotcrete according to the mapped Q-values.

2.3 Main difficulties during the construction phase

During the tender 3-no. additional drillholes were proposed on the uptrack tunnel alignment at First Street to fill in the gap previously identified and described above. However, due to site constraints such as the presence of a large seawater cooling main, congested utilities and existing temporary traffic management schemes only one drillhole was eventually completed.

Excavation of the uptrack tunnel started on 24 January 2011 and progressed from west to east. Due to the uncertainty regarding the ground conditions excavation proceeded using a combination of low angle long probes and high angle short probes in the tunnel crown at 10 m intervals. On 18 February 2011, excavation reached a zone where the advanced probing indicated the rockhead level ahead was less than 1D (6.2 m). A series of 3-no. 10 m long probes were drilled at 45-degrees in the tunnel crown and indicated 0.66D (4 m) of rock cover directly ahead. Excavation continued using lattice girders and spiles rebar for support, until the first blast on the morning of 24 February 2011 encountered a previously unidentified drillhole. Approximately 5 m^3 of sand and water flushed into the tunnel before the drillhole could be sealed with a packer. Meanwhile, a standpipe approximately 10m ahead of the excavation recorded a 1.7m drawdown in water level. Excavation continued but eventually stopped at Ch99782.4 on 15 March 2011 when advance probing indicated the rock cover ahead was now less than 4 m.

3 SUPPLEMENTARY GROUND INVESTIGATION AND GEOLOGICAL INTERPRETATION

3.1 Supplementary ground investigation

Due to the constraints described previously, a supplementary ground investigation was carried out from inside the tunnels. The investigation was undertaken using a combination of probing and rotary coring with full length sampling, from the uptrack tunnel face and transversely from the then fully excavated downtrack tunnel. A cross adit from the downtrack tunnel was excavated at a location where the rock cover was known to be adequate. This allowed works to proceed on two faces and further probing and coring was carried out from East to West. Samples were taken to a laboratory to test for PSD, moisture content, density, shear strength, permeability PH and Cl content. A summary of the investigation work is provided in Table 1 below:

Table 1: Summary of supplementary ground investigation

Type	Quantity	Method
Probe Holes	54	Horizontal and inclined probe percussion drilling logged on site by geologist
Rotary Drillholes	13	Horizontal and inclined rotary drilling with full length sampling using core barrel and mazier sampler.

3.2 Geological interpretation

A detailed geological model was developed based upon the investigation results and the joint sets identified on the face mapping. The additional ground investigation data was correlated with the geological structure and indicated a zone with low rock cover, 66m in length, directly ahead of the excavation face. This section of the

uptrack tunnel is affected by a main series of fractures, sub parallel to the tunnel alignment, particularly between Ch99792 – Ch99815 where the joints are very closely spaced. Preferential weathering of the joints and fractures significantly reduce the rock mass quality, bringing highly to completely decomposed granite into the tunnel crown and left wall. A long section of the geological conditions is provided in Figure 1 below. To facilitate the overall design, transversal cross sections were prepared every 3 m as shown in Figure 2a. The ground conditions were continuously reviewed with the latest jointing and weathering exposed on the excavated tunnel face (Figure 2b).

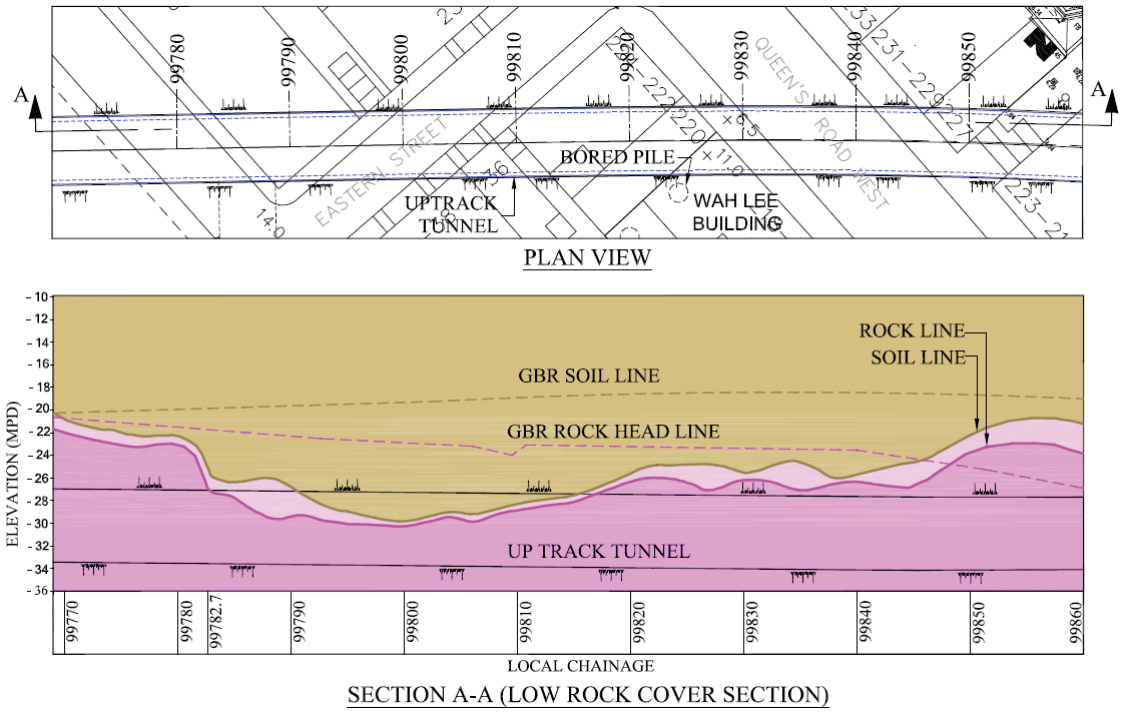
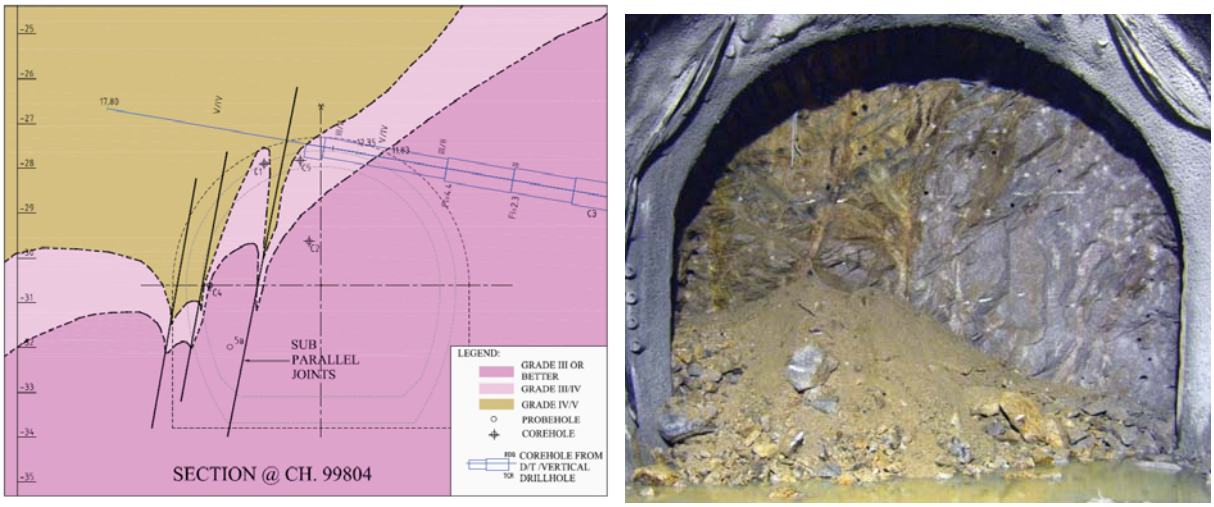


Figure 1: Longitudinal long section



(a) Anticipated Geological model (b) Excavated tunnel face with sandy spoil

Figure 2: Typical transversal geological models with corresponding tunnel face picture

4 RISK ASSESSMENT

4.1 Main risks and constraints

The following risks and constraints were identified:

- *Water drawdown and water inflow into the tunnel*: The water pressure above the tunnel crown is 3.5 bar. To manage the risk of uncontrolled water inflow, the design was evaluated and reviewed critically by both internal and external design consultants. According to laboratory tests on samples recovered during the supplementary ground investigation, the permeability of the soil mass was between 10^{-5} and 10^{-6} m/s. A permeability of $5.6.10^{-8}$ m/s or less was required to limit the water drawdown at surface to less than 1m. This corresponds to a very stringent requirement of <0.2 litre/min/m within any hole drilled into the soil mass in the tunnel crown. Therefore, it was necessary to grout within the tunnel crown before support installation.
- *Stability of Existing Building, settlement and collapse*: The temporary support design was based on the empirical Q-system approach. Additional checks by numerical analysis using PLAXIS and SEEP/W were carried out. The analysis concentrated on the Wah Lee Building which has one pile within 1m of the tunnel. According to the PLAXIS analysis a minimum cohesion (c') of 20 kPa was required for the grout treated completely decomposed granite. The results from laboratory tests on samples taken verified that treatment had improved c' to 90 kPa, therefore exceeding the pre-defined criteria. It should be noted that for building with shallow foundations (i.e. not found on rock) there was no direct load imposed on the bedrock and the tunnel excavation under an appropriate support system would not induce any adverse effects on these buildings.

4.2 Alternative solution

Four alternative construction methods were developed to continue the excavation. A detailed risk analysis was undertaken by comparing the design feasibility, planning impact, the cost and the main risks anticipated. The details are provided in Table 2.

Table 2: Developed methods for tunnel soft ground excavation

	Method	Design Consideration	Work duration (Month)	Cost	Major risk	Risk analysis
1	Grouting, canopy roof, mechanical excavation with partial blasting.	1. Ground treatment 2. Additional cross adit temporary rock support design and Blast Assessment Report	10	Lowest cost	Poor grout efficiency leading to water drawdown, settlement and / or collapse	High risk
2	Grouting / ground freezing and mechanical excavation	1. Same as method 1 2. Lab testing for ground freezing 3. Temporary rock support design for ground freezing	16	Highest cost	Overall project delay	Lowest risk
3	New TBM launch from SYP to SWL Shaft	1. Shaft civil works 2. TBM launching design 3. Ground treatment 4. Design enlargement tunnel for TBM assembly with BAR – design temporary support 5. TBM arrival design in SWL 6. Revised permanent lining	4	Lowest cost	Delays due to design approvals, lining works, logistic problems and interface works with other contract	Highest risk
4	Third TBM launch from SWL shaft to SYP (after 1st and 2nd original launch scope completion)	1. Same as method 3 except points 4 and 5.	7	Medium cost	Overall project delay	Medium risk

Methods 1 and 2 were planned from 2 working faces by excavating an additional cross adit from the downtrack to the uptrack tunnel. Method 2 minimized the risk of settlement and water drawdown but required a long site installation time before starting the excavation. Moreover the planning impact and cost were the highest. The TBM methods 3 and 4, required a long preliminary design and procurement phase significantly delaying the overall project planning and bringing problems with the lining works and the interface with the WIL 704 contract. Accordingly, none of these solutions were selected. Method 1 was the most efficient in terms of design feasibility with an acceptable programme impact and relatively low cost. However, due to the inherent uncertainty of ground treatment, this method brought high risk. This risk was to be managed by continuous site supervision to understand clearly the ground condition and adapt the design to ensure the stability of the excavation. Moreover, a freezing alternative was put in place in the event of grouting failure.

5 CONSTRUCTION PHASE

The 66 m of affected tunnel was divided in 7 vaults of 9 m length with canopy pipe roof and ground treatment required for each. Four canopies were constructed from the east side and three from the west side (from the additional cross adit) with a planned extra 3 m for the breakthrough.

5.1 Ground treatment

The basis of the design was to build a grout block of 1/2 the tunnel span above the crown and outside the side walls prior the installation of the temporary support. The main target of the ground treatment was to reduce significantly the rock and soil permeability to $5.6.10^{-8}$ m/s for ground stability during the support installation (Hartwell et al, 2011). The grout works has been adapted to improve the ground stiffness for the stability of the existing building. The ground treatment was conducted in two main stages:

- *Stage 1*: All the treatment was carried out from the downtrack tunnel, intercepting the uptrack tunnel; refer to the Figure 3 for the transversal section. The grout holes were drilled perpendicular to the sub parallel fractures at the uptrack tunnel for better treatment efficiency. A standard pattern of 90 holes in a grid of 1 m x 1.25 m was used for all vaults.
- *Stage 2*: Treatment holes were drilled from the uptrack tunnel face and transverse to the pattern in stage 1; refer to the Figure 4 for the transversal section. This second stage was used to check the permeability reduction and to treat any localized untreated features. A standard pattern of 30 holes in a grid spacing of 3 m x 3 m was used for all vaults.

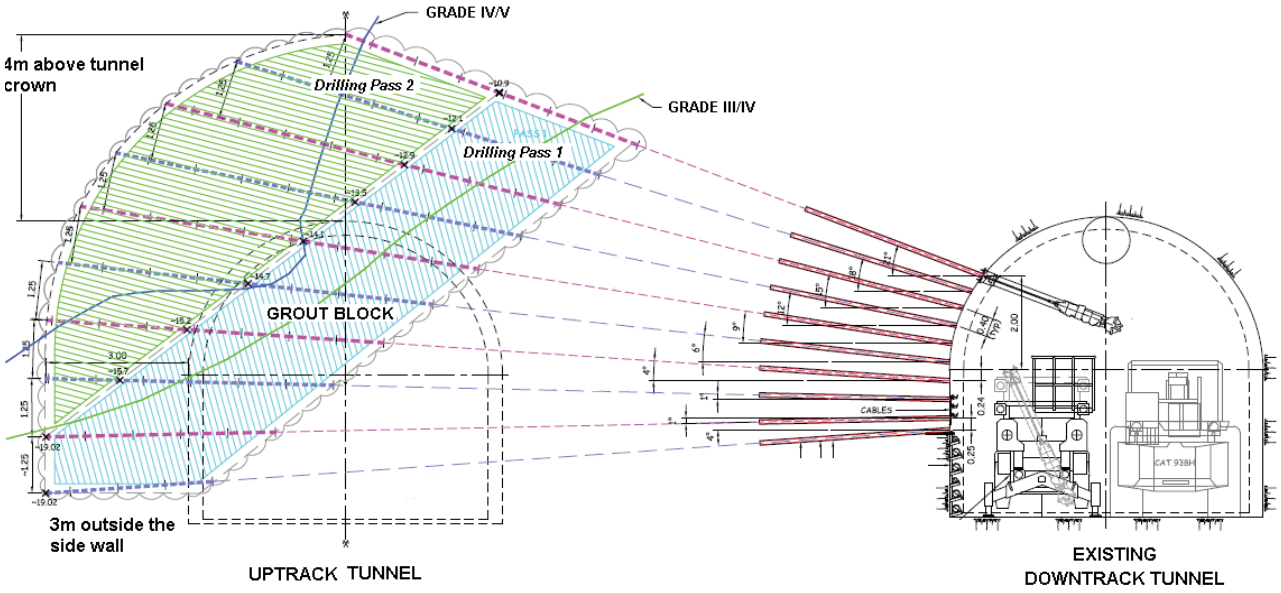


Figure 3: Transversal section of Stage 1 grout works

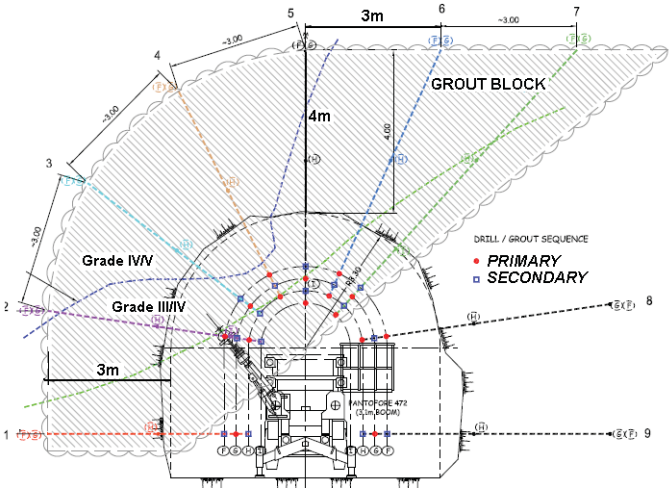


Figure 4: Transversal section of Stage 2 grout works

The grout layout plan included 2 successive stages of primary and secondary treatment. A tertiary stage was reserved for any area where anomalous grout take was recorded. Due to high water inflows observed when drilling in the soft ground, the grout holes were drilled using a multi stage drilling pass process. The injection was carried into an open hole with a mechanical packer (i.e. without a tube a manchette) because periodically the holes collapsed. Three different grout mixes were used:

- *Cement OPC bentonite* grout and *microfine* cement grout to open and seal the main fractures in the granite rock mass. A water cement ratio of 1.6 to 2.0 was adopted for better grout penetrability and water plugging.
- *Silicate* to impregnate the coarse grained soil. A stabilizer was used to ensure setting in less than 20 min.

The grout injection process was governed by both pressure and volume requirements. A sufficient volume of grout had to be injected to ensure the required replacement factors were achieved (10% for the weathered rock mass and 20% for the completely decomposed granite). The grout pressure needed to be high enough to overcome hydraulic resistance and create widening of the pre-existing joints. The grout performance was validated by a series of test holes and permeability testing (lugeon test) through the grouted block to check if the general water inflows were below the criteria.

5.2 Temporary support

A conservative modified empirical Q-system approach (Barton, 2002) was adopted to allow for potential adverse ground conditions. Different categories of support ranging from isolated feature dowels to lattice girders were implemented on site according to the as-mapped Q values. Where the rock cover is less than 1/3 of the span, a heavy duty lattice girder support was required. Prior to the suspension of works 7-no. lattice girders had already been installed. Where the rock cover was less than 0.8 m, the revised design comprised a combination of canopy pipe piles supported by steel rib arches spaced at 1.2m. The pipe piles for every excavation vault were extended to a maximum length of 12m as this is the maximum length that can feasibly be installed in mixed ground. An 3 m overlap of the pipe roof was maintained between vaults. A single layer of pipe piles, spaced at 300 mm, were installed in the mixed ground profile. A double layer spaced at 200 mm was installed where completely decomposed granite was encountered near the tunnel crown. Lastly, water drains were installed above the canopy to reduce the water pressure within the grouted zone and minimize any soil displacement during excavation.

5.3 Excavation stage

Because of the variability of the ground at the tunnel face, both mechanical excavation and rock blasting methods were used. The lower section of tunnel face comprised of competent granite and was excavated by drill and blast. A non blasting zone equal to half the diameter of the blasting zone was maintained between the upper limit of the blasting and the rockhead level. This method minimized the vibration within the soft ground and tunnel support area. The soft ground in the tunnel crown was excavated prior the blast of the lower bench.

6 ONGOING RISK MANAGEMENT DURING CONSTRUCTION PHASE

To minimize the risk during the construction phase, continuous technical supervision was provided by a team of geotechnical engineers, grouting engineers, site geologists and technical assistants. Coordination was established to check the latest ground conditions to adjust the ground treatment, temporary tunnel support design and excavation sequence as necessary.

6.1 Site technical information / main difficulties

The main information collected on a daily basis was:

- Detailed drilling logging of the grouting hole (Penetration rate / geological logging) to map precisely the rock cover.
- Water inflow measurements from every hole to define the main target zone for treatment and design a suitable grout mix.
- Detailed grout pressure and volume take variations from one stage to the next, to ensure complete treatment within the open joints and soil mass.
- A general water seepage assessment from the tunnel crown and walls in the proximity of the ongoing grout works to infer the water movements in the ground.
- Detailed logging of the pipe roof drilling to both ensure full coverage of the canopy in soft ground while optimizing the design to minimize installation into a competent rock mass.
- Geological mapping undertaken every 1.2 m after each steel arch installation. The blast and non-blast zones were defined at this stage.

The main difficulty encountered was the interception of sand during the drilling process with the occasional occurrence of heavy running sand. As a result grout methods were modified to improve the permeation.

6.2 Change in design from observation

The information collected was analyzed and interpreted in real time to optimize the design and planning while managing the risks. Changes in the design included:

For the Grout works:

- An increase or reduction of the treatment grid according to the water inflow measurements and ground conditions.
- The tube a manchette method was not adopted on site due to the collapsible nature of the soil. Instead, the method by advance stage drilling with a PVC casing installed in the grout hole to reach the unstable soil above the tunnel crown have been implemented. This method allows a selective injection within the soil mass.

For the Support installation:

- Pipe roof canopy for every vault was installed only in the exact location of the affected zones. Where soft ground was encountered, the pipe roof spacing was reduced.
- Additional grouting through the pipe roof was implemented to recompact the surrounding ground which had became disturbed due to the repetitive drilling of the pipe installation.

For the excavation:

- Mapping of the soil and rock interface in the face to allow blasting and reducing the amount of mechanical splitting.

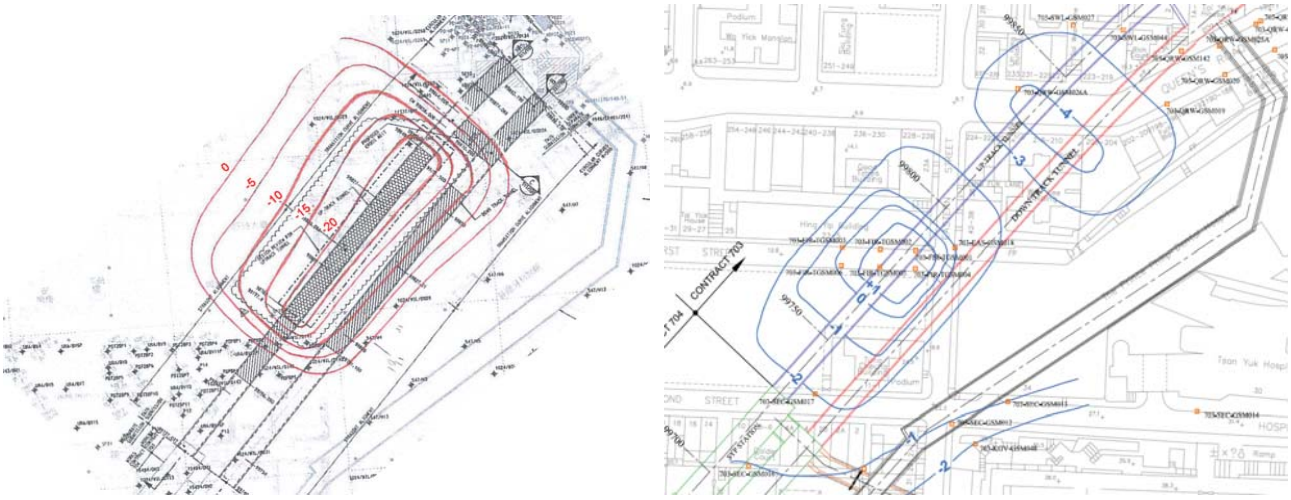
- Modification of the excavation cycle to avoid any disturbance. First the completely decomposed granite was excavated and safety shotcrete applied. Then the lower bench was blasted, followed by scaling, arch installation and final application of shotcrete.

6.3 Instrumentation and monitoring

During the work, systematic instrumentation monitoring was carried out. The following measurements were made on a daily basis:

- Building settlement and tilting
- Ground settlement and lateral movement
- Groundwater pressures and levels
- Convergence inside the tunnel
- Vibrations (during blasting)

In the event that any measurement exceeded a pre-defined Alert, Action or Alarm (AAA) level then an automatic SMS would be sent to the response team to take action according to agreed plan. During the works no instruments exceeded any AAA values. The ground settlement contour plans at design stage and work completion show very good performance in the Figure 5.



(a) Design stage settlement (b) Actual settlement

Figure 5: Settlement contour Plan

7 CONCLUSION

To effectively overcome the low rock cover conditions encountered a method of mechanical excavation with partial blasting, heavy temporary support and preliminary ground treatment was chosen. The risks of water inflow, tunnel collapse and damage to buildings and utilities were managed by continuous technical supervision. The grout works were optimized by precisely identifying where the soil mass was located during the drilling of grout holes and the pipe roof. The grout mix design and methods were adapted to achieve a permeability reduction to $5.6.10^{-8}$ m/s and stabilize the ground for the support installation. The pipe pile supports were exactly orientated in the affected section of the tunnel crown and properly post grouted to stabilize the disturbed soil. The excavation was inspected after each excavation cycle to ensure that was no seepage or instability in the exposed face. The excavation was completed in April 2012. No ground movement, building settlement or sign of building distress were recorded. No material instability, running sand and deformation was recorded in the tunnel face and crown.

ACKNOWLEDGEMENTS

The permission by MTR Corporation Ltd to publish this paper is gratefully acknowledged.

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Detecting Adverse Rock Condition ahead of Tunnels by Interpreting Jumbo Percussion Drill Logs

P. Barmuta & A.S. Maxwell
Maxwell Geosystems Ltd.

ABSTRACT

Most often geological site investigation for tunneling projects does not include directional cored bore holes covering the whole future tunnel alignment. In that case a need arises to test the rock mass condition ahead of the tunnel by short probes together with tunnel advance. In Hong Kong tunneling practice this need is recognized particularly in case of undersea tunnels and specified by the Client in conditions of contract. However, the understanding of probing ahead is limited to visual logging of color of flush, grade of chippings and penetration rate with the use of stop watch by the geologist during percussion drilling. Visual logging proved to be efficient and accurate in detecting extreme rock mass condition e.g. completely decomposed rock. Yet from the other hand it appeared to create a significant safety hazards for the logging geologist who works close to maneuvering booms and rotating string at noise level from hammers above 150 Db. In such hard working condition the quality of the records can also be affected. It is presented in this paper an interpretation of Jumbo percussion probing to detect adverse rock mass condition ahead of the tunnels as an alternative to visual logging.

1 INTRODUCTION

Analysis was carried out on data recorded by AMV Jumbo two boom machine: P – penetration rate (m/min), HP – hammer pressure (bar), FP – feed pressure (bar) and RP – rotation pressure (bar). Selected cases were used to test the developed data processing algorithm in addition to the automated interpretation provided by Bever Control software. The results of the tested algorithm show the adverse rock condition can be inferred from the percussion drill logs with confidence, hence the presence of the geologist at the face when drilling could be eliminated.

A significant advantage of this system was the ability to interpret not just probe holes but also the drill holes belonging to regular grouting rounds. As such, probing on HATS 2A project was undertaken as part of regular grouting works and did not cause any additional delay to the production cycle. The statistical interpretation methodology was largely based on works by Schunnesson (1997). In his approach the penetration and rotation pressure can be assumed as dependable variables while the rock resistance, hammer pressure and feed pressure as well as drill string length are the independent variables. By removing hammer, feed and length effect on penetration rate the remaining variations can be inferred to correspond to changes of rock resistance.

2 ALGORITHM OF INTERPRETATION AND COMMENTS

2.1 Combining a representative sample (Combined Sample - CS) from selection of probe logs

The sample that will provide a reference during further processing needs to include a representative of all expected penetration (P) hence the probes have to intersect all possible weathering grades, intensity of fracturing and rock lithological types. These extreme rates do not need to appear in the Combined Sample in large numbers yet they need to be present e.g. a thin seam of completely decomposed rock within predominantly fresh to highly weathered granite provides full range of penetration rate in the sample. In that sense completeness of the Combined Sample can be verified by geological mapping of the tunnel section from which the probes were selected. Practically it may not be possible to assemble a sample without a histogram bias towards higher or lower penetration rates. This effect can be mitigated when processing the data. Typical distribution of pairs (P vs HP) in the Combined Sample is presented on the Figures 1 and 2.

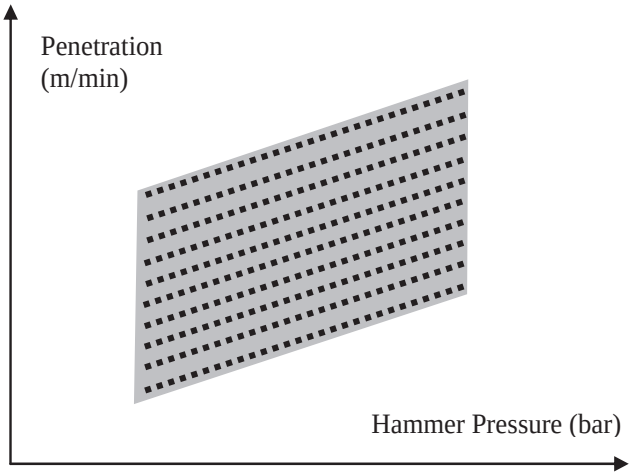


Figure 1: Hypothetical results of drilling through materials of various strengths

Note: Each string of dots represents dependency between P and HP for a uniform material of particular strength.

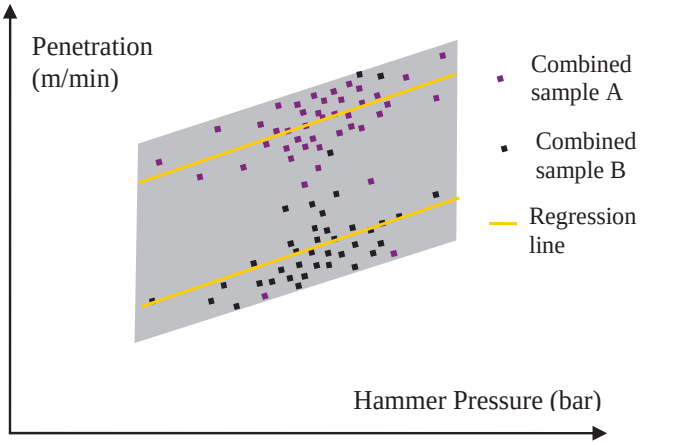


Figure 2: Graphical presentation of penetration and hammer pressure pairs of the combined sample illustrating the bias in data distribution.

Each string of dots on Figure 1 represents P vs HP dependency when drilling through uniform material of particular strength. The highest string of dots represents the strongest material while the lowest string of dots represents the weakest material. The Combined Samples A and B on Figure 2 are limited to a small "clouds" of all possible readings demarcated as grey area. Both include points corresponding to the weakest and the strongest material yet each shows significant bias of the distribution of P values, A towards the weaker material, B towards the stronger material.

2.2 Filtering out the readings related to factors other than drilling (e.g. changing rod)

Filter criteria are determined by analysis of graphic presentation: penetration P, hammer pressure HP and feed Pressure FP versus probe hole length L of all probes included in the Combined Sample. The selection of filter minimum and maximum threshold values can be also aided by analysis of histograms of P, HP and FP.

Filtering criteria are used as an input to Excel custom filter option and applied both to combined sample and particular probe log being investigated.

2.3 Correcting the Combined Sample and probes logs of length effect

Filtered data of the Combined Sample is used to obtain regression equation representing dependency between penetration and probe length. This dependency is machinery specific and is well researched. It is related to increasing weight of the drilling string and increasing friction area between the string and the probe hole wall. The linear regression represents the best fit for the effect of drill string length on penetration rate. The coefficient of determination R^2 is low and generally irrelevant due to large variation of the penetration related to other than length factors.

The factors obtained from Combined Sample linear regression are used to correct the data of the string length effect both in the Combined Sample and in the particular probe log being investigated. The correction is independent to the intercept of the linear regression, hence there is no distortion of the data related to Combined Sample not covering uniformly the different rock condition e.g. harder rock is represented by far more numerous readings than the weaker rock.

2.4 Correction of hammer pressure effect on penetration

The dependency between penetration and hammer pressure and penetration and feed pressure is determined by arranging the P and HP as well as P and FP pairs from the Combined Sample and obtaining linear regression. Hammer effect is usually the strongest and needs to be removed from variations of P. For the purpose of the interpretation the linear regression is assumed to adequately represent the relationship. The

coefficient of determination R^2 of the linear trend line is very low and is inapplicable in general due to large variations of penetration related to variables other than hammer pressure. The trend can be used to correct the penetration values by subtracting the portion explained by the regression i.e. the difference between the regression value at particular hammer pressure and any reference value e.g. average penetration $P_{av} = (P_{max} - P_{min})/2$. The selected reference level, as well as the intercept of the regression have no impact on the final result of the interpretation as they are reduced during final scaling of the penetration to the 0 – 1 a range when using Equation (1). The residual penetration rates can be understood as deviation from P_{av} (or other reference value) related to variations of rock quality. This regression trend does not represent rock mass of average quality (due to the likely bias of the Combined Sample) so the residual penetration can not be used directly for evaluating rock quality. It can be considered as an interim parameter only in the processing.

The correction of hammer pressure effect is carried out both on the Combined Sample and on particular probe log being investigated.

2.5 Scaling the L and HP corrected penetration to 0 – 1 range

Scaling of the corrected penetration (often found as normalization) can be performed by application of Equation (1) to L and HP corrected log of particular probe under investigation.

$$P_i^{scaled} = \frac{P_i - P_{\min}}{P_{\min} - P_{\max}} \tag{1}$$

The P_{max} and P_{min} for scaling purpose are derived from the Combined Sample after L and HP effect correction. Scaling the corrected penetration to the standardized range e.g. 0 – 1 provides an easy tool for interpreting the meaning of the parameter in terms of rock mass condition. Assuming the Combined Sample includes the whole range of expected penetration rates, P_{max} then $P_i^{scaled} = 1$ represents the worst condition (completely decomposed rock or extremely highly fractured rock with thick clay coating on joints) while the P_{min} and $P_i^{scaled} = 0$ correspond to the best rock condition (fresh an hard rock with few joints and scarce or no infill on joints). Using the scaled parameter instead of the absolute value of penetration allows also for comparing the logs and correlating the 0 – 1 range with Q-value or RMR scale, as well as with Rock Grade scale.

3 EXAMPLE OF PERCUSSION PROBING INTERPRETATION

A minor fault encountered in tunnel N of HATS 2A (Sandy Bay to Cyberport) (Figure 3) was used to test the efficiency of interpreting algorithm.

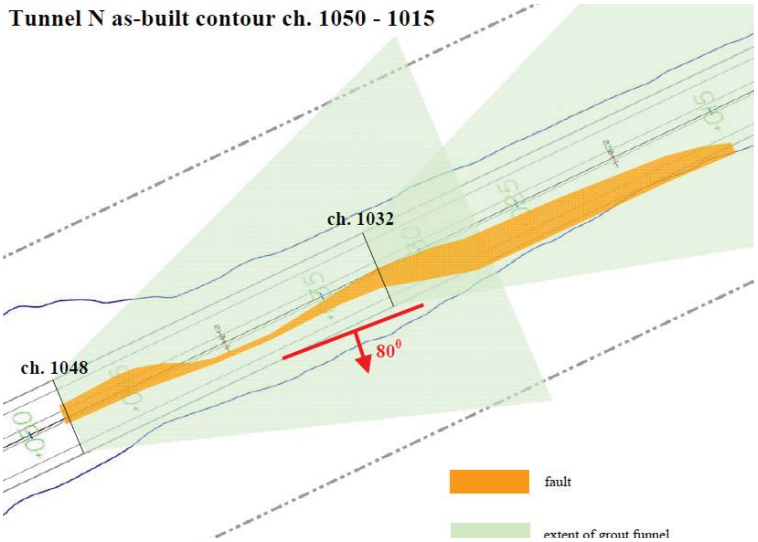


Figure 3: Fault zone encountered in Tunnel N

There were two distinctly different rock types covered by the probes that should appear in the interpreted probing results. The rock within the fault was extremely highly fractured, grade I to III, fine grained tuff with abundant unconsolidated clay coating on joints. Total clay content was evaluated as much as up to 50%. The surrounding rock was grade I, fine grained tuff, with scan line estimated RQD around 80%. Discontinuous decomposed rock coating was present on a small percentage of the joints.

From the positions of the probes against the fault zone (Figure 4) it was inferred that probe 12 was fully within the stronger rock mass while probe 14 encountered fault zone material along part of its length.

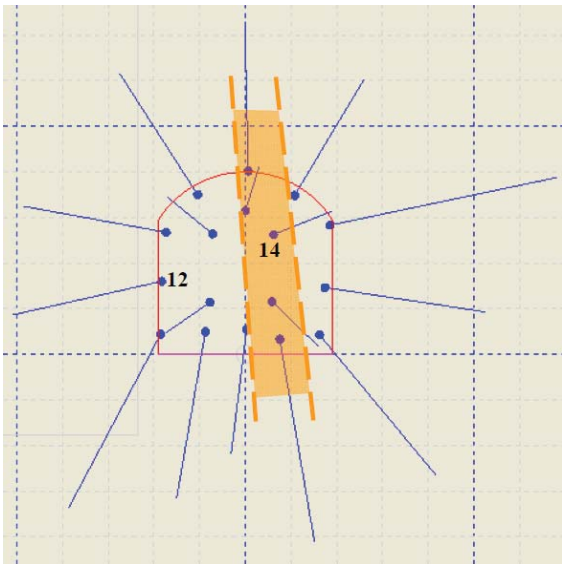


Figure 4: Position of probe holes of Probing 15 at chainage 1032

Note: Probe 12 is located fully within stronger rock. Probe 14 intersects weaker rock of the fault.

The Combined Sample consisted of data of all 17 grout holes drilled at chainage 1032 of Tunnel N. Filtering criteria – the threshold maximum and minimum values of P, HP and FP – were selected by analysis of raw data graphs of the probes and histograms. Example HP graph of probe 12 shows minor fluctuations of working HP within 100 – 110 bars (Figure 5) and peaks related to rod changing, hence data containing HP values below 100 bar could be safely removed from Combined Sample as well as from particular logs. The example log of penetration from probe 12 and 14 (Figure 6) indicates the penetration (and conjoined HP and FP) lower than 0.5 m/min could be filtered out from the Combined Sample and from logs. The upper range filter was assumed as 5 m/min as the upward peaks were related rather to probe cleaning. The histogram plot of penetration from the Combined Sample and from probe 14 (Figure 7) shows the expected different distribution. Note that the extreme values of penetration are still present in the Combined Sample.

Penetration logs of probes 12 and 14 were corrected for the effect of increasing drill string length (Figure 8) and subsequently for the effect of hammer pressure variations (Figure 9). The regression line shows Length correction significant as it may reach up to 0.7 m/min – 17% of the whole range of the filtered penetration 0.5 to 4.5 m/min. The P vs HP linear regression indicates the possible hammer correction up to 1.5 m/min (37%) within the range of working hammer pressure. The final scaled penetration parameter is independent from intercept of the linear trend.

The corrected values of penetration in the Combined Sample were used for selection of P_{max} and P_{min} values for further scaling the P record of particular probes 12 and 14 into a range 0 – 1. The obtained parameter after applying 6 point moving average (together 180 mm) is presented on Figure 10 and proves distinct difference between probes that coincides with their spatial position: 12 within surrounding competent rock, 14 within fractured an partially weathered rock of shear zone.

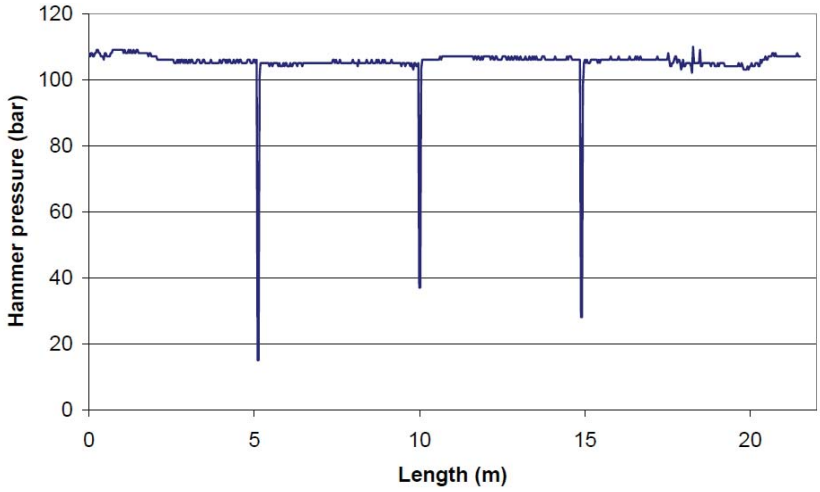


Figure 5: Example log of hammer pressure vs length in probe 12 shows minor fluctuations of HP and peaks related to rod changing

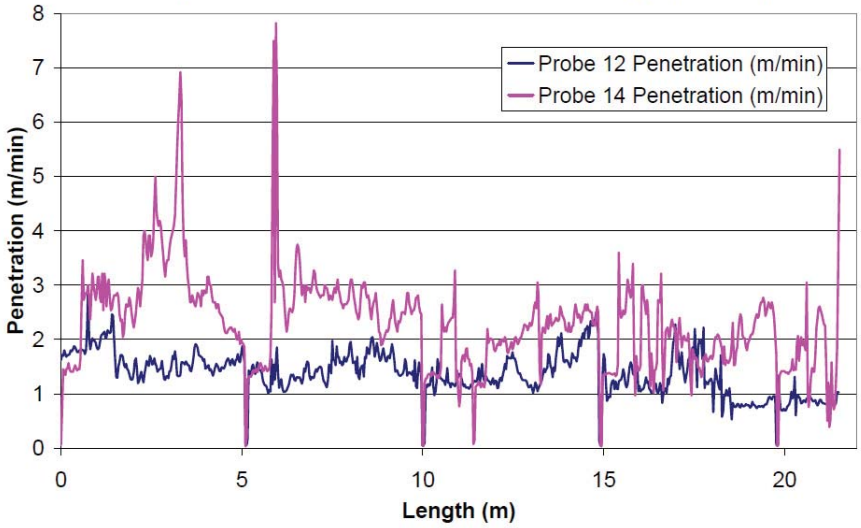


Figure 6: Penetration vs Length in probe 12 and 14 shows the range of penetration and the peaks related to rod changing

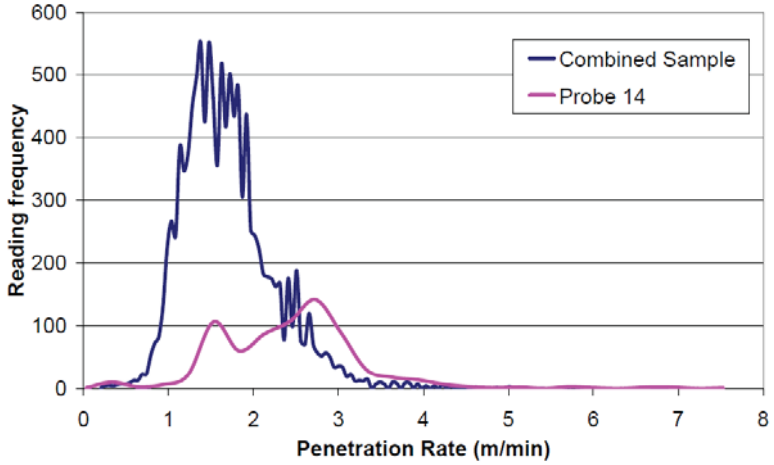


Figure 7: Histograms of penetration from Combined Sample and Probe 14

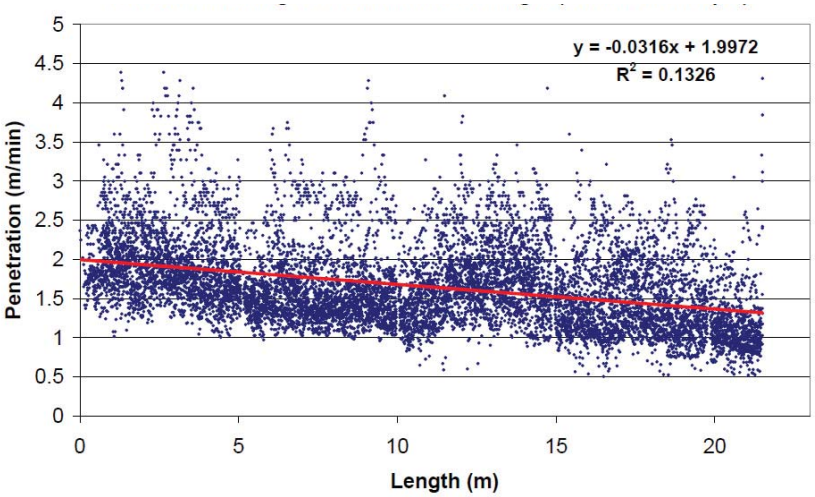


Figure 8: Effect of drill string length on penetration depicted by linear regression on data from Combined Sample

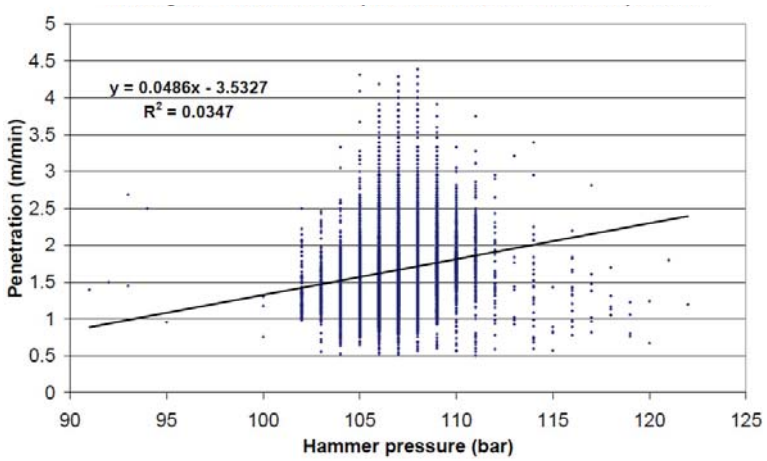


Figure 9: Effect of hammer pressure on penetration depicted by regression on data from Combined Sample

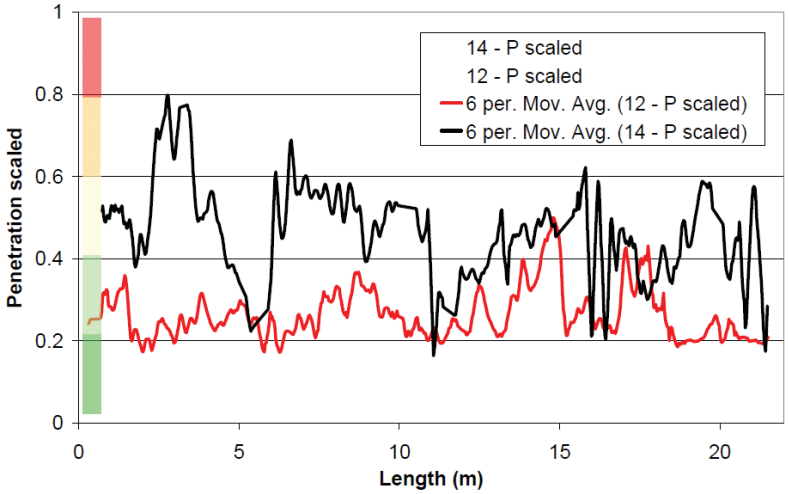


Figure 10: Penetration logs from probes 12 and 14 compared after correction of Length and HP effect and scaled to 0 – 1 range

4 DISCUSSION AND CONCLUSIONS

There are several sources of error in the proposed processing of the data to obtain scaled penetration as an indicator of rock properties. These are, for example:

- simplification of the P - HP dependency to linear and of constant a coefficient irrelevant to rock strength range (Figures 1 and 2)
- assuming hammer, feeder and rotation pressure entirely independent of each other
- assuming P – L dependency linear
- omission of feed and rotation pressure effect on penetration
- ignoring water flush effect and drill bit wear on penetration
- ignoring water inflow effect on Penetration
- Combined Sample not covering the whole range of possible penetration i.e. whole range of possible rock mass properties

However, from the objective point of view, which was detecting the extremely adverse rock condition, these simplifications and assumptions showed to be insignificant. The tested algorithm produces a parameter sensitive enough to detect the potentially hazardous rock condition. The completeness of the initial Combined Sample can be verified and amended during the project. Data from other project in similar rock condition can also be used to form the Sample.

The advantages of applying interpretative probing based on automated logs of drilling parameters are:

- eliminating safety hazards related to presence of geologist at drilling location
- obtaining results reliable and superior or equivalent to visual logging
- often eliminating a need of probing as additional activity in the production cycle (grout holes or other technical holes can be used as probes)
- easiness to produce various graphic reports

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Construction of Deep Circular Shaft within Urban Area

Freddie W.C. Chan, Lawrence M.P. Shek & Horace C.K. Cheuk
AECOM Asia Co. Ltd.
Danny D.S. Tang
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

The paper describes a case history of the construction of a deep shaft using diaphragm wall method which provides temporary support to an 85 m deep soil excavation in the urban area as part of the sewage conveyance system under Harbour Area Treatment Scheme Stage 2A. Complex geology consisting of deep and variable rockhead has been encountered. The design considerations are firstly discussed and the options of shaft are then evaluated. The construction method and ground improvement measures in term of toe grouting and fissure grouting with microfine cement are presented. The construction progress and performance review consisting of verticality results and pumping test results are also discussed. The comprehensive geotechnical monitoring arrangement indicates that there no undue settlement and abnormal groundwater drawdown throughout the shaft construction and soil excavation.

1 INTRODUCTION

The Harbour Area Treatment Scheme (HATS) is an environmental improvement project aiming at cleaning up the waters in the Victoria Harbour. Subsequent to the commissioning of the first stage in 2001, HATS stage 2A consisting of the construction of deep sewage conveyance system (see Figure 1) linking the preliminary treatment works (PTWs) located at urban areas in the northern and southwestern part of Hong Kong Island to the sewage treatment works (STW) at Stonecutters Island, commenced in 2009.

As part of the sewage conveyance system, a 12.3 m diameter junction shaft with depth over 155 m below the ground level was constructed at Sai Ying Pun to connect the sewage tunnels collecting sewage from the northern and the southern of Hong Kong Island and conveying it to Stonecutters Island STW (see Figure 1). The soil layer at this location was estimated to be about 85m deep and was supported using the diaphragm wall method. In this paper, it will discuss the design considerations, construction methods and performance review for the soil excavation of this deep shaft constructed in urban area.

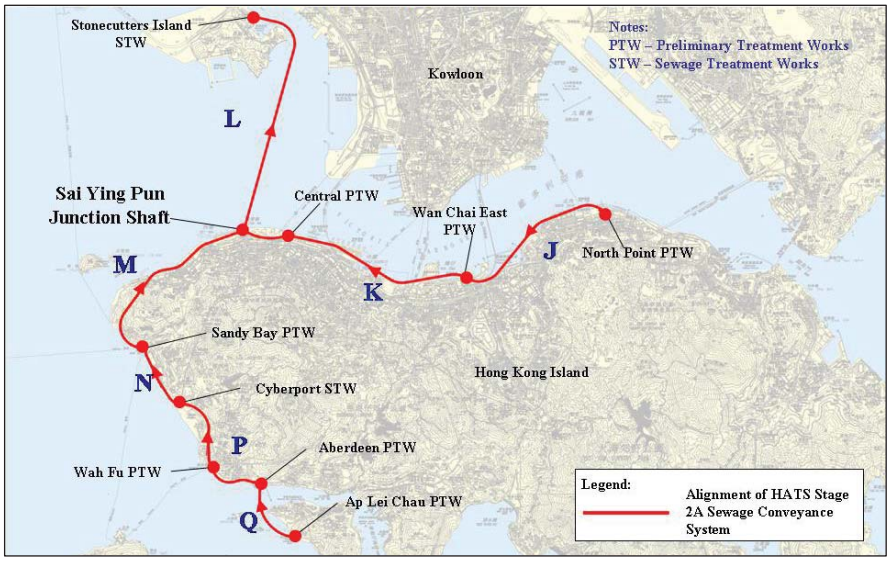


Figure 1: Alignment of HATS 2A Sewage Conveyance System

2 GEOLOGICAL CONDITIONS

The site is located in the area of reclamation underlain by varying thicknesses of marine deposits, alluvium and saprolite. Project specific boreholes have been sunk to reveal the ground conditions and the developed geological section is presented in Figure 2. The surface material at the shafts consists of reclamation fill, varying from approximately 20 m to 30 m thick which comprises loose to medium dense, fine to coarse sands and cobbles and gravels, with occasional construction waste materials such as wood and concrete fragments. The fill is locally underlain by marine deposits up to about 7m thick, which consist of firm marine clay and medium dense sand.

As revealed in the ground investigation, alluvium locally underlies the fill and marine deposit and varies from 0m to 7.5 m thick, consisting of interbedded layers of medium dense to dense, fine to coarse and soft to firm sandy silty clay. The marine deposits and alluvium have been partially replaced by fill. However, a soft layer with low SPT 'N' values is occasionally identified on the saprolite surface which could be in-situ or remoulded marine deposit or alluvium.

The superficial deposits are underlain by saprolite which varies from about 55 m to 65 m thick and consists of completely decomposed to highly decomposed granite which is moderately dense silty sand, becoming very dense with depth. Corestones are present in the saprolite which may give rise to shaft sinking difficulties as machine used to excavate soft material will not able to remove the corestones. The presence corestones may also create problems in obtaining adequate groundwater exclusion due to deviations in verticality of the individual wall panels and 'hang-ups' on corestones with more weathered material beneath.

The rockhead profile is considered to be relatively irregular ranging from 85 m to 95 m below ground level. Preferential weathering along sub-vertical joints, steeply dipping shears and sub-horizontal joints as revealed in the ground investigation information are likely to give rise to more irregularities than that shown in Figure 2. The highly variable rockhead profile would induce stability problem of the proposed shaft at the soil and rock interface. In addition, this may also cause difficulties in ensuring the watertightness of the proposed shaft at the soil to rock interface.

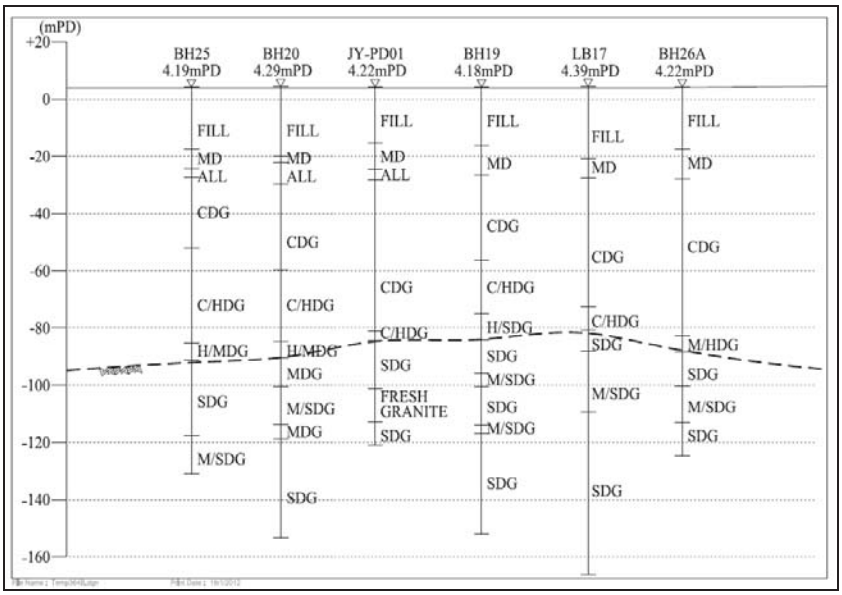


Figure 2: General geological conditions

3 CONSTRUCTION DIFFICULTIES AND SETTLEMENT CONCERN

In addition to the structural stability of the deep shaft, groundwater ingress is another key concern in the design and construction of this junction shaft. If groundwater ingress is not properly controlled, the shaft will act in a manner similar to wells and could result in radial drawdown of groundwater levels in the reclamation, superficial deposits and saprolite. The soil strata reach a total thickness up to 95 m and will be susceptible to ground displacements during shaft sinking and also settlement due

to groundwater drawdown if inflows into the shaft are not adequately controlled. The fine layers within the marine deposits and alluvium could act as aquicludes and may also be susceptible to consolidation settlement. The marine deposits existed below the layer of fill material will inhibit recharge from the sea to the material below the marine deposits. Precautionary measures in the form of grouting are considered. The need for grouting will depend on the permeability of the rock mass, likely extent and magnitude of groundwater drawdown, degree of potential settlement and the vulnerability of existing buildings, structures and utilities in the vicinity of the junction shaft.

4 DESIGN CONSIDERATIONS AND CONSTRUCTION ARRANGEMENT

The temporary support to the shaft in the soil layer was made up of reinforced concrete diaphragm wall panels virtually circular in plan. The circular shaft was generally designed to withstand the external radial load from the earth stress and the static groundwater pressure. The wall design took consideration of the behaviour of the shaft as a cylindrical shell that was to be constructed in chord segments. A cylindrical structure, when acted upon by the uniform exterior lateral pressures, would behave essentially as a compression ring. The compression force on the ring was calculated for the earth stress based on the at-rest soil condition.

The thickness of the diaphragm wall panel was controlled by the design compressive stress as a compression ring. The deviation from verticality of the panel will induce eccentricities from the mid-plan of the panel to the theoretical thrust line and it needs to be considered in the design. In the loading condition, each panel acted independently, and the joints will maintain tight under the effect of the outside pressures forming a loop of compression ring. A capping beam was designed at the top of the shaft to prevent the slippage of the joints, and the panel distortion from geometric imperfections and uneven pressures. The shaft is decided to be 1.5 m thick wall, 12.3 m internal diameter with target verticality tolerance at 1:380 (see Figure 3). To achieve the above stringent verticality requirement, the hydromill machine is selected for the construction of the diaphragm wall panels.

To cater for the different founding level of the toe level of the diaphragm wall panels with respect to the variable rockhead, installation of reinforced concrete ring beams starting at the highest toe level of the diaphragm for supplemental bracing are required.

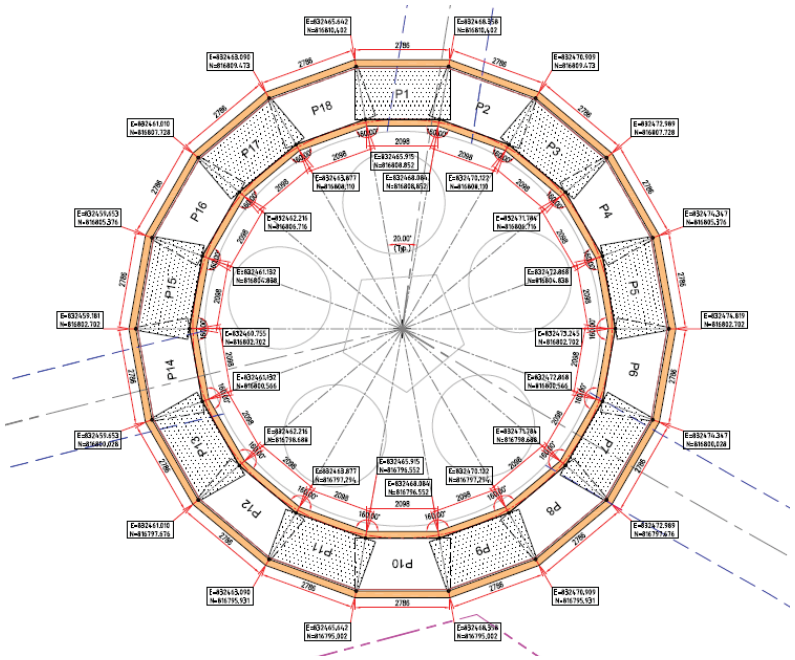


Figure 3: Layout of diaphragm wall panel of Sai Ying Pun junction shaft

As discussed in previous section, it is anticipated that the top 20 m to 30 m of fill consists of small and loose cobbles and is relatively porous. Ground improvement work in form of Tube-a-Manchette (TAM) grouting method has been carried out prior to diaphragm wall excavation to suppress the risks on bentonite leakage and improve the trench stability. As proposed by the contractor, 43 numbers of grout holes of 22m depth below ground were drilled with tube-a-manchettes installed. Multi-staged bentonite-cement grouting was proceeded and more than 500,000 litres of B/C grout were pumped to fill the void of possible leakage zone. With proper ground improvement and comprehensive bentonite slurry recycle system, steady slurry head was maintained during wall excavation.

5 CONSTRUCTION PROGRESS AND PERFORMANCE REVIEW

Installation of 18 numbers of panels for the circular cofferdam at Sai Ying Pun was successfully completed in about 7 months. On average, it took about 2.5 weeks for excavation, chiseling, bar-fixing and concreting for each panel. Alternate panels were constructed simultaneously to minimize the construction time. In order to maintain the trench stability the spacing of the alternate panels must be more than 3S, where S is the length of each panel, when constructed simultaneously. The most time consuming process was at the time when the excavation came over the hard material such as chiseling of the intermittent boulders and milling near the rock head level. The hydromill trench cutter was not efficient for excavation through rock and boulders, chisel hammer therefore was used by turns when boulders were encountered or rock head was nearly reached. Due to the limited working space of the site within urban area, only one hydromill and one crawler crane could be maneuvered for the circular cofferdam. No other major plant could be deployed to expedite the works progress. To effectively utilize the working space, the steel cages were prefabricated in an off-site bending yard and were delivered to site for immediate erection into the excavated trench. A summary table indicating the progress of diaphragm wall panels is presented in Table 1.

As mentioned above, the depth of the soft material is between 85m and 95m and the diaphragm will have to support the full depth of the soft material. In this regard, verticality control was of utmost importance in order to maintain the integrity and stability of the circular cofferdam during excavation. The hydromill machine selected was equipped with precise control mechanism to control the verticality of the excavated panel and the verticality of the excavation could be readily obtained from associated computer panel. Deviation from the vertical could be readily identified and immediately rectified as excavation proceeds. In addition, verticality during excavation between completed panels were maintained by the utilization of soft mill joint in the adjacent panels. In these ways, satisfactory result of verticality was obtained. The minimum achieved verticality has been approximate 1:600 (see Table 1).

For such a large scale dewatering within the deep circular cofferdam, a pumping test is generally warranted to obtain reliable data on the transmissibility, recharge, and capacity of wells and water-tightness of cofferdam, particularly important in this case. Furthermore, it is vital to ensure no adverse effect on the ground/structures in the vicinity.

Pump well, observation well, standpipe piezometer and triple-tipped piezometer were installed in accordance with approved design. The acceptance criteria for the pumping test were (1) no groundwater drawdown outside the cofferdam was more than 1 m and (2) steady state in groundwater drawdown was reached. Steady state shall be defined as the constant rate of pumping such that the rate of groundwater drawdown both inside and outside the cofferdam is less than 0.1 m over an hour. The pumping test was functionally completed and the water-tightness of the cofferdam was verified.

6 CONSTRUCTION MONITORING

Construction activities related to the deep shaft excavation may inevitably cause ground movement and groundwater drawdown. This may result in settlement of the ground surface, buried utilities and existing buildings and structures. To avoid any adverse impact on existing facilities and interruption to the construction programme, a comprehensive geotechnical instrumentation programme has been developed to monitor the possible influence from the construction activities. The proposed instruments were installed in advance of any construction activities to ensure they are functioning properly and sufficient time is provided to determine the baseline readings. Ground settlement markers, structure settlement markers, triple-tipped piezometers, magnetic probe extensometers and inclinometers had been installed to monitor the effect during diaphragm wall installation and shaft excavation. To facilitate immediate response towards abnormality,

corresponding Alert, Action and Alarm (AAA) Trigger levels of each monitoring instruments were established. Exceedance report with thorough review to examine the construction method and investigate the undue response of instrument should be prepared such that no adverse impact was made to the nearby facilities.

Based on the monitoring records throughout the period of diaphragm wall construction and subsequent soil excavation, no undue settlement and no abnormal groundwater drawdown are identified.

Table 1 – Summary of construction progress and achieved diaphragm wall panel verticality

Panel No.	Construction Progress		Founding Level (mPD)	Verticality
	Start	Complete		
P1	6-Feb-10	24-Feb-10	-84.1	1:2170
P2	24-Apr-10	12-May-10	-84.0	1:971
P3	4-Mar-10	23-Mar-10	-83.1	1:1941
P4	10-Jun-10	30-Jun-10	-83.1	1:991
P5	30-Mar-10	21-Apr-10	-82.8	1:2015
P6	8-Jul-10	22-Jul-10	-83.4	1:1223
P7	4-Jan-10	25-Jan-10	-84.0	1:883
P8	18-Feb-10	9-Mar-10	-84.0	1:844
P9	28-Jan-10	9-Feb-10	-81.0	1:686
P10	17-May-10	5-Jun-10	-78.9	1:865
P11	19-Mar-10	1-Apr-10	-82.2	1:2134
P12	21-Jun-10	13-Jul-10	-85.6	1:2804
P13	17-Apr-10	30-Apr-10	-85.6	1:1223
P14	29-May-10	17-Jun-10	-84.6	1:596
P15	27-Feb-10	16-Mar-10	-84.0	1:650
P16	5-May-10	25-May-10	-84.4	1:991
P17	6-Jan-10	2-Feb-10	-84.6	1:1085
P18	26-Mar-10	14-Apr-10	-84.6	1:3374

7 CONCLUSIONS

This paper presents a case history of the construction of a deep shaft using diaphragm wall method in urban area. The site constraints and difficult geological conditions are reviewed. The design considerations and construction methods are discussed. The comprehensive geotechnical monitoring arrangement indicates that there no undue settlement and abnormal groundwater drawdown throughout the shaft construction and soil excavation.

ACKNOWLEDGEMENTS

The authors wish to thank the Director of Drainage Services Department of the Government of the Hong Kong Administrative Region for permission to publish this paper.

Flexible Branch-out of Shield Tunnel for Underground Power Transmission

Shun-Min Lee & Tsung-Hai Chen
CECI Engineering Consultants, Inc., Taiwan

ABSTRACT

Owing to the continuous economic development of metropolitan areas, Taiwan Power Company invests tremendous capitals in upgrading the power transmission network on Taiwan. In recent years, transmission network has largely undergrounded to reduce the impact of construction to the densely populated urban environment. Although the shield tunneling technique has becoming prevailing in most alternative study, inevitably many vertical shafts for maintenance work are still required to branch out cables and the evacuation for workers. Since land acquisition and consumption for these out-let for underground facilities are restricted due to limited space for the branch shaft, some new development of design and construction technique are described in this paper.

Considering challenges from earthquakes that frequently occur in Taiwan, a durable connection mechanism has been incorporated between the vertical shaft and the tunnel. In addition, the location of shaft has been allocated as close as possible to the tunnel to greatly reduce the use of land and construction space. This technique has been successively adopted to a Φ5.23 m shield tunneling project of transmission cables in Kaohsiung, Taiwan.

1 INTRODUCTION

Reliable and stable power supply is critical to the living of people and continuous growth of economic condition. Taiwan Power Company (Taipower) is well-known of its performance of sustainable power supply for the development of Taiwan for decades. The power transmission system (Figure 1) connects among power plant, substation and users (clients) is a primary infrastructure for power utilities and all kinds of modern users. The cables to transmit the highest voltages (345 kV) of power in the system are sometimes called ultra-high voltage cables.

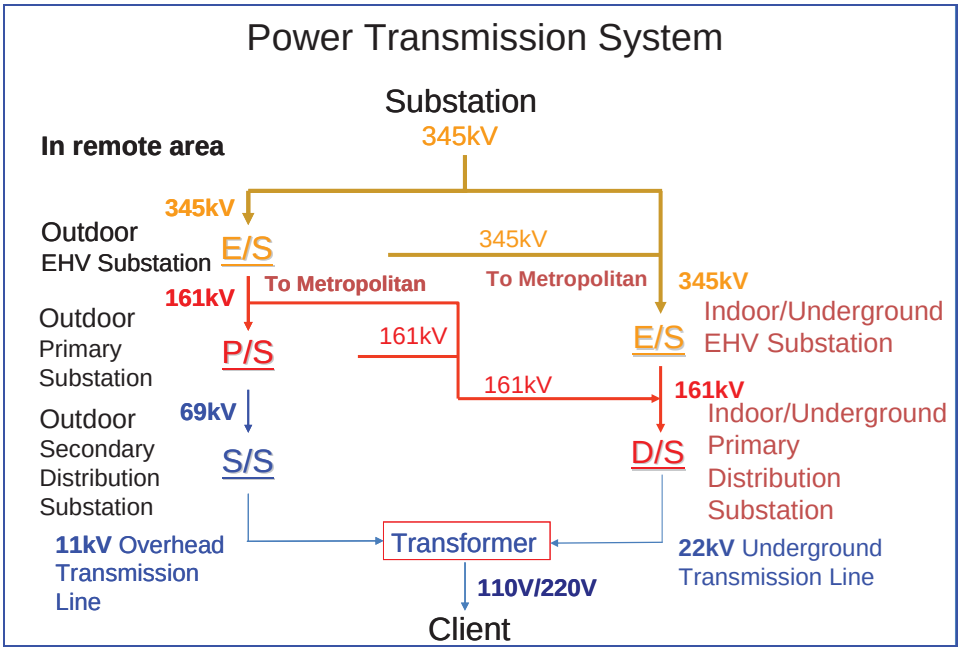


Figure 1: Taiwan Power Transmission System

In the past years, these ultra-high or high voltage cables were connected on grounds or hills along the line which are relatively simple to construct and maintain. However, after more and more people lives and gathers in the urban area, this kind of NIMBY (not in my backyard) facilities are frequently forced to move or construct underground. In fact, this trend is usually taken as modernization of urban environments.

Therefore, the newly constructed power transmission facilities are largely "hidden" below ground or behind fence and trees. The primary concern for construction work is the impact to people's living environment, especially in the densely populated cities. Nowadays trenchless methods are rapidly growing to comfort these concerns and, instead of cut-and-cover excavation, the shield tunneling technique has undoubtedly one of the prevailing alternatives for power transmission projects.

Once most length of power cables has gone underground, there are still some out-let to be constructed to accommodate the branch-out of cables and evacuation needs for workers. Depends on the regulation or code, these branch shafts are usually separated 0.5~2km apart along the power tunnel. The required depth of vertical shaft is at least from ground level to the nearby elevation of shield tunnel. If there are other concerns, such as flood protection, the exit of shaft might be elevated to certain height.

2 GAOGANG-WUJIA-KAOHSIUNG PROJECT

Kaohsiung metropolitan is the largest industrialized area in southern Taiwan. To improve the capacity and quality of power supply for Kaohsiung area, Taipower initiated, along with the "Sixth Power Transmission Project", the Gaogang-Wujia-Kaohsiung underground transmission project which includes four design/build construction lots to transmit 345kV power from the Kaohsiung Substation to the Gaogang Substation (Figure 2). Once completed, the project will enable a reliable power transmission of 3,300,000 kW to the Kaohsiung metropolitan (CECI, 2007).

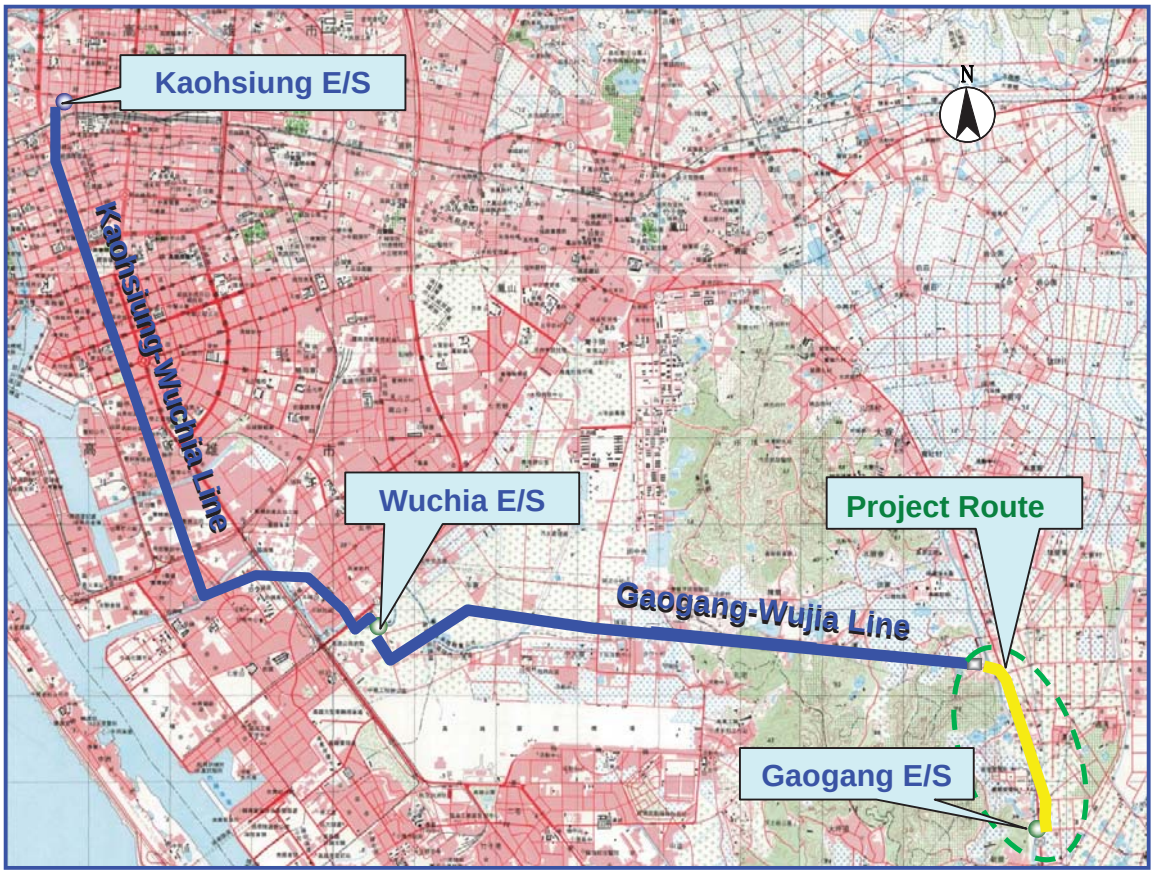


Figure 2: Route layout of Gaogang-Wujia-Kaohsiung underground transmission project

This paper illustrates the experience in the second D/B lot of Fenglin Road Shield Tunneling and the Gaogang Cooling Building project. The project route starts from the Gaogang Substation adjacent to vertical

shaft #1, crossed Zailai Ditch and runs north along the Fenglin 2nd Road up to an intersection with Provincial Highway No.88, and ends at shaft #2. Geotechnical information along the route, with total length of 1.56 km, is demonstrated in Figure 3. The Site is located at Daliao District of Kaohsiung which is a plain area with ground elevation of 10~15 m. The Holocene alluvium deposit is primarily consists of sand, silty sand, clay with portion of gravel. The groundwater table is at about G.L.-2~-5 m which is of concern during shield tunneling.

