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Keynote Speech

Tunnel Engineering in Hong Kong – An Outlook

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Mr Chairman, Fellow Members, Distinguished Guests, Ladies and Gentlemen,

I am greatly honoured in having this opportunity to address such a distinguished audience and to share with you some of my experiences and views on the subject of tunnel engineering in Hong Kong.

INTRODUCTION

With our continual growth and increasing ties with Mainland China, there is a strong demand for infrastructure development such as railways, highways, port facilities and public utilities in the territory. Owing to the hilly nature and the large and dense population especially in the urban areas and our new towns, engineers have always been facing with the challenges to preserve the environment and to work in congested developed areas where spaces to cater for developments and redevelopments of our infrastructure are normally very limited. Based on this background, there is an increasing demand to go underground by way of caverns and tunnels to satisfy our needs. Highways, railway and cross harbour tunnels are common features in Hong Kong. Apart from these, perhaps less known to the general public, we also have a large number of tunnels and caverns constructed for various utility services, such as water supplies, electricity ducts, sewage treatment works and stormwater systems. Today, as a professional engineer, it is very seldom that one in his or her career life has not been involved to some extent in some tunnelling works.

In fact, the technologies of tunnel engineering and associated services have also been experiencing the most rapid development in history. Thanks to the increasing awareness of the community in environmental protection and safety, tunnelling engineers have been pushed for making improvement in almost every aspects of the skills and technologies in order to meet the rising standards of the industry. At the same time, the tunnel engineering discipline has also been penetrating into many new areas of applications, to such an extent that the definition of tunnel engineering has undergone evolutionary changes in the last

decade or so and now virtually covers a wide range of techniques including tunnelling in hard or soft ground, cut-and-cover tunnel, micro-tunnelling, mine-tunnelling, pipe-jacking and various trenchless and no-dig technologies. The major techniques commonly employed in the tunnelling projects in Hong Kong are drill-and-blast method, tunnel boring machine (TBM) method and pipe-jacking, whilst various smaller-sized trenchless technologies, for example horizontal directional drilling, are being used increasingly.

DRILL-AND-BLAST

Let me first talk about the drill-and-blast method, which was used in the construction of Cheung Tsing Tunnel which I had been closely involved when I was the Chief Engineer of the Route 3 project in Highways Department in around 1993.

The drill-and-blast method, which dates back to the industrial revolution, has been used extensively and successfully in many infrastructure developments in Hong Kong for over half a century, and therefore should not be unfamiliar to us. The method makes use of the release of energy in a short time when an explosive detonates in a drill-hole in rock. The resulting high pressure due to the rapid expansion of the explosive reaction products gives rise to compression waves in the explosive and the surrounding rock material. These waves cause fragmentation of the rock. Normally a number of drill-holes filled with explosives in a pre-determined pattern are employed to maximize the blasting effect. Owing to the explosion involved in the tunnelling process, the blasting procedures are always under strict control with the blast precisely designed. At the same time, close monitoring on vibration, noise and settlement is also required during construction. As for Cheung Tsing Tunnel, we managed to achieve early completion of this first dual 3-lane 1.6 km long tunnel in Hong Kong with a breakthrough in only 9 months mucking out 400,000 m^3 of rock pieces.

Although not as modernized as the TBM method, the drill-and-blast method offers the advantages of simplicity in operation and higher flexibility in tunnel configuration, with no restraint on the turning radius of the tunnel alignment, and is a relatively cheap method of construction with an established expertise locally available in the industry. As a result, it is still commonly used in tunnelling works in Hong Kong, in particular in locations and areas with a lower sensitivity to vibration and noise arising from explosion.

TUNNEL BORING MACHINE (TBM)

Next comes the TBM method. Despite its relatively short history since the late nineteenth century, the TBM method has been successfully used in numerous major tunnel projects around the world. In Hong Kong, the method has also been employed in many major tunnel projects in recent decades. The TBM

method is a highly mechanized operation, with the machine basically tailor-made to cope with the anticipated ground conditions. The tunnel size of the TBM typically ranges from around 2 m up to well over 10 m, with a variety of designs to suit different ground conditions, such as shielded, double-shielded, slurry-type, earth-pressure-balance-type, hard rock, reamer and multi-head TBMs.

A typical hard rock TBM is a highly sophisticated and versatile invention consisting mainly of a cutterhead at the front, the shield, the main beam, the longitudinal thrust cylinders, the grippers and the conveyor belt for conveying the spoils. During excavation, the rock is cut by a set of roller disc cutters which are mounted on the cutterhead in a specific pattern. When the cutter tip penetrates into the rock face, the rock will fail by shear, with the fragments scooped by muck buckets into the head and conveyed to the rear of the TBM. In advancing the machine, the head is propelled forward off the grippers, which are hydraulically jacked against the excavated tunnel wall. Thrust cylinders between the grippers and the main beam are used to push the main beam and the cutterhead forward against the rock.

The above working mechanism becomes more difficult and complicated when soft ground and high groundwater table are encountered. Under such circumstances, care should be taken to select the suitable mode of operation, including close mode using slurry or steel bulkhead to balance the earth pressure and groundwater ahead of the machine, or open mode using hand tunnelling under either free or compressed air, or a mixed mode of the above two. Depending on the geology and groundwater conditions, the design and operation of TBM very often have to be modified in association with the support and finish of the tunnel so as to suit the changing conditions along the length of the tunnel.

As I have illustrated the drill-and-blast method with a highway project, I wish to quote some TBM projects carried out by the Drainage Services Department. One of the major tunnel projects in Hong Kong using the TBM method is the Harbour Area Treatment Scheme Stage 1, which comprises a deep sewage conveyance system 4 m in diameter with a total length of about 25 km to collect and transfer sewage from Kowloon and the north-eastern areas of Hong Kong Island to Stonecutters Island for centralized treatment. The tunnels mainly go through sound bedrock at depths ranging from 87m to 143m below sea level. Construction works started in 1995 with completion in 2001, at a project cost of HK$8.2 billion. A more recent project is the flood relief Kai Tak Transfer Scheme, which intercepts stormwater at Kowloon Tong and transfers it via a 1.45 km long 4.4 m in diameter tunnel to the Kai Tak Nullah for discharge. Works were commenced in 1998 with completion in 2000, at a total cost HK$336 million.

Putting aside the engineering constraints, it can readily be realized that the TBM method will become more favourable for longer runs, considering that its high erection and capital investment costs can be more evenly spread, hence

resulting in higher cost-effectiveness.

PIPE-JACKING

So much for the big tunnels. As for the smaller sized tunnels, the development in pipe-jacking techniques is equally encouraging. Apart from the obvious advantages in having highly reduced nuisance in terms of traffic disruption and environmental pollution compared with the traditional open-trench method, these sorts of trenchless techniques have been gaining increasing popularity in the installation of various public utility services due to its faster construction and lesser disruption to the public and adjacent properties.

Normal pipe-jacking is to push pipes, usually from around 400 mm to over 2000 mm in diameter, in straight line between two shafts using a steerable shield as lead pipe. The shield acts like a mini-TBM, with similar open, close and mixed mode of bulkhead design to suit different ground conditions. The pipes can be of any material, preferably with smooth finish to minimize resistance during jacking and rigid enough to transfer the thrust force from the jacking shaft to the shield. The resistance to jacking consists of different types of friction forces between the pipes and the soil or rock being encountered plus the required force to advance the shield. The lower the friction forces, the easier it is to advance the shield and make a longer span length of pipe jacking. To achieve this, lubrication materials such as bentonite or slurry are pumped into the excavated cavity surrounding the pipes. Apart from lubrication, the introduction of intermediate jacking stations can also facilitate in extending the span length. The intermediate jacking station consists of a number of smaller hydraulic jacks placed along the circumference of a sleeve to give additional thrust force to advance the shield.

Curved pipe-jacking is increasingly being used to minimize the number of jacking shafts and to suit the surrounding environment, and represents a new challenge to the operating engineers as well as the development of position guidance system. The design and the actual jacking operation have to take into account the soil conditions which should be able to take the additional horizontal reaction forces, the loading at the face of the shield and the appropriate location to start the curve during jacking. Despite the difficulties, the challenge is generally overcome, as reflected by the increasing number of curved pipe-jacking projects being completed around the world together with the use of shorter and shorter radius of curvature – now achieving about 12 times the pipe diameter.

One very successful application of the pipe-jacking method, again by Drainage Services Department, is in the recent construction of trunk sewer in the busiest areas in Wan Chai East and North Point districts. The project, at a total cost of HK$900 million, comprises the construction of trunk sewers at a depth of around 10 to 18 m below ground, involving in between a total of 27 nos. temporary shafts used either as a jacking shaft or as a receiving shaft to enable

the tunnelling works. One sewer, 2.9 km long, is in North Point while another, 1 km long, runs along the busiest streets near the Sogo Department Store in Causeway Bay. I wish to point out that the sewers include a section of S-shaped curve of 404 m length, known to be the first time achieved in Hong Kong. The alignments of the sewers have been carefully chosen to avoid disturbance to the foundations of existing buildings or structures in the areas. A big challenge emerged when the pipes are being jacked between the raking piles of the elevated Canal Road Flyover. Owing to constraints by the heavy traffic and congested underground utilities, the vector magnetic method of geophysical survey was chosen to define the pile locations, with 15 drill-holes sunk to detect through the magnetic effect of the steel reinforcement inside the piles within a 2 m radius. The final sewer alignment was designed to suit the survey results, with a minimum clearance of 450 mm from the piles. With the use of four TBMs ranging in size from 600 mm to 2000 mm, the trunk sewerage system in the area has been successfully completed in good time with the least disturbance to the public.

THE OUTLOOK

Looking at the near future, we shall continue to have a number of upcoming major tunnel projects calling for our participation and application of the latest tunnel engineering techniques, for example the Central & Wanchai Bypass Tunnel and the Central Kowloon Route Tunnel. But the future of these tunnels for various reasons is a bit uncertain. As the Director of Drainage Services, I again focus on the jobs under my control. Firstly, it is the Stage 2A of the Harbour Area Treatment Scheme, which comprises a sewage tunnel system of total length of about 20 km (diameter from around 0.85m up to 4.3m) to collect and transfer sewage from the northern and south-western areas of Hong Kong Island to Stonecutters Island for centralized treatment, with an estimated project cost HK$8.1 billion. In particular, in achieving the 100-year design life, considerations are being made in enhancing the maintenance of the tunnels including the prevention of siltation and corrosion. However, the fate of this project is subject to the acceptance by the community of the need for full recovery of recurrent costs under the Polluter Pay Principle through the sewage services charging system. Secondly, there are three other major drainage tunnel projects to be implemented, namely the Lai Chi Kok Transfer Scheme (4.9m diameter), the Hong Kong West drainage tunnel (6.25m – 7.25m diameter) and the Tsuen Wan drainage tunnel (6.5m diameter), each involving the construction of an extensive land-based tunnel for diverting the intercepted stormwater for off-site discharge so as to relieve flooding in the urbanized areas. These works are tentatively scheduled for commencement in 2007 and completion in 2011, with a total budget of nearly HK$4 billion. The above big tunnels will likely be constructed using TBM and/or drill-and-blast methods.

With the encouraging advancement in the various technologies and techniques in tunnel engineering, we are highly confident that the forthcoming

projects can be implemented smoothly in full compliance with the increasing requirements in safety, health and environmental aspects of the community. However, by looking at the abundant and sometimes painful experiences in various tunnel projects of our predecessors, it is not difficult to realize the importance of holistic planning and control in the management of major tunnel projects, whose success relies on the unrelenting support and input of all associated disciplines and specialists.

In connection with the inherent high uncertainties in underground conditions including geology, groundwater and obstacles, the need of sufficient site reconnaissance and ground investigation should be fully recognized. Comprehensive risk assessment and management are essential rather than any sort of add-on measures. Serious consideration should be given in adopting a formalized process of identifying hazards and evaluating their probability of occurrence and consequences, commonly in the form of a risk register for continuous monitoring through all stages of a project. Various risks identified should be thoroughly assessed, suitably shared among the parties and clearly allocated to the respective risk owners for the best treatment and control. The trend now involves the preparation of a so-called Geotechnical Baseline Conditions report at an early stage to set out the geotechnical conditions anticipated to be encountered during underground construction and provides a basis for comparing the actual conditions encountered with those assumed and allowed for by the contractor during tendering. Such provision not only facilitates the resolution of disputes during construction by clarifying the contractor's entitlement but also enables the procurement of insurance at more reasonable prices based on better-defined coverage. At the same time, the contract strategy should be critically reviewed with a view to ensuring the necessary control by the client whilst at the same time empowering relevant parties, including the contractors and specialists, in tackling with any problems and risks under their purview. In this regard, the characteristic structure of the New Engineering Conditions on risk allocation may offer a basis for deciding and allocating to the party best able to manage, estimate and carry the risks. This will greatly minimize unnecessary contractual dispute. On the other hand, irrespective of the contract form being adopted, stringent supervision of the tunnelling works should be enforced throughout the construction stage. In the process, safety and health of workers should be assigned with the highest priority, with ample allowance for relevant training and safe working systems and environment. Above all, the selection of contractor and specialists should be done through rigorous evaluation, with a view to ensuring their competence and capability in undertaking the job.

Facing the need for infrastructure development in the territory and the ever rising expectation of the public, engineers are obliged to identify the best and optimum engineering solutions, with a view to finding the technically suitable, environmentally acceptable and cost-effective solutions. Under this mission, the state-of-the-art tunnel engineering technologies and techniques have provided

good opportunities for us to explore innovative alternatives in contrast to conventional designs. The concept of interception of stormwater and bypassing the urbanized areas via drainage tunnels for off-site discharge for flood relief purpose under the Kai Tak Transfer Scheme quoted above is a good example. This innovative approach enables the smooth realization of the flood relief project for the benefit of the community, while greatly eliminating the need to carry out extensive open trenches in congested urban areas and the associated nuisances to the public. In Kuala Lumpur, even higher innovation is put into action under the project of Stormwater Management and Road Tunnel, in which a 12.8m diameter tunnel will operate both as a stormwater retention and bypass channel to divert floodwater away from the city centre and, at the same time, as a double-deck toll road facility to relieve traffic congestion during the dry days.

Project management in the construction industry has been well known to be both an art and science, requiring not only skill and knowledge but also engineering judgment and insight. Management of tunnel projects is certainly an even bigger challenge due to the various expertise involved and the inherent high uncertainties. Though not as specialized in tunnel engineering as most fellow engineers here, I hope the above views and outlook based on my experience in previous tunnel projects would offer some enlightenment when you face similar challenges ahead.

Ladies and gentlemen, I am sure that you will learn a great deal on cavern and tunnel engineering from the distinguished speakers later in the day. May I wish you a fruitful and successful experience! Thank you.

INTEGRATING Q-LOGGING WITH SEISMIC REFRACTION, PERMEABILITY, PRE-GROUTING, TUNNEL AND CAVERN SUPPORT NEEDS, AND NUMERICAL MODELLING OF PERFORMANCE.

Nick Barton[1]

Abstract: When the 'Q-system' was launched in 1974, the name referred to rock mass classification, with focus on tunnel and cavern support selection. Since that time 'a system' has indeed been developed. The Q-system now integrates investigation geophysics, rock mass characterization, input for numerical modelling, empirical design of support, and excavation performance assessment. The Q-value has proved easy to correlate with required support capacity, relative cost and time for tunnel construction, seismic P-wave velocity, deformation modulus, cavern deformation, and in modified form with permeability. Recent research has also shown encouraging links between Q, the depth dependant deformation modulus, and the *seismic quality* Q_{seis}, which is the inverse of attenuation. There are also indications that Q has captured important elements of the cohesive and frictional strength of rock masses. The above sensitivities are most likely because Q is composed of fundamentally important parameters that were quantified by exhaustive case record analysis. The six-orders-of-magnitude range is a reflection of the potentially enormous variability of geology and structural geology. Some of the empirical relationships are illustrated with a summary of Gjøvik Olympic cavern investigations, and of the *discontinuum* modelling of performance. The paper concludes with a critical assessment of the potential shortcomings of *continuum* modelling of highly stressed excavations in intact rock, and of shallow excavations in anisotropically jointed rock.

INTRODUCTION

This lecture will be an illustrated journey through some of the useful linkages and concepts that have been absorbed into the 'Q-system' during the last ten years or so. From the outset the focus will be on sound, simple empiricism, that works because it reflects practice, that can be used because it can be remembered, and that does not require black-box software solutions. Some of the empiricism will be illustrated by reference to investigations and to empirical and numerical modelling performed at the Gjøvik Olympic cavern in Norway.

Nature varies a lot and therefore Q does too

It is appropriate to start by illustrating contrasting rock mass qualities. Figure 1 shows a core box from a project that has not been *completed* during ten years of trying. The second project may not be *started* for at least ten years. The first should already have passing high-speed trains, the other high-level nuclear waste some time in the future. They are both from the same country and may have six orders of magnitude contrast in Q-value. A second pair of examples shown in Figure 2, requires a cable car for access on the one hand, and successive boat trips to fault-blocked flooded sections of tunnel on the other.

The contrasting stiffness and strength of intact rock and wet clay is easy to visualize. One may be crushed by one and drowned in the other. There are sad and multiple examples of both in the tunnelling industry. They merit a widely different quality description, as given by the Q-value.

[1] Nick Barton & Associates, Oslo, Norway

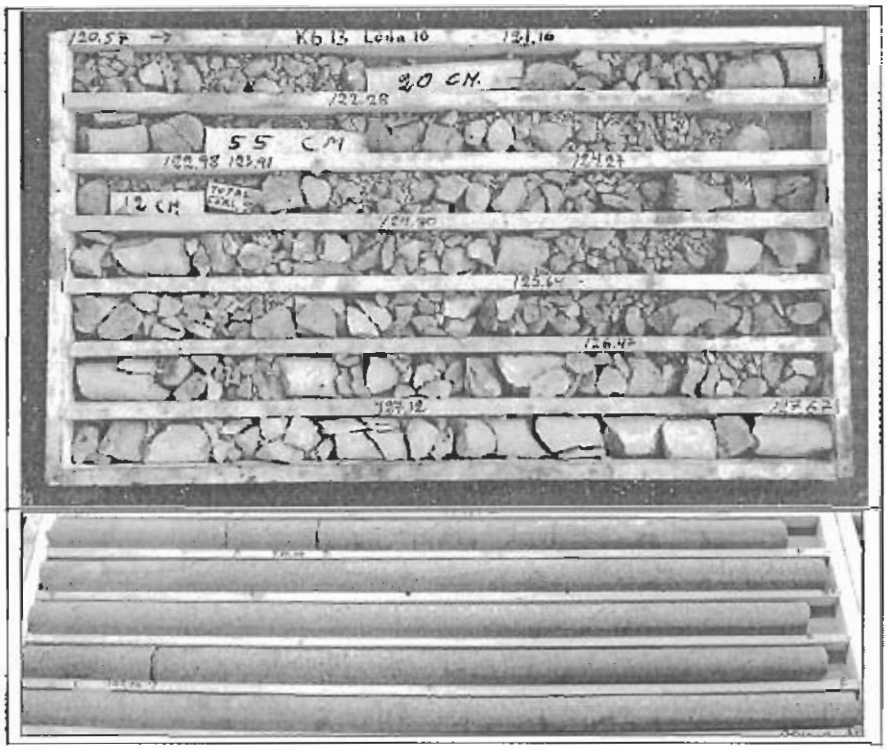


Figure 1 The obvious contrasts of these two core boxes suggest orders of magnitude differences in quality. Quantitative descriptions of shear strength and deformation modulus vary by orders of magnitude as well. Quality descriptors like RMR or GSI that suggest different qualities of about 5 and 95 are not then as appropriate as 0.001 and 1000.

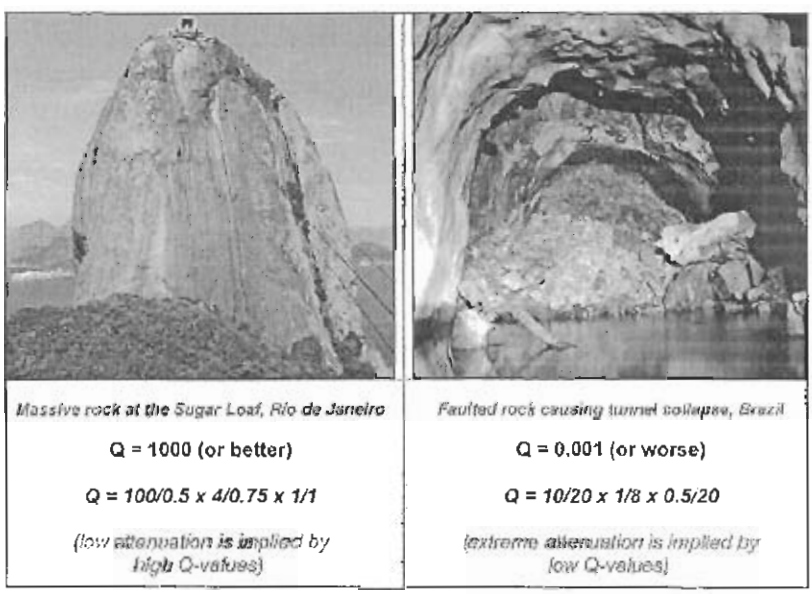


Figure 2 Respective access by cable car and by boat, emphasises the need for radically different magnitudes of 'rock quality', and also radically different magnitudes of 'seismic quality', the inverse of attenuation. Barton (2006). A single project beneath Hong Kong waters, demonstrated a 'length' of unjointed core of 57 m length, and an even wider regional fault zone. With such extremes, RQD values of 100% and 0% are clearly inadequate too, and can be improved by using local Q-values of, for example, 1000 and 0.001.