The inner diameter of the shield tunnel is 5.23m (outer 5.78m) to accommodate cables of 8@345 kV as well as 4@161 kV, cooling-pipelines, supporting racks, inspecting aisles, etc (Figure 4). The thickness of overburden soil on the shield tunnel is generally between 12~18m. The pit at the #1 shaft is used as the launch work shaft for shield tunneling to the existing #2 shaft, which provides cable out-let to connect towards the Wujia Substation. At meters before the #2 shaft on the route, there is a required branch shaft to make another out-let for the 161 kV cables to take their way to Fengsan Township.

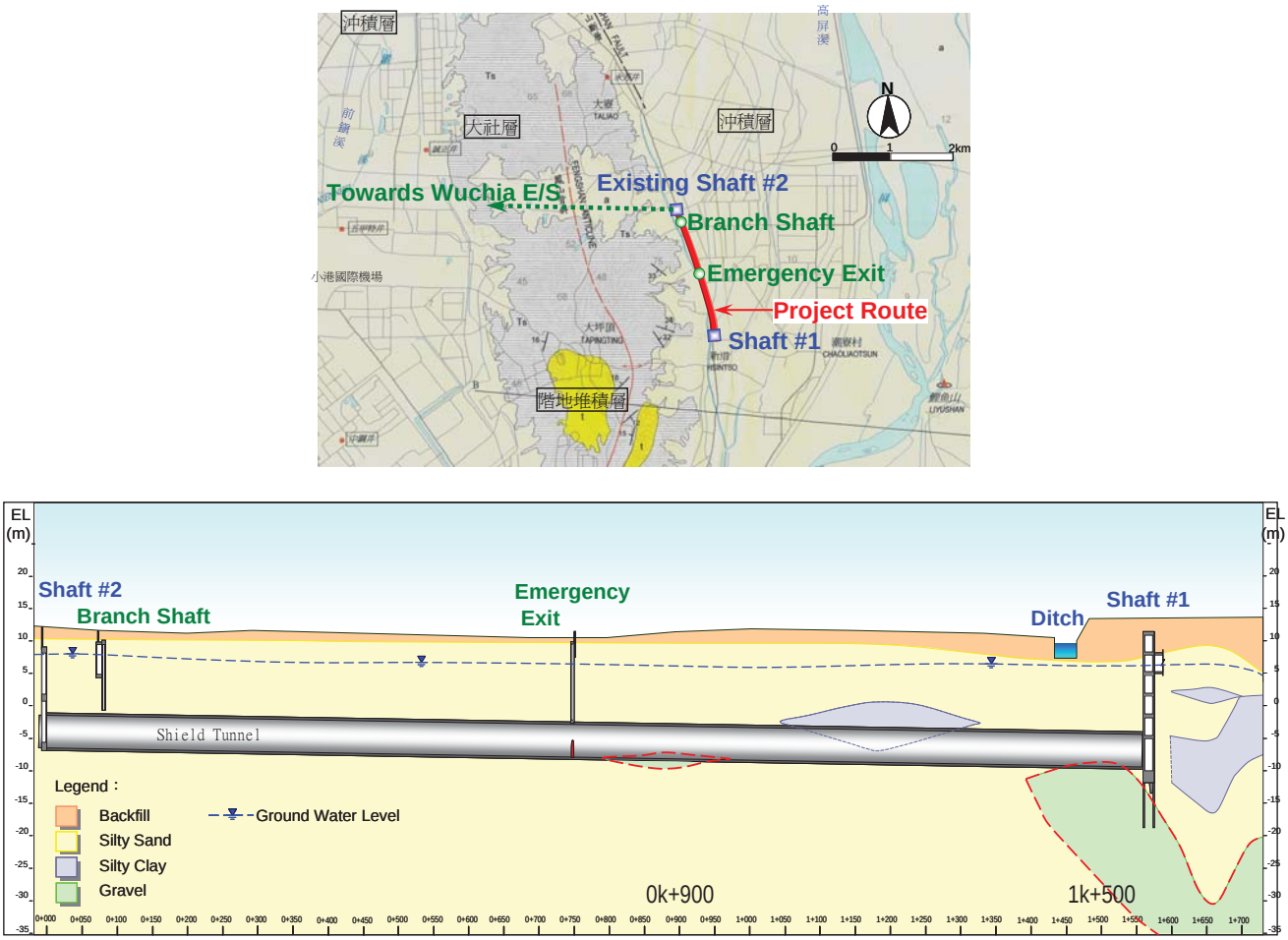


Figure 3: Geological information of the Gaogang-Wujia-Kaohsiung project

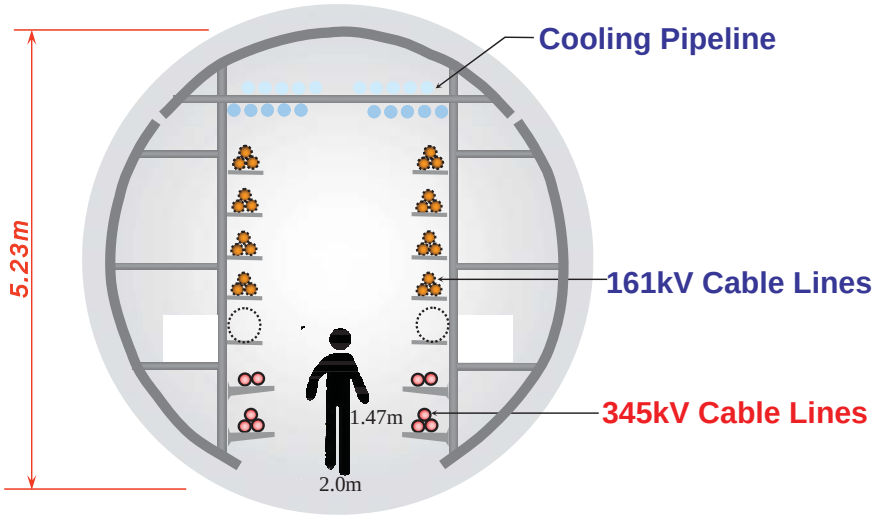


Figure 4: Inner cross section of transmission tunnel

3 DESIGN OF BRANCH SHAFT

Along with shield tunnels, it is not uncommon to construct divergent shaft or side passageway for operating purpose. However, due to space limitation, it is always worthwhile to make it compact and durable to be constructed within small portion of grounds.

Scheme of opening from one side of the tunnel and constructing a lateral culvert to a separated shaft is usually adopted in the past. This method used to take more space for construction or facilities, and the culvert normally consumes longer cable. Recent years, engineers start to use center walls or strengthened the linings of the tunnel to support the loading from vertical shaft above the tunnel. This method provide sufficient structural requirement for the connection to the shaft, however, the usage of inner space become less and restricted (see Figure 5). This type of connection also tends to trigger stress concentration issues at certain unfavorable situations.

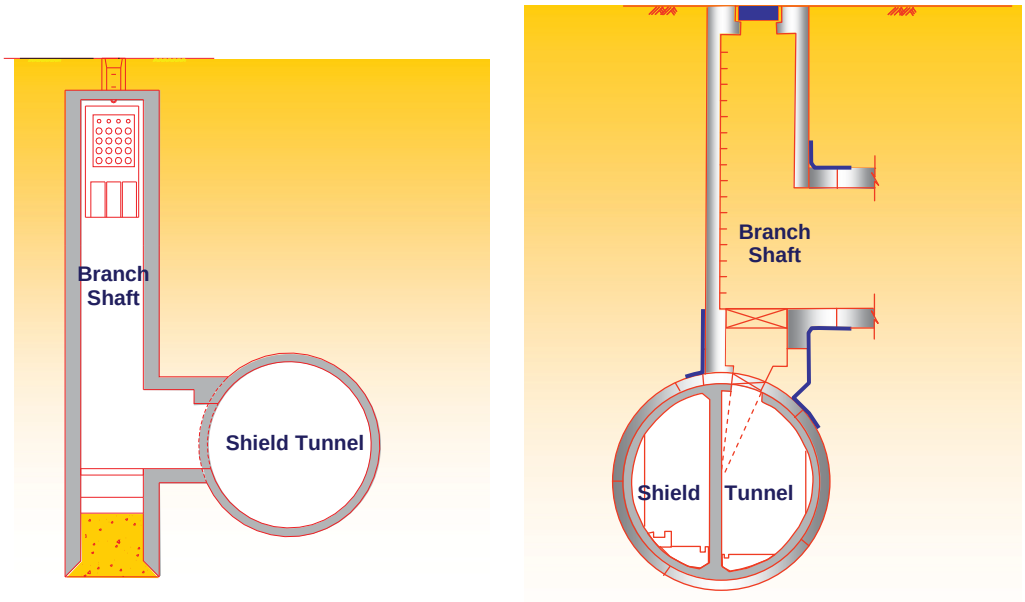


Figure 5: Conventional layouts of connection between branch shaft and shield tunnel

Conventional connection between branch shaft and shield tunnel takes time and space to construct. In addition, the stress-concentration situation often triggers non-uniform deformations, cracks and water leakage

problems. To conquer these problems, a versatile connection is developed to incorporate flexibility and constructability for the branch shaft, as illustrated in Figure 6.

The flexible connection beneath the concrete branch shaft is consisted of several steel rings which are circumvented by reinforced-rubbers (Figure 7). When necessary, this kind of flexible connection can absorb up to -5 cm~+10 cm vertical and 20 cm lateral displacements. For the purpose to retain these flexibilities, a steel tube is installed downward from the ground surface outside the vertical shaft. The branch shaft has widened flange on top to obtain bearing support from the soil beneath it, therefore reducing the stress applied on the shied tunnel lining from the branch shaft (Chen et al, 2011).

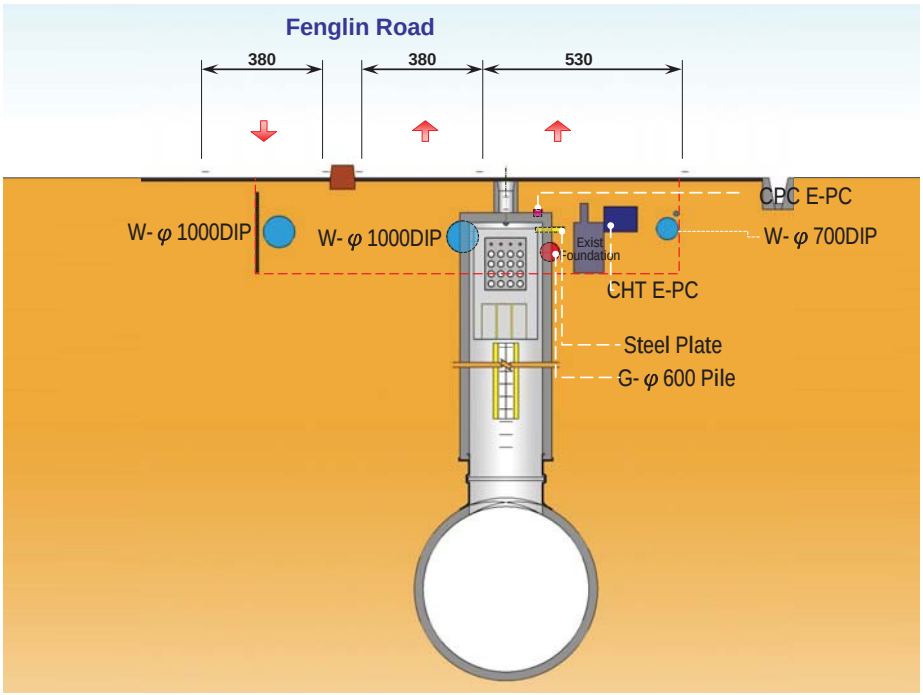


Figure 6: Flexible connection between branch shaft and shield tunnel

The lining for shield tunnel at this part has been changed to steel instead of concrete material. Its objectives are constructability (especially for opening and connection work), water-tightness and adaptive for curvature variation of tunnels.

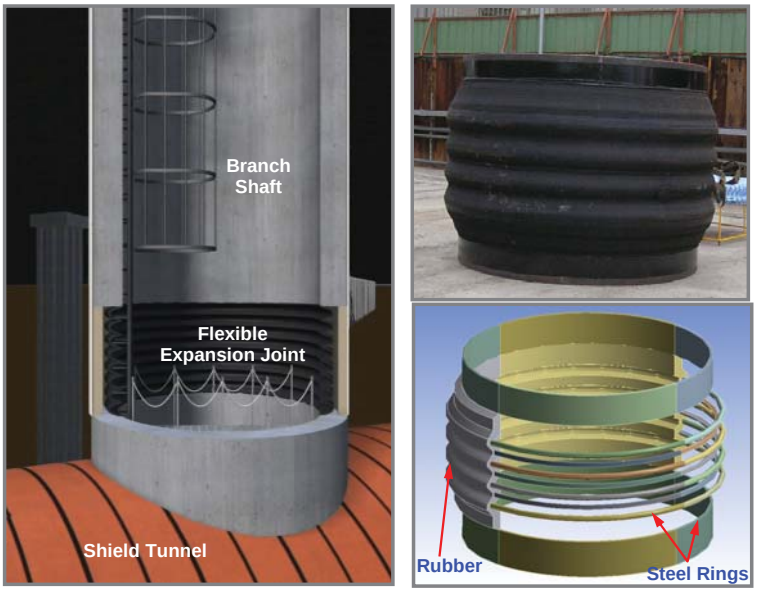


Figure 7: Flexible connection

4 CONSTRUCTION WORK

The structure of branch shaft is 12 m long with thickness of 0.3 m and outer diameter of 3 m. Before the construction of shaft, underground investigation has discovered pipelines of water, petroleum, gas, communication, etc. Because the ground is mainly sandy soil with high ground water pressure, to avoid detrimental effects to these pipelines, fully-casing excavation method and ground improvement grouting is adopted for branch shaft construction. In addition, considering risk of flushing into the shield tunnel, careful sealing procedure and quarantine measure are prepared and implemented during the construction.

5 CONCLUSIONS

The development of flexible connection between branch-shaft with shield tunnel provides an efficient construction to transmission network in urban area, especially to the site in populated district. The merit of space saving and durability against earthquake is of particular value to fault-zones cities.

The flexible connection beneath the concrete branch shaft is consisted of several steel rings which are circumvented by reinforced-rubbers. When necessary, this kind of flexible connection can absorb up to -5cm~+10cm vertical and 20cm lateral displacements. For the purpose to retain these flexibilities, a steel tube is installed downward from the ground surface outside the vertical shaft. The branch shaft has widened flange on top to obtain bearing support from the soil beneath it, therefore reducing the stress applied on the shied tunnel lining from the branch shaft.

Control of construction for tunneling is important, and is even more critical on the construction of divergent shaft or approaching the arrival shaft. Grouting for ground improvement is generally effective measure to overcome related operating risk.

ACKNOWLEDGEMENTS

The authors are grateful to the assistance kindly provided by the Taiwan Power Company and Kwong-Kee Construction Co., Ltd., during the design and construction phases for the engineering service project of the case history.

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Tunnelling Considerations for Hydro Electric Power Schemes in Shale Formations in Malaysia

N. R. Wightman
SMEC Asia Limited, Hong Kong
D. J. Steele & A.D. Mackay
Nishimatsu Construction Co. Limited, Hong Kong

ABSTRACT

Hydro Electric Power (HEP) is becoming an increasingly favoured and sustainable method of producing energy from natural renewable resources. Optimum locations for HEP scheme development are often in climates with high annual precipitation and mountainous terrain, with steep sided valleys and large water volume catchments, such as Sarawak, East Malaysia. Notwithstanding, these areas are largely unexplored, under developed and inaccessible and as a result have limited available ground model data, which is vital for the feasibility and risk assessment of an HEP scheme. Some of the risks associated with the ground, potentially impacting HEP schemes typical of this climate and terrain, are deep weathering, deep superficial deposition, adverse and complex structural geology, such as faulting and tectonic activity with regional metamorphism resulting from young mountain formation, and adverse hydrogeological conditions. Interpretation of the ground conditions and risk assessment is best carried out through a ground model development typically prepared at the project outset and regularly updated as relevant additional data becomes available. This paper presents some aspects of the feasibility design for the Sarawak Central HEP Scheme, Limbang province as an example of the use of a ground model to ascertain a suitable design assessment for the subsurface structures to be formed for the proposed HEP Scheme.

1 INTRODUCTION

The remote hilly locations located in the upland tropical area of the Limbang Province, central Sarawak, Malaysia (Figure 1) are ideally suited for HEP development. As typical of these locations, however, limited topographical and geological data is available. The risks of encountering adverse ground conditions impacting on the sub-surface structures during construction and operation are therefore increased.

This paper outlines the use of ground models used to assist the feasibility design for the Limbang HEP. The Scheme includes a dam and spillway, covering an approximate footprint of 10 square kilometre; diversion tunnels, about 1 km length; intake shafts and penstock tunnels, each providing between 100 m to 200 m head of water; and a 27 m span, 37 m height and 114m length cavern accommodating the power generation units. The published geology (Geological Survey, No date) comprised shale with minor coal seams and sandstone (Figure 2). Due to the inaccessibility of the terrain limited drillhole data was available; more reliable site investigation data was therefore obtained from accurate site reconnaissance and geological field mapping from available exposures. As typical of these locations the site was found to have complex structural geology, with variably orientated discontinuities, which influenced the rock's physical properties and the engineering design. The discontinuities mainly comprised bedding and steeply dipping joint sets, with an associated high degree of anisotropy particularly within the shales and widespread, localised weak zones were also present. A suitable degree of sensitivity was considered in the design used to assess suitable excavation sequences for the sub-surface structures with robust temporary excavation support and permanent lining. This design would need further assessment for the tunnel and cavern location selection as more information became available during the subsequent detailed design phase.



Figure 1: HEP locations, Limbang

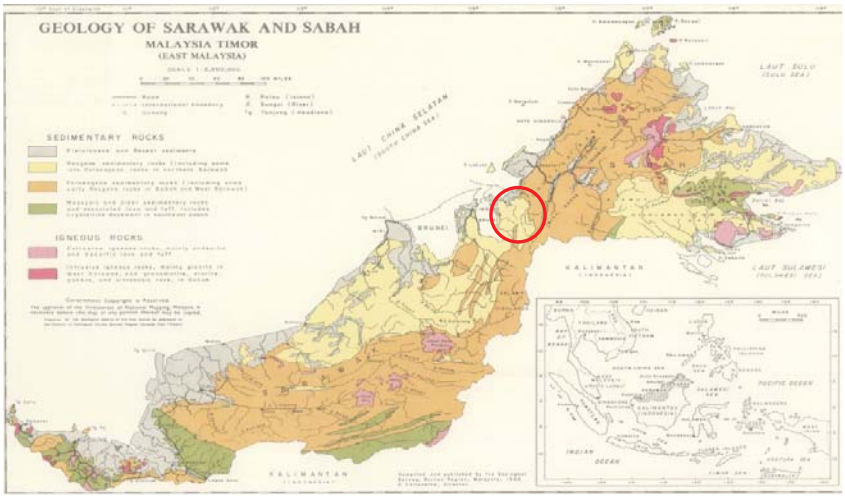


Figure 2: Geology of Sarawak with site location

2 HEP SETTING

2.1 Geological and Hydrogeological conditions

The Limbang HEP scheme involves two proposed areas for development, namely Limbang 1 and Limbang II (Figure 1). Both sites are situated on the Setap Shale Formation, which comprises inter-bedded shale, and sandstone (Figures 2 and 3) with a limited presence of mudstone, limestone, lignite, coal rich lenses with some marlstone, siltstone and calcareous sandstone. Other geological formations in close proximity to the Limbang 1 and 2 sites included the Mulu and Meligan Formations, which also comprise limestone, mudstones and shale and the Melinau Formation comprising limestone (Figure 3).

Following the desk study further site investigation (SI), comprising detailed geological mapping of exposures mainly located along the river banks (Plates 1 to 3) in the vicinity of the proposed HEP site, and a ground investigation comprising about 20 drillholes was carried out. The SI revealed thickly bedded sandstone and thinly bedded shale (Plate 1) with bedding thicknesses ranging from 1m to 20m, intense foliation and a high degree of anisotropy, and colluvium, influenced by river erosion (Plates 2 and 3).

Due to difficulties with continual access at this stage of the SI, piezometers were not installed. As rock was generally located near surface rapid surface water run-off was assumed with the permanent ground water level expected to be influenced by the river level with fully saturation of the regolith above rock head level.

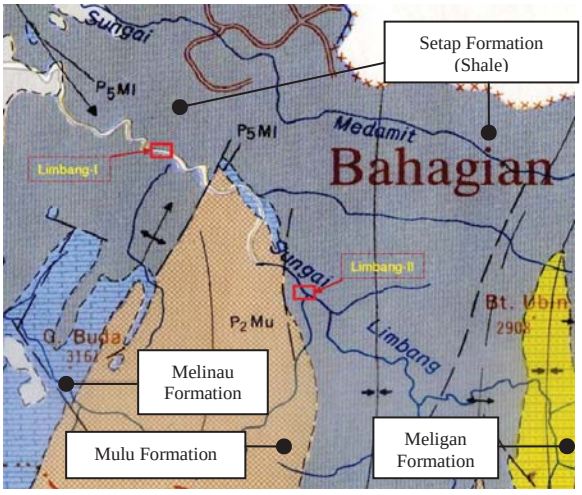


Figure 3: Geology at the Limbang HEP locations



Plate 1: Steeply dipping Shales Limbang River, Limbang II



Plate 2: View upstream towards the Limbang II dam axis



Plate 3: View upstream to Limbang I dam axis

Representative discontinuity assessment, ascertained through field inspection and available ground investigation data for the Limbang 2 site, are summarised in Table 1 below.

Table 1: Main discontinuity sets at Limbang 2 dam site

Discontinuity type	Dip direction (degrees)	Dip Angle (degrees)
Bedding	055 - 110	40 – 85
Joint set 1	300 – 010	65 – 90
Joint set 2	170 - 280	10 - 50
Joint set 3	160 - 210	65 - 90
Joint set 4	270 - 310	10 - 55
Joint set 5	020 - 060	10 – 45

The statistical assessment of the discontinuity data obtained for Limbang 2 (summarized in Table 1), was carried out using DIPS (Figures 4 and 5); and revealed one major discontinuity set, dipping 65 to 90 degrees, to be adversely orientated with respect to the cut slope formation. The stereoplots also provided a check of overlaps between the discontinuity set envelopes and failure zones. The shale rock core revealed very closely spaced discontinuities with a consistent orientation (Figure 6).

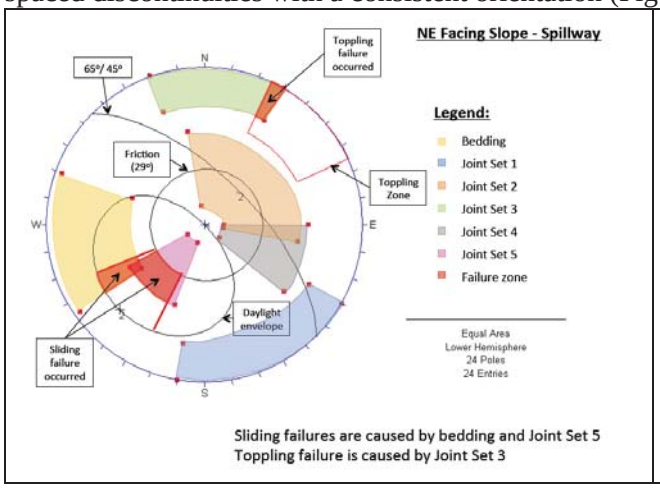


Figure 4: Potential toppling / planar failures

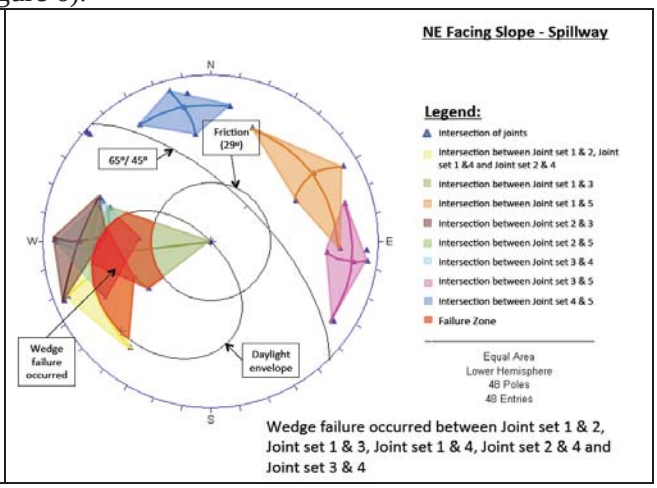


Figure 5: Potential wedge failures



Figure 6: Shale core (Setap Formation) showing the very close spacing of joints between the bedding

In addition to the instability associated with persistent discontinuities, a significant number of faults were anticipated in the vicinity of both Limbang HEP sites. Not only do faults present unique localised zones of instability they also have a major influence on the orientation, intensity, persistence and number of discontinuity sets in closer proximity to the fault zones. A potential increase in instability, i.e. toppling, planar and wedge failures within the rock, in both the slope and tunnel formations, are therefore anticipated. Recommendations were made to further identify the fault locations.

2.2 Design elements

The HEP configuration and main preliminary design layout elements for the Limbang 2 HEP Scheme, comprising intake shafts (typically about 50 m deep); penstock tunnels (typically about 100 m length) and the diversion tunnels (about 350 m long), are presented in Figure 7. The intake power tunnels, to be accommodated within the eastern dam abutment, had maximum and minimum design levels, below the maximum future flooded valley water level and above the current maximum tunnel design invert level respectively. Robust slope stabilisation in proximity of the intake portals was therefore needed (see Figures 8 and 9 for design elements with major slope stabilization design).

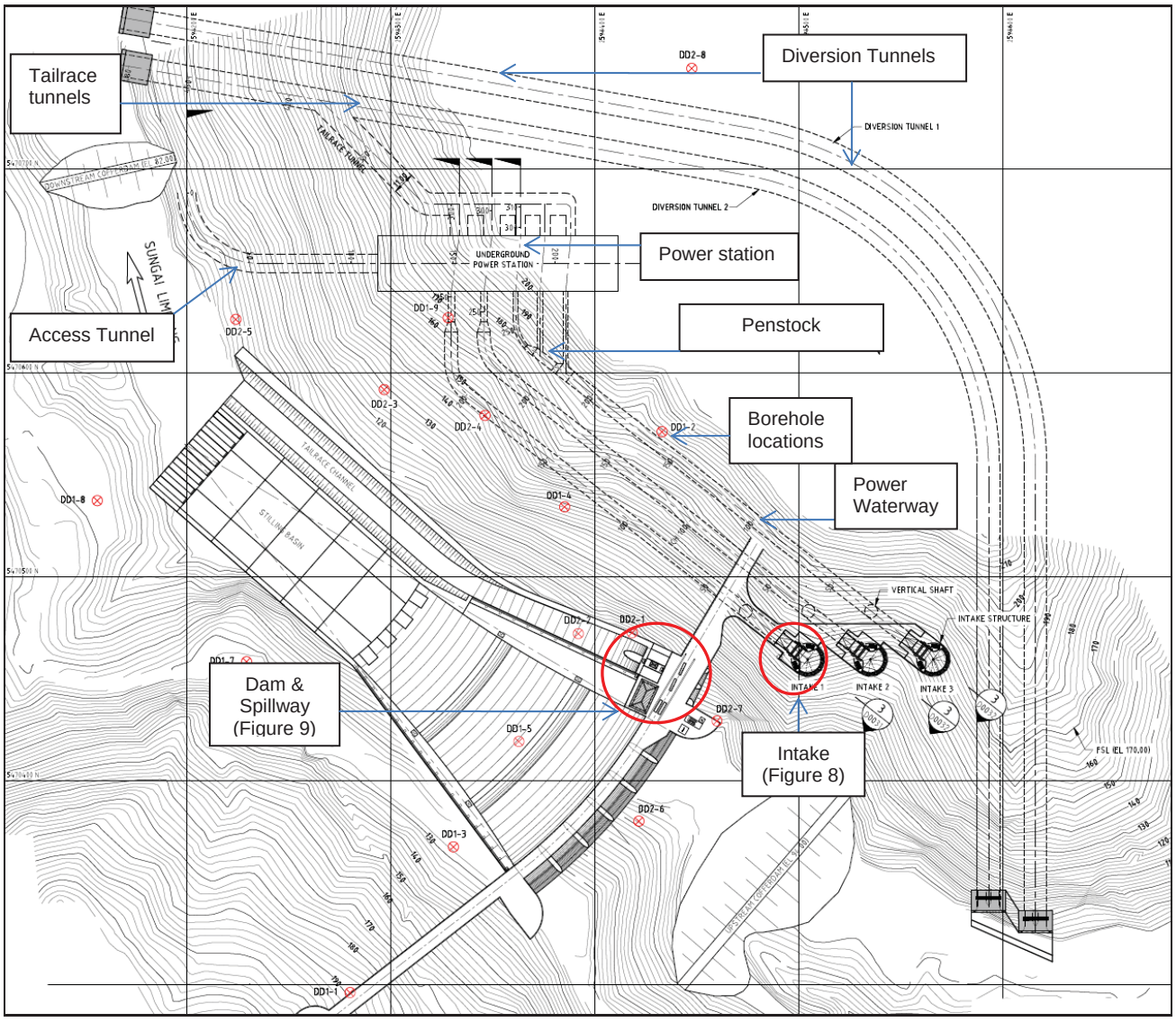


Figure 7: Preliminary layout of the Limbang 2 HEP scheme (main dam, spillway, diversion tunnels, penstock with intakes and powerhouse located in a cavern).

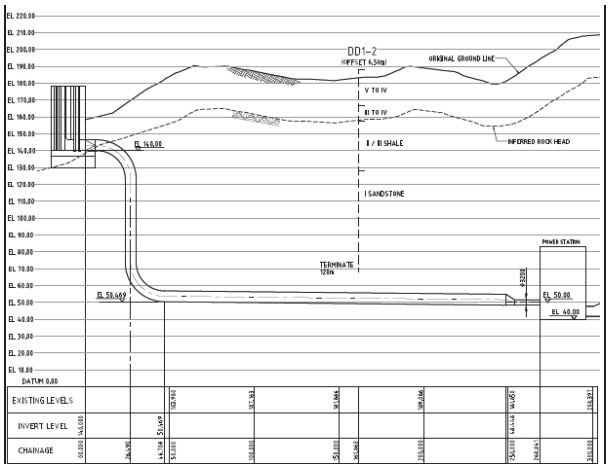


Figure 8: Slopes at the intake to the power tunnels

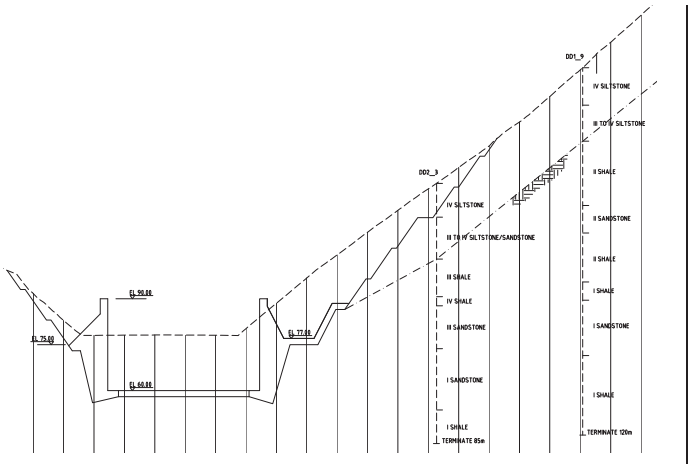


Figure 9: slope by the spillway

2.3 Properties of rock types

Based on the available site investigation data, the Setap Shale Formation revealed a high degree of physical variability compared to the sandstone. The shale is also prone to slaking and has a strength anisotropy ranging from a minimum and maximum strength parallel and perpendicular to the bedding respectively. Table 2 gives the range of physical properties of rock types present in the Setap Shale Formation, including indicative rock mass classification range using the Norwegian Geotechnical Institution (NGI) Q Index value, Barton (1989).

Table 2: Characteristics and Rock Mass Classification (Q value) for rock types found within the Setap Shale formation

Rock type	Is (PLT) / MPa (Ave)	UCS (MPa)	Joint frequency	Joint Roughness	Q value
Sandstone (all)	5.9	19.9 – 158.7	Close to very	Rough	0.83 to 3.33
Sandstone, Fresh (G1)	6.9	94.9 – 158.7	wide		
Shale (all)	1.45	4.4 – 23		Smooth to	
Shale, Fresh (G1)	1.68	11.6 - 23	Very close	polished	

The strength of the shale was lower than originally anticipated, due to a greater degree of weathering and / or the effects of intense folding and faulting in the more hilly terrain. The instability within the shale resulting from weathering effects and adversely orientated persistent discontinuities is presented in Plate 4.



Plate 4: Slope instability in the highly weathered shales, Limbang II

3 SUB-SURFACE STRUCTURE, PORTAL AND SLOPE DESIGN

3.1 Slopes and portals

The slopes adjacent to the spillway and powerhouse intake tunnel portals are expected to be formed in weak weathered shale (Figures 8 and 9). As the cut slope orientation does not readily coincide with the profile of the natural terrain, zones of locally increased excavation are required. The slope configuration comprises benches and berms (Figure 9) with bench gradients ranging from 55 to 70 degrees in fresh to moderately weathered rock decreasing to 45 degrees in highly to completely weathered rock, residual soil and colluvium. The anticipated weathering grades for the shale and sandstone are expected to range from fresh to completely decomposed rock (Grades, G, I to V, British Standard, BS, 5930:1999 and Australian Standard, AS, A2:2010, 2010); overlain by residual soil transported material (G VI and colluviums; BS, 5930:1999 and AS, A2:2010, 2010). From the interpreted ground model and slope stability assessment the entire slope required soil nail installation in addition to the slope support measures summarised in Table 3.

Table 3: Rock slope stabilising measures

Rock type	Slope Gradient (degrees)	Stabilising measures	Drainage measures
Decomposed rock / residual soil / colluvium	45	10 m long, 32 mm soil nails at 2 m centres and 100 mm shotcrete	10 m long raking drains at 2 m spacing / 10 degrees upward
Sandstone, (GII / III) & Shale, (GII / III)	55	5 m long, 40 mm rock bolts at 3 m centres and 75 mm shotcrete	10 m long raking drains at 2 m spacing / 10 degrees upward
Sandstone, Fresh (GI) & Shale, Fresh (GI)	68	Mesh cover to rock with fixings. Bolts for adverse joints	10 m long raking drains at 2 m spacing / 10 degrees upward

3.2 Tunnels and Caverns

The tunnel and cavern dimensions are minimized as much as practical to reduce construction and operation risk. The penstock tunnels were configured to allow partial or full operation of the power station in periods of low rainfall when limited water will be available for operation. To allow full operation of the tunnel the inside diameter is designed to be 8m. To assess the rock support the parameters for moderately weak shale, i.e. the poorest rock mass grade, were assessed to be Unconfined Compressive Strength (UCS) of 34MPa; Geotechnical Strength Index (GSI) of 35 and Lugeon values ranging from zero (massive rock) to 80 L/ minute (BS 5930:1999 and A2:2010, 2010). For poorer ground conditions the parameters were reduced and support requirements enhanced accordingly. Given the adopted parameters and poor rock conditions a final 700mm thick permanent reinforced concrete lining was assessed and temporary support, for the poorest rock conditions expected, to be 150 mm thick shotcrete with 5 m long, 32 mm diameter rock bolt installations. The underground powerhouse cavern requires 27 m wide by 37 m high by 114 m long to provide the space required and is anticipated to be located in poor condition shale. To allow construction the cavern excavation is staged, using split top heading and multiple benches (Figure 11) and pre-excavation grout comprising ordinary Portland and microfine cement to reduce permeability and ‘strengthen’ the rockmass to suitable limits, is required. Should weak zones and / or high groundwater inflows be encountered, pre-grouting and pre-support measures, such as spiling or heavier roof supports such as lattice girders or steel sets, will be installed.

3.3 Analysis

The use of finite element program (PHASE 2 software) provided a robust stability assessment for the cavern excavation in weak inter-bedded shale and sandstone. The parameters used for assessment included an Excavation Support Ratio (ESR) of unity and rock mass classification, Norwegian Geotechnical Institution (NGI) Q value range of 0.83 to 3.33, giving a support class between 5 to 6 (Barton et al, 1974 and Barton, 2002). From the findings of the analysis the anticipated support is 6.2m length rock bolts at 1.8m centres with 120mm thick fibre reinforced shotcrete with a typical 28-day design strength of 40 MPa. Following the preliminary analysis, Figure 9, a yield zone in the crown indicated excessive convergence with high bending moments being induced due to the weak nature of the shale. More robust measures were therefore adopted to

support the cavern roof and prevent excessive convergence. Figure 10 shows cavern crown and wall convergence, kept within acceptable limits by the installation of 9m long bolts at 1.5m centres in the crown and 6m long bolts at 1m centres in the cavern walls.

To further reduce convergence, the proposed construction involved sequential excavation of a divided cavern by an initial excavation along the centre drift followed by the left side slash, the right side slash and by 2m vertical height benches thereafter until the full excavation (Figure 11) is achieved. Support measures are to be applied immediately after each bench with a permanent lining of 1m with heavy reinforcement.

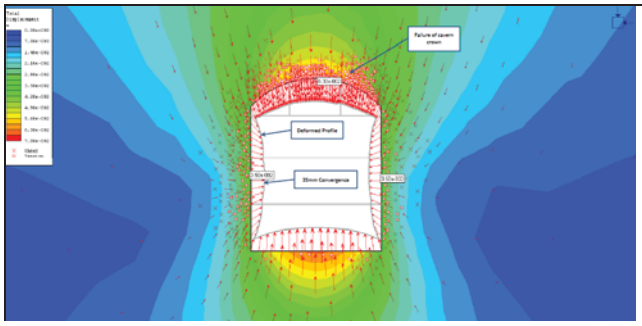


Figure 9: Excessive convergence of unsupported excavation in shale

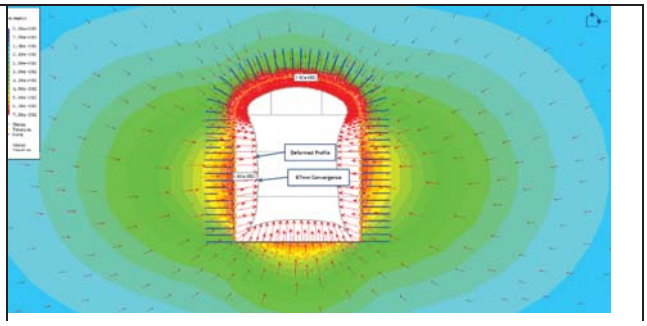


Figure 10: Reduced convergence, 9m long rock bolt and heavily reinforced, 1m thick, concrete crown

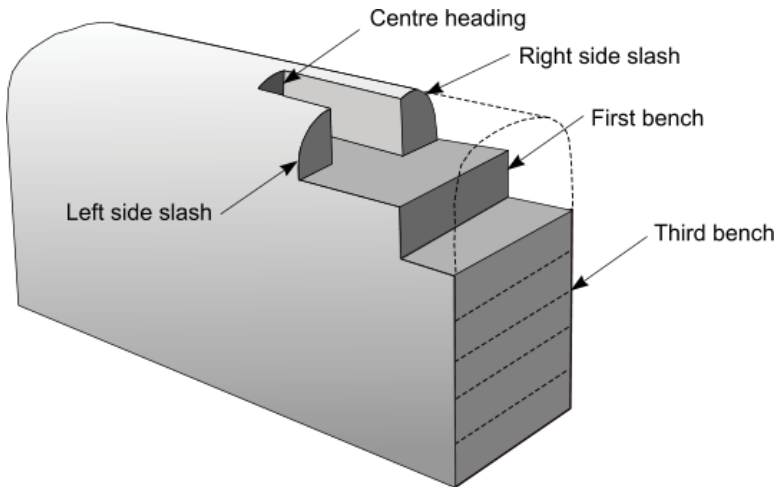


Figure 11: Control of convergence through construction sequencing with 2m benches (Heok, 2011)

4 CONCLUSIONS

Difficult ground conditions will be encountered in the tropical climate and terrain of the proposed HEP schemes with robust solutions being required for temporary excavation support and permanent tunnel and cavern lining. Based on the preliminary study for the Limbang 2 HEP Scheme and the anticipated weak rock, the feasibility design revealed that standard empirical temporary support measures, such as NGI Q System, may not provide adequate structural support. Carefully planned staged excavation is therefore of paramount importance for excavation and support for underground cavern excavations. In addition permanent lining installation will need to be in stages in order to progress the excavation sequence in a safe manner based on the data received at this stage.

Although extensive works are required, the high capital expenditure required for the construction of the HEP schemes is deemed to be justified to provide the power infrastructure necessary for the future development of industrial growth and development in Sarawak. The availability of reliable geotechnical and geological data is critical for the design of these underground structures and a carefully planned ground

investigation must be aimed at providing confirmation of the design parameters required for the detailed design stage.

From the initial assessment presented, it is expected for the final design assessments to include all new site data and that revisions will be required to the feasibility layouts prior to detailed construction drawings being produced. This staged process of design is critical in directing further ground investigation and producing robust designs adequate to support underground excavations for the schemes design life.

ACKNOWLEDGMENTS

The authors acknowledge the important input by the SMEC HK team particularly Ir Ricky Yim, Miss Emmy Hon, Ir Kent Li and Mr Martin Li in preparation of this paper, project director Mr Andreas Neumaier, project manager Mr Rudolf Naderer and the project sponsor Sarawak Energy Berhad (SEB). The opinions expressed in this paper are solely those of the authors and not of any other party.

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Numerical Modeling of Effects of Tunneling and Shaft Excavation on Adjoining Piled Foundations

Rupert K.Y. Leung & L. Tony Chen
Hyder Consulting Limited, Hong Kong
Johnson Chung
Geotechnical Consulting Group, Hong Kong

ABSTRACT

This paper presents two cases where 3D finite element modeling has been adopted to assess the effects of excavations on the piled foundations of existing structures. The first case involves soft ground TBM tunnelling underneath the piled foundation of an existing footbridge. The original design required as a protective measure a block of soil underneath the footbridge foundation and above the tunnel crown to be grouted prior to arrival of the TBM. With the help of 3D modeling, it was possible to justify elimination of the block grouting. The modelling also serves to determine the required face pressure for controlling the movement induced at the footbridge. The second case involves two connected 40 m deep diaphragm-walled excavations, one circular-shaped and 50m in diameter and the other rectangular in shape measuring 18 m by 12 m, which are to be carried out adjacent to a sensitive existing building. Because of the irregular shape of the excavation, 3D modeling has obvious advantages over 2D modeling in supporting the design with the excavation sequence and the actual arrangement of the struts in the two excavations duly accounted for.

1 INTRODUCTION

With the advance in processing capability of personal computers and the ready availability of commercial software, the use of three-dimensional numerical modeling has become more common in dealing with soil-structure interaction problems. The obvious advantage is that the problem can be more accurately modeled as compared with two-dimensional modeling which generally involves simplified assumptions, often conservative in most cases but can be unduly optimistic in others. Apart from a better understanding of the how the structures behave in the problem, 3D modeling can sometimes lead to more economical solutions.

The following sections presents two cases where 3D finite element modeling has been adopted to assess the effects of excavations on the piled foundations of existing structures. The first case involves soft ground TBM tunnelling underneath the piled foundation of an existing footbridge. The model took into account the various operations associated with TBM tunneling, including face support to the tunnel face and annulus grouting behind the behind the tunnel shield for determining the required face pressure for control of the movement induced at the footbridge. The second case involves two connected diaphragm-walled excavations, one circular-shaped and the other rectangular in shape, within soft ground and adjacent to a sensitive existing building. Because of the irregular shape of the excavation, 3D modeling was chosen over 2D modeling in supporting the design with the excavation sequence, the internal strutting arrangement of the two excavations and the effect of the existing building foundation duly accounted for.

2 STUDY CASE 1: TBM TUNNELLING UNDERNEATH A PILED FOUNDATION

2.1 Case description

The alignment of the twin tunnels of a proposed railway passes underneath the foundation piles supporting the eastern end of an existing footbridge. The tunnels are to be constructed by closed face tunnelling using a tunnel boring machine with an 11 m shield. The twin tunnels, namely the Northbound and Southbound

tunnels, have an external diameter of 9m and the tunnel crown is only 3 m (i.e. 0.33D) away from the closest pile of the footbridge. To minimize the effects of the tunnel construction on the piles and footbridge, the original design required a block of soil underneath the piled foundation to be grouted. In order to investigate whether a more economical design with the proposed grouted soil block eliminated would be feasible, 3D finite element modeling using the software "Plaxis 3D Tunnel v2.4" was employed to study the tunnel-pile interaction.

2.2 Finite element model

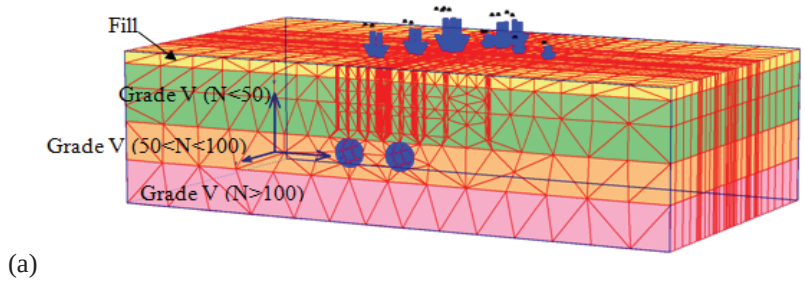
The ground profile consists of a 4m thick superficial Fill layer underlain by over 30m of Grade IV/V Tuff or foliated Meta-tuff soil layers. Table 1 summarizes the geotechnical parameters adopted.

Table 1: Summary of geotechnical parameters adopted in Study Case 1

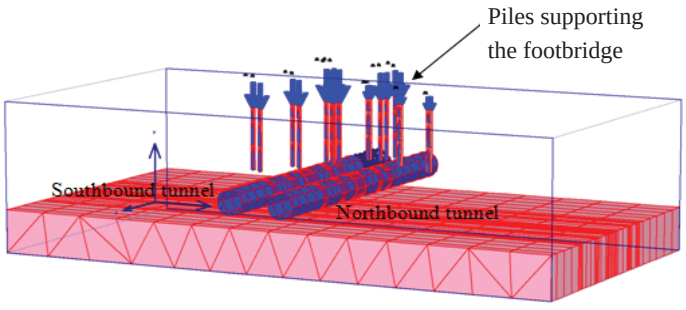
Soil	Top Level (mPD)	$\gamma_{dry}/\gamma_{sat}$ (kN/m^3)	ϕ' ($^\circ$)	c' (kPa)	K_o	R_{inter}*	E_{ref}** (kPa)	E_{inc}** (kPa/m)	ν
Fill	+6.0	18/19	35	0.1	0.43	1.00	10,000	N/A	0.30
Grade V (SPT N<50)	+2.0	18/19	33	5	0.46	1.00	8,426	2,106	0.30
Grade V (50<SPT<100)	-17.5	18/19	33	5	0.46	1.00	90,475	3,850	0.30
Grade V (SPT N>100)	-30.4	18/19	33	5	0.46	1.00	200,200	5,500	0.30

* R_{inter} = interface strength reduction factor
** E_{ref} = Soil modulus at top of layer; E_{inc} = increment of E with depth

The soils are modelled using Mohr Coulomb soil model. The piles are modelled by continuum elements and the tunnel lining by plate elements. The groundwater table is conservatively taken at the ground surface (i.e. +6.0 mPD). The finite element model used in this study is presented in Figure 1.



(a)



(b)

Figure 1: Finite element model for Study Case 1: (a) all soil layers shown; (b) part of the soil layers not shown

2.3 Analyses and results

The excavated tunnel section for the closed face tunnelling is temporarily supported by a supporting pressure which is equivalent to the hydrostatic water pressure plus a pre-defined over-pressure and the lining is then

erected behind the shield. The over-pressure can be varied in accordance with the varying ground condition along the tunnel alignment. At sensitive locations an over-pressure in the range 20 to 60 kPa is often adopted.

The contractor targets at maintaining the ground loss resulting from a single tunnel bore in greenfield condition to less than 0.7%. In practice, a ground loss of 1% is often assumed for typical tunnelling conditions. The first stage of the analysis is to determine the relationship between ground loss and tunnel over-pressure. The over-pressures corresponding to 0.7% and 1.0% greenfield ground loss are then used to investigate the effect of constructing the two tunnels on the footbridge and its piled foundation.

Single-bore Tunnelling under Greenfield Condition

A ground model with no piles or pile loads is used to determine the relationship between over-pressure and ground loss for the construction of a single tunnel under greenfield condition. Three analyses are conducted respectively with an over-pressures of 10 kPa, 20 kPa and 40 kPa.

A "progressive tunnel excavation" method is adopted in the analyses. The "progressive tunnel excavation" method involves modeling the tunnel boring process in a series of incremental lengths beneath the footbridge. A length increment of 11 m is adopted as this is the same length as the distance between the tunnel face and the point at which the lining is in place supporting the ground. For each incremental length of tunnel excavation the excavated ground is supported in the finite element model using a pressure loading which is equivalent to the hydrostatic pressure plus the selected over-pressure. The pressure at the excavation face represents the face pressure and the pressure around the 11 m long tunnel perimeter represents the tail grouting pressure. The over-pressure is assumed to be uniform across the tunnel face and to be uniform along the tail grouting zone. As each successive increment of excavation is modeled, tunnel lining is included in the section of the tunnel which has been previously excavated.

Table 2 summarizes the induced ground loss and maximum settlement induced by a single bore of TBM tunneling under greenfield condition.

Table 2: Simulation summary and computed volume loss and settlement in greenfield condition

Case	Over-pressure (kPa)	Ground Loss V_L (%)	Maximum Settlement δ_v (mm)
GF_p10	10	1.95	39.9
GF_p20	20	0.90	17.4
GF_p40	40	0.43	8.3

Figure 2 shows the computed ground surface settlement profiles. The computed settlement matches fairly well with the Gaussian settlement tough using a trough width parameter (K) of 0.4. The relationship between the induced ground loss and maximum settlement with over-pressure is presented graphically in Figure 3. For the purpose of the further analysis described in the following section, over-pressure corresponding to 0.7% and 1.0% ground loss are taken as 28.0kPa and 21.5kPa respectively.

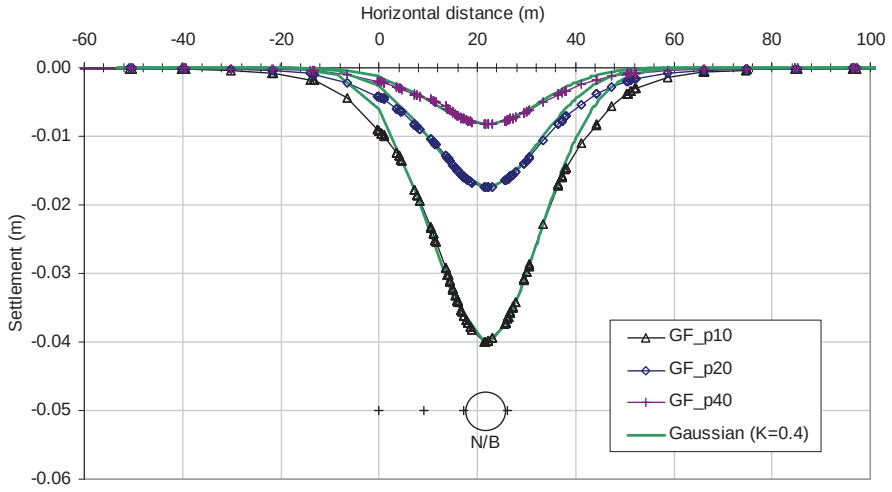


Figure 2: Ground surface settlement for greenfield cases

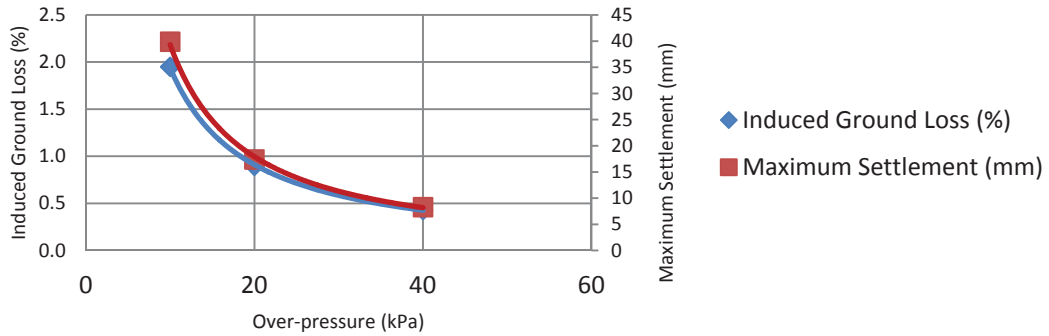


Figure 3: Relationship between induced ground loss and maximum settlement with over-pressure

In practice it would be necessary to ensure that the minimum over-pressure is maintained at the top of the TBM face and also that an additional over-pressure of approximately 20kPa will be adopted to allow for variations in ground conditions during tunneling.

Tunnelling underneath the Piled Foundation

Progressive tunnel excavation for the Southbound then Northbound tunnel underneath the existing piled foundation of the footbridge are modeled. The predicted ground loss and maximum ground surface settlement resulting from construction of the two tunnels are summarized in Table 3. Figure 4 shows the ground surface settlement in the case of over-pressure corresponding to 0.7% greenfield ground loss.

Table 3: Ground loss and settlement for twin-bore tunnelling

	V_L (%)		δ_v (mm)	
Case	1st	1st + 2nd	1st	1st + 2nd
Pile_VL0.7%	0.51	1.20	10.0	22.1
Pile_VL1.0%	0.64	1.83	12.8	35.8

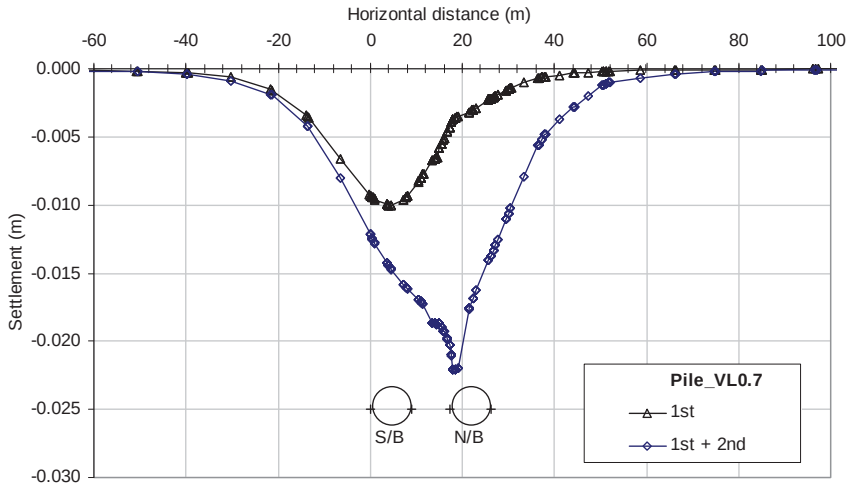


Figure 4: Ground surface settlement for twin-bore tunnelling (in Case Pile_VL0.7)

For both cases analyzed, ground loss caused by the first tunnel excavation is less than that under greenfield condition. This can be attributed to the fact that the existing piles, located laterally away from the Southbound tunnel, provide restraint to ground settlement to a certain extent. The ground loss and settlement resulting from the excavation of the second tunnel (i.e. Northbound tunnel) are greater than those resulting from the first tunnel drive in both cases, and for the case of Pile_VL1.0%, respectively approximately 20% and 30% greater than that of the greenfield condition. These increases can be explained by the presence of loaded piles closer to the Northbound tunnel and that soil shear strain has been mobilized by the first tunnel drive.

3 STUDY CASE 2: DEEP SHAFT EXCAVATION ADJOINING A PILED FOUNDATION

3.1 Case description

This case involves two inter-connected excavations, which are to be carried out within soft ground and adjacent to a sensitive existing building (EBS). One of the excavations is circular in shape (CE), measuring 50 m in diameter and 40 m in depth, and the other is rectangular in shape (RE), measuring 10 m x 15 m on plan and 35 m in depth. The excavations are supported by diaphragm walls, with several layers of reinforced concrete ring beams for CE and reinforced concrete permanent struts and steel temporary struts for RE.

Numerical modeling using software "Plaxis 3D Foundation v2.2" was carried out to study the effects of change in construction sequence of the two excavations from the original sequence of having the circular excavation completed first to an alternative of having both excavation proceeded concurrently.

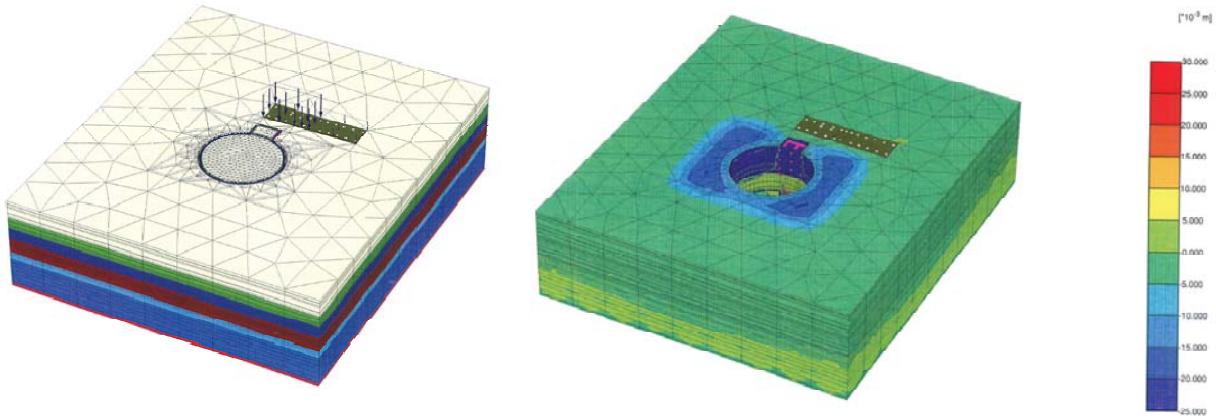
3.2 Finite element model

The geological profile and geotechnical parameters adopted in the study are summarized in Table 4.

Table 4: Summary of geotechnical parameters adopted in Study Case 2

Soil	Top level (mPD)	γ (kN/m^3)	ϕ' (^{o})	c' (kPa)	K_o	R_{inter}	E (MPa)		ν
FILL	+5.5	18.0	30	0	0.5	0.67	20		0.3
Marine Deposits	-7.2	17.0	28	6	0.53	0.5	8		0.3
Alluvium	-15.5	19.0	35	3	0.43	0.67	25		0.3
CDG (N<200)	-24.0	19.0	37	5	0.40	0.67	20m to 30m	30	0.3
					0.40	0.67	30m to 40m	40	0.3
					0.40	0.67	>40m	100	0.3
CDG (N>200)	-40.0	19.0	38	5	0.38	0.67	200		0.2
Rock	-60.0	22.0	N/A	N/A	N/A	N/A	5000		0.2

The soils are modelled using Mohr Coulomb soil model, the diaphragm walls using plate elements, and struts and waling beams using beam elements. The finite element model is shown in Figure 5.



(a) before excavation (b) after excavation, vertical displacement shown

Figure 5: Finite element model for Study Case 2

To account for the building stiffness in the soil-structure interaction, the building is modelled as an equivalent base slab of 5m thickness loaded by an UDL, based on the following equation proposed by Franzius et al (2006):

$$(EI)_{bldg} = E_s \sum_1^{m+1} \left(I_s + A_s \cdot {H_m}^2\right) \quad (1)$$

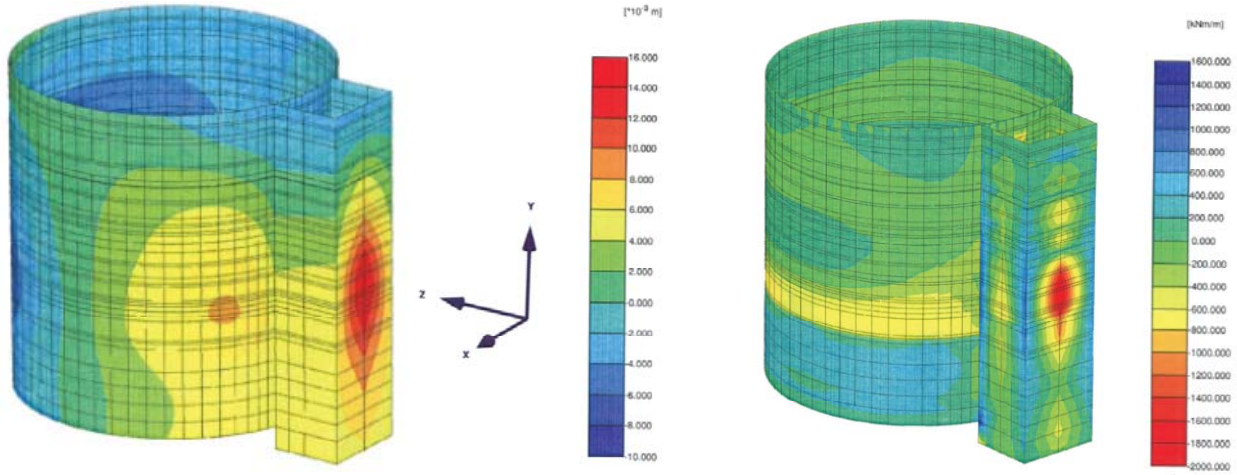
$$(EA)_{bldg} = (m+1)(E_s A_s) \quad (2)$$

where E_s = Young's modulus of slab, I_s = moment of inertia of slab, A_s = cross-sectional area of slab, m = number of storeys, H_m = vertical distance between the building neutral axis and the slab neutral axis.

The piles supporting the buildings are modelled as embedded pile elements with built-in interface elements to model the interaction between the pile and the soil.

3.3 Analyses and Results

A construction sequence with both the circular and the rectangular excavations proceeded concurrently and the permanent struts constructed as excavation proceeds downward is adopted in the analysis. Figure 6 shows the displacement of the diaphragm wall and bending moment along the wall panels upon completion of the excavation. The analysis also shows the maximum ground settlement at the EBS to be less than 5mm and the maximum pile settlement of the EBS to be less than 2 mm.



(a) displacement in z-direction (b) bending moment along D-wall

Figure 6: Results of the analysis

Sensitivity analyses have been carried out to investigate the effect of the EBS and its piled foundation and the interface strength on ground settlement. In the first case, the model is analyzed with the EBS and its piled foundation removed. In the second case, the interface strength reduction factor R_{inter} is reduced to 0.1. In addition, alternative excavation sequences with the circular excavation proceed slightly in advance of the rectangular excavation has also been studied.

4 CONSLUSIONS

The two cases described in this paper have demonstrated that 3D finite element modelling is an effective tool in assessing the effect of excavations on adjoining existing structures and their foundations. It has an advantage over empirical method in that loading from the existing structures and the reinforcing effect of the structures and their foundations can be accounted of. It is also an obvious choice over 2D modelling when geometry of the problem creates doubt in simplifying it into a plane strain problem. Caution should however be taken that proper calibration and sensitivity check be carried out to validate the model.

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Modelling of Tunnelling beneath a Piled Building - Comparison of 2D and 3D Analyses with a Case History

S.W. Lee & C.K.M. Choy
Geotechnical Consulting Group (Asia) Limited, Hong Kong

ABSTRACT

Shield tunnelling in the urban environment is often carried out in close proximity to existing pile foundations and utilities/services. Assessment of the effect of tunnelling on existing piles and building structures often uses two-dimensional (2D) finite element (FE) analysis. The process of tunnelling and the geometry of piles and building structure/loads are a 3D problem, and to model them as a 2D soil-structure interaction problem can give inappropriate predictions. This paper aims to shed light on this issue by carrying out 2D and 3D FE analysis of a well-document tunnelling case history in Hong Kong in the 1980s. It is found that the 3D analysis predicts ground/building settlements in good agreement with the measurements, whereas the 2D analysis predicts higher ground/building settlements and pile response. The reasons behind the 2D over prediction are given.

1 INTRODUCTION

Construction for underground metro lines in the urban environment is often carried out in close proximity to existing building foundations, structures, utilities/services, etc. Shield tunnelling by means of slurry support or earth pressure balance (EPB) method is often used in soft ground tunnelling partly due to its ability to control tunnel face stability and ground movement. To protect existing buildings, tunnelling-induced ground/building movements are often limited to in the order of millimetres for serviceability reasons.

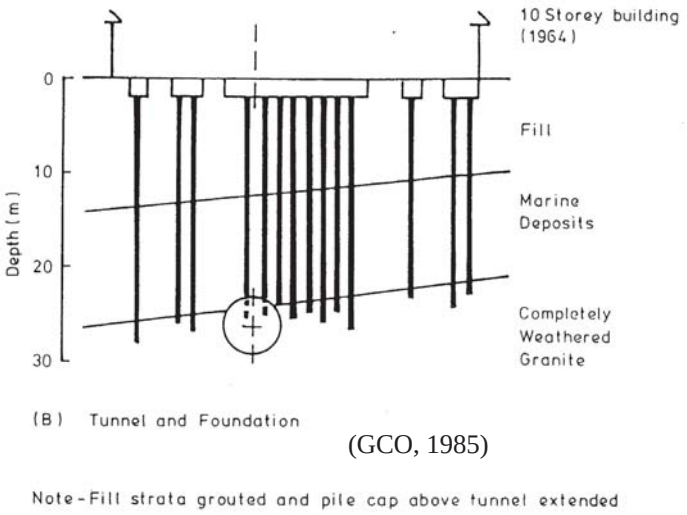
There are different analytical methods to assess the soil-structure interaction problem involving tunnel-pile-building interaction. The most rigorous method is the FE/finite difference method, and 2D FE analysis is often used. However, tunnelling is a 3D problem and in certain circumstances 2D modelling can give inappropriate predictions. This paper aims to shed light on the differences in prediction between 2D and 3D modelling of tunnelling in close proximity to pile foundations. The modelling is based on a well-documented tunnelling case history involving a piled building in Sheung Wan during the construction of the MTR Island Line in the 1980s. The limitations of 2D modelling of tunnelling will be explained.

2 BACKGROUND INFORMATION

The Hua Tai Building in Sheung Wan was constructed in 1964. It was a 10-storey reinforced concrete frame structure supported by 73 nos. of 0.457 m diameter Franki piles. Plate 1 shows a photo of the building before it was demolished recently for construction of the MTR West Island Line. Construction for a 6 m diameter overrun tunnel of the Sheung Wan Station, MTR Island Line was carried out in the 1980s (GCO, 1985), see Figure 1. The overrun tunnel was driven using an open-face shield machine under 2.6 bar compressed air pressure, see Figure 2. The ground conditions beneath the building generally comprise Fill, Marine Deposits (MD), Completely Decomposed Granite (CDG) and rock. The Standard Penetration Test (SPT) N values recorded in the adjacent boreholes are shown on Figure 3. The average groundwater level is +1.7 mPD.

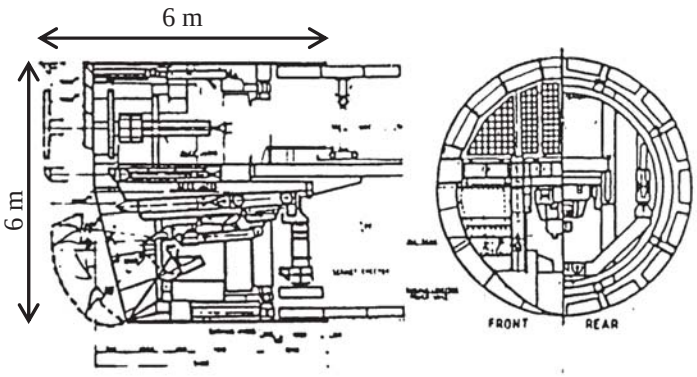


Plate 1: Photo of Hau Tai Building



(GCO, 1985)

Figure 1: Overrun tunnel and building foundation



(GCO, 1985)

Figure 2: Schematic diagram of open-face tunnelling shield

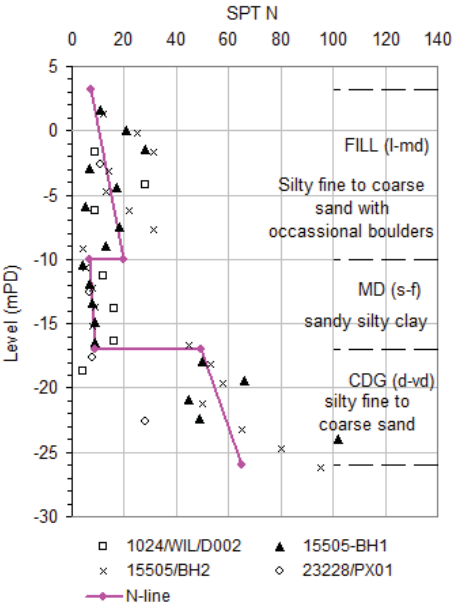


Figure 3: SPT N profiles

The overrun tunnel alignment intersected the building piled foundation, hence 17 nos. of the Franki pile toes were trimmed as the tunnel passed by. Polystyrene pads were installed at the bases of the trimmed piles to protect the tunnel. Prior to the pile trimming, improvement works were carried out at the building by grouting the underlying Fill to enhance its bearing capacity/stiffness (which made the existing piles redundant) and increasing the size of the central raft. Measurement for settlements of the ground and building was carried out during the tunnelling.

3 2D AND 3D FE MODELLING

The 2D analysis has been carried out using Plaxis 2D V9.02, and the 3D analysis using Plaxis 3D V2011.01. Figure 4 shows the 2D and 3D models. The input parameters for both 2D and 3D analyses are identical. All soils have been modelled using the linear elastic, perfectly plastic Mohr Coulomb model. The soil Young's moduli (E) adopt E=1.5N for Fill and MD, and E=3N for CDG (Chan, 2003). Table 1 summarises the soil input parameters.

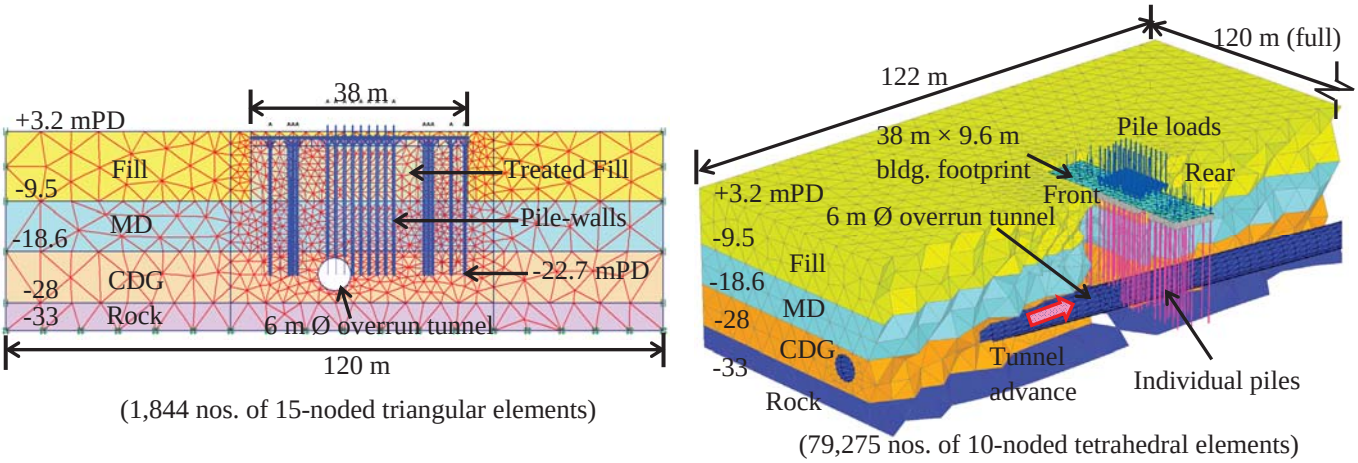


Figure 4: 2D and 3D models

Table 1: Soil input parameters

Soils	Material Type	Material Model	γ (kN/m^3)	E_{ref} (kPa)	ν (-)	c' (kPa)	ϕ' (Deg)	y_{ref} (mPD)	E_{inc} (kPa/m)	R_{inter} (-)
Fill	Drained	MC	18/19	11,250	0.3	0	32	3.2	1421	0.67
Treated Fill	Drained	MC	19	30,000	0.3	100	32	-	-	0.67
MD	Undrained	MC	16	9,900*	0.3*	40	0	-10	450	0.50
CDG	Drained	MC	20	148,500	0.3	5	35	-17	5133	0.67
Rock	Drained	Elastic	23	2.0×10^6	0.2	-	-	-	-	-
Pile caps	Non-porous	Elastic	24	14.2×10^6	0.2	-	-	-	-	-

Notes: MC = Mohr Coulomb, γ = unit weight; E_{ref} = reference stiffness, ν = Poisson's ratio, c' = effective cohesion, ϕ' = effective friction angle, y_{ref} = reference level, E_{inc} = increment of stiffness with depth so that the operating E = $E_{ref} + E_{inc}(y_{ref} - z)$ where z is the level below y_{ref}, R_{inter} = soil/structure interface strength.
*Plaxis will automatically convert the input effective E' and ν' to their undrained E_u and ν_u, and add the bulk modulus of water to the stiffness matrix when the material type is set undrained (Plaxis 2D V9.0).

The SGI tunnel linings are modelled using "Plate" structural elements. The building superstructure rigidity is modelled using a 'Plate" element placed on the top of the pile caps following Potts & Addenbrooke's (1997) approach. Table 2 presents the input parameters for the "Plate" elements.

Table 2: "Plate" input parameters

Structures	EI (kNm^2/m)	EA (kN/m)
SGI linings	1.8×10^4	6.6×10^6
Tower	9.2×10^7	1.7×10^7
Podiums	4.6×10^7	8.3×10^6
"Pile-wall" in 2D	2.0×10^7	1.6×10^6

Note: I = moment of inertia, A = cross sectional area

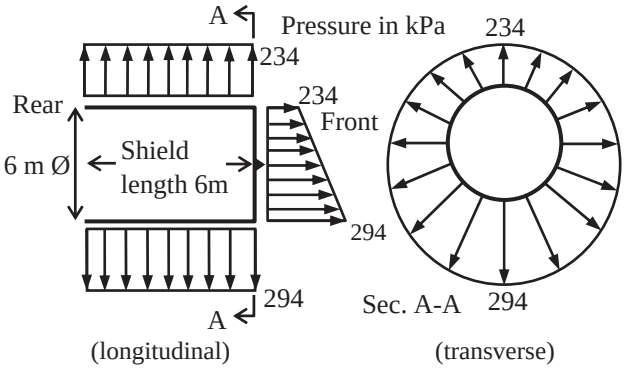
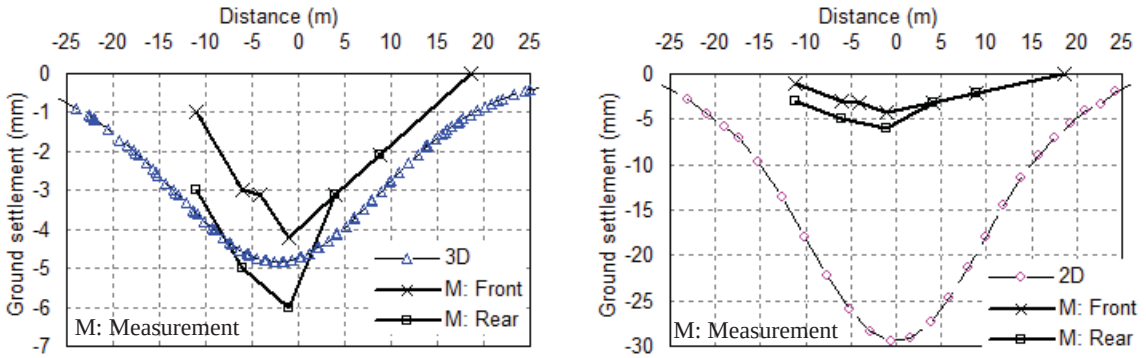


Figure 5: Tunnel support pressures modelled

In the 2D analysis the piles can only be modelled as "pile-walls" with the "Plate" input properties calculated from the EI and EA values per pile divided by the 1.5 m spacing of the piles into-the-plane, see Table 2. In the 3D analysis the 0.457 m diameter Franki piles are modelled using "Embedded Piles" structural elements with an E of 14.2 GPa and the shaft friction and end-bearing capacities dependent on the soil strength. For modelling the trimming of pile toes, a lower section of the piles between level -19.4 and -22.7 mPD was deactivated when the tunnel passes by. The tower loading is 428 kN per pile and the podium loading is halved.

In the 2D analysis the tunnel excavation is infinitely long into-the-plane, i.e. without specifically modelling the ground relaxation on the tunnel face and along the section of the tunnel boring machine (TBM). In the 3D

analysis the progressive advance of the TBM is modelled in a step-by-step manner (Lee et al, 2009). Figure 5 shows the support pressures modelled around the TBM for the 2D (transverse plane only) and 3D analyses. The support profile increases linearly from the crown 234 kPa to the invert 294 kPa (average 264 kPa being close to the quoted 260 kPa compressed air), and is equivalent to an overpressure of 20 kPa (i.e. pressure in excess of hydrostatic water pressure). In the 3D analysis the 6 m long TBM is advanced at 1.5 m increments, and for each advancement a 1.5 m wide lining ring is correspondingly erected behind the TBM with the support pressure removed in the lined tunnel section. This process is repeated as tunnelling progresses.



(a) 3D prediction (b) 2D prediction

Figure 6: Comparison of greenfield settlements

4 COMPARISON OF PREDICTIONS

To check the prediction of greenfield settlements, separate 2D and 3D analyses have been carried out in which the piles, pile cap and superstructure rigidity are not activated. The 3D analysis has modelled the progressive advance of tunnelling. Figure 6 shows that the 3D analysis predicts a maximum greenfield settlement of 5 mm, compared to the measured maxima of 4 – 6 mm. The predicted 3D settlement profile is wider than the measured profile. This discrepancy is likely caused by the limitation of the linear elastic, perfectly Mohr Coulomb model which does not consider the behaviours of non-linear soil stiffness from very small strains and soil anisotropy. The 2D analysis predicts a maximum settlement of 29 mm, being 6 times higher than the measurement. This is because the 2D analysis models an infinitely long tunnel excavation into-the-plane. In reality, the TBM is approximately 6 m long and during the tunnel excavation soil arching occurs both in the transverse and longitudinal directions (see Figure 7). The 2D analysis does not take into account the soil arching in the longitudinal direction, resulting in the predicted higher greenfield settlements.

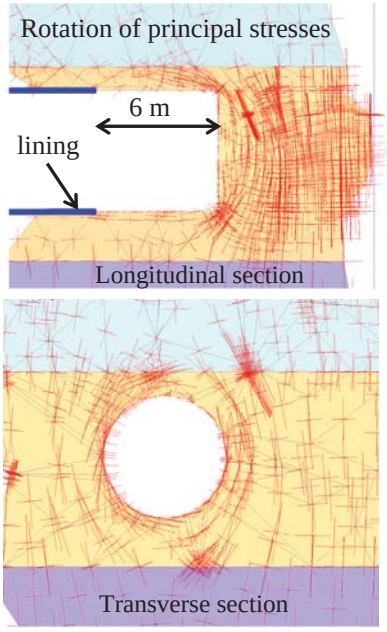


Figure 7: Soil arching around tunnel excavation

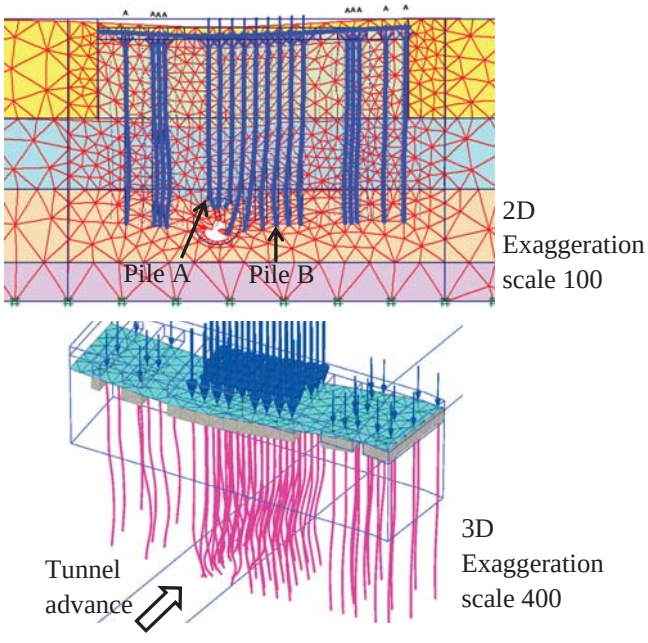


Figure 8: 2D and 3D mesh deformation

Figure 8 shows the deformation of the 2D and 3D models with the building and piles in place. Figure 9 compares the measured building settlements with the 2D and 3D predictions. The building settlement profiles predicted by the 3D analysis are very similar to the measured profiles, indicating that the superstructure stiffness influences more the predicted building settlement profile rather than the soil stiffness. The 3D analysis correctly predicts that the building rear settlements (maximum 7.5 mm) are higher than the building front settlements (maximum 6.5 mm), compared to the measured maxima of 9 mm and 6 mm respectively. The 2D analysis over predicts the building settlement with a predicted maximum of 11 mm, mainly because of the plane-strain tunnel excavation modelled resulting in higher ground settlements.

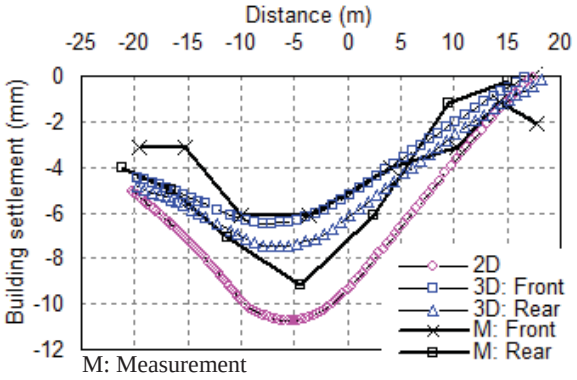


Figure 9: Comparison of building settlements

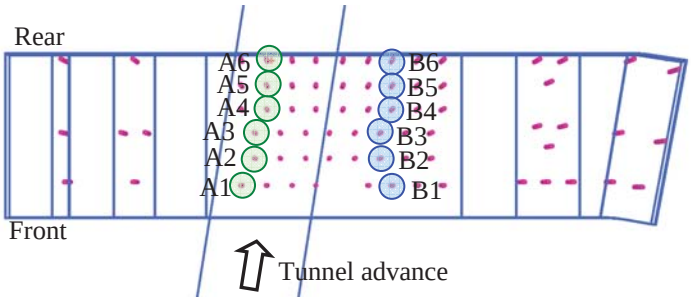


Figure 10: Pile locations in 3D analysis

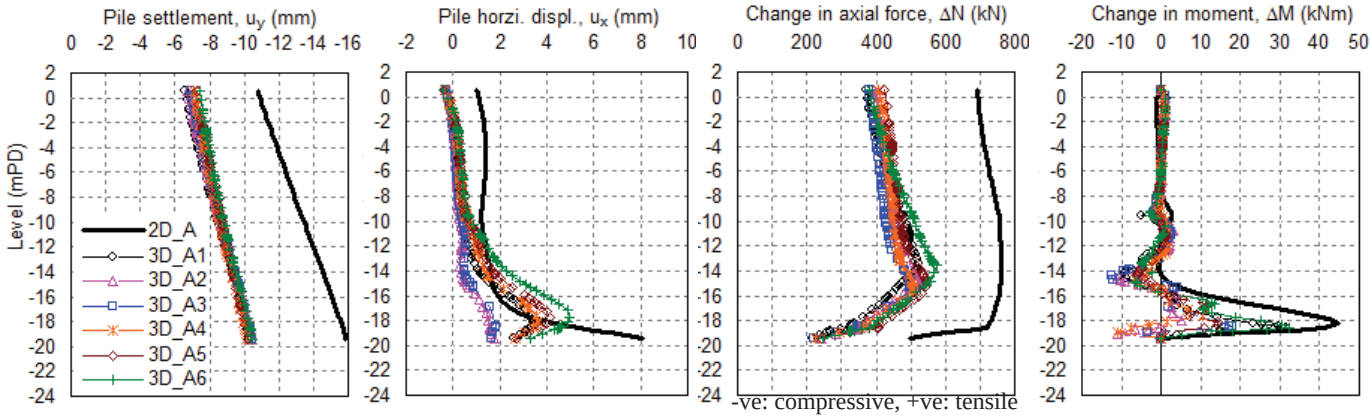


Figure 11: Comparison of pile behaviours of Piles Row A between 2D and 3D analyses

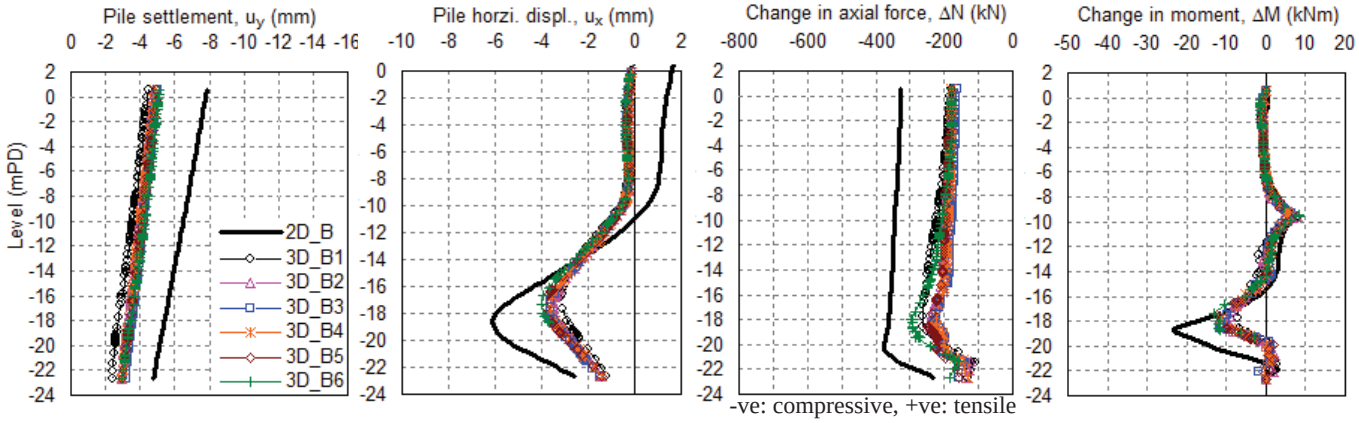


Figure 12: Comparison of pile behaviours of Piles Row B between 2D and 3D analyses

The locations of the individual piles in the 3D analysis are referred to Figure 10. Figure 11 compares the behaviours of Piles Row A between the 2D and 3D analyses involving the toe trimming. In terms of pile settlements (u_y) and transverse horizontal displacements (u_x), the 2D predictions are higher mainly because of

higher ground movements predicted by the plane-strain tunnel excavation. In terms of change in pile axial forces (ΔN) (positive for tensile force) and change in pile bending moments (ΔM), the 2D analysis predicts higher ΔN and ΔM. The predicted higher ΔN is because in 2D the modelled "pile-wall" has a larger surface area in contact with the surrounding soil than a real circular pile. The 2D analysis therefore tends to unrealistically mobilise a higher shaft friction along the "pile-wall", resulting in a larger change in the pile axial force. The ΔN in tensile mode is due to the tunnel excavation below the trimmed Piles Row A, reducing their toe bearing pressure and resulting in the pile settlements being larger than the soil settlements (i.e. positive skin friction is mobilised). The predicted higher ΔM in 2D is related to the predicted higher curvatures of u_x.

Figure 12 compares the behaviours of Piles Row B located at a horizontal distance of 3.5 m from the tunnel edge. The 2D analysis predicts higher u_y, u_x, ΔN (negative for compressive force) and ΔM than the 3D predictions. The ΔN in compressive mode is due to the negative skin friction generated on the piles, arising from the soil settlements being larger than the pile settlements at the location of Piles Row B.

5 DISCUSSION AND CONCLUSIONS

Table 3 summarises the predictions from the 2D and 3D analyses in terms of their predicted maximum values.

Table 3: Comparison of maximum values between measurements, 2D and 3D analyses

Items	u_y			u_x		ΔN		ΔM	
	Measured (mm)	2D (mm)	3D (mm)	2D (mm)	3D (mm)	2D (kN)	3D (kN)	2D (kNm)	3D (kNm)
Greenfield	6	29	5	-	-	-	-	-	-
Building	9	11	8	-	-	-	-	-	-
Piles A	-	16	10	8	5	761	575	45	32
Piles B	-	8	5	6	4	376	293	23	13

Note: u_y = settlement, u_x = transverse horizontal displacement, ΔN = change in axial force, ΔM = change in moment.

Using the same set input parameters and modelling conditions in the 2D and 3D analyses, the 3D analysis predicts the greenfield and building settlements very close to the measured settlements with a discrepancy of only 2 mm. In the absence of the measured pile behaviours, the 2D analysis predicts the maximum pile movements, changes in the pile axial force and bending moment which are 30% - 70% higher than those predicted by the 3D analysis. The limitations of the 2D analysis are summarised in Table 4 below.

Table 4: Comparison of 2D and 3D modelling of tunnelling near piles

Items	2D	3D
Ground/building movements	• Model plane-strain tunnel excavation into-the-plane. The effect of soil arching along the TBM length and around the tunnel face is not modelled. • Larger ground/building movements are predicted. • Model an infinitely long building structure and loads into-the-plane.	• Model progressive advance of TBM step-by-step. • Model soil arching effect in both transverse and longitudinal directions along TBM and around tunnel face. • Realistic ground/building movements are predicted. • Model the exact geometry of building relative to the tunnel alignment (e.g. skew orientation) and building loads.
Pile movements/forces	• "Pile-wall" is modelled, unrealistically providing a larger surface area for mobilising shaft friction. A higher change in pile axial forces is predicted due to a higher mobilised shaft friction. • "Pile-walls" result in compartmental effect, i.e. no soil movement across the pile walls. This will affect the predicted "pile-wall" movements. • Predict pile horizontal displacements in the direction perpendicular to the tunnel alignment only.	• Individual piles are modelled, modelling the real geometry of piles and hence realistic mobilisation of shaft friction. • Allow soil movement between piles, hence more realistic pile movements will be predicted. • Predict pile horizontal displacements in both directions perpendicular and parallel to the tunnel alignment.

The tunnel excavation beneath the Hau Tai Building in the 1980s has been analysed using both 2D and 3D finite element analyses. It is found that the 3D analysis predicts greenfield and building settlements in good agreement with the measurements, including the correct prediction of the observed building settlements being larger than the greenfield settlements. The 2D analysis over predicts the measured greenfield/building settlements and the pile response compared to the 3D prediction.

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3D Numerical Modeling of Development of Tunneling-induced Ground Arching

L. Tony Chen
Hyder Consulting Limited, Hong Kong

ABSTRACT

Tunneling will disturb the ground and cause stress redistribution. The stresses of the ground mass above the tunnel roof will reduce when it moves downwards, while those of the adjoining masses will increase, due to the arching effect. The arching effect plays a positive role in reducing ground loading on temporary support for tunneling. The mechanism of arching development during tunneling has been investigated by several researchers. The outcomes of these research works have provided useful insights about the topic but are however not comprehensive enough to address some practical issues. In this paper a series of 3D finite element analyses has been undertaken to provide further understanding of the topic and the results are discussed. It is shown that the extent of the arching effect is significantly influenced by both the tunnel heading position and the tunnel depth. The height of the arching zone above the tunnel roof can be estimated using a design chart which has been developed based on the analysis results presented in the paper. The ratio of the initial over the final arching height is found to vary between 0.5 and 0.6 within the parameters examined. The application of the design chart to a case history is also discussed.

1 INTRODUCTION

The phenomenon of ground arching is well recognized in the construction of underground structures such as tunnels. Arching can be described as a transfer of stresses between a yielding ground mass and its adjoining stable masses, resulting in a redistribution of ground stresses (Terzaghi, 1943). When the yielding mass tends to move downward, the shearing resistance will act upward and reduce the stress at the base of the yielding mass, as illustrated in Figure 1.

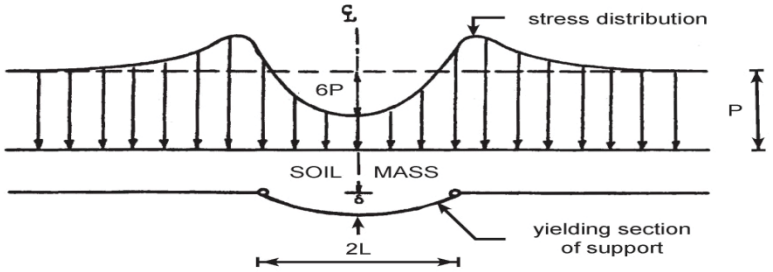


Figure 1: Stress distribution in the soil above a yielding base (Bjerrum et al, 1972; revised by Evans, 1984)

Tunneling will disturb the ground and cause stress redistribution. The stresses of the ground mass above the tunnel roof will reduce when it moves downwards, while those of the adjoining masses will increase, due to the arching effect. The arching effect plays a positive role in reducing ground loading on temporary support for tunneling.

The ground loading on the support system is often estimated based on the arching theory proposed by Terzaghi (1943) which classifies the ground into several categories and provides simple equations for estimating ground loads. Terzaghi's theory involves simplified assumptions and is based on a two dimensional condition which cannot provide an insight into how the ground arching develops during tunneling. It is

expected that the extent of the arching effect around an active heading will be affected by ground conditions, tunnel heading position, size of excavation, cross section, depth of tunnel and round length.

Several researchers have attempted to study the mechanism of arching development via numerical methods and centrifuge model tests, e.g.; Lee et al (2006) and Chen et al (2011). The outcomes of these studies have provided an insight into the behavior of arching associated with tunneling but are however not comprehensive enough to address some practical issues.

The purposes of this paper are three-fold, namely: firstly, to provide further understanding of the arching mechanism during tunneling through a series of 3D finite element analyses; secondly, to develop a design char for estimating the arching extent for practical design use; and thirdly, to illustrate the applicability of the design chart.

2 METHOD OF ANALYSIS

Figure 1 shows the dimensions of a tunnel under consideration, both the span B and height H_t being approximately 6m. These dimensions are similar to those of a real tunnel project currently under design and construction, as will be further discussed later in the paper. The depth to tunnel roof is denoted as H_c, while the vertical extent of the arching zone is denoted as h_a.

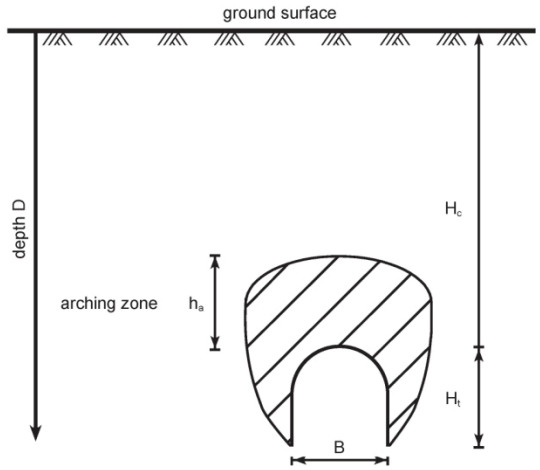


Figure 2: Tunnel geometry under consideration

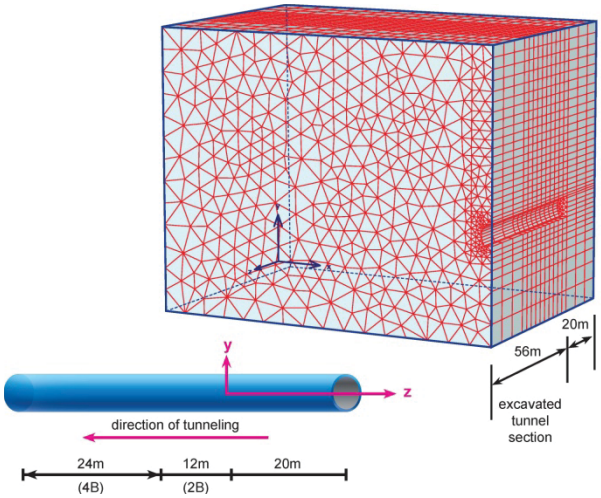


Figure 3 Adopted 3D numerical mesh

For simplicity, the ground is assumed to consist of only rock having a GSI value of 25, representing a weak rock. The rock is simulated as an elastic-perfectly plastic material obeying the Mohr Column failure criterion. The associated effective cohesion c', friction angle ϕ' and Young's modulus E_m are estimated based on Equations (1) – (3) as proposed by Hoek & Brown (1997) and the adopted values are presented in Table 1.

$$\sin \phi' = (k-1)/(k+1) \quad (1)$$

$$c' = \sigma_{cm}/(2\, k^{0.5}) \quad (2)$$

$$E = (\sigma_{ci}/100)^{0.5}.10^{(GSI - 10.40)} \text{ (in MPa)} \quad (3)$$

Where, k is the slope of the line linking σ_1' and σ_3'. σ_{cm} is the uniaxial compressive strength of the rock mass and σ_{ci} is the intact rock strength.

Table 1: Adopted geotechnical parameters for rock having GSI of 25

GSI	Cohesion c' (kPa)	Friction angle ϕ' (degree)	Young's modulus E_m (MPa)
25	250	18	120

The 3D numeral modeling is carried out via Plaxis 3D Tunnel version 2.2. Figure 3 shows the adopted 3D Plaxis model and its coordinate system with the z axis being in the tunnel longitudinal direction. Due to its symmetry, only half of the problem is considered in the Plaxis model and the symmetry boundary (on the right hand side) is set to align with the tunnel centre line. To minimize boundary effects, the left boundary is set at 50m away from the tunnel centre line, while the bottom boundary is set at 20 m below the tunnel invert level. The depth to tunnel roof H_c will be varied to investigate its effects on the analysis results.

The tunnel heading advances in the z direction, from + z to – z with its origin coinciding with the observation plane (hereafter referred to as OP) where the ground stresses and settlements due to tunneling are under investigation. The effects of heading progress are investigated for a tunnel length of 6B, consisting of 2B behind (positive z) and 4B ahead (negative z) of OP. To minimize boundary effects in the z direction, the following assumptions are made:

- Before the concerned tunnel excavation is started, a 20 m long section of tunnel has been excavated and properly supported.
- The concerned tunnel excavation stops at 20 m before the z boundary.

3 ANALYSIS RESULTS

As a first case scenario, the depth of the tunnel roof is chosen as 100m. Analyses for other tunnel depths will also be carried out to investigate the effects of the depth on the analysis results.

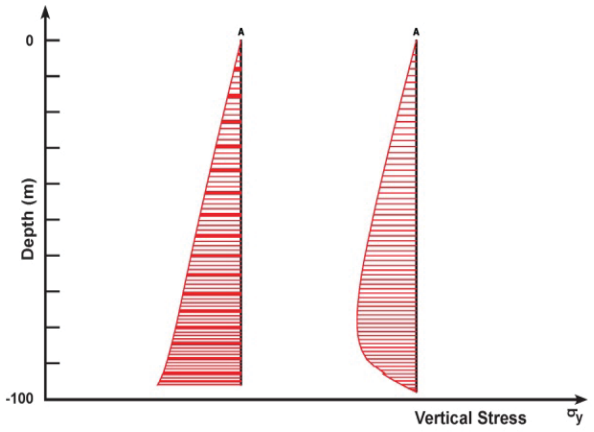
Due to space limitations, discussions of the analysis results will be limited to the vertical stresses and settlements above the tunnel roof.

3.1 Typical results

To illustrate the influence of the tunnel heading position, the analysis results corresponding to two distances, i.e. z = -1B (i.e 6m behind OP) and 1B (i.e. 6m ahead of OP), will be compared.

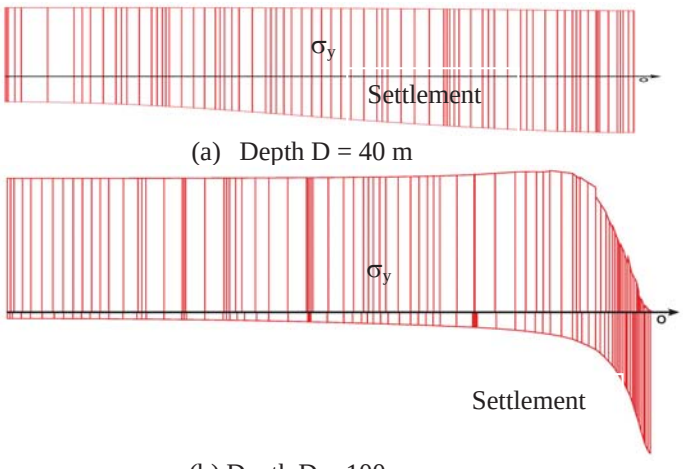
Figure 4 (a) shows the vertical stress profile for z = -1B, while Figure 4(b) shows that for z = 1B. It can be seen that the former profile remains almost linear, while the latter profile becomes non linear within a zone above the tunnel roof, exhibiting the arching effect. This appears to indicate that the tunnel heading position has significant effects on the redistribution of stresses.

Where arching occurs, the inflection point between the linear and non linear portions of the vertical stress profile may be used to indicate the extent of the arching effect, i.e. h_a.



(a) z = -6 m (b) z = 6 m

Figure 4: Typical vertical stress profiles



(a) Depth D = 40 m

(b) Depth D = 100 m

Figure 5: Typical horizontal profiles of settlements and stresses

To further illustrate the behavior of the ground where arching occurs, a comparison of the horizontal distributions of settlements and vertical stresses within and above the arching zone, represented by a depth D = 40 m and D = 97 m, respectively, is shown in Figure 5. The following observations can be made;

(1) Within the arching zone (i.e. D = 100 m), the settlement and vertical stress above the tunnel roof sharply increases and decreases, respectively. The settlement decreases rapidly while the vertical stress becomes almost constant outside of the tunnel side boundaries.
(2) Above the arching zone (i.e. D = 40 m), the change in both the settlement and vertical stress profiles is insignificant.

3.2 Estimation of arching extent

The curves of the vertical stress along the tunnel centre line versus the normalized depth are plotted in Figure 6 for various distances of the tunnel heading away from OP. As shown, the arching effect is not developed until the tunnel heading moves to very close to OP, and becomes more and more pronounced as the tunnel heading moves away from OP. It becomes almost fully developed when the heading arrives at approximately four times the tunnel span (i.e. 4B) away ahead of OP, beyond which the condition becomes two dimensional.

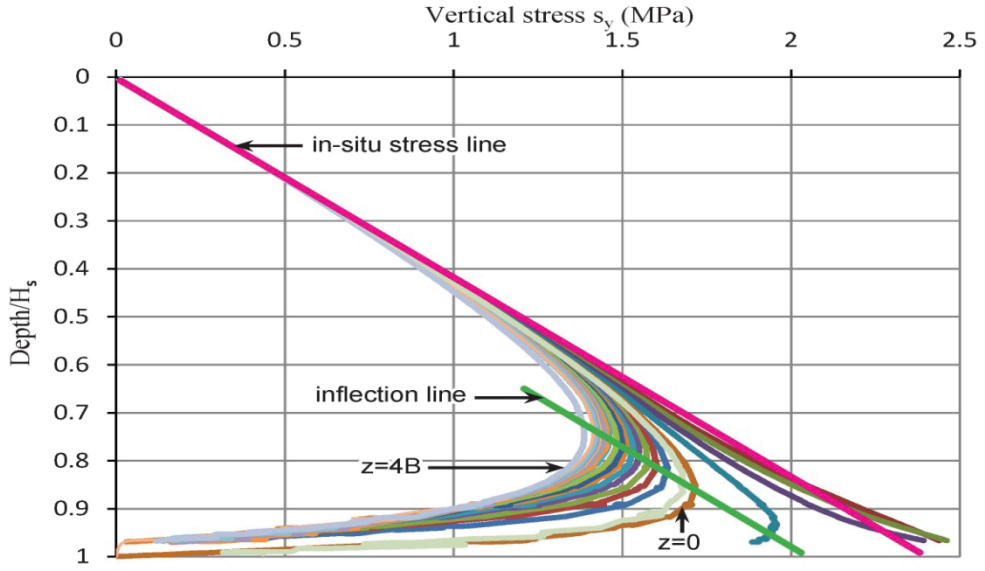


Figure 6: Vertical stress vs normalized depth for H_c = 100 m

For simplicity, the arching is assumed to start developing at the tunnel heading location (i.e. z = 0) and become fully developed when the heading is at a distance of 4B ahead of OP. The overburden force, F_a, acting on the tunnel roof, considering the arching effect, can be expressed by Equation (4), similar to that proposed by Terzaghi (1943).

$$F_a = \gamma h_a \quad (4)$$

Equation (4) can be used to estimate either the initial or final overburden force, corresponding to the initial or final arching zone.

From Figure 6, the arching extent h_a can be estimated to be approximately 13m and 25m, respectively, resulting in a ratio of 0.52. From Equation (4), the initial force F_{ai} is roughly 52% of the final force F_{af}. The difference can be expressed by a reduction factor, R, as defined in Equation (5). For this case, R is equal to 0.52.

$$R = F_{ai}/F_{af} \quad (5)$$

Interestingly, it is found that the line (hereafter referred to as Inflection Line) linking the inflection points of the profiles exhibiting the arching effect is approximately parallel to the in-situ stress line, as shown in Figure 6. This relationship can be used to determine the arching extent as will be discussed in detail below.

Similar analyses have also been undertaken for other tunnel cover depths, including H_c = 20 m, 50 m and 150 m. The vertical stress profiles corresponding z = 0 and 4B for different tunnel depths are plotted in Figure 7.

As shown, the vertical stress-depth curve for H = 20 m is not smooth, indicating that the arching effect may not be developed if the cover depth is too shallow. On the other hand, the vertical stress-depth curves for other deeper tunnel depths are smooth and exhibit similar features to those discussed above for H = 100 m. Figure 7 also shows that the arching extent increases with tunnel depth.

From an examination of the curves shown in Figure 7, it is found that the Inflection Line for any tunnel depth can be approximately expressed by Equation (6).

Inflection Line:

$$Y = C_1X + 0.14/\cos(\text{atan } C_1) \quad (6)$$

where C1 is the slope of the in-situ stress line.

The lines (hereafter referred to as Initial and Final Arching Line, respectively) linking the inflection points of the initial and final vertical stress – normalized depth curves, respectively, for different tunnel depths can be expressed by Equations (7) and (8), respectively.

$$\text{Initial Arching Line: } Y_i = 0.007X_i + 0.84 \quad (7)$$

$$\text{Final Arching Line: } Y_f = 0.027X_f + 0.70 \quad (8)$$

By comparing Equations (7) and (8), it is found that the reduction factor, R, varies between about 0.5 and 0.6 within the parameters examined.

Equations (6) to (8) can be used together to determine the extent of the arching zone corresponding to either the initial or final arching zone, for a given tunnel depth, which can then be used to estimate the corresponding force acting on the tunnel roof.

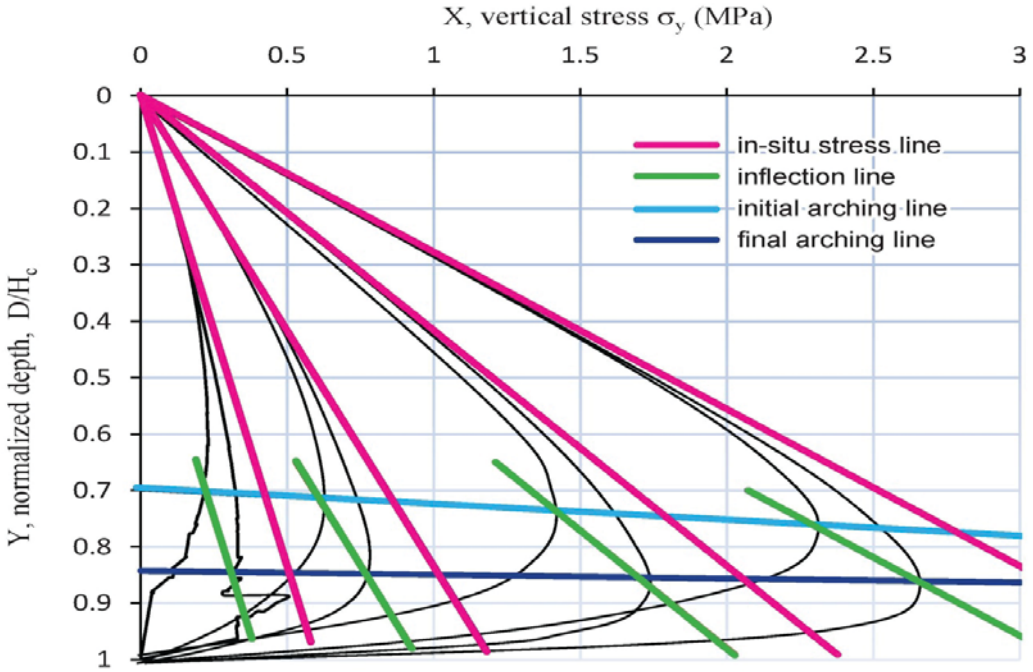


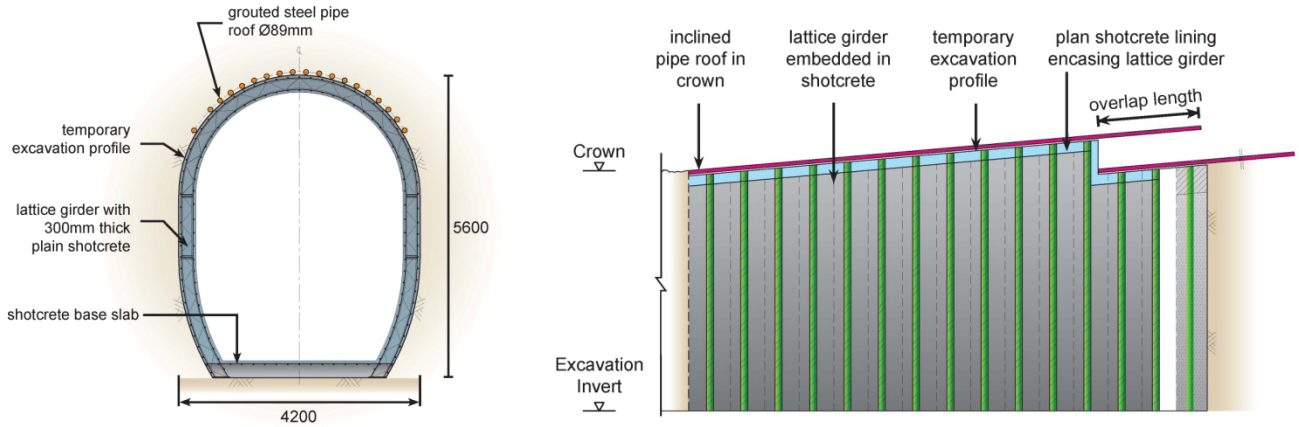
Figure 7: Vertical stress versus normalized depth for different tunnel depths

4 APPLICATION TO A CASE HISTORY

The design chart shown in Figure 7 has been recently applied to the design of a temporary support system for a tunnel project in Hong Kong. The tunnel is located at about 45m below the ground level and runs through fault zones in some locations. The temporary support system consists of horizontal pipe roof supported by lattice girders and shotcrete. The adopted geotechnical parameters are similar to those shown in Table 1, while the details of the temporary support system are shown in Figure 8.

The 300 mm thick shotcrete layer is to be applied in two phases, each being 150 mm thick. The tunnel excavation is advanced at 1m round length and the distance between the tunnel face and the last installed girder is limited to 1.5 m.

Following Equations (6) and (8), the arching extent when it is fully developed can be estimated to be 11 m which gives an estimated rock load of 210 kN based on Equation (4). The reduction factor R is taken as 0.65. The two case scenarios shown in Table 2 were analyzed for the temporary support design. The forces developed in the support system were calculated via Phase 2 and were used in the structural design of the lattice girders and shotcrete layer.



(a) Cross section (b) Long section

Figure 8: Details of temporary tunnel support system for a case history

Table 2: Cases analyzed for design of temporary support

Case	Longitudinal distance (m)	Shotcrete thickness (mm)	Shotcrete strength (MPa)	Reduction factor R
1	0.5+ 0.75=1.25	150	33	0.65
2	0.5+0.5=1	300	50	1

5 CONSLUSIONS

In this paper the mechanism of arching development during tunneling is investigated via a series of 3D finite element analyses. The following major conclusions can be made based on the analysis results presented:

(1) The arching effect starts to develop at a very short distance ahead of the tunnel heading and becomes fully developed at about four tunnel span behind the face.
(2) The inflection point of a vertical stress profile above the tunnel roof may be used to define the extent of the arching zone.
(3) The arching extent increases with increasing tunnel depth.
(4) A design chart (Figure 7) has been developed for estimating the height of the arching zone corresponding to either the initial or final arching zone. For a given tunnel depth, this can be done by drawing three lines defined by Equations (6) to (8). The arching heights can then be used to estimate the forces acting on the tunnel roof.

(5) The ratio of the initial overburden force over the final overburden force is about 0.5 to 0.6 within the parameters examined.

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Recent Experiences of Numerical Prediction & Assessment – Excavation over a Tunnel of Unbolted Segmental Tunnel Lining

J.B. Wang & Leslie Swann
Jacobs (China) limited
Lawrence S.Y. Lee
Geotechnical Engineering Office, Civil Engineering and Development Department, Hong Kong SAR
Stephen Reynolds
Jacobs (UK) limited

ABSTRACT

A 700 m long Grade Separated Road (GSR) is proposed to link the new Terminals and on to a new set of remote stands at an Airport in UK. Construction of the GSR involves excavation adjacent to and over the existing live underground tunnels. To investigate the impacts on the unbolted concrete segmental tunnel lining from the proposed GSR work, a 3D model together with a series of 2D models were developed considering the site history and construction sequences proposed. Both short and long term behaviour of the existing underground tunnels during and after the GSR construction was investigated, showing the proposed GSR construction method was a viable solution to minimise the impacts on the existing tunnels.

1 INTRODUCTION

As part of the airport development, a Grade Separated Road (GSR) is proposed to link new Terminals and on to a new set of remote stands. The road has an overall length of 700 m and comprises two taxilane underpasses to avoid conflict between road vehicles and aircraft. Construction of the proposed GSR involves the installation of embedded secant pile retaining walls, casting bridge decks across the underpass sections, and subsequent excavation within the walls to depths of up to 11m with temporary propping, in places, to reach formation level for casting the base slab.

The GSR passes over and runs sub-parallel to the existing underground tunnels which were built in the 1970s using an unbolted concrete segmental tunnel lining. The excavation for the GSR, over and adjacent to the live tunnels, causes great concern over safe operation of the trains. ELS scheme was developed to limit the impacts on the existing tunnels associated with the GSR construction consisting of installing 900mm diameter secant pile walls as temporary retaining walls during the excavation stage and also as permanent retaining walls for the GSR. Plaxis 2-D Numerical analyses were carried out at selected critical sections to estimate the associated ground movements and to assess the impacts to the underground line. A FLAC ^{3D} model was also established to investigate the impacts where the proposed GSR crosses over the existing tunnels at a skew direction.

This paper presents the case study from the numerical modelling aspect, concentrating on the 3-D analysis in particular. Compared with 2-D analysis, the benefits of the 3-D modelling for prediction with higher accuracy are highlighted.

2 SITE AND GROUND CONDITIONS

Ground conditions at the site, comprise Made Ground, underlain by River Terrace Deposits overlying London Clay. The existing ground level is generally flat, approximately at 23.1 m AOD. Groundwater levels recorded on site were typically at 3m below ground level.

The thickness of the River Terrace Deposits varied between 3.5 m and 4.7 m, and are described as medium dense to very dense sandy gravel with occasional cobbles. London Clay has typically been described as firm to stiff, grey extremely closely to closely fissured clay with occasional silt laminate and partings and occasional medium sand to fine gravel size shell fragments and is heavily overconsolidated.

The GSR is about 700 m in length and 12 m wide, lying above the existing live tunnels. The GSR ramp crosses over the tunnels at an angle of 57 degrees and then runs approximately parallel to the tunnels with a minimum 4.3 m spacing. At the crossing area, the GSR ramp is about 3 m deep and is approximately 8 m above the tunnels. The depth of the GSR increases to about 8 m at the down ramp near the proposed left taxilane underpass. The internal diameter of the Piccadilly Line tunnels is 3.81 m, formed by 22 numbers of precast concrete segments. The tunnel rings are approximately 0.6 m long and 152.5 mm thick. Airbus A380 / Boeing 747 aircraft loading of total 9000 kN is considered in the design of the bridge and retaining walls.

3 METHOD AND SEQUENCE OF CONSTRUCTION

It is proposed that the excavation for the GSR be supported by secant pile walls and that the secant pile walls will also form the permanent walls of the GSR. The secant piles above the tunnels will terminate at approximately 6 m above the tunnels and the piles shall also be a minimum of 3 m away from the tunnel structure as required by the tunnel exclusion zone.

One to two layers of temporary props are proposed to support the secant pile walls during GSR construction. At the underpass area, with a bridge deck at the top, no props are proposed during excavation. Construction of the GSR shall adopt top-down method under the bridge deck and bottom-up construction sequence for other parts. The typical construction sequence using bottom up sequence for the GSR sections outside the Taxilane Underpass is presented below:

(1) Install secant piles
(2) Excavate to 0.5m below bridge deck
(3) Install /casting bridge deck
(4) Excavate to final excavation level
(5) Cast base slab and skin walls and connect it with the secant pile walls

4 3-D NUMERICAL MODELLING

Excavation for the construction of the GSR will change the stresses and induce ground movement around the existing tunnels. The western ramp of the GSR crosses over the existing tunnels at a skew, which requires a more sophisticated study on the likely impacts on the existing tunnels. A three-dimensional numerical model was developed and analysed with Finite difference software FLAC ^{3D} version 3.1.

4.1 Modelling domain and boundary conditions

The three-dimensional model developed is a rectangular block of 102 m long x 100 m wide x 63 m deep comprising a total of around 60,000 soil zones. A general view of the domain is presented in Figures 1 & 2 below.

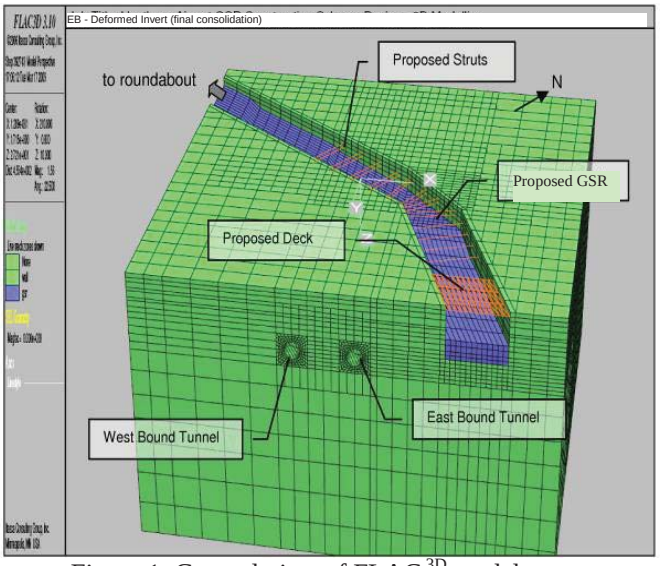


Figure 1: General view of FLAC ^{3D} model

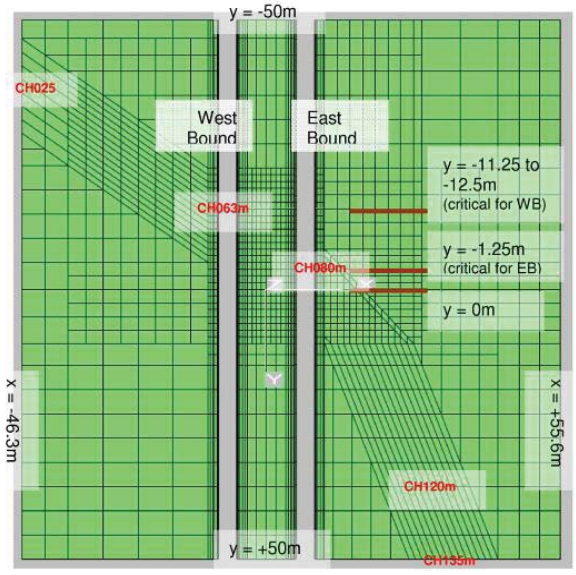


Figure 2: Plan view at 13.9 m bgl

The model simulates the western portion of the proposed GSR which runs above the existing twin tunnels at an acute angle, between chainage of 25 m and 135 m as shown in Figure 2. A 100 m length of the twin tunnels are covered in the model, running parallel to the y-axis from y = -50 m to y = +50 m.

For the four vertical external boundaries, horizontal movement perpendicular to the plane is restricted while movement in the plane of the vertical boundary is allowed. The vertical movement is not permitted at the base boundary of the 3D model.

4.2 Soil elements and soil properties in the model

The ground under consideration comprises London Clay overlain by Made Ground and River Terrace Deposit. A constitutive model of liner elastic perfectly-plastic with Mohr-Coulomb failure criterion was adopted for the Made Ground and the River Terrace Deposit. The nonlinear elastic properties of the London Clay shows a strong dependency of the soil stiffness on the strain levels experienced, i.e. higher stiffness at lower strains. The nonlinear tangent elastic properties of the London Clay can be described by the following equations, given by Jardine et al (1986):

$$E_u = c_u \left\{ A + B\cos\left(\alpha I^{\gamma}\right) - \frac{B\alpha\gamma I^{\gamma-1}}{\ln(10)}\sin\left(\alpha I^{\gamma}\right) \right\} \quad (1)$$

where
c_u = undrained shear strength
$I = \log_{10}(\varepsilon_a / C)$
A, B, C, α, γ = material-specific constants, and
$\varepsilon_a = \frac{\sqrt{2}}{3}\left\{ (\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2 \right\}^{0.5}$
ε_1 , ε_2, ε_3 = principal strains (or principal deviatoric strains)

The soil parameters adopted in the 3D model are summarized in Tables 4.1 and 4.2 below:

Table 4.1: Soil Parameters

Soil Type	γ (kN/m^3)	υ' / υ	k_o	c' /c_u (kPa)	ϕ' (°)	E' (kPa)	k (m/s)
Made Ground & River Terrace Deposits	20	0.2	0 – 1.6mbgl: 0.5 1.6 – 5.5mbgl: 0.4	0	36	37,500	2 x 10^{-4}
London Clay	20	0.49	5.50 – 11.25mbgl: 2.6 11.25 – 31.75mbgl: 2.2 31.75 – 63.00mbgl: 1.2	Increase with depth (86 kPa at London Clay top	-	-	2 x 10^{-9}

Table 4.2: Jardine Model Parameters for London Clay

A	B	C (%)	α	γ	ε_{min} (%)	ε_{max} (%)
1350	1350	0.001	1.319146	0.66336	0.0011	0.3

4.3 Tunnel lining In the model

The existing tunnel lining comprises 22 unbolted pre-cast concrete segments with a ring length of 600mm. Since it is impractical and unnecessary to build a 3D model with those lining segments modelled precisely, an "equivalent stiffness method" by Muir Wood (1975) is adopted, which suggests:

$$I_e = I_{joint} + (4 / n)^2 \times I_{segment} \quad (2)$$

where
I_e = equivalent moment of inertia for a continuous "liner" element in model
I_{joint} = moment of inertia of joints between lining segments (= 0 if no structural connection)

$I_{segment}$ = moment of inertia of segments
n = number of segments (n = 22 in current case)

The tunnel lining were modelled as a liner structure element in the FLAC ^{3D} model that can take the tunnel hoop stress, bending moment and shear stress and also friction between the soil and the tunnel lining. The secant pile wall and base slab of the GSR were also modeled as liner structure elements, having isotropic linear elastic properties of the concrete material. Beam structure elements were adopted for the struts modelling.

4.4 Proposed works and modelling sequence

(a) Establishing initial conditions
To establish the existing conditions of the ground and the existing underground structures, the 3-D model was set up to include the major construction history of the existing tunnels including the construction year and tunnel face volume loss during tunnelling. The West Bound (WB) Tunnel was constructed first and then East Bound (EB) Tunnel. The tunnel face volume loss was simulated by excavating the tunnel area allowing a 2% volume loss in term of tunnel cross-sectional area. Tunnel lining is then installed to support soil stresses around.

The tunnel construction stage is followed by long-term consolidation in which any excess pore pressure generated due to tunnel excavation would be fully dissipated, assuming a permeable tunnel lining. After full consolidation, present (or existing) condition of the domain has been arrived at. All displacement vectors are then set to zero before commencement of the proposed works.

(b) Modelling of construction Sequence
The excavation and GSR construction are designed to be carried out in 3 bays. Subsequent to the installation of secant pile wall which is assumed as "wished-in-place", excavation to 1mbgl and installation of the first layer of 711 x 14 CHS struts and bridge deck are modelled. Construction of the GSR is then carried out bay by bay in three stages as shown in Figure 3. Further excavation, installation of second layer of 711 x 14 CHS struts and casting of base slab in one bay would then be completed, before a similar operation for the next bay.

Upon completion of base slab construction for the Bay 3, all struts would be removed. Construction of the airfield pavement would then be modelled. Finally, a long-term consolidation is then carried out. The construction stages modelled are detailed in Section 3.

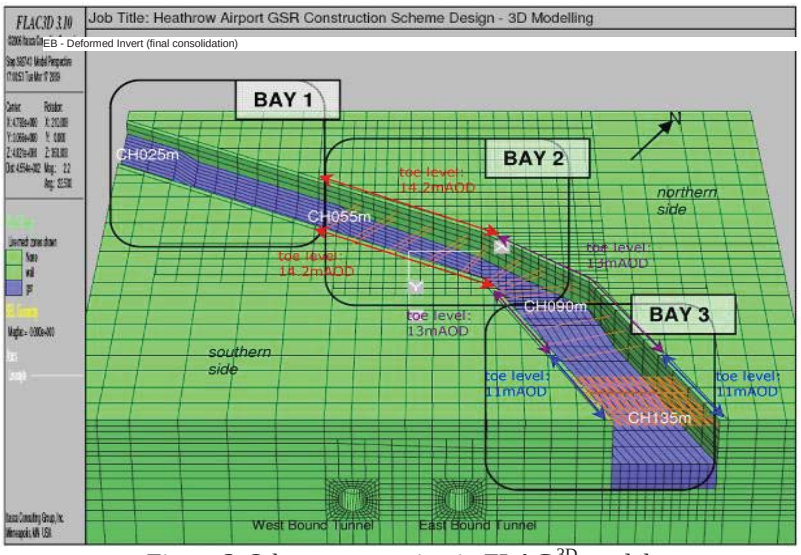


Figure 3: 3-bay construction in FLAC ^{3D} model

5 RESULTS AND DISCUSSION

5.1 Predicted tunnel movement

Long term ground movement in a horizontal plane is presented in Figure 4 below. It can be observed that the line of zero transverse (in x-direction, see Figure 4) ground movement vector generally matches the centreline

of GSR. The ground heave effect including ground movement towards the excavation from both sides is illustrated in Figure 5 below.

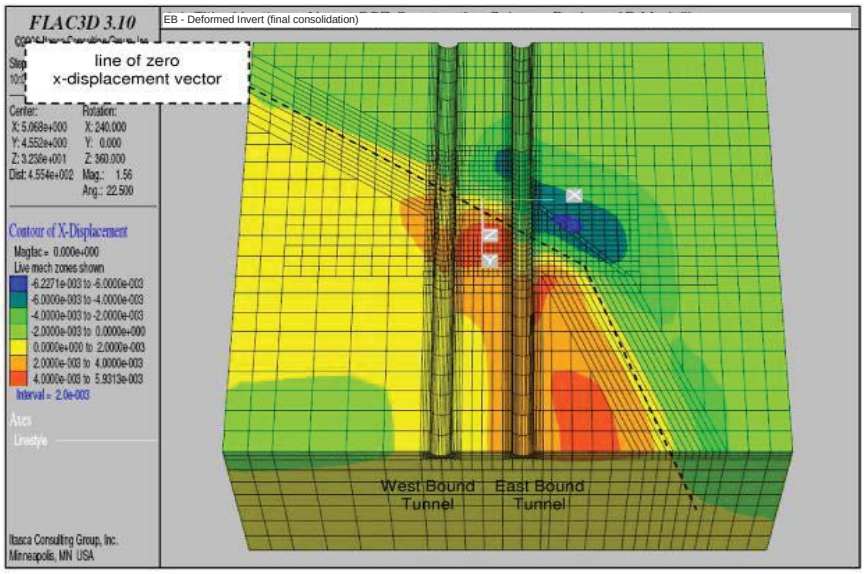


Figure 4: Contours of transverse ground movement in horizontal plane

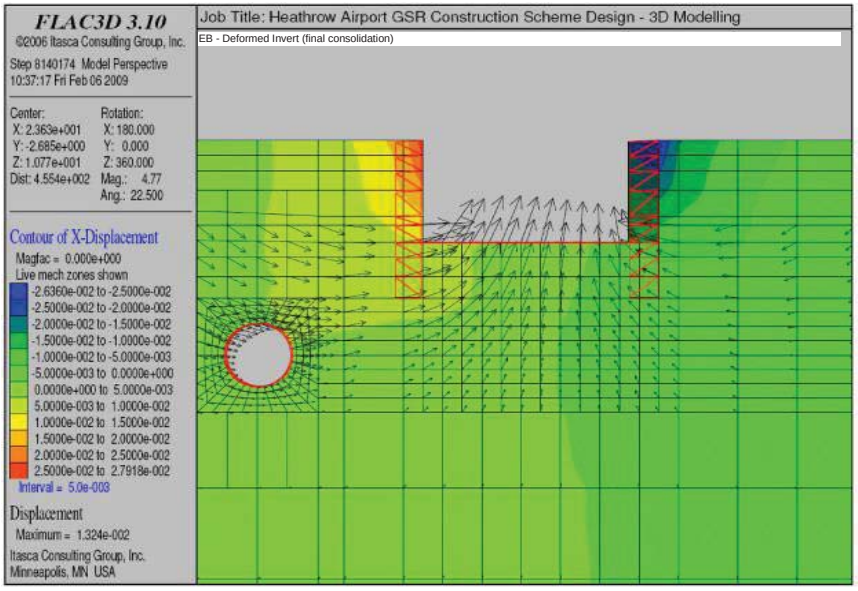


Figure 5: Typical ground movement vector below proposed GSR

(a) Eastbound tunnel

The predicted maximum tunnel lining deformation of the crown and invert along the longitudinal direction for the EB Tunnel is given in Figure 6. The tunnel lining deforms in a similar way to the surrounding ground. Maximum deformation of the tunnel lining occurs at the area where the proposed GSR runs across the EB Tunnel. It experiences an upward movement of around 8.5 mm and 1.8 mm at the crown and invert respectively at this critical location, inducing a differential vertical movement of approximately 6.6 mm (moving apart). It is also worthwhile to notice that such maximum deformation occurs at the end of excavation to the final excavation level in Bay 2, when the excavation is directly above the tunnel.

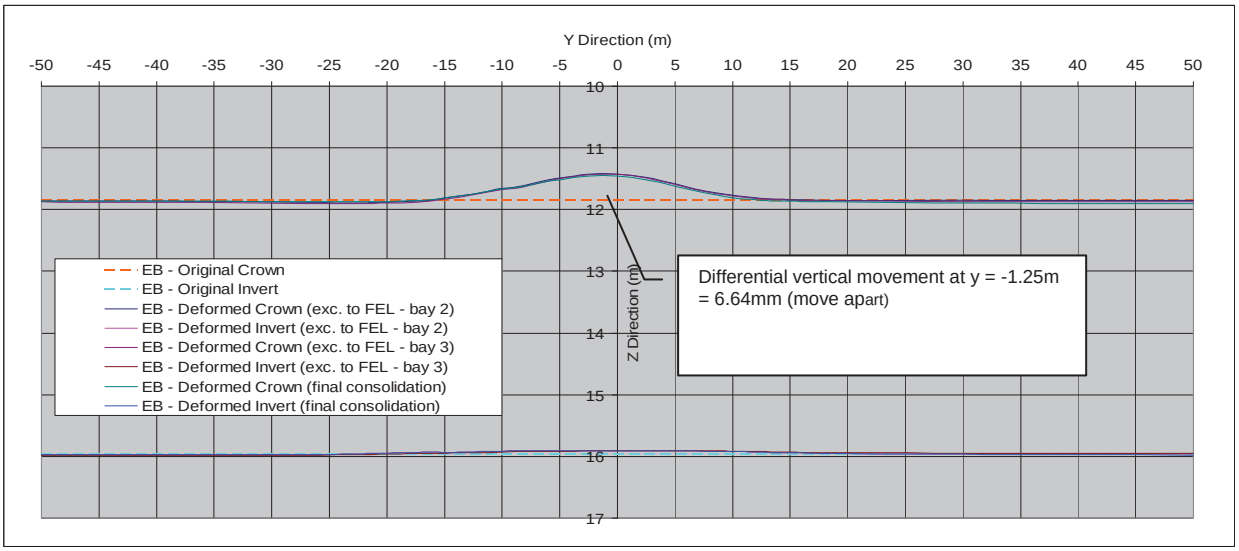


Figure 6: Predicted lining deformation along EB Tunnel (exaggerated by 30 times)

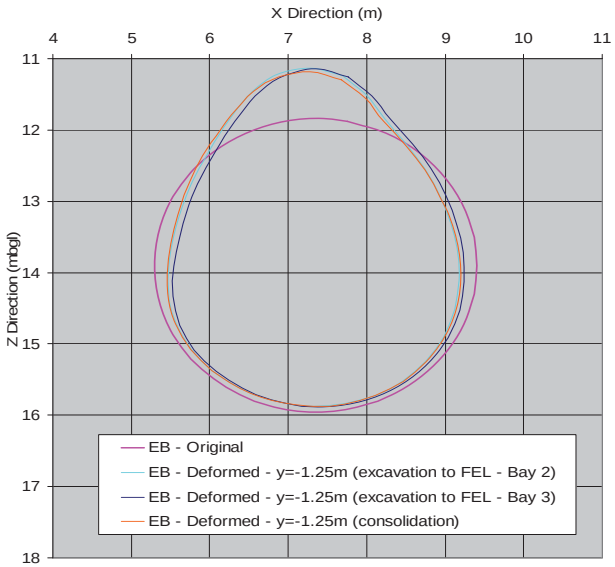


Figure 7: Predicted transverse lining deformation (exaggerated by 30 times)

The predicted tunnel lining deformation of the critical section for the EB Tunnel is plotted in Figure 7. The tunnel is squeezed by approximately 4.8 mm at the end of excavation to final excavation level in Bay 2.

(b) Westbound tunnel

Maximum lining deformation of 4.1 mm and 1.4 mm are predicted respectively at the crown and invert along the longitudinal direction for the WB Tunnel, inducing a differential vertical movement of about 2.7 mm (moving apart). Again, the maximum deformation occurs at the end of the excavation to the final excavation level in Bay 2, when the excavation is directly above the tunnel. The predicted maximum differential horizontal movements of the tunnel lining side walls is 1.9 mm with a maximum lateral movement of 2 mm occurring at the eastern side wall of the tunnel after excavation to final level in Bay 2.

5.2 Predicted tunnel lining forces

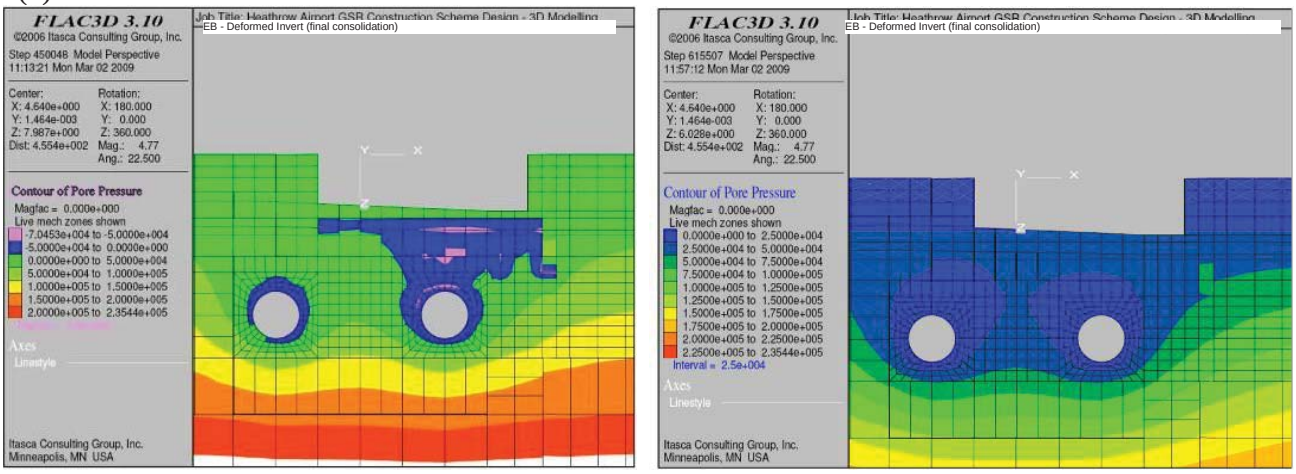
The predicted bending moment of the tunnel lining in the transverse (hoop) direction after excavation completion remains small throughout the GSR construction as expected. The hoop force of the tunnel lining is reduced slightly after GSR excavation, from average 403 kN/m of the tunnel lining compression at the existing

condition down to average of 397 kN/m. Therefore the overall structural force change of the existing tunnels associated with the proposed GSR construction is expected to be small.

5.3 Predicted long term behaviour

Figure 8(a) below presents the contours of total pore pressure along critical section (i.e. y = -1.25m) at the end of proposed construction. Negative excess pore pressure is developed in London Clay due to clay swelling upon unloading in the undrained condition.

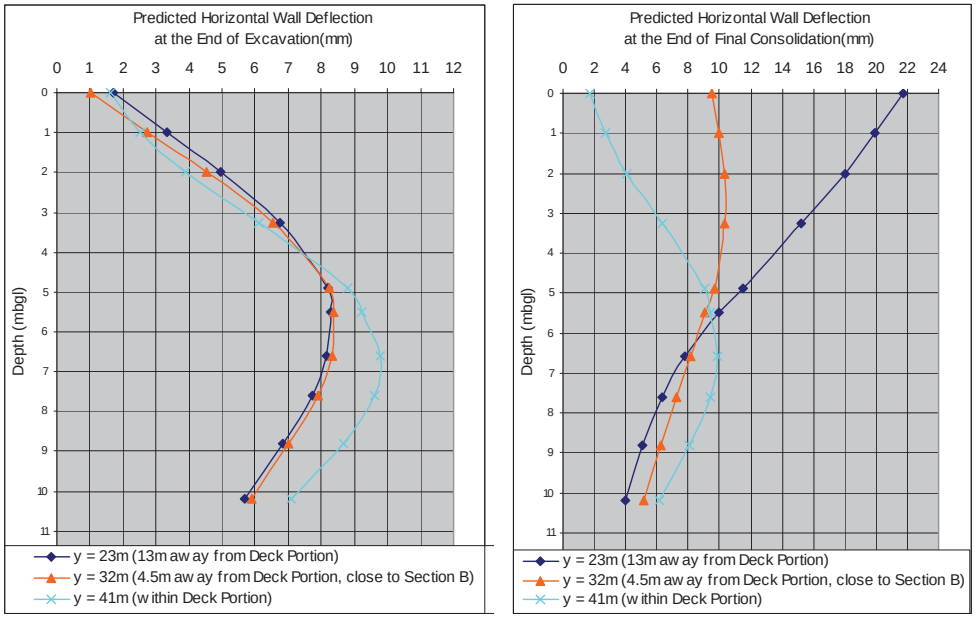
Subsequent to completion of all proposed construction activities in the model, consolidation is carried out for the long term behavior. Excess pore pressure generated in previous construction stages would be dissipated within around 10 years time. Contours of total pore pressure at the end of full consolidation are given in Figure 8(b).



(a) End of GSR construction (b) Long term consolidation

Figure 8: Predicted total pore pressure distribution at different stages

Figure 9 below shows a development of the lateral movements of the secant-pile wall at various locations during and after GSR construction. The restraining effects on the wall deflection from the bridge deck of the taxilane underpass are shown in the Figure 9. The predicted wall top deflection at the end of excavation from 3D modelling is similar at various locations as indicated in Figure 9 (a). However, the long term wall top deflection developed from less than 2mm at the decking area, to around 9.5 mm and 22 mm respectively at 4.5 m and 13 m away from the underpass decking (see Figure 9(b)).



(a) During GSR construction (b) After GSR construction

Figure 9: Development of lateral movements of the secant-pile wall

2D analysis at section B which is about 4.5m away from the decking with a y value of 32 m (see Figure 2) was also carried out using geotechnical software Plaxis. The Plaxis 2D analysis adopted Hardening Soil Model with small strain stiffness (HSsmall) for London Clay while Mohr-Coulomb soil model having properties as given in Table 4.2 was used for other soil stratus. Conventional bottom-up construction sequence with one layer strut was followed at section B. The 2D analysis results give a wall top deflection of 23 mm at long term compared with less than 10mm predicted by 3D analysis. The 2D analysis of the section B is unable to consider the restraining effects from the adjacent underpass deck, hence significantly overestimate the wall movements. It is also interesting to note that, from 3D modelling, the wall top deflection reached 22 mm at the location of about 13m away from the underpass deck, which is similar to the deflection predicted by 2D analysis at section B. Thus the restraining effect on the wall deflection from the underpass deck decreases to negligible at a distance of about 1.5 times of the retaining height away from the restraint. Maximum of 22 mm and 23 mm of the wall deflection were predicted by 3D analysis and 2D modelling respectively, both occurring at wall top and having similar values.

6 CONCLUSION

Impacts on the existing live underground tunnels caused by the proposed GSR construction were assessed with numerical modelling including a 3D modelling. The 3D model analysis results suggest that the deformation of the segmental tunnel lining is small and the structural force change of the tunnel lining is also insignificant. Therefore, with the ELS scheme developed and the construction in sequential bays, the likely impact on the existing tunnels associated with the proposed GSR construction is expected to be small. Comparison of the 3D and 2D analysis results for the long term behaviour of the retaining wall near the underpass deck illustrated the 3D modelling ability to analysis problems in 3D environment.

ACKNOWLEDGEMENT

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development of the Government of the Hong Kong Special Administrative Region.

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Settlement due to Under-drainage: Transient Characteristics and Control Measures

Angus Maxwell & Graham Kite

Maxwell Geosystems Ltd

ABSTRACT

The flow of water into tunnels and the lowering of ground water levels is a transient process governed by the permeability of the ground, the storage of the various reservoirs and the available recharge. The resulting settlement is a function not only of the compressibility of the deposits but also of their ability to drain. This paper draws on a large database of information both from Hong Kong and worldwide to examine the transient behavior of the ground during drawdown and reviews the effectiveness of surface recharge systems.

1 INTRODUCTION

Under drainage is the process by which a tunnel (sink) drains water from an aquifer which has limited immediate recharge. This can be as a consequence of an impermeable upper layer, an aquiclude, and/or due to lateral recharge being restricted.

The net result of under-drainage is a lowering of groundwater pressure in materials, some of which may be compressible. This paper looks beyond the conventional steady state view of under-drainage and addresses transient characteristics. It summarises the authors' experience of rocks exposed in an uninterrupted transect across 20km of Hong Kong during mining of the Strategic Sewage Disposal Scheme Phase 1.

2 FLUID FLOW IN HONG KONG STRATA

Hong Kong is a mountainous region with a thick mantle of residual soil and saprolite, draped in complex sediments comprising intercalations of terrestrial colluvium and alluvium with marine sedimentary deposits. Reference to Fookes (2007) illustrations on geomorphology and Leeder (2011) on sedimentary process show immediately the difficulties faced when attempting to assess the impact of a sink installed within this ground. Imagine a thick sequence of marine deposits overlaying the model shown in Figure 1 and it is clear how inadequate conventional analyses are. Predominantly models are two dimensional or if three dimensional they are so simplistic as to be misleading.

The following provides a list of the typical characteristics of materials and their impact on the analysis of ground water flow:

Water: Tests carried out during SSDS Phase 1 shows that three types of ground water exist in Hong Kong. Freshwater from rainfall percolating downwards, seawater percolating sideways as the freshwater table varies and deep groundwater which has been in place for millions of years and whose chemistry changes in response to hydrothermal incursions from depth, through diffusion and through in-situ reaction with the ground. Seawater and fresh water may percolate downwards only after this ancient water is displaced. The characteristics of seawater and freshwater are well known but the deeper water is characterized by very high biochemical oxygen demand, high Fe content and conductivity, the natural consequence of which is to promote corrosion and rapid deposition of salts, in particular Ferric Hydroxide on exposure to air.

Rock: May be massive or closely jointed but back analysis shows that bulk rock permeabilities in Granites and Tuff vary from 5×10^{-8} m/sec to 5×10^{-7} m/sec with an average of 1×10^{-7} m/sec being equivalent to steady state tunnel inflow of 1 litres/minute/metre of tunnel at 100 m of head. The porosity of igneous rocks is considerably less than 1% (except for some rare tuff breccias) and increases with weathering. Flow tends to

occur along discontinuities which may be cooling joints, un-roofing joints, tectonic joints or volcano-sedimentary features such as tuff breccias. Typically, rapidly cooled igneous/volcanic rock tends to be more closely jointed and more permeable than granites cooled gradually. Those rich in volatiles such as Rhyolites may also have open structures and close jointing and as a result are difficult to drill. The actual permeability in rock depends as much on the size of the discontinuities as on their connectedness and this is most influenced by the extent to which mineral deposition has or has not taken place. Deposition of quartz, calcite, chlorite, various iron compounds and clays minerals including the ubiquitous kaolin are hard to predict and occur in several phases. Faults may not always be the main conduits so often assumed since they are frequently observed to possess significant clay in the gouge and decomposition and therefore annealing on one or other side of the fault plane. These may sometimes behave as dams and it may be that the inrush of water experienced on hitting faults is a function of the high head and not necessarily of high permeability.

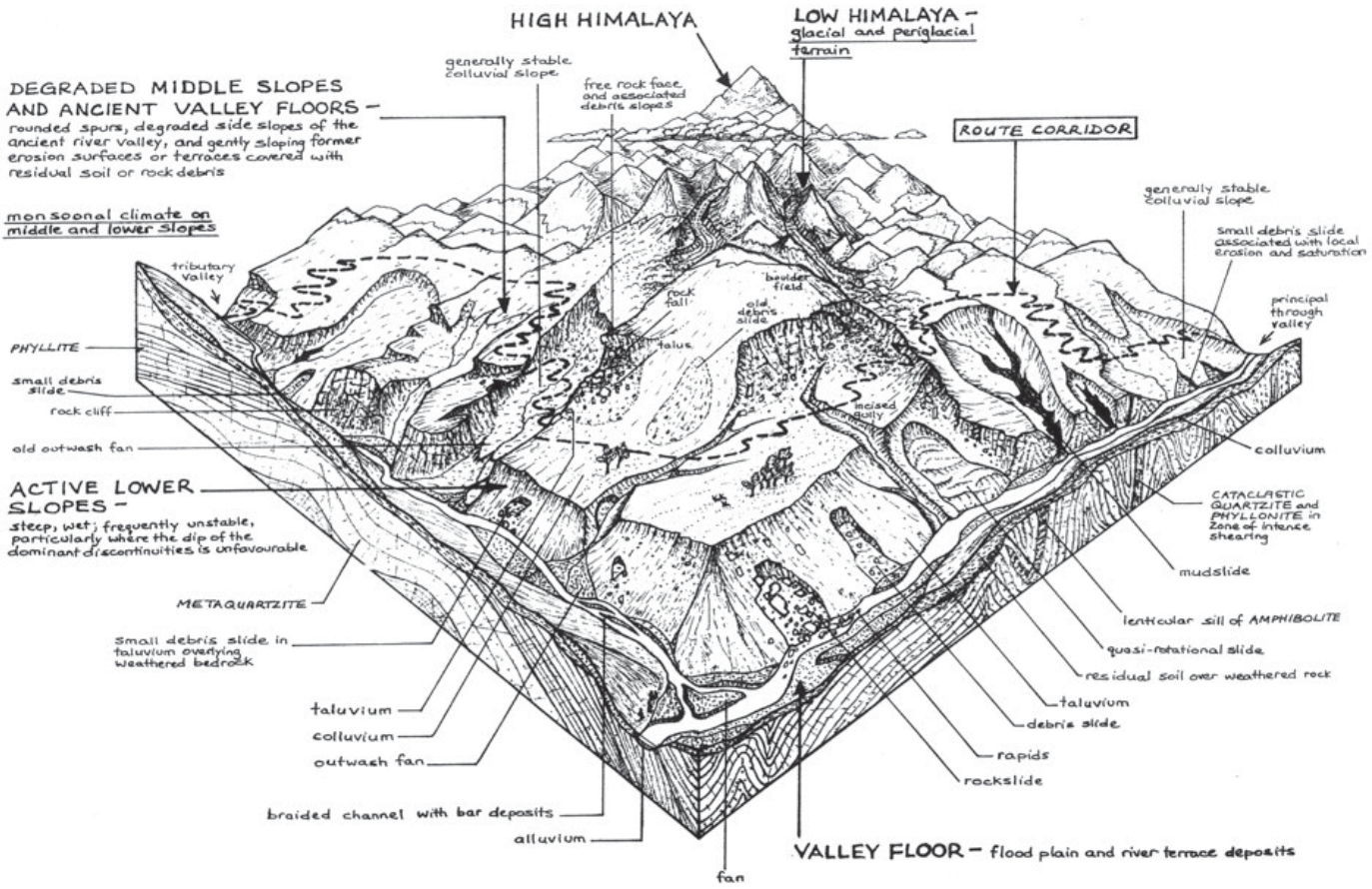


Figure 1: Fookes' Mountain Model (Fookes et al, 2007)

Decomposed rock and Saprolite: For the purpose of this classification we group all decomposed rock from grade IV to VI within this category and as such the permeability can vary typically from 1 x 10^{-7} to 5 x 10^{-6} m/sec. Porosity varies from <5% to up to 15%. Flow in decomposed rock is governed by relict fissures and soil pipes which may or may not be connected since the arrangement of soil pipes may also be governed by the terrain of the decomposed rock surface.