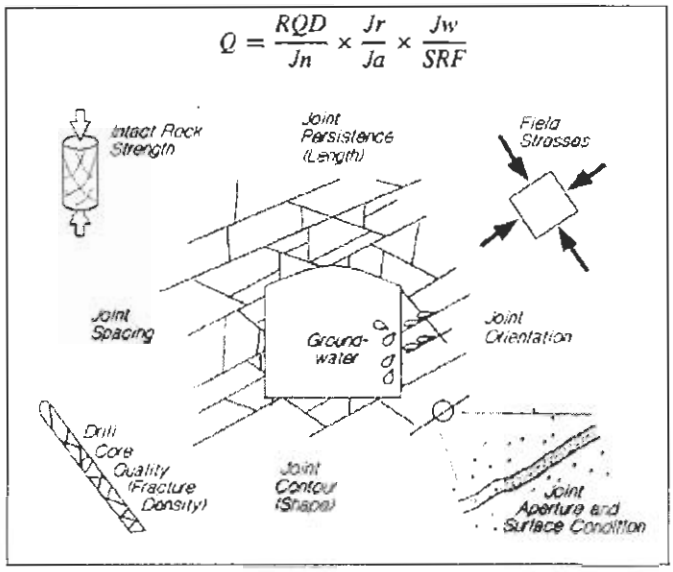


Figure 3 A pictorial impression of the sources of input data that should be captured in the Q formulation. From Hutchinson and Diedericks (1996).

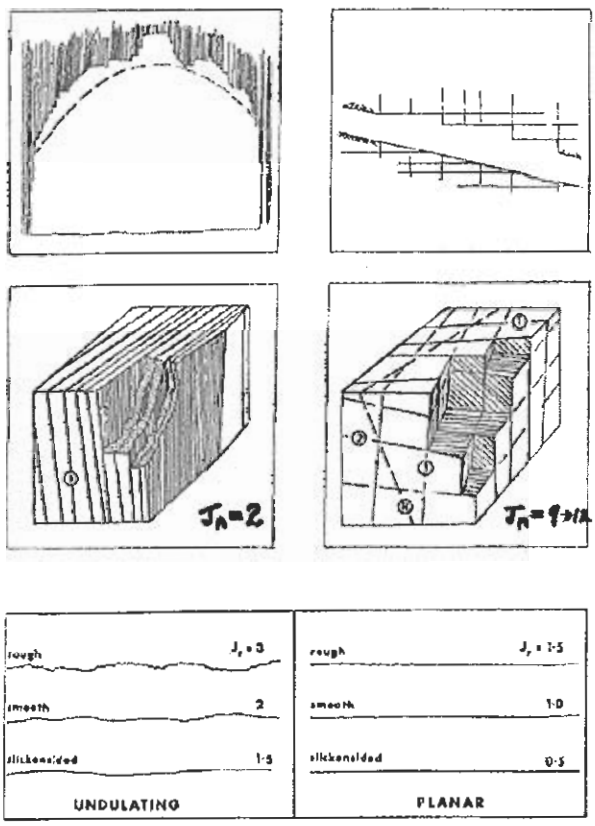


Figure 4 Two of the most important components of Q and of rock mass stability: the number of joint sets (or degree of freedom for block definition and fall-out), and the joint roughness (or inter-block release-or-hold mechanism). The general level of over-break and ease of characterization are also fundamentally affected by these two parameters.

RELATING SEISMIC ATTRIBUTES TO Q

Since there is a limit to how many boreholes can be drilled, how many cores can be logged, and how many permeability tests can be performed, it is useful to have alternative ways of estimating and extrapolating these 'point sources' of information.

We will start by looking at correlation between velocity and measures of quality, with Sjøgren et al. (1979) as a *very* useful starting point for the case of investigations in hard rock, using seismic profiles (totalling 120 km) and local core logging results (totalling 2.1 km of core).

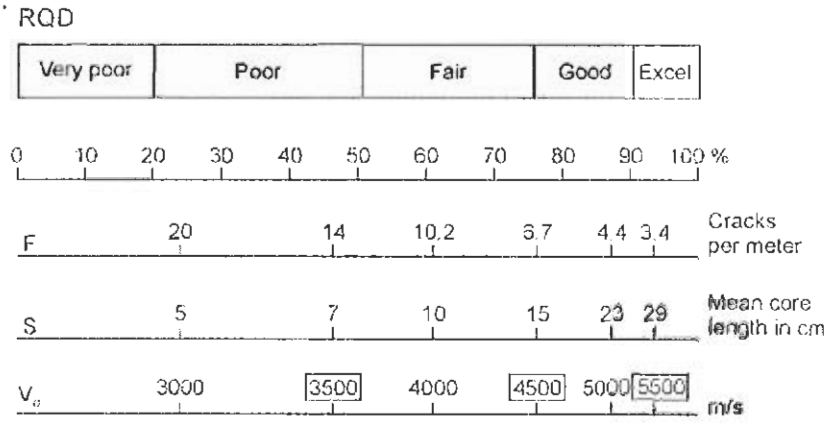


Figure 5 Hard rock, shallow refraction seismic. Sjøgren et al. (1979) combined 120 km of seismic profiles and 2.1 km of core logging to derive these mean trends.

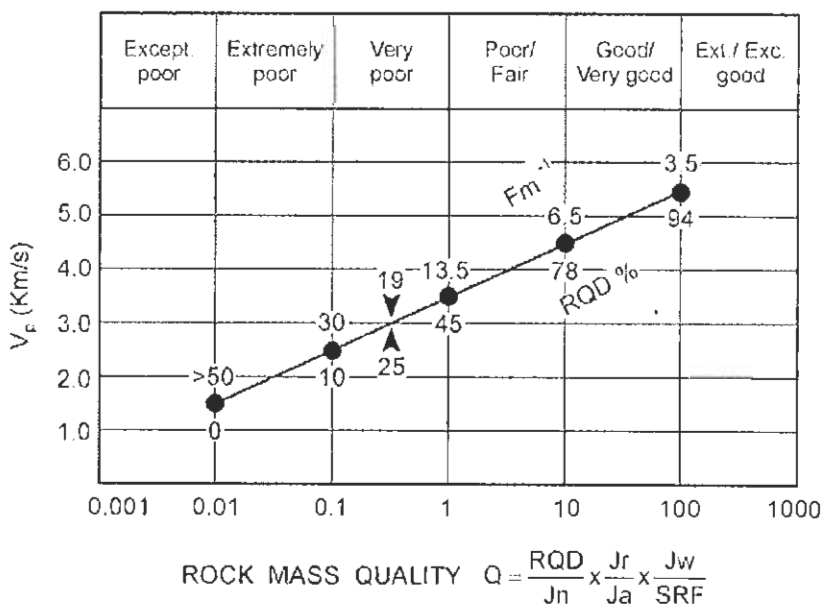


Figure 6 Hard rock, shallow seismic refraction mean trends from Sjøgren et al. (1979). The Q-scale was added by Barton (1995), using the hard rock correlation $V_p \approx 3.5 + \log Q$. By remembering Q = 1: Vp ≈ 3.5 km/s, and V_p = 3 km/s: Q ≈ 0.3, the Q-Vp approximation to a wide range of qualities is at one's fingertips (e.g. for hard, massive rock: Q = 100: Vp ≈ 5.5 km/s, and V_p = 5 km/s: Q ≈ 30).

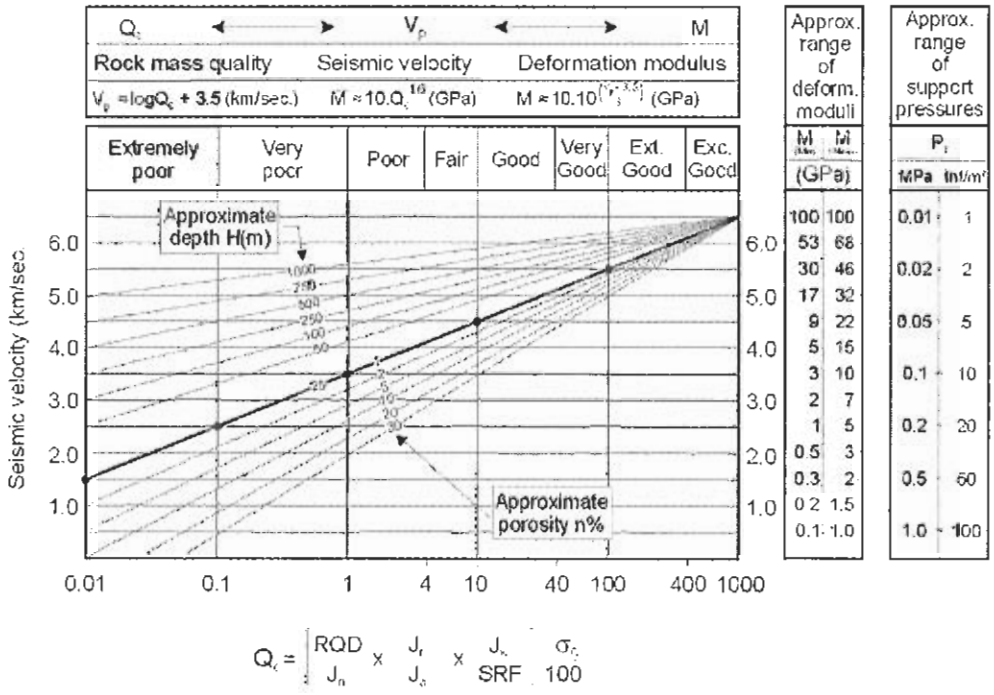


Figure 7 An integrated empirical model for linking Q-value (via Q_c) to P-wave velocity, depth, matrix porosity, deformation modulus, and approximate support pressure (based on a mean Jr = 2). With this simplification, the Barton et al. 1974 support pressure formulation, and the Barton, 1995 deformation modulus formulation suggest inverse proportionality between support pressure and deformation modulus. This is logical, but the simplicity is nevertheless surprising. Barton (2002).

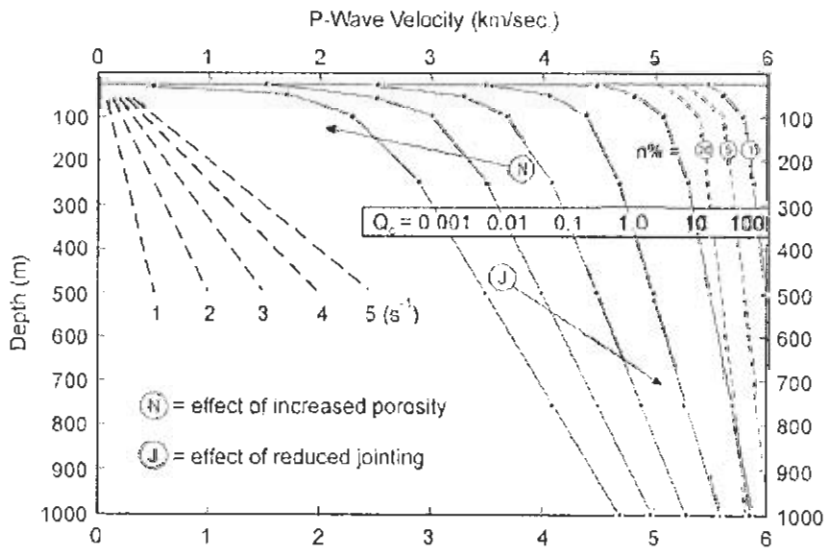


Figure 8 The depth-velocity trends for different Q_c values. This diagram explains why faulted rock ahead of a deep tunnel may sometimes be 'invisible' or of such 'high' velocity, like 4 km/s that it is misinterpreted. It may subsequently cause tunnel collapse, or trap a TBM. In fact such rock is still probably displaying an important contrast to the surrounding rock mass which also shows signs of joint closure. In the case of soft rock, acoustic closure prevents such differentiation. Barton (2006).

RELATING PERMEABILITY TO Q AND TO Q H20

Here we move into more difficult territory, since there are potential problems of flow-channels that have suffered erosion or solution-effects, and there are joints that may be clay-sealed, therefore having both low permeability *and* low Q-value.

For hard, low porosity, jointed rock masses without clay, the approximate Lugeon scales shown in Figure 9 may have some practical merit, when 'out in the field' and also away from colleagues who make a living from permeability measurements. They know best, and we need their data too.

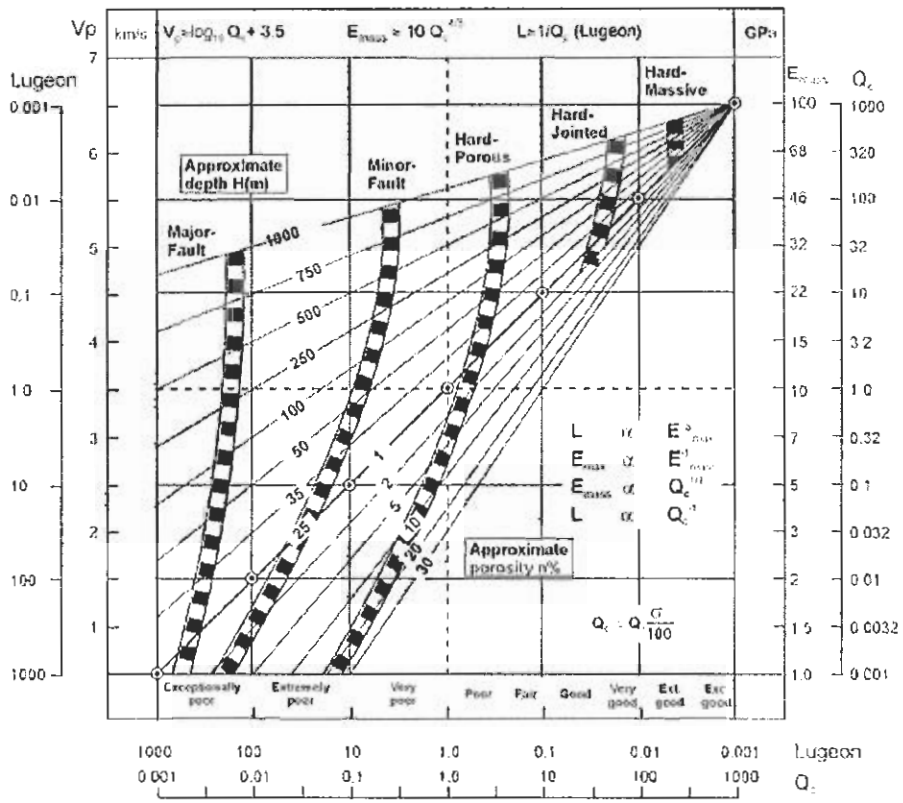


Figure 9 An extension of Figure 8 to include a very approximate estimate of Lugeon value, strictly for the case of rock masses without clay-filling (and therefore flow-blockage) of the joints. For a more general case, the modified term Q_{H2O} is used. This is shown in Figure 10. Note the 'type-curves' for e.g. 'massive rock' and 'jointed rock'. Table 1 shows a collection of potential inter-relationships, where proving them 'wrong' is also useful as anomalies may thereby be uncovered. Barton (2006).

Table 1 A set of inter-related approximations that are useful when assessing results in the field.

Q_c	0.1	1	10	100
Lugeon	10	1	0.1	0.01
K (m/s) ≈	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_p (km/s)	2.5	3.5	4.5	5.5

Table 2 The two versions of 'Q-permeability' estimation. It should not need to be emphasised that both are approximate. Both are presently based on limited test data.

$L \approx 1/Q_c$ (1 Lugeon $\approx 10^{-7}$ m/s $\approx 10^{-14}$ m^2) (hard, jointed, clay-free rock masses)	General case, with depth/stress allowance, and consideration of joint wall strength
$Q_c = RQD/Jn \times Jr/Ja \times Jw/SRF \times \sigma_c/100$	$Q_{H2O} = RQD/Jn \times Ja/Jr \times Jw/SRF \times 100/JCS$ $K \approx \frac{2}{1000 \times Q_{H_2O} \times 10^{5/3}}$

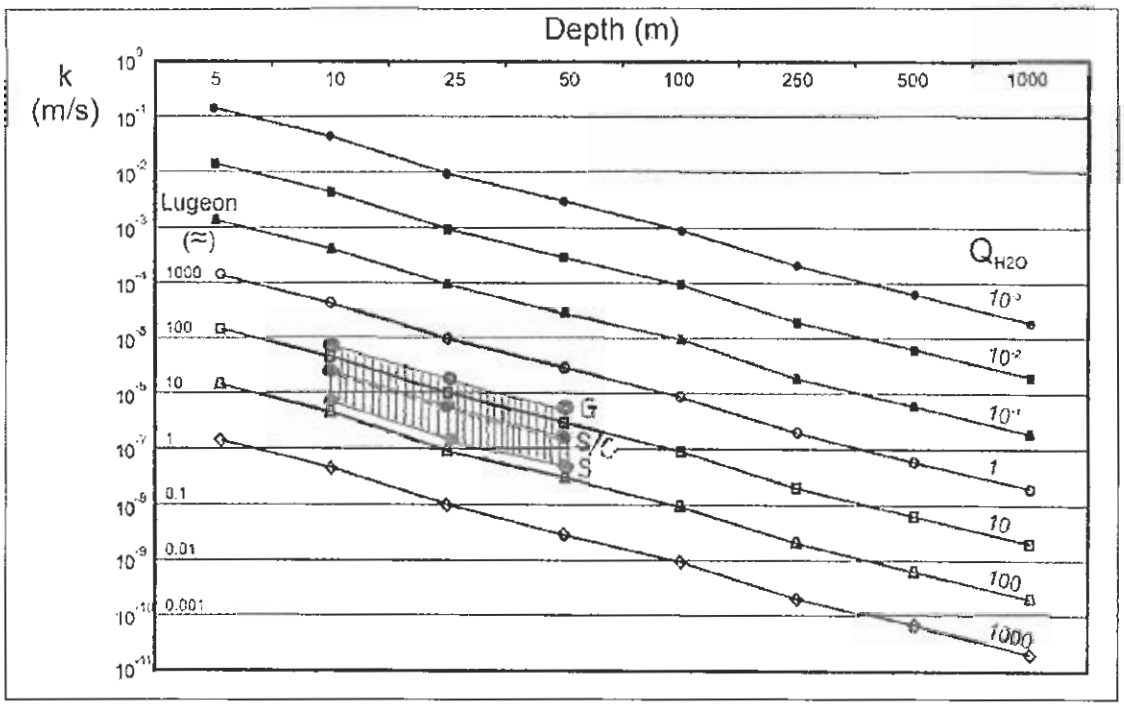


Figure 10 An example of Q_{H2O} and permeability estimation based on more than 1 km of core logging. Independently measured permeabilities ranged from 0.1 to 100 Lugeons, in the depth range 10-50 m. NB&A contract report (2006).

Example of Q_{H2O} estimation

Clay-bearing, well-jointed rock at 100 m depth, with a low assumed JCS of 10 MPa:

Regular Q-value = $\frac{50}{9} \times \frac{1.5}{4} \times \frac{0.66}{1} = 1.4$, i.e. 'poor'

$$Q_{H_2O} = \frac{50}{9} \times \frac{4}{1.5} \times \frac{0.66}{1} \times \frac{100}{10} = 98$$

$K \approx 10^{-8}$ m/s (at 100 m depth)

(Quite low permeability due to clay coatings, and compressible joint walls, despite the extensively jointed nature of this rock mass).

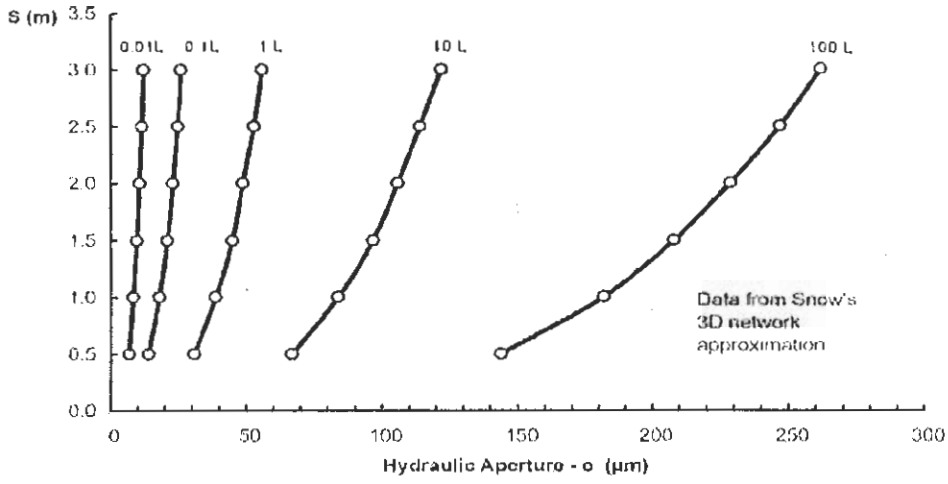


Figure 11 Based on a simplified cubic network model from Snow (1968), it can be shown that K≈10-8 m/s (approx. 0.1 Lugeons), may be composed of (mean) hydraulic apertures (e) of between 15 and 25 μm, for assumed mean conductor spacings of between 0.5 and 3.5 m. Such results suggest the need for high-pressure pre-injection and use of ultrafine cements. Barton (2004).