Colluvium: The distribution of colluvium is dependent on the palaeogeomorphology of the decomposed rock/rock surface even where hidden by other sediment. Colluvium may be intercalated with alluvium at the margins of upstanding areas of rock. Colluvium can be highly heterogenous and comprise transported boulders and/or chaotic slide materials on slopes and/or debris flow materials which are strongly channelized in their upper reaches but are spread thinly over wide areas at their distal ends. The key characteristic of colluvium is that it is discontinuous and is unlikely to behave either as an aquiclude or as a significant pathway for water migration.

Alluvium: Alluvium is an often mis-used catch all phrase and is often considered to be a single material for the purpose of hydrological assessment. In reality this is a complex arrangement of gravels, sands, silts and clays which form an alluvial plain. Permeabilities can range from 1 x 10^{-5} m/sec in gravels to as low as 1 x 10^{-10} m/sec in consolidated clays. Porosity can also vary from 20% to 40%. Of all materials it is the alluvium which most influences the movement of groundwater during under drainage.

Marine deposits: Marine deposits are deposited over the alluvium during sea level rise and may vary from well winnowed medium sands to silts and clays depending on the energy of the marine environment in which they are deposited. Small amounts of clay and silt can significantly affect the permeability and particle size distribution curves should be used to extend in situ and lab permeability tests in order to estimate relative permeability.

Fill: It is often assumed that fill is highly permeable, however this will very much depend on the type of fill, marine sand or CDG, and level of compaction. As a result permeability can vary from 1 x 10^{-6} to 1 x 10^{-5} m/sec. The vertical permeability of the fill will also largely be controlled by layers of re-precipitated mud which may have accumulated at the base of the reclamation and within the reclamation during storm periods. It is often assumed that dredged channels for seawalls are always pathways for recharge whereas in reality the foundation alluvium material may not be particularly permeable even discounting the potential for considerable thickness of re-precipitated mud.

Artificial materials: The presence of linear construction projects, which often comprise kilometers of diaphragm wall, will severely affect the natural drainage paths within the soil strata. Assumptions of lateral drainage, particularly from hillside catchments, will be incorrect. The effect of retaining systems on natural drainage has been seen on countless projects, where installation of D-walls has resulted in elevated water levels and significant (up to 5 m) differences in piezometers from one side of an excavation to another. Nevertheless in most designs water pressure is assumed to exert evenly on both sides of an excavation.

3 MODELLING AND PREDICTING FLUID FLOW

3.1 Simple models

The conclusion appears to be that it is too complex to model. Certainly any simple model cannot be considered to be in any way to represent what will happen except if that model is constructed on a small scale and a lot of GI data is available to confirm that the materials between source and sink are hydrogeologically consistent. On a basin scale analyses will be grossly misleading. There is still room for modeling but only as a test of sensitivity. Particular factors to identify are: What are the key drainage materials? What is the recovery time likely to be for various assumptions of recharge boundary, aquiclude permeability and thickness? What is the specific storage in these key materials and how will they be affected by depressurization?

Once these key permeable horizons have been identified it is necessary to try and identify where they are situated, the connectivity between them, and the potential rock conduit (if the tunnel is in rock) and to areas which may be sensitive to depressurization.

3.2 Mapping fluvial and alluvial networks

A concept that is common in geology is the facies map. Used frequently in petroleum and groundwater exploration this maps units not by their geological unit or formation name but by material or origin. A map therefore can represent the distribution of material type at a particular time, depth or relating to a particular stratigraphic event, ie an unconformity.

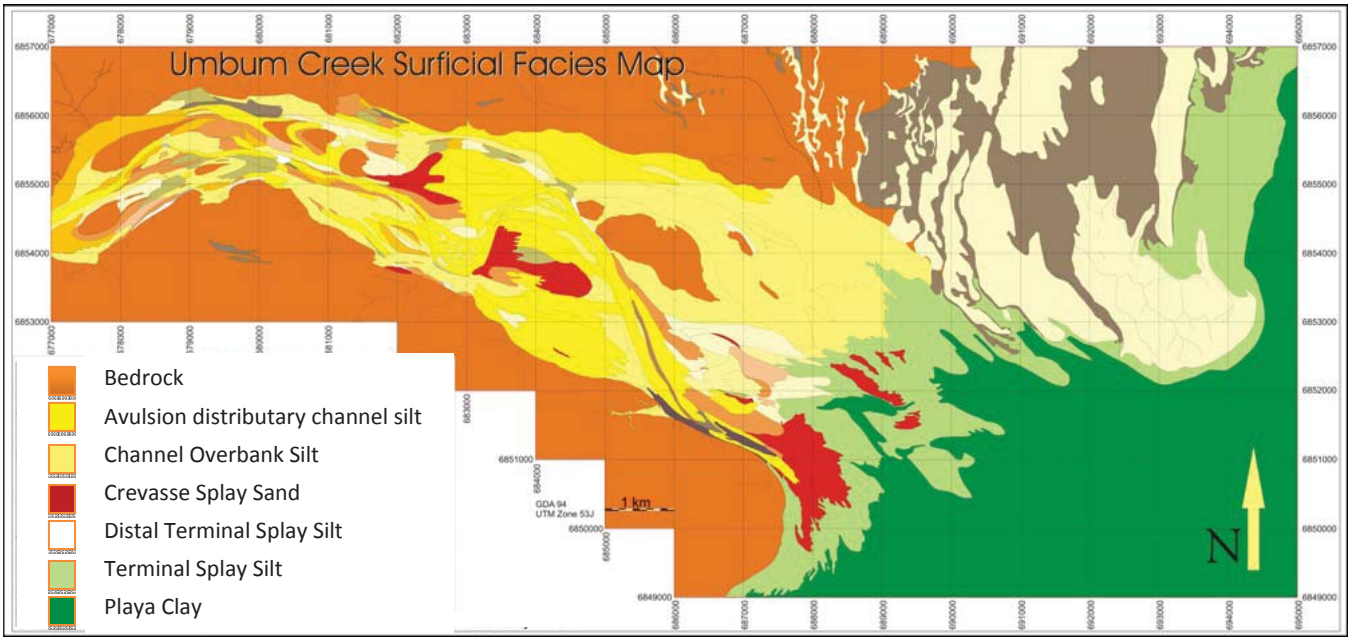


Figure 2: Sedimentological facies map of alluvial fluvial deposits (Reilly 2007)
Note: The map shows the distribution of material types and depositional environments at a particular chronostratigraphical boundary.

Such maps are easily prepared from GI logs by picking a chronostratigraphical boundary and mapping the distribution of material types at that boundary. Key horizons are: the material at the base of the alluvium which is unconformable on the decomposed rock and the alluvial material immediately below the base of the marine deposit which sits unconformably on the alluvium. These two horizons will determine the connection to the rock/decomposed rock flow paths and the locations where depressurization will affect the compressible marine deposit. Facies maps provide an assessment on whether the assumptions used in numerical modeling are likely to be correct. Consider the example in Figure 2. For any arbitrary section, is it sensible to assume all the flow is in the line of the section? Is it sensible to assume that the materials identified are in homogenous layers or will flow tend to be concentrated on particular pathways? In reality fluvial systems are far too complex to be modeled on a catchment scale by simple layer cake models. It is far better to identify a realistic hydrogeological model using basic geological tools and relationships and use this as a basis for simpler experiments to investigate the potential sphere of influence of a groundwater sink and the likely consequences.

3.3 Transient vs steady state

When water is encountered in a tunnel the flow path is immediately determined by the high permeability geological units. The speed of reaction is determined by the porosity of the medium. In rock, highly connected and conductive pathways can produce drawdowns in piezometers several hundred metres away whilst rock piezometers several 10's of metres away remain unchanged. Rock fracture porosity is so low that a small quantity of water extracted will have a large effect.

The effect will be dependent of how deep within the rock the piezometer tip is placed ie away from recharge from nearby more permeable horizons, and how close this is to the tunnel sink. Commonly these preferential water pathways are associated with deep weathering and some recharge will be drawn from overlying saprolitic soils producing depressurization and consolidation. Whilst compressibility is relatively low, these layers can be thick and consolidation is rapid, typically completed within 2-3 months.

The most important pathway for depressurization is that which connects fissure flow in the rock to highly permeable gravel and cobble horizons in the lower alluvium. From here connectivity is assured since the deposits originated in a fluvial/alluvial environment and are probably still operating as conduits for ground water drainage.

3.4 Typical observations

For open tunnels in rock the first interception with water is through probe holes. These will produce immediate drawdowns in some rock piezometers which will recover once grouted. Drawdown in overlying compressible deposits will not occur if the holes are allowed to drain for periods of only a few hours. Longer periods of drainage may start to produce drawdowns which will start consolidation at rates governed by the permeability of the consolidating material. The amount of consolidation will depend on the length of time the change in stress operates. This is termed drawdown days and is determined by the length of time the sink is in operation and the rate at which the ground will recharge once the sink is removed (Figure 4).

In the example in Figure 3, the rate of discharge is related to the permeability of the aquifer tapped by the sink and in this case the initial rate of drawdown is chosen to be 2m/day and it is stopped after 2 days. The rate of recharge is determined by the surrounding permeabilities at the recharge boundaries and the specific storage of the aquifer. It is set at $^2/_3$ of the drawdown rate in this example which was typical of the recovery of piezometers in SSDS after transient depression due to probe hole drilling. Note that actual recharge rates may be many times lower than this value.

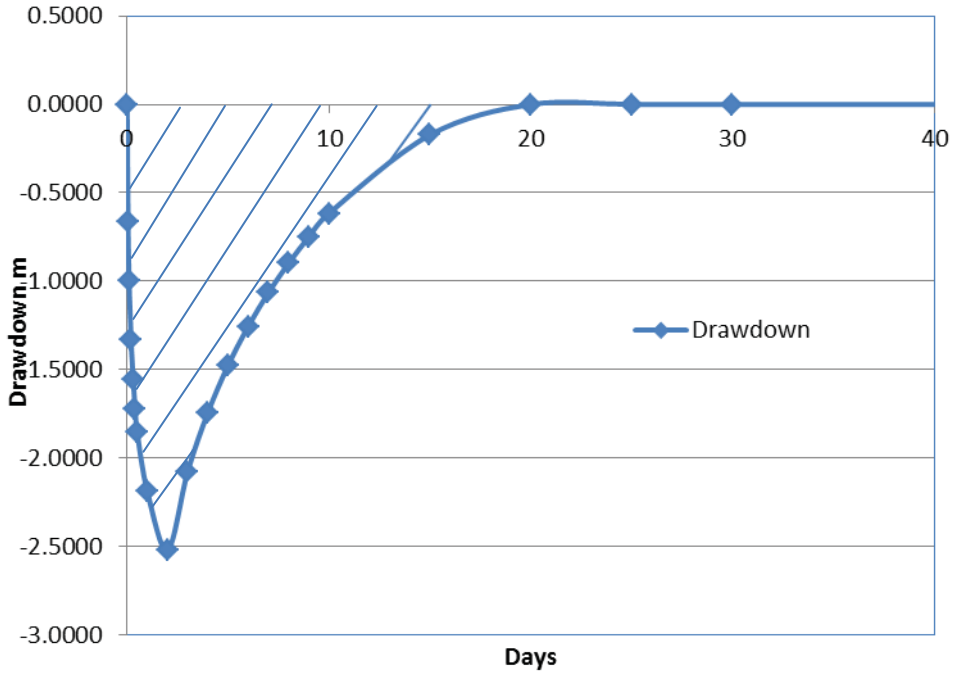


Figure 3: Draw down days (shaded) is a balance between the rates discharge and recharge

Settlement can be calculated by integrating the curve for the number of drawdown days and applying simple one-dimensional consolidation theory the result of which are shown in Figure 4. Table 1 summarises some typical soil compressibility for Hong Kong materials. Those for materials with high permeabilities are taken from the results of field observations including both drawdown related consolidation and rebound. It is noticeable that the results of back calculation of M_v and C_v values for decomposed soils suggest significantly higher compressibility and lower coefficients of consolidation for decomposed soils particularly within fault zones.

Table 1: Typical parameters for settlement analysis

	HDG	CDG/CDV	Colluvium	Alluvial Sand	Alluvial Clay	Marine Sand	Marine Clay
Permeability m/sec	5.00E-06	2.00E-06	1.00E-05	1.00E-05	1.00E-09	2.00E-05	1.00E-08
Mv m^2/MN	0.01	0.1	0.0001	0.014	0.05	0.02	2
Cv m^2/yr	50	400	100000000	1000000	100	10000000	2

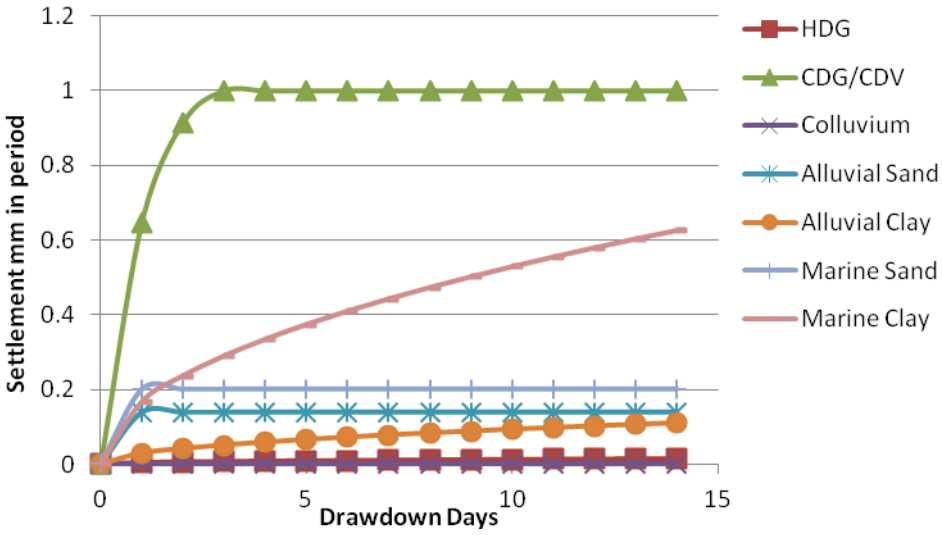


Figure 4: Calculated settlement produced after x drawdown days (for 1m drawdown) for common Hong Kong soils types based of back analysis

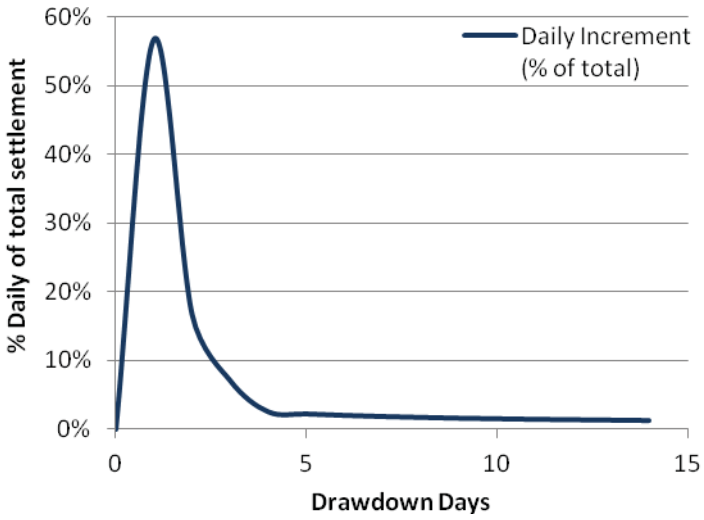


Figure 5: Cumulative daily settlements expressed as % of settlement over the initial period of drawdown
Note: Note the initial spike.

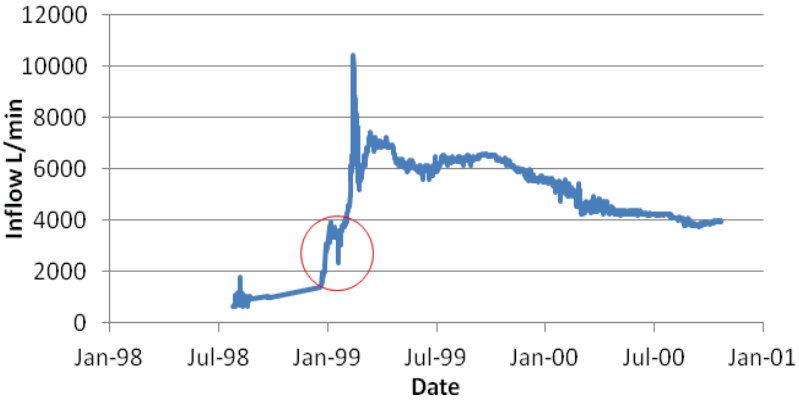
The values of M_v are based on back analysis of field observations combined with laboratory test data from Hong Kong. Values for compressibility of sands are based on observations of comparative density from SPT data. The values back analyses for CDG and CDV are relatively high and are in contrast to their higher stiffness. The observation of rebound on water level recovery suggests that the observed consolidation includes significant shrinkage. In the short term, CDV, CDG, confined loose sands particularly those with clay intercalations and re-deposited clays pose the most significant threat of short term settlement (Figure 5).

4 EXAMPLES FROM SSDS TUNNELS

The following example is taken from SSDS tunnel the details of which are published in TDD (2000). Tunnel C was excavated at -90 mPD between Tseung Kwan O and Kwun Tong across Junk Bay. Several instrumented reclamations were nearby as shown in Figure 6. In this example water inflow increased markedly from chainage 700 to 800 and was constant up to chainage 1200 (shown in red in Figure 6(a)).



(a) Location of SSDS Tunnel C
Note: Adjacent piezometers and areas of high inflow are indicated



(b) SSDS Tunnel C - tunnel infow

Figure 6: SSDS Tunnel C – Location plan and tunnel inflow

The graphs in Figure 7 show the change in piezometer head against radial distance to tunnel face which occurred as a result of mining in this section. In rock there is a clear drawdown in only one of the piezometers and this is the nearest to the tunnel. A steep drawdown curve can be seen in some piezometers but others show no response despite being relatively close. This is due to the control of rock structure and connectivity on transient water flow. In the decomposed rock (CDV) a wider drawdown curve can be identified with affects seen as far as 400 m from the tunnel face but still several piezometers are unaffected as would be expected with the decomposed rock inheriting many of the characteristics of the rock.

The alluvium by contrast shows no recognizable drawdown curve but affects are observed as far as 700 m from the tunnel source. Many piezometers are unaffected and there is no correlation with radial distance. Since the marine deposit also relies on the alluvium for its connection to the sink it is no surprise that the marine deposit shows the same relationship.

The results suggest that the use of simple layer cake homogenous hydrogeological models is not valid for transient water inflow studies. Their use should be restricted for assessment of steady state water balance analyses only and not used for assessment of the likely lateral extent of water drawdown effects. For this more regional hydrogeological drainage models should be constructed.

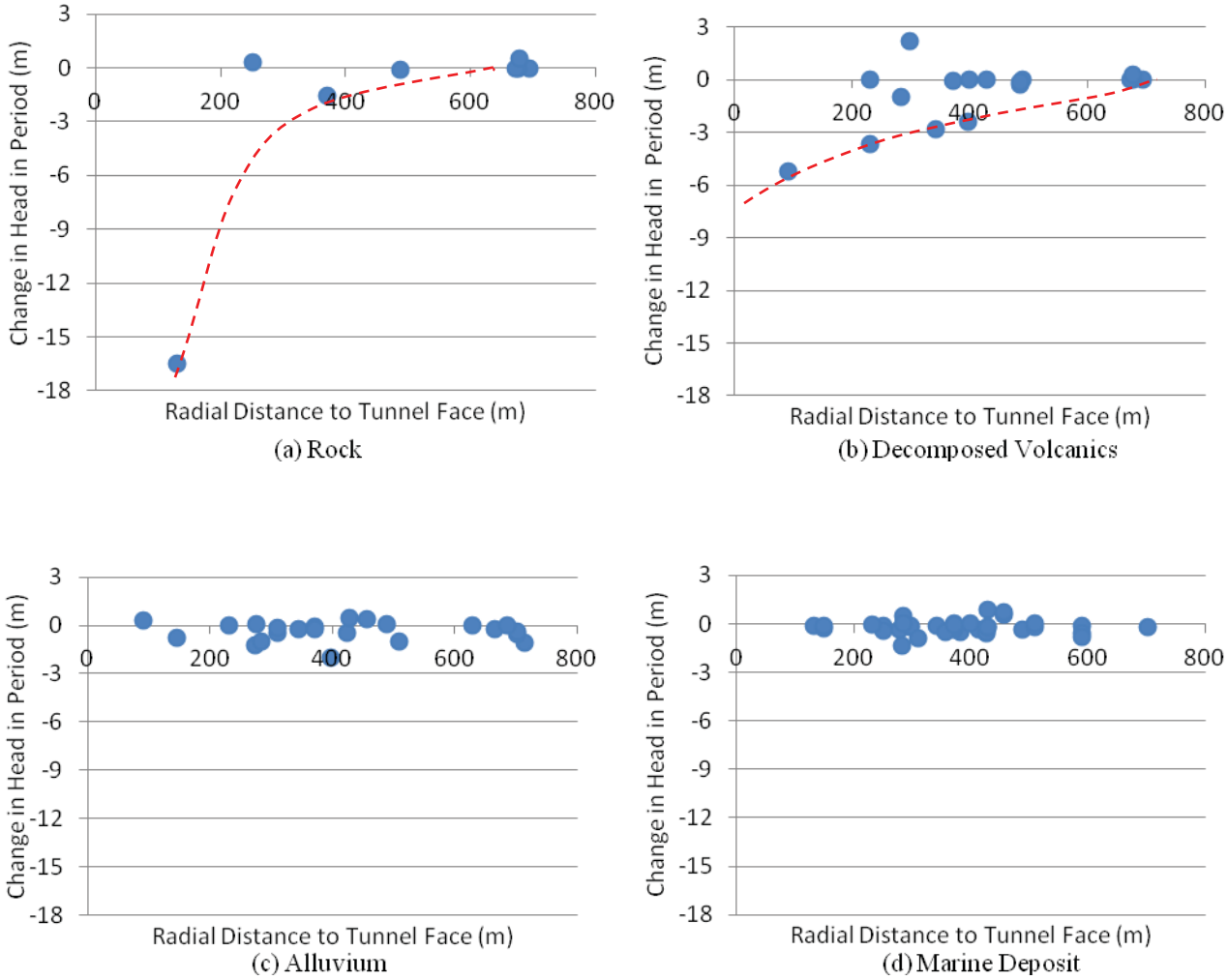


Figure 7: Change in head in piezometers at a radial distance from Tunnel C during 2000 L/min increase in total inflow (Chainage 700-800)

5 CONTROL MEASURES

Control measures implemented within deep ecognize projects are typically limited to pre-grouting at pre-determined intervals, based on simple models of rock permeability combined with remediation grouting when inflows continue and / or exceed the rate of allowable discharge. During shallow tunnel construction, for instance by cut and cover methods, groundwater is typically controlled by implementation of a cut off (diaphragm walls, secant pile walls, grout curtains etc) and a recharge system, which is often a series of recharge wells places at pre-determined spacings with little understanding of the complexity of the hydrogeological conditions.

The difficulties in creating a successful recharge system are well recorded, notably in CIRIA C515 (Preene et al, 2000). The primary consideration is whether compressible materials that are of concern can be recharged directly or whether recharge should be targeted at surrounding, more permeable facies which will prevent under-drainage in the first place. The concept that low permeability materials will not readily accept recharge is logical yet apparently overlooked in the majority of recharge system designs.

In addition to targeting recharge wells to intercept suitable facies, issues of groundwater chemistry and suspended solid content within the recharge water often leads to clogging and bio-fouling of the wells with the net result that a systematic programme of maintenance is essential and many contingency wells must be installed, in addition to the original design number, in order to maintain the design quantity of wells in

operation while maintenance is ongoing. Recharging too close to the sink may also result in excessive feedback, where additional grouting / pumping solutions are then required within the tunnel to deal with the extra inflows.

Recharge schemes fail due to a limited understanding of the hydrogeological complexities of the surrounding environment and the assumption that an holistic recharge of that environment is necessary and can be achieved. In addition to the recommendation that facies maps are adopted during the planning and design stages to model groundwater movement those receptors sensitive to groundwater drawdown in the surrounding environment should be ecognized and targeted for analysis. Where groundwater drawdown is controlled by fissure flow and palaeo-channels, and not simply primary permeability of overlying units, drawdown may not occur above or adjacent to tunnel alignments but also at some lateral distance from the tunnel. Having undertaken a facies mapping exercise and recognized the potential flow paths across the wider area surrounding the alignments, control of groundwater around specific sensitive receptors should then be considered in addition to any cut-off, grouting or recharge adopted at the tunnel site.

Sensitive receivers may vary from concern for surface habitats, including protection of river systems' base flow, particular ecosystems or crops, to excessive settlement, distortion, damage or collapse of slopes, utilities, infrastructure and buildings. In addition to consideration of annual mean rainfall, protection of surface habitats could include the use of shallow trench recharge systems or sprinkler systems. Targeted protection of slopes, utilities, infrastructure and buildings may rely on a combination of local groundwater cut-off and recharge systems around the receptor of concern, taking into account the underlying hydrogeological conditions, foundation type and particular risk of that receiver. The authors are aware of at least one current project in Hong Kong where an existing tunnel is being protected in this way while the construction of new tunnel proceeds beneath it.

6 CONCLUSION

In most tunnel projects where water is strictly controlled, it is the transient behavior of ground water which is of key concern. In this regard it is the higher permeability materials which are most important and in particular those which are compressible and have restricted recharge. These include CDG/CDV deposits in fault zones, sand pockets in alluvial systems and some re-deposited clays including those in seawalls and beneath dredged reclamations.

In the transient case detailed engineering geological and hydrogeological subsurface mapping is recommended ahead of numerical modeling. In complex hydrogeological settings modeling should be aimed at sensitivity testing and identifying the likely relative contributions of recharge and discharge in certain key areas.

Groundwater control measures must take into account the complexity of the model produced and target recharge at appropriately receptive facies in areas where key sensitive receptors have been identified.

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Design of Temporary Lining to Resist High Water Pressure Acting on a Drill-and-blast Tunnel

L. Tony Chen, Rupert K.Y. Leung & John W.Y. Yeung
Hyder Consulting Limited

ABSTRACT

This paper discusses the analysis and design of temporary lining to resist high water pressure acting on a drill-and-blast tunnel in Hong Kong. The interaction between the lining and ground is modeled using the two dimensional finite element program Phase2. The section forces from the analysis are used to check the adequacy of lining thickness and its reinforcement (if any). A great effort has been spent to explore ways to achieving cost and program savings. The effects of cross section geometry, water pressure, rock quality and temporary rock dowels on the lining design have been investigated numerically and are discussed in the paper. For a given set of loadings, it is found that the cross section geometry has the most significant influence on the lining design. To limit forces in the lining, cross section geometries shall be curvilinear, consisting of compound curves wherever possible.

1 INTRODUCTION

Control of groundwater inflows into tunnels excavated by the drill and blast method is critical in both minimizing ground movement and ensuring a dry tunneling environment. Grouting is normally employed to control groundwater inflows and has been proved to be effective in most cases. However, for some difficult ground conditions the use of grouting alone may be inadequate to limit water ingress for a relatively lengthy construction period and thus the installation of a temporary lining may become necessary.

This paper discusses the analysis and design of this type of temporary linings for a drill and blast tunnel project in Hong Kong. The tunnel is located at over 100 m below the ground surface and is subject to significant hydrostatic pressure. Where the tunnel alignment passes through severe fault zones and beneath sensitive structures, a temporary shotcrete lining, in addition to pre-excavation grouting, is to be installed to limit water inflows prior to casting of the permanent lining. Although not required to take the rock load, the temporary lining does rely on the ground for its support due to both non symmetrical cross section geometry and loading.

2 GROUND CONDITIONS AND GROUNDWATER TABLE

2.1 Ground conditions

The geological profile along the tunnel alignment consists of fill, marine deposits, alluvium, CDG and rock. The rock is described as strong to very strong, pinkish grey spotted black, slightly decomposed, medium to coarse grained GRANITE. Major fault zones are anticipated at some locations. Figure 1 shows a typical geological profile of the site.

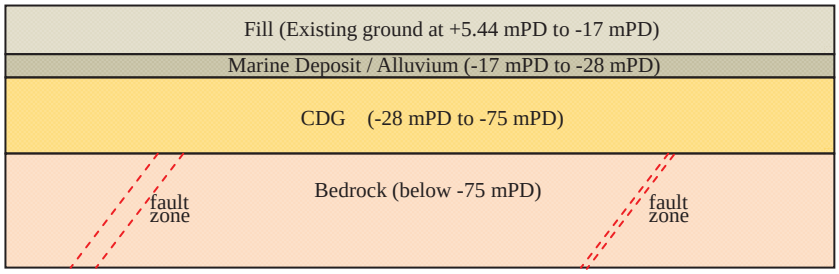


Figure 1: Typical geological profile

2.2 Groundwater table

According to the piezometer records for the project, the highest and lowest recorded ground water levels are approximately 0.5m and 2m, respectively, below the existing ground surface. The design groundwater level is taken as the highest existing ground level plus 1.0 m. For the temporary lining design, full hydrostatic pressure is adopted.

3 METHOD OF ANALYSIS

The span, height and shape of the tunnel vary along the tunnel alignment due to different functional and operational requirements. A typical tunnel profile with a temporary lining is shown in Figure 2. The lining may be constructed of plain concrete or reinforced concrete depending on the structural design.

3.1 Numerical model

The soil-structural interaction of the temporary lining is analyzed using the two dimensional finite element program Phase2 Version 6. The lining is represented by a series of beam elements connected to form the lining shape. The results from the analysis are used for the structural design of the lining. The adopted 2D finite element model under hydrostatic pressure is shown in Figure 3.

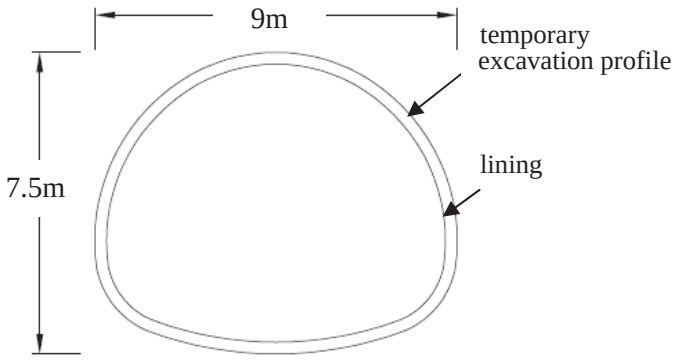


Figure 2: Typical tunnel profile with temporary lining

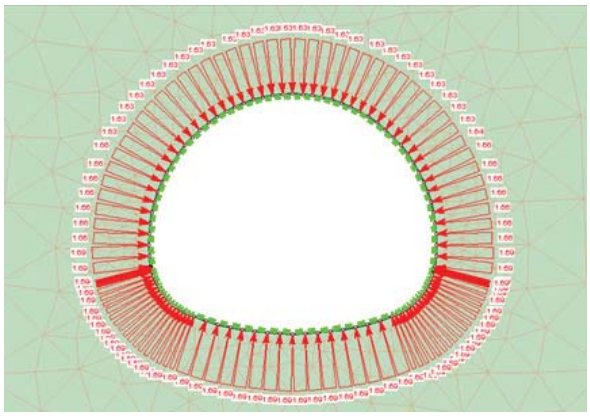


Figure 3: Adopted numerical model

3.2 Geotechnical parameters

The quality of the rock mass is classified according to the Q system, while the failure of the rock mass is modeled following the Hoek-Brown failure criterion, as described briefly below. Table 1 presents the adopted geotechnical parameters for two adopted Q values.

Table 1: Adopted geotechnical parameters

	Q	RMR	E_{rock} (GPa)	n	g (kN / m^3)	GSI	m_i	m_b	s	a
1	1	44	7.08	0.23	27	39	32	3.62	0.00114	0.5
2	0.025	11	1.05	0.23	27	6	32	1.11	0	0.621

Young's modulus of rock (E_{rock}) is determined by the following equations:

For RMR > 55, according to Bieniawski (1978)

$$E_{rock} = 2\,RMR - 100 \quad (1a)$$

For 10 < RMR < 55, according to Serafim & Pereira (1983)

$$E_{rock} = 10^{(RMR-10)/40} \quad (1b)$$

where RMR = 9 ln Q + 44 is the Rock Mass Rating according to Bieniawski (1976).

The Hoek-Brown (1997) parameters are determined by the following equations:

$$m_b = m_i e^{(GSI-100)/28} \quad (2)$$

where GSI = RMR - 5 is the geological strength index and $m_i = 32$ is the intact rock constant for granite.

For GSI ≥ 25

$$s = e^{(GSI-100)/9} \text{ and } a = 0.5 \quad (3a)$$

For GSI < 25

$$s = 0 \text{ and } a = 0.65 – GSI/200 \quad (3b)$$

3.3 Structural design

The structural design follows Eurocode 2 Part 1-1 for plain concrete members and BS8110 for reinforced concrete members. For the case of plain concrete, the eccentricity of the loading is limited to 0.3h, where h is the structural depth of the section considered.

The adopted shotcrete parameters are presented in Table 2.

Table 2 Adopted shotcrete parameters

Strength (MPa)	Yong's modulus (GPa)	Possion's ratio
50	27.7	0.20

3.4 Cases analyzed

In order to achieve economical design, five cases have been analyzed to investigate the effects of cross section geometry, water pressure, rock quality and temporary rock dowel on the lining design, as summarized in Table 3.

Table 3 Cases analyzed

Case	Cross section geometry	Water load (kPa)	Q	Rock dowel (m)	Remark
1	See Figure 2	1630	1	-	typical analysis
2	See Figure 4	1630	1	-	for study of geometry effect
3	See Figure 2	1630	0.025	-	for study of Q value effect
4	See Figure 2	815	1	-	for study of water load effect
5	See Figure 2 and 5	1630	1	2	for study of rock dowel effect

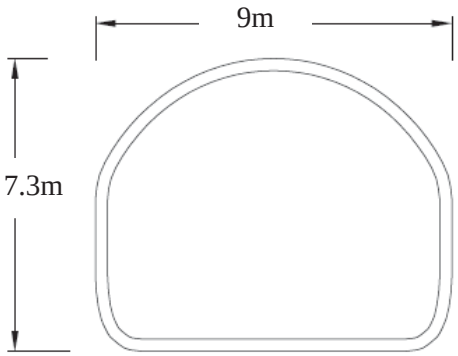


Figure 4: Tunnel geometry with sharp corners

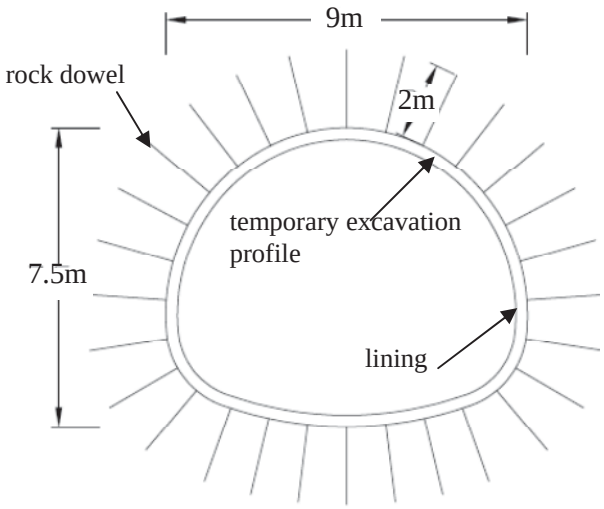


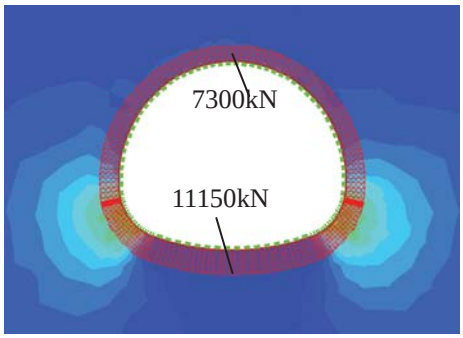
Figure 5: Arrangement of temporary rock dowels

4 ANALYSIS RESULTS

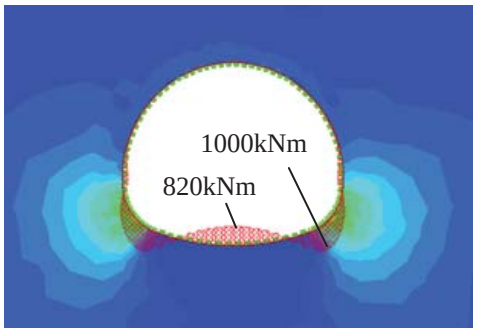
The major outputs from the Phase2 analyses include distributions of axial force, bending moment and shear force in the lining. These structural forces can be used to check the adequacy of the structural design.

4.1 Typical results

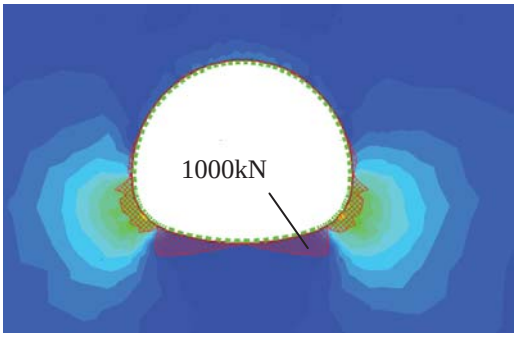
The calculated distributions of axial force, bending moment and shear force in the lining for Case 1 are presented in Figure 6 as typical analysis results.



(a) Axial force profile



(b) Bending moment profile



(c) Shear force profile

Figure 6: Calculated force distributions for Case 1

As shown in Figure 6(a), the axial force along the whole lining is in compression with its maximum value occurring at the base. Figure 6(b) shows that the bending moment is relatively small at both the crown and the side walls, but is quite significant at the base due to the relatively large span and non-circular shape of the

base, with hogging moment occurring at the centre and sagging moment occurring at the corners. Figure 6(c) shows that the shear force is also relatively small at both the crown and the side walls, but is quite significant at the base and the corners.

For this case, it is found that the lining can be designed as plain concrete at both the crown and walls with a thickness of 300 mm, and as reinforced concrete at the base with a thickness of 500 mm and 1.5% steel reinforcement.

4.2 Results of sensitivity studies

The calculated maximum forces for the base lining for different cases are summarized in Table 4 and also shown graphically in Figures 7 and 8 for comparison. The following observations can be made:

(a) As compared with Case 1 which has a relatively smooth geometry, for Case 2 the bending moment and shear force are much greater at both the base and the corners. Clearly the cross section geometry has significant effects on the calculated structural forces, hence the lining thickness. The structural design indicates that the lining for this case would need to be reinforced concrete with a much greater thickness and larger amount of steel reinforcement. A more economical design would be to smooth the geometry by deepening the centre of the base. Although the base centre is deeper, the lining thickness, hence the overall excavation volume, can be reduced significantly.

(b) As compared with Case 1, for Case 3 the axial force is slightly smaller, while both the bending moment and shear force are larger, indicating that the change of Q value from 1 to 0.025 has some influence on the lining design. This is expected as a softer ground tends to increase the deformation in the lining, resulting in larger bending moment and shear force.

(c) As compared with Case 1, the calculated forces are significantly reduced for Case 4 where the water pressure is only half of that for Case 1. This is expected as the water pressure is the main loading acting on the lining.

(d) Since rock dowels are usually installed as a temporary measure to stabilize the rock mass, it is of interest to investigate if the dowels can help to optimize the lining design. The results are encouraging and indicate that by modeling the dowels as support, the calculated forces become smaller resulting in smaller lining thickness.

Table 4: Calculated forces of different cases

Case	Moment (kNm)	Axial Force (kN)	Shear Force (kN)
1	1000	11150	1000
2	3360	8800	3550
3	1520	10500	1465
4	380	5920	395
5	520	10110	590

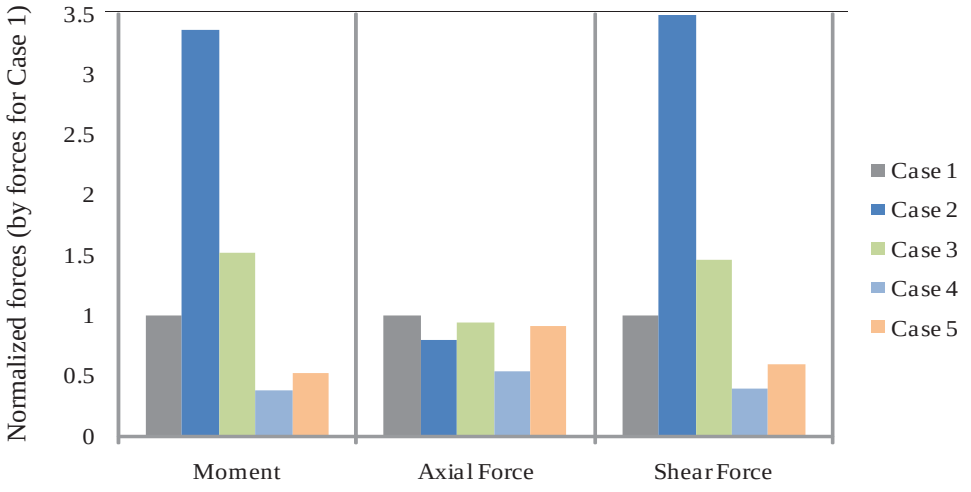


Figure 7: Comparison of calculated forces of base lining

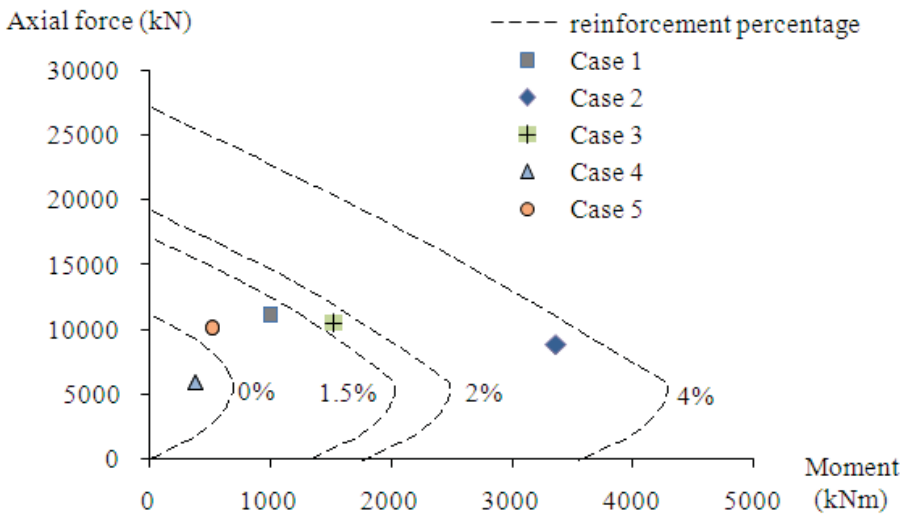


Figure 8: Axial force and moment interaction diagram for base lining design

5 CONCLUSIONS

This paper discusses the analysis and design of temporary lining to resist high groundwater pressure acting on a drill-and-blast tunnel in Hong Kong. The interaction between the lining and ground is modeled using the computer software Phase2. The section forces and stresses from the analyses are used to check the adequacy of lining thickness and its reinforcement (if any). An investigation into the effects of various factors, including cross section geometry, water pressure, rock quality and temporary rock dowel, on the lining design has been carried out using the two dimensional finite element program Phase2. The investigation results indicate that, for a given set of loadings, the cross section geometry has the most significant influence on the lining design. To limit forces in the lining, cross section geometries shall be curvilinear, consisting of compound curves wherever possible. Also, where temporary rock dowels are used to stabilize the rock mass, the lining design can be optimized by modeling the dowels as support to the lining.

ACKNOWLEDGEMENT

The authors would like to thank their colleagues Mr TS Leung and Mr Nick Gibbs for carrying out some of the numerical analyses discussed in the paper.

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Effect of End Wall on the Deflection of Diaphragm Wall

L.W. Wong
AECOM Asia Co. Ltd

ABSTRACT

Based on the wall deflection profiles of 8 inclinometers in the diaphragm wall located in the vicinity of the end walls, the effect of reduction in wall deflections near the end wall is studied. Empirical relationship between the wall deflections and the distances to the end walls has been established.

1 INTRODUCTION

The lateral deflections of the diaphragm wall supporting deep excavation in urban areas have been the prime concern of the profession as excessive ground movements would affect the adjacent structures. It has been observed that the ground settlements around the end walls would be less than those occurring at the central sections. Wong & Patron (1993) reported the 3-dimensional effect of the ground settlements adjacent to the excavation supported with the diaphragm wall. The reduction in wall deflections around the end walls would be favourable for ground movement control. This paper presents the observed wall deflection profiles of the diaphragm wall located in the vicinity of the end walls. A relationship between the maximum lateral wall deflections and the distances to the end walls has been established.

2 GEOLOGICAL PROFILES

A 0.9 km section of a dual three lane trunk road tunnel was built along the northern shore of the Hong Kong Island. The ground on which this tunnel section was built was an 18 hectare reclaimed land, of which the geological conditions have been reported by Wong (2012). Reclamation for this site of interest was conducted in 2 phases, i.e., the initial phase at the middle and the final phase at the two ends. The sloping seawalls mainly composed of rock fill of 400 mm maximum in size were constructed at the boundaries between the initial and the final reclamation areas. In the initial reclamation area, sand fill with the mean grain size (d_{50}) of 0.9 mm was placed. In the final reclamation areas, land-based granular fill of 200 mm maximum in size were used. The fill layer has the thickness of about 20 m. The mean sea level is at 1.2 mPD (metres above Principal Datum).

In the descending order, below the fill stratum are the alluvial deposits of the thicknesses ranging from 1.5 m to 2 m, the saprolite soils of completely decomposed granite (CDG) to highly decomposed granite (HDG) and the bedrock of slightly decomposed granite (SDG).

3 CUT-AND-COVER CONSTRUCTION

3.1 Tunnel box structure

The trunk road tunnel of this case of interest has the lowest road level of -12 mPD. The bottom-up cut-and-cover method was adopted for construction. As depicted in Figure 1, the diaphragm wall panels and the central barrettes of 1.2 m in thickness formed the integral part of the tunnel box structure, which is typically 32 m in width and 11 m in height. The wall panels and barrettes are embedded at least 300 mm into SDG. Table 1 summarizes the rockhead levels of the wall panels installed at the locations of the inclinometer casings.

As depicted in Figure 2, the excavation was typically conducted in Stage 1 to Stage 4. The struts were removed in Stage 5 and Stage 6. The depths of excavation for Stage 1 to Stage 4 were typically 7.3 m, 13 m, 16 m and 19 m. The lower 2 levels of struts were removed in Stage 5 after casting of the base slab. The uppermost struts were dismantled in Stage 6 after casting of the roof slab.

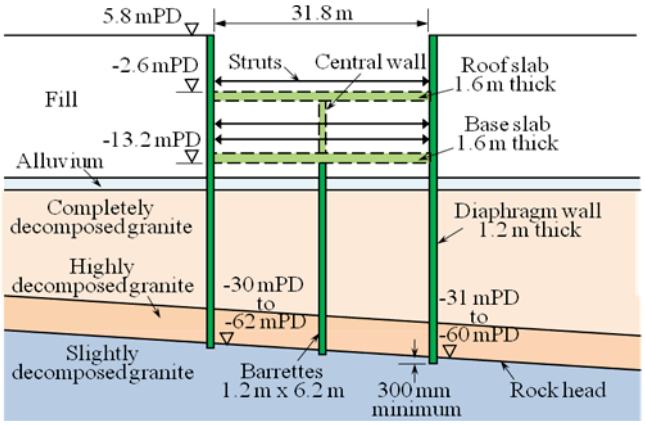


Figure 1: Typical section for the tunnel box

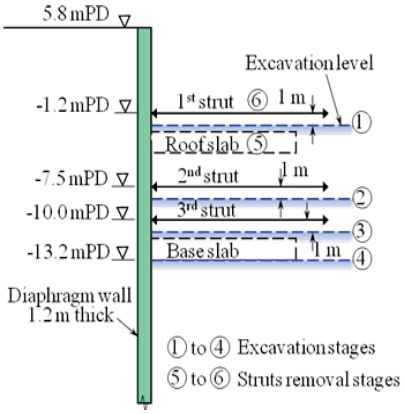


Figure 2: Construction stages

Table 1: Rockhead levels at various diaphragm wall panels

Inclinometer no.	Fill area	Distance to end wall (m)	Ground level (mPD)	Rockhead level (mPD)	
				North wall	South wall
N5, S5	Rock fill	6.3	6.26	-42.00	-38.78
S90	Sand fill	4.4	5.34	-49.33	-51.38
N96	Sand fill	6.9	5.21	-52.06	-40.83
N102	Sand fill	4.1	5.77	-48.95	-42.35
N123	Sand fill	9.7	6.07	-30.79	-33.61
S128	Rock fill	9.9	5.64	-35.17	-41.86
N152	Gravel fill	12.3	5.50	-54.63	-50.44

3.2 Stiffness of struts

The cut-and-cover excavation was supported with 3 levels of struts, of which the horizontal spacing varied from 5.2 m to 8.2 m. Each strut consisted of a pair of H-beams. Table 2 presents the typical normalized stiffness values of the struts. The stiffness is defined by $2AE/SBH$ where A is the sectional area, E is the Young's modulus of steel, S is the spacing of the struts, B is the length of the struts or the width of excavation and H is the maximum depth of excavation.

Table 2: The stiffness values for the struts

Inclinometer number	Total sectional area of struts A (cm^2)	Length of strut B (m)	Spacing S (m)	Depth of excavation H (m)	Strut stiffness $2AE/SBH$ (MN/m^3)
N5, S5	1634	31.8	6.5	16.3	19.9
S90	1634	31.8	8.2	18.9	13.6
N96	2639	35.2	5.4	18.8	30.3
N102	1634	36.6	5.2	19.0	18.5
N123	2083	31.8	5.3	19.0	26.7
S128	2811	31.8	7.3	18.4	27.0
N152	1725	34.6	4.8	18.3	23.3

3.3 Instrumentation

The instruments installed in the vicinity of the tunnel box mainly comprised piezometers, inclinometer casings in ground and in the diaphragm the wall panels and vibrating-wire strain gauges on the struts. Figure 3 shows the locations of the inclinometers.

Fourteen numbers of inclinometer casings were installed in the diaphragm wall panels and 6 numbers were installed 2 m to 5 m behind the wall in ground. The toes of the inclinometer casings were typically installed to 5 m below the wall toe levels. Among the 14 numbers of inclinometers in wall, 8 of them were located within 13 m to the end walls or to the bulkheads and only 6 of them were located in the typical central sections. It is noted that these temporary bulkheads are demolished after the base slabs of the adjacent sections are cast.

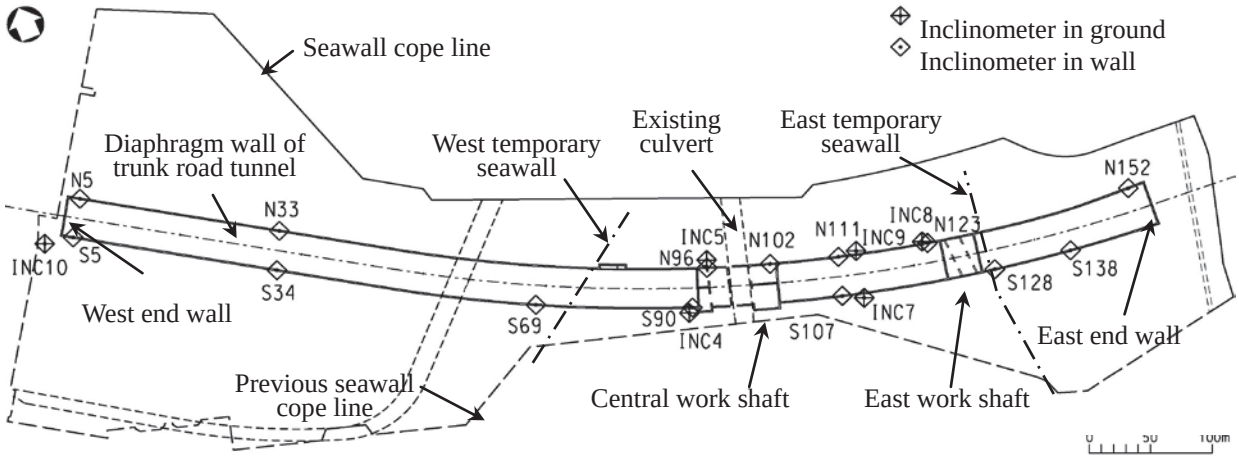
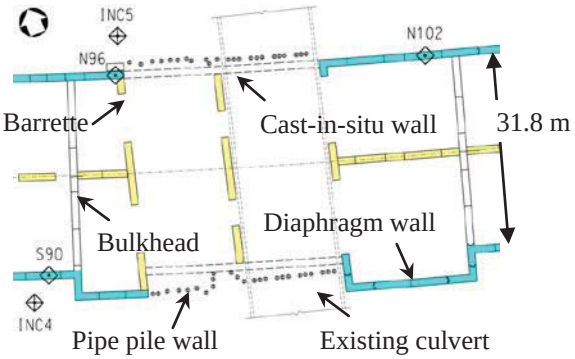


Figure 3: Location of inclinometers

3.4 Central work shaft

There are existing utilities crossing 2 sections of the tunnel alignment. Near the central portion the utilities comprised a stormwater culvert of 15 m in width and 7 pressurized water mains of the diameter ranging from 300 mm to 1200 mm. At the eastern portion, there are 6 water mains. While these utilities would obstruct diaphragm walling, the pipe pile walls were adopted for the temporary earth retaining system. Figure 4 depicts the layout of the earth retaining structures at the central work shaft.

Inclinometer no. N96 was installed in a diaphragm wall panel immediately next to a row of intermittent pipe pile wall and to a barrette of 1.2 m x 2.8 m. Installed through the gaps between the water mains and through the base and the roof slabs of the culvert, the pipe piles are tubular steel pipes of 610 mm in diameter in-filled with steel H-piles of 305 mm x 305 mm x 283 kg/m. Jet grout piles of 2 m in diameter and cement-bentonite grouting were conducted behind the pile walls to achieve water-tightness. The barrette was socketted 300 mm into the rockhead and was cast up to the base slab level. Excavation for the pipe pile wall on the west side of the culvert was supported with 5 levels of struts.



Notes: The existing water mains, jet grout piles and cement-bentonite grouting are not shown for sake of clarity.

Figure 4: Layout of the earth retaining structures at the central work shaft

4 DEFLECTIONS OF DIAPHRAGM WALL

The wall deflection profiles observed from the 8 inclinometers near the end walls are presented in Figures 5 to 7. As the wall toes are socketted 300 mm minimum into SDG and the toes of the inclinometer casings are extended 5 m below the wall toes, the toes of the inclinometer casings could be considered as the fixed points for calculating the wall deflection profiles.

As shown in Figures 5 to 7, the wall deflections gradually develop as excavation proceeds in Stage 1 to Stage 4. After casting of the base slab the wall deflections virtually cease in Stage 5 and Stage 6. The maximum wall deflections in various construction stages are summarized in Table 3. In Stage 4 to Stage 6, the maximum wall deflections observed from the inclinometers range from 20.3 mm to 57.0 mm.

Figure 6 shows that the maximum deflection for the hybrid earth retaining wall observed from inclinometer no. N96 after installing the 5th level strut in Stage 5 was 34.4 mm, which is comparable to the wall deflections ranging from 22.7 mm to 46.0 mm observed from the other 3 inclinometers in the sand fill area.

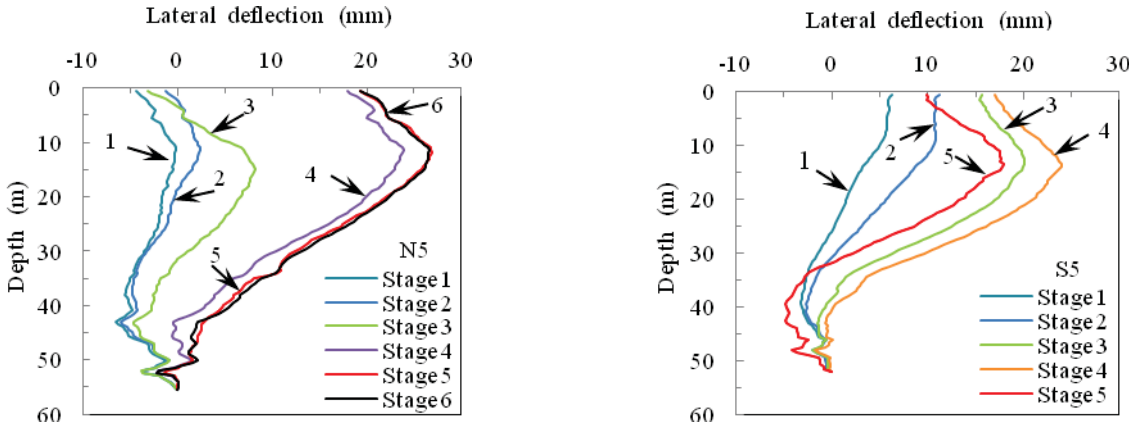


Figure 5: Wall deflection profiles next to the west end wall in the rock fill area

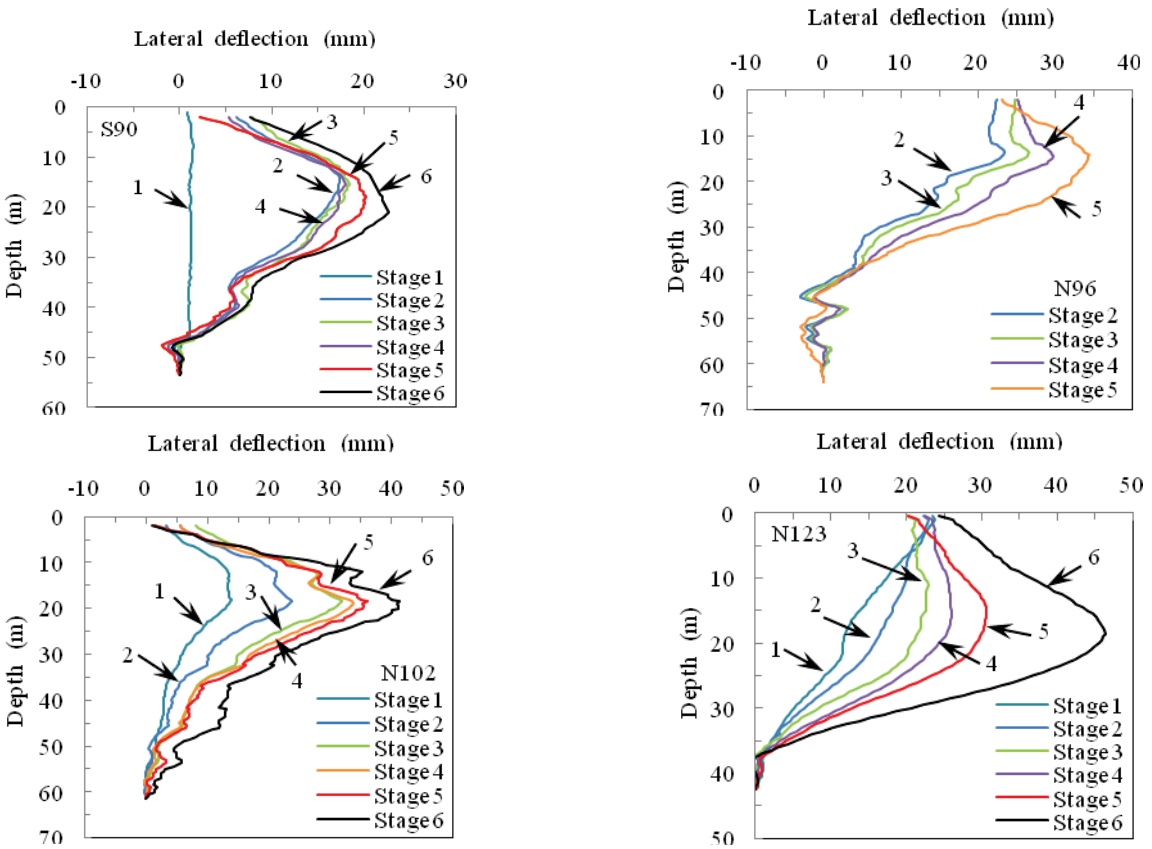


Figure 6: Wall deflection profiles in the sand fill area

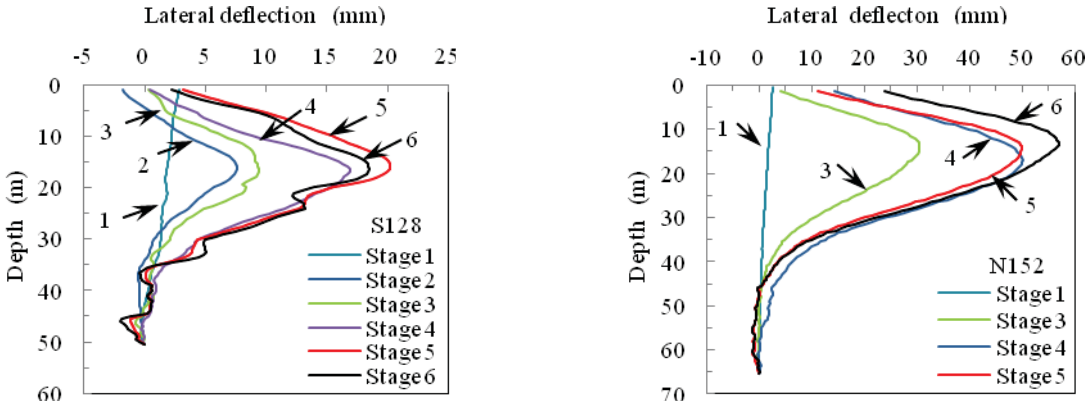


Figure 7: Wall deflection profiles near the east work shaft and the east end wall

Table 3: Maximum wall deflections observed from inclinometers in various stages

Stage	Depth of excavation (m)						Maximum deflection (mm)						
	N5, S5	N152	S90	N102	N123	S128	N5	S5	N152	S90	N102	N123	S128
1	8.3	4.9	6.4	7.4	7.7	6.9	1.2	6.3	-	-	13.9	23.9	2.9
2	10.5	12.9	13.9	14.3	13.1	13.5	2.4	11.2	-	17.4	23.7	22.8	7.6
3	13.5	15.9	16.4	17.3	17.1	16.5	8.3	20.1	30.5	18.3	31.8	22.9	9.4
4	16.3	18.3	18.9	19.0	19.0	18.4	23.9	24.0	50.1	18.0	33.7	26.0	17.0
5	16.3	18.3	18.9	19.0	19.0	18.4	26.8	20.7	50.0	20.2	36.0	30.6	20.3
6	16.3	18.3	18.9	19.0	19.0	18.4	26.6	-	57.0	22.7	41.3	46.3	18.5

5 EFFECT OF END WALL

The maximum wall deflections of those 8 inclinometers located in the vicinity of the end walls are compared with those occurred in the central sections. Wong (2012) reported the maximum wall deflections occurred in 6 inclinometers located at 50 m or larger to the end walls. Excluding the data for inclinometer no. S107 located next to an underpass, the maximum wall deflections observed from the 13 inclinometers in the strut removal stages are summarized in Table 4.

Table 4: Maximum wall deflections occurred in various fill areas in struts removal stages

Fill area	End wall sections			Central sections			
	Distance to end wall (m)	Depth of excavation (m)	Maximum deflection (mm)	Distance to end wall (m)	Depth of excavation (m)	Maximum deflection (mm)	
						Range	Average
Sand fill	4.1 ~ 9.7	18.9 ~ 19.0	22.7 ~ 46.3	50	18.3	66.0	66.0
Gravel fill	12.3	18.3	57.0	70 ~ 175	18.0 ~ 18.8	60.4 ~ 73.7	67.1
Rock fill	6.3 ~ 9.9	16.3 ~ 18.4	20.3 ~ 26.8	120	18.4	43.7	43.7

Figure 8 presents the distribution of maximum wall deflections of the 13 inclinometers against the distances to the end walls. It is noted that the distances to the end walls, x, are normalized with the width of excavation, B. There is the trend for the wall deflections to approach zero at the end wall. This effect of reduction in wall deflection gradually diminishes at farther distances to the end walls. The variation of wall deflections against the distances to the end walls could be expressed by the empirical Eq. (1) with the hyperbolic function of δ_c and b:

$$\delta_x = \delta_c / (1 + b\, B/x) \quad (1)$$

where δ_x is the maximum wall deflection occurs at the distance x to the end wall, δ_c is the maximum wall deflection occurs beyond the end wall influence and b is an empirical coefficient. Adopting the δ_c value of 67

mm for the walls in the sand fill and in the gravel fill areas, the *b* value of 0.1 could be determined by curve fitting. Similarly, applying the δ_c value of 44 mm for the walls in the rock fill area, the *b* values of 0.1 is obtained.

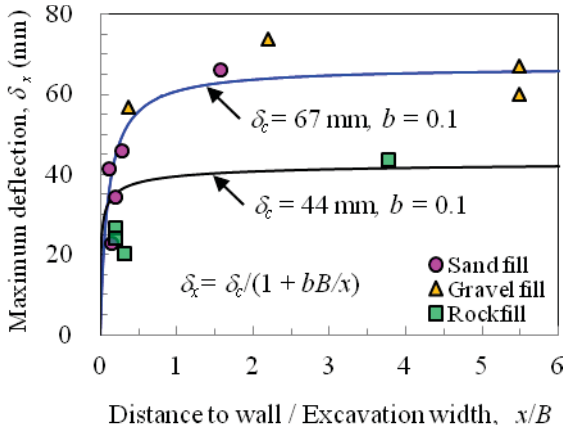


Figure 8: Variation of maximum wall deflections

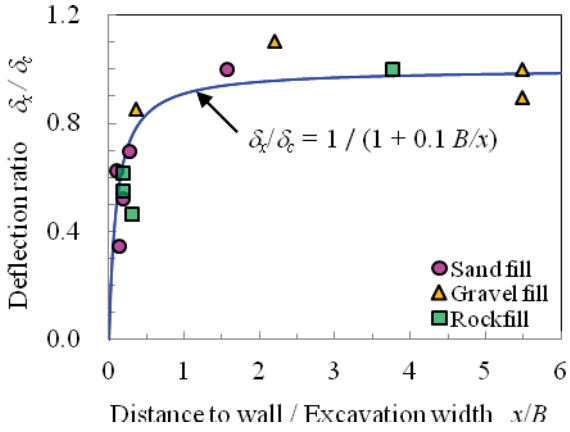


Figure 9: Distribution of normalized wall deflections

Normalizing the δ_x values with the respective δ_c values for the sand fill, the gravel fill and the rock fill areas and inputting the *b* value of 0.1, Eq. 1 is re-arranged into Eq. 2.

$$\delta_x/\delta_c = 1 / (1 + 0.1\ B/x) \qquad (2)$$

The variation of the δ_x/δ_c ratios against the distances to the end walls are presented in Figure 9, showing that the normalized wall deflections could be represented by Eq. 2. According to this equation, at the distance of 0.9*B* to the end wall, the wall deflection is 90 % of those occur in the central sections. This equation is applicable for assessing the effect of reduction in wall deflections near the end walls. The δ_c value could however be estimated in advance by 2-dimensional analysis such as the PLAXIS and the FREW analysis.

6 CONCLUSIONS

Based on the wall deflection profiles observed along a cut-and-cover tunnel in a coastal reclamation area, the effect of the end wall on the lateral deflections of the diaphragm wall has been studied. The following concluding remarks could be drawn:

(1) The lateral deflections of the diaphragm walls located in the vicinity of the end walls are less than those occur in the central sections of the cut-and-cover tunnel. The influence of the end wall effect diminishes at a distance of 1 time the width of excavation from the end wall.

(2) The reduction in the wall deflections could be expressed by an empirical hyperbolic relationship between the wall deflections occur at the central tunnel sections and the distances to the end walls.

Although this study is based on the results of one case history on cut-and-cover construction, the established empirical equation is expected to be applicable to excavation projects constructed in similar geological conditions.

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Effect of Earth Pressure Imbalance on Diaphragm Wall Deflections

L.W. Wong
AECOM Asia Co. Ltd

ABSTRACT

Diaphragm wall was employed to support a cut-and-cover tunnel of 19 m maximum in depth. The observed wall deflections next to an existing underground structure were about a half of those occurred in the wall without the structure. The observed effect of earth pressure imbalance was compared with an analytical case reported in the literature.

1 INTRODUCTION

Cut-and-cover excavation supported by the diaphragm wall has been the construction method for underground works such as rail tunnels, metro stations, expressway tunnels and basements. The ground displacements behind the diaphragm walls have been the prime concern in urban area as excessive ground movements would affect the adjacent structures. On the other hand, the presence of existing underground structures may reduce the earth pressures acting on the adjacent underground works. This paper presents the case history on a trunk road tunnel constructed by the cut-and-cover method. The deflections of the diaphragm wall with and without adjacent underground structures were closely monitored. The deflection profiles for various instrumented sections are presented and the effect of earth pressure imbalance on the wall deflections is critically reviewed.

2 GEOLOGICAL PROFILES

A 0.9 km section of a dual three lane trunk road tunnel was built along the northern shore of the Hong Kong Island. Prior to construction of the trunk road tunnel, a vehicular underpass of about 75 m in length was completed. The locations of the trunk road tunnel and the underpass are shown in Figure 1. The clearance between the underpass and the trunk road tunnel vary from 1 m to 11 m.

The ground on which the trunk road tunnel was built was an 18 hectare reclamation area. The original seabed levels varied from -10 mPD to -14 mPD (metres above Principal Datum). Prior to filling, the clayey marine deposits of approximately 4 m in thickness were removed by dredging.

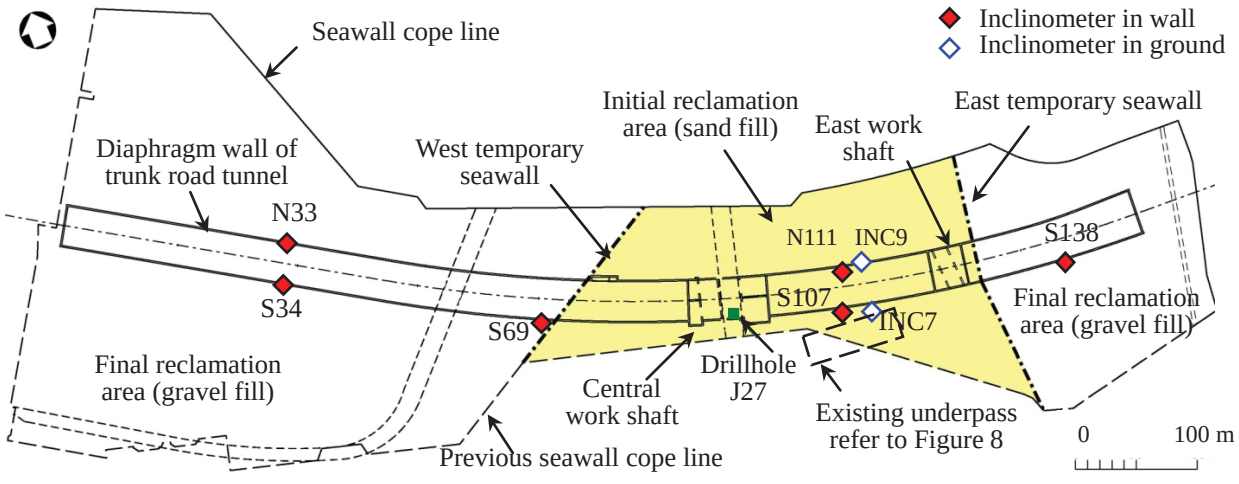


Figure 1: Extent of reclamation and the trunk road tunnel

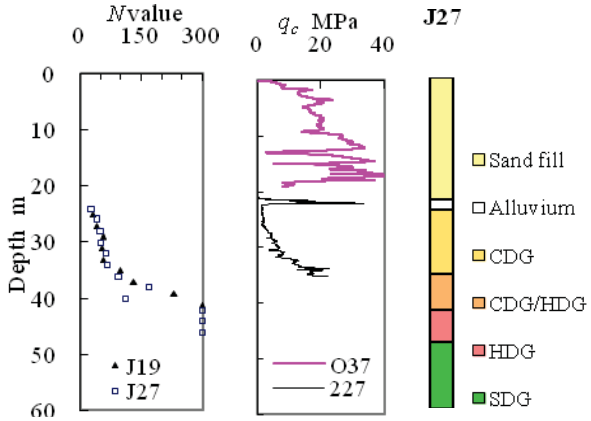


Figure 2: Typical subsoil condition

Reclamation for this site of interest was conducted in 2 phases, the initial phase at the middle and the final phase at both ends. The sloping seawalls mainly composed of rock fill of 400 mm maximum in size were constructed at the boundaries between the initial and the final reclamation areas. In the initial reclamation area, sand fill with the mean grain size (d_{50}) of 0.9 mm was placed from the dredged level to the elevation of 5 mPD. In the final reclamation areas, land-based granular fill of 200 mm maximum in size was used.

The typical subsoil profile in the reclamation area is depicted in Figure 2. In the descending order, below the fill stratum of about 20 m in thickness are the alluvial deposits of the thicknesses ranging from 1.5 m to 2 m, the saprolite soils of completely decomposed granite (CDG) to highly decomposed granite (HDG) and the bedrock of slightly decomposed granite (SDG). The ground level, for example, at drillhole no. J27 that located on the south side of reclamation area was 4.0 mPD. The mean sea level is at 1.2 mPD.

3 CUT-AND-COVER CONSTRUCTION

3.1 Tunnel box structure

The trunk road of this case of interest has the lowest road level of -12 mPD. The bottom-up cut-and-cover method was adopted for construction. As depicted in Figure 3, the diaphragm wall panels and the central barrettes of 1.2 m in thickness formed the integral part of the tunnel box structure, which is typically 32 m in width and 11 m in height. The wall panels and the barrettes are embedded at least 300 mm into SDG. Table 1 summarizes the rockhead levels of the wall panels at the locations of the inclinometers.

As depicted in Figure 4, the excavation was typically conducted in Stage 1 to Stage 4. The struts were removed in Stage 5 and Stage 6. The depths of excavation for Stage 1 to Stage 4 were typically 7.3 m, 13 m, 16 m and 19 m. The lower 2 levels of struts were removed in Stage 5 after casting of the base slab. The uppermost struts were dismantled in Stage 6 after casting of the roof slab.

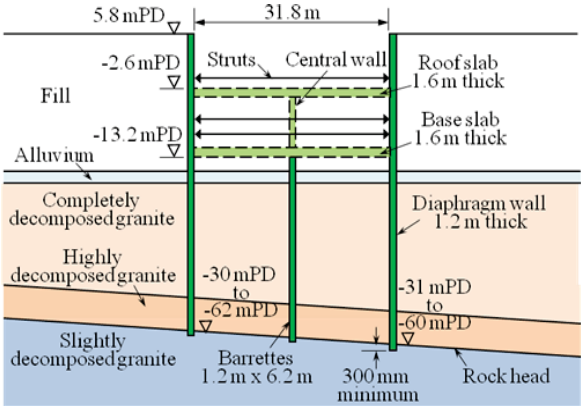


Figure 3: Typical section for the tunnel box

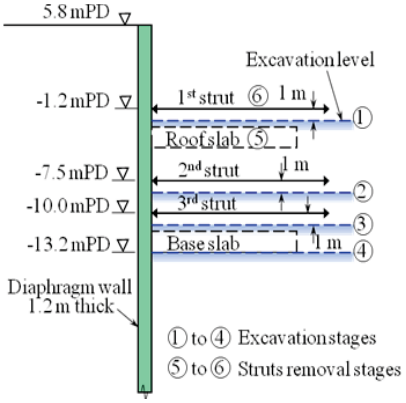


Figure 4: Construction stages

Table 1: Rockhead levels at various diaphragm wall panels

Inclinometer no.	Fill area	Ground level (mPD)	Rockhead level (mPD)	
			North wall	South wall
N33, S34	Gravel fill	5.89	-41.21	-43.68
S69	Rock fill	4.82	-48.79	-53.47
N111, S107	Sand fill	5.43, 4.81	-40.37	-41.79
S138	Gravel fill	5.15	-40.02	-39.52

3.2 Stiffness of struts

The cut-and-cover excavation was supported with 3 levels of steel struts, of which the spacing varied from 5.5 m to 8.3 m. Each strut consisted of a pair of H-beams. Table 2 presents the normalized stiffness values of the struts. The stiffness is defined by $2AE/SBH$ where A is the sectional area, E is the Young's modulus, S is the spacing, B is the width of excavation or the length of the struts and H is the maximum depth of excavation.

Table 2: Stiffness of struts for the typical central sections

Section with inclinometer	Strut level	Member	Sectional area A (cm^2)	Spacing of struts S (m)	Depth of excavation H (m)	Strut stiffness $2AE/SBH$ (MN/m^3)
N33, S34 & S69	1	2 x UB 610 x 305 x 238 kg/m	607.6	8.2	18.8	13.7
	2	2 x UB 914 x 305 x 289 kg/m	737.6			
	3	2 x UB 610 x 229 x 113 kg/m	289.0			
N111, S107	1	2 x UB 610 x 305 x 238 kg/m	607.6	5.6	18.3	26.2
	2	2 x UB 914 x 305 x 289 kg/m	737.6			
	3	2 x UB 914 x 305 x 289 kg/m	737.6			
S138	1	2 x UB 610 x 305 x 238 kg/m	607.6	8.3	18.0	14.9
	2	2 x UB 914 x 305 x 289 kg/m	737.6			
	3	2 x UB 610 x 305 x 149 kg/m	380.0			

3.3 Groundwater conditions

Instruments installed in the vicinity of the tunnel box mainly comprised piezometers, inclinometer casings in ground and in the diaphragm wall panels and vibrating-wire strain gauges on the struts. Figure 1 shows the locations of the inclinometers casings installed at 4 instrumented sections. The toes of the inclinometers casings were typically installed to 5 m below the wall toe levels.

Figure 5 shows the readings for the vibrating wire piezometer no. PZ05 located on the south side of the tunnel box near inclinometer no. S107. The upper and the lower sensors are installed in the bases of the fill and of the HDG strata respectively. The daily readings indicate that the groundwater levels are under tidal influence. Prior to excavation in October 2009, the piezometric levels in fill varied between -0.5 mPD and 1.8 mPD, with an average of around 1.0 mPD. The piezometric levels in HDG varied between 0.5 mPD and 1.9 mPD, with an average of around 1.4 mPD. When the maximum depth of excavation of 19 m was reached in September 2010, the average piezometric levels in the fill and in the HDG strata were lowered to about -0.5 mPD and 0 mPD respectively. This cut-and-cover tunnel was completed in October 2011.

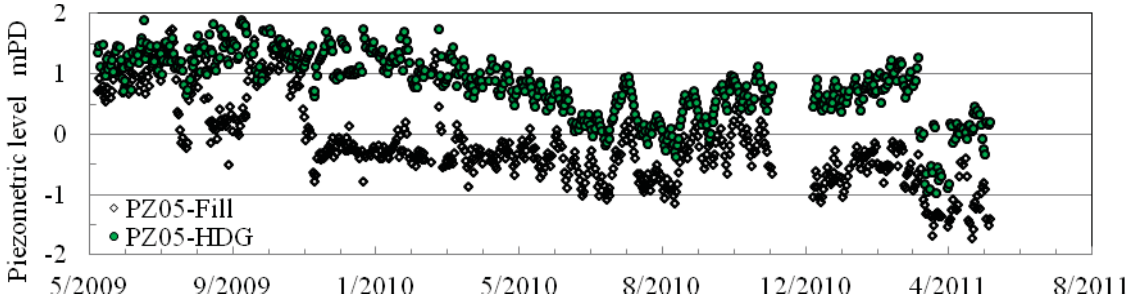


Figure 5: Variation of piezometric pressures near inclinometer no. S107

4 DEFLECTIONS OF DIAPHRAGM WALL

Figure 6 presents the deflection profiles for inclinometers no. N33, S34, S69 and S138. Table 3 summarizes the maximum deflections at various depths of excavation observed in these 4 inclinometers. As the wall toes are socketted 300 mm minimum into SDG and the toes of the inclinometer casings are extended 5 m below the wall toes, the toes of the inclinometer casings could be considered as the fixed points for calculating the wall deflection profiles.

Inclinometers no. N33, S34 and S138 are located in the gravel fill area. Prior to installation of the 1st level strut, the walls were cantilevers and the largest deflection at the top ranged from 32.9 mm to 34.8 mm, with an average of 33.8 mm. In Stage 4 to Stage 6 when the largest depths of excavation were reached, the maximum wall deflections ranged from 60.4 mm to 73.7 mm, with an average of 67.1 mm. It is noted that after the base slab and the roof slabs were cast in Stage 5 and Stage 6 respectively, the wall deflections continued to increase due to struts removal.

Inclinometer no. S69 is located at the temporary sloping seawall that mainly composed of rock fill. The maximum wall deflection of 43.7 mm in Stage 3 is slightly lower than those mentioned above. It is noted that an outward toe movement of about 10 mm was observed from this inclinometer at the depth of 61 m. This outward movement would be caused by struts removal in Stage 5.

Table 3: Wall deflections for typical central sections

Stage	Depth of excavation (m)			Maximum deflection (mm)			
	N33, S34	S69	S138	N33	S34	S69	S138
1	7.3	5.9	6.5	34.8	32.9	22.7	-
2	13.8	13.5	12.8	41.9	48.7	25.8	-
3	16.7	16.5	15.8	57.6	61.5	43.7	47.0
4	18.8	18.4	18.0	58.1	62.6	40.0	73.7
5	18.8	18.4	18.0	60.4	65.7	40.5	68.0
6	18.8	18.4	18.0	57.4	67.3	40.7	72.0

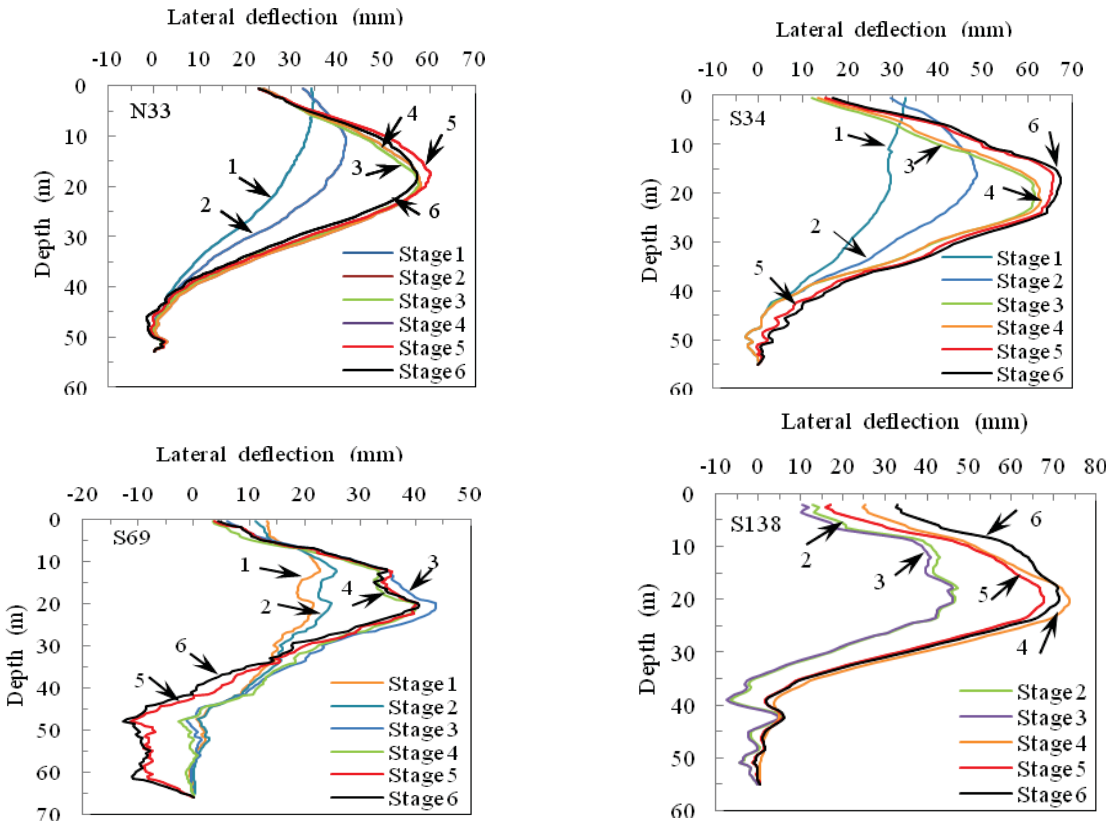


Figure 6: Wall deflection profiles for central sections – gravel fill and rock fill area

5 EFFECT OF EARTH PRESSURE IMBALANCE

Figure 7 presents the deflection profiles for inclinometers no. N111 and S107, which are located in the sand fill area. Table 4 summarizes the maximum deflections occurring in various construction stages. The reduction in wall deflections is observed on the southern wall. In the strut removal stages, the maximum wall deflections in inclinometers no. N111 and S107 were 66.0 mm and 31.8 mm respectively. The maximum deflection of 66.0 mm for inclinometer no. N111 is similar to the average deflection of 67.1 mm observed from inclinometers no. N33, S34 and S138. However, the maximum deflection of 31.8 mm for inclinometer no. S107 is only 48 % of that occurring on the north side of the excavation.

Similar reduction in ground movements is observed from the inclinometers in ground. Inclinometers no. INC7 and INC9 were located 16 m and 20 m east of inclinometers no. S107 and N111 respectively and 4 m behind the diaphragm wall panels. As summarized in Table 4, the maximum lateral ground deflections in the strut removal stages in the north and the south sides were 71.0 mm and 26.1 mm respectively.

The reduction in wall deflections could be attributed to the imbalance in earth pressures on the north and on the south sides of the excavation. As shown in Figure 8, the existing vehicular underpass of 25 m in width and 9 m in height is located immediately south of the trunk road tunnel. The clearance between the underpass and the tunnel box is 7 m near inclinometer no. S107. The underpass is supported on reinforced concrete bored piles of 1.5 m in diameter spacing at 6.8 m. An existing seawall founding on a rock fill mound is located further south to the underpass.

There is the trend of outward movement on the southern wall. Figure 9 presents the trend of deflections occurring at the first strut level of the wall panels. After the 1st level struts were installed, the deflections for inclinometer no. N111 increased from 23.9 mm in Stage 1 to 55.8 mm in Stage 6, with the net inward deflection of 31.9 mm. The deflections for inclinometer no. S107 decreased from 18.2 mm in Stage 1 to 6.8 mm in Stage 6, with the net outward deflection of 11.4 mm. The 1st level strut was shortened by 20.5 mm (the difference between 31.9 mm and 11.4 mm).

The trend of outward deflections of the southern wall is also observed at the lower strut levels. After installing the 2nd and the 3rd level struts, the net outward deflections observed from inclinometer no. S107 in Stage 6 were 9.6 mm and 6.8 mm at the 2nd and at the 3rd strut levels respectively.

The effect of earth pressure imbalance has been reported in the literature. Hwang et al. (2012) conducted numerical analysis on cut-and-cover construction using the PLAXIS software. In one of the cases studied, the wall deflections on both sides of an underground metro station of 20 m in depth and 40 m in width next to an existing underground car park of 10 m in depth were calculated. The car park was supported with mat foundation. The computed results show that when the metro station is excavated to the depth of 20 m, the wall deflections next to the car park and on the opposite side are 71.5 mm and 89.5 mm respectively. The wall deflection next to the car park is 0.8 times of that on the opposite side of the excavation.

In this case of interest, the maximum wall deflection observed from inclinometer no. S107 was 0.48 times of that from inclinometer no. N111. The effect of earth pressure imbalance for this case of interest is more prominent than the analytical case presented by Hwang et al. (2012). The larger reduction in wall deflections for this case of interest may be attributable to the presence of the underpass and the seawall. As a result, the earth pressure acting on the south side of the trunk road tunnel would be significantly less than that on the north side. A numerical analysis could be conducted on this case of interest to confirm the effect of the piles, the underpass and the existing seawall on the wall deflections.

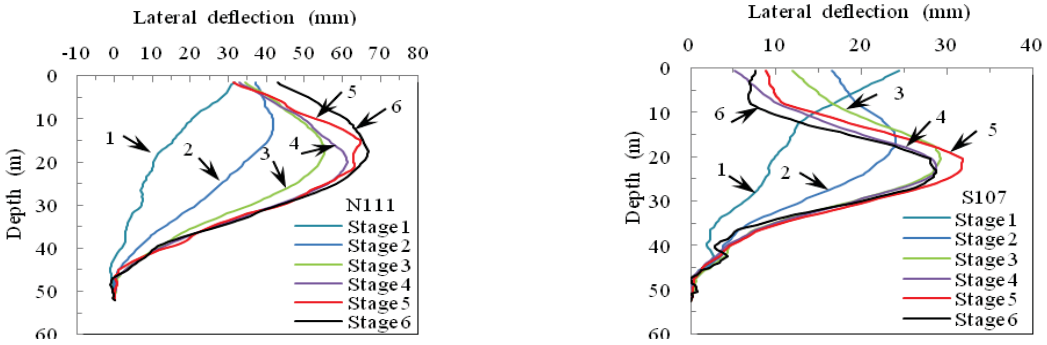


Figure 7: Wall deflection profiles for central sections - sand fill area

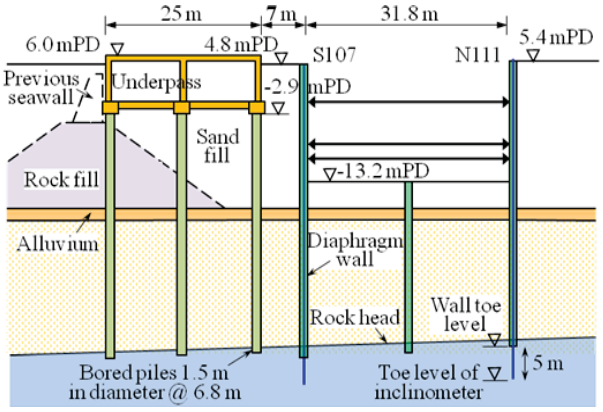


Figure 8: Excavation adjacent to an underground structure

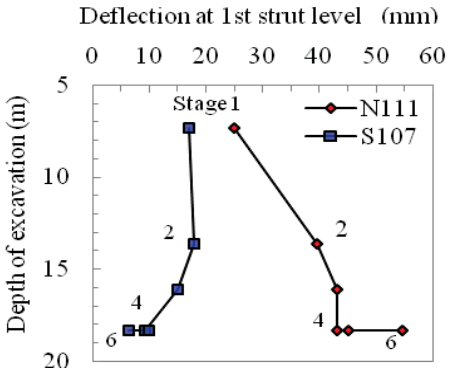


Figure 9: Trend of wall deflections at the first strut level for N111 and S107

Table 4: Wall and ground deflections near the underpass

Stage	Depth of excavation (m)	Maximum deflection (mm)			
		North side		South side	
		N111	INC9	S107	INC7
1	7.3	31.6	19.7	24.4	24.5
2	13.6	41.9	36.0	24.0	21.4
3	16.1	55.5	-	29.3	24.1
4	18.3	61.2	-	28.8	23.8
5	18.3	64.8	69.9	31.8	26.1
6	18.3	66.0	71.0	28.7	24.1

6 CONCLUSIONS

A cut-and-cover tunnel of 0.9 km has been constructed in a coastal reclamation area. Study of the inclinometer monitoring results would lead to the following conclusions:

(1) For the excavation of 19 m in depth supported with the diaphragm wall of 1.2 m in thickness, the maximum lateral wall deflections range from 60 mm to 74 mm, with an average of 67 mm in the sand fill and in the gravel fill areas. For the excavation in the rock fill area, the maximum wall deflection is about 44 mm.

(2) Due to the effect of earth pressure imbalance, the deflections of the walls in the vicinity of the underground structures would be less than those of the typical wall sections.

The effect of earth pressure imbalance observed in this case of interest deserves further studies. The effect of existing foundations and earth retaining structures on the performance of cut-and-cover construction shall also be considered.

REFERENCES

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Consideration for Sensitive Design of GINA Gaskets for Immersed Tunnel

Joseph Y.C. Lo, H. Sakaeda, C.K. Tsang & Y.M. Hu
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

Rubber gasket is the key component of the flexible joints of immersed tube tunnel and there have been no specific established design criteria for the gasket and discussed for each immersed tube case. It not only can ensure the water tightness during the connection of tunnel units but also can allow the tunnel to facilitate its movements due to differential settlement, creep and shrinkage of concrete, temperature effects and earthquake loads during the operation period. Nowadays the depth of water for immersed tube selected becomes deeper and deeper and requirement for the behavior during a bigger earthquake becomes more severe. Accordingly, required characteristics for rubber gasket used for key joint element for each tunnel unit has become more stringent. In the present paper, the properties of some commonly used Gina gaskets (one of rubber gasket types) are described first. The design methodology with sensitive consideration of Gina Gasket is then illustrated by using the case of the on-going project, Guangzhou Zhoutouzui immersed tube tunnel.

1 INTRODUCTION

Several techniques are available for the construction of river or sea crossing tunnels. Among them, immersed tube tunnel offers the following key advantages in comparison with TBM tunnel:

(1) No specific requirements for ground condition other than medium to hard rocks.
(2) Shallow soil cover making the approach tunnel length at both ends shorter with the limit of the gradient imposed by vertical alignment. Flexible to select the cross section shape to suit a desired purpose. For example, rectangular cross section, if selected, can have a better arrangement for traffic spatial requirements.
(3) Less geotechnical and construction risks as the tunnel is formed on the pre-excavated trench where the nature of the ground has already been exposed.
(4) Evacuation access to the safer adjacent duct for an emergency case can be easily made by providing doors between two ducts.

An immersed tube tunnel consists of numbers of prefabricated tunnel elements that are first fabricated in a fabrication yard. After the tunnel elements are constructed, they are floated and towed to the site, installed one by one, and connected to one another under water. An immersed tunnel is generally installed in the trench that has been dredged previously in the bottom of the sea. The space between the trench bottom and the invert of the tunnel is filled with sand foundation in later stage or the elements is set on the pre-formed sand or gravel mat foundation on the trench. As construction proceeds, the tunnel is backfilled. The completed tunnel is covered with a protective layer over the roof.

The immersed tunnel elements prefabricated in the fabrication yard or dry dock will be sealed at both ends by installation of bulkhead. In addition, rubber gasket will be mounted on the collar plate at one end of tunnel element. Rubber gasket is a very important element for preventing water ingress during the period of jointing the tunnel elements together. On the other hand, if the rubber gasket is used as part of permanent structure as flexible joint, the joint is able to allow the tunnel to facilitate its movements due to differential settlement, creep of concrete, temperature effects and earthquake loads during operation stage.

In this paper, the application of Gina gasket (one type of rubber gasket) in immersed tunnel will be described. Then the properties of some commonly used Gina gaskets and its corresponding terminologies will

be discussed. Finally the sensitive consideration for design methodology of Gina gasket will be illustrated by using the information of the Guangzhou Zhoutouzui immersed tube tunnel.

2 APPLICATION OF GINA GASKET TO GUANGZHOU ZHOUTOUZUI IMMERSED TUNNEL

After the Gina gasket and bulkheads are installed in the tunnel element, the tunnel element is ready for floating and finally towing to the immersion site. During the immersing operation, the new element is pulled closer to the previously sunken element. After its position is checked, it is then placed on the temporary supporting bracket on the other element. Finally the element is pulled nearer by the pulling jack until the nose of Gina gasket is just crashed to secure temporary water seal (i.e. primary jointing). Then the element is further pushed against the precedent by the unbalanced water pressure, which will be generated by drained out the water trapped in the chamber between two bulkheads facing each other. (i.e. secondary jointing). At this moment, the sunken element receives a hydrostatic pressure of about several thousand tonnes in the axial direction of tunnel alignment, so that waterproof is completely secured at the joint. The jointing operation of IMT elements is illustrated in Figure 1.

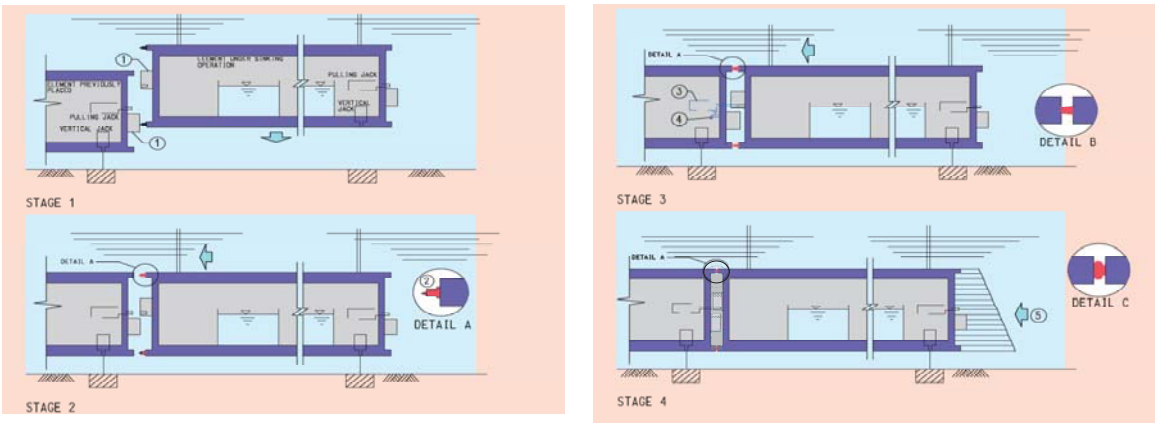
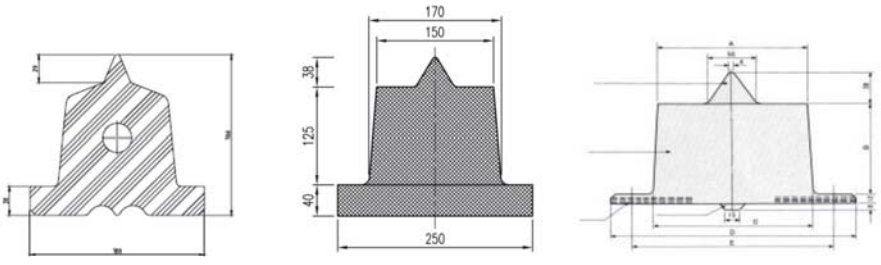


Figure 1: Application of Gina gasket for joining immersed tunnel

3 OVEREVIEW OF GINA GASKET

3.1 Type of Gina gasket

There are two well-known manufacturers of Gina gasket. They are Trelleborg of the Netherlands and Yokohama of Japan. Gina gaskets are usually made of rubber, either natural or artificial. A typical Gina seal generally includes a triangular nose, a trapezoid main body and bottom flanges on both sides for installation purpose. The commonly used Gina gaskets are shown in Figure 2.



Trelleborg Type ETS Trelleborg Type G Yokohama Type Y

Figure 2: Cross sections of Gina gasket

The Trelleborg Type ETS Gina gasket is made of artificial rubber and is normally blend of NR (natural rubber) and SBR (styrene-butadiene rubber). The inner hollow hole is designed to reduce the overall stiffness and hardness of gasket. It is said that although SBR is cheaper in cost, it is inferior to the natural rubber mainly in viscosity, impact resilience, tearing resistance and flex cracking resistance. Both Trelleborg Type G

and Yokohama Type Y are made of natural rubber. Instead of using a thick flange for clumping fixing in the Gina seal from Trelleborg, reinforcement layers are inserted in the flanges of Gina seal from Yokohama to resist the stress concentration by adopting bolt fixing to secure the accurate position of the gasket. The bottom cross section area of the Gina seal from Trelleborg is reduced in order to obtain a larger pressure between Gina gasket and steel frame and hence a better watertightness. Similarly, there is a specially designed water seal protuberance at the bottom of the Yakohama type.

3.2 Design consideration of Gina gasket

The design life of Gina seals should be incorporated with that of corresponding immersed tube tunnel which is usually over 100 years. In addition, the selected Gina gasket should satisfy the following conditions during its whole design life: 1) Should have sufficient resistance to transfer the hydrostatic loads at high water level within the maximum compression capacity of the Gina profile; 2) Sealing at all water levels even when there is less compressed strain due to variation in smoothness/flatness of the tunnel faces, rotation of immersed tunnel elements, creep and shrinkage of the concrete, thermal effects and seismic action; 3) The maximum repeated compression is still within the tolerance even during seismic motion. The stress and strain of steel plates and bolts in steel clamping system should be within specified limit without adverse stress effect on the gasket body; 4) To have enough tear strength; 5) To have enough elongation capacity; 6) Shall not be sudden combustible and not generate poisonous gas during fire; and 7) Less stress relaxation and permanent residual compressive set.

The material properties used in the design should include the effect of stress relaxation (creep) in long term, permanent residual deformation against behaviour especially during seismic motion which is verified by the laboratory tests under relevant standards. A considerable margin of life span should exit for materials.

The relative lateral and vertical displacements of two adjacent immersed tube units might introduce shear force to the Gina seal. But, in reality, the close tolerances of Gina shear displacement is employed in the design of both vertical and lateral shear key so that the joint would become rigid after a certain allowable displacement.

3.3 Load deformation curve

The initial compression is defined as the deformation of Gina gasket under static water pressure right after the immersion of the corresponding tunnel unit. As long as the tide level and unit position are confirmed, the load applied on the Gina gasket (F_i) can be determined.

$$F_i = \gamma_w h A / L \tag{1}$$

where γ_w is water density; h is the vertical distance between tide level and the centroid of tunnel section; A is the area of tunnel section; and L is the length of Gina gasket. The initial compression can then be found from a load deformation curve which is normally provided by the manufacturer. Figure 3 shows a typical load deformation curve of a Gina gasket.

Due to the existence of triangular nose, the rate of deformation increment is much larger when the total deformation is small with smaller compressive force. This is designed to facilitate the initial water-tightness by use of pulling jack for compression of Gina gasket. The rate decreases all the way with the increase of axial deformation with larger compressive force. The end of this curve shown in Figure 3 only shows the total deformation equivalent to the normal design load. Due to super-elasticity of the nature of rubber the maximum compression capacity of the Gina gasket is much larger while the total deformation does not increase much. However, in the design safety factor need to be applied so that Gina gasket is not allowed to deform to the final failure.

3.4 Minimum compression

In order to ensure water-tightness, a minimum axial deformation (in other word: contact stress) of Gina gasket should be achieved. The required minimum compression increases with the increase of outer pressure (Figure 4). Minimum compression deformation (stress) usually is not a critical concern at the construction stage but is crucial at the operation stage in long term. Due to earthquake loads, temperature variation, concrete shrinkage

and differential settlement of tunnel units, the gasket movement joints at each unit joint are likely to move away from each other. The minimum compressive deformation and its contact stress should always be achieved under any design cases.

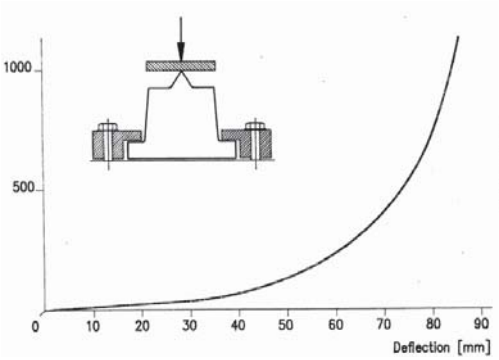


Figure 3: Load seformation curve of Gina gasket

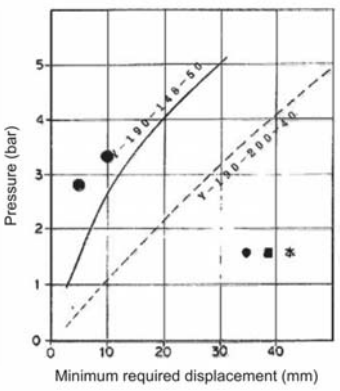


Figure 4: Water-tightness performance curve

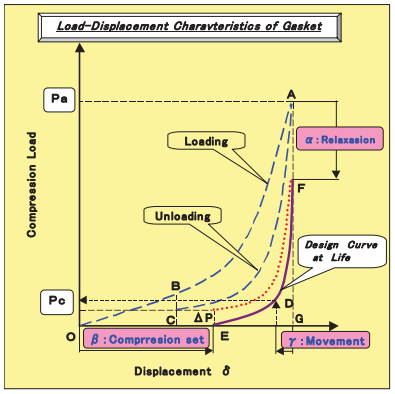


Figure 5: Revised load-deformation curve by including stress relaxation and cyclic load

3.5 Time-dependent behaviour

The curves in Figure 3 & 4 do not include any time effect. As discussed in design consideration, the service life of an immersed tunnel is expected to be over 100 years. It is well known that rubber is a material which is easy to undergo stress relaxation in the long term. Permanent deformation of the rubber gasket is expected due to stress relaxation. On the other hand, the loads applied on rubber gasket are obviously not constant due to temperature variation; tide level variations and the characteristics of earthquake. Under cyclic loading, permanent deformation of rubber gasket would also occur. In the design, in order to ensure the water tightness (i.e. minimum compression) during the design life of tunnel, it is important to estimate accurately the long term stress-deformation relations by including relaxation and aged deterioration of Gina gasket to ensure the water tightness (i.e. minimum compression) during its whole tunnel life. Figure 5 shows a typical revised load-displacement curve by considering of both stress relaxation and cyclic loading history. Nowadays, it is widely assumed empirically that the permanent deformation due to relaxation (Δ_r) is 15% of the initial compression by the end of its service life.

$$\Delta_r = 0.15\delta_i \tag{2}$$

4 PROJECT BACKGROUND OF ZHOUZOUZUI IMMERSED TUNNEL

The Guangzhou Zhoutouzui immersed tunnel project is composed of 422 m open ramp and 1372 m Cut and Cover Tunnel in both Haizhu District and Fangcun District, and 340 m Immersed Tunnel underneath the Pearl River (Figure 6). The project is proposed to alleviate the traffic congestion in the downtown of Guangzhou, P.R. China. AECOM has been assigned as the chief designer of this key infrastructure project in Southern China.

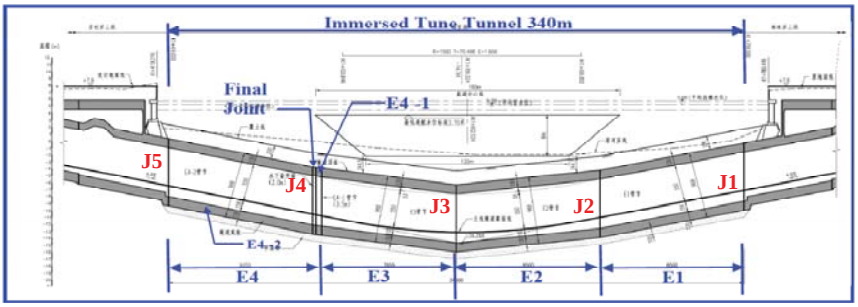


Figure 6: Longitudinal profile of ZTZ immersed tube tunnel

The 340 m long immersed tube tunnel, with a rectangular cross section of 31.4 m wide and 9.68 m high, is

divided into 4 tunnel units (E1 to E4) connecting with flexible joints. The lengths of E1 and E2 are both 85m while the length of E3 is 79.5 m. Total length of E4 is 90.5 m which consists of E4-1 (3.5 m), E4-2 (85 m) and final joint (2 m) between E4-1 and E4-2. The longitudinal section of the immersed tunnel is shown in Figures 6. One type of Gina gasket was assigned at Joint 1 and 2 while another type of Gina gasket was applied at the other joints. The level of the riverbed is -2.0 m, the 1 in 100 tide level, the average high tidal level and the lowest low tide level are respectively +7.72 m, +5.68 m and +3.36 m.

5 AXIAL DEFORMATION OF FLEXIBLE JOINTS

In this section, all these possible axial movements would be discussed in details. A summary is shown in Table 1 for Zhoutouzui project.

5.1 Variation in smoothness/flatness of the steel frame (Δ_1)

According to the allowable construction tolerance of steel frame as specified in the design report, the variation in smoothness/flatness of the steel frame is less than 3 mm.

5.2 Rotation of immersed tunnel element (Δ_2)

The opening or closure of flexible joint due to rotation of immersed tunnel element is cause by the differential settlement of tunnel units. The differential settlement was calculated by longitudinal analysis (AECOM, 2005a). As long as the rotations of different units were obtained, the opening or closure of flexible joint due to rotation of immersed tunnel element can be calculated by the following equation:

$$\Delta_2 = \theta H/2 \tag{3}$$

where θ rotation of immersed tunnel element and H is the height of tunnel element.

5.3 Creep and shrinkage of concrete (Δ_3)

The creep and shrinkage of the tunnel units as a whole will cause reduction in length and thus gap opening. To assess the effect of creep and shrinkage, a simplified method of assuming a temperature change of 15°C is used to estimate the movement caused by creep and shrinkage of concrete. Since the tunnel units will be fabricated in dry dock before it is immersed in the river. It is reasonable to assume 1/3 of creep and shrinkage will occur during the service life of tunnel. The amount of joint opening can be determined by the following equation:

$$\Delta_3 = \alpha \Delta t (L_r + L_l)/2 \tag{4}$$

where α is the thermal expansion coefficient and is equal to 0.00001/°C; Δt is the equivalent temperature change and is equal to 5°C; L_r is the length of half tunnel unit on the right side of joint; and L_l is the length of half tunnel unit on the left side of joint.

5.4 Temperature effect (Δ_4)

The overall increase or decrease of the temperature of the tunnel unit will lead to longitudinal deformation. Since flexible compression joint are not restrained to longitudinal deformation of the concrete, therefore temperature variation causes both extension and contraction of units (i.e. both gap opening and closing). A temperature change of 10°C is considered in the estimation of joint movement by temperature variation according to equation (Equation 4).

5.5 Earthquake movement (Δ_5)

The movement of flexible joint due to earthquake loading was calculated based on the history of earthquake record of Guangdong Province. The design earthquake loading of 0.144g was adopted as suggested by

Guangdong Province Earthquake Engineering Laboratory Center. Details of the earthquake analysis can be found in Report of Seismic Analysis (AECOM, 2005b).

5.6 Permanent deformation (Δ_6)

As discussed in Section 3.5, it is assumed empirically that the deformation due to relaxation is 15% of the initial compression by the end of operation period.

Table 1: Summary of axial deformation

Joint No.	Δ_1		Δ_2		Δ_3		Δ_4		Δ_5		Δ_6	
	close	open	close	open	close	open	close	open	close	open	close	open
J1	3.0	-3.0	0.7	-0.7	0.0	-2.1	4.3	-4.3	4.4	-3.9	0.0	-11.0
J2	3.0	-3.0	0.2	-0.2	0.0	-4.3	8.5	-8.5	1.8	-2.5	0.0	-10.8
J3	3.0	-3.0	0.0	0.0	0.0	-4.1	8.2	-8.2	7.4	-11.1	0.0	-11.0
J4	3.0	-3.0	0.0	0.0	0.0	-4.3	8.5	-8.5	4.6	-4.1	0.0	-10.7
J5	3.0	-3.0	0.4	-0.4	0.0	-2.3	4.5	-4.5	11.7	-12.9	0.0	-10.7

6 SELECTION OF GINA GASKET

The Gina gasket selected should satisfy the two principles. (1) Maximum allowable compression should not be exceeded in order to prevent compressive failure of rubber based on supplier's information; and (2) Minimum allowable compressive deformation (stress) should be achieved to maintain water-tightness in both construction stage and the operation stage based on the water-tightness performance curve.

Three design cases were considered. In case I, the average high tide level is used with considering all the possible closing movements in order to check whether the maximum allowable deformation is exceeded. In case II, the lowest low tide level is used with all the possible open movements in order to investigate whether the minimum allowable deformation is fulfilled. In these two cases, a loading factor of 0.7 was applied to the deformation due to temperature variation in order to consider the combined loading effect. In case III, 1 in 100 tide level is selected without considering the earthquake effect, which is regarded as an accident case.

The total deformation of Gina gaskets according to the above three design cases are summarized in Table 2. It can be noted that all deformation fulfil the design principles and hence the selected Gina gaskets are regarded as satisfactory.

Table 2: Summary of final deformation

Joint No.	Case I (mm)		Case II (mm)		Case III (mm)	
	Close	Open	Close	Open	Close	Open
J1	84.980	55.912	84.665	49.847	79.405	44.587
J2	85.880	47.435	82.920	45.275	79.330	41.685
J3	87.330	49.711	89.821	38.640	87.821	36.640
J4	85.350	47.372	85.070	43.492	81.260	39.682
J5	83.750	54.916	89.701	38.985	84.601	33.885

7 CONCLUSIONS

Gina gasket is the key element of an immersed tube tunnel. It prevents water ingress into the tunnel not only during the construction period but also during the operation period. Hence, it is important to propose a reliable design method which can estimate the opening and closure of the Gina gasket during the whole tunnel life accurately. The paper presents the principles, procedures and considerations of Gina design through an on-going project, Guangzhou Zhoutouzui immersed tube tunnel. Particularly, the present method included the time effect (i.e. stress relaxation and cyclic loading history of Gina gasket), which could yield more reliable results. However, more research is still required in order to have a clearer understanding of the kinematic behaviour of Gina gasket during seismic motion.

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AECOM 2005b. *Seismic Analysis Report of Guangzhou Zhoutouzui Immersed Tube Tunnel*, AECOM Asia Co. Ltd.

Ground Improvement for a Large Jacked Box Tunnel

A.M. Pearson, & A.S.K. Au,
Benaim URS, Hong Kong
A.N. Lees
Benaim URS, Brisbane
J. Kruger
Thiess John Holland JV, Brisbane

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. Benaim, 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 inimizes 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.

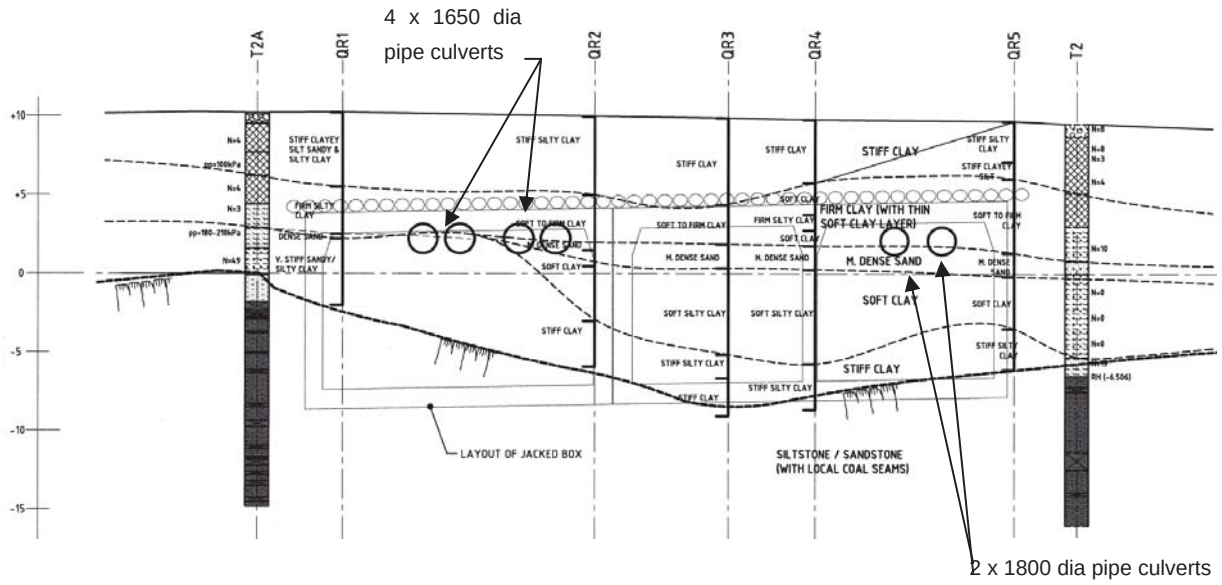


Figure 1: Transverse geological section

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.

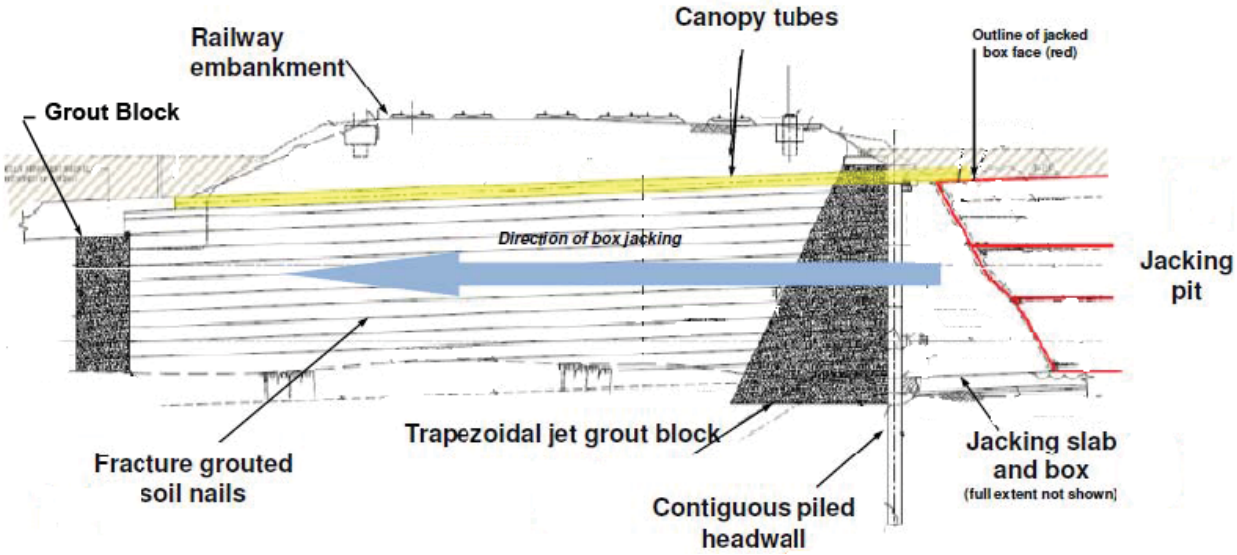


Figure 2: Arrangement of ground improvement regime to facilitate jacking

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 inimizi 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 inimizi 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 inimizing 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 Bjerruml (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.

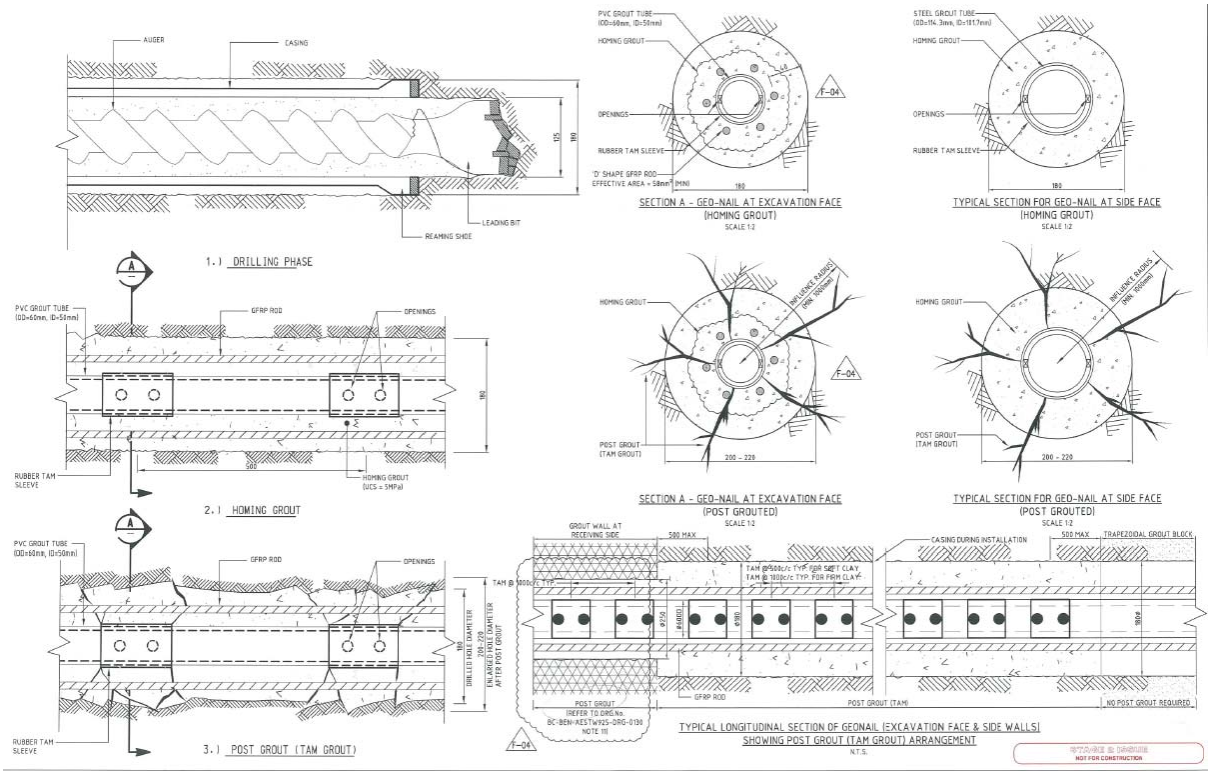


Figure 3: Detail of geonails

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:

$$E_u = q_{net}\, b\, (1- vs^2)\, Is/\delta p \qquad (1)$$

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.

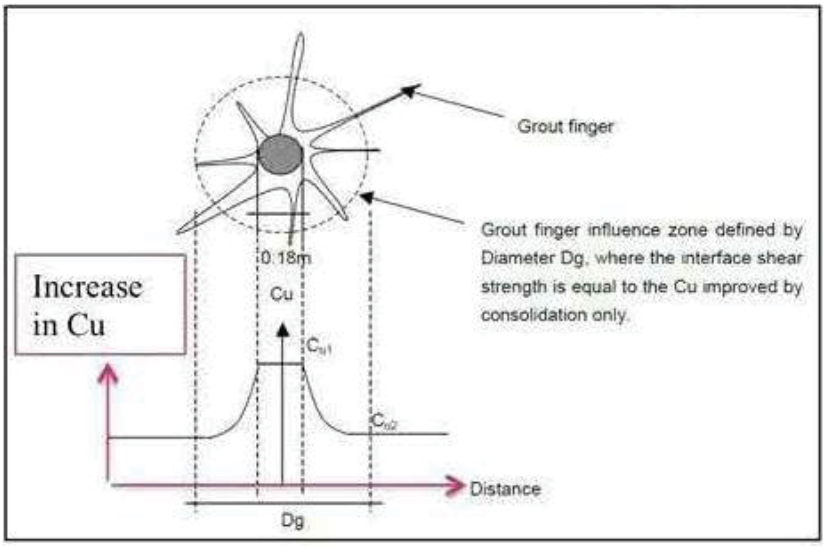


Figure 4: Effect of soil improvement with distance from the nail body

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.

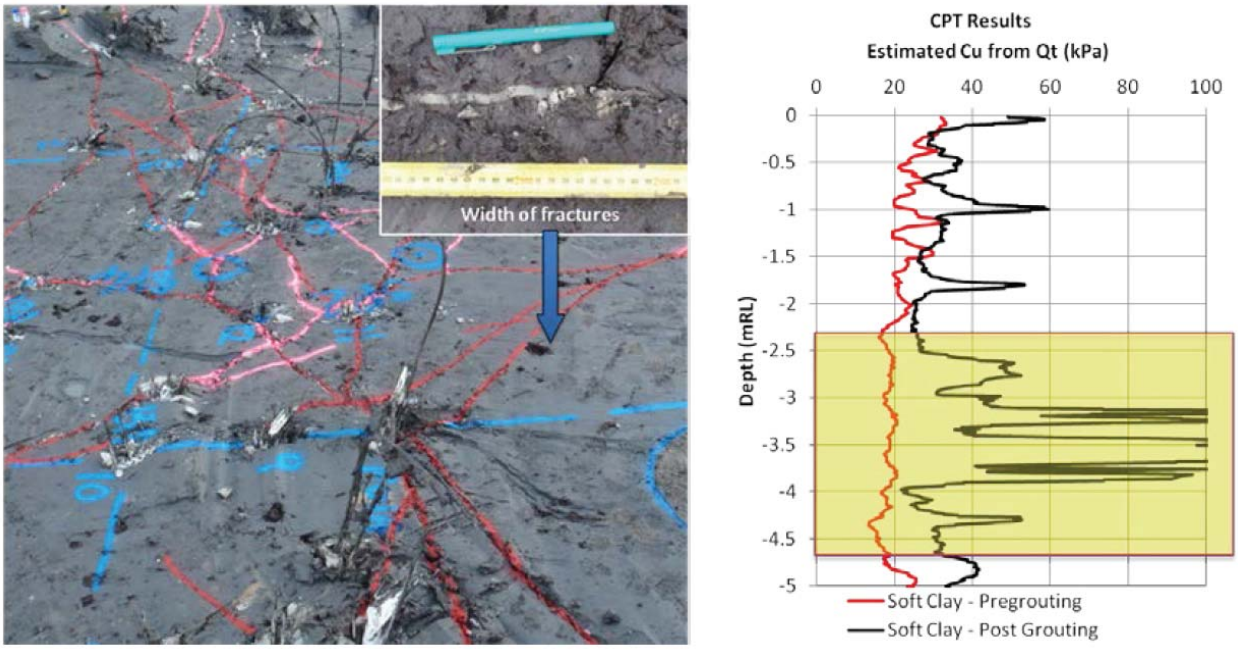


Figure 5: Exposed fractures from nail trial & CPT testing showing strength improvement

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.

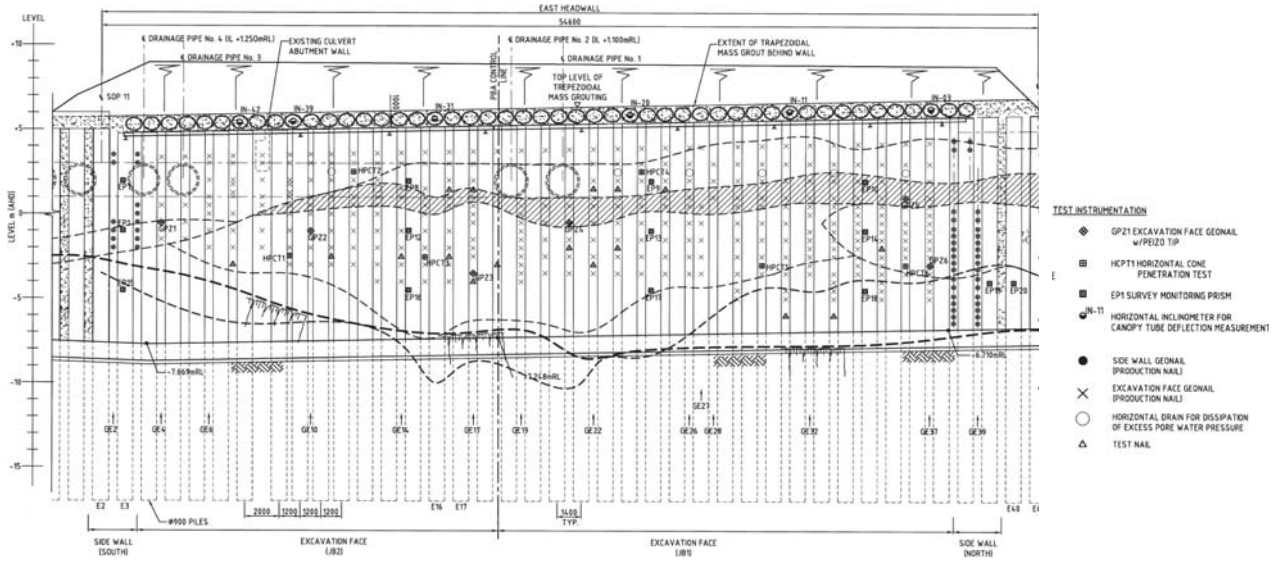


Figure 6: Elevation of headwall showing geonail & instrumentation layout

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.

ACKOWLEDGEMENTS

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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Implementation of Comprehensive Geotechnical Monitoring Programme against Ground Displacement before and during Construction of the HATS Project in Hong Kong

S.W.B. Mui, S.W.K. Wong & C.S.M. Choy

AECOM Asia Co Ltd, Hong Kong

R.K.F. Seit

Drainage Services Department, Hong Kong

ABSTRACT

Many tunnelling infrastructure works are on-going in Hong Kong recently, including the construction of 21 km of deep sewage tunnels under the Harbour Area Treatment Scheme Stage 2A project. Although water ingress has been tightly controlled by comprehensive pre-excavation grouting, any drawdown of the groundwater level during construction of the sewage tunnels may cause ground settlement which can subsequently affect existing buildings or structures in the vicinity. In recognition of this potential risk, an extensive geotechnical monitoring programme is implemented to monitor the ground conditions against possible displacement impact to existing structures and utilities. It can also provide forewarning for carrying out necessary protective measures to existing buildings and structures, particularly to those with historical or archaeological values.

This paper presents the types of the geotechnical monitoring stations including ground settlement markers (GSM), piezometers, automatic groundwater monitoring devices (AGMD), utility monitoring points (UMP), structure settlement markers (SSM), extensometers and vibration monitoring point (VMP) as well as the special monitoring on sensitive building with historical value nature using visual survey and thermographic imaging survey. The difficulties and constraints to implement the monitoring scheme and additional monitoring identified during the construction stage are also reviewed.

1 INTRODUCTION

The Harbour Area Treatment Scheme (HATS) aims at improving the water quality in the Victoria Harbour by collecting sewage from urban areas on both sides of the harbour for centralized treatment at Stonecutters Island Sewage Treatment Works (SCISTW). HATS Stage 1 covering urban Kowloon, Kwai Chung, Tsing Yi, Tseung Kwan O and eastern part of Hong Kong Island was commissioned in December 2001. Since then, the water quality of the Harbour has substantially been improved. Construction of HATS Stage 2A, which includes 21 km of sewage conveyance system (SCS) collecting sewage from the northern and south-eastern part of the Hong Kong Island, commenced in July 2009 and is in progress. The method of rock excavation is by means of the drill and blast method.

For the construction of the SCS, the Drainage Services Department (DSD) has implemented a comprehensive monitoring programme to monitor ground conditions, existing structures and other possible impacts which may arise as a result of the works. The monitoring programme includes the installation and monitoring of Ground Settlement Markers (GSM), Structure Settlement Markers (SSM), Utility Monitoring Points (UMP), Vibration Monitoring Points (VMP), Extensometers and Piezometers equipped with Automatic Groundwater Monitoring Device (AGMD).

The monitoring stations are in general so located to extensively cover an area within a distance of 400m from both sides of the SCS alignment which is shown in Figure 1.

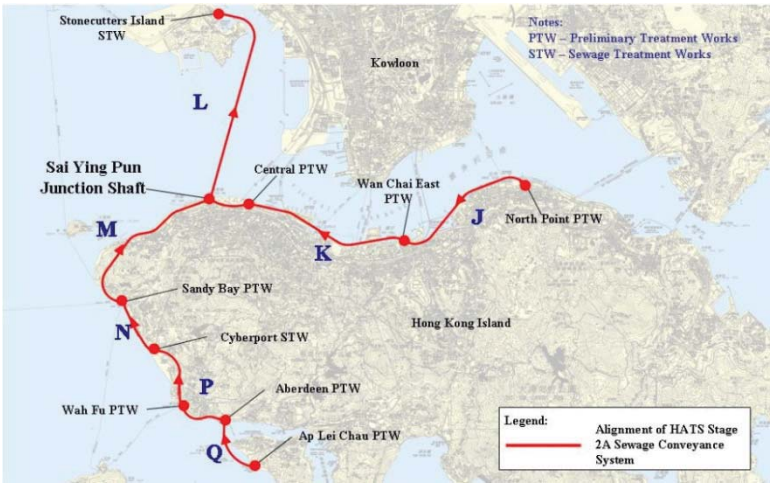


Figure 1: Plan of HATS2A Sewage Conveyance System

2 TYPICAL INSTRUMENTATION

2.1 Ground Settlement Marker (GSM)

Ground Settlement Marker (see Figure 2) is used to monitor ground surface movement. The ground settlement marker type 1 is installed at ground surface for measurement of soil surface settlement. It comprises a 50 mm long x 15 mm diameter surveying nail with a plastic collar top and epoxy bottom.

The ground settlement marker type 2 is installed at concrete paved or artificial hard surface. It consists of a 25 mm outer diameter steel rod whose lower end is welded to 250 x 250 x 12 mm steel plate that is firmly embedded in crushed stones. The upper end of the rod is hemispherical in shape from which vertical displacement could be measured. The steel rod is protected by 75 mm diameter PVC pipe and installed 0.4m below concrete pavement.

2.2 Utility Monitoring Point (UMP)

Ground settlement may lead to settlement of buried utilities and inflexible pipes are particularly susceptible to damage. Hence, Utility Monitoring Point (UMP) (see Figure 2) is proposed to be installed at gas mains and water mains which are less flexible. For utilities within zones susceptible to high settlement risk, the UMPs were installed at 50m spacing along the tunnel alignment.

The UMP basically consists of a steel pipe flange with a vertical section of steel riser pipe, which is installed on top of existing utilities pipe in a 125 mm diameter borehole. Centralisers are placed at 3m intervals within the steel riser pipe. A PVC sleeve is used to isolate the steel riser pipe from the soil and the space in between the PVC sleeve and the riser pipe will be filled with bentonite slurry. The UMPs are also placed at the existing gate valve chamber, the cap of gate valve or any fixed point on the gate valve for monitoring of any settlement of the existing pipe.

2.3 Structure Settlement Marker (SSM)

Structure Settlement Marker is used to monitor the movement of the existing buildings/ structures which may be affected by the works. Featuring a spherical head and a plated body that minimize damage to the building, the wall-mounted marker is installed around the external facade of the building.

To adequately monitor the settlement of Existing Building Structure (EBS) (see Figure 2), the markers were proposed based on the condition survey conducted in pre-contract stage such that any differential settlement of the EBS and hence the orientation of the deformation, could be recorded. For highway structures, the markers were installed at the two ends (mainly at the abutment which is normally rested on footing) with one in the middle to record the settlement. Some SSMs were proposed at the approach ramps to the Cross-Harbour Tunnel at Causeway Bay and Western Harbour Crossing at Sai Ying Pun. Tilting markers have been also installed on some buildings adjacent to the shaft.



Figure 2: Examples of Ground Settlement Marker (GSM), Utility Monitoring Point (UMP) and Structure Settlement Marker (SSM)

2.4 Piezometer/AGMD

Multi-level piezometers containing up to three tips with tips installed at a minimum depth of 10m into bedrock, in saprolitic soil and in the superficial deposit layers to detect any change in pore water pressure in various strata. The piezometers are equipped with automatic groundwater monitoring devices (AGMD) which measure the piezometric data electronically. The data obtained are transmitted off-site to a remote computer system by a Wireless Data Transmission Unit (WDTU). During tunnel excavation, it is necessary to continue monitoring potential drawdown of the groundwater table over an influence zone of approximately 400m in radius from the tunnel face. Although more costly than a conventional manual measurement by a dipmeter, the AGMD can record pore water pressure in a preset time interval with instant wireless data transmission. The typical details and schedule of installations are shown in Figure 3.

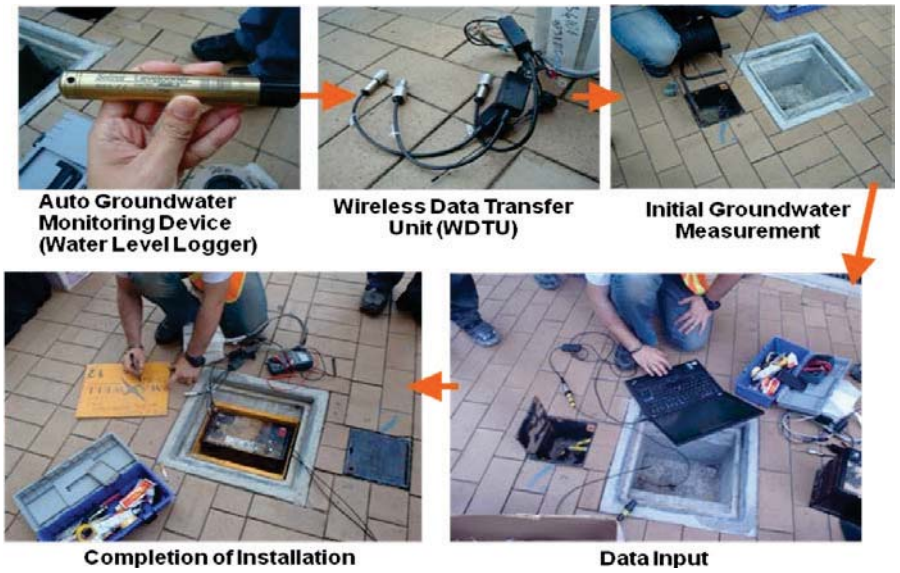


Figure 3: Installation procedure of AGMD and WDTU

2.5 Vibration Monitoring Point (VMP)

Excessive blasting vibration may cause damage to buildings, structures, utilities, slopes, retaining walls, natural terrain and even boulder fields. Hence, the blasting is required to be monitored such that it is executed within the limit set by the regulatory authority. In fact, the blasting vibration recording is a requirement of the blasting permit. The seismograph used is required to have 3 directions channels for vibration monitoring and a fourth channel for air overpressure measurement. The contractor is required to locate the seismograph

vibration sensors and air overpressure sensors at locations of sensitive receivers as determined in the Blasting Assessment Report or as directed. A set up of the seismograph is shown in Plate 1. In addition, real time vibration monitoring has been installed inside the MTRC Tsuen Wan Line Tunnel as shown in Plate 2.



Plate 1: Set-up of seismograph vibration sensors and air overpressure sensors



Plate 2: Set-up of seismograph for real time vibration monitoring inside MTRC Tsuen Wan Line Tunnel

2.6 Extensometer

Subsurface deformation monitoring is being carried out in the Cyberport Waterfront Park area to determine any settlement occurring in the compressible fill, marine deposit and alluvium layers. This was done through the installation of rod-type extensometers in two boreholes in the area. To ensure good anchorage in the soil layers, 'borros' type anchors were used. The 'borros' type anchor has 3 prongs that can protrude approximately 150 mm from the body with applied hydraulic pressure. These anchors are connected to the surface mounted reference head by measurement rods which are protected from the grout by PVC sleevings to ensure their free movement. The magnitude of deformation is determined by measuring the movement of the rods attached to the anchors relative to the head of the extensometer anchored at the top of the borehole. Installation of the extensometer is shown in Plates 3 and 4.



Plate 3: Connecting 'borros' type anchor to rod extensiometer



Plate 4: Installation of rod extensiometer

3 PROPOSED ALERT, ACTION AND ALARM (AAA) LEVELS

Under the construction contracts, the contractors are responsible for proposing the Alert, Action and Alarm (AAA) trigger levels for monitoring purpose. For monitoring of groundwater drawdown which is the primary cause for possible ground movement, it is specified that the changes in piezometric pressure head and groundwater table should not be greater than a serviceability limit equivalent to one metre head of water below the baseline. In this connection, the contractor is required to avoid any undue settlement by controlling

groundwater inflow to the tunnel such that the maximum ground settlement recorded by any GSM or UMP is not greater than 50 mm and that recorded by any SSM is not greater than 25mm under normal situation. The Alert, Action and Alarm levels being 50%, 80% and 100% of the total allowable limits. As for blasting, the vibration limit for existing buildings and structures constructed up to current standard is 25 mm/s while the maximum allowable air overpressure is 120dBL. More stringent requirements for vibration limit are stipulated for more sensitive structures such as power stations, water retaining structures and even significant monument structures. The Alert, Action and Alarm levels for vibration and air overpressure measurements are 90%, 95% and 98% of the allowable limits.

Geotechnical monitoring for the different instruments consists of standard and active monitoring at different frequencies depending on the location of the tunnel face from the monitoring stations.

4 GEOTECHNICAL MONITORING DATABASE

The monitoring data for each type of instrument described above are available for viewing 24 hours a day during construction. This is made possible because of the use of a web-based geotechnical instrumentation database (see Figure 4) which can store, analyze and present the data obtained both on screen and in report formats. All instrumentation data collected manually are uploaded into the database no later than one day after survey while piezometric pressure data collected by AGMD are transmitted every half an hour through WDTU. The database have the capacity of auto notification once AAA level is exceeded.

Graphical plots or tabulated monitoring data of any monitoring device can be easily generated from the database. The user-interface also allows data from different instruments of interests to be overlain for analysis purpose. Such functions allow the user to easily interpret if the ground settlement is caused by changes in piezometric head which indicate changes in groundwater condition.

5 IMPACT ON EXISTING BUILDING STRUCTURE (EBS) AND PROTECTION AGAINST DAMAGE TO EBS

5.1 Visual survey

The EBS along the alignment of the sewage tunnels are divided into 4 different categories, namely Category A, B and C and heritage resources for surveillance purpose under the condition survey. However, the list of existing buildings and structures all involved a pre-condition visual survey prior to the commencement of tunnel excavation. Vibration monitoring points for blasting near heritage resources was set at suitable locations with agreement from Antiquities and Monuments Office. The visual survey record (see Plate 5) provides an assessment of the physical conditions of the affected property by identifying its defects and deficiencies. The pre-condition survey is considered absolutely essential so as to protect all parties involved in the construction works against future claims.

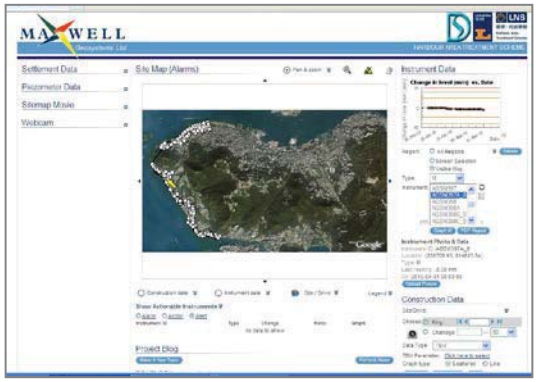


Figure 4: Geotechnical instrumentation database



Plate 5: Condition of an existing crack recorded during the visual inspection survey

5.2 Thermographic imaging survey

For Category A EBS, which is highly sensitive existing building structure adjacent to the tunnel alignment, thermographic imaging survey is required in addition to the conventional visual survey. The thermographic survey is a quick and non-destructive testing to examine old building structures. It identifies the debonding of finishing and water leakage problem, and therefore can provide an efficient and effective evaluation about the building condition.

In general, areas with air voids in defective concrete have higher temperature than normal concrete under solar radiation. Thermograph image shows these slight temperature differences in locating the problematic area. Thermographic imaging survey has been carried out at a Category A EBS, namely the batteries adjacent to the Victoria Road, and the record is illustrated in Figure 5.

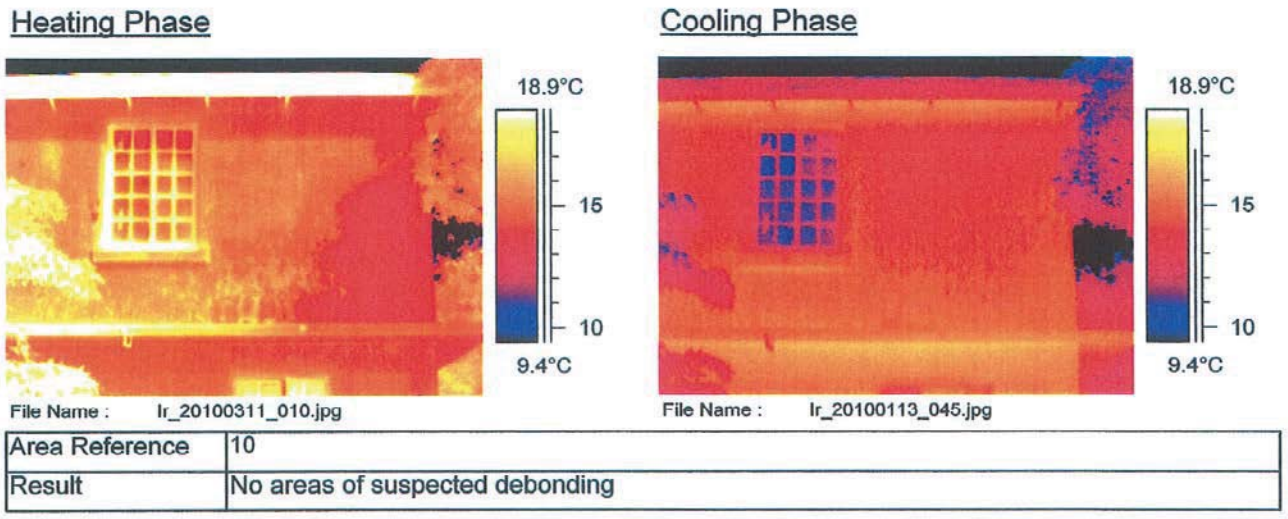


Figure 5: Thermographic Imaging Survey at Category A EBS, Batteries near Victoria Road

6 DIFFICULTIES AND CONSTRUCTION CONSTRAINTS

As the piezometers used to monitor groundwater condition are installed deep below ground near the SCS whose alignment is in close proximity to the sea, the water in the piezometer tube is saline in nature. This variation in the density of the water in the tube warrants the need of regular calibration to correlate the pressure recorded by the transducer of the AGMD against the true depth of the water column to ensure correctness of water level data.

During the construction of the two deep diaphragm wall shafts in the Sai Ying Pun area, it was noted that the piezometers/AGMD installed deep into the bedrock nearby reacted by showing a drop in piezometric pressure through real time transmission. The observed piezometric pressure head drop was interpreted to be caused by the presence of voids in the rockhead allowing water seepage into the shafts. This led to the instruction of extensive toe grouting at rockhead level of about -80 mPD in the course of construction of the shafts. The additional grouting proved to be critical in making the shafts watertight as supported by the presence of grouting material filling the joints and cracks in the rockhead in subsequent excavation of one of the shaft as shown in Plate 6.



Plate 6: Exposed rockhead showing presence of micro-fines cement grout in joints and cracks

The benefit of having real time monitoring as exemplified above is that decision for correction can be made at the first sign of problem.

It is noteworthy to mention that the piezometric head in the rock could be affected significantly even by drilling for pre-excavation grouting at a distance of 150 m if packers were not applied immediately to stop the inflow from connected flowpaths. The observed problem was due to site constraint of limited working space thus restricting timely installation of packers. The piezometric head drop eventually recovered after the hindered drillholes were grouted.

During the construction of the HATS2A project, a number of other projects including the MTRC West Island Line, Hong Kong West Drainage Tunnel (HKWDT) and Laying of Western Cross Harbour Main and Associated Land Mains from West Kowloon to Sai Ying Pun projects were in progress in different areas along the SCS alignment. As such, regular meetings with these parties were needed to exchange monitoring data and construction programmes. Joint surveys were also carried out with Western Harbour Crossing and Stonecutter Island site.