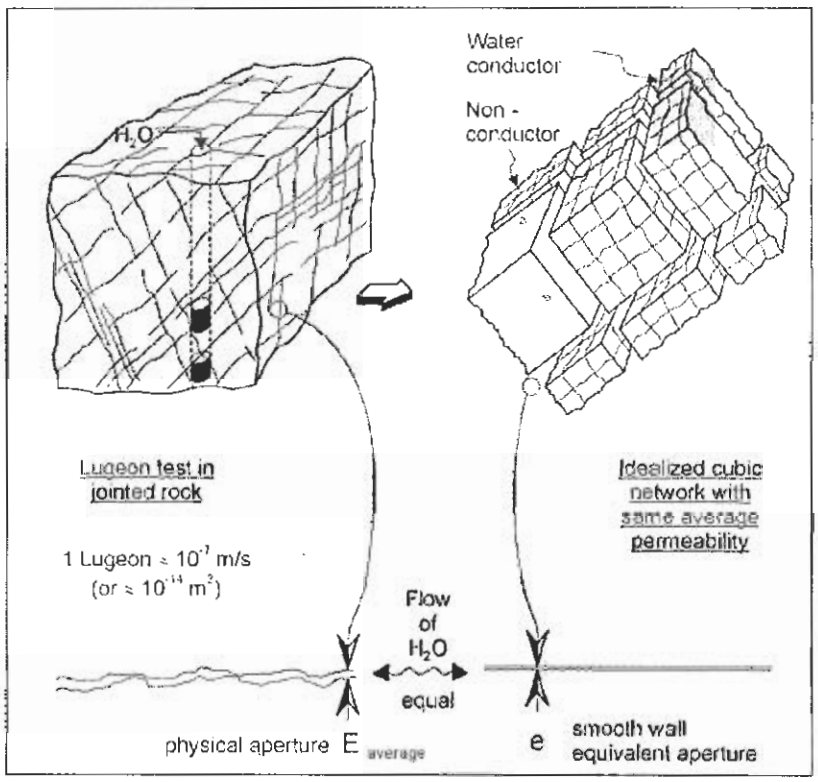


Figure 12 A cubic network approximation to flow and leakage in rock masses ('the engineering approach'). The permeability shown in Figure 11 is based on $k=e^2/12$ for one conductor, $K_1=e^2/12 \times e/S$ for one set, and $K_2=2e^3/12S$ for the rock mass, i.e. two of the three sets controlling flow on average. Snow (1968) with additions by Barton (2004). Note $E \approx \sqrt{(e^2 \times JRC^{2.5})}$ and $E \geq 4d_{95}$ for successful grouting – often requiring grouting pressures of 5 to 10 MPa.

Figure 13 A station cavern showing the need of pre-injection for both water control, and reduction of over-break. Permeability measurement and extrapolation using Q_{H2O} can be used for selecting grouting materials and grouting pressures. When Lugeon values are typically from 10 to 100, less difficulties with pre-injection are expected, but consequences of no pre-injection are greater.

THE BASIC TUNNEL AND CAVERN SUPPORT CHART, Q AND RMR CONVERSION

There is such widespread use of RMR, often in parallel with Q, that it is appropriate to address possible inter-relationship between the two. This of course has been the subject of a lot of publications. One 'camp' utilizes the 'ln' (natural logarithm) format, as in equation 1, in Figure 14, while the other 'camp' utilize the 'log' format, as in equation 2 in this figure. Since the latter format is simpler, and gives a more tangible range of RMR in relation to the Q-scale (avoiding the negative values that occur below Q = 0.01), it has been used by the writer also in relation to stand-up time and deformation modulus conversion between the two systems.

Stand-up time using simple log Q-RMR conversion

While there are admitted pitfalls when attempting to utilize Q to RMR conversion, due to some serious differences in structure and parameter weightings between the two systems, it is nevertheless considered that the advantages may outweigh the disadvantages. For this reason, both stand-up time estimates from Bieniawski (1989), and deformation moduli trends with RMR have been utilized, in an attempt to add to the tools available. Since we are engineers, and not scientists, our craft is the ability to make realistic approximations, leaving all decimal places on the calculator.

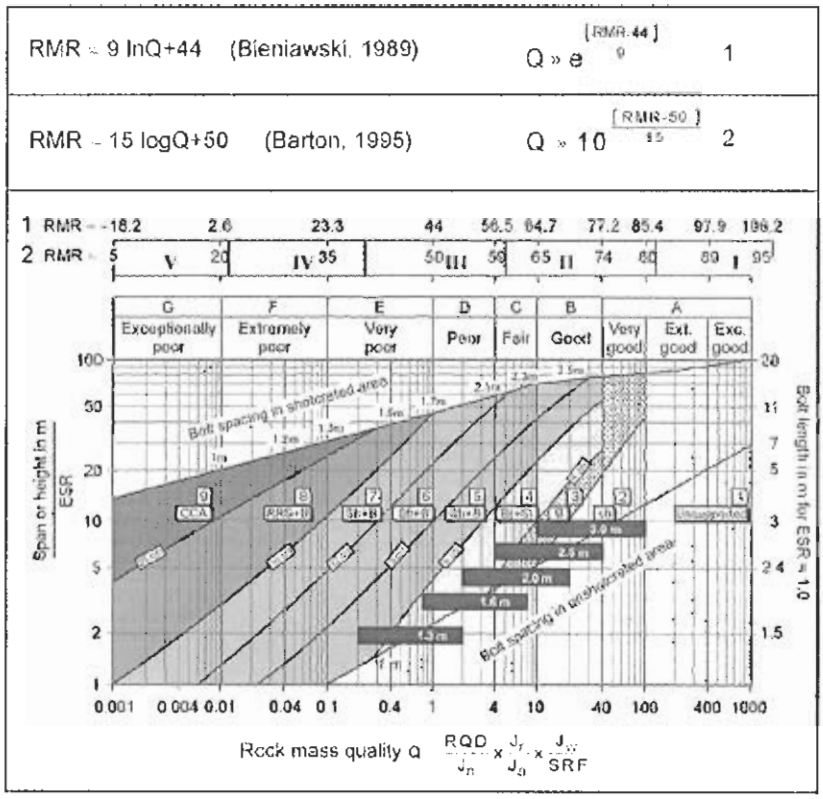


Figure 14 The Q-support chart from the Grimstad and Barton (1993) update for S(fr) in place of S(mr). The black areas referto aspects of TBM tunnel-support, where Q-values are easy to misinterpret due to lack of overbreak (Barton, 2000). The two Q-RMR formulæ refer to the 'standard' Bieniawski (1989) format, and a suggested improvement that is easier to estimate in the field. It also avoids an unwanted negative value of RMR when Q < 0.01.

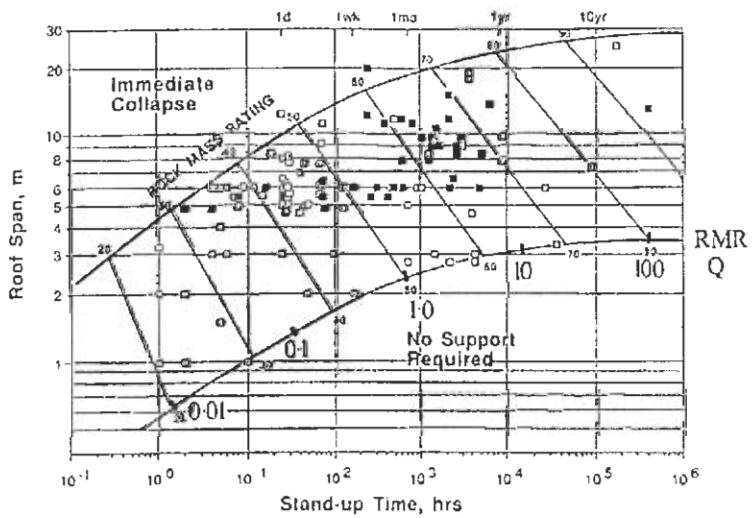


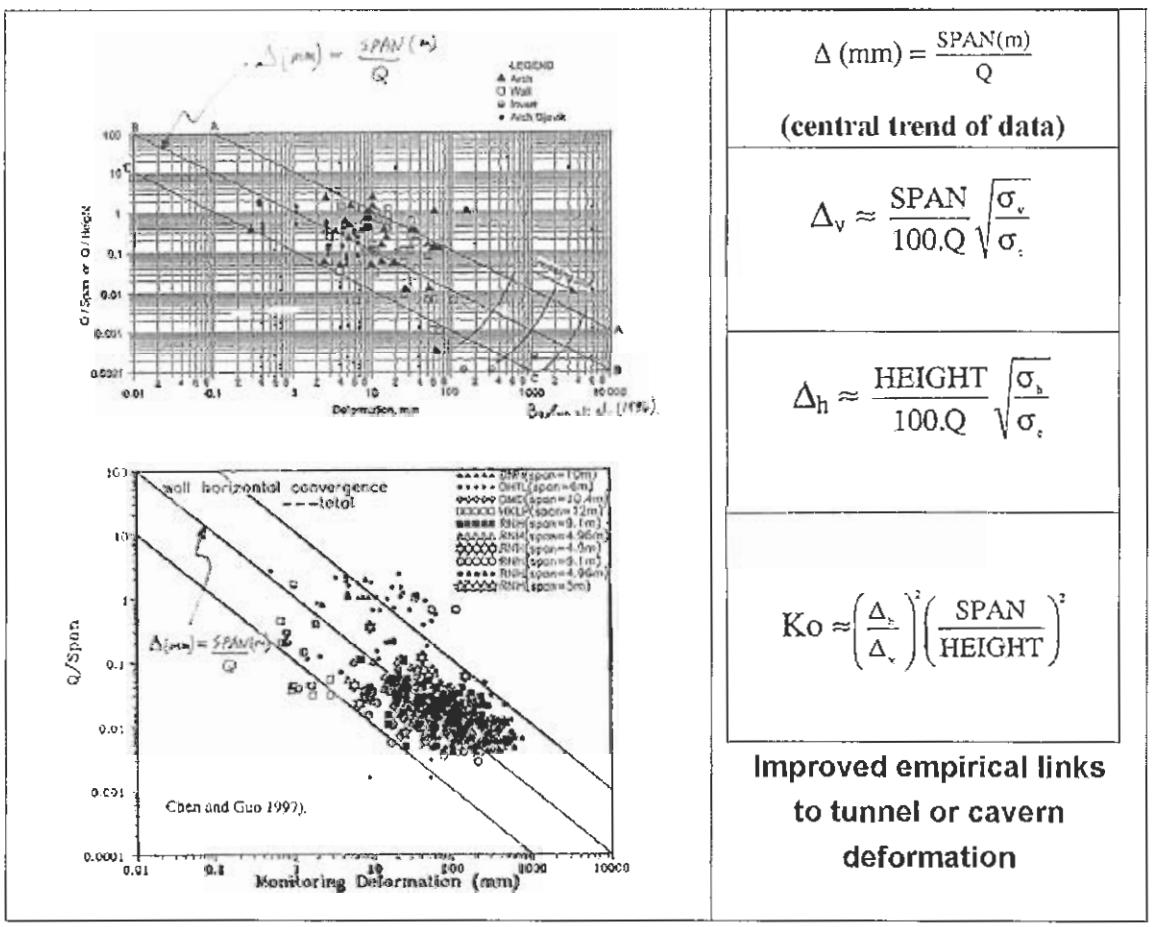
Figure 15 Bieniawski(1989) stand-up time estimations. Note the large Q-value approximations next to the small RMR numbers. The conversion is based on Figure 14 (equation 2).This diagram suggests that when the Q-value is as low as e.g. 0.01, or RMR as low as 20, the stand-up time for a <1m advance may be a matter of minutes, with collapse imminent or immediate if an advance of 2 to 3 m was made – against better judgement, in such poor conditions. Use of spiling combined with rib-reinforced shotcrete RRS arches, and immediate robotic S (fr) application is suggested.

DEFORMATION ESTIMATION BASED ON Q

For many years the writer has assembled information on cavern deformations, cavern size, and Q-values. Barton et al. (1994) showed an updated version, including performance data from various stages of the Gjøvik Olympic cavern excavation. By good fortune, researchers Chen and Guo (1997), followed the same (Q/SPAN) plotting format, and kindly provided the writer with a large set of data from tunnels in Taiwan, driven in significantly poorer rock conditions.

The two sets of widely different deformation data are shown in the left side of Figure 16, with some hand-annotations to emphasise the unexpectedly simple common trend that was discovered when the Taiwan data was added:

$$\Delta \text{ (mm)} = \text{SPAN (m)} / Q$$



$$\Delta \text{ (mm)} = \frac{\text{SPAN(m)}}{Q}$$

(central trend of data)

$$\Delta_v \approx \frac{\text{SPAN}}{100.Q} \sqrt{\frac{\sigma_v}{\sigma_c}}$$

$$\Delta_h \approx \frac{\text{HEIGHT}}{100.Q} \sqrt{\frac{\sigma_h}{\sigma_c}}$$

$$Ko \approx \left(\frac{\Delta_h}{\Delta_v}\right)^2 \left(\frac{\text{SPAN}}{\text{HEIGHT}}\right)^2$$

Improved empirical links to tunnel or cavern deformation

Figure 16 Q/SPAN versus deformation was a plotting format used by the writer for many years. The top-left figure from Barton et al. (1994) includes some Gjovik Olympic cavern data concerning pilot tunnels and top headings. The bottom-left figure from a Chinese language article by Chen and Guo (1997), adds hundreds of data from tunnels in Taiwan that showed significant deformation problems. The equations given on the right-side of the figure show the equation of the central trend line, followed by some empirical improvements for reducing scatter, using the 'competance factor' principle (i.e. stress/strength). This has a similar role to stress/modulus, but is simpler to estimate.

Example application of the deformation formulæ

Nathpa Jhakri hydropower cavern

SPAN = 20 m, Height = 50 m, representative Q-value = 3,

σ_c= 35 MPa, σ_h = 4 MPa, σ_v = 6 MPa

$\Delta_v = 20{,}000/(100 \times 3) \times (6/35)^{0.5} = 28$ mm (i.e. within range of MPBX measurements in arch)

$\Delta_h = 50{,}000/(100 \times 3) \times (4/35)^{0.5} = 56$ mm (i.e. within range of MPBX measurements in walls)

$K_o = [56/28]^2 (20{,}000/50{,}000)^2 = 0.64$ (measured $K_o = 4/6 = 0.66$)

RELATING SUPPORT COSTS AND TIME TO Q

Since the Q-system support recommendations appear to be widely used, it may be of interest to present two independent assessments of the extra costs and time involved, when Q-values are low, compared to the 'plateau' where higher Q-values cause minimised support costs and excavation times.

The first example from an NB&A contract, shows the cost estimates for three sizes of tunnel cross-section, using the quantities of arch *and* wall support given by direct application of Q-system-based 'rules' that were published in the 1970's. (See Table 3).The standard support elements were different combinations of B + S(fr), and the quantities were obtained directly from Figure 14 support chart format.

The permanent support recommendations are strictly following NMT support philosophy, i.e. that combinations of B + S(fr) represent permanent support, when inflow of water has been solved by cheaper means than a concrete lining and membrane. Pre-injection, or free-standing bolted PC-elements with outer membrane, are two solutions to avoid having to design to take full water pressure. Dry-but-drained is the key concept. Barton and Grimstad (1994, 2004).

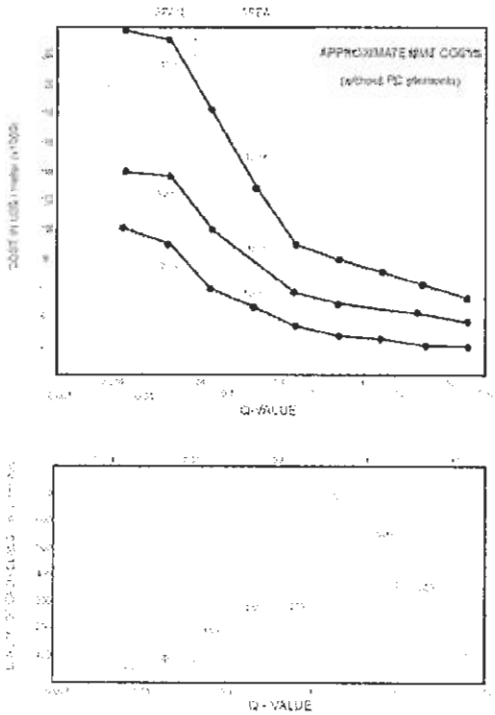


Figure 17 Direct application of Q-system permanent support recommendations in different Q-value ranges, using Norwegian unit prices for B, S(fr), drill-and-blast, mucking and 3 km transport.

In Figure 17, the support costs have a superimposed Q-value statistic based on NB&A logging of numerous boreholes along the tunnel route. The Q-histogram method of collecting and sorting data was used. Total costs for any of the three tunnel alternatives (e.g. ramps, two-lanes, and three-lanes), can be obtained by multiplication of the meters of tunnel of a particular Q-class, and the US $ /m (x 1000) costs.

Table 3 Empirical rules for estimating wall support, and general temporary support.

1.	*Temporary Support*	a) b) c)	increase ESR to 1.5 × ESR increase Q to 5Q (arch) Increase Q_w to $5Q_w$
2.	*Wall Support* (based on modified quality Q_w for walls)	a) b) c)	select Q_w = 5Q (when Q > 10) select Q_w = 2.5Q (when Q < 10) select Q_w = 1.0Q (when Q < 0.1)
Note 1 Use total excavation height (H) for wall support design. Note 2 Q is the general rock quality observed when inspecting the arch *or* walls of a tunnel. For local variations of rock quality (arch or walls), map locally and change support as appropriate. (Q_w is *not* the observed value of Q in a cavern wall.)			

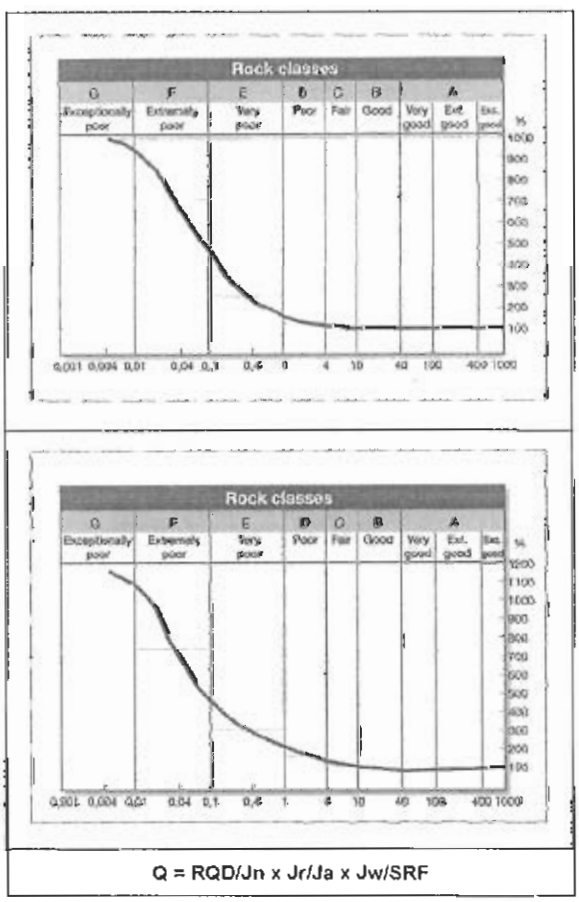


Figure 18 Relative time and cost in relation to Q-values, based on a contractor's calculations for a long 100 m^2 tunnel. These diagrams are useful for arguing the merits of high pressure pre-grouting, which helps to bring the *effective* Q-value down the curve, to the right. Roald et al. (2001). By arguing for small individual improvements in Q-parameter values, it is possible to deduce a 'snowball' increase in effective Q-values as a result of effective pre-grouting. This results in potential increases in seismic velocity, deformation modulus and shear strength, and of course a reduction in permeability. Support needs are reduced: longer rounds can be used.

SOME LESSONS FROM THE GJØVIK OLYMPIC CAVERN INVESTIGATIONS

In the last section of this paper, some of the lessons learned and techniques applied when investigating ground conditions and modelling planned excavation and support of this large cavern will be briefly reviewed. Reference will be made to some of the Q-correlations given earlier in this paper. A start will be made with the comparison of core logging (cross-hatched area: Figure 19) and the Q-logging performed in existing near-by excavations (black area: Figure 19). Note the 'tail' on RQD logged in the case of core, and the lack of a 'tail' logged in existing excavations. This was caused by lack of data where shotcrete covered the poorest rock, when logging existing excavations.

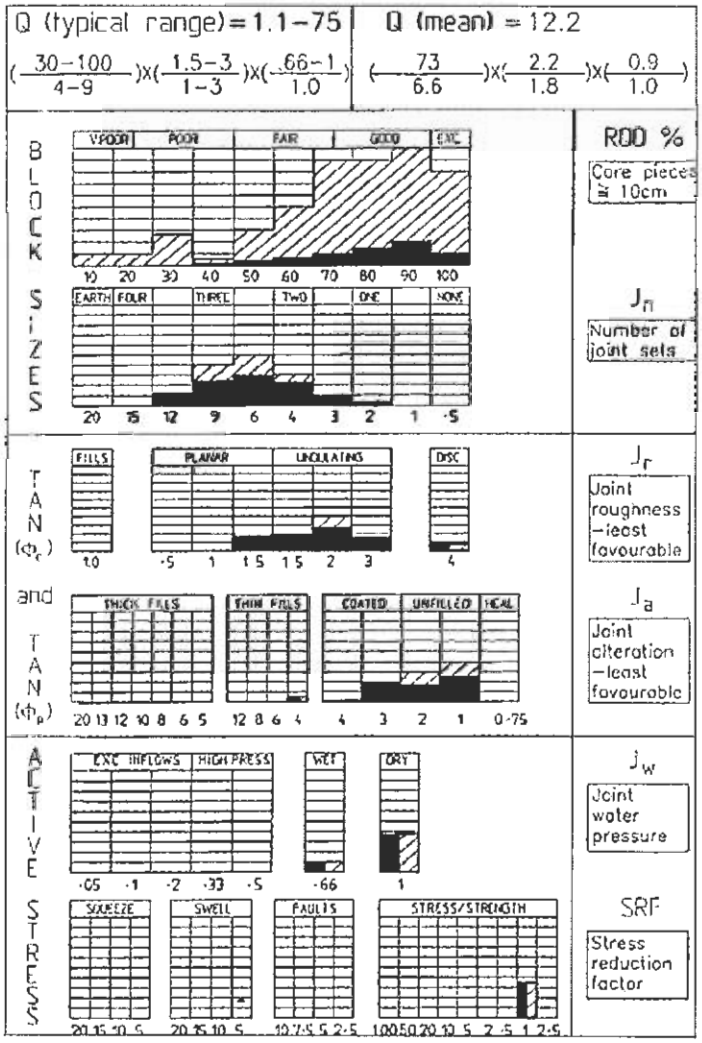


Figure 19 Q-histogram logging of core (4 holes) and existing local excavations, performed by different loggers at different times. Barton et al. (1994).

The boreholes used for core recovery were permeability tested (K mostly $\approx 10^{-7}$ to 10^{-8} m/s), and were also used for cross-hole seismic tomography. Two examples are shown in Figure 20. The expected increase in velocity with depth, from about 3.5 to 5.0 km/s is shown. What was unexpected was that the rock quality (RQD, F m^{-1}, and Q) did not show a corresponding general increase in quality, as can be ascertained by studying Figure 21, which shows the velocities interpreted close to one of the boreholes.