As regards vibration monitoring, the Contract states that the influence zone is 132m based on attenuation of blast vibration to 5 mm/sec on the use of 6kg maximum instantaneous charge (MIC). However, AMO requires that vibration monitoring should be carried out whenever the proposed blasting is within 200 m from a target buildings or structure. The influence zone for buildings and structures with historical and archeological values identified along the alignment of the SCS was later extended to meet AMO's requirement. These liaison works and on-going monitoring works are crucial to the project progress and stakeholders.

7 CONCLUSION

In support of the construction of deep seated sewage tunnels under the HATS 2A project, a comprehensive monitoring programme has been implemented according to Contract requirement. Monitoring of disturbance to ground, utilities and building structures at greenfield locations along the tunnel alignment before commencement of excavation works provides a baseline from which the effect of ground movement due to tunnelling can be assessed. The installed piezometers equipped with AGMD provide first hand information of the piezometric pressure through real time transmission to guard against potential effect of surface settlement due to consolidation of the underlying soil strata. The deep piezometers installed in the bedrock are most sensitive to change of piezometric pressure in the pore spaces of the rock and considered instrumental in monitoring groundwater inflow to tunnels during their construction. The condition survey protects the interest of all parties involved in the construction in the event of a third party claim. In this connection, both visual inspection and thermographic imaging survey were applied depending on the categories of the EBS. The use of a web-based database greatly enhances the application use of the geotechnical instrumentation.

ACKNOWLEDGEMENTS

Special thanks to the Director of the Drainage Services Department, the Government of the Hong Kong Special Administrative Region for permission of publication of this paper.

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Instrumentation Monitoring of TBM Tunnelling Effects to Adjacent Pile Foundation for HATS 2A Project

Y.T. Liu & A. Cheung
Ove Arup & Partners Hong Kong Limited
W.L. Chan
Drainage Services Department, Government of the Hong Kong SAR

ABSTRACT

The rapid development and redevelopment of the urban area usually demand the use of deep foundation. With the need of infrastructure being planned underground and constructed in the form of tunnelling method, there is a high possibility that the tunnel alignment may become very close to these adjacent existing foundation. Prediction of adjacent pile responses and ground deformations caused by tunnelling is therefore important as part of the design. This Paper presents the instrumentation and assessment of the impact on adjacent rock socketted pre-bored H-pile and ground movements arising from a tunnel construction in the Harbour Area Treatment Scheme Stage 2A project. Field data in comparison with predictions under theoretical methods is also included. Back-analysis of volume loss is given and it is suggested that 2% of volume loss shall be used for design under similar geological condition of reclaimed land area.

1 INTRODUCTION

The Harbour Area Treatment Scheme Stage 2A (HATS 2A) is a major government infrastructure project in Hong Kong aiming to upgrade the existing facilities to treat the sewage caused by urban development around Victoria Harbour, improve the water quality and maintain a healthy and sustainable marine environment. Under HATS 2A, an Interconnection Tunnel was to be constructed linking the existing and the new main pumping station inside Stonecutters Island Sewage Treatment Works (SCISTW). The tunnel was aligned across the future Northern Sludge Cake Silos and Sludge Dewatering Building which the foundation for the two new buildings had been constructed prior to TBM construction. A 10m width protection zone was reserved for the TBM construction as well as to minimize the TBM effect to the adjacent piles.

Tunnel construction invariably causes ground movements and changes in the field stress conditions. Many attempts had been made to simulate the tunnelling effect in centrifuge model tests but few papers discussed the real tunnelling-induced responses and compared the real responses with the theoretical prediction especially in Hong Kong. This Paper presents: (i) the details of instrumentations and monitoring on the newly constructed piles and adjacent areas during the tunnel construction, (ii) tunnelling-induced responses of single pile from the monitoring and comparisons with theoretical methods, (iii) ground deformation patterns obtained from the monitoring and comparisons with theoretical predictions, (iv) analysis on the field data and comparison with the theoretical data and predictions adopted in the design stage, and (v) suggestions for future geotechnical assessment of similar construction effects.

2 BACKGROUND INFORMATION

The site has a typical offshore geology in Hong Kong which is generally underlain by Fill, Marine Deposits, Alluvium and Completely Decomposed Granite (CDG). The bedrock level is between 50 m and 70 m deep. The new piled foundation adjacent to the tunnel comprises more than 240 numbers of 610 mm rock socketted pre-bored H-piles and were installed prior to the tunnel construction (Figure 1). The pre-bored H-piles were socketted into bedrock with maximum socket length of 4.5m with pile capacity up to 5,500 kN.

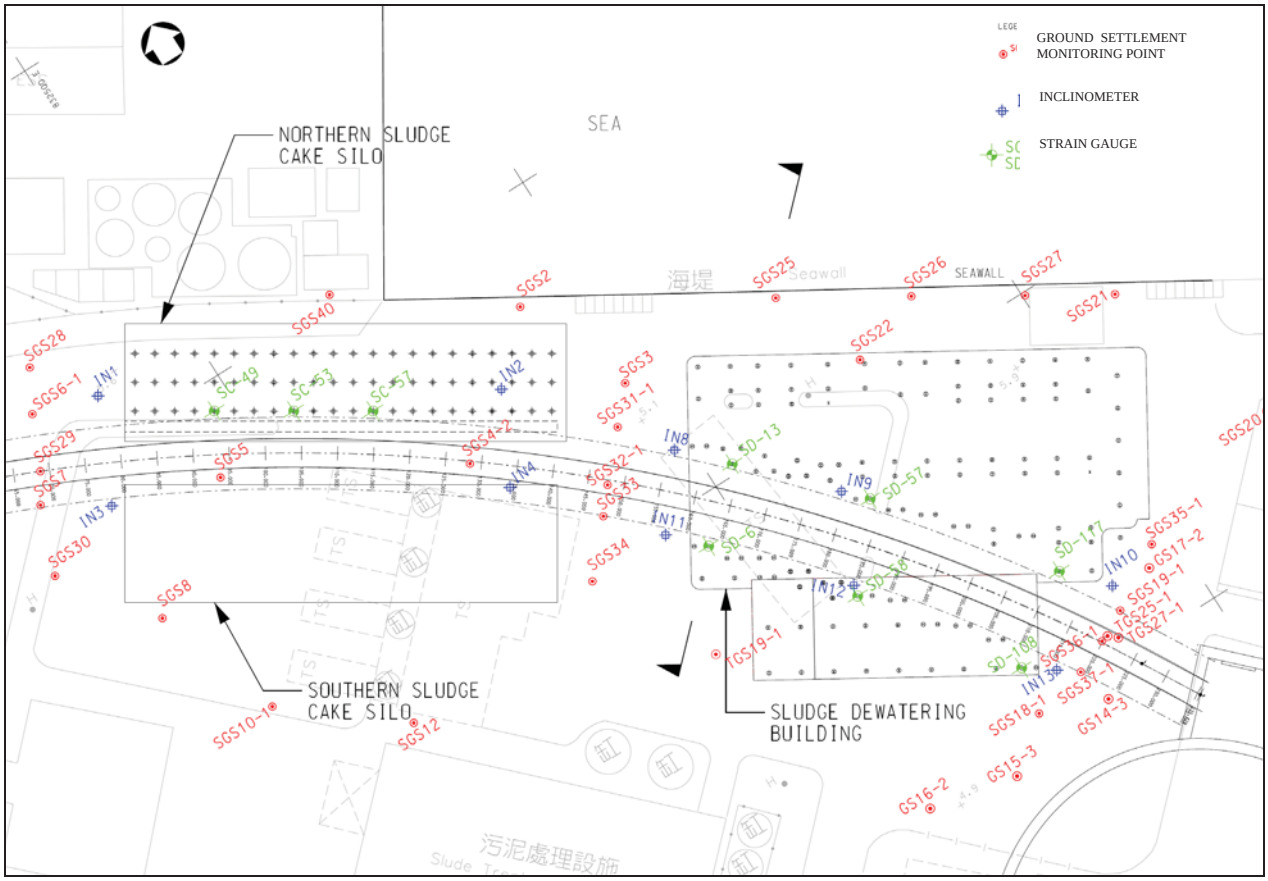


Figure 1: Tunnel layout plan

Note: Cross-section is shown in Figure 2.

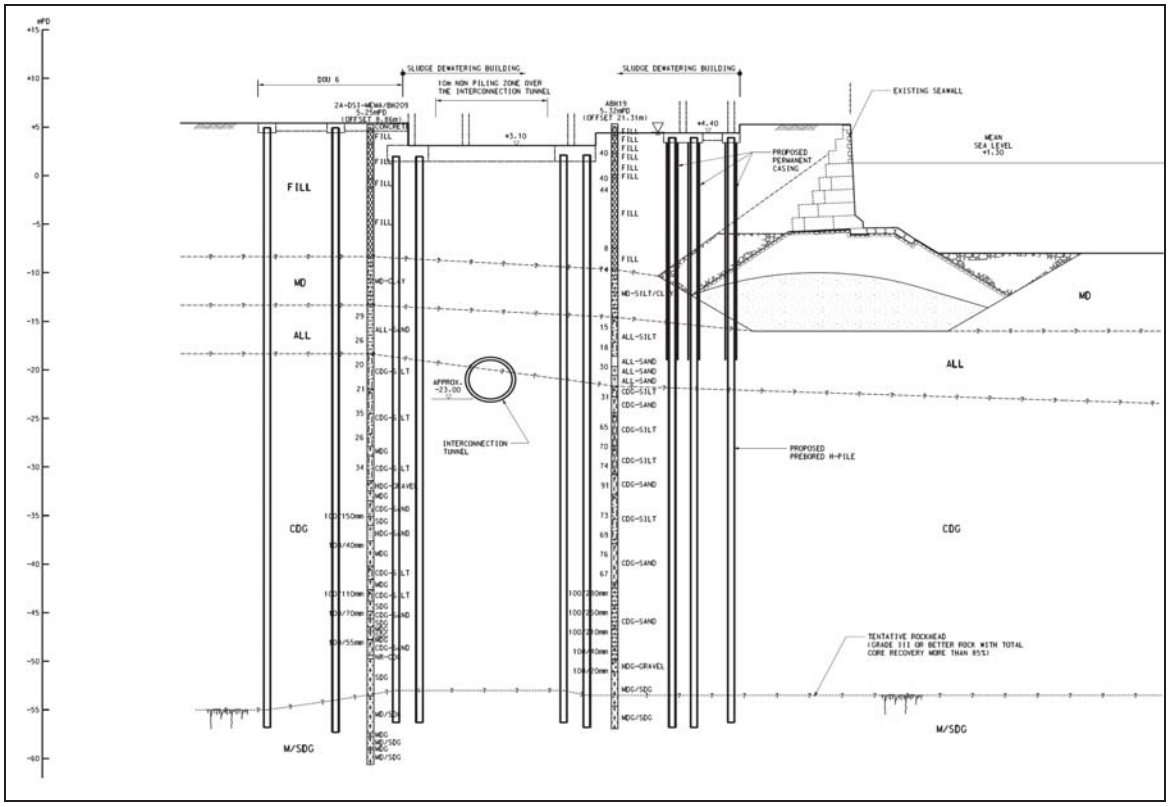


Figure 2: Cross section showing the tunnel and the adjacent pile foundation

The proposed tunnel has an invert level of about 28m below the current ground level and having an internal diameter of 4 m and around 240 m in length. The ground stratigraphy revealed that the tunnel is located within the alluvium layer and occasionally CDG (Figure 2). The tunnel was to be built by Earth Pressure Balanced type Tunnel Boring Machine (TBM). The typical advance rate for this TBM was around 4 m per day. The volume loss was estimated to be within 4% at the design stage.

In order to monitor the ground condition and the effect of the TBM construction, there were various instruments installed around the pile foundation locations. It consisted of ground settlements, building settlements, standpipes/piezometers, strain gauges installed along newly constructed piles and inclinometers. In addition, the TBM face pressure, excavated material, measured grout volume were also recorded and reviewed during the construction stage. However, only the monitoring of the induced pile responses and the adjacent soil movement will be presented in this Paper for the sake of readability.

3 MONITORING OF INDUCED PILE RESPONSES

Vibrating wire type strain gauges were installed to the 9 numbers of steel H sections prior to installation which were used to measure the induced strains (Figure 3). 5 sets of strain gauges were installed along each selected pile at prescribed levels -11, -16, -21, -26 and -31 mPD and each set consisted of 4 strain gauges installed on the inner face of the H-pile flange at each prescribed level (Figure 3).

The strain gauge produces the strain of the pile at the prescribed level and the stresses can be obtained by the stress-strain relationship. The axial force at the prescribed level is then calculated by the average measured stresses from the 4 strain gauges at the same level times the steel sectional area. Besides, the bending stress developed in the pile is the net value of the average stresses from the strain gauges A/B and C/D (Figure 3). The bending moment will be determined by the following simple equation:

$$M = \frac{\sigma I_x}{y} \tag{1}$$

where M = the bending moment, σ = bending stress, I_x = the second moment of area about the neutral axis x, y = the perpendicular distance of strain gauge to the neutral axis.

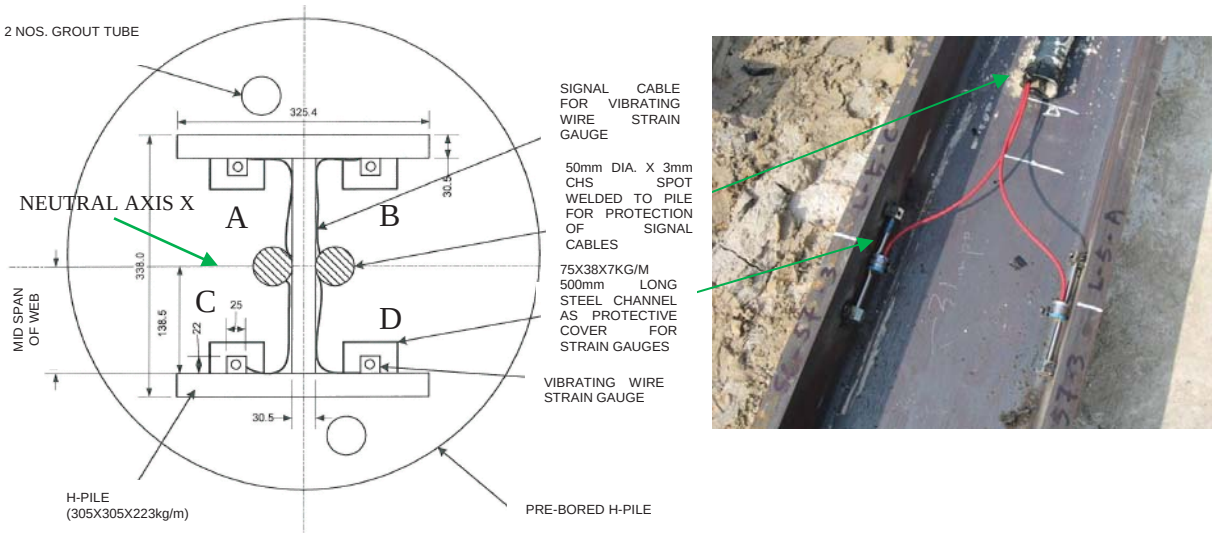
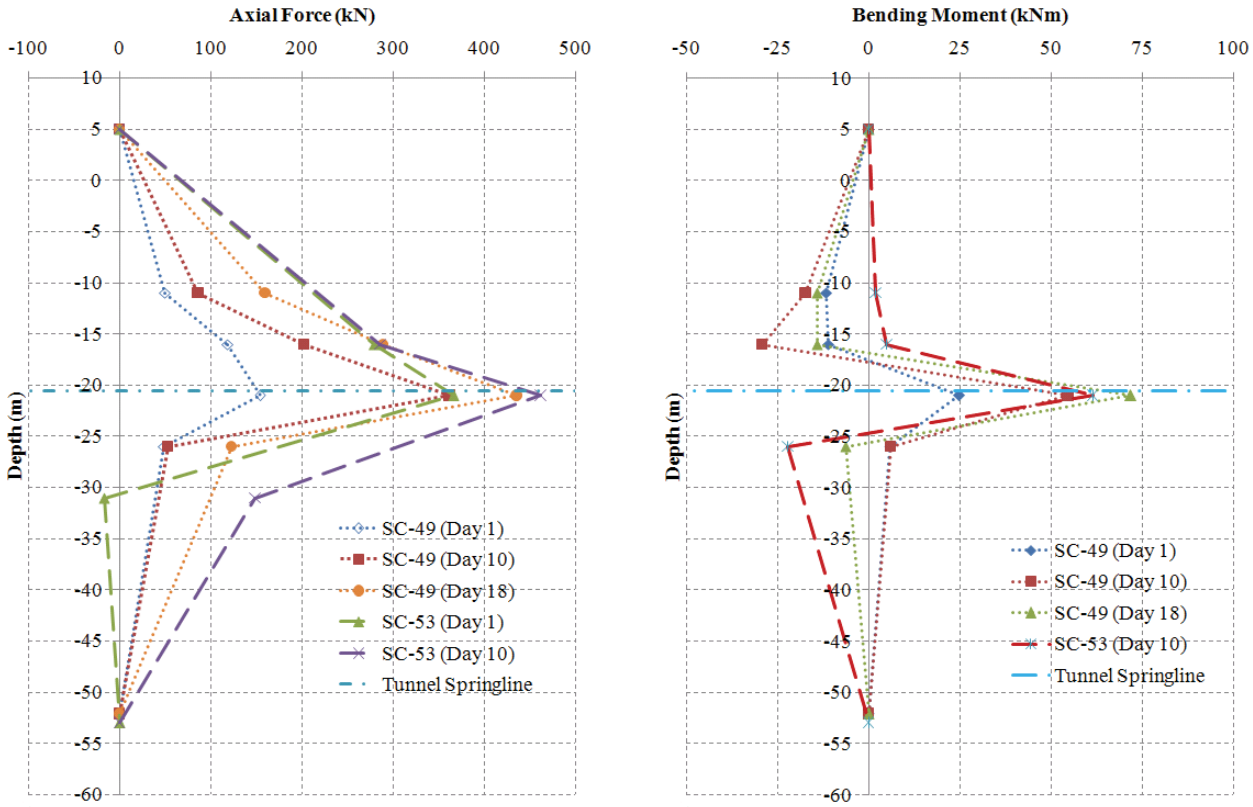


Figure 3: Arrangement of strain gauges on pre-bored H-pile

By the time of preparing this Paper, the TBM had passed through three of instrumented piles. Figure 4 shows typical profiles of the induced axial force and bending moment due to the period of adjacent tunnelling passed by at the 1st day, 10th day and 18th day. Herewith 1st day is defined as the first day TBM passed by the instrumented pile at the closest 3 m distance. The interpreted profiles have the following features:

- From the Figure 4(a), it is observed that the induced pile axial force increases downward and reaches a maximum value at the level of tunnel springline.
- The reading in Figure 4A reveals that there is a significant increase in maximum axial force, 435 kN at 18th day for SC-49 and 461 kN at 10th day for SC-53 which is equivalent to a maximum of 8.4% of the axial working load capacity (5,500 kN) within 3 m from pile edge to tunnel edge.
- Figure 4(b) indicates that the interpreted bending moment profile has a double curvature, with the maximum value occurring at the level of the tunnel springline and the trend matches with the result derived from simplified boundary element analysis by Loganathan & Poulos (1999).
- In Figure 4(b), the induced bending moment increases to 72 kNm (18th day) for SC-49 and to 62 kNm (10th day) for SC-53 within 3 m from pile edge to tunnel edge.
- The majority of axial load and bending moment has been developed at the first 10 days after the TBM passing by the instrumented pile while there is small portion of increment occurred between 10 and 18 days, as shown in Figure 4.

In deriving the pre-bored H-pile design, 600 kN axial compressive load and 100 kNm bending moment have allowed to cater for the effects of tunnelling in the vicinity. From the instrumentation results, it can be seen that these values are considered adequate.



(a) Induced Pile Axial Load (b) Induced Pile Bending Moment

Figure 4: Induced pile responses

4 MONITORING OF SOIL MOVEMENT

A comparison of surface settlement troughs obtained using various methods such as Mair (1993) and Loganathan & Poulos (1999) and measured data is shown in Figure 5.

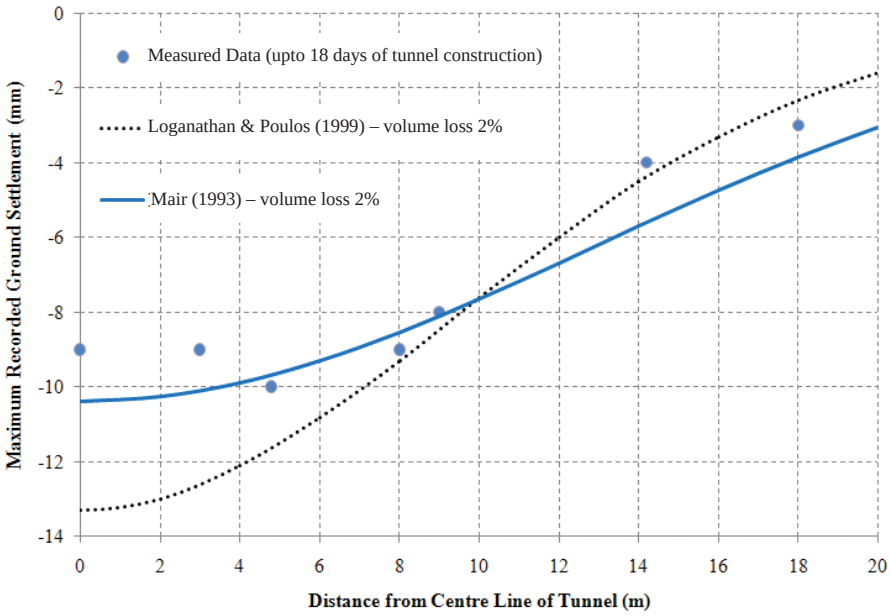


Figure 5: Comparison of surface settlement predicted and observed values

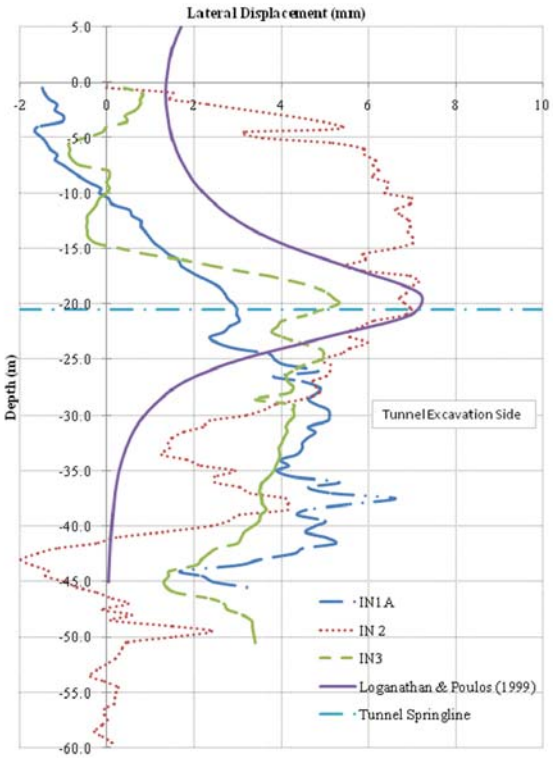


Figure 6: Induced lateral displacement at the 3 m from tunnel

The maximum surface settlement occurred above the centre of the tunnel was measured to be 9 mm and becomes less gradually as the distance from the tunnel increases. Using a volume loss of 2%, Gaussian distribution function and i=0.5z suggested surface settlements derived by Mair (1993) and analytical method by Loganathan & Poulos (1999) are introduced for comparison with the measured data. As shown in Figure 5, it is noted that the measured immediate surface settlement trough at 18th Days after excavation more or less follows the well-established Gaussian distribution. Although the analytical method by Loganathan & Poulos (1999) is more commonly adopted for tunnel designs in Hong Kong, it is observed that the empirical method by Mair (1993) has a better fit for the tunnelling induced settlements in reclaimed land under the conditions of this project.

Ten inclinometers were installed adjacent to the interconnection tunnel and the TBM had passed through six of them. Figure 6 shows the typical lateral displacement profiles for several inclinometers. The maximum lateral soil displacement occurs at the tunnel springline with a maximum value of 7 mm. The observed data comparing with the analytical prediction by Loganathan & Poulos (1999) with a volume loss of 2% and distance of 3 m from the tunnel edge is shown in Figure 6. The maximum predicted displacement value is in good agreement with maximum measured displacement. It is also noted that the lateral soil movement has similar trend as the analytical predictions as illustrated in Figure 6.

Therefore from this result, it is observed that the assumption of 2% volume loss is reasonably adequate for estimating the lateral soil displacement.

5 CONCLUSION

Although according to Ran (2004) and Pang et al (2005), the long term effect of the tunnelling may be notable, it is considered not quite applicable to this project since there are several occasions in the past showing the surface settlement occurred almost immediately within 2-3days after the construction activities. The paper presents the results covering the period of 3-4 days before and as long as 18 days after the TBM passed by the instrument. As such this period is considered to be sufficient to include the immediate effect and even the delay response of the ground.

This Paper presents the typical rock socketted pre-bored steel H-pile responses within a distance of 3m from the tunnel edge in reclamation area. The monitoring data reveals that the induced maximum pile axial force and bending moment due to the tunnel construction are 461 kN and 72 kNm respectively. Both the maximum axial force and bending moment are located at a level close to the tunnel springline. Adequate allowance of additional axial force and bending moment was provided in the original pile design.

In this Paper, the available field data for estimating the tunnelling-induced ground movement for the HATS2A project are assessed and reviewed. The surface settlement is described with reasonable accuracy by Gaussian distribution proposed by Mair (1993) for tunnel in reclamation area with 2% volume loss.

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Risk Management for Ground Engineering Works: the Role of Independent Instrumentation Monitoring Consultant

Angus Maxwell & William Tai
Maxwell Geosystems Ltd

Arthur So
Mass Transit Railway Corporation

ABSTRACT

The engineering community has successfully completed many exceptionally challenging construction projects. Unfortunately, history has shown that on occasions, political, time and monetary pressures have exceeded those of the water and ground, sometimes leading to failure. Authorities have attempted to mitigate these risks through the implementation of a variety of independent design checkers and verifiers and through the provision of supervisory teams on site. These organizational systems have resulted in improvements and one of them is the instrumentation monitoring. However, a common complaint is that the monitoring information is received too late and in forms which are not readily analysed or checked by the engineers. The Express Rail Link (XRL) is the first project in Hong Kong to have a role provided for an independent professional body to check, audit and deliver the project monitoring data to the project stakeholders. This paper will report on its method of implementation, benefits to the project and provide the guidance for those considering the management of project risk on future projects.

1 INTRODUCTION

The Mass Transit Railway Corporation (MTRC) has developed the use of the Independent Instrumentation Monitoring Consultancy (IMC) as part of a project wide risk management initiative. This paper describes the origins of the role, its original concept and the way in which the role has evolved to become a key part of the MTRC systems. The key elements of construction monitoring can be divided into:

(a) Design and prediction of movements
(b) Installation of monitoring instruments
(c) Instrument monitoring and communication of measurement results and their likely causes
(d) Comparison of measurements with predictions
(e) Review of predictions to define the next level of critical movements whilst dealing with any existing critical movements
(f) Change of works procedure or methods to confine further movements to within the next level of predictions.

2 MANAGEMENT STRUCTURE

In all construction cases, design and prediction is provided by the design engineer with review from the owner or his site teams and associated regulatory bodies. The designer will design instrumentation to suit the monitoring of these parameters and to enable sufficient data to be collected to feedback information of the performance of the design. This is then passed to the site teams for implementation. At this time a variety of bodies become involved in the project delivery process. These are indicated in Figure 1.

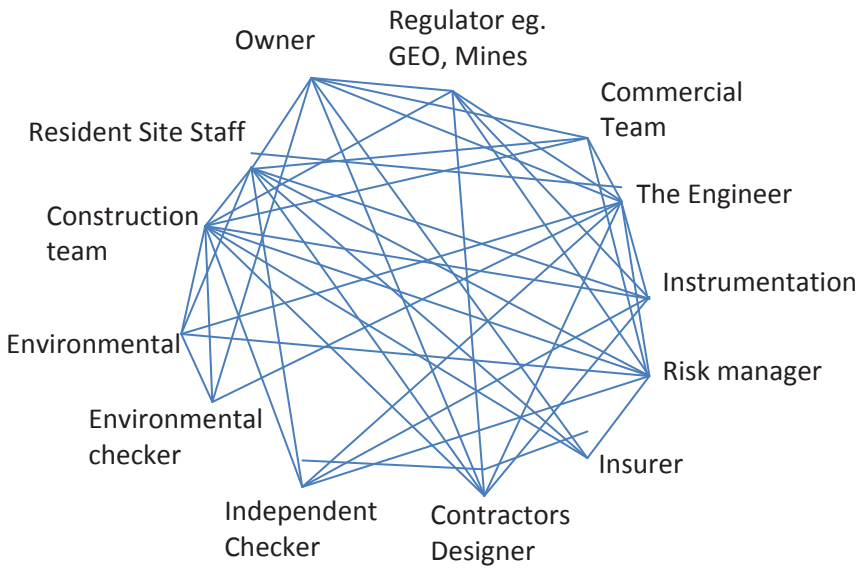


Figure 1: Complex interaction of teams at Construction Stage

The communications and interactions between these various bodies are complex but at this stage the construction is live and decisions will need to be made within a tight time framework. The ability of the management to react to change will be governed by the systems in place.

2.1 Management challenges

With such a complex interaction, management of projects faces several challenges. Much effort is put into ensuring that the objectives of all the team members are aligned towards common safety, technical and commercial goals by the use of partnering and in some cases alliancing. Historically little focus has been placed on the management of information flow between the various parties to the project.

Ultimately deviations from the prediction for the works sometimes lead to technical and commercial conflict and parties may justify their positions by "cherry picking" information to suit a particular argument. As the result, neither party has a complete set of information to form an overall picture of the matter. In some cases two sets of records exist and this is counter-productive.

In fact many of the issues boil down to information such as:

(a) Handling the flow of information
(b) Handling the huge quantity of information
(c) Uncertainty as to the reliability of the information received: especially if it is being recorded by separate parties who may have very specific agendas
(d) Independence of the party undertaking the work
(e) Checking the information
(f) Duplication of information
(g) Ability to interpret the information quickly and accurately

2.2 Improvements to management systems

Change is a natural part of construction and the management of this change should be embraced within the project management scheme. Since this is an expected occurrence, the management structure should be geared to respond proactively rather than defensively. The objective should be to maximise the amount of time spent on engineering interpretations rather than operating a computer.

Better information sharing and communication is required and the delivery of "agreed" factual information should be speeded up. There needs to be better cross pollination with information from other teams and facilitation of back analysis and comparison against design. Risk management should be integrated into the

systems rather than being separated. Better communication, agreeing, sharing and condensing of data leads to smaller more effective teams. Efficiencies relieve time and monetary pressure which impact on quality.

2.3 Independent monitoring versus contractor monitoring

In Hong Kong all construction instrumentation is carried out by the contractor, and is not uncommon using specialist subcontractors. For most projects, an estimate of about 1% of the construction cost is often set aside in the budgets for the instrumentation of ground engineering projects within the urban environment. This should be considered as a "lower bound" and in some very complex high risk projects up to 5% has been set aside.

Monitoring within Asia tends to be seen as an imposition by the owner/designer and the authorities on the contractors to safeguard against failure or damage to the environment. Such a policing approach has not engendered buy in from the contractors and as such many will opt for the cheapest solution. If results are inconclusive or instruments fail, this is seen as removing restrictions from the contractor's working environment. A change in attitude to one where the monitoring is considered a help would require that the contractors take some benefit from the monitoring when the design is not over-conservative or under-provided. Such observational engineering requires careful application but if applied would give incentive to produce quality and reliable information.

In Hong Kong, the instrumentation contractor is normally subcontractor of the main contractor. If time or monetary pressure is felt by the main contractor, this pressure may be passed on to the instrumentation contractor. This can be risky if the contractor's instrumentation subcontractor is also constructing the temporary works. The MTRC has addressed such a conflict by requiring the instrumentation contractor to be independent of the geotechnical works.

In Singapore, all instrumentations for government works are contracted directly to the owner. This removes any potential pressure the contractor may bring to bear but also removes any direct involvement of the instrumentation contractor in the construction process thereby breaking the feedback loop.

3 THE INDEPENDENT MONITORING CONSULTANT

3.1 Origins of the IMC role

When the KCRC and the MTRC merged in 2009, all rail ownership and rail project delivery was brought under one roof. Since this was a part public company a certain level of review was required for government projects to be sub vented to the MTRC for development. Independent monitoring of environmental compliance has been there for some time but additional areas covering technical monitoring, finance and design were added.

Initially the independent monitoring of the West Island Line was issued as a works contract given the high proportion of measurement over consultancy services. The second independent monitoring contract was for the Regional Express Line and by this time it was issued as a consultancy reflecting the increased focus on the engineering services. The consultancy comprised:

(a) Physical monitoring of up to 17% of the contractor's monitoring
(b) Provision of a Unified Web database for the presentation of all monitoring results
(c) Review of all monitoring designs
(d) Review of on-going monitoring and production of weekly and monthly reports

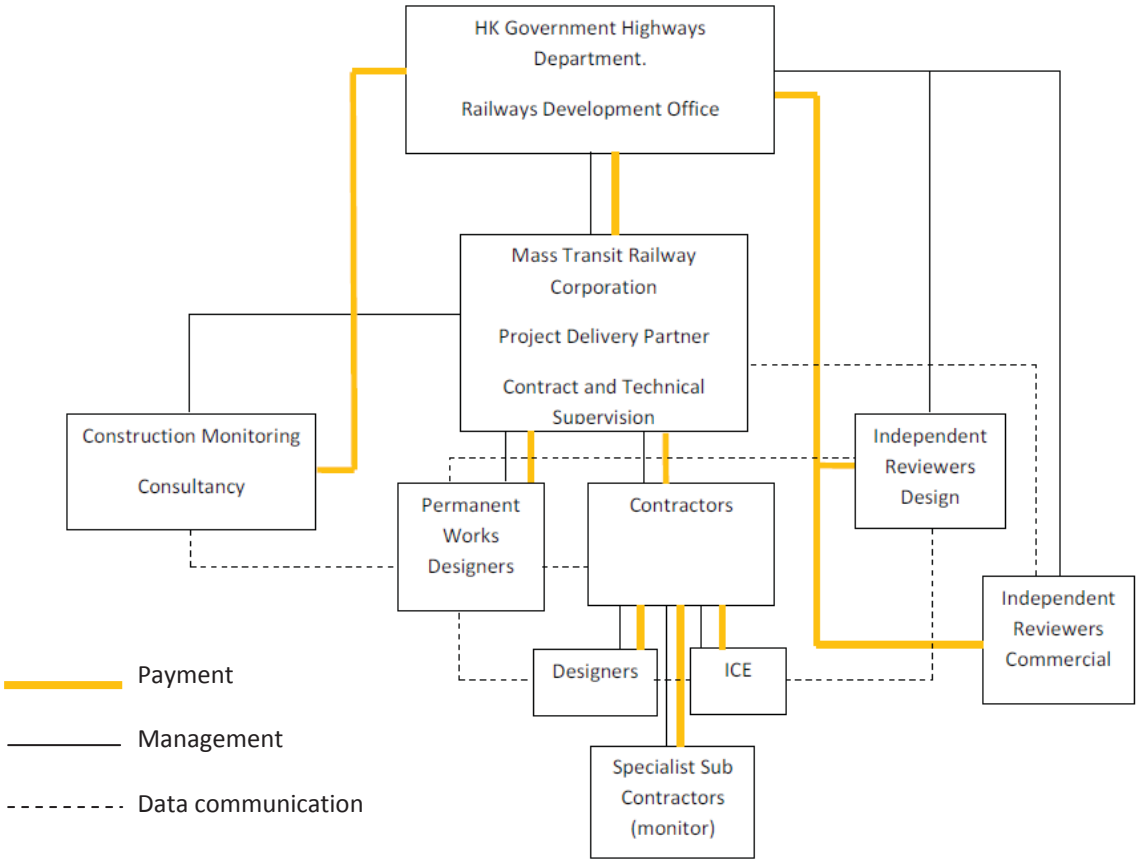


Figure 2: The Regional Express Line Contract Structure

3.2 Independent physical monitoring

Low prices put pressure on quality. In some cases where resources are just not available data may be extrapolated or fudged. Often there is a focus on providing good looking graphs rather than truly representing the data. The advent of independent physical monitoring helps to ensure that the instrumentation and survey personnel provide the required frequency of measurement and that the measurements are undertaken with the required levels of accuracy.

3.3 Provision of a unified system for management of data and independent data processing

The provision of a central unified system for the publishing of data to all the project members is the foundation of a new method of construction risk management. Provided by an independent third party, this system acts as the published repository for construction data which is accessible over the web to all. Secure layers are set such that on multi-contract projects parties can only see information relevant to their contract and to the contracts adjacent.

The setup of the system is designed to ensure the maximum independence of the data and speed of processing. Key aspects are:

(a) All data is received as raw data
(b) Data can be received in a number of structured formats such that the production of data for the system should require no additional steps for the suppliers
(c) The data can be provided from different teams within a contract such that survey can provide survey data and geotechnical can provide geotechnical data, thus remove delays from combined submission
(d) The data is independently processed
(e) Real time data is provided to the system in real time
(f) Data shall be available to the public within 30 minutes of loading

(g) The system shall allow unusual data to be quarantined for investigation if required but quarantined data shall be visible on the web site if it would cause an alarm
(h) A GIS shows the location of the instruments and their current alarm state
(i) Individual items of construction work are identified and their progress is shown on the GIS
(j) The system is able to identify IMC data from contractors data and easily plot this data together for comparison
(k) The system is able to plot large data volumes efficiently (at least 10000 records in 10 seconds)
(l) The system also allows progress information to be added to construction elements so that the actions which may be causing adjacent instrumentation responses can be identified and interrogated
(m) AGS data or, if unavailable, PDF logs of boreholes are also added to the system to enable the geotechnical significance of movements to be determined
(n) Alarms are sent by SMS and Email but also registered to the online system as part of a weblog. First response to the alarm is to the weblog and comprises a confirmation that the alarm is correct and observations as to work going on in the area
(o) Alarm reports are generated from the system as required by any user with those access rights

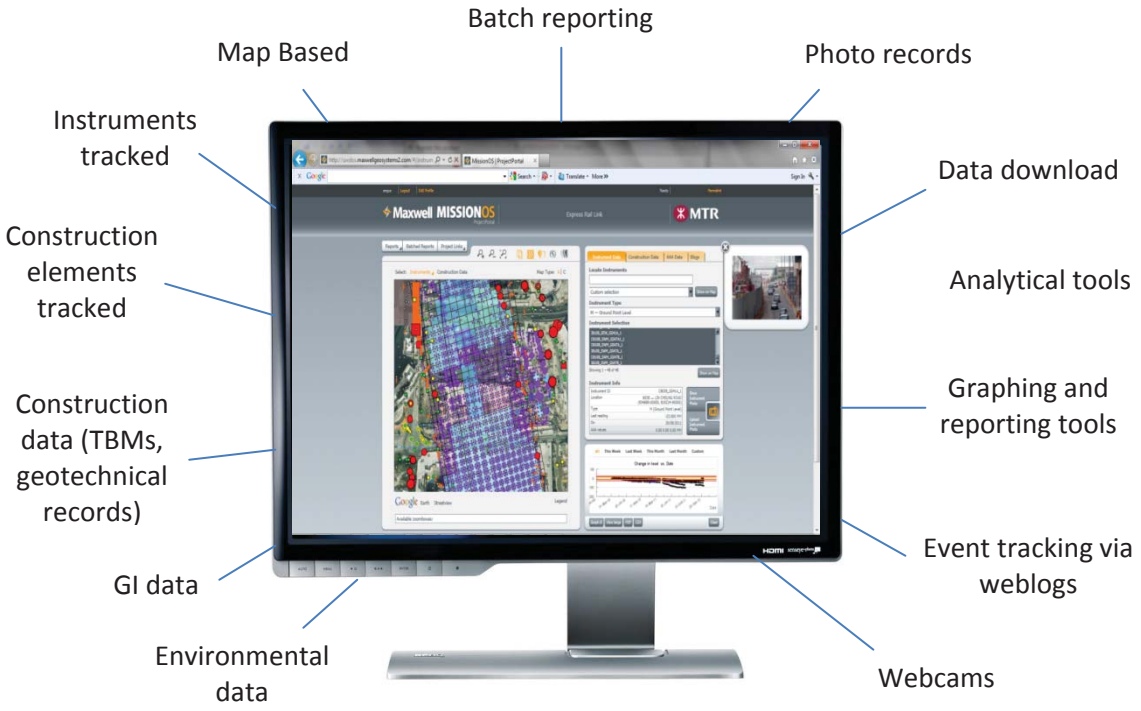


Figure 2: The Unified Web database (UWD)

These capabilities significantly extend those required by the original MTRC specification. This highlights the difficulties common with specifying a system implementation. Unless there is in depth knowledge of what can be done the Engineer tasked with specifying a capability has no knowledge of how difficult or much time this will take to implement, particularly when dealing with a general software house. In this case the supplier reacted to the intentions of the MTRC and provided them with achievable solutions.

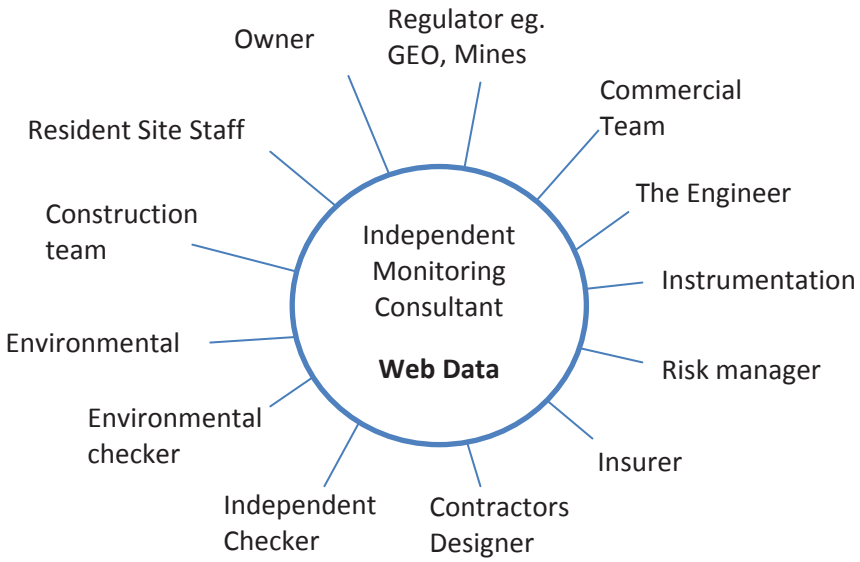


Figure 3: Complex interaction of teams simplified using the UWD

Whilst the public portal is most users interface to the data this is only the tip of the iceberg. Behind the scenes systems are designed to cope with imperfect and incomplete data so that data can be presented in a timely fashion. Such systems include:

(a) Automatic revision of instruments and calculation of carry over value
(b) Easy revision of inclinometer for variation in top of pipe level, change of probe, blockage etc
(c) Processing of inclinometer from top or bottom
(d) Processing of magnetic extensometer data from any reference position or from top of pipe
(e) Bias removal from inclinometer
(f) Correction of depth based readings (piezometers, extensometers, inclinometers) for variation in top level (including interpolation)
(g) Correction of all points for benchmark fluctuations
(h) Identification and compensation for natural fluctuations.
(i) Automatic filtering against credible ranges in both magnitude and rate
(j) Automatically assigning default parameter so that all instruments are registered to the system and not forgotten
(k) Automatically updating the system for new instruments as they appear in the data
(l) Automatic audit facilities to check for data missing key information or with quarantined data or data outside of limits
(m) The facility to combine instruments in any way to produce further instruments which may have engineering significance, such as angular distortion of utilities. To interpolate and discount data to ensure that calculations are meaningful
(n) The facility to group instruments and display data in plan, section or 3D
(o) The facility to search and locate any instrument in question instantly in plan for investigation

3.4 Review of monitoring designs and on-going project performance

Key aspects of the physical monitoring undertaken are that the results reduce and not increase uncertainty. To ensure that monitoring is effective it is important for the project team to appreciate:

(a) What can be achieved with the type and distribution of instruments chosen and what cannot
(b) Whether external factors will affect the performance of the instruments
(c) Whether the instruments are installed and set up for best recovery of quality data
(d) Additional processing steps to improve data quality

As a specialist in this field the IMC provides useful advice. A further important area of independent review is the choice of alarm levels. These are normally linked to design predictions and tolerances and it should be clearly identified what is the basis for the alarm eg: tolerance of a building or structure or predicted movements of a piece of temporary works and this indicated on any AAA report. In some cases it may be neither. Most of these alarms are based on the primary parameter – settlement, deflection, load, draw down. In many cases this is not the crucial parameter. The advantage of systems is their ability to track other derived parameters. In most cases this is distortion and resulting tension.

The use of systems greatly assists the on-going review of project performance. The relationships between changes in instrumentation and changes in the works progress can be identified easily and subtle geographical and temporal relationships revealed though combining data together and even animating. The web access allows experienced engineers to view the data from remote sites and provide feedback based on accumulated knowledge some of which may be directly related to strata into which the project is being constructed.

4 CONCLUSIONS AND NEXT STEPS

Whilst the Independent Instrumentation Monitoring Consultant service is focused mainly on the instrumentation results, the holistic web management of data from constructions is already a reality. Total data management systems are already in place on projects for Drainage Services Department and include all aspects of production and technical data. These are in the process of being combined with commercial management, programme management and risk management systems to provide a single project resource for information. The ability of the systems to enhance communication and facilitate decision making may support future use of observational engineering but in the meantime the availability of data in structured system guarantees its availability for the engineering of the future.

The provision of independent monitoring, processing and reporting of ground and structure movements linked to the provision of truly independent advice has improved the risk profile of the Mass Transit Railway projects leading to a trend of lower insurance premiums.

Construction of Underground Lift Shafts and Tunnels underneath a Declared Monument, The Heritage 1881, Hong Kong

Chris Cheung, Alan Lai & P.L. Leung
AECOM

ABSTRACT

The Heritage 1881 is a new landmark in Tsim Sha Tsui. It is a sustainable development project to revitalize a declared Monument into a boutique hotel and a shopping arcade. Innovative excavation and lateral support systems such as horizontal tie-backs were adopted to facilitate this redevelopment. Two underground lifts connected with two tunnels were constructed within the monument to connect the Kowloon Park Drive and the elevated monument. The access tunnels to these underground lifts were constructed by tunnelling underneath the monument. Construction with horizontal pipe-piles to form the access tunnels with a soil cover of less than 1.5 times the width of the tunnel immediately below the footing foundation of the monument was one of the greatest challenges of this project.

1 INTRODUCTION

A declared monument "Former Marine Police Headquarter" (Main Building) was redeveloped into a new landmark in Tsim Sha Tsui. The Main Building was situated on an elevated platform and approximate 10m above the surrounding ground and it was founded on a shallow foundation system. In order to construct the new building around the perimeter of the Main Building, a deep excavation was required. The excavation and lateral support (ELS) system to support the 12m deep excavation consisted of soldier pile wall and an innovative lateral support system of horizontal steel ties (HST). After the excavation was completed, horizontal tunnels, in associate with vertical lift shafts were constructed underneath the Main Building and between the HST (Plate 1 refers). The entrances of the tunnels were located at street level of Kowloon Park Drive. Openings were required to be formed on the soldier pile wall to facilitate the construction of the tunnels. Modification/ strengthening works were required for the formation of the openings (Plates 2 to 4 shows the completed excavation works). The design not only had to cater for the lateral earth pressures, but also, more importantly had to minimize movements to the Main Building above. Both geotechnical and structural computer programs were required in the analysis. This paper presents the design approach for the mentioned tunnels and the observation of the behavior of the monument during construction.

1.1 Background

The Main Building of the monument consisted of 3-storey buildings constructed between 1881 and 1920. It was declared as monument in 1994 under the Hong Kong Antiquities and Monuments Ordinance. The building was founded on shallow foundation (strip footing) on an elevated platform which was about 10m higher than the surrounding street level. In order to facilitate the redevelopment in Year 2003, the original platform at +14.0 mPD around the Main Building was removed. The excavation was generally 12m deep with deeper areas required for the basements. The adopted ELS system consisted of soldier pile wall at 0.7m spacing supported by horizontal steel ties (HST). Each HST comprised 4 numbers of 50mm diameter rebar (similar to mini-pile) at horizontal spacing not more than 4.9m and vertical spacing of not more than 2.5m. The HST was installed by directional drilling method to ensure the alignment of the HST.

1.2 Proposed lift and tunnel

Two lifts were proposed at Kowloon Park Drive so that the public could access the Main Building from Kowloon Park Drive by means of underground tunnels and underground lifts. The width of the lift shaft and

the tunnel were 3 m and 3.5 m respectively. These structures needed to be constructed underneath the Main Building.



Plate 1: Eastern view

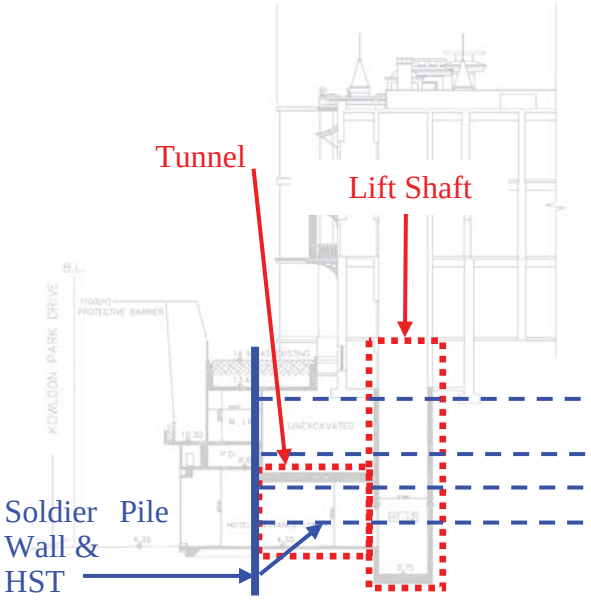


Figure 1: Cross section of the tunnel and the lift shaft

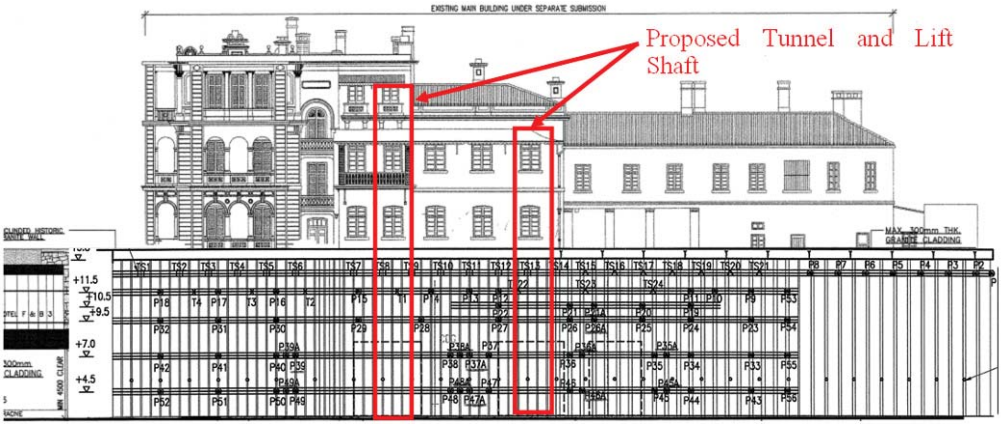


Figure 2: East elevation

1.3 Site constraints

The constraints identified during the design stage are summarized as below;

(a) The Main Building was a sensitive structure, as advised by the structural engineer, the tolerable differential movement should be limited to less than 1:1,000. In addition, it was sensitive to vibration as well.
(b) Lift shaft was situated between HSTs.
(c) The existing building was founded on shallow foundation. The works required to expose the existing footing and install the cofferdam wall, i.e. pile wall, immediately next to the footing.
(d) Limited head room (maximum 3 m) for piling plants which required modification of the piling rigs, removal of ground floor slab and local excavation to increase the headroom prior to installation of the pipe piles.
(e) Formation of the openings on the already installed soldier pile wall.

1.4 Ground condition

The geological condition underneath the footing of Main Building is tabulated in Table 1. Based on the monitoring records, the average groundwater level was +2.0m PD, which is below the existing road level and the final formation level.

Table 1: Ground condition

Elevation (mPD)	SPT 'N' value	Material Description
+13.8 to top ground level (+14.5 mPD)	-	Masonry/ Granite block
+13.8 to +8.0	8 to 48	Completely Decomposed Granite (CDG)
+8.0 to -5.0	> 100	Completely to Highly Decomposed Granite (C/HDG)
Below -5.0	-	Moderately Decomposed Granite (MDG)

2 EXCAVATION SEQUENCE

The ELS to facilitate the construction of the permanent structure was separated into 3 main stages, which consisted of excavation of vertical lift shafts, forming the openings on the soldier pile wall and excavation of the horizontal tunnel. In order to allow more feasibility for construction in terms of construction sequence and programme, as requested by the client, the tunnel or the lift shaft was designed to be constructed independently, whereas the openings on the soldier pile wall was carried out prior to tunnel construction. The following sections present the construction sequence and difficulties encountered during construction.

2.1 Excavation for lift shaft

The existing Main Building, which was supported on the footings, was the sensitive receiver. It, including the footings, was sensitive to vibration, settlement and lateral movement. Therefore, small sized replacement piles, steel pipe piles with outer diameter of 219 mm, was selected as the cofferdam wall to minimize the vibration effect to the existing structures. Odex method was adopted for the installation of the vertical pipe piles. Horizontal steel diagonal struts were adopted as the shoring to laterally support the cofferdam wall. Prior to the installation of the pipe piles, all locations of nearby footings had to be checked by means of ground radar and also trial pits. Limited head room, which was about 3 m, with the only access points, i.e. the window of the Main Building and the door, caused more difficulty in the installation of the pipe pile. Modification of piling rigs, and the partial removal of the ground floor slab with some local excavation was required to allow more headroom for the installation of the pipe piles. Breakdown and reassembly of the piling rigs were required to mobilize the piling rigs to the designated locations.

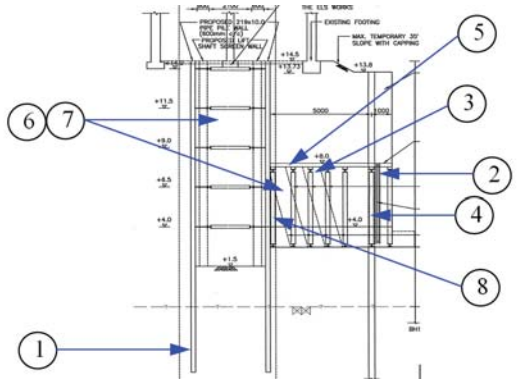
HSTs were carefully installed closely monitoring their alignment. The installation of the vertical pipe pile wall was then carried out with a tighter control on the vertical alignment as well to avoid any disturbance or damage to the existing HST. Upon completion of the pile installation, the subsequent excavation was carried out stage by stage until the final formation level was reached.

2.2 Excavation for tunnel with soldier pile strengthening frame

Tunnels were formed by means of horizontal pipe piles, which were laterally supported by vertical steel frames at 1 m c/c. Prior to breaking through the soldier piles to proceed with the tunnel excavation, strengthening of the soldier piles with a steel frame, on the outside of the soldier pile wall and TAM (Tube-A-Manchette) grouting was carried out to strengthen the soil behind the soldier pile wall in order to enhance the stability of the tunnel face during excavation inside the tunnel.

2.3 Overall excavation sequence

The lift shafts and the tunnel excavation were designed to be carried out concurrently in order to shorten the construction programme. Figure 3 summarizes the overall sequence of excavation.



Labels:
(1) Install vertical pipe piles for lift shaft
(2) Install strengthening frame at soldier pile wall
(3) Carry out TAM grouting to the soil
(4) Cut the soldier pile wall within the frame
(5) Install horizontal pipe piles for tunnel
(6) Excavation for the tunnel and the lift shaft
(7) Construct the permanent lift shaft and tunnel breakthrough the vertical pipe piles of the lift shaft and construct the structure at the connection

Figure 3: Overall excavation sequence

3 ANALYSIS

The better way to analyse a complicated 3-dimensional situation is to carry out 3-D computer modelling. However, at the time of the project, 3-D computer programs were not common and there were many limitations on the available 3-D computer programs. Therefore, 2-D analysis, using both structural and geotechnical models, with engineering judgment was carried out to simulate the 3-D effects.

Various situations were considered, they were:
(1) Redistribution of structural forces and the load path when the soldier pile wall was cut,
(2) Lateral/axial forces on the HST after redistribution,
(3) Face stability of the steep excavation in C/HDG (70° to horizontal) during tunnel excavation,
(4) The impact and influence of lift shaft and tunnel on each other.

For item 1 and item 2, the analysis was eventually carried out with a2-D computer program (Plaxis) adopting calibrated apparent stiffness values for the lateral support. It is considered that when the soldier piles are cut, the remaining part of the soldier pile wall above the opening will kick-out, and the force will transferred on to the strengthening frame; and subsequently part of these forces will be transferred to the bottom part of the soldier piles. Once the strengthening frame is in loading, the force will transfer to the adjacent waling and then HST. To avoid overload to the HST and minimize the deformation, additional HST was required as shown in Figure 4 below.

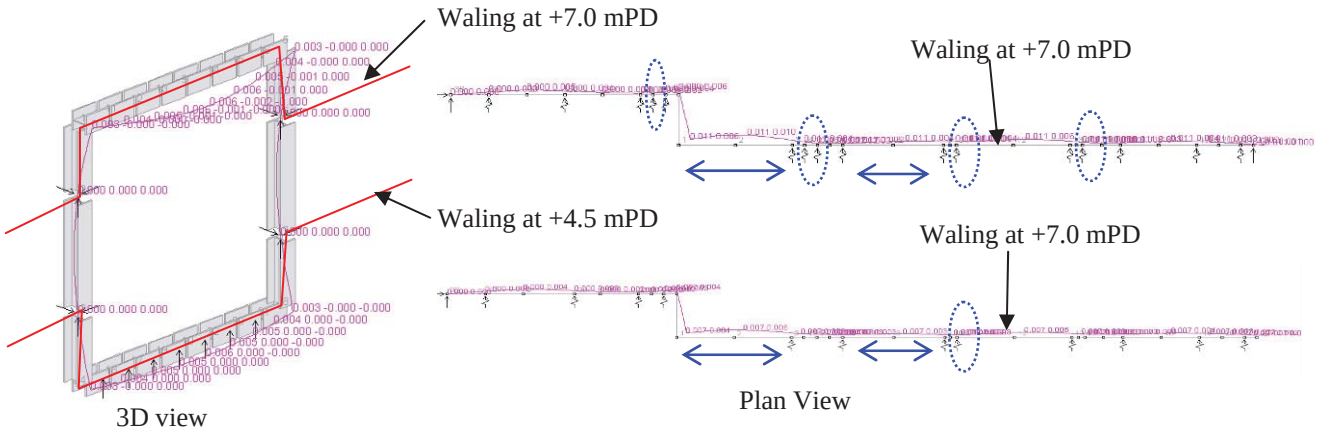


Figure 4 – Calibration of apparent stiffness of the strengthening frame by structural computer program
Note: ⇔ Strengthening Frame ⬭ Additional HST

For item 3, the face stability of 70°cutting slope during tunnel excavation was carried out by SLOPEW. The design parameters for C/HDG was c' = 5 - 10 kPa, and ϕ' = 36° with a Young's Modulus value greater than 100 MPa; the material anticipated in the tunnel excavation. The analysis concluded that the Factor of

Safety (FOS) was lower than the required. Therefore, TAM grouting was proposed to enhance the face stability. Since the tunnel was above the ground water table, the water pressure was not a major concern in the ground treatment but the strength was the major concern. Therefore, only cement/bentonite grout has been adopted TAM grouting. Based on the required FOS, which was determined based on the consequence to life and the construction was under close monitoring and supervision, the soil had to be improved to c' = 50 kPa and ϕ' = 36°.

For item 4, it is understood that the excavation of both lift shaft and tunnel may affect each other, therefore, transverse (Figures 5 and 6 below) and longitudinal (Figure 7 below) analyses were carried out as shown below.

3 DISCUSSION

Monitoring works were carried out over the construction period and the construction reviews were carried out to verify if the deformations were consistent with those predicted. Measured (by inclinometers, siturated between the two tunnels) and predicted (by Plaxis) wall deflections were extracted for comparison, Figure 8 and 9 shows the measured and predicted wall profile respectively. Based on the observation, the deflection curvature is similar.

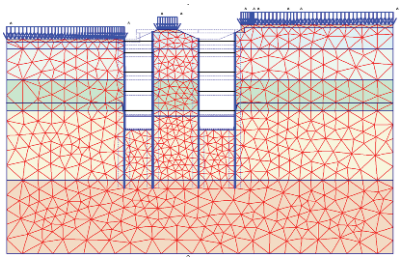


Figure 5: Section across two lift shafts (Section A-A)

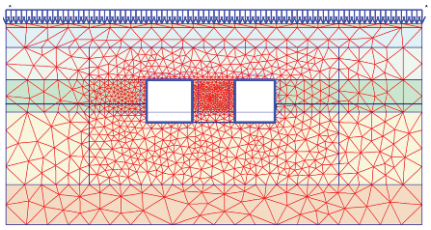


Figure 6: Section across two tunnels (Section B-B)

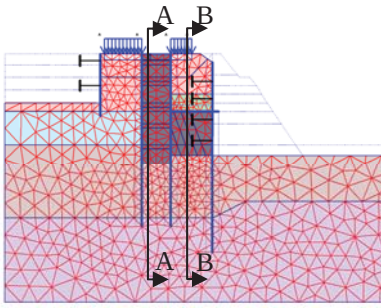


Figure 7 – Section for analysis



Plate 2: View of the tunnel



Plate 3: View of the lift shaft (via bottom)

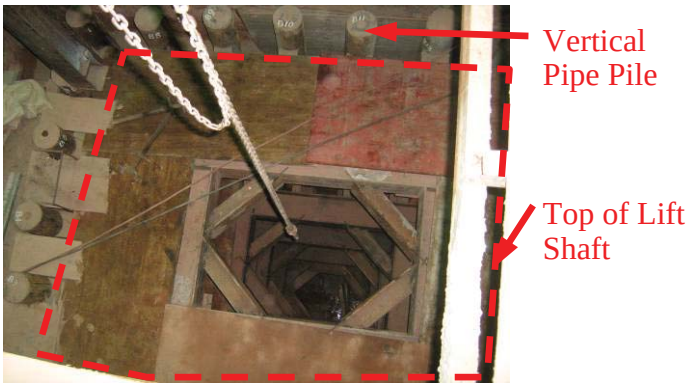


Plate 4: View of lift shaft (from top)

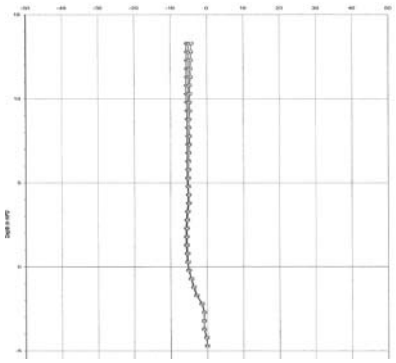


Figure 8: Measured lateral movement is 5.6 mm

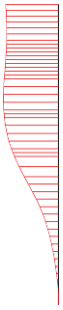


Figure 9: Predicted lateral movement is 11.0 mm

4 CONCLUSION

Although there were a lot of site constraints for the construction of the lift shafts and the tunnels in the development of The Heritage 1881, both tunnels and the lift shafts were constructed successfully and within the predicted and tolerable limit of deformation and settlement. Lessons learnt from this project for further tunneling work close to monuments are:

- Set up a clear and continuously communication between Antiquities and Monuments Office (AMO), BD and the project team.
- Comprehensive site investigation including desk study, field testing and laboratory testing with some advanced monitoring to investigate the ground conditions, verify the findings of desk study and the condition of the monument.
- Based on the site investigation develop a set of reasonable and practical tolerable limits for the monument against vibration, deformation and settlement.
- Select suitable construction schemes by experience engineers.
- The designers of ELS should have both geotechnical and structural knowledge and skill so as to establish a robust design. Computer programs have become more powerful nowadays; hence 3D modelling could be adopted in the future project so that the arching effect in the tunnel could be more precisely determined which may lead to more cost effective design.
- Understanding of the limitations of the computer programs is crucial. Splitting a complicated mechanism into different but simpler models may lead to obtain a more sensible result.
- Perform sensitivity checks when modeling/analysing the construction scheme.
- Use simple hand calculation to verify the analyzed results obtained from sophisticated programs.
- Assessment of potential impact to the affected facilities is important. Comprehensive monitoring with suitable mitigation measures should be proposed beforehand – before construction commences.
- Actual performance should be closely monitored throughout the construction period. Communication between design team and site supervision team is important. Always staying alert with respect to the scheme adopted, design methodology used, sequence of work and the actual performance.

ACKOWNLEDGEMENTS

We wish to express our gratitude to the Client (Flying Snow Ltd.) for permission to publish this technical paper and acknowledge the project team and their contribution to the successful completion of the project. The kind support from Dr. Suraj De Silva is grateful acknowledged.

Tunnel Construction by Horizontal Pipe Pile for MTR Choi Hung Park and Ride Development

Chris Cheung & Alan Lai
AECOM Asia Co. Ltd., HKSAR

Philip Lee
Formerly AECOM Asia Co. Ltd., now Leighton Asia Ltd., HKSAR

ABSTRACT

A new subway connecting existing Mass Transit Railway (MTR) Choi Hung Station and the basement of a new residential building was constructed in the project "MTR Choi Hung Station Park and Ride Development". The subway is located beneath the Clear Water Bay Road, which is a 6-lane major road, and is close to a number of MTR structures. Therefore, careful planning and special measures were required during the construction of the subway to minimize ground movements.

Horizontal pipe pile (HPP) with grouting treatment works was adopted for the excavation of the subway adit. This paper describes the design and construction details of the project, highlighting the technique of horizontal pipe piling and the effectiveness of the grouting treatment works to control the ground movement, particularly on the criteria established for the assessment of the grouted material.

1 INTRODUCTION

1.1 Background

The MTR Choi Hung Park and Ride Development consists of a 48-storey residential block, 10-storey podium floors and a 2-level basement. A new pedestrian subway (about 4.5 m x 4.5 m), which connects the basement of the new development and the existing MTR Choi Hung Station (see Figure 1), was designed in 2003 and constructed in 2005. As part of the project requirement, the construction of the subway could not be constructed by the conventional cut and cover method. Instead, a novel "tunneling" method was adopted following the construction sequence as listed below. This paper presents the details of the excavation and the temporary support works of the tunnel construction, i.e. item (c).

(a) Construct new basement by vertical pipe piles.
(b) Breakthrough the temporary vertical pipe pile wall to facilitate tunnel construction.
(c) Carry out grouting (9 m x 9 m) and excavation (6 m x 6 m) to facilitate the tunnel construction.
(d) Construct the tunnel structure

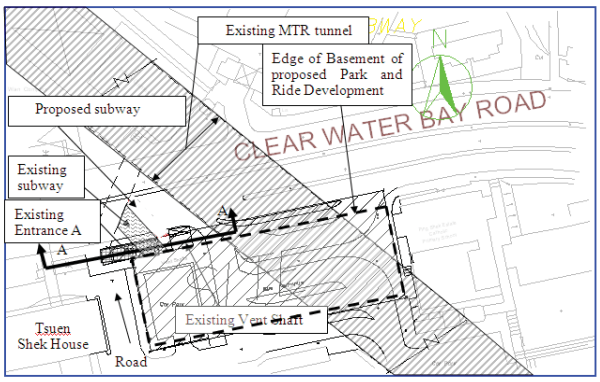


Figure 1: Plan of the new subway adit in the MTR Choi Hung Station Park and Ride Development

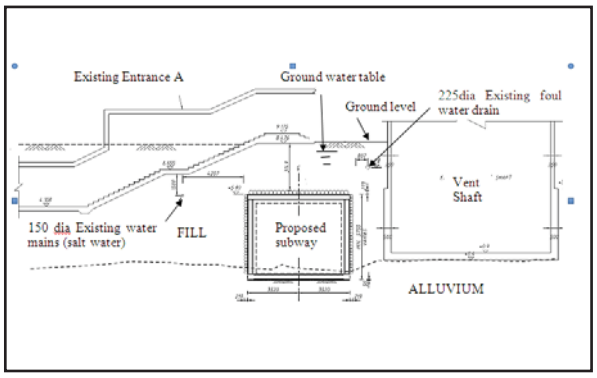


Figure 2: Cross-section A-A showing the subway adit and its adjacent structures

1.2 Geological conditions

Ground investigation consisting of 13 nos. of boreholes was carried out for the design of the tunnel excavation works. A cross-section (A-A) through the existing entrance A, the existing vent shaft, and the proposed subway is shown in Figure 2. The ground is covered by about 9 m of fill, and is underlain by 5 m of alluvium, 5 m of completely decomposed granite (CDG) and finally bedrock in sequential order. The groundwater varied from 1 m to 3 m below the existing ground level. The excavation for the subway mainly involved excavation in fill and the alluvium below the groundwater level.

1.3 Site constraints

The sensitive receivers in the vicinity of the site are listed as below, and are also shown in Figure 1.

(a) existing residential building - Tsuen Shek House;
(b) underground utilities such as fresh and salt water mains were located above the proposed subway;
(c) existing MTR tunnel, the shortest distance between it and the proposed subway was only 4.5m;
(d) vent shaft, an on-grade structure, located at 1.1 - 2.5 m from the edge of the proposed subway;
(e) MTR Entrance A, an on-grade structure, with its staircase approximately 3.7m above, and
(f) existing subway to which the proposed subway would connect.

Table 1: Summary of design soil parameters

Soil type	Density (kN/m^3)	Friction angle (degree)	Cohesion (kPa)	Young's modulus (MPa)
Fill	19	35	0	15
Alluvium	19	35	0	25
CDG	19	37	2	50

2 DESIGN AND CONSTRUCTION

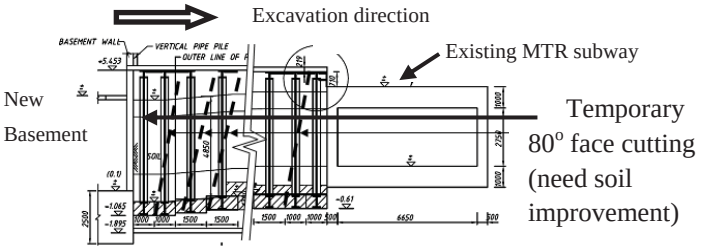
2.1 Design

Horizontal pipe pile wall was proposed as the lateral support system to the excavation. Accuracy of alignment was important due to the shallow soil cover above the adit and the proximity to existing utilities. Due to the oblique angles at intersection between the existing tunnel and the proposed tunnel, and at intersection between the proposed tunnel and the new basement respectively, the pipe piles have different lengths. It was important to ensure that the installation of the pipe piles would not cause any adverse effects to the existing subway. Therefore, directional drilling was adopted for the installation of some pipe piles i.e. at every 5 nos. of pipe pile. The remaining pipe piles were installed without piloting system but with an interlock connected to the installed pipe piles so that the alignment could follow the pipe piles which had been installed by more precise directional drilling. Small slots were pre-fabricated on the pipe piles so that the soil zone disturbed by the pipe pile installation could be strengthened by pressurized grouting from inside the pipe piles. Steel portal frames at 1.5 m c/c with a series of 80 degree temporary cut slopes were constructed to facilitate the tunnel excavation. The whole excavation was larger than the permanent structure to allow sufficient working space for the construction of the permanent structure; at least 700 mm clearance was allowed outside the outer edge of the permanent structure. Typical longitudinal section is shown in Figure 3. Ground improvement by Tube-a-Manchette (TAM) grouting was carried out. The major objective of the grouting operation around the excavation zone was to prevent water ingress, and to strengthen the soil mass to be excavated so as to ensure the stability of the cutting slope. Through grouting, the voids in the fill material were sealed up thereby preventing water flow into the excavation which could cause water drawdown outside the excavation of the connecting subway.

In 2003, 3D computer modeling was not commonly used for design purposes due to the high demand for computational resources. In order to simulate the 3D effect due to tunnel excavation, both transverse and longitudinal sections of the excavation were modelled by FLAC, which is a 2D finite difference model (see Figures 4(a) to (b)). The Mohr-Coulomb model was adopted to define plastic yielding criteria of the soil mass. Since the same Young's modulus (E) was used for the soil in both loading and unloading conditions, significant soil swelling/heaving could be resulted at the base of the excavation when excavation was mimicked in the analysis i.e. the whole frame and the ground surface would heave together. This would in

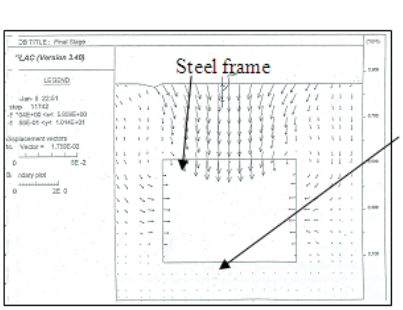
turn underestimate the ground movement and affect the planning of mitigation measures with respect to ground settlement. To overcome the problem, a simplified model with a fixed base boundary at the bottom of tunnel was used. This modeling approach neglected the swelling effect at the base of the excavation so that the frame would not heave and the ground settlement would be slightly over-estimated. The over-estimation was considered to be on the conservative side and the planning of the mitigation measures for the ground movement would also be on the safe side. On the other hand, another model without any base fixity was also set up so that the swelling effect due to excavation could be quantified. The loadings exerted on the frame were chosen from the envelope of the two aforementioned models; the design values were taken as the larger one between the two models at different locations. Besides the estimation of settlement and the determination of loadings on the frame, stability analyses were also carried out for the temporary cut slopes. A computer model, SLOPEW, which is based on limit equilibrium approach, was adopted for the checking the factor of safety (FOS). The shear strength of the soil was characterized by the Mohr-Coulomb model and the Morgenstern-Price method was adopted to calculate the FOS. A surcharge of 20 kPa at the ground surface was assumed to simulate the heavy traffic loading, and the groundwater table was assumed to be at 1m below ground. In view of the high consequence-to-life (CTL) associated with slope failure, a FOS of 1.4 was adopted as the minimum requirement in the slope stability assessment. It was however found that the FOS of the proposed cut slopes was less than 1.4. The ground therefore had to be improved prior to tunneling which led to the grouting operation as described above.