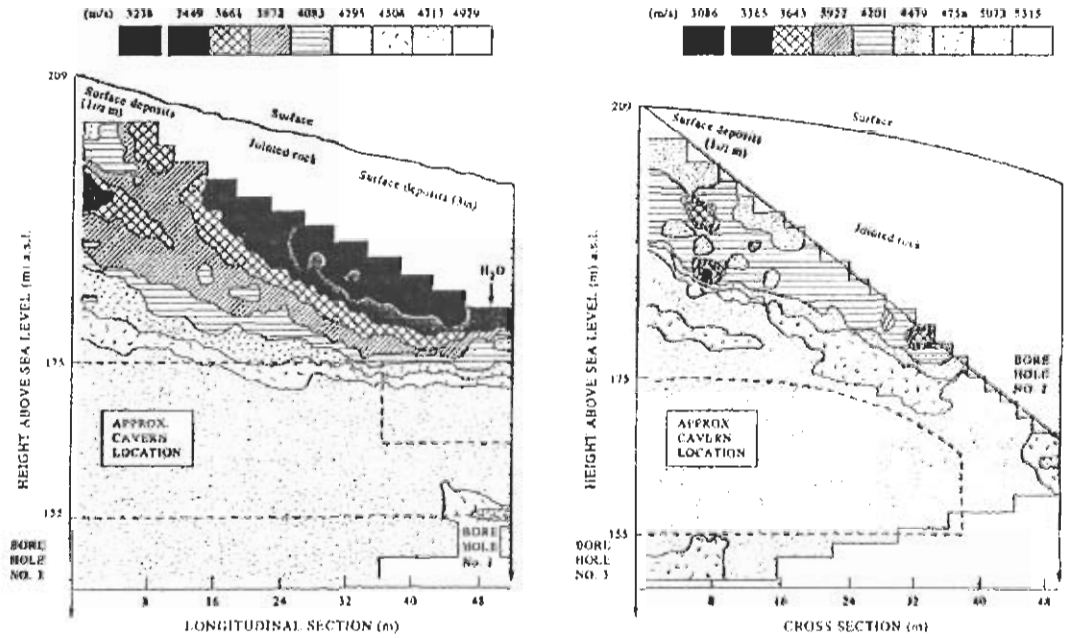


Figure 20 Crosshole seismic tomography between two pairs of holes at the Gjovik cavern site.

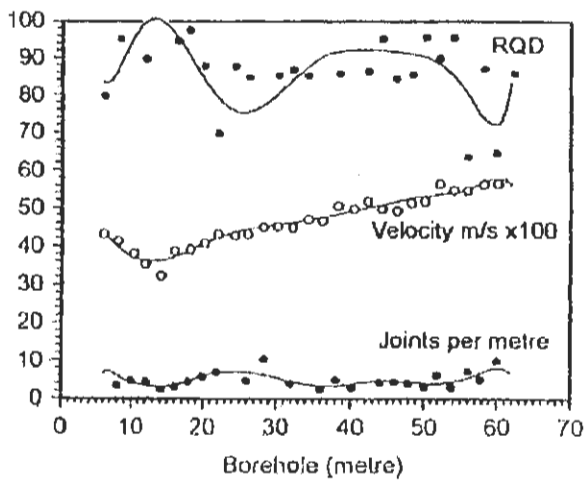


Figure 21 Note lack of a general rock quality improvement with depth, compared with the consistent rise in P-wave velocity. The Q-value logged down the holes mostly varied between 1 and 30, with a mean of 10 to 12, and showed no tendency for improved quality below about 5 m. Barton et al. (1994).

The Q-Vp-depth models shown earlier in Figures 7 and 8 indicate how velocity increase can occur without the need for Q-value increase. However, between 25m and 50 m the predicted increase in velocity is relatively minor, such as 4.5 to 5.0 km/s. The increase of closer to 2 km/s between 10 and 60 m depth suggested in Figure 21, may be explained by a measured or interpreted horizontal stress increase, which was as much as 5 MPa over this same depth range.

This increase, with little assumed change in rock quality, is possible because of the rough state of the conjugate jointing (high Jr and JRC), and due to the relatively sound tectonised gneiss, with UCS about 90 MPa, and JCS about 75 MPa. In softer rock like chalk, 'acoustic closure' (in relation to Vp) occurs at much shallower depths than this.

SOME IMPORTANT ASPECTS OF NUMERICAL MODELLING

When confronted with the modelling of a very large cavern, such as Gjøvik's 62 m span, with (arguably) hundreds of thousands of individual interlocked blocks involved, there are those who would prefer to apply continuum FEM, FLAC or PHASES, using the traditional argument that such a large number of blocks 'cannot' be modelled as a discontinuum, and therefore that a much simpler continuum approach can be used.

A glance at the comparison of numerical models performed by an NGI colleague (Figure 22, from Backer, pers. comm.(1998), should be convincing enough to suggest that little may be learned from the continuum models. Besides differences in mode and magnitude of deformations, there are fundamental differences in the stress distributions too, when modelled jointing is actually capable of normal and shear adjustments, as in UDEC-BB, rather than a pseudo-jointing that does not 'disturb' the continuum modelling.

This should be born in mind when the argument proposed for performing the simpler continuum models is that 'at least you get the stress distribution'. You may not in fact get *realistic* stress distributions.

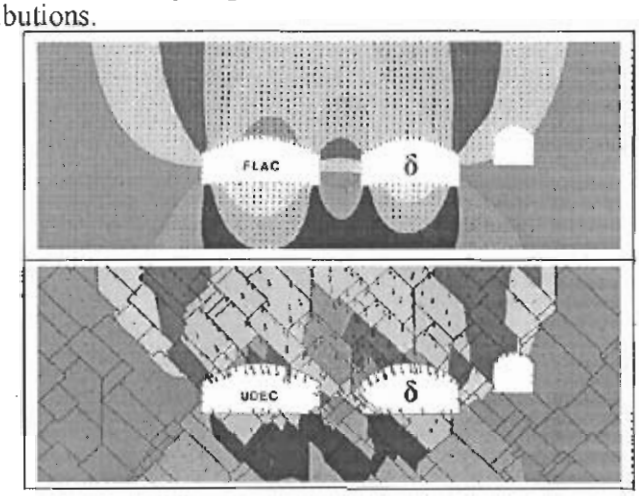


Figure 22 A demonstration of some of the limitations of continuum models as compared to discontinuum models, for showing the potential deformation modes (and support needs) in large tunnels and caverns. Backer, pers. comm.(1998).

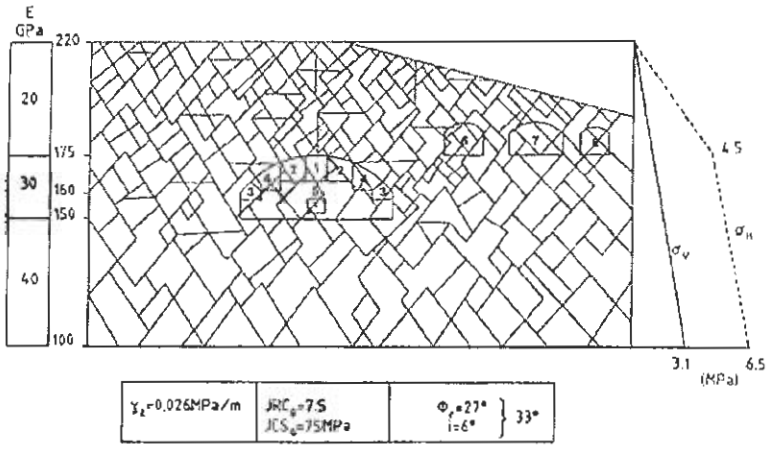
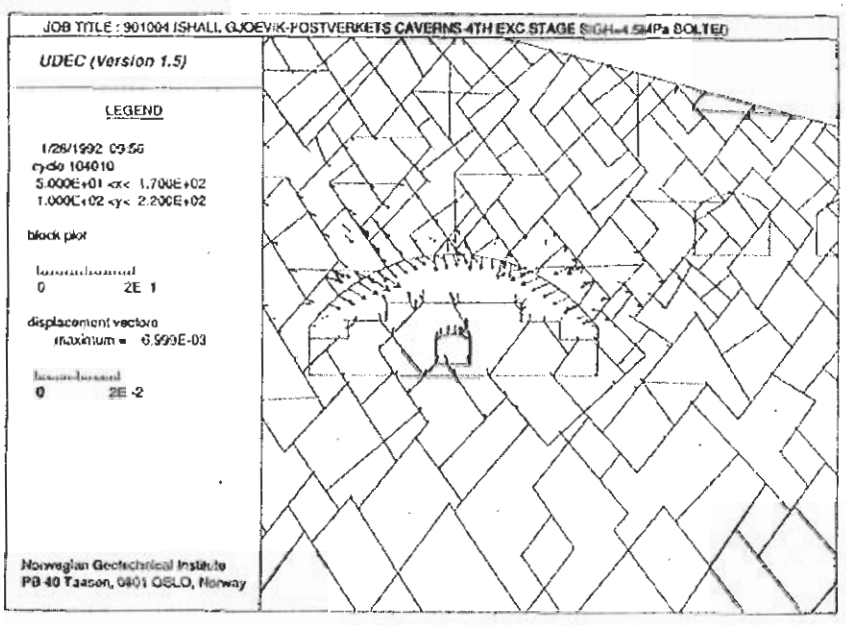


Figure 23 The joint geometry, modulus variation with depth, and BB joint input data used in the UDEC-BB modelling of the Gjøvik Olympic cavern. Physical modelling had earlier shown the possibility of ground heave ('negative' vertical deformation) with similar high levels of horizontal stress. Upwards or downwards deformation depended on the joint pattern. The modelled 7 to 9 mm downwards directed deformation matched the unknown measured result almost perfectly. Barton et al. (1994).



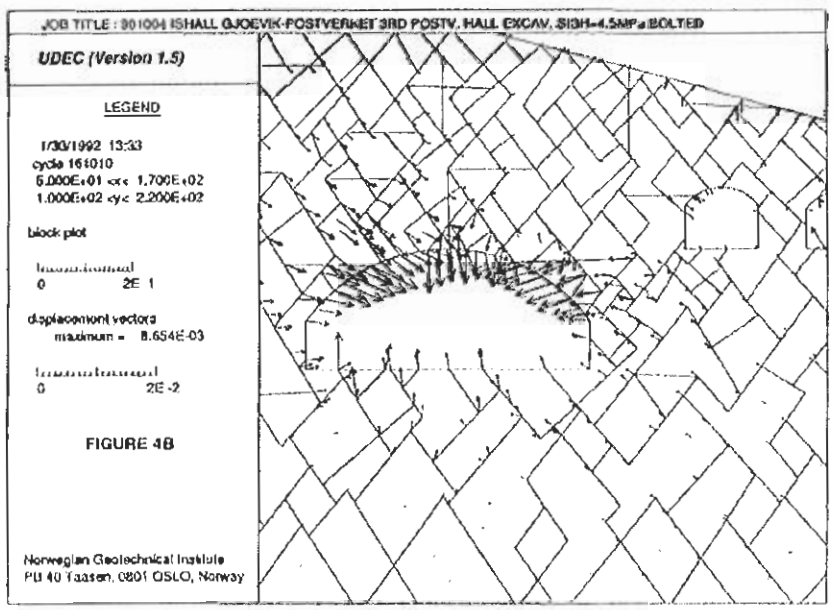


Figure 24 Two stages of the multiple excavations at Gjøvik, which included final excavation of adjacent Postal Service caverns, which caused some subtle changes to the main cavern. Note the maximum modeled deformations of 7.0 and 8.7 mm for the huge (62 m span) 'top heading' and final stages, when the total height was 24 m. UDEC-BB modeling by Chryssanthakis, of NGI. Barton et al. (1994).

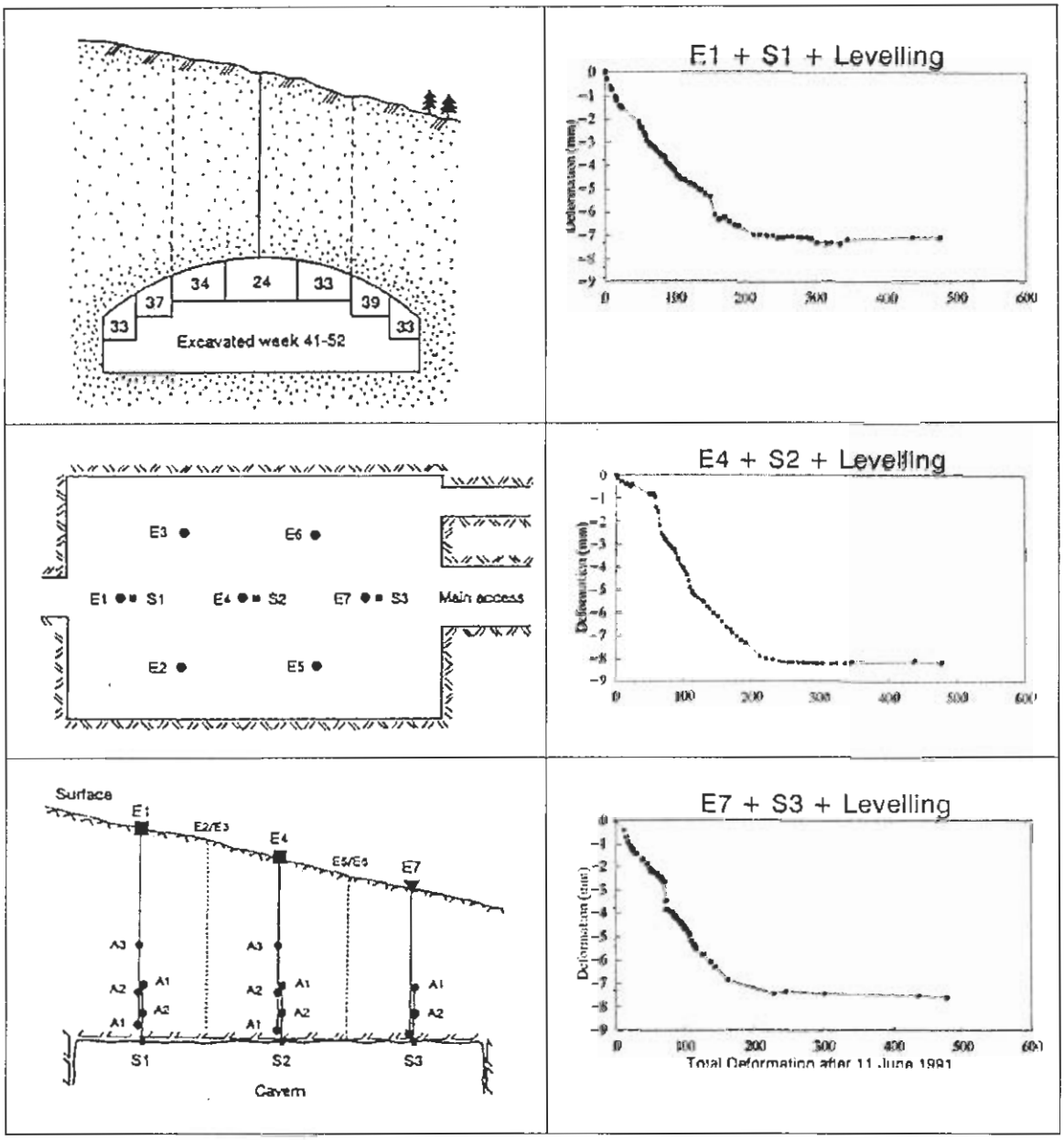


Figure 25 Three views of the cavern that show the location of the ten MPBX. On the right-hand side are the first 500 days of monitoring, obtained by adding the surface leveling to the 'dilation' of the 'E' (external) and 'S' (short internal) extensometers. Maximum measured deformations of 7 to 8 mm were an excellent validation of the class A (blind) modeling with UDEC-BB (7 to 9 mm), and were also a form of validation of BB input data. Key components of the rock mass for stable conditions being achieved with 'normal' NMT Q-based rock support, were the roughness of the jointing, and the fairly high horizontal stress (see Figure 23). Joint roughness was typically Jr = 2, and JRC_0 = 7.5, with larger scale undulation that was assumed to give a 'Patton i-value' of 6°.

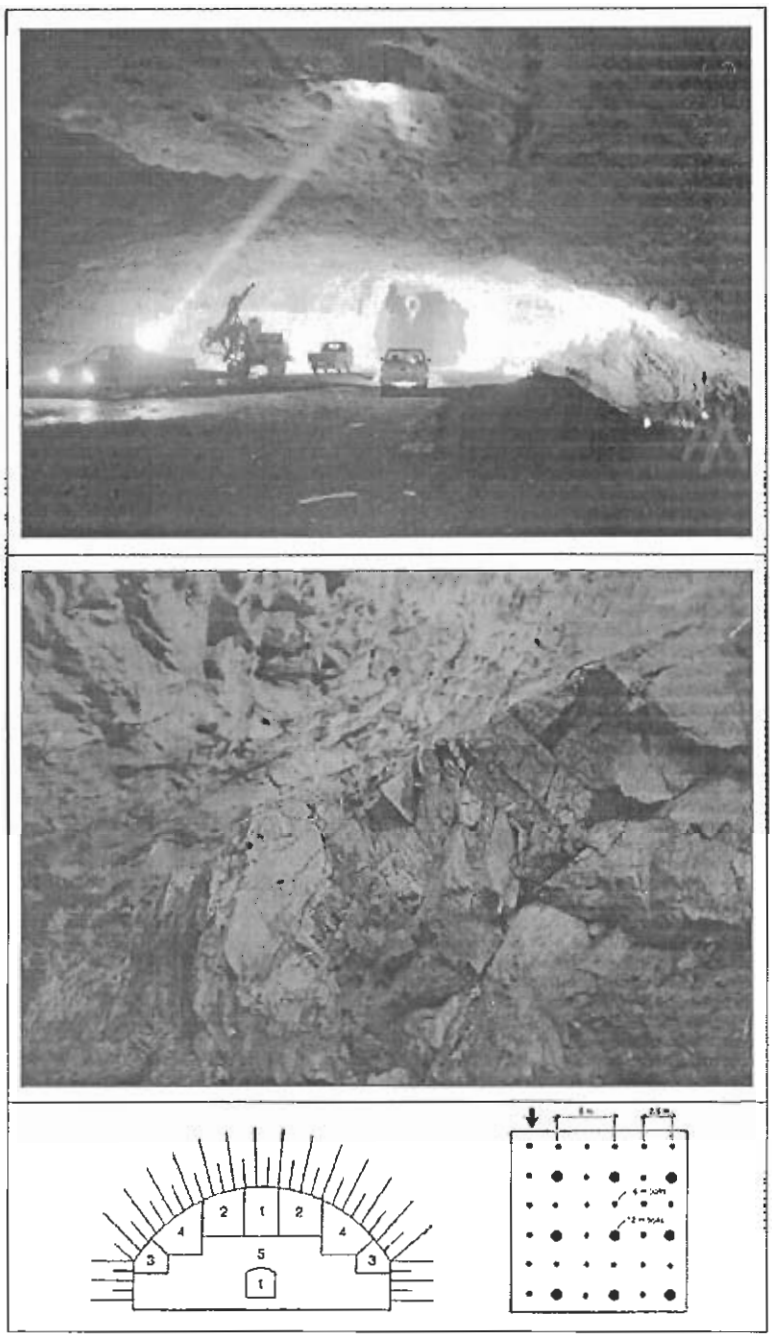


Figure 26 Views of the top-heading shown in Figures 24a and 25a. Note the conjugate jointing in the gneiss, which was responsible for more than 1.5 m overbreak in places in the arch, some 20 m above this lower photo, which was taken from the floor of Level 3. The NMT permanent support was S(fr) 10 cm (a mean 9.8 cm from numerous control borings), plus the bolting and light cable (2 x 16 mm twin-strand) depicted in the lower diagram. The 140,000 m^3 (62 x 24 x 120 m) cavern was excavated and supported in roughly 7 months. (See week-numbers and excavation stages in Figure 25a).

A CRITICAL DISCUSSION OF INPUT DATA FOR CONTINUUM MODELLING

A long time ago, in the late 1960's, there was a move in some rock mechanics circles to try to move beyond the confines of continuum modeling, and focus on the possible effects of jointing on the performance and reinforcement needs of rock excavations, be they tunnels, slopes or dam abutments.

Thanks to the late 1960's modeling developments of Goodman and his colleagues with joint elements in FEM codes, followed by Cundall in the early 1970's, first with μDEC, then UDEC and later with 3DEC, this focus could be fulfilled by an increasing number of rock mechanics practitioners around the world. However, utilizing these codes correctly, with realistic input data, needs experience, time and therefore budgets to match.

Presumably as a result of time and budgetary pressures, and also the developing need to model large-scale mining problems, there has been a marked 'backwards' movement into the use of convenient continuum codes, which also have particularly good graphics presentation. Simple software packages for handling the unjustifiably complex input data calculations (e.g. Figure 27) are also provided, so that a smart user might theoretically need only limited understanding of rock mechanics principles to use the codes 'successfully'.

$E_m\ (GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{(GSI-10)/40}$	$E_m \approx 10 \times Q_c^{1/3}$
$\sigma'_{cm} = \sigma_{ci} \times \frac{(m_b + 4s - a(m_b - 8s))(m_b/4 + s)^{a-1}}{2(1+a)(2+a)}$	$\sigma_{cm} \approx 5\gamma Q_c^{1/3}$
$\phi' = a\sin\left[\frac{6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a) + 6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}}\right]$	$"\phi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$
$c' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right]\left(s + m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1 + \left(6am_b\left(s + m_b\sigma'_{3n}\right)^{a-1}\right)/((1+a)(2+a))}}$	$"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$

Figure 27 The extraordinarily complex formulæ (left), for developing input data for some recent continuum models, and comparison to some of the less developed, and equivalent Q-based formulæ. For a non-mathmatician, in other words for a simple rock engineer, there is no possibility to have any feel for the influence of local rock quality on the rock mass compression strength, friction angle or cohesion, when formulations require software, rather than estimation for their evaluation.