Due to the requirement that the operation of the existing MTR subway had to be maintained, the breakthrough of the tunnel could only be carried out after the whole new subway was constructed. The busy road above the adit had avoided the construction of a receiving pit, i.e. the excavation could only be carried out in one direction. The gap between the new and the existing tunnel was stabilized by grouting carried out from existing subway only. In other words, the pipe piles were supported by steel frames at one end and the other end was free hanging in soil prior to tunnel excavation. Tunnel excavation therefore required the support from the temporary cutting slope and the installed frame throughout the excavation process. Therefore, analyses for longitudinal sections were set up to model the staged excavation and sequential installation of portal frames. Similar to the analysis for transverse sections, two models were set up with different fixities at the bottom of the tunnel. In addition to overall stability, flotation was also a concern. The drawdown of groundwater more than 1m was not allowed to avoid any consolidation settlement which could adversely affect the sensitive receivers. Therefore, water pressure underneath the excavation was high. The stability against flotation was controlled by the thickness of grouting and the soil cover above excavation zone and the side friction of the pipe pile. The minimum grouting thickness at the bottom was then determined.

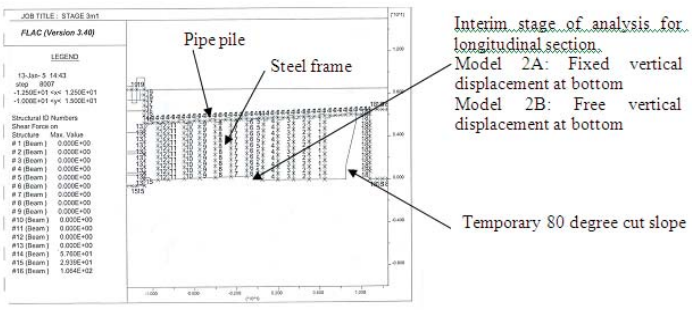


Member Type	Member Size
Top Beam	356x308x177 kg/m UC
Side Column	305x305x137 kg/m UC
Bottom Beam	356x308x177 kg/m UC
Pipe pile	219mm diameter, 6mm thick

Figure 3: Typical longitudinal section - Temporary support arrangement



(a) Typical transverse section (Model 1A and 1B)



(b) Typical longitudinal section (Model 2A and Model 2B)

Figure 4: FLAC analyses models

2.2 Construction

Construction of the tunnel was carried out from 2003 to 2005. Permeation grouting, which is a technique in which the pore fluid is replaced with grout injected at a steady injection pressure and therefore would not disturb the soil structure, was adopted. Hydraulic drilling rigs with a top hammer drilled with temporary steel casing had proved to be necessary to penetrate the onerous ground conditions to permit the installation of the grout tubes. 50mm Tubes-a-manchette (TAM) were adopted to be installed within the cased holes and then low strength bentonite cement grout (sleeve grout) was pumped into the hole while the casing was extracted. TAM grouting is commonly used for injection in soil. They have the advantage that each injection point can be re-grouted. TAM grouting also allows the injection volume and pressure at numerous injection points to be monitored and controlled. Injection of grouts was performed through TAM. Double packers were positioned on each side of the injection point and inflated to form a seal. Grout was injected into the isolated section of the TAM tube and pressure increased until the grout broke through the sleeve grout and injected into the ground. Grouting was executed in two phases. In the first phase, bentonite cement (B/C) grout was used to fill the larger voids and to 'repair' any damage caused to the ground during the drilling activities. Bentonite cement grout is the most common grout to be used because of its comparative low price and good supply. Chemical grouting was then conducted in the second phase through the same grout holes to fill the remaining small voids in soil. The chemical grout radiated away from the injections points and improved the strength of the treated soil and reducing the permeability. Sodium silicate and 600C hardener were adopted to form the chemical grout which had been widely used in the previous MTR Station projects. 600 C hardeners are methyl and/or ethyl diesters formed from the action of aliphatic diacides mixtures on methanol and/or ethanol. Sodium silicate has to be mixed with a reagent 600C hardener for it to gel. The choice of the reagent and the proportion in which it is mixed with the sodium silicate has a significant effect on the viscosity evolution and the strength of the resulting grout. The target gelling time of the silicate grout was 50 – 90 min. Grouting would be stopped when the design grout intake volume had been injected, or the specified pressure was achieved, or the grout broke through to the ground surface, or excessive surface heave was recorded, or any damage to surroundings was observed. The spacing between grout holes depended on the grout pressure and the ground geology. Originally, 1.2m c/c grout holes, i.e. clearance was about 1 m, were proposed during the design stage. It was then reviewed to adopt a spacing of 750mm c/c in general. The performance of the grout was verified by carrying out trials at different locations with different pressures and locations to ensure that the grouting parameters were suitable to this project. After the grouting parameters were confirmed (B/C grout: (Bentonite: Cement: Water = 2.5 kg: 90 kg :1 20 L) and chemical grout: (Water: Sodium Silicate: 600C hardener = 111 L : 80 L : 9 L), grouting was carried out for the whole tunnel alignment. Inclined grouting was also carried out at the bottom of the tunnel to achieve the required grout zone thickness determined from floatation checking. Actual performance of the grouting was also verified after the completion of the grouting operation prior to bulk excavation. The key parameters adopted in the assessment are shown in Table 3 (reference is made to Karol, 2003). It is not uncommon that the actual soil parameters may deviate from the design values. Sensitivity study was carried out to generate alternative acceptance criteria in advance (see Table 4).

Comprehensive monitoring composed of piezometers, ground settlement markers, building settlement markers, vibration monitoring points was provided. Slight ground heave (maximum was 32mm) was observed during the grouting operation and B/C generally caused more heaving than chemical grout. The monitoring results indicated that the ground was being compressed by the pressurized grout. Hydrofracture may occur during permeation grouting if the injection pressures were too high and could lead to ground heave and affect the stability of adjacent structures or services. Therefore, inspection to the utilities nearby was carried out. No damage to the utilities was noted. Local resurfacing at the road was carried out to avoid disturbance to the road operation. Pumping rates were limited to 8-10 l/min at the top two rows of the grout holes and 10-12 l/min at the other lower rows. The pumping rates were limited to avoid the development of high water pressures within the ground. In view of the B/C grouting having more significant effect on the ground heave, the target grout intake volume of the B/C grout was reduced from 120 l/m to 60 l/m. The reduced grout intake volume was compensated by the same amount of the chemical grout and therefore the target grout intake volume of the chemical grout was increased from 180 l/m to 240 l/m. The frequency of the monitoring on settlement checkpoints was increased to one-hour intervals. In case either +/- 5mm difference in one checkpoint by two successive readings or the accumulated difference in the checkpoints greater than +/- 20mm or the leakage of grout on the ground surface was observed, the grouting works would be suspended

and reviewed. After considering the monitoring records and the grouting assessment, excavation could then be carried out.

Table 2: Summary of design models

Model	Fix vertical displacement at base	Determine cumulative effect due to stage excavation longitudinally	Determine loading	Determine maximum settlement
1A (transverse)	Y		Y	Y
1B (transverse)			Y	
2A (longitudinal)	Y	Y	Y	Y
2B (longitudinal)		Y	Y	

Table 3: Summary of key parameters in the assessment of trial grout

Description of parameters	Objective	How to obtain
Uniaxial compressive strength (UCS)	To ensure minimum strength of the treated soil mass required for stability is achieved	Laboratory testing-UCS
Comparison between target* and actual volume intake	To evaluate extent of grouting into soil mass	-Target volume can be estimated based on the soil properties* -Actual measured grout volume intake
Effectiveness of penetration of grout to the soil mass	Counter-check on the penetration of grouting	Spray of chemical indicator- simple measure to assess if grouting penetrate to the soil mass
Bulk density	Another evidence to assess if soil mass had been grouted	Laboratory test
Young's modulus (E)	Critical for settlement review	Pressuremeter test; Laboratory test
Pressure	To verify if there is any underground path which causes significant grout loss	From grouting record
Permeability	To verify effectiveness against water seepage	Comparison of the permeability test results of the soil before and after the grouting works. Should the permeability of soil be decreased by more than 10times of the original after grouting works, the grouting works on improving the ability on water inflow will be treated as successful

Note: * The design grout intake volume was determined based on the equation introduced in Baker (1982), V_z (n x F) (1 + L), where V_z is the total volume of the treatment zone; n is the soil porosity; F is the void filling factor and L is the grout loss factor. The porosity of the existing fill materials varied from 0.3 to 0.4. The void filling factor varied from 0.85 to 1.0 depending on various factors such as the grading of the soil, grouting sequence and the injection pressure, etc. The grout loss factor varied from 0.05 to 0.15, depending upon the shape of the grouted zone, the frequency of injection points per unit volume, and the presence of highly porous layers in the soils. The design grout intake volume was therefore determined within 0.3 V_z and 0.4 V_z.

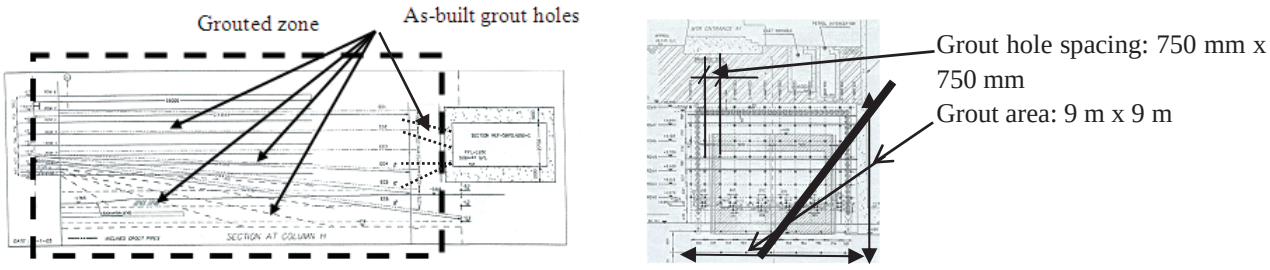


Figure 5: Actual grout zone for the excavation work

A total of 70 numbers of steel pipe piles were installed to form the pipe pile roof structure. Optical piloting system was adopted in this project. Since it was an innovative idea to adopt directional drilling to control the alignment of horizontal pipe piles, trial installation of HPP, both on site and off site, were carried out to verify the suitability of the piloting system prior to the installation of other HPP by directional method. Results showed that the piloting system was appropriate.

Table 4: Summary of results under different assessment criteria

Description	Original	Alternative	Actual
Unit weight, γ (kN/m^3)	24	20	>20
Young's modulus, E (MPa)	200	100	94*
Friction angle, ϕ, of treated soil(°)	45	35 (assuming no change due to grouting)	>35
Soil cohesion, c (kPa)	500	50	>50(correlated by UCS)
Predicted settlement at ground surface (mm)	12.3	13.42	18
Factor of safety of slope at cutting face	>5	1.44	Stable

Note:* Additional analysis based on the actual data of the slightly reduced E-value, 94MPa, was carried out. It confirmed that the displacement was also similar to alternative criteria and was considered to be still acceptable.

After the performance of grouting was found satisfactory, excavation began. It took 60 days to complete all the 20 cut-and-support cycles for the 28m long subway. In general each cut-and-support cycle was completed within 3 days. The success of the subway excavation relied on the effective functioning of the grouted zone. No significant seepage was noted. The cutting face showed a good self-standing ability. The grouted soil was very firm that it required the excavator mounted with hydraulic breaker for breaking it down and excavation. Frequently, horizontally distributed cement grout intrusions were seen. A maximum settlement of 18mm was observed during the excavation which was in a similar order of the predicted value. Taking into account the ground heave during grouting, the cumulative ground movement was 18mm settlement to 19mm heaving. Regular inspections were conducted continuously. The movement of the existing MTR subway and the vent shaft were monitored on a daily-basis. Vibration monitoring was also performed during the modification work of the existing MTR subway. All the results were within tolerable limits of sensitive receivers.

3 CONCLUSIONS

The excavation works associated with the MTR Choi Hung Station Park and Ride Development was completed without any significant adverse effects to the surroundings during excavation. The key to the success is summarized as follows:

(a) Successful trial grouting and the use of two phases grouting - first with bentonite cement grout and secondly chemical grout - can improve the properties of the treated soil effectively through reduction in permeability and an increase in strength;
(b) Sensitivity analysis to set up alternative acceptance criterion;
(c) Timely review of the design assumptions;
(d) Successful installation of horizontal pipe piles by directional drilling;
(e) Supervision by competent person with detailed records of observations of each phase of the grouting operation for continuous review of the design assumptions;
(f) Comprehensive assessment criteria were set up to assess the performance of grouting;
(g) Prior to the commencement of the works, condition survey was carried out to record the conditions at ground surface, structures and utilities in the vicinity for the assessment of the impact to surroundings.
(h) Comprehensive monitoring with appropriate action plans was set up and implemented at the right time.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude for MTR Corporation Ltd. and Rich Resource Development for the permission to publish this technical paper. The kind supports from Ir. Calvin TH Cheong, Mr. Ricky YS Lee and Dr. Johnny Cheuk are grateful acknowledged.

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Influence of Utilities for Cut-and-Cover Tunnelling Works

Tony Cheung & Ryan Mo
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

Hong Kong has already installed extensive utilities networks in urban areas. Currently, underground infrastructure developments such as metro, railway and vehicular access will affect or be affected by existing utilities networks. Cut-and-cover tunnel and station construction methods are adopted commonly to enhance the transportation networks. The influence of utilities on cut-and-cover tunneling works shall be considered during the planning, design and construction stages.

One of the metro development project for construction of cut-and-cover tunnel required excavation to depth about 20m below ground level. A number of existing utilities are located within footprint of the cut-and-cover tunnel near busy truck road, such as water mains, box culvert, sewerage & stormwater mains, gas mains etc. All of these utilities were required to remain operational during the construction. This cut-and-cover tunnel project involved the construction of about one thousand meters long cut-and-cover tunnel.

This paper presents the geotechnical design principle, construction sequence & consideration, monitoring system and project management of the cut-and-cover tunneling works influenced by extensive utilities networks.

1 INTRODUCTION

Utilities and transportation networks are essential elements for the development of urban city. Hong Kong is well-developed urban city with extensive utilities networks. Currently, underground infrastructure developments such as metro, railway and vehicular access will affect or be affected by existing utilities networks.

Cut-and-cover tunnelling works are selected as one of the construction methods for tunnel and station of metro & railway networks. Most of them are so-called “fast-track project”. It implies utilities diversion option may not be a best solution due to the relative longer construction time for utilities diversion works. This makes geotechnical engineers play a challenging role in the design & construction of cut-and-cover tunneling works for metro development in last decade. This cut-and-cover tunnel project involved the construction of about one thousand meters long cut-and-cover tunnel in urban area.

This paper presents the geotechnical design principle, construction sequence & considerations, monitoring requirements and project management of the cut-and-cover tunneling works influenced by extensive utilities networks in Sections 4 and 5 through two case studies of the cut-and-cover tunnel project. It also provides discussion of relevant findings & recommendations to Employers, Engineers, Contractors and utilities owners involved in cut-and-cover tunnelling works for the electing and carefully implementing in the project.

2 PROJECT DESCRIPTION

About one thousand meters long cut-and-cover tunnel project located in close proximity to the existing structures & utilities was constructed at an urban area of West Kowloon. In order to deal with specific problems in the execution of the project, different types of temporary works were adopted to suit the site constraints and construction methodology. Cut-and-cover tunnel were constructed by conventional cofferdam walls, such as sheet piles, pipe piles and diaphragm walls, together with adoption of utilities diversion works, and supporting in-situ works on existing utilities.

The typical cut-and-cover construction of the southern end of the project and located close proximity of busy road required excavation to depth about 20m below ground level. This area is generally covered with FILL, Marine Deposit, Alluvium overlaying the completely decomposed granite and bedrock. The saprolite overlies the rockhead which is predominantly completely decomposed granite. The thickness of the predominantly completely decomposed granite saprolite varies from about 5 m to at least 30 m. The bedrock was encountered at depths of up to –50 mPD.

A number of existing utilities are located within footprint of the cut-and-cover tunnel, such as (1) 1200 mm diameter water main, (2), 800 mm diameter water main, (3) 600 mm saltwater main, (4) 600 mm cooling main, (5) 4.8 m x 4 m box culvert, (6) 1800 mm diameter stormwater main, (7) 1350 mm diameter sewerage main, and (8) electricity cable, telepcom cables, gas mains.

All of these utilities were required to remain operational during the construction of cut-and-cover tunnel. A tremendous design and construction challenges were tackled. The alignments of the captioned utilities are shown in Figure 1**.**

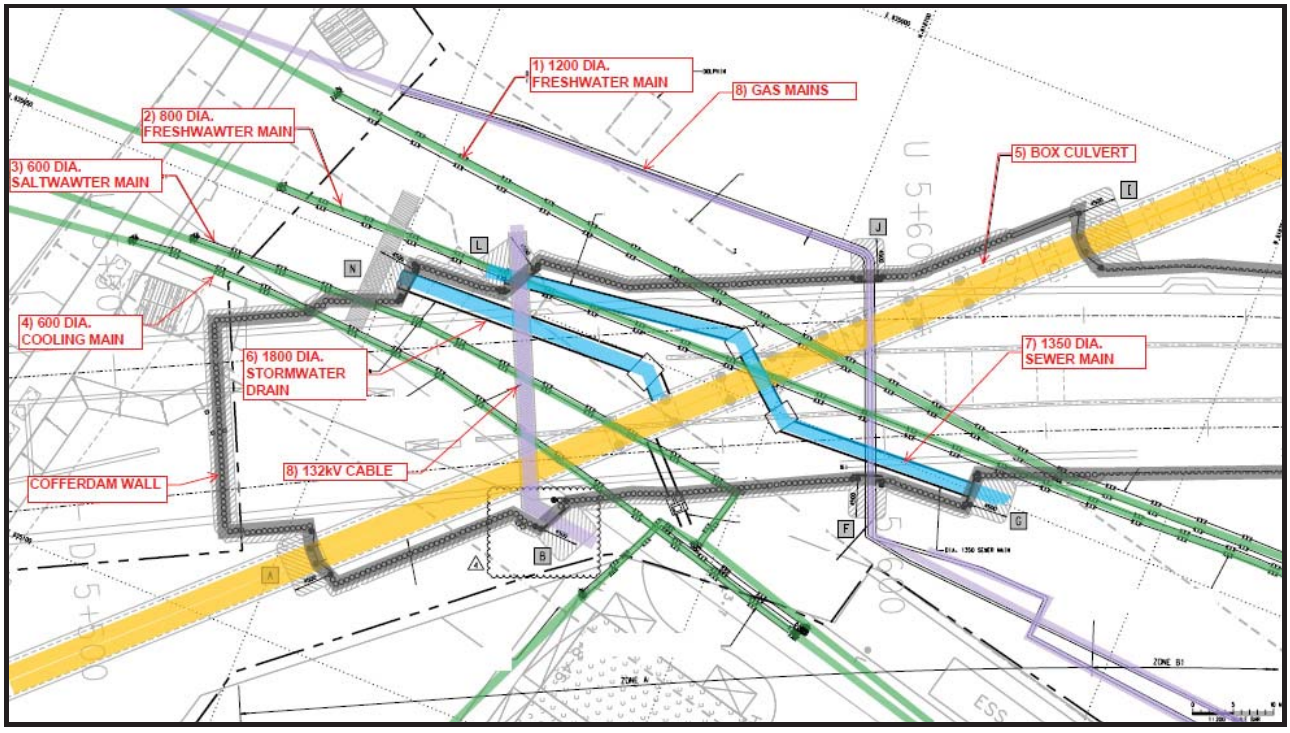


Figure 1: Alignment of utilities within cofferdam

Temporary walls including pipe pile walls and sheet pile walls were adopted as lateral support for this cofferdam excavation area. Steel struts and walings would be installed at regular intervals to provide lateral support to the temporary walls. However, pipe pile wall was not contiguous; water seepage has to be handled by grouting treatment at the back of cofferdam walls. In additional, grouting treatment at the utilities gap openings of cofferdam walls were used to provide water cut-off and soil strengthen purpose.

Utilities diversion works of water carrying services were adopted to remove their impacts during cofferdam excavation works. Discussion is presented in Section 4.

Owing to the existing utilities intersection with cofferdam wall, nine openings were presented in the cofferdam wall alignment. Strengthening soldier piles and grouting treatment were adopted in these openings. Works for openings of sewerage box culvert is discussed as case study in Section 5.

3 INITIAL CONSIDERATION

Utilities networks in the vicinity or intersection of cut-and-cover tunneling works and restriction of their deformations can be the most risky construction operations undertaken. In addition, there are important aspects in design cut-and-cover tunnel in the vicinity and intersect with existing utilities, which shall be

considered during planning, design, and construction stages, i.e. deformation limit, method of construction, construction duration. In general, the decision-makers of tunnel alignment will select the best option for construction in respect to the construction program and land issues. However, it will pose a number of geotechnical challenges and require geotechnical engineers to review and implement in construction stage.

However, most of the geotechnical constraints on cut-and-cover tunneling works can be mitigated in the initial stage of the project. Cooperation with different parties will help these constraints of existing utilities to be mitigated and removed. Before deciding on any temporary works for cut-and-cover tunnelling works, it is essential that the required deformation values of utilities should be considered. In addition, methods of construction are left largely to the Contractor. This should not, however, prevent the contract requirement from prescribing a particular type of construction scheme if it is clear from a risk, safety and quality point of view that this would benefit the project as whole.

4 UTILITIES DIVERSION

In order to mitigate the construction risk on the existing water carrying pipes (1) 1200 mm diameter water main, (2) 800 mm diameter water main, (3) 600 mm saltwater main, and (4) 600 mm cooling main, review on the possibility of the diversion option instead of support in-situ option was carried out in the early stage of construction works. After close liaison with Water Services Department (WSD), clients and contractor, diversion option to the water carrrying pipes was adopted instead of supporting in-situ option. In addition, a big bend box (approximate 4 m x 4 m in area) connected with the existing 1200 mm and 800 mm water mains was located within the footprint of cofferdam. Effects on the operation and construction vibrations were considered in temporary works design. The diverted alignments of the water mains are away from the cofferdam excavation and shown in Figure 2.

Cooperation with different parties could remove the utilities constraints on cut-and-cover tunnelling works in initial stage of construction.

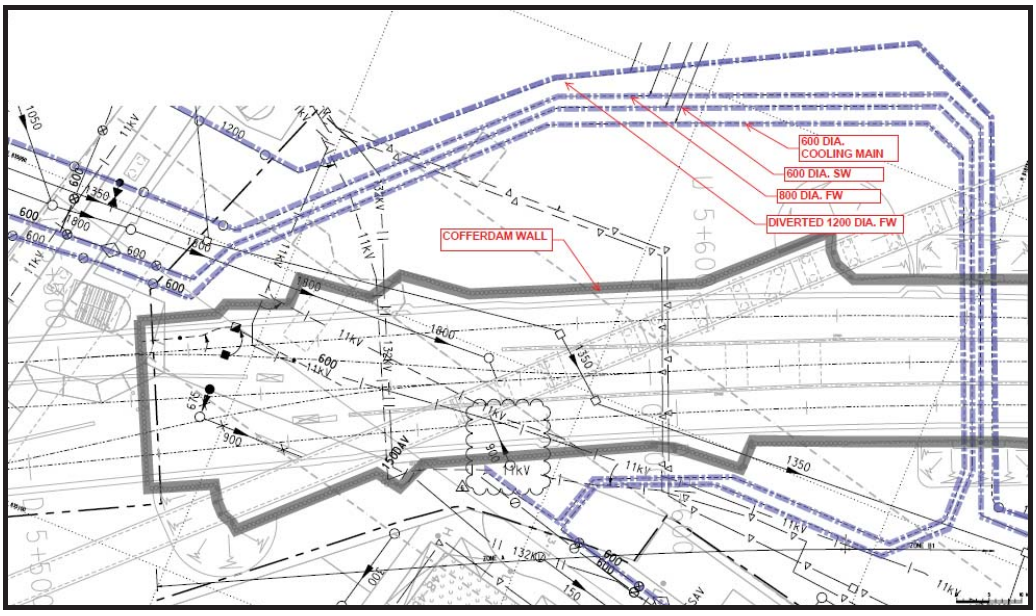


Figure 2: Utilities Diversion for Water Mains

5 TEMPORARY WORKS FOR BOX CULVERT

As shown in Figure 1, there were many utilities gap openings along the alignment of cofferdam wall. Apart from the diversion works on four water carrying pipes, eight numbers of utilities gap openings were required to consider in cofferdam wall design. Owing to the width of openings were ranged from 3.5 to 5.8 m, cofferdam walls were strengthened by soldier piles at the two sides of openings, and grouting treatment were adopted to provide water cut-off purpose.

The largest width openings of cofferdam wall were in the box culvert. Typical soldier piles system and grouting treatment at the edge of opening cannot provide adequate supporting system. In order to minimize the adverse effect on the existing box culvert during cofferdam excavation works, two additional soldier piles were constructed within the footprint of box culvert to enhance the wall stiffness, together with steel waling and strut were provided to support the soldier piles. In addition, a temporary supporting system was used & separate with the cofferdam wall.

After close liaison with Drainage Services Department (DSD), two soldier piles were constructed underneath the existing box culvert, and installation works was agreed to be proceed during dry season only. Schematic arrangement is shown in Figures 3, 4 and 5.

The construction sequence is as follows:

- Expose the top slab of box culvert for installation of 2 soldier piles (see Figure 4);
- Install 4 number soldier pile (2 at the middle of the opening, and 2 at sides of box culvert);
- Grouting treatment behind the soldier pipe to form grout curtain for water cut-off purpose;
- Excavate and install lagging wall in 500 mm spacing. Excavate and install waling struts stage by stage, and down to final excavation level.

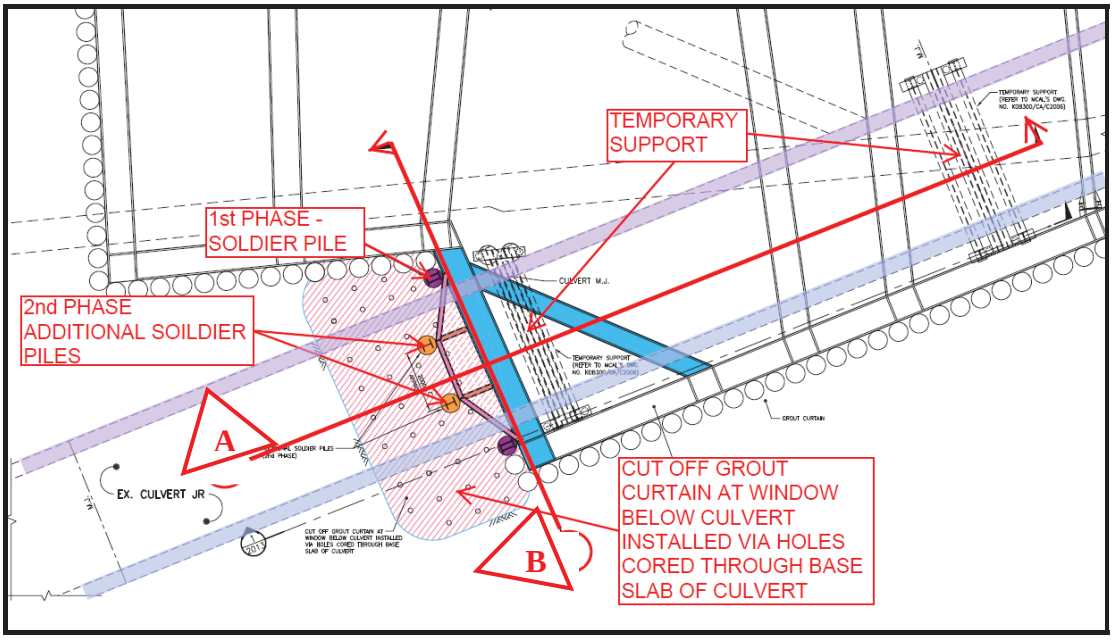


Figure 3: Temporary works arrangement for utilities window opening of box culvert
Note: Sections A and B are shown in Figures 4 and 5 respectively.

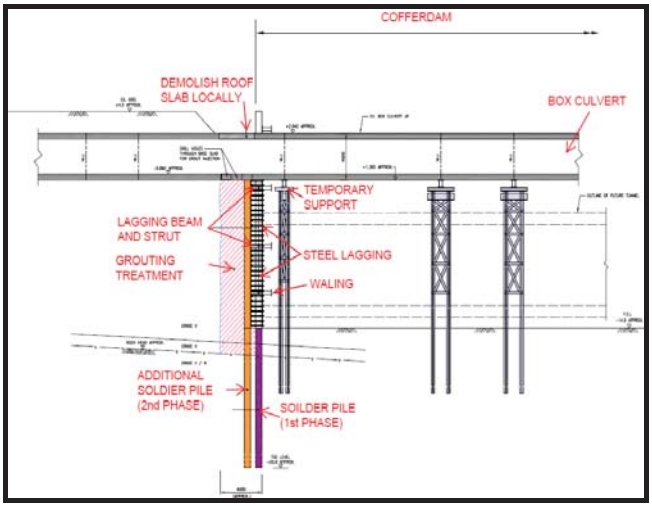
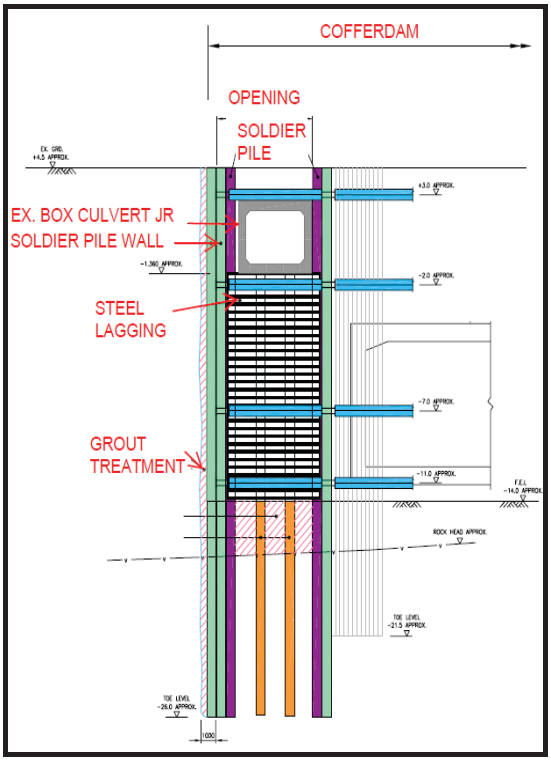


Figure 4: Temporary works arrangement for utilities window opening - Section A



Figures 5: Temporary works arrangement for utilities window opening - Section B

6 ALLOWABLE DEFORMATION VALUES FOR UTILITIES

The allowable deformation limits of existing utilities are generally based on agreement from the owners of utilities. The structural limit of the utilities or facilities should be examined to determine the allowable deformation values of utilities. Owning to there is no historical deformation records of utilities, the strictly allowable deformation limits were adopted. For this project, the allowable deformation values are shown in Table 1.

Table 1: Allowable deformations for utilities

Utilities	Allowable Deformation Value (mm)
Water Mains	5
Box Culvert	20
1800mm diameter Stormwater Main & 1350mm diameter Sewerage Main	20
Gas Main (steel pipe)	25
Gas Main (PE pipe –HDPE)	50
Electricity Cable with Cable Trench	25
Other Utilities (Flexible)	100

A comprehensive instrumentation and monitoring plan are required to ensure the works to be carried out safety and to monitor the possible deformations of utilities during cut-and-cover tunnelling works.

Based on the stress condition of the pipeline or facility of existing utilities, the allowable deformation of utilizes can be reviewed and set to reasonable value. Most of them are buried and not allowed for inspection until the exposure during the construction stage. For captioned diverted portion water mains, allowable 75mm deformation value was agreed as the allowable deformation value as the stress condition of newly installation utilities can be monitored in subsequent construction. Thus, the allowable deformation value can be increase to a reasonable value.

As it is impossible to carry out the review and assessment on stress condition of utilities in the planning and design stages. The review and assessment processes shall be carried out in the early stage of construction.

It is highly recommended to review the stress condition of the utilities pipes, especially for the water carrying pipes in the early stage of construction.

7 CONCLUSIONS

The construction method of cut-and-cover tunneling works in associated with the influence of existing utilities indicates that great benefits can be achieved if review of existing utilities is carried out at the early stage of the planning process of the project, by avoiding high risk on construction works, and by selecting an appropriate alignment and method of construction with optimum design options. It is suggested that geotechnical input for planning of tunneling project should be carried out and implement in construction stage.

The review would identify and assess the impacts of existing utilities that could influence a tunnelling project. In addition, partnership arrangements involving client, contractor, engineer and utilities undertakers are particularly beneficial when the partnership is engaged in cut-and-cover tunneling works, as it means that the influence of utilities constraints can be solved or removed adequately.

Experience Sharing for Micro-tunnelling Projects Implemented by CLP Power

Alan N.L. Wong
CLP Power Hong Kong Limited
W.Y. Wong
Fugro (Hong Kong) Limited

ABSTRACT

CLP Power Hong Kong Limited (CLP) employs micro-tunnelling technique to install underground electricity cables at strategic locations where there are overall economic and engineering incentive to the electricity infrastructure projects as well as mitigating disturbance to the public. The role of the Designer and the Independent Checking Engineer (ICE) are crucial in these works, and therefore requirements are stipulated in the statutory control system entitled "Control of Trenchless Works by Utility Undertakers affecting Public Roads" jointly drawn up by the Highways Department and the Geotechnical Engineering Office.

Since the majority of these trenchless works[(1)] projects will be carried out underneath public roads, the project team needs to ensure a safe and smooth implementation of the trenchless works to avoid imposing risks to working personnel or causing undue disturbances to the public. In this connection, CLP has developed a comprehensive Safety, Health, Environmental and Quality (SHEQ) management system for trenchless works to guide the professionals in properly executing their specific tasks.

This paper depicts the works by ICE from a consultant firm in proper checking of the engineering process, and outlines the above-mentioned SHEQ system developed by CLP as a client in order to ensure safe execution of all its projects. Example of a completed project is included to outline the SHEQ system for enhancing safety management and risk assessment for the works.

1 INTRODUCTION

In order to avoid causing potential inconvenience or disturbances to the public due to trench excavation works in densely trafficked areas or to overcome site constraints, e.g. electricity cables to be installed below and across a nullah, CLP has adopted micro-tunnelling techniques over a decade for installing underground electricity cables in some of their cable network projects in Hong Kong. Depending on the geological conditions and constraints of individual sites, different micro-tunnelling techniques, including slurry-operated tunnel boring machines (TBM), horizontal directional drilling (HDD), hand-shield tunnels, hand-dug tunnels supported by steel frames and horizontal pipe piles, have been adopted by CLP.

Since May 2008, a statutory control system entitled "Control of Trenchless Works by Utility Undertakers affecting Public Roads" (hereinafter called "Control System"), jointly drawn up by the Highways Department (HyD) and the Geotechnical Engineering Office (GEO), has been exercising over the design and supervision of trenchless works. Design proposal of trenchless works satisfying Control System, prepared by a Designer and certified by an ICE, is to be submitted to the HyD for approval before commencement of relevant construction works. The organization of Control System is as shown in Figure 1.

(1) Excavation within unallocated government land and affecting public roads, under public roads, and/or within a horizontal distance from public roads equivalent to the ground cover of the works refers as "trenchless works".

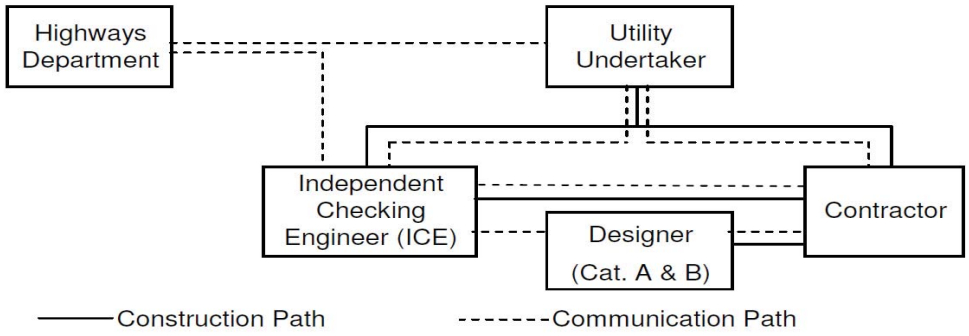


Figure 1: Organization of Control System

This paper depicts the works undertaken by the ICE, and introduces a Safety, Health, Environmental and Quality (SHEQ) management system developed by CLP in implementing their trenchless works projects. An example of a completed trenchless works project by tunnel boring machine (TBM) method in the Kai Tak Nullah and Choi Hung Road is illustrated to share the experience of the ICE as well as to highlight the effectiveness of the SHEQ system for enhancing the safety management and risk assessment for works undertaken inside a confined space environment.

2 ROLE OF INDEPENDENT CHECKING ENGINEER (ICE)

To ensure the trenchless works to be carried out in a safe and proper manner, the ICE conducts independent checking on the works, during design and construction stages. The ICE's role is to perform an independent check of the site investigation (SI), design and construction of proposed trenchless works. Apart from certifying the design, the ICE certifies the construction method statements and procedures, risk control limits and the respective mitigation measures, monitoring and site supervision plan to ensure that the proposed trenchless works are satisfactory, and meeting all relevant government requirements and standards.

During the course of trenchless works, Category A and Category B site supervisors, appointed by the Designer, supervise the works. The ICE carries out site inspections or audits the site on a regular basis to verify the content of the Category A supervisor's reports and to confirm adequacy of the design review, prepared by the Designer, and that proper risk control actions have been taken. The ICE is also responsible for ensuring that appropriate and timely actions are taken to prevent and mitigate risks to public life and property.

3 SAFETY, HEALTH, ENVIRONMENTAL AND QUALITY (SHEQ) MANAGEMENT SYSTEM

3.1 General

While the "Control System" is focusing on the safety of the construction design and implementation methodology, CLP extended the system to cover the prevention of personnel injuries and fatalities in trenchless works. A SHEQ management system which sets out a systematic process and practices for management of the safety, health, environment and quality aspects to reduce risk, to increase the operation efficiency and to minimize any adverse impacts to the environment throughout the trenchless works was thus developed in CLP. A supervision team is formed by CLP, ICE and Contractor together with the Designer to carry out the monitoring mechanism for safety control.

In order to align the safety standards of the supervision team, CLP has been arranging various safety trainings and workshops, such as Safety Leadership Training for the supervision personnel, which includes ICE, Category A site supervisor, Category B site supervisor and Contractor's site supervision staff (Plate 1). Regulatory and statutory requirements as well as the additional safety and quality expectations from CLP were presented in the training. For instance, the elements of Safe Systems of Work (SSoW) are thoroughly explained and relevant case studies are discussed (Plate 2).

Case studies for sharing the working experience in trenchless works and workshop for various micro-tunnelling techniques of trenchless works are also arranged by CLP to enhance safety standard and awareness of supervision staff, ICE, Designers, Contractor and Subcontractor frontline staffs (Plate 3).



Plate 1: Safety training

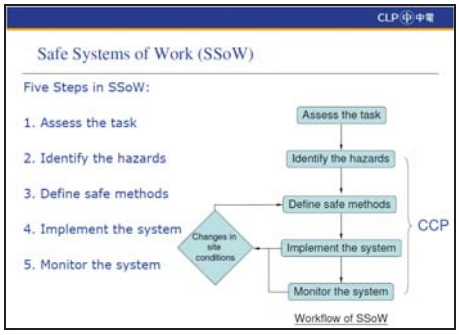


Plate 2: Safe Systems of Work

Furthermore, risk assessment training courses are also arranged by CLP on a regular basis to enhance the supervision team's risk management ability. Through the training course, the supervision team has acquired a better understanding of risk management and is equipped with practical knowledge of the tools and techniques for hazard identification.



Plate 3: Workshop for trenchless works

3.2 Critical Control Point (CCP) for micro-tunnelling projects

In view of high risk activities associated with trenchless works, CLP has raised a number of control measures on some high-risk activities associated with trenchless works to supports the implementation of the SSoW for trenchless work. A list of Critical Control Points (CCPs) has been established to identify the hazards, define safe working methods, implement the methods and monitor the safe working methods. The CCP adopts a systematic and preventive approach to control and eliminate the risks of identified potential hazards. The CCP gives specific details on how to perform the work-related tasks and outlines the parties to be responsible for the necessary tasks. Samples of CCP under the SHEQ system are shown in Table 1.

Table 1: Sample of Critical Control Point for micro-tunnelling works under the SHEQ system

CCP Elements	**CLP / ICE**	**Contractor and Designer Staffs**	**Workers**
01: Site Layout Plan	A / M	S / M	I
02: Lifting Operation	A / M	S / M	I
04: Site Specific Risk Evaluation	A / M	S / M	I
07: Confined Space Criteria	M	M / I	I
08: Safety Drill	A / M	M / I	I
09: Ground Treatment	M	S / M	I
13: Shelter for Lifting Operation	A / M	S / M	I
15: Trial Hole on Sheetpile		M	I
16: Settlement Monitoring	M	S / M	I
17: Preventive Measures on Sleeve Pipe Backward Movement	A / M	S / M	I

Legend: S = Submission, I = Implementation, A = Approval, M = Monitoring

4 CASE STUDY IN IMPLEMENTATION OF CCP IN MICRO-TUNNELLING WORKS CROSSING KAI TAK NULLAH AND CHOI HUNG ROAD

This trenchless project was to facilitate the installation of new 132kV cable circuits running across the Kai Tak Nullah and Choi Hung Road. The works comprised the construction of a 103-m long, 1650-mm diameter concrete sleeve pipe jacked across the strategic crossing location by TBM method. And two temporary working pits were constructed as a jacking pit and a receiving pit for tunnelling works. Locations of pits and pipe jacking alignment are shown in Figure 2.

The depth of jacking pipe was about 13.3 metres below ground. The ground conditions generally consisted of fill underlain by alluvium and the groundwater table was at approximately 2.3 metres below ground. Working pits construction and the jacking pipe were mainly in alluvium (Figure 3).

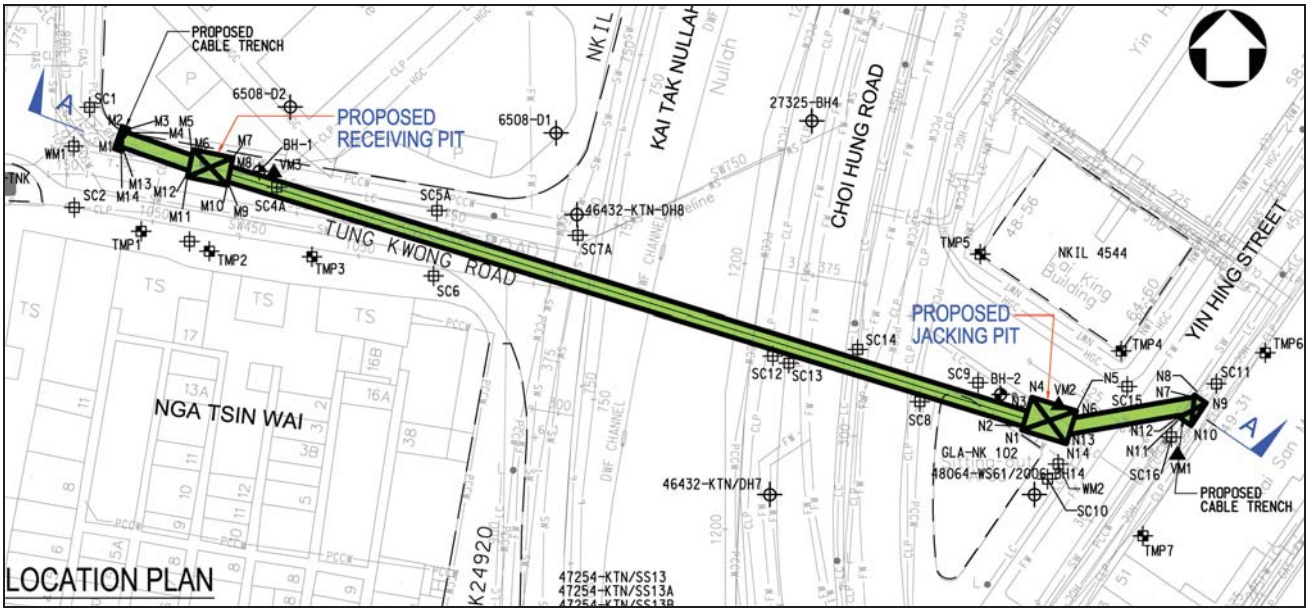


Figure 2: Location of pits and alignment of jacking pipe

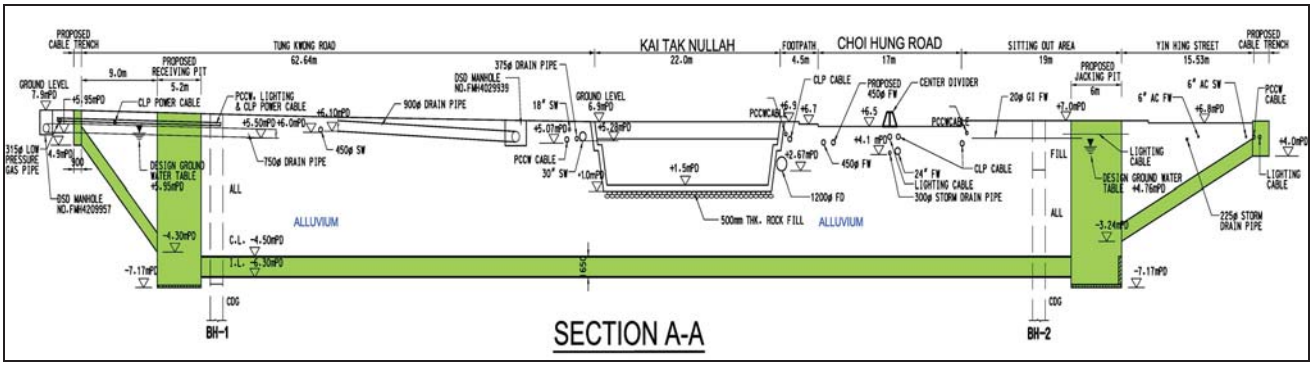
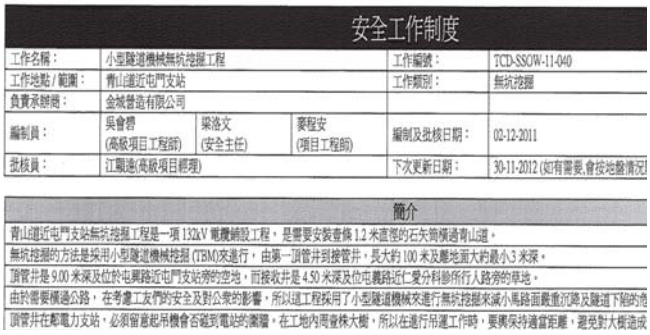


Figure 3: Longitudinal section of jacking pipe

In order to minimize the hazards arising out of the works, risk evaluation was to be site specific (Plate 4). Also, trenchless works usually involved working inside confined space environment. The trenchless works, therefore, was carried out in compliance with proper confined space procedure. Safety drills at working pits and inside the jacking pipe were conducted on a regular basis to enhance the safety awareness of the workers (Plate 5).



安全工作制度

工作名稱：	小型隧道機械無坑挖掘工程			工作編號：	TCD-SSOW-11-040
工作地點 / 範圍：	青山道近屯門支站			工作類別：	無坑挖掘
負責承辦商：	金城營造有限公司				
編制員：	吳會禮 (高級項目工程師)	梁洛文 (安全主任)	麥程安 (項目工程師)	編制及批核日期：	02-12-2011
批核員：	江駿逸(高級項目經理)			下次更新日期：	30-11-2012 (如有需要,會按地盤情況)

簡介

青山道近屯門支站無坑挖掘工程是一項 132kV 電纜鋪設工程，是需要安裝直徑 1.2 米直徑的石矢筒橫過青山道。

無坑挖掘的方法是採用小型隧道機械挖掘 (TBM)來進行，由第一頂管井到接管井，長大約 100 米及鄰地面大約最小 3 米深。

頂管井是 9.00 米深及位於屯興路近屯門支站旁的空地，而接收井是 4.50 米深及位屯興路近仁愛分科診所行人路旁的草地。

由於需要橫過公路，在考慮工友們的安全及對公衆的影響，所以這工程採用了小型隧道機械來進行無坑挖掘來減小馬路面最重沉降及隧道下陷的危

頂管井在鄰電力支站，必須留意起吊機會否碰到電站的護罩，在工地內需豎株大樹，所以在進行吊運工作時，要與保持適當距離，避免對大樹造成

Plate 4: Site specific SSoW



Plate 5: Safety drill inside jacked pipes

To ensure safety of workers in the pit during lifting operation, a shelter was constructed as a protection area for workers (Plate 6) and a buffer zone at the entrance of the pit was also established (Plate 7). Besides safety concern on specific works conditions, CLP implemented Registered Site Supervisor (RSS) system to enhance the safety standards of the site supervisors. Qualified RSS was deployed at each trenchless works site for full-time site supervision to ensure that the works was carried out in a safe and proper manner. For example, RSS would be required to supervise lifting operations to ensure proper lifting operations.



Plate 6: Shelter

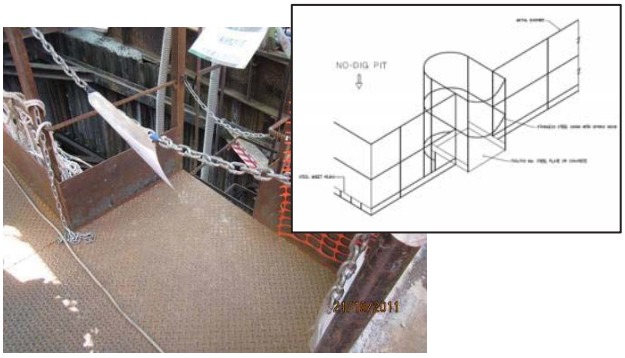


Plate 7: Buffer Zone

Prior to the commencement of trenchless works, monitoring instrumentation was to be installed as per construction drawing and then surveyed and monitored in frequent intervals (Plate 8). If any results of the monitoring checkpoints reached the alarm level, all construction works was to be stopped immediately and remedial measures were to be carried out to prevent the situation from deterioration. Moreover, to seal off hydraulic barrier, minimize the ingress of groundwater and prevent the collapse of the natural ground around the entrance ring during the commencement of the excavation works at the pit, ground treatment (curtain grouting) was to be carried out (Plate 9).



Plate 8: Settlement / Vibration Monitoring

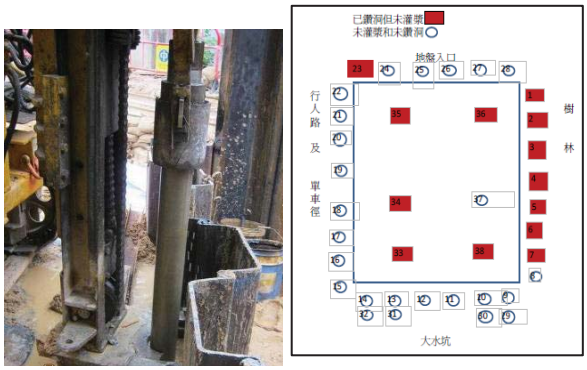


Plate 9: Ground Treatments

In order to mitigate potential risks arising from sheetpile opening for pipe jacking works, a number of small holes were drilled through the sheetpile to make sure that there was no excessive ingress of groundwater. If excessive ingress of water still persisted, horizontal drilling and subsequent grouting would be carried out until ingress of groundwater could be contained to an acceptable level (Plate 10). To prevent micro-tunnelling and jacked sleeve pipe from moving backward due to high groundwater pressure, after completion of each sleeve pipe jacking operation, welding works were carried out between steel guide rail and steel end plate of the jacking sleeve pipe to uphold the position of the sleeve pipe (Plate 11).



Plate 10: Grouting works before forming an opening on the sheetpile

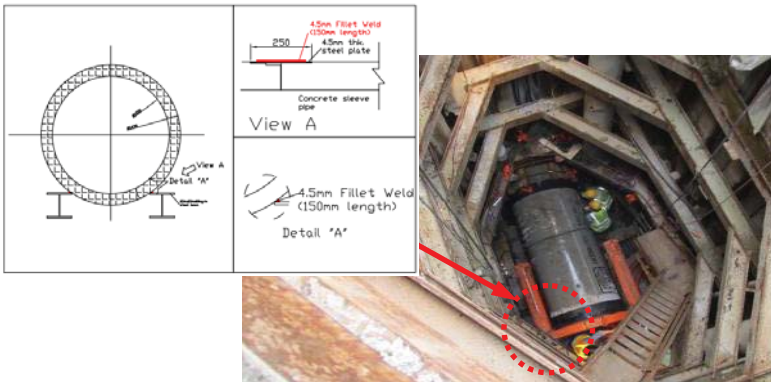


Plate 11: Precaution measure to prevent backward movement of jacking pipe

5 CONCLUSION

The use of micro-tunnelling techniques to install underground electricity cables has been used at strategic location in CLP project over a decade to cater for the situations where there are overall economic and engineering incentive to the projects as well as mitigating disturbance to the public. In view of the nature of micro-tunnelling works and the associated risks, which may lead to serious injuries, CLP has developed a comprehensive SHEQ management system, in addition to ICE's monitoring system, to ensure the trenchless works to be carried out in a safe and proper manner.

ACKNOWLEDGEMENT

The paper is published with the permission of the management of CLP Power Hong Kong Limited and Fugro (Hong Kong) Limited. Tributes are also paid to many practitioners whose wealth of experience has continued to contribute to improvements in the practice of trenchless works in Hong Kong.

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Centrifuge Modelling of Tunnel Excavation over an Existing Perpendicular Tunnel

K.S.G. Lim, T. Boonyarak & C.W.W. Ng
Civil and Environmental Engineering Department, The Hong Kong University of Science and Technology

ABSTRACT

Nowadays, tunnels are constructed in an increasing rate due to rapid development in urban areas. Consequently, soil-structure interaction problems due to tunnelling have become a major concern. Limited studies are conducted on twin tunnel interaction considering only two-dimensional tunnel excavation and simulating the effect of volume loss only. Therefore, the stress transfer mechanism caused by multi-stage tunnel advancement and weight loss on an existing tunnel was not fully understood. When a new tunnel is excavated above an existing tunnel, effect of weight loss caused by removal of soil weight inside the new tunnel lining should be considered. In this study, a three-dimensional centrifuge model test is conducted to investigate the effects of volume and weight losses on the interaction between perpendicularly crossing tunnels. Three-dimensional stress ground displacement, deformation of the existing tunnel and bending moment induced on tunnel lining are reported and discussed.

1 INTRODUCTION

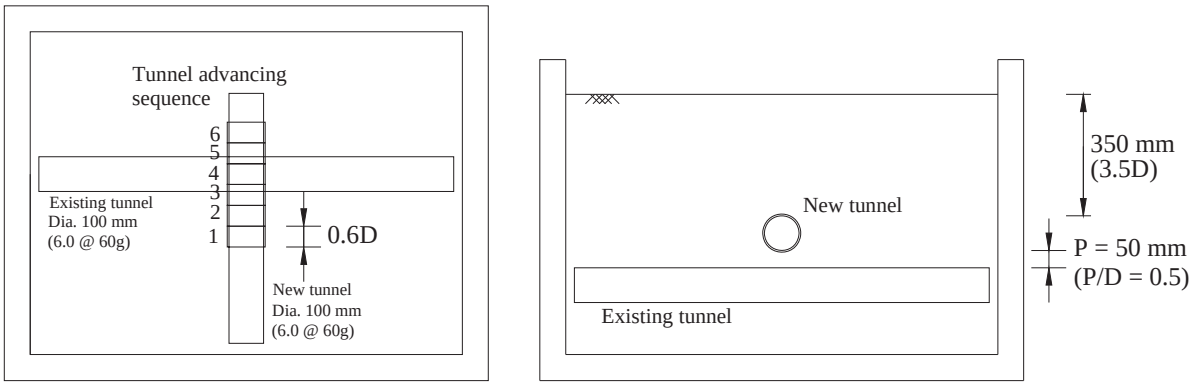
Tunnels are constructed for transportation, communication and other purposes. It is sometimes inevitable whereby multiple tunnels are constructed in a closely spaced area in order to develop more efficient and environmentally friendly infrastructure in congested urban cities. Studies on multiple tunnel interaction are getting popular these days as the impact on the existing tunnels due to newly constructed tunnels may be tremendous in terms of serviceability and safety problems. Liu et al (2011) carried out an investigation of a new tunnel excavation above an existing tunnel based on a case study in Nanjing. The study focuses on application of jet grouting slab and effect of skew of the crossing tunnels. Kim et al (1998) carried out physical model tests on parallel (side-by-side) and perpendicular (cross-cutting) tunnels. The limitation of the test is that the actual stress condition in prototype scale cannot be replicated since the tests were carried out at 1 g and hence the behaviour of soil in the model box is different from that in prototype.

However, the soil-structure interaction problems arising from perpendicularly crossing tunnels attract relatively little research attention in the past. Only limited studies are carried out and the effects of both weight and volume losses due to different tunnelling sequence for perpendicularly crossing tunnels constructed in sand has not been fully understood. Therefore, it is important to investigate the effects of tunnelling on nearby existing tunnels so that the existing tunnels can continue to operate safely both during and after construction of the new tunnel. In this study, three-dimensional physical model test was carried out in the state of the art geotechnical centrifuge at The Hong Kong University of Science & Technology (HKUST) (Ng et al. 2001). The study considers the effect of volume loss, weight loss and three-dimensional tunnel excavation to investigate the soil-structure interaction problem due to a new tunnel excavation above an existing perpendicular tunnel.

2 CENTRIFUGE MODEL PACKAGE AND TEST PROCEDURE

Figure 1 shows the centrifuge model package for this study. A new tunnel is excavated above an existing tunnel with cover-to-diameter ratio (C/D) equal to 3.5 and 5, respectively. A pillar depth-to-diameter ratio (P/D) of 0.5 is adopted, whereby the pillar depth is the distance between the outer lining of the twin tunnels. The elevated gravity in the centrifuge is 60g. For a model tunnel with an external diameter of 100 mm tested at 60 g, the external diameter is equivalent to 6m in the prototype. The lining thickness is 0.18 m in prototype

scale. All the two model tunnels is made of aluminium-alloy tubes. Proper scaling is adopted to ensure correct bending rigidity (EI) of each tunnel simulated in the centrifuge tests. After scaling the EI, the tunnel lining thickness is equivalent to a concrete tunnel lining with 265 mm thickness. The new tunnel is excavated in 6 steps. Each excavation length is 3.6 m in prototype or 0.6 D. To simulate a novel three-dimensional tunnel simulation technique, six independent pairs of volume-controllable rubber bags containing heavy fluid are adopted. An inner rubber bag is used to control weight loss, whereas an outer rubber bag is adopted to simulate volume loss. In this study, volume loss of 2% is considered and simulated. Heavy fluid is drained out from the inner and outer rubber bag simultaneously in six stages (1-6) to simulate tunnelling process. The test is carried out in dry Toyoura sand. Seven potentiometers are installed to measure the displacement and deformation of existing tunnel whereas eight pairs of strain gauges are installed to measure and the bending moment induced on the existing tunnel lining. In addition, ground surface settlement is measured by using Linear Variable Differential Transformers (LVDTs) in-flight. All the instrumentations are installed at 1g. A relative density of 68% is achieved using the sand raining technique.



(a) Plan view (b) Elevation

Figure 1: Centrifuge model tunnel package

3 CENTRIFUGE TEST RESULTS

All results are converted into prototype scale, unless stated otherwise.

3.1 Transverse ground surface settlement

Figure 2 shows the measured transverse (x-direction) ground surface settlement at six advancing stages. The tunnel advancement is in the y-direction represented by y/D=-1.5, -0.9, -0.3, 0.3, 0.9 and 1.5 for the corresponding six excavation stages, respectively. It can be seen that as the tunnel advances, the transverse ground surface settlement increases. After the last excavation i.e. y/D=1.5, the maximum induced ground surface settlement is approximately 26mm in prototype scale. Also, the settlement trough width is approximately 3D away from the new tunnel centre line. The measured ground surface settlement trough is then fitted with the Gaussian distribution curve through most of the measured points with the same maximum induced settlement. The actual volume loss is then deduced using the following equation:

$$V_s = \sqrt{2\pi} i_x \delta_{max} \quad (1)$$

The volume loss deduced is 1.3% which is lesser than the expected volume loss for Greenfield case which is 2%. This may be due to the presence of existing tunnel below the new tunnel which stiffens the ground and result in smaller volume loss.

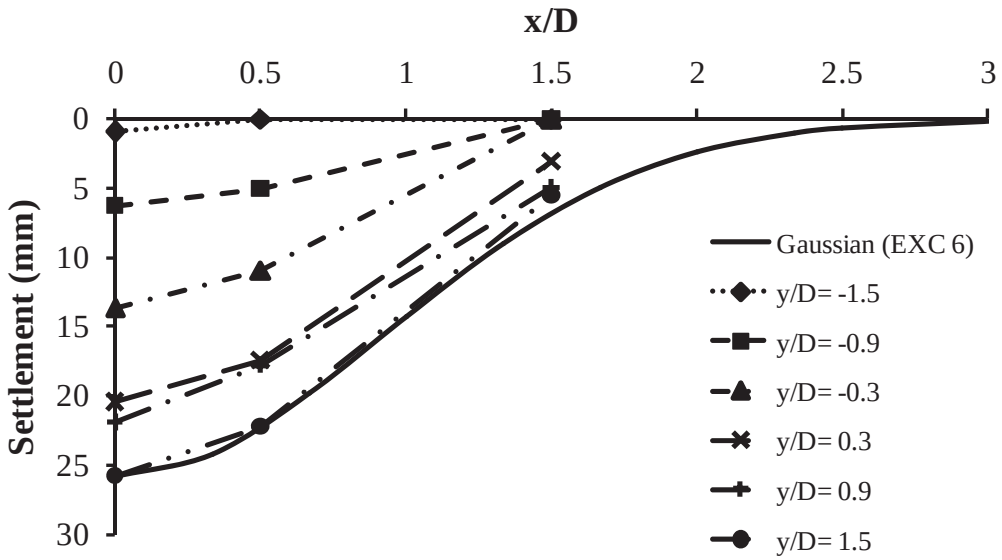


Figure 2: Comparison of transverse surface settlement troughs at six tunnel advancing stages

3.2 Displacement and deformation of existing tunnel

Figure 3 shows the measured incremental displacement and deformation of existing tunnel by the seven potentiometers due to the construction of new tunnel. The existing tunnel experiences vertical elongation and horizontal compression due to stress relief as the excavation of new tunnel advances. The vertical diameter of the existing tunnel lining increases by 5mm while the horizontal diameter decreases by 1.5mm at the end of excavation. The vertical elongation and horizontal compression of the existing tunnel is verified by the measured bending moments by strain gauges later. .

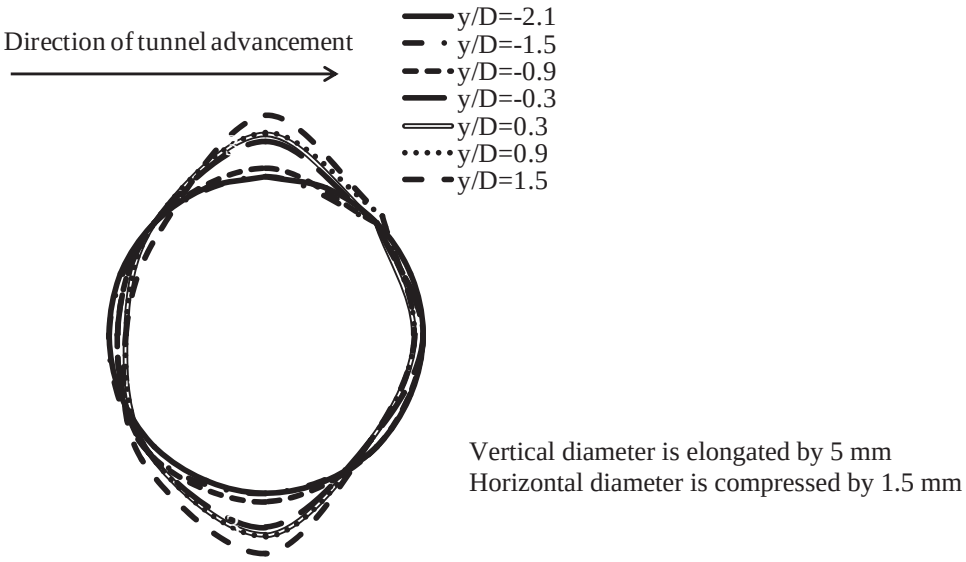


Figure 3: Incremental displacement and deformation of existing tunnel due to tunnelling

3.3 Bending moment induced on existing tunnel lining

Figure 4 shows the incremental normalised transverse bending moment induced on the existing tunnel lining during tunnelling. Positive bending moment denotes that the existing tunnel deformed outwards (i.e. outer face of tunnel lining in tension) while the negative bending moment denotes that the existing tunnel deformed inwards (i.e. outer face of tunnel lining in compression). The bending moment induced on the existing tunnel

lining is normalised by the ultimate bending moment calculated based on the applicability of simple beam theory represented by the following equation:

$$M_u = 1/6\sigma_c h^2 \quad (2)$$

where σ_c is the compressive strength of the tunnel lining and h is the thickness of the tunnel lining. The compressive strength of the concrete, σ_c used for the tunnel lining herein is 40 MPa and the thickness of the lining, h is 0.265 m. By using equation (2), the ultimate moment capacity, M_u can be calculated and is equal to 470 kNm/m. It can be seen from the figure that the measured bending moment increases at the crown and invert while the bending moment decreases at the springline as the new tunnel face advances. However, the bending moment decreases slightly after the new tunnel passes the monitoring section. This may be due to stress redistribution due to soil arching when the excavated section passes the existing tunnel. The measured results show that the outer face of the existing tunnel lining is in compression at the springline but in tension at the crown and invert. This results in the elongation of the existing tunnel lining i.e. the vertical diameter of the tunnel increases but the horizontal diameter of the existing tunnel lining decreases as shown in Figure 3. The transverse normalised bending moment has no significant changes after the new tunnel passes the monitoring section. The maximum normalised transverse bending moment occurs at the crown due to the effects of stress release when the new tunnel is driven above the existing tunnel.

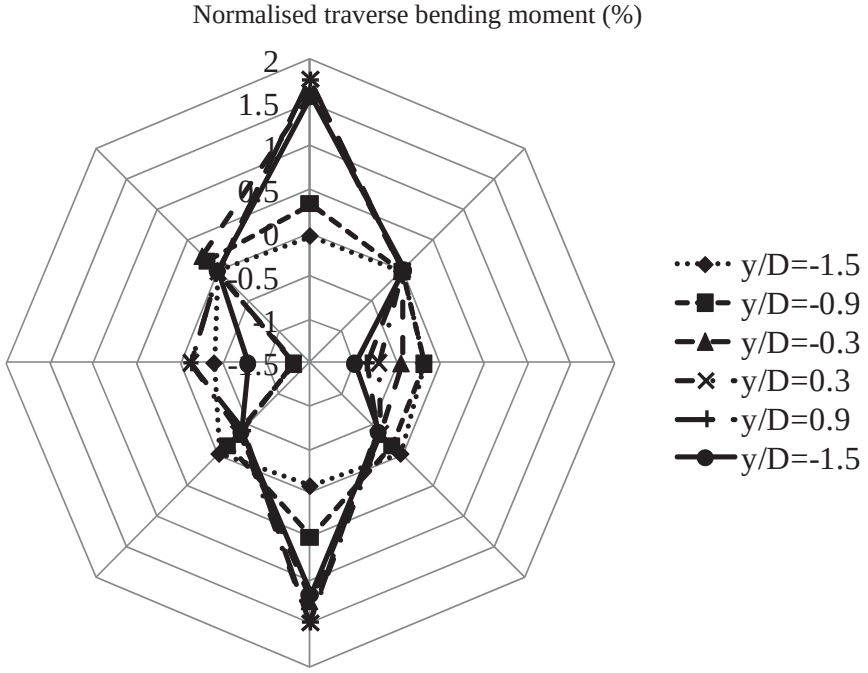


Figure 4: Normalised incremental transverse bending moment induced on existing tunnel during tunnelling

4 CONCLUSIONS

(1) The maximum induced ground surface settlement is approximately 25 mm (prototype) and the observed settlement trough width is approximately 3D away from the centre line of the new tunnel. The deduced volume loss due to tunnelling is equivalent to 1.3%, which is less than the prescribed volume loss of 2%. This may be due to the presence of the existing tunnel which is much stiffer than the existing soil in the ground. .

(2) The vertical diameter of the existing tunnel lining increases by 5 mm while the horizontal diameter decreases by 1.5 mm at the end of excavation. This observation is caused by stress relief induced by removal of equivalent soil weight inside the tunnel lining and displacement of soil around the new tunnel lining.

(3) The maximum normalised transverse bending moment occurs at the crown due to effects of stress relief when the new tunnel is driven above the existing tunnel. The critical section of the new tunnel

advancement on the existing tunnel occurs when the excavated section is directly above the existing tunnel. The transverse bending moment decreases slightly after the new tunnel passes the monitoring section. This may be due to stress redistribution resulting from soil arching when the excavated section passes the existing tunnel.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided by the General Research Fund 617410 from the Research Grants Council of the Hong Kong SAR.

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Centrifuge Modelling of the Effects of Twin Tunnelling on a Loaded Pile Group

C.W.W. Ng, M.A. Soomro & S.Y. Peng
Department of Civil and Environmental Engineering,
The Hong Kong University of Science and Technology, Hong Kong

ABSTRACT

Tunnels are often constructed to reduce traffic congestion and environmental impact in urban cities. It is inevitable that some tunnels have to construct near existing pile foundations inducing three-dimensional tunnel-soil-pile interaction problems. Various studies have been carried out to investigate tunnel-soil-pile interaction by simplifying it as a two-dimensional problem (i.e. the plane strain conditions) and focusing on the response of a single pile due to the construction of a tunnel. Three-dimensional physical modelling of twin tunnel-soil-pile interaction is rarely reported. This paper describes and reports a three-dimensional centrifuge model test investigating the response of an initially loaded 2 × 2 pile group during in-flight excavation of two parallel tunnels. During the advancement of each tunnel, induced settlement of pile group and tilting of pile cap (in both transverse and longitudinal directions) were measured. The measured results are reported and discussed.

1 INTRODUCTION

Due to shortage of lands, high-rise buildings are preferred to meet the development and economical growth in major cities. The construction of high rise buildings often requires deep foundation such as pile group when the underlying soil and rock strata do not have sufficient bearing capacity. Similarly, tunnels are in demand to minimize traffic congestion on roads and to reduce environmental impacts in these cities. Very often, tunnels have to be constructed close to pile foundations in urban areas. The construction of tunnels will inevitably induce stress changes in the ground and may induce excessive ground settlement, tilting of pile cap and reduce the load carrying capacity of piles. To study the tunnel-soil-pile interaction, a number of researches have been carried out including field monitoring, centrifuge and numerical modelling as well as analytical solution. Bezuijen & Schrier (1994) carried out centrifuge tests to determine the influence of bored tunnels on pile foundations. Loganathan et al (2000) assessed tunnelling induced ground deformations, induced axial forces and bending moments in a single and a pile group in clay. Jacobsz et al (2004) investigated the adverse effects of tunnelling on a pile located above the tunnel in dry sand. Lee & Chiang (2007) studied the tunnelling induced bending moment of a single pile in saturated sand. Their tunnels were embedded at depths of various cover-to-diameter ratios, to investigate tunnelling induced bending moment and axial force of the single pile. Moreover, numerical analyses and analytical solutions have been reported in the literature to study pile-tunnel interaction problem (e.g. Chen, 1999; Mroueh & Shahrour, 2002; Lee & Ng, 2005).

Most of centrifuge tests reported in the literature were carried out under the plane strain condition (i.e. two dimensional) and limited to the response of pile foundation due to the construction of a single tunnel. In this study, therefore, a three-dimensional centrifuge test was carried out to investigate the effects of the construction of two parallel tunnels on a nearby 2 × 2 pile group. Induced settlement of the pile group and titling of the pile cap due to advancement of two parallel tunnels are reported and discussed.

2 CENTRIFUGE MODELLING AND TEST PROCEDURE

2.1 Experimental set-up

The centrifuge test was carried out at the Geotechnical Centrifuge Facility of the Hong Kong University of Science and Technology (Ng et al, 2001 & 2002). The centrifuge has a capacity of 400 g-ton, with an arm radius of 4.2 m. The test was conducted at an acceleration of 50 g.

Figure 1 shows a schematic elevation view of the centrifuge model. A 2 × 2 pile group was located at centre of the model container. Diameter (d_p) and length of each pile (L_p) were 20 mm and 600 mm in model scale, respectively. The embedded depth for each pile was 500 mm. The corresponding pile diameter and embedded depth are 1 m and 25 m in prototype scale, respectively. Each model pile had an axial rigidity E_mA_m of 3517 N and a bending rigidity E_mI_m of 113 Nm^2. The corresponding prototype E_pA_p and E_pI_p are equal to 8.8 MN and 706 MNm^2, respectively. All four piles were rigidly connected to a 130 mm × 130 mm × 20 mm pile cap. The pile cap was made of 20 mm thick aluminum plate, representing 1 m thick reinforced concrete pile cap. A dead weight of 17.5 kg (corresponding to 22 MN in prototype scale) was placed on the top of pile cap to simulate working load applied to the pile group.

Two model tunnels, i.e. the first (left) and the second tunnels (right) are also shown in the Figure. Diameter (D) of each tunnel was 152 mm in model scale, representing 7.6 m diameter tunnels in prototype. The center of each model tunnel was located at the same level as pile toe. The horizontal distance from the center of each tunnel to the front piles was 165 mm (1.1 D).