The writer has often used the 'Chinese method' of rapidly thumbing from the back of a consultant's report to the front, whereby the coloured appendices of endless stress distributions and deformation patterns, can be read almost as in a film. Does all this 'colour' represent anything real? Would the numerical modelers know how to input a neglected clay seam – without 'smoothing-it-out' in a continuum approximation? Would equations c) and d)

for φ' and c' in Figure 27 change very much?

An additional problem is that the failure mode involved in rock breakage is actually highly unlikely to be governed by the Mohr-Coulomb style of strength criterion 'c + tan φ', because rock fails at small strain, breaking the cohesion, followed by mobilization of friction at larger strain. It is for this reason that continuum models with such strength criteria (i.e. with the '+' term) make such a remarkably poor job in modelling actual rock failure, as documented in careful experiments. This important fact was emphasised by Martin et al. (2002). The obviously poor performance is shown in Figure 28.

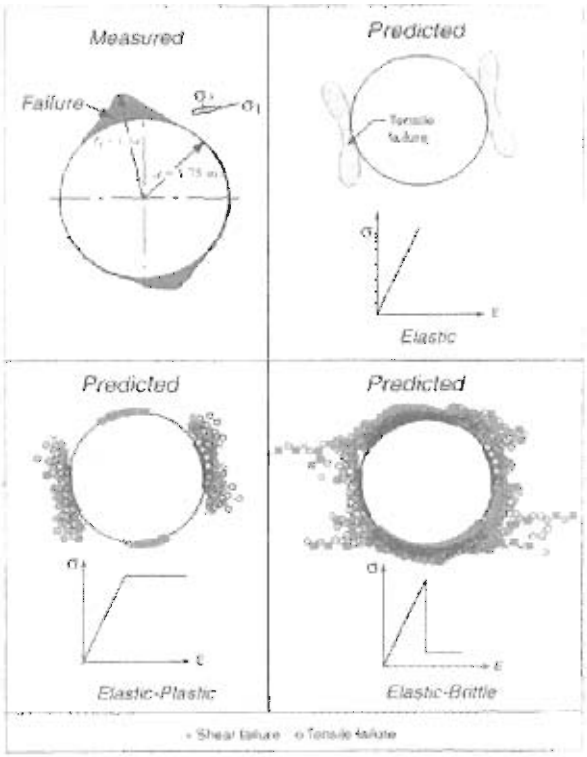


Figure 28 Attempts to model the rock breakage observed in a line-drilled tunnel at the URL in Canada, using continuum models having strength criteria of the form 'c + tan φ', when the actual breakage is by a process similar to 'c *then* tan φ', due to cohesive failure at small strain followed by frictional mobilization at larger strain. Martin et al. (2002).

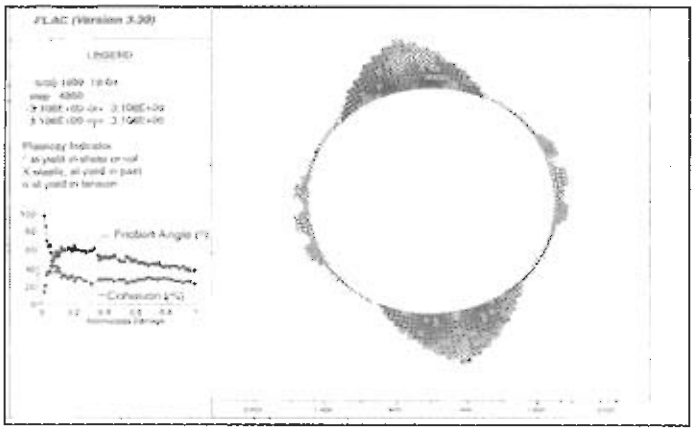


Figure 29 Degradation of cohesion at small-strain, and simultaneous mobilization of friction by means of a 'manual' entry into a FLAC calculation by Diederich, (Martin et al. 2002), shows excellent matching to the URL break-out position and depth. A more valid fracture mechanics approach may perhaps be achieved by FRACOD. (Shen

In Figure 27, simple Q-based equations for 'c' and 'φ' were shown, that are actually found to be composed of each 'half' of the Q_c-formulation. They have the advantage of not requiring software for their calculation – they already exist in the Q-parameter logging data. We define them as follows:

'cohesive component' CC = RQD/Jn x I/SRF x σ_c/100

'frictional component' FC = $\tan^{-1}$[Jr/Ja x Jw]

RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	FC°	CC MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.8	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5
10	20	1	6	0.5	5	0.008	10	0.0008	5°	0.01	0.4	0.9

Figure 30 Five hypothetical rock masses with reducing quality from top to bottom of the tabulation. Note the difference between Q and Q_c due to normalization by σ_c/100. The P-wave velocity and (pseudo-static) deformation modulus estimates are from the central diagonal, near-surface (25 m depth) inter-relationships given in Figure 7. Barton (2002).

Plate loading tests taken to such high stress levels that rock mass failure occurs are rare. The measurement of P-wave velocity at such sites may allow tentative extrapolation to other sites through a common rock mass quality estimate. Such data can then be a source of tentative rock mass strength ($\sigma_{c\,mass}$) estimation. The small table below suggests compressive (and cohesive) strengths in rock masses somewhat different to those usually assumed. They also show some implicit variation from the values set up in Figure 29 (from specific Q-parameter combinations), but reinforce the idea of potentially very high cohesive strengths (e.g. 10's of MPa) in competent rock masses.

Table 4 Plate load tests driven to failure, with corresponding velocity and modulus data for the different rock masses. Savich et al. (1974).

Velocity V_p (km/s)	2.3	3.7	4.0
Modulus E_{mass} (GPa)	1	3	15
Rock mass σ_{cm} (MPa)	4	20	50

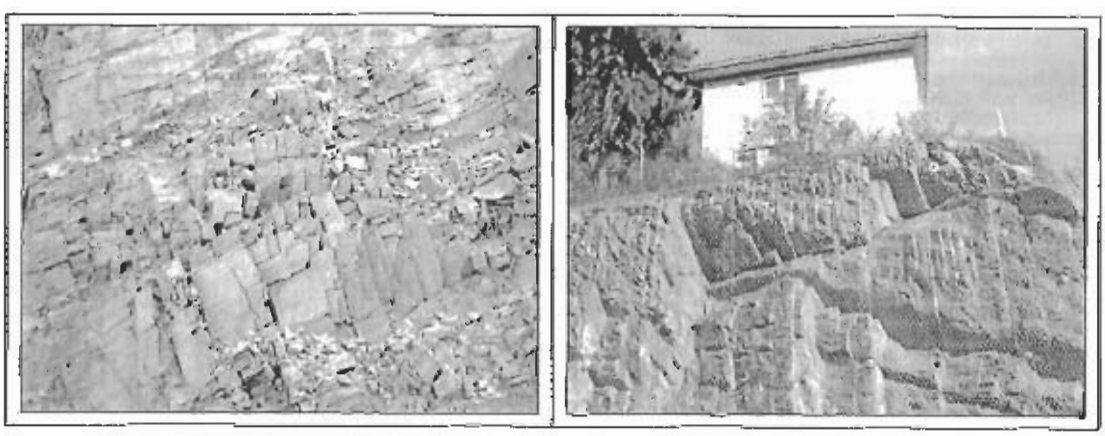


Figure 31 Examples of rock masses with particularly low CC (left), and particularly low FC (right). These require relatively more shotcrete (left) and relatively more bolting (right). The original Q-system case records have apparently reflected these different needs, and the Q-parameter ratings developed have given the possibility of realistic CC and FC values.

CONCLUSIONS

1. Q-system linkages to parameters useful for design are based on sound, simple empiricism, that works because it reflects practice, and that can be used because it can be remembered. It does not require black-box software evaluation.

2. The wide range of Q-values (0.001 to 1000) reflects to some degree the very wide range of geological conditions, and is probably responsible for the fact that empirical equations based on the Q-value are particularly simple.

3. An integration of Q with seismic data is useful because there is a limit to how many boreholes can be drilled, how many cores can be logged, and how many permeability tests can be performed. The ability to extrapolate these 'point sources' of information helps to project rock quality classes along a tunnel, or to different parts of a large cavern.

4. Due to the effect of increased stress at greater tunnel or cavern depth, it must be expected that deformation modulus and seismic velocity will increase. Eventual sonic logging or cross-hole tomography ahead of a tunnel face may therefore give a higher velocity than the rock quality may suggest.

5. The most simple and least accurate approximation for permeability is that the number of Lugeon: $L \approx 1/Q_c$. This is strictly for the case of clay-free, jointed, low porosity rock masses. A more generally applicable approximation uses an inverted Ja/Jr term and 100/JCS to give a better link to permeability. A high value of Q_{H2O} implies low permeability, and a general reduction of permeability with depth is also modelled.

6. There are other surprisingly simple relationships that have their origin in empirical links to Q-values. Support pressure appears to be inversely proportional to deformation modulus, and a central trend for tunnel deformation is that Δ in millimetres is equal to span in meters divided by Q. An improved fit to the quite scattered deformation data incorporates the stress-to-strength ratio, with differentiation of vertical and horizontal stress, for estimating arch or wall deformation.

7. As expected from a system that has it's origin in tunnel and cavern support selection, there is a strong correlation of time for construction with Q, and cost of construction with Q. The strongest correlation, where the curves of time and cost are steepest, is where the Q-value is between 0.01 and 1.0. It is here that the greatest benefit of high pressure pre-injection may be obtained, with effective, apparent improvements in many of the Q-parameters, and therefore in correlated properties like increased velocity and modulus, reduced support needs, and increased round lengths.

8. Application of discontinuum codes like UDEC-BB gives much more understanding and more relevant behaviour predictions than continuum codes. The example of the Gjøvik Olympic cavern of 62 m span is given, where the blind prediction of displacements was remarkably accurate, despite the possibility of either upward or downward displacements, that depended upon the interaction of joint orientations, their strength and stiffness, and horizontal stress levels.

9. Strength criteria of the form 'c + tan φ' used in continuum codes, which have remarkable complexity requiring software for evaluation of their components, have in addition the problem that when supposedly simulating shear failure, the reality is cohesion reduction at small strain, and friction mobilization at larger strain. Mohr-Coulomb and Hoek-Brown strength criteria may therefore need to be of the form 'c *then* tan φ'.

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MODERN SOFT GROUND TUNNELLING TECHNOLOGY - NEW SOLUTIONS OR NEW PROBLEMS?

J. Nick Shirlaw[1]

Abstract

Over the last thirty years soft ground tunnelling technology has been developing rapidly, particularly with the widespread use of Earth pressure balance and slurry shields. It is common to use precast concrete, segmental tunnel linings for this type of tunnel. However, there have been numerous problems of local loss of ground with such machines, and some more severe incidents involving major tunnel collapses, as well as other problems. These may be ascribed to the speed of the development of the new technology, and the time taken to understand the limitations of that technology and the organization and procedures required to tunnel using it.

INTRODUCTION

Over the past thirty yeas there have been major changes in the methods used to excavate and support soft and mixed ground tunnels. The pace of development has been rapid. This has allowed the construction of larger, deeper tunnels in worse ground and at a faster rate than was previously the case. The new developments have also resulted in, generally, better control of settlement due to tunnelling.

Although there have been significant advances in the use of shotcrete lining methods, as applied to soft ground, dramatic developments have been in machine tunnelling. This has led to the widespread adoption of Earth Pressure Balance (EPB) and slurry machines for tunnelling in difficult ground conditions. These machines are now commonly used in potentially unstable ground, such as soft clay, sands below the water table and weathered rocks. Other types of shield are still used, but EPB and slurry machines have become the most common methods for tunnelling through such ground conditions.

The increasing pace of development is illustrated in Figures 1, 2 and 3. Figure 1 is a reproduction of a plate showing the shield machine that was used to construct the St Clair River Railway tunnel, between Canada and the USA, in 1890. The machine was driven forward by hydraulic rams, shoving off the segmental lining. The segments were erected using a segment erector. Face support through the soft clay under the river was provided by compressed air, supplemented by face timbering as necessary. Excavation was by hand, using spades. Removal of the muck was by rail mounted wagons, pulled by horses.

Figure 2 shows an open face shield that was used in 1986 for the construction of the running tunnel between City Hall and Bugis Stations in Singapore. The machine is recognizably the same type as that used nearly 100 years earlier in Canada. Some improvements have been made, such as the incorporation of a backhoe for excavation of the face below the central deck. Hydraulically operated hood rams and manually operated face grids have also been provided, to allow rapid support of the crown and the face above the central deck. However, support of the soft marine clay encountered during tunnelling was still by the provision of compressed air, and excavation of the upper part of the face involved the use of cheese wire and spades. The muck cars were now pulled by locomotives, rather than horses, but the general tunnelling method is still

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recognizably the same as that used nearly 100 years earlier. Similar shields, usually operating under compressed air, were used to excavate the majority of the tunnels in soft and mixed ground conditions for the Modified Initial System, Tsuen Wan Extension and Island lines in Hong Kong, in the 1970s and early 1980s.

For comparison, Figure 3 shows EPB machines of the type used to excavate the majority of the tunnels for the North East line in Singapore, in the late 1990s. Similar machines were used in Hong Kong for the construction of tunnels for West Rail, also in the late 1990s. In this type of shield, excavation is fully mechanical and support to the face is provided by pressurizing the cut material within the plenum chamber of the machine. Compressed air is generally not used for normal operation, but air locks are provided for access to the plenum chamber, to inspect and change the cutting tools.

For small diameter tunnels, such as those commonly used for the laying of utilities by ‘trenchless methods’, support to the tunnel is commonly provided by jacked pipes.

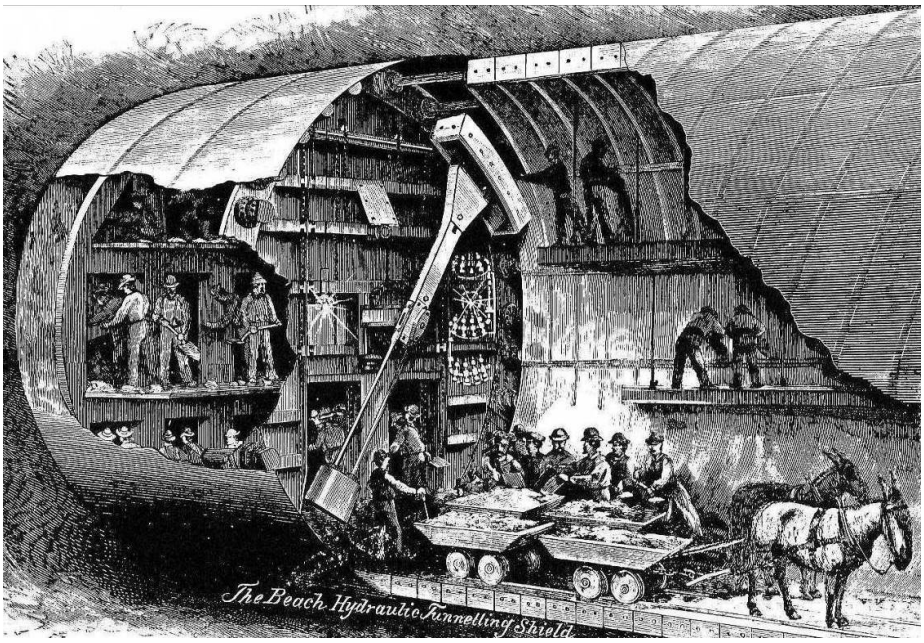

Figure 1. The ‘Beach’ shield used to construct the 1890 St Clair River tunnel from Canada to the USA

For large diameter tunnels, reinforced, precast concrete segmental linings are a common method of providing permanent support. Although pre-cast concrete segmental linings have been in use for about seventy years, there have been significant changes in the design of such linings, primarily to enhance durability, water tightness and ability to withstand fire.

Overall, there have been tremendous changes in soft ground tunnelling technology in the last 30 years. However, the speed of change has meant that there has been little time to understand the limitations and problems that come with those changes. This paper will discuss some of the issues that have arisen in machine tunnelling, with pre-cast reinforced concrete segmental linings. The paper is particularly written around experience in Singapore, but will make reference to similar documented issues elsewhere. To provide context for the experience in Singapore, a brief summary of the ground conditions, and behaviour of the ground in relation to tunnelling, will be provided before discussing specific issues.

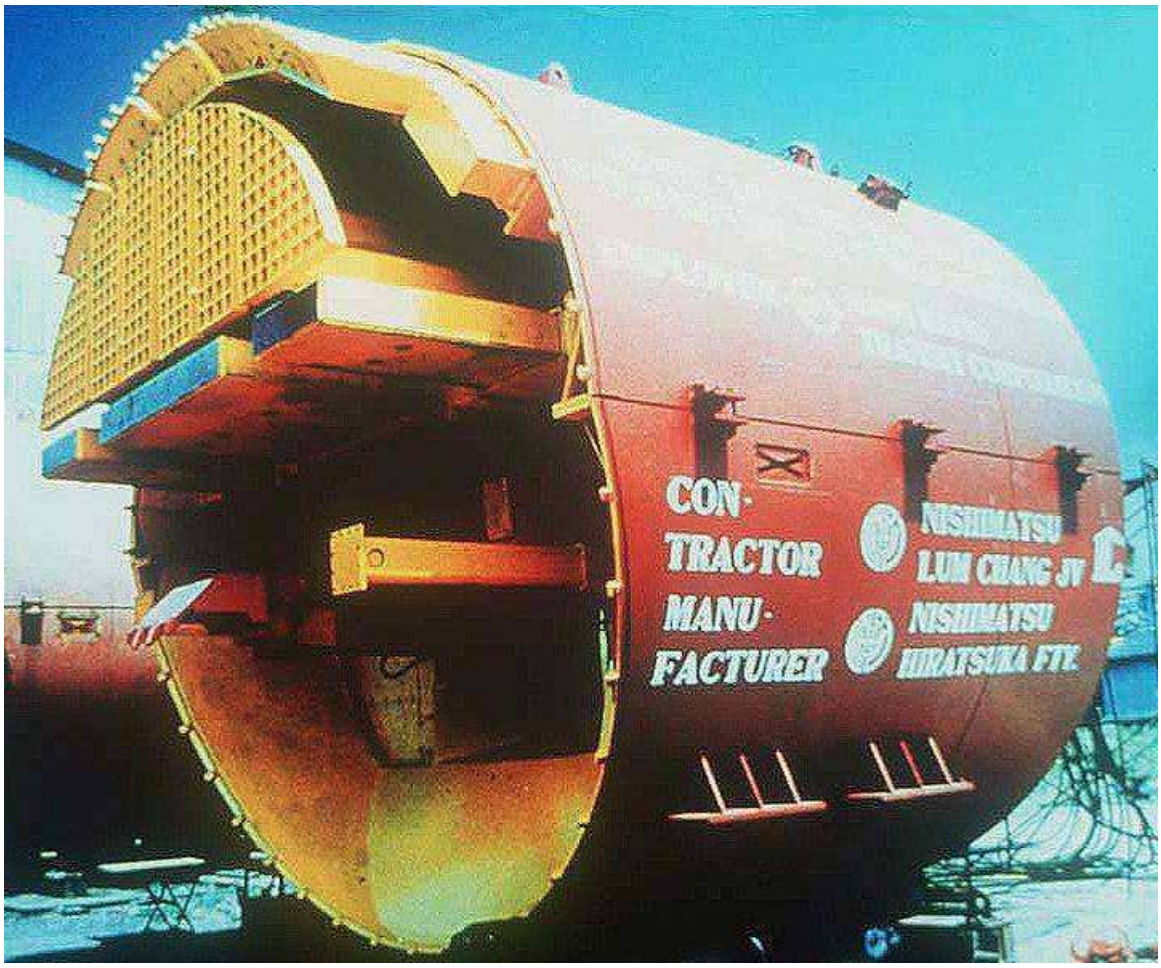

Figure 2. An open face shield used in 1986 to drive between City Hall and Bugis Stations in Singapore

Figure 3. Examples of Earth Pressure Balance Shields used during the Construction of the North East Line in Singapore

GROUND CONDITIONS FOR TUNNELLING IN SINGAPORE

The geological map of Singapore is shown in Figure 4.

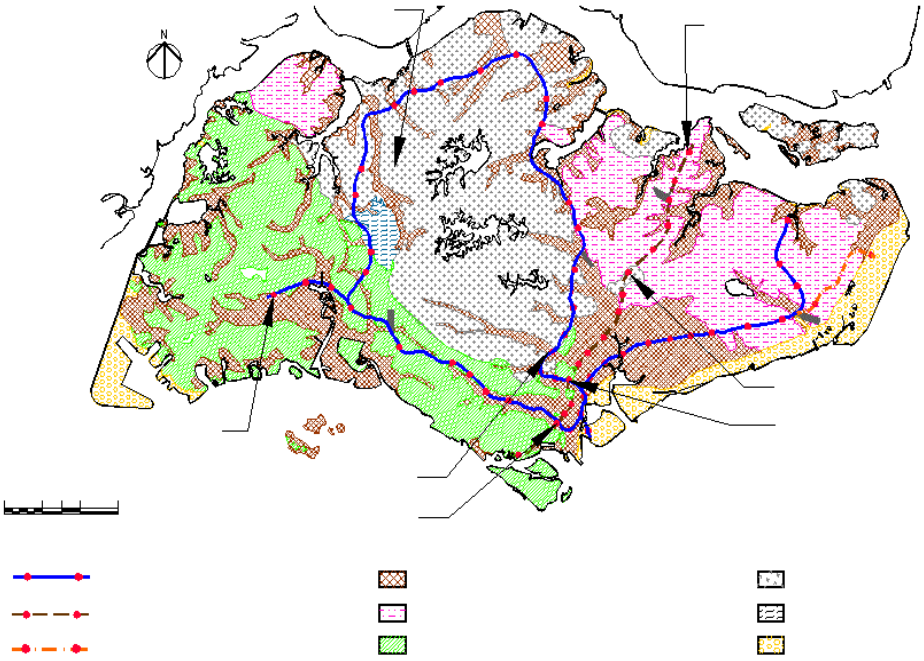

Figure 4. The geology of Singapore (based on PWD 1976)

References to the nature of the units are given below; the observed behaviour during tunnelling is also summarized:

Kallang Formation soils (Holocene and late Pleistocene).