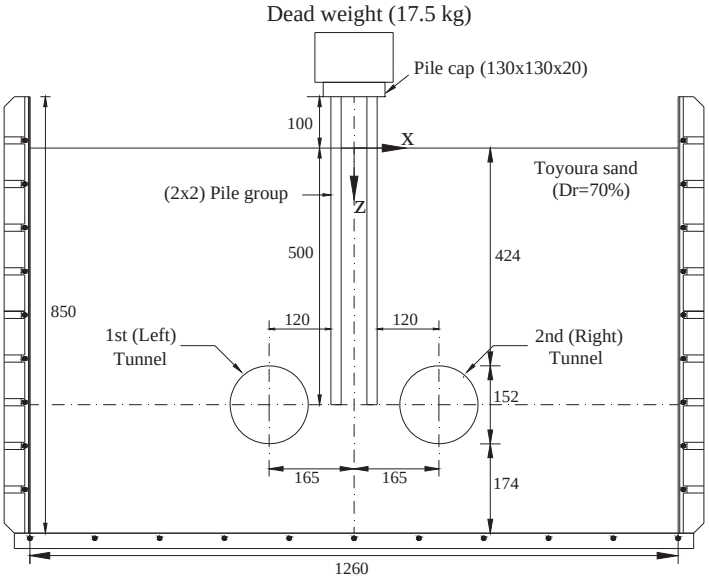


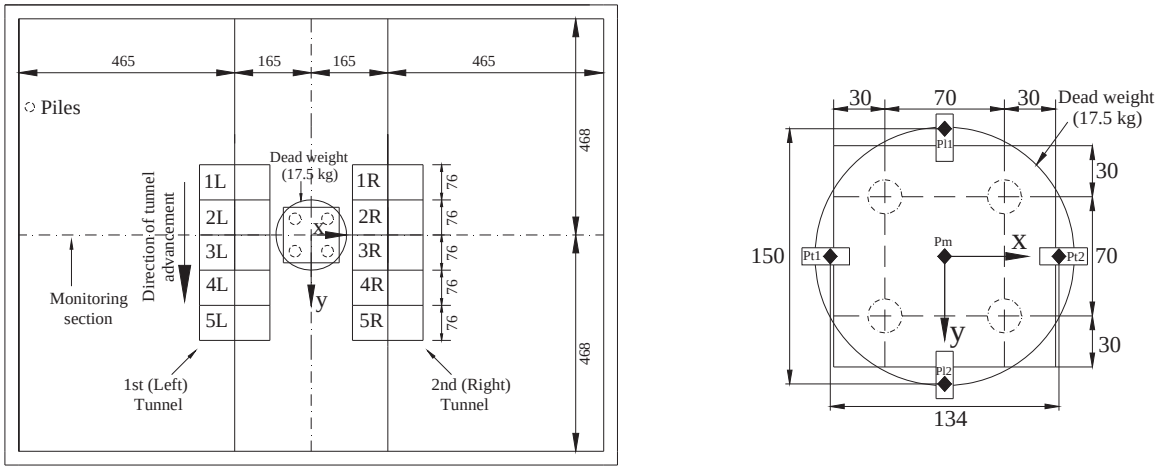
Figure 1: Schematic elevation view of centrifuge model
Note: All dimensions are in mm in model scale.

2.2 Simulation of tunnel advancement and instrumentation

Figure 2(a) shows the plan view of model. The longitudinal length of each tunnel was 380 mm (2.5D). Excavation of each tunnel was divided into five stages (i.e. 1L -5L and 1R-5R). Each stage had an advancing distance of 0.5D. Three-dimensional tunnel advancement of both tunnels was modelled by controlled equivalent volume loss. Each model tunnel consisted of five cylindrical rubber bags, which were filled with de-aired water. By using fully filled five rubber bags, the advancement of each model tunnel was simulated by releasing a well-controlled amount of water, equivalent to 2% of volume loss. The tunnel on the left side of the pile group was excavated first and then the tunnel on the right side. A monitoring section was at centre line of pile group (i.e. y = 0) for reference of tunnel advancement.

Figure 2(b) shows the configuration of potentiometers mounted on the dead weight. A potentiometer (P_m) was installed on middle of the dead weight to measure the settlement of pile group. A pair of potentiometers (P_t1 & P_t2) were mounted along transverse direction of tunnel to capture tilting in transverse direction.

Another pair of potentiometers (P_l1 & P_l2) were installed to measure tilting along the longitudinal direction of tunnel adcancement. The distance between P_t1 & P_t2 and P_l1 & P_l2 were 134 mm and 150 mm, respectively.



(a) Centrifuge model (b) Pile cap

Figure 2: Plan view of (a) centrifuge model and (b) pile cap instrumented with potentiometers
Note: All dimensions are in mm in model scale.

2.3 Sample preparation and test procedure

Dry Toyoura sand was used in the test. The specific gravity (G_s) of sand grains is 2.65. The minimum and maximum void ratio (e_{min} and e_{max}) of Toyoura sand are 0.977 and 0.597, respectively (Ishihara, 1993). Sand was rained into strongbox from a hopper from constant height of 500 mm. A fairly uniform density of 1550 kg/m^3 (i.e. D_r = 70%) was achieved. After model preparation, dead weight (i.e. 17.5 kg) was put on top of pile cap. Then acceleration of the centrifuge was increased to 50 g. After achieving equilibrium, advancement of first (i.e. left side of pile group) was simulated by designed volume loss of 2%. Each of five advancing stage was simulated in-flight by releasing a well-controlled amount of water from rubber bag one by one. Subsequently, excavation of second (i.e. right side of pile group) tunnel was simulated by the same procedure as first tunnel. The induced settlement of pile group and tilting of pile cap in both directions (i.e. transverse and longitudinal directions of tunnel advancement) was recorded during each stage of advancement.

3 TEST RESULTS AND DISSCUSSION

All results presented here are in prototype scale unless stated otherwise.

3.1 Induced pile group settlement due to twin tunneling

Figure 3(a) shows the induced settlement of the pile group during advancement of the first tunnel. The measured settlement and the distance from tunnel face to centre of the pile group (y) are normalized by pile diameter (d_p) and tunnel diameter (D), respectively. The induced settlement was measured by potentiometer (P_m) mounted on the center of dead weight, as shown in Figure 2(b). The pile group starts to settle as first tunnel advances. At the end of the first excavation of the left tunnel (i.e. y/D = -0.5), the induced pile group settlement is 0.17%d_p. Larger pile group settlement (1.26%d_p) is induced due to the second, third and fourth excavation stages (i.e. y/D = -0.5 to 1.0). This settlement is 77% of the total settlement induced to the pile group after completion of the first tunnel. The induced settlement due to the fifth excavation stage (i.e. y/D = 1.0 to 1.5) is only 0.20%d_p, as this section is further away from the pile group. The induced settlement of the pile group is 1.63%d_p at the end of the first tunnel construction.

As shown in Figure 3(b), the magnitude of settlement induced to the pile group due to advancement of second tunnel is quite similar to that of the first tunnel. Incremental pile group settlement induced due to only the second tunnel is 1.62%d_p. From the results, it appears that pile group response due to second tunnel (i.e. right tunnel) is independent of excavation of first tunnel (i.e. left tunnel) in terms of settlement. The final induced settlement of pile group after the twin tunnelling is about 32.5 mm (3.25%d_p). This settlement is

larger than the allowable settlement (i.e. 25 mm) for typical residential buildings (Wahls, 1994; Zhang & Ng, 2005).

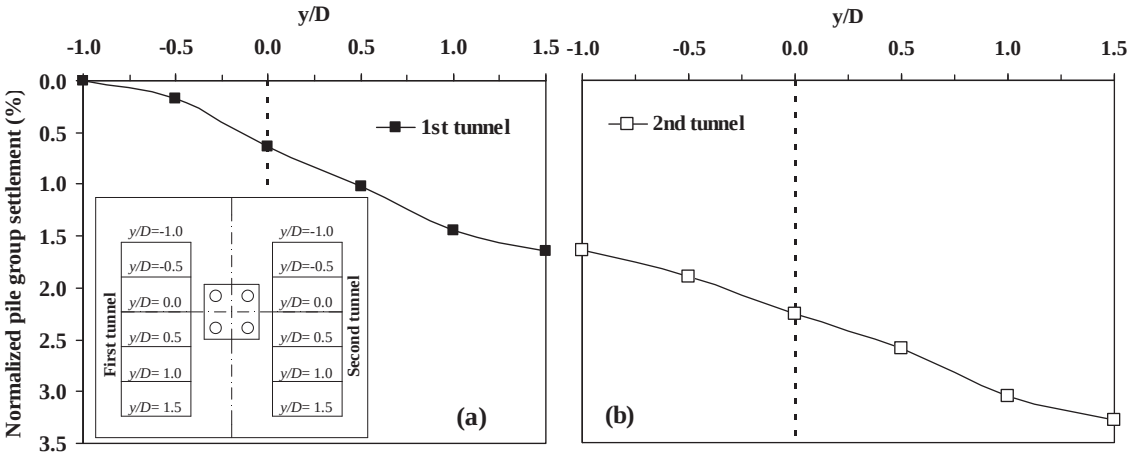


Figure 3: Induced settlement of the pile group during advancement (a) the first tunnel and (b) the second tunnel

3.2 Tilting of pile cap due to twin tunnelling

Tilting in transverse direction of each tunnel

Figure 4 shows the tilting of pile cap in the transverse direction of tunnel advancement during excavations of the twin tunnels. Potentiometers P_t1 and P_t2 were installed on the pile cap to measure settlements at two locations of the pile cap, as shown in Figure 2(b). Tilting is defined as the ratio of differential settlement between measured by two potentiometers (P_t1 & P_t2) to the distance between them. Positive value of tilting in the transverse direction is defined as the pile cap tilts towards the first tunnel.

It can be obsreved from Figure 4(a) that as first tunnel advances towards monitoring section, the pile cap starts to tilt towards the tunnel. During the first excavtion stage (i.e. y/D = -1.0 to -0.5), the pile cap tilting is quite small. However, when the tunnel face reaches between y/D = -0.5 and 1.0, the pile cap tilts significantly towards the first tunnel as a result of stress release due to these excavations. The most critical stage is when tunnel face is between y/D = -0.5 and 0.5, when the largest magnitude of incremental tilting is induced. The magnitdue of tilting is about 0.20% when tunnel face is at y/D = 1.0. Incremental tilting due to the fifth excavation (i.e. y/D = 1.0 to 1.5) is only 0.02%. Thses observations cannot be captured in a centrifuge test carried out under the plane strain conition. The magnitude of titing after excavation of first tunnel is 0.22%.

As shown in Figure 4(b), the magnitude of tilting in the transverse direction reduces as the second tunnelling is carried out. It means that the pile cap tilts back towards the second tunnel. The reduction in tilting due to the second tunnelling is 0.24%. The magnitude of the reduction is very close to that induced by the first tunnelling. As a result, the pile group tilts back to its original position at the end of the twin tunnelling. Therefore, the most critical stage in terms of tilting in the transverse direction is at the end of the first tunnelling. The maximum magnitude of tilting of about 0.22% occurs at this stage.

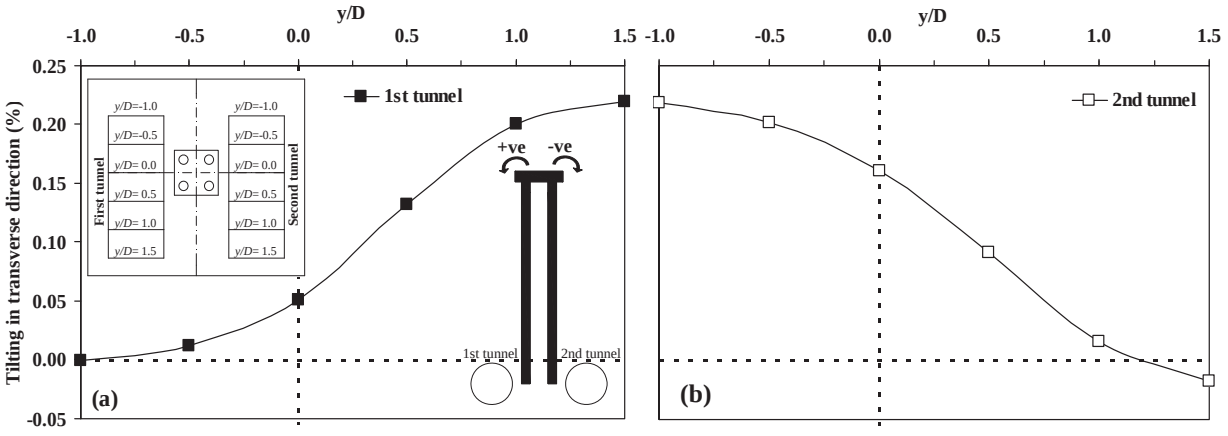


Figure 4: Tilting of pile cap in transverse direction during advancement of (a) the first tunnel (b) the second tunnel

Tilting in longitudinal direction of each tunnel

Figure 5(a) shows the induced tilting of pile cap in longitudinal direction during advancement of the first tunnel. Potentiometers (P_l1 and P_l2) were mounted on pile cap (see Figure 2(b)) to measure differential settlement in the longitudinal direction. Positive value is defined as pile cap tilts in the opposite direction of tunnel advancement. From the figure, it can be seen that positve pile cap tilting is observed during the first and second stages (i.e. y/D = -1.0 to 0.0). This means the pile cap tilts towards the first excavation in the longitudinal direction (towards excavation 1L in Figure 2(a)). The second stage of tunnelling induces larger tilting as compared to the first stage, because it is closer to pile group. The maximum tilting in the longitudinal direction is about 0.10%, which occurs at y/D = 0.5. As tunnel face passses beyond y/D = 0.5, pile cap tilt back slightly.

Figure 5(b) shows the tilting of pile cap during advancement of the second tunnel. The first and second tunnelling stages (stages 1R and 2R in Figure 2(a)) induce positive incremental tilting to the pile cap. The maximum tilting is about 0.12%, which occurs at y/D = 0.0 (i.e. the end of excavation stage 2R). The maximum value of longitudinal tilting is not as significant as that in transverse direction, which is 0.22% as shown in Figure 4(a). The magnitude of longitudinal tilting reduces as the tunnel face advances beyond the center of the pile group (y/D = 0.0 to 1.5). The final tilting in the longitudinal direction is about 0.06% after the completion of the twin tunnelling.

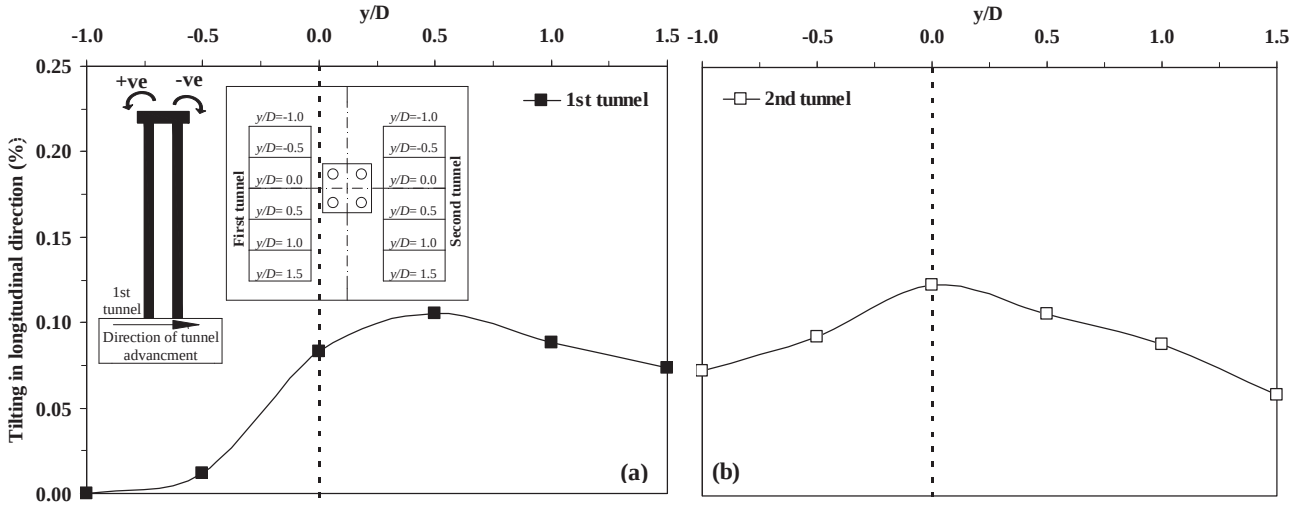


Figure 5: Tilting of pile cap in longitudinal direction during advancement of (a) the first tunnel and (b) the second tunnel

4 CONCLUSIONS

Based on the centrifuge test, the following conclusions may be drawn:

(1) Induced settlement of the pile group after the excavation of first tunnel is 1.63% of pile diameter (d_p). The most significant increment settlement occurs when the tunnel face advances between y/D = -0.5 and 1.0. The induced settlement during these stages of excavations is 77% of the total settlement at the end of the first tunnelling. The induced incremental settlement of the pile group during the advancement of the second tunnel is similar in magnitude to that during first tunnelling. The pile group experiences a final settlement of 3.25%d_p after excavation of both tunnels.

(2) The pile cap tilts towards the tunnel in the transverse direction during the excavation of the first tunnel. The maximum tilting in transverse direction is about 0.22%, which occurs at the end of the first tunnelling. During the second tunnelling, the pile cap tilts towards the second tunnel (i.e., away from the first tunnel). As a result, the magnitude of tilting in the transverse direction is reduced during advancement of the second tunnel.

(3) As expected, tilting of pile cap in the longitudianl direction of the tunnel also occurs during the contruction of the twin tunnels. The maximum of tilting is 0.12% when the tunnel face of second tunnelling reaches the center of the pile group (i.e., y/D = 0.0). After the completion of both tunnels, the final tilting reduced to 0.06%. Relatively speaking, the observed longitudinal tilting is substantially smaller than that measured in the transverse direction.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided by the General Research Fund 617608 from the Research Grants Council of the Hong Kong SAR.

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Effects of Twin Tunnel Construction at Different Elevations on an Existing Loaded Pile in Centrifuge

H. Lu & C.W.W. Ng
Department of Civil and Environmental Engineering, The Hong Kong University of Science and Technology, Hong Kong SAR

ABSTRACT

Tunnelling activity inevitably induces stress changes and ground deformation, which may affect nearby existing pile foundations. Although a number of studies have been reported to investigate the effects of tunnelling on existing piles, the excavation of only a single tunnel is often considered. This paper describes a three-dimensional centrifuge model test to investigate the response of an existing loaded single pile due to twin tunnel excavation located at different elevations in dry Toyoura sand. In addition, another centrifuge test was carried out to determine the load capacity of the pile. The first tunnel was simulated in-flight near mid-depth of pile shaft whereas the second tunnel was excavated near pile toe in-flight. The diameter of each tunnel simulated in centrifuge was 6.1m in prototype. The diameter and length of the pile simulated was 0.8 m and 19.6 m, respectively. The clear distance between the pile and each tunnel was 1.1m. By using five rubber bags filled with water, the advancement of each tunnel was simulated in five stages by releasing a well-controlled amount of water, equivalent to 1% of volume loss. Measured ground surface and pile head settlements and induced bending moments induced in the pile are reported and discussed.

1 INTRODUCTION

Tunnels are often preferred for underground transportation systems in densely populated areas. It is almost inevitable for tunnels to run close to some existing pile foundations in these areas. Since any tunnelling activity may induce stress change and soil movement in the ground, nearby piles may suffer from additional axial force, bending moments, and settlements. Estimation of the effects of tunnelling on existing pile foundations of buildings poses a major challenge to designers. It is particularly vital to estimate the tunnelling effects when two new tunnels are to be built near an existing pile.

In the literature, some centrifuge model tests have been carried out to investigate tunnelling effects on piles. Bezuijen & Schrier (1994) studied the influence of bored tunnels on pile foundations. Loganathan et al (2000) assessed tunnelling induced ground deformations and their adverse effects on pile foundations in clay. Jacobsz et al (2001) investigated the adverse effects of tunnelling on a pile located above the tunnel in dry sand. Lee & Chiang (2007) studied the tunnelling induced bending moment of a single pile in saturated sand. The tunnels were embedded at depths of various cover-to-diameter (C/D) ratios, to investigate tunnelling induced bending moment and axial force of the single pile. In addition, some numerical analyses have also been reported in the literature to study the tunnelling effects on pile foundations (e.g., Mroueh & Shahrour, 2002; Lee & Ng, 2005).

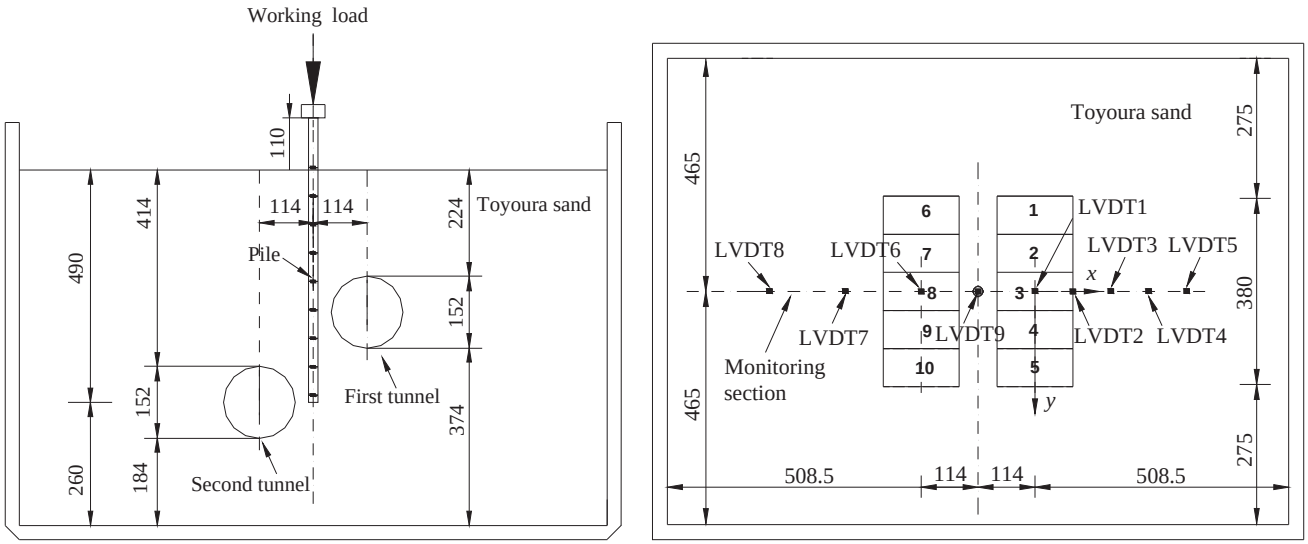
Pang (2007) reported the field monitoring and numerical study of the effects of twin shield tunnelling on an adjacent pile foundation in Singapore. A northbound tunnel and a southbound tunnel were constructed near piles one after the other. The smallest clear distance between the tunnels and piles was 1.6 m. Results of the field study showed that the piles were subjected to a large dragload and bending moment due to an induced soil movement in residual soil. However, most of the previous studies are limited to the responses of pile due to single tunnel construction and no centrifuge test is carried out investigating twin tunnelling effects on piles. In this study, a centrifuge test (Test ST) was carried out to assess the responses of a single pile due to twin tunnelling at different depth. In addition, another centrifuge test (Test L) is carried out to obtain the load settlement curve of the single pile without tunnelling effects. Three-dimensional tunnel construction including five advancing stages for each tunnel excavation was simulated in-flight. A volume loss of 1% was well

controlled for each segment of each tunnel. The ground surface and pile settlement were recorded. Instrumented model piles were used to capture the bending moments induced by tunnelling at different advancing stages. Three-dimensional twin tunnelling effects on single pile in different advancing stages is analysed and reported.

2 CENTRIFUGE MODELLING

2.1 Experimental setup

Both the centrifuge model tests were carried out at the Geotechnical Centrifuge Facility of the Hong Kong University of Science and Technology (Ng et al, 2001a; Ng et al, 2002). The 400 g-ton centrifuge has an arm radius of 4.2 m and is equipped with a two-dimensional hydraulic shaking table and a four-axis robotic manipulator. Both the centrifuge tests were carried out at an acceleration of 40 g.



(a) Elevation (b) Plan

Figure 1: Schematic diagram of centrifuge model
Note: All dimensions are in mm in model scale.

Figure 1(a) shows a schematic elevation view of the centrifuge model in Test ST. A single pile was located at centre of the model container. The pile had a diameter of 20 mm. The length of the pile was 600 mm. The pile cap was elevated by 110 mm, therefore the embedded depth for the pile is 490 mm. In prototype scale, the corresponding pile diameter is 0.8 m, the embedded depth is 19.6 m.

As shown in Figure 1(a), the advancement of the first tunnel was simulated near mid-depth of pile shaft and the second tunnel was excavated near pile toe in-flight. The diameter of each tunnel was 6.1 m in prototype. The tunnel diameter (D) was 152 mm, which is corresponding to a diameter of 6.08 m in prototype scale. The horizontal distance from the centre line of the tunnel to the front pile row is 0.75D. The clear distance between the pile and each tunnel was 1.1m in prototype. Test L has the same configuration with Test ST but only without the model tunnel.

Figure 1(b) shows the plan view of the model in Test ST. the longitudinal length of each tunnel was 380 mm, which was equivalent to 2.5D. The tunnel excavation was simulated in five stages, with the tunnel face advancing by a distance of 0.5D in each stage. The three-dimensional effects of each advancing stage on the single pile were investigated.

2.2 Simulation of tunnel advancement

Each model tunnel consisted of five cylindrical rubber bags (see Plate 1). Each rubber bag was filled with de-aired water. Three-dimensional tunnel construction was simulated in-flight by draining away a controlled

amount of water from each rubber bag one by one. The amount of water drained away from each rubber bag was controlled to induce an equivalent volume loss of 1.0% in each stage of tunnel construction.

2.3 Model piles and instrumentation

Each instrumented model pile was fabricated from an aluminium tube. Nine levels of strain gauges were installed to measure the bending moments along the entire pile length. The strain gauges were protected by a thin layer of epoxy. The outer diameter of each pile was 20 mm, which is corresponding to a pile diameter of 0.8 m in prototype scale. The model pile had an axial rigidity (E_mA_m) of 2,154 kN and a bending rigidity (E_mI_m) of 112 Nm^2. The corresponding E_pA_p and E_pI_p values were 3,446 MN and 261 MNm^2 in prototype, respectively.

A vertical load was applied to the pile using a hydraulic jack. A load cell was installed in the piston of the jack to control the applied load. Settlement of the pile was measured by a linear variable differential transformer (LVDT) located at the pile head.

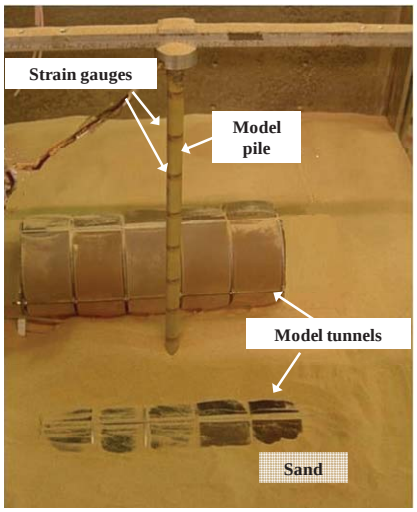


Plate 1: Model tunnels and model pile

2.4 Model preparation

Dry Toyoura sand (G_s = 2.65, e_{max} = 0.977, e_{min} = 0.597, ϕ'_{cv} = 31°) (Ishihara, 1993) was used in the test. The centrifuge model was prepared by pluvial deposition method. Sand was rained from a hopper which was kept by 500 mm above the sand surface. The measured relative densities of sand in the two tests are 65% and 60% for Test ST and Test L, respectively.

2.5 Test procedure

After model preparation, the acceleration of the centrifuge was increased to 40 g. The model pile was loaded in-flight at 40 g in a number of steps. In each step, an incremental vertical load of 100 N (160 kN in prototype) was applied. Each load increment was maintained for three minutes. Once the load had reached the working load (1,200 N), tunnel construction with the designed volume loss of 1.0% was carried out. Five construction stages were simulated in-flight by draining away water from each of the rubber bags one after the other. The ground surface settlement, the settlement of the single pile and the induced bending moments along each instrumented pile were recorded.

3 TEST RESULTS

All the test results are presented in prototype scale unless stated otherwise.

3.1 Determination of the axial load carrying capacity of the pile

Prior to tunnelling, it is necessary to obtain the capacity of the pile so that the working load can be deduced. A pile load test (Test L) was carried out. Figure 2 shows the measured load-settlement relationship. The load applied to the pile cap was gradually increased to 4 MN at increments of 100 kN in each step. The ultimate axial load capacity was determined based on a displacement-based failure load criterion proposed by Ng et al (2001b). This failure load criterion is expressed as follows:

$$\delta_{\mathrm{ph,max}} \cong 0.045 d_{\mathrm{p}} + \frac{1}{2}\frac{P_{\mathrm{h}} L_{\mathrm{p}}}{A_{\mathrm{p}} E_{\mathrm{p}}} \tag{1}$$

where $\delta_{\mathrm{ph,max}}$ is the maximum pile head movement which defines the ultimate load, P_{h} is the pile head load, L_{p} is the pile length, E_{p} is the elastic modulus of pile shaft, A_{p} is the cross-sectional area of the pile, and d_{p} is the pile diameter. As shown in the figure, the ultimate load capacity of the pile is 2.88 MN. A working load of 1.92 MN was adopted with a factor of safety of 1.5. A pile settlement of 1.6% d_{p} is observed due to the applied working load.

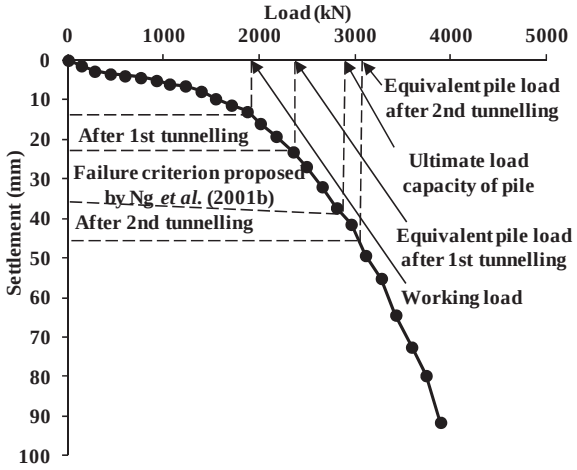


Figure 2: Load settlement relationship obtained from in-flight pile test

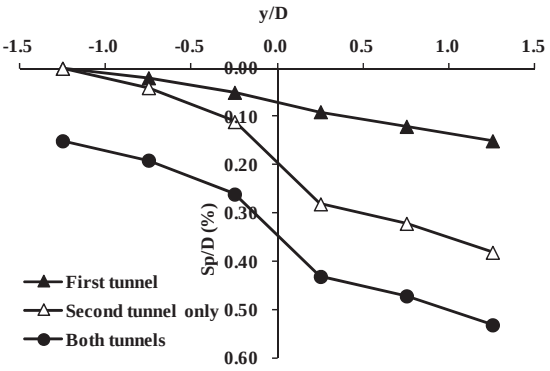


Figure 3: Pile settlement due to tunnel excavation

3.2 Tunnelling-induced settlement of the pile and apparent loss of pile capacity

Figure 3 shows the development of normalized pile settlement (S_{p}) during each tunnel construction stage in Test ST. Location of the tunnel at any stage is indicated by the distance between tunnel face to the centerline of the pile (y). Both the measured S_{p} and the distance from the tunnel face to the centerline of the pile (y) were normalized by the tunnel diameter (D).

During the excavation of the first tunnel, the induced settlement increases almost linearly as the excavation of the first tunnel progressed at C/D = 1.5. After the excavation of the first tunnel, a pile settlement of 0.15%D (1.1% of the pile diameter) was measured.

When the second tunnel is excavated, as the tunnel face advances at a depth of C/D = 2.7 from y/D = -1.25 to -0.25, a pile settlement of 0.11%D (0.8% of the pile diameter) was induced. A significant increase in pile settlement (0.17% D) occurs when the tunnel face advances from y/D = -0.25 to 0.25. When the tunnel face reaches y/D = 1.25, the pile settlement increases to 0.38% D (2.9% of the pile diameter). After the excavation of the both tunnels, the cumulative pile settlement induced by twin tunnelling is about 0.53% D (4.0% of the pile diameter).

By comparing the pile settlement induced by the first and the second tunnel individually, it can be observed that the profile of pile settlement induced by the two tunnels is different. The pile settlement increases almost linearly with the advancement of the first tunnel, whereas a significant increases of pile settlement occurs when tunnel face is between y/D = -0.25 and -0.25 during the excavation of the second

tunnel. The significant increase of pile settlement may imply yielding of soil around the pile toe occurs when the tunnel face reaches the pile toe. Moreover, the pile settlement induced by the second tunnel is about 2.6 times of that due to the first tunnelling. This is due to the fact that the first tunnel is located near the mid-depth of pile shaft, whereas the second tunnel is excavated near pile toe. Lee & Chiang (2007) also reported that pile settlement induced by tunnelling near the mid-depth of pile shaft is smaller than that induced by tunnelling near the pile toe. The test results are consistence with that presented by Lee & Chiang.

Since pile capacity is often interpreted using settlement criteria, the induced pile settlement due to tunnelling can be considered as an apparent loss of pile capacity (ALPC). Based on the results of pile settlement shown in Figure 3, the pile can be thought of as being subjected to an equivalent load of 2.32 MN (obtained from the load settlement curve in Figure 2). The equivalent load on the pile increases by 0.40 MN due to the tunnel excavation. Since the ultimate load carrying capacity of the pile group is 2.88 MN as obtained from the load settlement curve using the displacement-based failure criterion proposed by Ng et al (2001b), it can be considered that an ALPC of 14% occurrs due to the tunnel excavation. The ALPC increases to about 36% after the second tunnel is constructed. The ALPCs suggest that the serviceability limit state of the pile after tunnelling should be considered.

3.3 Tunnelling-induced bending moment along the pile

Figure 4 shows the measured bending moments along the pile in Test ST. The depth (z) is normalized by the diameter of the tunnel (D). Bending moments are taken as positive if tensile stress is induced at the side which is facing the first tunnel.

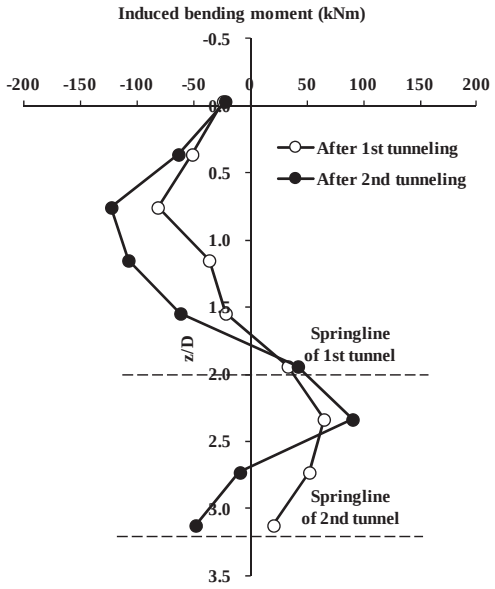


Figure 4: Tunnelling induced bending moment along pile

After excavation of the first tunnel, both positive and negative bending moments are induced to the pile. The positive bending moment occurs along upper part of the pile (z/D < 1.7), whereas negative bending moment happens along the lower part of the pile shaft (z/D < 1.7). The maximum induced bending moment locates approximated at z/D = 0.75. The magnitude is 81.7 kNm, which is about 10.2% of the moment capacity of the pile (800 kN).

After the excavation of the second tunnel, the bending moment near pile toe (z/D < 2.7) turns to be negative due to the soil movement induced tunnelling near pile toe. However, the maximum bending moment still occurs at z/D = 0.75. The magnitude of maximum bending moment increases to 122.6 kNm (15.3% of M_{yield}). It can be observed that the induced bending moment to the pile by twin tunnelling is still relatively small. This is consistent with the numerical results presented by Lee & Ng (2005). They reported that the bending moment induced by a tunnel near pile toe is insignificant as compared to the bending moment capacity of pile. Based on the test results in this study, it is evident that the bending moment induced by twin tunnels are also relatively insignificant.

4 CONCLUSIONS

One in-flight pile load test and one centrifuge model test were carried out to investigate the effects of twin tunnel excavations at different elevations on an existing loaded pile. The advancement of the first tunnel was simulated near the mid-depth of pile shaft whereas the second tunnel was excavated near pile toe in-flight. Based on test results, the following conclusions may be drawn:

(a) Induced pile settlement due to the excavation of the first tunnel is about 1.1% of the pile diameter. The pile settlement induced by the excavation of the second tunnel is about 2.6 times of that induced by the first tunnel. The cumulative pile settlement induced by twin tunnels is about 4.0% of the pile diameter.
(b) Based on the displacement-failure load criterion proposed by Ng et al (2001b), the apparent loss of pile capacity (ALPC) is about 14% after the construction of the first tunnel construction and it increases to about 36% (cumulative) after the construction of the second tunnel.
(c) The induced bending moment due to twin tunnelling is not significant. The maximum bending moment induced in the pile by the excavation of single and twin tunnels is about 15.3% of the ultimate bending moment capacity.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided by the General Research Fund 617608 from the Research Grants Council of the HKSAR.

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Centrifuge Modelling of Three-dimensional Tunnelling Effects on Buried Pipeline

J. Shi & C.W.W. Ng

Department of Civil and Environmental Engineering, The Hong Kong University of Science and Technology

Y. Wang

Department of Civil and Architectural Engineering, City University of Hong Kong

ABSTRACT

Tunnel excavation may induce soil displacement around adjacent buried pipelines and causes additional loading and damage to the pipelines. The interaction between tunneling and adjacent buried pipelines has attracted growing research attention, in which the tunnel-pipe interaction is frequently simplified as a two-dimensional problem (i.e., plane strain problem). This paper presents a three-dimensional centrifuge model test to investigate the influence zone in the tunnel-pipe interaction and pipe responses induced by underneath tunnel construction. Details of the test are presented, including the test program and setup, model pipe and instrumentation, model preparation, and test results (e.g., measured ground surface settlement and induced pipe bending strain). The centrifuge test results are shown to agree well with a dimensionless relationship between the relative pipe-soil stiffness and ratio of maximum pipe curvature to maximum ground curvature. This dimensionless relationship is recently developed, based on extensive finite element simulations, for direct estimation of pipeline responses to tunneling-induced ground movement.

1 INTRODUCTION

The underground space in urban areas is frequently congested with utilities, including pipelines and conduits that are affected by underground construction, e.g., tunneling. Tunnel excavation may induce soil displacement around adjacent pipelines and causes additional loading and damage to the pipelines. It may disrupt the conveyance of important services and threaten safety of urban inhabitant. A number of studies have been carried out on the interaction between tunneling and adjacent buried pipelines (e.g., Attewell et al, 1986; Vorster, 2005; Klar et al, 2008, Marshall et al, 2010, and Wang et al, 2011). For example, Vorster (2005) and Marshall et al (2010) carried out centrifuge tests to investigate the tunneling effects on buried pipelines with consideration of different volume loss, C_p/D_p ratio (i.e., cover-to-pipe diameter ratio) and relative pipe-soil stiffness. Klar et al (2008) carried out numerical parametric study of tunneling effects on jointed pipelines by considering relative pipe-soil stiffness, relative pipe-joint stiffness, and location of joints in relation to tunnel centerline. However, the pipe-soil-tunnel interaction was simplified as two-dimensional problem (i.e., plane strain problem), and spatial variation of the pipe responses and influence zone were not studied.

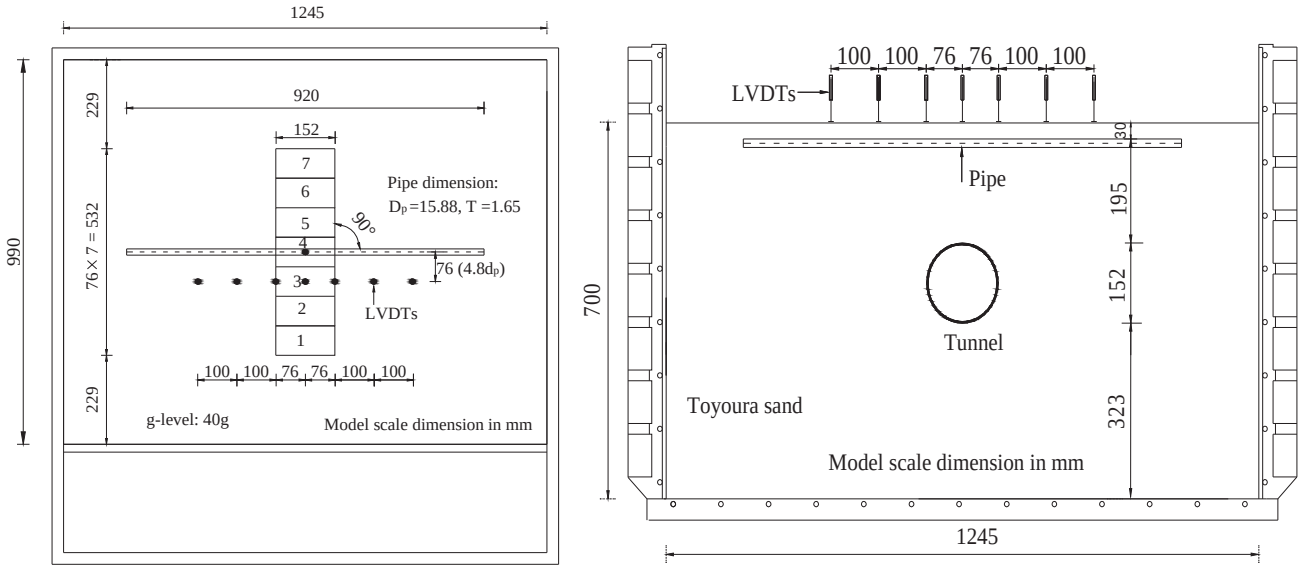
Wang et al (2011) carried out a series of numerical parametric study with 900 finite element (FE) simulation runs to encompass various combinations of ground settlement profiles, pipe dimensions, material properties, pipe burial depth, and soil properties. The Greenfield ground settlement was described by Gaussian distribution. The different pipe-soil responses to uplift and downward movements were simulated explicitly in the FE analyses with two separate nonlinear force–displacement relationships. One dimensionless relationship between relative pipe-soil stiffness versus ratio of maximum pipe curvature to maximum ground curvature was developed, and it can be used to directly estimate the maximum pipe bending strain. This study carried

out a three-dimensional centrifuge test to validate the dimensionless relationship proposed by Wang et al (2011) and to investigate the spatial variation of the pipe responses and influence zone.

2 EXPERIMENTAL PROGRAM AND SETUP

The centrifuge test was carried out at Geotechnical Centrifuge Facility of Hong Kong University of Science and Technology (Ng et al, 2001 & 2002). Three-dimensional model box was used, and its length, width and depth were 1.245 m, 0.99 m, and 0.85 m, respectively. The g-level used in this test was 40g, g = gravitational acceleration.

Figures 1a & b showed plan and elevation views of the centrifuge model. The intersection angle between pipe and tunnel centerline was 90°. The length of the model pipeline was 920 mm which corresponded to 36.8 m in prototype. The model tunnel had an outer diameter (D) of 152 mm and consisted of seven sections. The excavation length of each section was 76 mm (0.5D). The model tunnel was buried in Toyoura sand at a depth of 700 mm, which corresponded to 28 m in prototype. The pipe outer diameter (D_p) and cover depth (C_p) measured to pipe crown were 15.88 mm and 30 mm, respectively, and they corresponded to 0.635 m and 1.2 m, respectively, in prototype, resulting in a pipe cover-to-diameter ratio (C_p/D_p) of 1.9. The outer diameter and cover depth (C) of model tunnel were 152 mm and 225 mm, respectively. They were equivalent to 6.08 m and 9 m in prototype and resulted in a tunnel cover-to-diameter ratio (C/D) of 1.5.



(a) Plan view

(b) Elevation view

Figure 1: Schematic view of centrifuge model

3 MODEL PIPE AND INSTRUMENTATION

One aluminum alloy tube with Young's modulus of 70 GPa was used as the model pipe in the test. The diameter, thickness and length of the pipe were 15.88 mm, 1.65mm and 920 mm, respectively. They corresponded to 0.635 m, 0.066 m and 36.8 m in prototype. To monitor the pipeline responses to the tunneling-induced ground displacement, 17 pairs of strain gauges were mounted at the outer surface along pipe crown and invert for the measurement of longitudinal bending strain. The distance between each pair of strain gauge was 50 mm from center to center.

In addition, as shown in Figure 1, two rows of Linear Variable Differential Transformers (LVDTs) were installed on the ground surface, one right above the existing pipeline and the other at the side of the pipeline. The distance between the row of LVDTs at the side of the pipeline and the pipe centerline was 76 mm (i.e., 4.8 D_p). Previous studies have suggested that ground settlements measured at such distance were not affected by the existence of the pipeline (e.g., Yeates, 1984; Attewell et al, 1985).The ground settlement measured from this row of LVDTs, therefore, can be considered as Greenfield surface settlement.

4 MODEL PREPARATION AND TESTING PROCEDURE

Dry Toyoura sand was used in this test. It is a uniform fine sand consisting of subrounded to subangular particles. It has a mean grain size D_{50} = 0.17 mm, a maximum void ratio of 0.977, a minimum void ratio of 0.597, a specific gravity of 2.65 and an angle of friction at critical state of φ'_{cv}=31°(Ishihara, 1993). In order to achieve a uniformly medium dense sand, Toyoura sand was rained into model container from a hopper keeping a constant distance of 500 mm above sand surface. The relative density of soil sample was 70%.

After the model preparation and a final check, the centrifuge was spun up to 40 g. In flight, the advancement of model tunnel was simulated using seven rubber bags fully filled with water and by releasing a well-controlled amount of water from each rubber bag in each section. The tunnel advancement was modeled in seven successive sections, each of which corresponded to releasing of water from one rubber bag (see Figure 1). The volume loss in each section is controlled as 2% in the centrifuge test.

5 RESULTS OF CENTRIFUGE TEST

All the results presented here are in prototype.

5.1 Surface settlement

Figure 2 shows variation of surface settlement measured from the LVDTs located at the side of pipeline. As discussed before, this surface settlement can be considered as Greenfield surface settlement. Excavations of all sections are successful except for section 6. Note that transverse surface settlement increases with advancement of tunnel construction. The maximum surface settlement is 27.6 mm. When the tunnel face approaches the locations of LVDTs, the measured settlements increase rapidly. Then, the settlements only increase slightly when the tunnel face advances far beyond the monitoring section. The maximum incremental settlement occurres when tunnel face was located exactly underneath monitoring section. When the tunnel face is located from -1.25D to +1.25D, 95% of total settlement occurs. The influence zone, therefore, can be identified as -1.25 D to +1.25 D. Within this influence zone, 83% of total settlement occurs when the tunnel face is located from -0.75D to +0.75D.

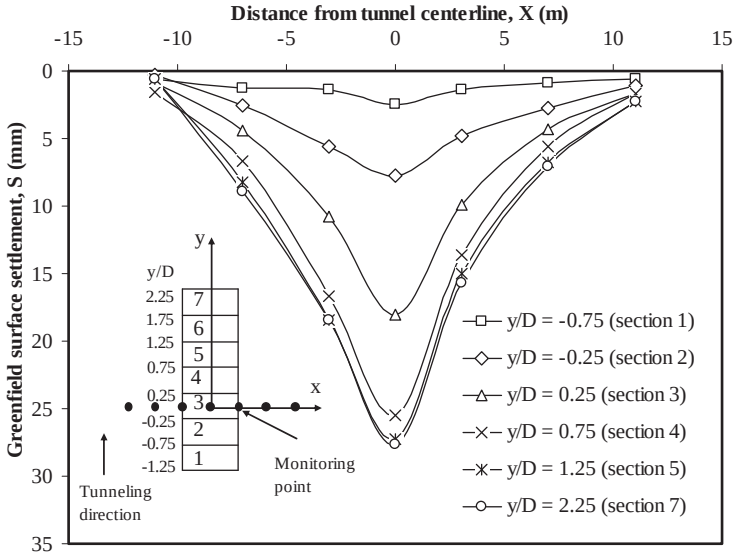


Figure 2: Variation of greenfield surface settlement

Figure 3 shows variation of ground surface settlement measured above the pipeline. The ground settlement patterns are similar to those shown in Fig. 2 for the Greenfield condition. The maximum induced surface settlement is 20.0 mm. The incremental settlements are about 0.8 and 0.9 mm when tunnel face is located from -1.75D to -1.25D (section 1) and from +1.25 D to +1.75 D (section 7), respectively. The surface settlements induced by excavations of sections 1 & 7 are about 5% of total settlement, so the influence zone can be identified as -1.25 D to +1.25 D. Within this influence zone, 75% of total surface settlement is induced

when tunnel face is located from -0.75 D to +0.75 D. Compared with Greenfield settlement, surface settlement above existing pipeline is much smaller. This is due to stiffening effects provided by the existing pipeline.

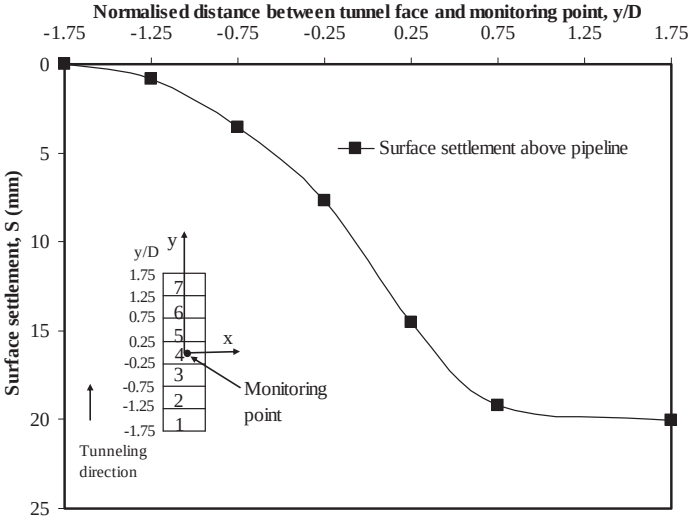


Figure 3: Variation of surface settlement above pipeline

5.2 Longitudinal bending strain along pipeline

Figure 4 shows variation of the longitudinal bending strain along the pipeline during tunneling. The pipe is symmetry with respect to tunnel centerline, and as expected that the profiles of longitudinal bending strain along the pipeline are also symmetry. Sagging moment occurs at the center of pipeline, while hogging moment occurs at other regions. The maximum pipe strain (ε_{pmax}) at sagging regions is about two times of that at hogging regions. It is therefore conservative to use the maximum pipe strain at the sagging regions as the design parameter.

When the tunnel face approaches the monitoring section, measured bending strain along the pipeline increases rapidly. Then, the bending strain only increase slightly when the tunnel face advances far beyond the monitoring section. The maximum incremental strain occurs when the tunnel face is located exactly underneath monitoring section. This is consistent with the variation of ground surface settlements. The maximum pipe strain is 203uε. Only 1.8% of total bending strains is induced by excavation of section 1. In addition, excavation of section 7 does not induce any additional bending strain. Therefore, the same influence zone (i.e., -1.25D to +1.25D) as that for the ground settlement can be identified accordingly. Within this influence zone, 92% of total strain occurs when the tunnel face is located from -0.75D to +0.75D.

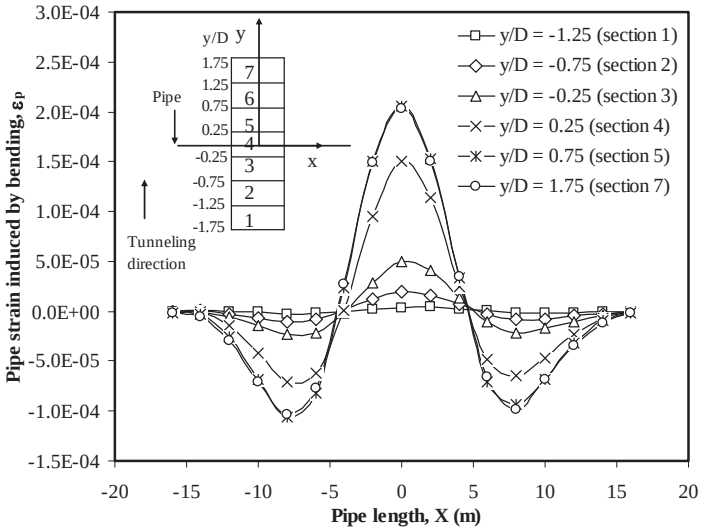


Figure 4: Variation of pipe longitudinal strain

5.3 Validation of numerical results

Figure 5 shows the regression curves recently developed by Wang et al (2011) and comparison of centrifuge and field test results reported in literature. E_pI_p is pipe flexural stiffness, S_{max} and i are the maximum settlement and width of the Gaussian settlement trough. K_u and K_d are soil subgrade modulus, and they are functions of soil internal frictional angle, soil density, pipe diameter and burial depth. The maximum ground and pipe curvatures can be calculated as S_{max}/i^2 and $2\varepsilon_{pmax}/D_p$, respectively. Details of the regression curves can be found in Wang et al (2011).

Because the measurements of the Greenfield settlement and bending strain along the pipeline are located at sections 3 and 4, respectively (see Figure 1), the measurements of the Greenfield settlements correspond to the measurements of the bending strain at the following section. For example, the Greenfield settlement at section 2 corresponds to the bending strain at section 3. The measured Greenfield surface settlements at sections 2, 3, and 4 are well fitted by a Gaussian function with respective S_{max} and i values of (7.8 mm, 4.48m), (18.0 mm, 3.56 m) and (25.2 mm, 3.62 m). This leads to the maximum ground curvatures of 3.89E-4 m^{-1}, 1.42E-3 m^{-1} and 1.92E-3 m^{-1} for the measured Greenfield settlement at sections 2, 3, and 4. The corresponding bending strain measurements along the pipeline are sections 3, 4, and 5, and the corresponding maximum pipe curvatures are 1.45E-4 m^{-1}, 4.63E-4 m^{-1} and 6.19E-4 m^{-1}, respectively. The ratios of max pipe curvature to max ground curvature therefore are 0.373, 0.326 and 0.322.

The burial depth to pipe centerline is 1.52 m, and the mean effective stress is about 16 kPa based on an assumption of an at-rest lateral earth pressure coefficient of 0.5. Based on the triaxial test results from Fukushima & Tatsuoka (1984), soil peak friction angle is estimated as 43°. The values of K_u and K_d are 2.25×10^3 kPa and 3.12×10^4 kPa, and the relative pipe-soil stiffness (i.e., $[E_pI_p/(K_u^{0.9}K_d^{0.1}i^4)](S_{max}/i)^{0.5}$) of these three cases are 1.20E-2, 5.13E-02, 5.63E-02, respectively. The centrifuge test results are plotted in Fig. 5, and they are in good agreement with the regression curves developed by Wang et al. (2011). In addition, Fig. 5 also includes field and centrifuge results from Takagi (1984) and Vorster (2005). It is evident that the regression curves are consistent with the results from field and centrifuge tests, and they can be used to directly estimate the pipe responses to tunneling-induced ground movements.

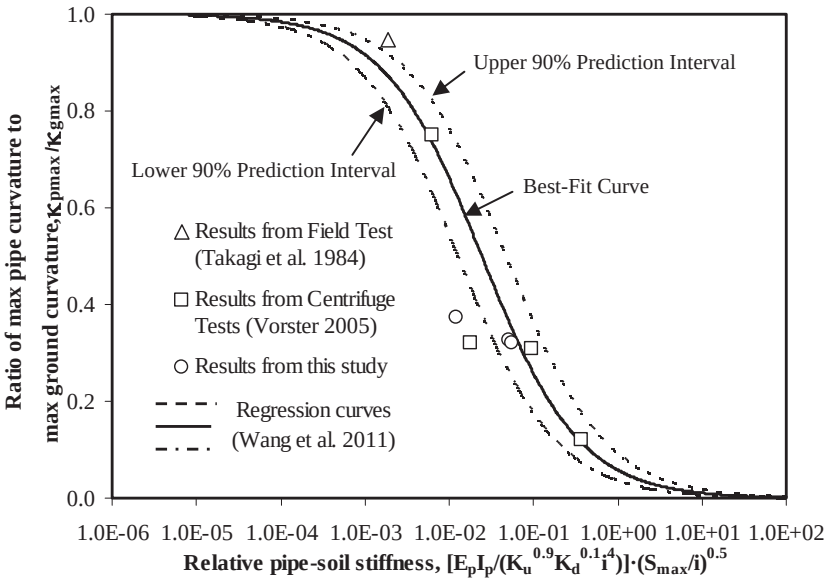


Figure 5: Validation of numerical results

6 CONCLUSIONS

This study carried out one three-dimensional centrifuge test to investigate the pipe responses to tunneling-induced ground movements. The following conclusions can be drawn.

(1) Based on ground surface settlements and pipe longitudinal bending strain, influence zone can be identified as -1.25 D to +1.25 D. Within the influence zone, 83% of Greenfield surface settlement, 75% of surface settlement above existing pipeline and 92% of bending strain occur when the tunnel face is located from -0.75D to +0.75D.

(2) Sagging and hogging moments occur at the center of pipeline and other regions, respectively. The maximum pipe strain at sagging region is about twice of that at hogging regions and it is conservative to use the maximum pipe strain at sagging regions as the design parameter.
(3) With known ground settlement profile, pipe dimension, pipe material properties, pipe burial depth, and soil properties, pipe strain can be directly estimated by using the proposed regression curves. The centrifuge test results have shown that the regression curves provide accurate estimate of pipe responses to tunneling-induced ground movements.

ACKNOWLEDGEMENTS

The work described in this paper is supported by two research grants [Project No. 9041260 (CityU 121307) and Project No. 617610 (HKUST)] from the Research Grants Council of the Hong Kong Special Administrative Region, China. The financial supports are gratefully acknowledged.

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Passive Failure Mechanisms and Ground Deformations of Shallow Tunnel in Sand and Clay in Centrifuge

K.S. Wong & C.W.W. Ng
Department of Civil and Environmental Engineering, Hong Kong University of Science and Technology, Clear Water Bay, Kowloon, Hong Kong SAR

ABSTRACT

There has been an increasing demand for new tunnels in respond to rapid growth and needs in urban areas. It is well recognised that tunnel face stability is vital for the safe tunnel construction in soft ground, particularly at shallow depths. Numerous studies have been carried out to investigate active failure mechanisms of tunnel face in sands and clays. Relatively speaking, fewer researches have been conducted to investigate passive failure mechanisms and ground deformations in front of a tunnel face. The objectives of this paper are to study possible mechanisms of passive failure and ground deformation of shallow tunnel face using earth pressure balance or slurry shield in sand and clay. Two centrifuge model tests were carried out to simulate a 4.46 m diameter tunnel located at cover over diameter (C/D) ratios equal to 2.1 and 2.2 in clay and sand, respectively. In each test, a hydraulic piston (i.e., tunnel face) was used to simulate and create passive failure due to shield tunneling. Passive failure load was measured by a load cell located behind the tunnel face. The particle image velocimetry (PIV) and close-range photogrammetry was used to capture passive failure mechanisms. In this paper, observed failure mechanisms, measured passive failure pressures as well as ground deformations are reported and discussed.

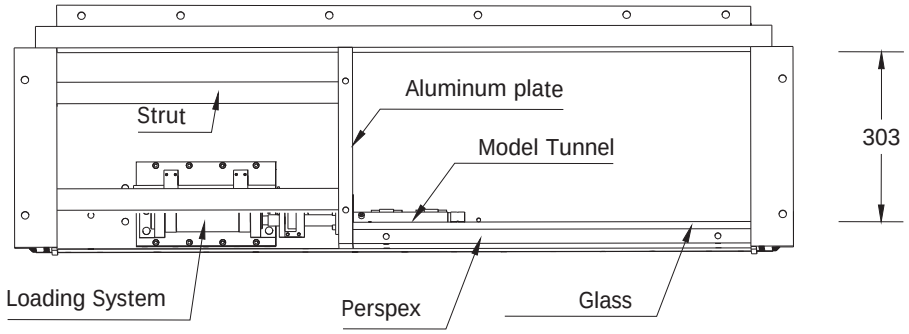
1 INTRODUCTION

There has been an increasing demand for new tunnels to meet the demands of transport systems and underground utilities in respond to rapid growth in urban areas. Some of tunnels are located in shallow depth because of accessibility, serviceability and economy. It is well recognised that tunnel face stability is vital for the safety of tunnel construction in soft ground. The shield tunnelling method is commonly used in soft ground to improve stability and safety. Since water and earth pressures at the tunnel face have to be balanced by a supporting medium, either pressurized fluid for slurry shield or spoil of excavated soils for earth pressure balance shield are commonly adopted. No matter what type of shield is used, the design and control of applied pressure at an excavated tunnel face require the pressure to be large enough to maintain face stability (i.e., preventing active failure) but not too large to avoid blow-out at the face (i.e., passive failure). Over the decades, many studies have been carried out to investigate active failure mechanisms of tunnel face in sands and clays. Relatively speaking, fewer researches have been conducted to investigate passive failure mechanisms and ground deformations in front of a tunnel face. The objectives of this paper are to study and compare passive failure mechanisms of shallow tunnel face and to investigate ground deformations in sand and in clay. In this paper, observed failure mechanisms, measured passive failure pressures as well as ground deformations will be reported and discussed.

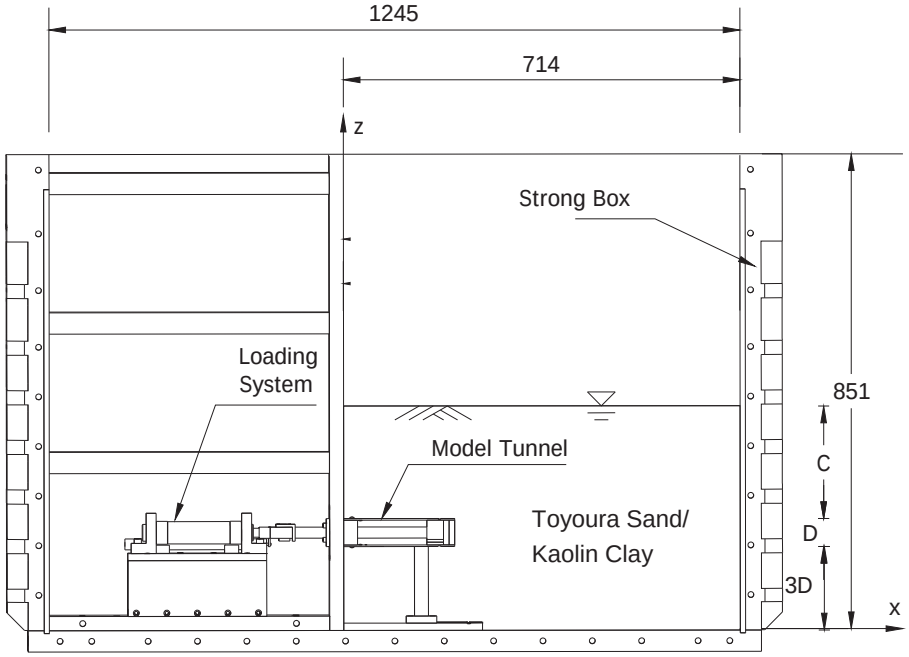
2 CENTRIFUGE MODELLING

Two centrifuge model tests were carried out in the Geotechnical Centrifuge Facility (GCF) at the Hong Kong University of Science and Technology (HKUST). The geotechnical centrifuge at HKUST is equipped with a unique biaxial hydraulic shaker (Ng et al, 2004) and a computer controlled four-axial robotic manipulator. (Ng et al, 2001). The 4.46m diameter tunnel was located at a cover over diameter (C/D) ratios equal to 2.1 and 2.2 in clay and sand, respectively.

Figure 1 shows the centrifuge model package used in this study. A rectangular model container was used in the centrifuge model tests. The model container had an internal plan area of 1245 mm by 350 mm and an internal height of 850 mm. An acrylic viewing window was fitted to the front wall of the container. A 12.7 mm thick glass, measuring 850 mm by 714 mm, was bolted to 25.4 mm thick perspex with similar dimensions to form a composite panel. The composite panel was attached to the front wall. The side of the composite panel that was in contact with the soil formed a vertical plane of symmetry. In clay, the composite panel was replaced with a piece of perspex of similar dimensions. An aluminum plate braced by six struts was used to separate the soil from the loading system.



(a) Plan view



(b) Elevation

Figure 1: Basic configuration of a centrifuge model
Note: All dimensions in millimetres.

By making use of plane of symmetry, only half of the tunnel was modelled in the tests. To simulate and create passive failure due to shield tunnelling, a hydraulic piston in the loading system was used to push the tunnel face towards soil at 0.2 mm/s when the centrifuge speed was maintained at a radial acceleration of 100g. A load cell located behind the tunnel face was installed to measure the tunnel face pressure. Particle image velocimetry (PIV) and close-range photogrammetry (White et al, 2003) was used to capture the failure mechanisms on the vertical plane of symmetry. Linear variable differential transformers (LVDTs) were installed on the ground surface to measure the surface displacement.

For test in sand, Toyoura sand with relative density of 63% was used. The corresponding saturated unit weight, γ_{sat}, is 19.0 kN/m^3. The sand has the maximum and minimum void ratios of 0.977 and 0.597 respectively with a specific gravity of 2.65 (Verdugo & Ishihara,1996). The critical state angle of friction, ϕ_{cs}', for Toyoura sand is 31°. Lightly overconsolidated kaolin clay was used for test in clay. The clay layer has a saturated unit weight of 16 kN/m^3. Following Bolton & Powrie (1987), the critical state angle of friction for kaolin clay is 22°.

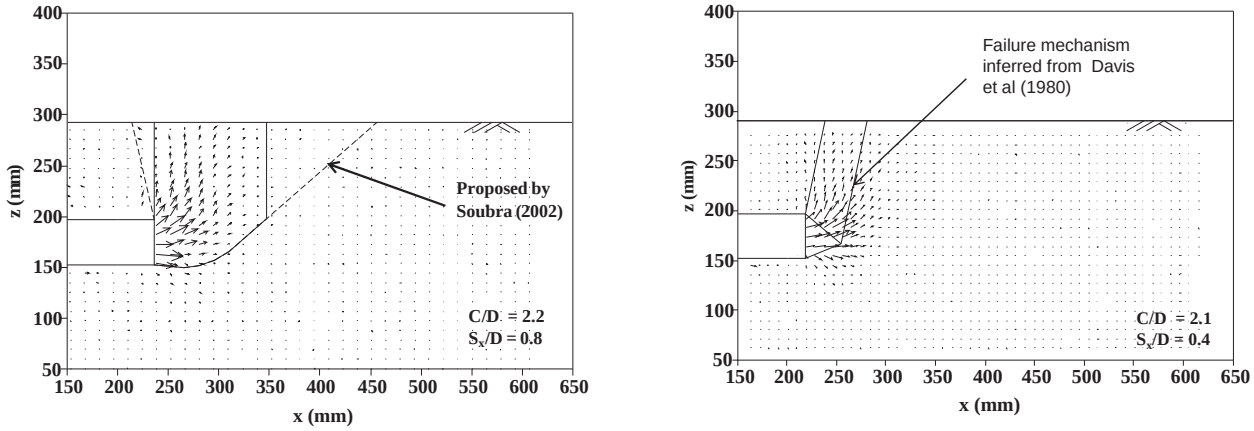
Details of the model setup and preparation as well as testing procedures are given in Wong et al (2012) and Ng & Wong (2012).

3 CENTRIFUGE TEST RESULTS

3.1 Failure mechanism

Figure 2a shows the measured normalised displacement vectors on the vertical plane of symmetry at normalised tunnel face displacement, S_x/D of 0.8 for tunnel located at C/D ratio of 2.2 in sand. The displacement vectors are normalised by the tunnel face displacement. The ordinate system adopted is illustrated in Figure 1. It can be seen from Fig. 2a that the advancing tunnel face displaces the soil in front of the tunnel face whereas the soil further away from the tunnel face is forced upwards to the ground surface. The observed failure mechanism is compared to a five-block failure mechanism (dashed lines in the figure) proposed by Soubra (2002) in obtaining upper bound solutions. It is clear that the proposed five-block failure mechanism is much wider than the observed failure mechanism. Based on the measured displacement vectors, an alternative funnel-type failure mechanism may be postulated as illustrated by the solid lines.

Figure 2b shows the measured normalised displacement vectors on the vertical plane of symmetry at S_x/D of 0.4 for tunnel located at C/D ratio equal to 2.1 in clay. The observed failure mechanism revealed by displacement vectors is somewhat similar to that observed for tunnel located in sand. A narrower funnel-type failure mechanism may be postulated for passive failure in clay. In this paper, a two-block failure mechanism inferred from Davis et al (1980), as illustrated in the figure, is adopted to calculate upper bound passive pressure. The comparison between the measured and the upper solutions is given later.



(a) Sand (b) Clay

Figure 2: Normalised displacement vectors for shallow tunnels located in (a) sand, and (b) clay

3.2 Passive failure pressure of tunnel face

Figure 3 shows the variations of tunnel face pressure, σ_t with S_x/D for shallow tunnels located in sand and clay. For tunnelling in sand, σ_t increases with S_x/D but at a reducing rate. Calculated passive failure pressures by using the lower and upper bound solutions, which were derived by Leca & Dormieux (1990) and Soubra (2002) respectively, are also included for comparisons. The calculated passive failure pressure using the upper bound (UB) solution underestimates the measured failure pressure. As expected, calculated pressure by the lower bound (LB) solution is smaller than the measured failure pressure. This suggests that some improvements on the lower and upper bound solutions may be considered. For tunnelling in clay, tunnel face pressure increases with S_x/D but at a reducing rate and reaches a steady state at about S_x/D equal to 0.2. The

lower and upper bound solutions derived by Davis et al (1980) are used to estimate the passive failure pressures. It is found that the measured passive failure pressure is closely bracketed by the best upper and lower bound solutions.

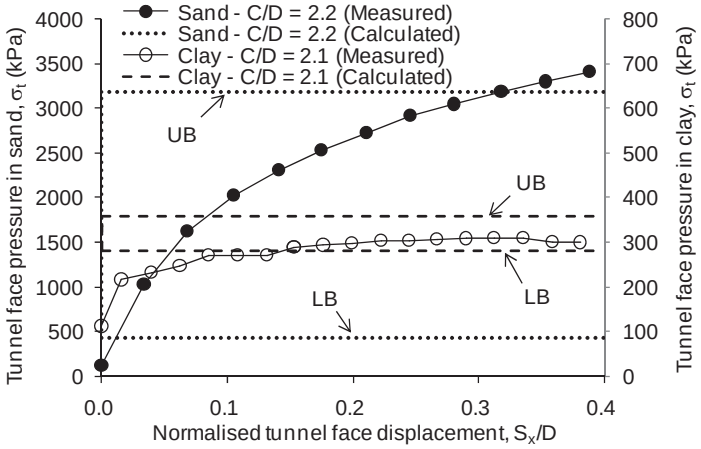


Figure 3: Variations of tunnel face pressure with tunnel face displacement for shallow tunnel located in sand and clay

3.3 Surface displacement

Figure 4 shows the measured normalised surface heave, Δ/D, on the vertical plane of symmetry for shallow tunnels located in sand and clay, at normalised tunnel face displacement, S_x/D of 0.3. The measured surface heave along the longitudinal direction was obtained from the PIV analyses and LVDTs in sand and clay respectively. Gaussian distributions are obtained by setting K equal to 0.27 and 0.4 in sand and clay, respectively. Mair & Taylor (1997) found that the K value varies from 0.25 to 0.45 and 0.4 to 0.6 for sand and clay, respectively. The adopted K values in obtaining the Gaussian distributions fall within these ranges. The maximum Δ/D used in the Gaussian distributions is deduced from measured heaves, which are 2% and 1.3% in sand and clay respectively.

For tunnelling in sand, the extent of heave is at about 4D in front of the initial position of the tunnel face while the location of the maximum heave is at 1.7D from the initial position of tunnel face. Although the extent of heave in clay is also at about 4D in front of the initial position of the tunnel face, it extends 2D behind the initial position of the tunnel face. The location of the maximum heave is at 1.1D from the initial position of the tunnel face. The measured heave in sand and clay can be well described by the Gaussian distributions. It can be seen from the figure that surface heave ridge induced by tunnelling in sand is narrower and the peak heave is about 50% higher than that observed in clay. The observed difference in the amount of heaves in sand and clay might be attributed to the strong dilation in sand and the consolidation settlement in clay during the advancement of tunnel face to 0.3D (i.e., about 8 days in prototype). Based on the measured long-term settlements reported by Ng & Wong (2012), the estimated consolidation settlement is about 0.1% of the tunnel diameter.

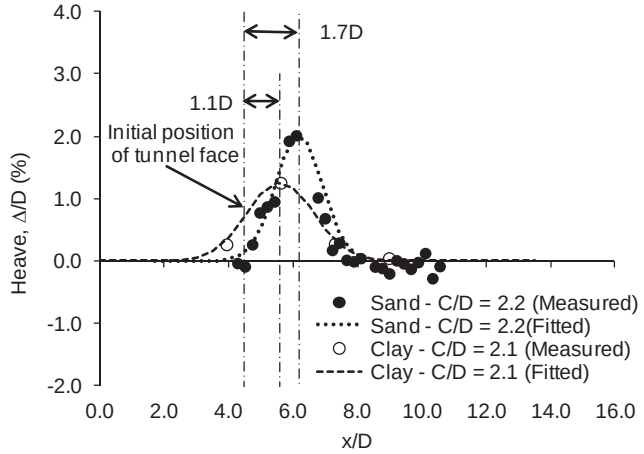


Figure 4: Normalised surface heave at tunnel face displacement of 0.3 for shallow tunnels located in sand and clay

4 CONCLUSIONS

For shallow tunnelling in sand and clay, a funnel-type passive failure mechanism is observed. Existing upper and lower bound solutions appear to be able to estimate passive failure pressure in clay but not very well in sand. For tunnelling in sand, the extent of heave and the location of the maximum heave are at about 4D and 1.7D in front of the initial position of the tunnel face, respectively. Although the extent of heave in clay is also at about 4D in front of the initial position of the tunnel face, it extends at 2D behind the initial position of the tunnel face. The location of the maximum heave is at 1.1D from the initial position of the tunnel face. The maximum measured surface heave is 2% and 1.3% of tunnel diameter in sand and clay respectively. Gaussian distributions can be adopted to characterise the profiles of surface heaves in both sand and clay well.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the research grants 617410 and 617608 from the Research Grants Council of Hong Kong SAR.

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