The most common of the Kallang Formation soils is a near normally consolidated marine clay, which has been deposited in two phases. These are known as the lower and upper marine clays. The formation also includes loose fluvial sands, moderately stiff fluvial clay and soft organic clays. More details of the deposition and engineering properties of the Kallang Formation can be found in Tanaka et al (2001) and Bird et al (2003).

The marine clay is of low strength, typically in the range from 15 kPa at 15 m depth to 60 kPa at 40m depth. The soil will squeeze uncontrollably if no face pressure is applied. The soft organic clays behave in a similar manner. The fluvial sand layers are classified as 'flowing ground'. The fluvial clay is variable in strength, but typically ranges from fast squeezing to slow squeezing behaviour. Essentially, all of the soils of the Kallang formation have to be considered as potentially highly unstable, and require significant face pressure to prevent uncontrollable loss of ground in an unsupported face. During compressed air tunnelling the face pressure applied was typically sufficient to balance the water pressure, which amounted to about 60% of the total overburden pressure.

Old Alluvium (Pleistocene)

The Old Alluvium consists of lightly cemented alluvial sands and clays that generally have the strength of a weak or very weak rock. The upper 10 m to 30 m of the formation has been subjected to tropical weathering, leading to a reduction in cementation and strength, and increase in permeability. Details of the deposition, engineering properties and behaviour of the Old Alluvium can be found in Gupta et al (1987), Peart et al (2001), Shirlaw et al (2000), and Chiam et al (2003).

A significant proportion of the Old Alluvium consists of slightly cemented, clayey sand or very weak to weak sandstone. This material is excellent for tunnelling, and typically can be excavated without face support. Beds of slightly cemented clayey silt, or very weak to weak mudstone, are also commonly encountered. Also present in the formation are beds of sand or sand and gravel, with a low (<10%) content of silt and clay. These beds of sand or sand and gravel, unlike the rest of the formation, act as flowing ground below the water table, and require a face pressure to balance the water pressure at tunnel level.

Fort Canning Boulder Bed (Pleistocene)

The Fort Canning Boulder Bed is an old colluvial deposit consisting of strong to very strong quartzite boulders in a hard clayey silt matrix. The boulders typically comprise about 25% of the material, by volume. The origin and properties of the Fort Canning Boulder Bed are described in Shirlaw et al (1990), Han et al (1993) and Shirlaw et al (2003c).

The shear strength of the clayey silt matrix is reported as being in the range of 150 to 1500 kPa by Shirlaw et al (2003c). The material has been successfully tunneled, in free air, using open face shield and shotcrete lining methods, demonstrating the stability of the formation for tunnel construction. Broms & Lai (1995) report a layer of water bearing gravel at the interface of the boulder bed and the underlying Jurong Formation, during caisson construction. A similar layer would probably not be stable if encountered during tunnel construction.

Jurong Formation (Upper Triassic/Lower Jurassic)

The Jurong Formation consists of sedimentary rocks, including mudstone, siltstone, sandstone, conglomerate and limestone. The rock exhibits varying degrees of metamorphism; shale and quartzite have been encountered. The rock has been heavily folded. The dip on the beds encountered during tunnelling has been as high as 80 degrees. The rock has been subjected to intense tropical weathering. The mudstones and shales fragment as they weather, while the quartzites are relatively resistant to weathering. More details of the properties and behaviour of the Jurong Formation can be found in Zhao et al (1999) and Shirlaw et al (2000) and Jeyatharan et al (2003).

The behaviour of the Jurong Formation depends both on the degree of weathering and the particular nature of the bed. Commonly, there is a thick mantle of residual soil; this is typically from 10m to 30m in thickness (Zhao et al, 1999). The residual soil is typically a moderately stiff to very stiff clayey silt. Tunnelling in free air has been carried out, using open face shields, in the residual soil (Grade VI) of the Jurong Formation. However, free air tunnelling cannot be safely carried out if the tunnel crown is close to the overlying soils of the Kallang Formation. There needs to be sufficient thickness of residual soil between the crown of the tunnel and the overlying soft soil to form a stable arch over the tunnel advance.

In contrast to the residual soil, some beds of the Jurong Formation can be unstable if weathered to Grade V (completely weathered) or Grade IV (highly weathered) rock. Grade V sandstone behaves like a bed of sand, and, below the water table, is potentially unstable in an unpressurised face. The mudstone and siltstone beds fragment as they weather. Grade IV mudstone or siltstone can form a mass of sugar lump or gravel sized fragments, which can behave as raveling or fast raveling ground. Similar ground behaviour is commonly observed in the numerous faults which are found in the Jurong Formation. Figure 5 shows the ground settlement experienced over open face shield tunnels driven through a small fault in the Jurong Formation; the settlement exceeded 400mm.

Limestone is found in the western part of Singapore. Currently, it is included as part of the Jurong Formation although Pakianathan & Jeyatharan (2005) argue that it should be treated as a distinct formation. The limestone exhibits karstic features, in

particular cavities, sink holes and slump features. The cavities and sink holes are commonly infilled with soft or loose material which would be highly unstable if encountered during tunnelling

Figure 5. Settlement caused by open face shield tunnelling though a fault in the Jurong Formation

Bukit Timah Granite (Lower/Middle Triassic)

The Bukit Timah Granite includes various igneous rocks, particularly granite, microgranite and granodiorite. The rock has been affected by intense tropical weathering. The weathering profile can be of the typical 'corestone' type, exhibiting the six weathering zones outlined in BS 5930 (1999). Zhao et al (1995) record 'stratigraphic' weathering of the Bukit Timah granite. In stratigraphic weathering there can be an abrupt transition from Zone 1 to Zone 6, with no evidence of the intermediate zones. Details of the engineering properties and behaviour of the Bukit Timah granite are given in Zhao (1993), Zhao et al (1995) and Shirlaw et al (2000).

Due to the low relief of Singapore, there is generally a thick mantle of residual soil (Grade VI) over the central areas of Singapore where the Bukit Timah Granite is found (Figure 1). The Grade VI Granite is typically a moderately stiff to very stiff clayey silt. Free air tunnelling, using open face shields, drum diggers and SCL methods have been used successfully in the Grade VI material. Grades V and IV are potentially flowing ground, particularly in the zone just above the rock grades of the weathered granite. Good tunnelling conditions are typically found in the rock grades: caverns have been constructed in the rock (Tan et al, 2001).

Groundwater conditions

The main urban areas of Singapore are typically located about 3 m above sea level near the coast, 4 m to 5 m above sea level further inland. With a high annual rainfall, ground water levels are typically close to the ground surface. Except under the few hills, the groundwater is generally found about 1.5 m below ground level. The salinity of the pore water in the upper marine clay is close to that of sea water (35,000ppm), while that of the lower marine clay can be about half of this value. Significant chloride levels can be found in the Jurong Formation. Chloride levels of 18,200 ppm have been measured in the weathered rocks of the Jurong Formation (LTA 2001). This is even higher than the 16,600 ppm measured in the upper marine clay.

TUNNELLING ISSUES

General

The focus of this paper is on the use of modern, EPB or slurry shields, erecting segmental linings to build tunnels in soft and mixed ground. Some of the problems and unresolved issues in this form of construction will be discussed. The paper will concentrate on the immediate area of the shield advance, minimizing the effects on third parties and ensuring the water-tightness of the tunnel. Issues such as shaft sinking, material handling and erection of the lining, while important, will not be covered. The objective will be that of the owner: to achieve a stable, durable tunnel with minimal adverse effect.

There are a number of steps required to build, successfully, a tunnel using an EPB or slurry shield erecting segmental linings. Each of these steps has to be planned and executed correctly to achieve the objective. The steps discussed will be:

(a) controlling the stability of the face of the tunnel;
(b) minimizing ground losses due to over-cutting and steering;
(c) detailing the segmental linings to minimize leakage; and
(d) grouting the lining.

These items are associated with different areas of the tunnel construction. All of these items affect the settlement that will occur due to the tunnelling, while the last two will affect the durability of the lining and equipment installed inside the tunnel.

While this paper will concentrate on TBM tunnelling, there are usually some elements related to TBM tunnels that still have to be built using SCL or more traditional, hand methods, such as timbered headings. Typical elements that require alternative construction methods are cross-passages, escape refuges, ventilation connections and initial drives. The skills required to construct tunnels through difficult ground using such methods are quite different to those needed for TBM tunnelling. The ancillary tunnels often form a relatively minor part of the overall works, but carry a particular risk.

Ground control and settlement are directly related to the understanding of the ground conditions along the alignment. If there is good knowledge of the ground conditions, the type of shield and the tunnelling procedures can be selected appropriately. If, however, there is inadequate understanding of the ground conditions, then there is a serious risk that conditions will be encountered for which the machine and/or operating procedures are unsuitable.

Controlling the stability of the face of the tunnel

During EPB and slurry shield tunnelling the normal mode of operation involves providing face support by pressurizing either the spoil from excavation (EPB) or slurry. However, the face also needs to be supported during man access into the plenum chamber, for inspection and maintenance. During access the spoil or slurry in the plenum chamber has to be removed, partially or fully. Support to the ground is, most commonly, provided by compressed air. Other methods, such as treating the ground or dewatering, may also be used.

Particular issues involved in maintaining face stability, both during normal advance and during face access, are:

(a) What is the appropriate face pressure?
(b) Can the appropriate face pressure be consistently applied?
(c) Why might the appropriate face pressure not be applied?
(d) What can be done to reduce the risk that the appropriate face pressure is not applied?

If the appropriate face pressure is not applied, then, in unstable ground, there will be major loss of ground at the face. Such a loss of ground typically causes a highly localized area of very large ground settlement, commonly referred to as a 'sinkhole'. Examples of large, local settlement and a sinkhole are shown in Figure 6).

Figure 6. A Sinkhole (left) and a Local Area of Large Settlement, Both over EPB Driven Tunnels

There have been a significant number of such incidents over EPB and slurry shields over the last 20 years. Shirlaw & Boone (2005) record 57 such incidents over 76.9 km of EPB driven tunnels in Singapore and Canada, or one incident per 1.35 km of tunnel. Anon (2005) received information on over 100 incidents, involving both EPB and slurry shields. It can be seen that the consequences of applying inadequate face pressure, even for a relatively short period, can be serious, and that this has been happening with an unacceptable frequency. The answers to the questions above are therefore important.

Determining the appropriate face pressure

The face pressure required depends on a number of factors, including:

(a) the depth of the tunnel;
(b) the diameter of the tunnel;
(c) the strength of the ground in and over the face;
(d) the groundwater level at the tunnel horizon, and the permeability of the ground; and
(e) the sensitivity of the area to settlement

Anagnostu & Kovari (1996) provide methods of assessing the face pressure required for slurry and EPB tunnelling. They provide simple equations, derived from limit equilibrium analysis, that can be used to calculate face pressures. It should be noted, however, that the values derived are those required to avoid collapse of the face, not to those required to minimize settlement. The calculations for EPB shields tunnelling in granular soils are based on steady state seepage conditions developing in the chamber and the ground ahead of the face. In most granular soils this is not a practical option, due to the instability that would result before steady state conditions can be established. In practice, in granular soils, soil conditioning generally has to be used to reduce the permeability of the spoil. For EPB shields operating with soil conditioning in granular soils below the water table, the methods proposed by Anagnostu & Kovari (1996) for assessing the appropriate face pressure for slurry shields are more appropriate than those given by them for EPB tunnelling.

For tunnelling in clay, Kimura & Mair (1981) provide charts that can be used to assess the pressure required to avoid collapse at the face of the tunnel. The charts can also be used to relate face pressure to volume loss, and therefore to settlement. However, to assess total settlement other contributory factors, such as overcut closure, partial

closure of the annular space around the ring, and consolidation also have to be considered.

Ideally, planning of face pressures should be based both on calculation and on past experience of tunnelling through similar ground conditions. For Singapore, Shirlaw et al (2003b) provide a summary of the settlements during EPB tunnelling for the North East line, and relate these to the face pressure normalized by the overburden pressure at the axis level of the tunnel. A face pressure equal to about the water pressure at tunnel level was sufficient to avoid collapse when tunnelling in soft clays and granular soils. Weathered rocks that, in free air, would behave as flowing, raveling or fast raveling ground, also required a face pressure equal to about the water pressure at tunnel level. For the soft clays and loose sands of the Kallang Formation, the use of such a pressure would avoid outright collapse, but significantly greater face pressure was required to reduce surface settlement to a reasonable level. In consistent, stiff or hard cohesive soils, such as Grade VI weathered granite or the Fort Canning Boulder Bed, no face pressure had to be applied to control face stability. However, the surface settlements reduced with increasing face pressure.

Anon (2005) provide a general recommendation for EPB and Slurry machines that:
'*It is recommended that operating pressures are always greater than the prevailing hydrostatic pressure and sufficient to ensure that the plenum is full.*.'

Based on experience in Singapore, for a wide range of ground conditions, the use of a pressure greater than the hydrostatic pressure should always provide sufficient support to avoid major loss of ground at the face. In soft clay or loose sand, to control settlement the pressure needs to be significantly higher than hydrostatic pressure, and has to approach full overburden pressure.

Can the appropriate face pressure be consistently applied?

There are a number of possible reasons why the appropriate face pressure cannot be applied in practice. This typically only occurs locally, as the associated problems are usually so severe that measures have to be taken to correct the pressure. Particular reasons for not being able to apply sufficient pressure would include:

(a) the nature of the ground;
(b) mechanical failure of the machine;
(c) loss of the support medium / slurry; and
(d) inability of the machine to advance with the required face pressure.

EPB and slurry shields have limitations on the type of ground in which they can operate successfully. For EPB machines, there is a limit to the ability to transform clean, granular soils into a spoil suitable for EPB operation. The use of soil conditioning agents, such as foam, typically requires about 30% of the cut material to be silt or clay size particles to form an effective EPB spoil. Where the natural ground includes only a small percentage of fine particles, such particles can be injected into the chamber. However, this is a time consuming process that can greatly reduce the rate of tunnel advance.

A particular problem with EPB machines has been tunnelling through mixed faces of rock and soil grades of weathered granite. The rock grades are cut into cobble sized pieces by the machine. If the proportion of Grade V rock in the face is small, there will be insufficient fine particles in the spoil to form an effective spoil for EPB operation. Failure to balance the water pressure will lead to instability of the Grade V granite.

For slurry machines, if the ground is of very high permeability, there will be loss of support as the slurry penetrates into the ground. Anagnostu & Kovari (1996) provides methods for calculating the effectiveness of slurry support.

Modern, closed face shields are complex machines; if part of the machinery fails to function properly or becomes worn, then the ability to maintain pressure in the plenum chamber may be compromised. Examples of mechanical problems that have led to loss of face pressure and consequent loss of ground, using EPB shields, are given in Table 1, which is based on Shirlaw and Boone (2005).

Table 1. Mechanical Problems that have led to high Localised Settlement on 7 Projects involving EPB shields

Cause	Number
Abrasion of screw conveyor	3
Failure of main pressure bulkhead	1
Jamming of discharge gate by obstructions	2
Failure of tail-seals	1

The operation of EPB and slurry shields in pressurized mode is dependent on having the plenum chamber filled with spoil (EPB) or slurry. If the spoil or slurry is lost, then the shield can no longer be operated in pressurized mode. Examples of situations where this can occur would include:

(a) when old wells or drill holes are encountered;
(b) when cavities are encountered; these may be natural, such as karstic features, or manmade; and
(c) if there is an open path into the launch or recovery shaft.

If the machine has not been properly designed, or if ground quite different to that envisaged is encountered, then it may not be possible to advance the machine under the necessary face pressure. The total thrust required to advance the machine equals the sum of:

(a) the force required to react to the face pressure;
(b) the friction on the shield;
(c) the force required to move the trailing gear; and
(d) the force that has to be applied to the cutting tools to excavate the ground.

If the force required for items 2 to 4 is high, there may be insufficient thrust remaining to provide reaction to the desired face pressure. This generally occurs only in rock or mixed face conditions, and if the rock requires significantly higher thrust to excavate than anticipated. It may be questioned why it is necessary to apply a face pressure when tunnelling in rock, and whether the general requirement to balance hydrostatic pressure is still applicable in rock. Most tunnelling in rock is carried out without requiring face pressure, so face pressure would not be considered a normal requirement. However, there are times when a face pressure is necessary, even when tunnelling in rock. Examples would include:

(a) mixed face conditions, if either the soil or rock requires a support pressure for stability;
(b) where the interface between soil and rock is close enough above the crown to the tunnel to require a support pressure;
(c) rock that exhibits raveling, or, particularly, fast raveling behaviour, in locations where settlement has to be controlled; and
(d) where there is soft clay over the tunnel, and consolidation settlement must be controlled: without face pressure there will be seepage into the tunnel face.

If any of these circumstances occur, and the machine does not have adequate thrust, then it may not be possible to maintain the required face pressure.

Why might the appropriate face pressure not be applied?

Circumstances are listed, above, in which the required pressure might not be applied for factors related to the ground or the machine. However, there is also a human

factor, and the required face pressure may not be applied because the operator either chooses not to do so, or may not be capable of doing so.

If the use of a face pressure greater than the hydrostatic pressure at tunnel level provides a high degree of security against a major loss of ground at the face, then why might an operator not apply that pressure? Possible reasons include:

(a) concern over abrasion;
(b) failure to use, or ineffective use, of soil conditioning additives to create an effective spoil for EPB operation;
(c) concern over the effects of high compressed air pressures when entering the chamber for inspection and maintenance; and
(d) because the operator is overwhelmed by information during rapidly changing ground conditions.

Abrasion on the cutting tools, head, pressure bulkhead, screw conveyor (for EPB shields) and pipework (slurry) shields can be costly, both in terms of materials and time delays. For EPB shields there is evidence that the face pressure and the rate of abrasion are related. Concern over abrasion has been used as a reason for minimizing face pressure. In Singapore, the most abrasive ground conditions have been provided by the Granite, beds of quartzite within the Jurong Formation, the Fort Canning Boulder Bed and the Old Alluvium. In these ground conditions, the majority of the tunnelling can be carried out successfully without the need for face pressure. However, there are locations where face pressure is necessary. Such locations would include mixed faces of soil and rock grades of the weathered Granite and where beds of clean, uniform sand are encountered in the Old Alluvium. Such variable, but abrasive, conditions pose particular problems for the operation of the TBM. If the pressure is maintained throughout the drive, the consequent abrasion will be severe. Alternatively, if the face pressure is reduced, there is a risk that it cannot be increased quickly enough to avoid a local loss of ground, if the ground conditions deteriorate suddenly.

Soil conditioning is an important aspect of EPB tunnelling. However, there are many different products in the market, and relatively little guidance about what product works best in what circumstances (Boone et al, 2005). For slurry shields, additives may be required in highly permeable ground to avoid loss of the slurry. Trial mixes of soil and conditioner, or slurry and additive, prior to tunnelling, are important, so that the changes in type or proportion of conditioner/additive as the ground varies can be planned (Anon, 2005). Finally, however, there is a degree of optimization in response to observed performance during tunnelling. Failure to respond quickly enough to changing ground conditions can lead to an incorrectly conditioned spoil, or loss of slurry, and the possibility of loss of face pressure as a result.

During shield launch the face pressure has to be developed from zero initial pressure in the launch shaft. During shield recovery, the face pressure has to be reduced from the normal operating pressure to zero in the recovery shaft. There is a risk during launching and recovery that an inadequate face pressure will be applied at some point during the transition from zero to full pressure, or vice versa. There is also a risk that the ground will enter the shaft through the annulus around the shield, rather than through the face.

A particular issue in relation to face pressure is that required for access to the plenum chamber for inspection and maintenance. If the machine is being operated under pressure, then, logically, the face access should also be under pressure. The pressure for access into the head is generally provided by the use of compressed air, although alternative methods are available. Where compressed air is, or could be, necessary to provide face stability, there is a risk that the air pressure applied will be less than actually necessary. This may be because of :

(a) concern over the effects on health of high compressed air pressure;

(b) requirements related to the length of time that can be worked in high compressed air pressures;
(c) legal limits on the compressed air pressures that can be applied; and
(d) the need for highly specialized staff, equipment and procedures for work in very high pressures.

The operator of a modern tunnelling machine needs to check many parameters during shield advance. A typical machine will record anything from 50 to over 100 different operating parameters, which should be available to the operator in the on-board computer. Practically, however, the operator's primary duties for EPB tunnelling are:

(a) steering the machine to keep to the required alignment;
(b) checking that the thrust and torque used will not damage the machine;
(c) checking the operating parameters for any indication of excessive wear or damage;
(d) controlling the machine advance and spoil removal such that the required face pressure is maintained;
(e) visual checks on the nature of the spoil dropping from the discharge of the screw conveyor;
(f) controlling the degree of opening of the discharge gate;
(g) varying conditioner dosage to change the properties of the spoil; and
(h) checking excavation volume, so that there is no major over-excavation.

For tunnelling using slurry shields the list is largely similar, if the items relating to the screw conveyor are excluded.

In consistent ground conditions, for the operator to check and fine-tune the items on the list should not prove a problem. However, in rapidly changing ground conditions, the operator can simply become overwhelmed by the amount of information and by the need to make constant adjustments. In such conditions in Singapore it has been found advantageous to station engineers, full time, to assist the operator during shield advance. The assessment of the information can then be shared among the small team of operator plus one or two engineers, and critical decisions are made by the team, rather than an individual operator.

What can be done to reduce the risk that the appropriate face pressure is not applied?

The number of incidents involving loss of ground and excessive settlement during EPB and slurry shield tunnelling has led to various studies. In particular, the British Tunnelling Society's closed face working group prepared 'Closed face tunnelling machines and ground stability, a guideline for best practice' (Anon, 2005). Some of the recommendations in that report relating to face stability are given in Table 2:

This table is a summary of some of the main recommendations of the report in relation to maintaining face stability. Issues such as tail void grouting and site investigation have not been included in the table as these will be covered below. Reference should be made to the report for the full recommendations. Reference can also be made to Shirlaw et al (2005a, 2005b) for some recommendations in addition to those in Anon (2005).

All of the recommendations in the report are valuable, and these should provide a useful basis on which future machine tunnelling should be based. However, many of the current problems arise from human factors. While these are apparently covered in the last line of the table above, all of the tunnels where there has been occasional poor control of face stability have been managed and driven by experienced tunnellers, so simply requiring a certain degree of experience does nothing to change the status quo. Particular issues that need to be addressed are;

(a) inadequate planning for key areas of risk, such as those given in Shirlaw & Boone (2005);

(b) general optimism about anticipated performance, leading to ineffective risk assessments;
(c) inadequate knowledge of the ground conditions, or the consequences of those ground conditions on the tunneling; and
(d) inadequate knowledge of, and testing of, soil conditioning agents; this is not a criticism of individuals, but of a lack of knowledge in the industry as a whole

Table 2. A Selection of Recommendations from Anon (2005)

Item	Recommendations
Choice of shield type	Based on: • Gradations of ground anticipated • Soil permeability • Hydrostatic head at the tunnel horizon
Operating Pressure	• Keep operating pressure > hydrostatic pressure and sufficient to ensure chamber is full • Operating pressure < pressure to heave the ground surface • Plan and justify operating pressures • Minimum three pressure cells in plenum chamber, one near the top
Setting operating parameters	Provide operator with 'drive sheet', prepared by tunnel manager, including: • Operating pressures • Grouting parameters • Thrust forces • Cutterhead torque • Excavation quantities • Foam/slurry parameters • Alignment Also provide information on geology, groundwater and structures
Excavation quantity	For slurry shields: monitor flow and density of slurry in in-bound and out-bound lines For EPB shields: • Use at least 3 belt weighers, count muck skips, consider laser profiler • Provide audible and visual alarm in case of excessive excavation
Spoil conditioning	For slurry shields: use slurry with polymer For EPB shields: • Provide one cutterhead injection point per metre of diameter • Keep continuous record of quantities injected • Test foams under pressure
Risk management	• Four stages of risk management • Aim for risk reduction or elimination • Correct choice of equipment, including TBM
Staff and personnel	• Minimum criteria for experience of key staff • Training of new operatives

The changes in the tunnelling technology over the last thirty years mean that the industry has been on a steep learning curve. Organizational changes to adjust to the new technology are still being made. Inevitably, during the process of learning, mistakes have been, and are still being, made. The major benefit of studies such as Anon (2005) and Shirlaw et al (2005a) may not be in specific recommendations, but in publicizing the number of incidents of major settlement due to locally poor control of face pressure. There have been so many of these incidents, on so many projects, in so many countries, that the problem is clearly systemic.

Ancillary tunnels constructed by methods other than TBM

A contract package that is primarily for TBM tunnelling, usually also includes some ancillary tunnels that cannot readily be constructed by TBM. Examples include initial drives, cross-passages, sumps, escape refuges and ventilation connections. The skills required to excavate and support tunnels using SCL or more traditional methods are quite different from those required to construct TBM tunnels. It is common for such work to be carried out before or after the main drives, and it is quite common for the work to be carried out by the same crew that worked on the TBM tunnel. This simplifies staffing. However, if the TBM crew has little or no experience of this type of work, then there is a high risk of stability problems developing during the construction of the ancillary tunnels. Such stability problems can also affect the stability of the main, TBM driven, tunnel. Examples of the problems that can occur during the construction of such ancillary tunnels have been the massive collapses that occurred in Shanghai in 2003, and in Kaoshiung in 2005.

The use of gangs specializing in this type of work, rather than the regular TBM crew, has been implemented on some projects, and seems sensible, particularly in high risk areas and/or in difficult ground conditions.

Ground losses due to overcutting, curves and ploughing

It is common practice for the excavated diameter to be slightly larger than the diameter of the shield skin. This minimum overcut reduces the friction on the TBM, reducing the thrust needed. It is also common to provide soft and mixed ground TBMs with the facility to increase the overcut, using extendable overcutting tools. The effect of overcutting is to leave a large void outside the shield skin. In soft clay or loose sand this void is liable to close before the tail void grouting is carried out. Other causes of voids outside the skin are the effect of driving round curves and ploughing, where the shield is driven with ‘look-up’ or ‘overhang’. The potential for voids outside the skin can be expressed in terms of volume loss, where the volume of the void is expressed as a percentage of the unit volume of the tunnel. The potential volume losses from these factors for a particular TBM are given in Table 3. The TBM had a minimum cut diameter of 6.46 m, a skin diameter of 6.44m and a length of 8.1 m.

Table 3. Potential size of voids around a 6.46m diameter TBM, expressed as volume loss

Potential cause of void around Skin of TBM	Potential size of void, expressed as volume loss, %
Difference between minimum cut diameter and skin diameter	0.6
Extending overcutters by 70mm	5.00
200m curve	0.62
50mm ‘look-up’	0.79

These potential values for volume loss are significant, given that Anon (2005) suggests that 1% is achievable, as a value for the total volume loss, for well conducted tunnelling, irrespective of ground conditions. In Singapore, this value has not been achieved for the majority of the tunnelling in the soft clay and loose sands of the Kallang Formation (Shirlaw et al, 2003b). However, the tunnelling in Singapore has generally not included the systematic injection of bentonite grout around the shield skin recommended in Anon (2005); such injection should significantly reduce the volume loss from these causes.

Detailing the segmental linings to minimize leakage

Pre-cast concrete segmental linings include an enormous number of joints. A typical 6 m diameter tunnel with five plates and a key, with 1m long segments, includes over 18 kilometers of joint for every kilometer of tunnel. Ensuring that these joints are properly sealed is important, particularly if no secondary lining is to be used. Leakage at

the joints will, potentially, lead to corrosion of the lining and of equipment installed in the tunnels. Waterproofing of joints between segments is primarily reliant on gaskets installed on the segments. As discussed in Anon (2005) such gaskets are now commonly made of either Ethylene Polymer Diene Monomer (EPDM) or hydrophilic materials, which swell when in contact with water.

Over the last 22 years, a number of different gaskets have been used in Singapore. Shirlaw et al (2006) list the various gaskets that have been used for the subway tunnels in Singapore. These are summarized in Table 4.

Table 4. Gaskets used for the Singapore subway tunnels

Phase or Line	Gaskets used
Phase 1	One of (depending on construction contract) Bitumastic sealing strips Composite neoprene and bitumastic sealing strips Neoprene gaskets Hydrophilic gaskets, with up to 10 times swelling capacity
Phase 2	Hydrophilic gaskets, with up to 10 times swelling capacity
North-East line	Two seperate gaskets, one EPDM, one hydrophilic with up to 4 to 6 times swelling capacity
Circle Line	Single gasket formed by co-extruding EPDM and hydrophilic material

The gaskets used for the early tunnelling included bitumastic and composite neoprene and bitumastic sealing strips. These performed poorly. Leakage and corrosion have been evident, and regular on-going maintenance has been necessary.

For the North East Line, opened in 2003, the specifications required two gaskets on every segment: one EPDM and one hydro-swelling. The overall performance of these gaskets, based on the amount of back-grouting required to meet the specifications, was better than that of the gaskets used for the earlier tunnels. However, particular problems included:

(a) high expansion (6 x or more) hydro-swelling gaskets could not meet the requirements for long-term performance;
(b) low expansion (less than 6 x swelling) hydro-swelling gaskets could not seal the joint over the full range of possible gaps at the gasket location; and
(c) problems with ring grouting resulted in large steps developing on some joints, up to 75mm on the circumferential joints. Some of these steps were so large that the gaskets were either not in contact or had so little contact that they could not perform satisfactorily.

For the Circle Line composite gaskets are being used. These comprise an EPDM ‘carrier’, with co-extruded hydro-swelling material on the meeting face of the gaskets. Compared with the double gaskets used for the North East line, there is only a single line of defense, so any damage to the gasket will cause a breach in the waterproofing system. However, the advantages of the gasket are that:

(a) based on testing, the gasket should provide a seal over the full range of the gap that can occur due to the maximum design deformation of the lining, and over the design life of 120 years;
(b) the use of hydro-swelling material should help to seal at the corners of the segments;
(c) the use of a single gasket allows a wider gasket than either of double gaskets; a wider gasket gives greater tolerance to steps;

Another major change for the segmental linings used in Singapore has been the change from a caulking groove to a channeling groove. Caulking is a traditional means of improving the watertightness of segmental linings. However, unless the whole tunnel is caulked, leaks can bypass any local areas of caulking, and simply migrate to other areas. A simpler concept is to modify the old caulking groove, so that water can be channeled along the circumferential joints and into the scupper drains. The channeling groove is modified from the old caulking groove, to allow a seal to be pressed into the groove, and retained there, while leaving a channel beneath the sealant. The purpose is to allow leakage to flow around the groove, but prevent it spreading over the face of the segments or dripping onto rails or other equipment in the tunnel.

Tail void grouts

As discussed above, effective tail void grouting is a critical factor in keeping the lining in shape and avoiding excessive steps, and thus to the performance of the gaskets. Poor grouting practice can have consequences even more severe than this. Delayed grouting around the segmental lining was reported for a tunnel that collapsed in Hull (Grose & Benton, 2005). Recommendations for grouting in Anon (2005) include:

(a) the use of grout pipes that extend along the tail skin of the TBM;
(b) a minimum of 4 injection ports;
(c) double grout lines at every port; and
(d) control primarily by pressure, but with volume also being recorded.

The recommendations are good in terms of ensuring that the tail void is filled, and filling occurs as quickly as possible. Grouting on this basis should also limit settlement due to inward movement of the ground as the ring comes out of the skin. However, in addition to these items, it is necessary to ensure that the grout holds the ring in place during the shoves for subsequent rings. This will not be the case if the grout is fluid, and has low internal shear, for a long period after the grout is injected. Bezuijen & Talmon (2005) have measured the large flotation forces that the lining is subjected to before the grout sets. If the grout is fluid and has low internal shear, the ring will be free to move around under flotation and shove forces. To prevent this it is necessary to use a grout that either sets rapidly after injection, or a high shear grout. High shear grouts were studied as part of the EUPALINOS project (AFTES, 2002). Shirlaw et al (2004) discuss the tail void grouts, and the performance of those grouts, for a number of projects.

Site investigation for soft and mixed face TBM tunnels

The BTS Joint Code of Practice for Risk Management of Tunnel Works (Anon, 2003) requires there to be ‘sufficient’ site investigation carried out by or on behalf of the Client before the work is tendered for construction. This raises the issue of what is ‘sufficient’.

The US National Committee on Tunnelling Technology (1984) recommended, as a general rule-of-thumb, there should be 1.5 feet of borehole for every one foot of tunnel. Subsequent studies have been carried out on the tunnelling for the Sheppard Subway (Westland et al, 1998) and Singapore’s North East Line (Shirlaw et al, 2003a). Both of these studies came to the conclusion that about 0.8m of borehole for every 1m of twin tunnel appeared to give adequate information to limit risk. The latter studies were for cities with a long history of underground construction. This prior experience was probably a major factor in reducing the optimum borehole frequency compared with areas without such experience.

Borehole frequency is only the crudest possible measure of the extent of a site investigation. Apart from the general stratigraphy along the alignment and typical geotechnical parameters, such as strength, stiffness and permeability, particular issues for machine tunnelling include:

(a) grain size, for machine type and conditioner selection;
(b) groundwater levels and gradients;
(c) abrasivity;

(d) tensile strength (for rock);
(e) gas, either free or dissolved;
(f) chemical contamination (for effect on gaskets and grouts);
(g) faults;
(h) free swell potential (for rock and weathered rock);
(i) old wells, or other man-made open paths that could lead to loss of slurry or compressed air;
(j) natural open paths, such as karstic features; and
(k) obstructions, such as piles or old sea walls

On one major project in Singapore, the Deep Tunnel Sewer System (DTSS), every one of the six tunnelling contracts was completed late. The drive on one tunnel was completed ten months after the project was supposed to have been handed over, which would indicate that the full delay would be between one and two years. All of the contracts involved EPB tunnelling. The delays were ascribed to '*Mr Ground is totally worse than anyone had predicted* (sic)' (Paulo, 2004), which would suggest that a very limited initial site investigation, provided to tenderers, was a factor in the delays.

CONCLUSIONS

In answer to the question posed in the title of this paper, the use of modern TBMs for soft and mixed ground tunnelling provides both solutions to old problems and new problems as to how to best use that new technology.

In Singapore, there has been extensive soft ground tunnelling over the past nine years, mostly using EPB and (more recently) slurry machines. Incidents of large, localized loss of ground have been occurring with greater frequency than was the case using more traditional methods. There have also been major delays on some EPB driven tunnels. Many of these problems are solvable, and the publication of the BTS study (Anon 2005) should help to set a higher benchmark for TBM tunnelling, and to avoid many of the problems.

Many of the problems have occurred simply because it takes time for the industry to appreciate how best to utilize the new technology. Identification of the ground conditions and the machinery required to tunnel successfully in those conditions are import. Equally important are the organization and the planning for the tunnelling.

There is a risk that we become dazzled by the sophistication of the new technology, and forget that the human factor is still critical to successful tunnelling. It is worth remembering that the various problems outlined in this paper have all occurred on tunnels driven by experienced contractors, with experienced tunnel managers and operators. A well selected machine and an experienced TBM operator are a good basis for success, but are no a guarantee of it.

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ARE WE PREPARED FOR CAVERN DEVELOPMENT IN HONG KONG?

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Abstract

Since 1982, the Geotechnical Engineering Office of the Civil Engineering and Development Department, in collaboration with the relevant Government departments and regulatory authorities including the Planning Department, the Buildings Department, the Fire Services Department and the Lands Department, has conducted a series of studies on the planning, design, construction and regulatory control on the development of underground caverns in Hong Kong. The outcome of some of these studies has been incorporated in the Buildings Ordinance, the Hong Kong Planning Standards & Guidelines, Geoguide 4 (Guide to Cavern Engineering) and the Guide to Fire Safety Design for Caverns. The Office also subsequently assisted Government in considering and processing cavern development proposals for some infrastructure projects. The current state-of-practice of cavern development is that established in mid-1990. While there are administrative and technical issues that need to be addressed to facilitate enhanced use of underground space, from the cases of projects using cavern solutions, the potential for more active use of caverns for infrastructure development in Hong Kong still remains.

INTRODUCTION

Land is a scarce resource in Hong Kong. The hilly terrain of Hong Kong offers a great opportunity for placing public facilities in underground caverns. Use of caverns helps meet the land demand for urban development with relatively minimal environmental impact. Consideration of using underground caverns by private companies for oil storage began in the 1970s. Since early 1980s, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department has carried out the necessary technical preparation work for cavern development in Hong Kong. Feasibility studies, which established the viability of cavern development, have been carried out, in collaboration with the relevant Government departments and regulatory authorities including the Planning Department, the Buildings Department, the Fire Services Department and the Lands Department. Technical guidelines and standards on planning, investigation, design, construction and regulatory control for caverns have been published. Various issues on cavern development, including potential uses of rock caverns, design and construction of rock caverns, and planning and finance for rock caverns were covered in Malone and Whiteside (1989). Howells and Chan (1993) and Malone and Chan (1993) presented the development of regulatory framework and

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the status of rock cavern development in Hong Kong respectively.

The massive hard crystalline igneous rocks underlying most parts of Hong Kong are particularly suitable for large-scale cavern development. Furthermore, the successful construction of a number of public facilities in Hong Kong, namely, the Tai Koo MTR concourse, Mount Davis refuse transfer station, Stanley sewage treatment plant and Kau Shat Wan explosives depot, have demonstrated that caverns can be cost-effective alternative, while yielding additional environmental, safety and security benefits for many civil engineering applications.

This paper first reviews the past work on cavern related activities in Hong Kong, including (i) technical studies, (ii) institutional arrangements, (iii) technical guidelines and (iv) cavern construction. Second, the paper discusses the prospect of promoting the use of underground caverns for public and private facilities in Hong Kong.

TECHNICAL STUDIES

General

Since early 1980s, the GEO has commissioned or undertaken a series of studies to examine the technical feasibility of using underground space for a wide range of public and private facilities. These studies are summarized below.

Underground Oil Storage Study

In response to interest expressed by oil companies to use underground rock caverns for oil storage, in early 1980s the GEO commissioned a study to examine the geotechnical aspects of such operations (Neste 1982a, 1982b). The study described in general terms engineering geological zoning of areas for construction of underground oil storage caverns and provided general guidelines on geotechnical aspects for control of siting, design, construction and operation of underground oil storage caverns. The study did not identify any significant geotechnical problems, but the proposals were not taken forward by the developers at that time.

A Study of the Potential Use of Underground Space

Investigation of potential viable uses for large man-made underground spaces in rock caverns was an initiative of the Metroplan for the 1990s, being a response of the Government's urban planning strategy to the continuing demand for usable land. The Metroplan recognized that a systematic study was necessary to establish the opportunities for cavern development in Hong Kong. Between 1989 and 1990, the GEO commissioned a study of the potential use of underground space (SPUN), which comprised two phases. Phase 1 examined the potential use of space underground and Phase 2 carried out outline feasibility studies for seven schemes to test the concept of cavern development at four sites (OAP 1990). The seven schemes considered at four sites were (i) sewage treatment works and refuse transfer station at Mount Davis, (ii) container freight station, tractor and trailer park, and oil and LPG storage at Tsing Yi, (iii) warehousing at Chai Wan, and (iv) commercial, and government, institution or community (GIC) development at Quarry Bay.

The studies were based primarily on existing geotechnical information, supplemented by engineering geological mapping as appropriate. The studies demonstrated that the schemes considered were not only technically feasible but that they could give substantial economic

savings through release of land and environmental benefits to the community in general.

Cavern Project Studies

A further study, referred to as CAPRO (Cavern Project), was carried out in early 1990s to determine the engineering feasibility and develop preliminary designs, cost estimates and programmes for cavern proposals for a Government warehouse at Chai Wan and a refuse transfer station at Mount Davis with a view to confirming the engineering suitability and economic viability of the cavern proposals. Besides reviewing existing information, the studies involved site investigation and laboratory testing works, geological mapping, and consultation with the relevant regulatory agencies and Government departments. The studies confirmed the viability to construct the two facilities in a rock cavern as an alternative to an above-ground scheme (OAP 1991a, 1991b).

Cavern Area Studies

The GEO undertook an in-house project, referred to as Cavern Area Studies (CAS), between 1992 and 2000. The project classifies the suitability of land for rock cavern development at North Lantau, Hong Kong West, Hong Kong East and Kowloon, based on the consideration of surface developments, underground facilities, engineering geological conditions and topographical constraints (e.g. Roberts 1993; Table 1). The two main engineering geological criteria are proximity to faults and land elevation, reflecting the importance of major weak zones or cavern stabilization requirements.

Table 1. Cavern Land Classification used in CAS (Roberts 1993)

Cavern Land Class	Suitability	Description
IA	High	above 60 mPD - at least 50 m from major underground installations, photolineaments or faults
IB	High	below 60 mPD - at least 50 m from major underground installations, photolineaments or faults
IIA	Medium	within 50 m of major photolineaments or faults
IIB	Medium	within 50 m of major underground installations
III	Low	beneath surface development

INSTITUTIONAL ARRANGEMENTS

General

With the technical feasibility of using underground space demonstrated through the SPUN and CAPRO studies, the Hong Kong Government has then formulated the necessary administrative arrangements to help promote the use of cavern as a viable option for a variety of facilities.

Town Planning Guidelines

In recognition of the potential of the use of underground space, a set of planning guidelines was prepared in 1991. Section 2 of Chapter 12 of the Hong Kong Planning

Standards and Guidelines (HKPSG) (PD 2002) outlines the role of Planning Department, Lands Department, the GEO and other relevant Government departments in the planning and development of underground rock cavern.

Under the guidelines, a CAS is the first of a number of steps in the planning process for rock cavern development. Upon identification of suitable sites, the GEO is to carry out pre-feasibility studies on these sites with a view to determining the suitable location, orientation, layout, access and rough cost estimate. The planning process also includes planning procedure (site reservation), project planning (engineering considerations and utility provisions), land disposal and project implementation.

Public sector works should follow the standard Government procedures as given in the guidelines. For private cavern project, Planning Department will prepare a planning brief listing the development parameters and technical requirements. The planning brief will form the basis for drawing up the lease conditions and the project should follow the normal approval procedures as surface development.

Buildings Ordinance

In Chapter 123, Section 2(1) of the Buildings Ordinance, "....building includes....caverns adapted or constructed to be used for the storage or petroleum products,....cavern or any underground space adapted or constructed for occupation or use for any purpose including its associated access tunnels and access shafts.....". As such, underground caverns come within the ambit of the Buildings Ordinance and relevant submissions and approvals are coordinated by the Building Authority in consultation with other relevant Government departments. Furthermore, the fire safety requirements stipulated in the Code of Practice for Minimum Fire Service Installations and Equipment published under Section 16(1((b) of the Buildings Ordinance are also applicable to underground cavern situations.

Practice Note PNAP 177 (BD 1995) makes reference to the SPUN study. With respect to guidance on cavern construction, PNAP 177 lists HKPSG, Guide to Cavern Engineering and Guide to Fire Safety Design for Caverns. The contents of the last two documents are described below.

TECHNICAL GUIDELINES

General

Corresponding to the institutional arrangements, the regulatory framework also includes the development of technical guidelines to provide guidance for the implementation of cavern works. These include the publication of guidance on cavern engineering and fire safety design. A few technical guidance notes, which are of relevance to cavern development, have been promulgated recently.

Geoguide 4: Guide to Cavern Engineering

Following the SPUN and CAPRO, GEO commissioned the preparation of Geoguide 4 "Guide to Cavern Engineering" (GEO 1992). The document (i) concentrates on the geotechnical aspects of cavern engineering and aims at providing guidance on cost-effective and safe-integrated designs, and (ii) covers geological conditions and its implications for cavern planning and design, site investigation, design concepts and practices, and monitoring and maintenance.

Guide to Fire Safety Design for Caverns

The "Guide to Fire Safety Design for Caverns" (BA and FSD 1994) provides design guidelines on fire safety in underground caverns. It notes that the guidelines would be expanded and updated with experience gained in the construction and operation of underground cavern projects. The guidelines are applicable to underground rock caverns for use by public utilities of low population nature such as sewage treatment works, refuse transfer stations and water service reservoirs, where an effective management scheme is operated and a satisfactory contingency plan for emergency situations is in place.

GEO Technical Guidance Notes

Technical Guidance Note (TGN) No. 5 (GEO 2004) provides the sources of information for the planning of site investigation in Hong Kong. For cavern development, of particular importance are the geological maps and memoirs prepared by the Hong Kong Geological Survey of the GEO and the ground investigation data housed in the Geotechnical Information Unit of the Civil Engineering Library. The guidance given in TGN 24 "Site Investigation for Tunnel Works" (GEO 2005a) and TGN 25 "Geotechnical Risk Management for tunnel Works" (GEO 2005b) is also applicable to cavern development. These technical guidance notes are updated regularly to incorporate the latest technological development for geotechnical practice.

CAVERN CONSTRUCTION

General

In Hong Kong, the "first generation caverns" (Malone and Chan, 1993) constructed in 1980s were tunnel networks for water supply (e.g. valve chamber of Western District Aqueduct in 1984) or railways (e.g. MTR stations at Sai Wan Ho and Tai Koo in 1985). The

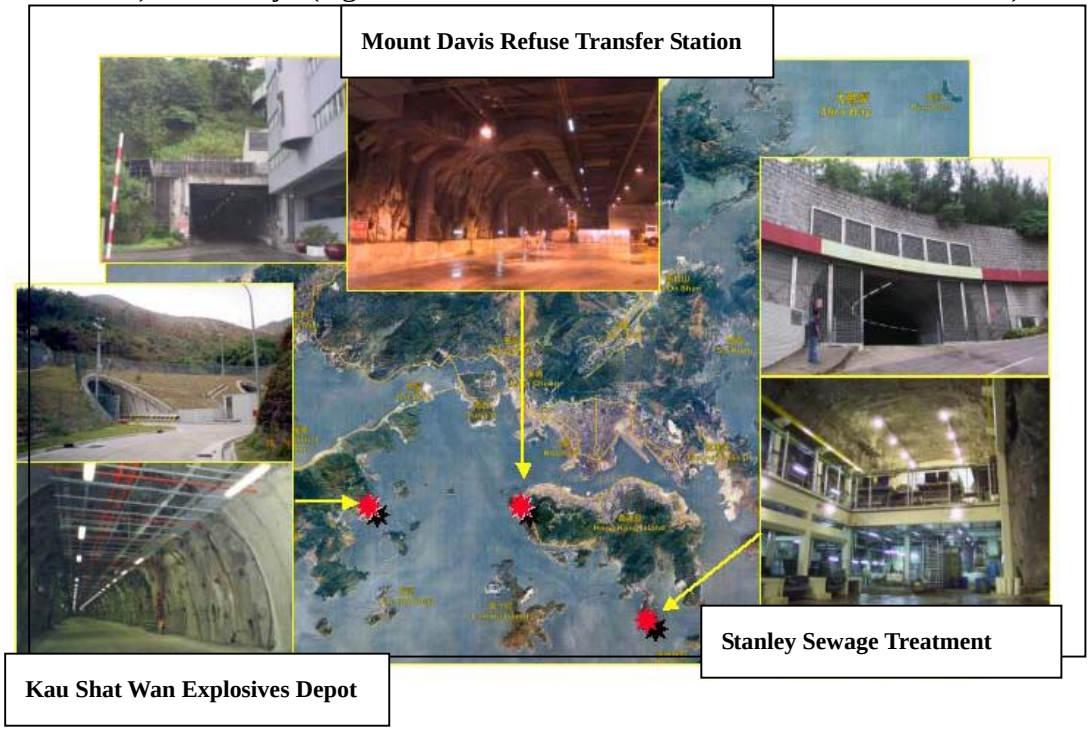

Figure 1. Locations and general views of the three "second generation caverns".

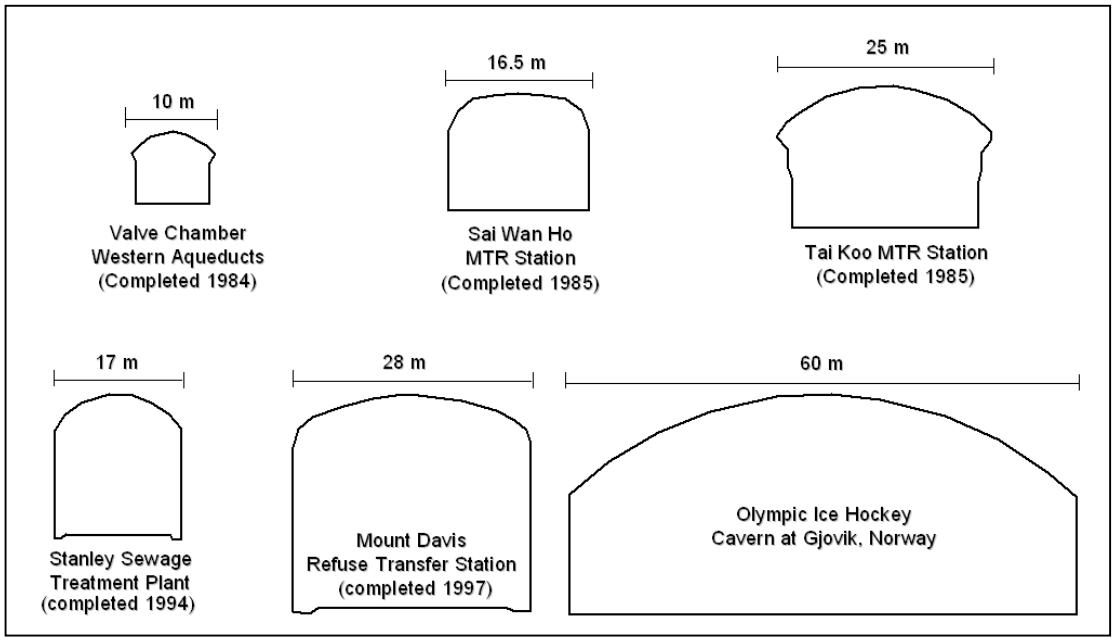

Figure 2. Cross Sections of Rock Caverns in Hong Kong and the Olympic Ice Hockey Cavern at Gjovik in Norway.

three "second generation caverns" constructed in 1990s were purpose-built for environmentally unattractive facilities (Figure 1). Figure 2 compares the cavern cross-section in Hong Kong with the Olympic ice hockey cavern in Gjovik of Norway.

Stanley Sewage Treatment Plant

The underground cavern for the Stanley Sewage Treatment Plant was constructed at the northern end of Stanley peninsula, adjacent to Stanley Prison. The basic arrangement comprises three general areas – main treatment and disinfection, services and an access tunnel. The main cavern, with maximum dimensions of 17 m wide, 17 m high and 130 m long, is formed in massive granite with widely spaced joints and a 10 m-wide fault zone.

The Sewage Treatment Plant serves a population of 27,000 in Red Hill, Tai Tam, Ma Hang, Chung Hom Kok and Stanley. The factors determining the choice of an underground option were (i) lack of a suitable surface site in surrounding area to accommodate the plant, and (ii) the scheme was relatively cheap as compared with land surface option. Excavation of the cavern commenced in 1992 and completed in mid-1994. Conventional drill and blast techniques were the method of construction and multiple heading and step-by-step benching were used to excavate the fault zone (Oswell et al. 1993).

Kau Shat Wan Explosives Depot

Due to large-scale urban development planned on the West Kowloon Reclamation and in the vicinity of Stonecutter Island, a new explosives complex was required to replace the facilities on Stonecutter Island. A site at Kau Shat Wan on Lantau Island was identified to be technically feasible for storage of explosives underground. Ten small caverns (each 13 m wide, 6.8 m high and 20 m long) connected through passageways were formed in granite and feldsparphyric rhyolite, which were relatively free from major geological weak zones.

The cavern option for the explosives depot was selected because it would (i) offer substantial environmental and security benefits for the scheme and (ii) require less extensive

site formation and reclamation works, and (iii) demand lower capital and operational costs. Excavation of the caverns was by conventional drill and blast techniques. The works commenced in 1994 and the whole underground complex was completed in 1997.

Mount Davis (Island West) Refuse Transfer Station

The search for a site to house a refuse transfer station serving the western part of Hong Kong Island began in 1991. The objective was to find a waterfront site for refuse waste compaction station, which could facilitate rapid transportation of the waste by sea for disposal at the West New Territories Landfill. A site under Mount Davis was identified for its operational, technical and financial viability.

Two caverns and associated access tunnels were excavated in volcanic tuff of generally good rock quality with a few fracture zones, using conventional drill and blast techniques. The cavern housing the tripping hall is 28 m wide, 12 m high and 66 m long and the one housing the compactor hall is 12.5 m wide, 7 m high and 160 m long. The works commenced in 1995 and the whole transfer station was completed in April 1997.

DISCUSSION

General

From the successful implementation of the tunnel-related "first generation caverns" in 1980s to the purpose-built "second generation caverns" in 1990s, it has demonstrated that caverns for some public facilities are the right solutions on economic, environmental and, in some situation, security grounds. The HKPSG guidelines (PD 2002) note that rock cavern development is considered suitable for land uses such as commercial, industrial, GIC and utilities (Table 2). However, the use of underground space needs to be competed with other traditional options of placing facilities on surface land. The following sections examine the prospect for enhanced use of underground space in Hong Kong.

Benefits and Cost

Underground cavern construction is generally considered as capital intensive with the project cost, programme and risk of adverse consequences being directly related to the ability to manage the ground and groundwater conditions. Depending on the types of facility, operating facilities constructed underground may require special considerations such as possible constraints on access and traffic arrangement, acoustic treatment to reduce noise level, ventilation and suppression measures to improve air quality, specific fire safety measures, and monitoring and maintenance of cavern structures. However, there are some distinct benefits of using underground space.

(1) Caverns sites made available to house new public facilities or to re-provide old facilities for which acceptable sites could not be found on surface land, e.g. facilities in need that are environmentally objectionable, unattractive, or that require a large piece of land to support.
(2) Land released for other uses thereby reducing pressure for land and development costs, and possibly enhancing Government land revenue, especially at prime sites.
(3) Caverns can be a cost-effective solution, especially for environment-, safety- and security-sensitive facilities.
(4) Use of caverns could help contain landslide risks in the long term as the need for site formation and developments close to steep hillsides could be correspondingly reduced.

(5) Placing facilities in underground space could also relieve the pressure for extensive reclamation for developable land.
(6) The rock excavated to form the caverns will provide a source of rock products such as aggregates for concrete, seawall armour rocks, etc.

Table 2. Possible uses for rock cavern development (PD 2002)

Land Use	Possible	Unsuitable
Residential		X
Commercial		
Office	X	
Retail	X	
Entertainment	X	
Hotel		X
Ancillary parking	X	
Commercial / Residential		
Commercial elements	X	
Residential elements		X
Industrial		
Industry	X	
Storage / warehousing	X	
Oil bulk storage	X	
LPG bulk storage	X	
Education		X
Vehicle Parking	X	
Open Space		X
Government / Institution / Community (GIC)		
Abattoir	X	
Civic centre / community centre	X	
Hospital / polyclinic /clinic	X	
Incinerator		X
Indoor games / sports hall	X	
Market (wholesale and retails)	X	
Refuse transfer facilities	X	
Sewage / water treatment	X	
Service reservoir	X	
Transport connections and networks	X	
Cemetery / crematorium		X
Columbarium / mausoleum / mortuary	X	
Utilities		
Power stations / electricity sub-stations	X	

Key Issues for Enhanced Use of Caverns

In December 2003, the Geotechnical Division of the Hong Kong Institution of Engineers formed a Working Group on Cavern and Tunnel Engineering (previously called Interest Group on Cavern Engineering), which comprises representatives from the GEO, learned and professional societies, academia, consultants and contractors. One of the Working Group's

terms of reference is to identify key issues on and actions required for appropriate use of caverns and tunnels in Hong Kong. Based on a review of state-of-practice as outlined in the previous sections, the Working Group has identified the following issues.

(a) Planning issues – possible measures include streamlining the planning process, and designating selected sites for cavern development and inviting development proposals from both public and private sectors.
(b) Land matters – streamline the process with respect to land grant/land sale for private development, special lease conditions, costing of surface land released, and land premium for caverns.
(c) Project development – launch strategic public projects suitable for adopting cavern option, e.g. sewage treatment plant, refuse transfer station, service reservoir, etc.
(d) Cavern Engineering – update and expand Geoguide 4 (GEO, 1992) to include recent advances in cavern engineering, such as analytical methods for support design, blasting design and assessment, groundwater effects, and construction techniques.
(e) Fire Safety – provide additional guidance for caverns to be used for housing high population facilities.
(f) Radon Gas – develop local code with respect to radon emissions for caverns to be used for housing high population facilities.

Progress

The administrative and technical issues identified above would take time and concerted efforts by many parties to systematically address them. In late 2004, with a view to promoting the wider use of underground cavern, the ETWB (Environment, Transport and Works Bureau) has requested the works department to identify a list of planned or anticipated projects that would involve provisions of refuse transfer station, water treatment plant, sewage treatment plant, service reservoirs, dangerous goods storage, incinerator and abattoir, that local communities may be reluctant to accept in their neighbourhood. Placing these facilities underground would minimise the nuisance to the communities and their impact on the environment. Several potential projects have been identified. This identification process furnishes an impetus for works projects to consider cavern option at the earliest possible stage and perhaps would provide the thrust for developing the “third generation caverns” in the 2000s and beyond.

CONCLUSIONS

The massive hard crystalline igneous rocks underlying most parts of Hong Kong’s hilly terrain are suitable for large-scale cavern development. Regulatory framework has been in place and technical expertise has been established through the construction of the first and second generation caverns. Since completion of the three purpose-built caverns in mid-1990, rock cavern option has not been seriously considered for public infrastructure development in Hong Kong. The apparent geotechnical uncertainties of constructing facilities underground probably play a part in this lack of consideration. However, the clear benefits of housing nuisance facilities underground and the release of surface land could outweigh the geotechnical uncertainties, which are generally surmountable with careful site selection and geotechnical studies. To further promote the use of underground space in public and private sectors, particularly for housing high population facilities, it would require concerted efforts in dealing with the administrative and technical issues identified.

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RISK ASSESSMENT AND MANAGEMENT OF TUNNEL PROJECTS IN HONG KONG

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Abstract

This paper draws upon experience gained from a series of major tunnel projects in Hong Kong that highlight the current state of the art of risk identification, assessment and management in Hong Kong. A Risk Management Plan is outlined that has been used to conduct detailed risk analysis of three recent major tunnel projects. The results from this work highlights the processes employed; the quantities of different types of risks identified before and after mitigation measures are applied; and many of the high risks anticipated for different types of tunnel related works. The projects include a cable tunnel to be driven by TBM; an underground railway project in urban areas and a drill and blast highway tunnel in Hong Kong's semi-rural areas. The analysis yields some interesting results.

INTRODUCTION

Tunnelling conditions in Hong Kong, as illustrated previously by McFeat-Smith et al (1985) vary substantially from rock tunnelling in rural areas in massive competent granite and highly fractured volcanic rocks to extreme mixed face tunneling below intensely developed urban areas. The latter has led to the application of several hard rock Earth Pressure Balance TBMs (EPBMs) and hard rock Slurry TBMs. Both conventional tunnelling in areas of low rock cover and EPBM applications have on occasion resulted in sinkholes to street level (McFeat-Smith, 1998, 2001) whilst the slurry machine applications appear to be more successful in the local mixed face conditions. Works for recent highway tunnel projects such as the Eagles Nest Tunnels have involved conventional excavation of large span tunnels (up to 18m) in mixed face conditions requiring close control of ground movements through detailed excavation sequencing and rapid temporary support installation in conjunction with close ground movement monitoring.

Water inflows in local tunnels (see McFeat-Smith et al 1998) have become a major concern following the completion of the Tai Po to Butterfly Valley aqueducts where cumulative inflows of up to 250 l/s at 800 m head were successfully managed, in sharp contrast to the works for Phase 1 SSDS contracts (Grandori et al, 2001) where inflows from sub-aqueous rock tunnels resulted in settlements in the order of 900-1500mm in urban areas as well as a series of legal disputes. The latter has given rise to concerns about the current one sided Hong Kong Conditions of Contract that in extreme cases places (and even generates) unnecessarily high risks for contractors. This appears to be leading to an improved outlook from ETWB on the subject of risk allocation and risk sharing for contracts involving tunnelling works.

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In summary, the local tunnelling industry undertakes a relatively high volume of tunnelling works in variable ground conditions, in many cases with extreme risks. Contactors have matched this with constantly improving technology. Nevertheless the scale, complexity and associated risks for tunnelling works planned for the following decade will present new demands and challenges upon all parties concerned with the works as well as third parties. Hence it is essential that much higher levels of risk control and risk sharing (see example in McFeat-Smith, 1986) be implemented on all new tunneling contracts if further failures and increasing levels of litigation are to be curbed. This paper attempts to show how this can be achieved by identifying best practices, opportunities and some of the more evident pitfalls. The risk management plan outlined in this paper presents what is considered to be a good quality risk assessment system in regular use by a few Contractors. It has been developed from that proposed by Hong Kong's MTR Corporation Ltd (Kam & Lai, 2001). The system relies upon the involved parties identifying risks for particular projects and helps in the quantification and mitigation of such risks as discussed in the following sections of this paper. The risk management system must be implemented as an integral part of the overall project management system, to provide assurance that the project shall be completed on time, on budget and in compliance with the specified contract and statutory requirements for materials and workmanship, environmental protection and health and safety.

RISK ASSESSMENT AND MANAGEMENT PLAN

Throughout the development of tunnel projects the parties involved are required to hold meetings to review risks associated with the various planning, design and construction methodologies and strategies being developed for the project. Through these review meetings the parties involved should endeavor to identify and quantify all significant risks that have been identified. The case histories analyzed in the following section provide a guide for this purpose that can be developed and structured in accordance with IMS risk evaluation system for financing and insuring tunnel projects (McFeat-Smith & Harman, 2005). For each risk identified mitigation and/or contingency measures to reduce the risks to a tolerable level need to be defined and priced as part of the estimation process. The proposed solution should represent the optimum solution, with the risks minimized and the mitigation and/or contingency measures priced. Potential risks can be categorized as R1 "unacceptably high" through to R4 "acceptable" risks as outlined on Table 1, which indicates the necessary timing of the action required. Quantification of each risk level is given on Table 2.

Scope and Objectives of the Risk Management System

The parties should implement a risk management system for the project that covers all elements of the works, with the primary objective of protecting the interests of all parties who may be involved in, or effected by, the project, including the General Public and the Client Although the risk management process should be basically the same for each type of risk, for ease of reference the risks can be categorized under: Programme; Health and Safety; Environmental; and Commercial Risks (see Table 2). The risk management system should ensure that:

(a) risks are identified for all aspects of the project;

(b) identified risks are evaluated as a product of their "frequency" and "consequence";

(c) identified risks are each allocated strategic treatment options from the choices of avoidance, transfer, mitigation or acceptance, then explained in detail;

(d) emergency and Contingency plans are established for all R1 and R2 level risks;

(e) resources are focused on mitigating the most significant risks;

(f) ownership for management of the risk and its related treatment and contingency measures are implemented by more than one staff but supervised and reported on by one manager;

(g) risk status is reviewed on a scheduled basis to identify if changing circumstances alter risk priorities and treatment for alteration of resources;

(h) participating staff are trained in all risk aspects related to their processes;

(i) records of all risk management activities are maintained;

(j) audits of risk management activities are implemented and effective; and

(k) senior management must drive risk management activities.

Table 1. Risk Levels and General Action Plans

Risk Level	Action And Timescale
R1	All initial risks at this level (i.e. prior to the application of mitigation measures) require an ***Outline Emergency and Contingency Plan***. Method Statements detailing mitigation measures are required for all such risks and these are to be cross-referenced to the Risk Register. Work shall not begin until mitigation implementation has been completed and verified. Mitigated risks must be reduced to an R3 level.
R2	All initial risks at this level (i.e. prior to the application of mitigation measures) require an ***Outline Emergency and Contingency Plan***. Method Statements detailing mitigation measures are required for all such risks and these are to be cross-referenced to the Risk Register. Work shall not begin until mitigation implementation has been completed and verified. Mitigated risks must be reduced to an R3 level or reported to the Board for resolution.
R3	Mitigation must be implemented before works begin.
R4	Mitigation may be implemented before works begin.

Risk Mitigation

Measures proposed to mitigate risks should be subject to review by a Risk Management Committee including all concerned parties. All mitigation measures need to be adequate to reduce the level of risk to a tolerable residual level. If in the opinion of the committee this is not achievable then the proposed methods or sequence of construction should be revised to provide a construction solution for which residual risks are tolerable. The committee should monitor and review the identified risks and their related timeframes, mitigation and implementation actions. The methodology for carrying out the Risk Review Meetings may for larger projects involve decentralization for management purposes. Register owners should aim to update their registers within 2 days of the meeting and during as new information becomes available.

Risk Register Review Meeting Procedure

The Agenda should cover the following items:

(a) identification of New Risks;
(b) review of Risk Register to confirm status of mitigation for Active Risks;
(c) status of action statements from previous meeting;
(d) present for discussion risk management performance indictor statistics;
(e) confirm action statements from the current meeting;
(f) discuss performance measurement of risk management success; and
(g) other actions.

Table 2. Risk Matrix

CONSEQUENCE ASPECT	CONSEQUENCE TYPE	CONSEQUENCE						
		7	6	5	4	3	2	1
		Trivial	Negligible	Marginal	Serious	Critical	Catastrophic	Disastrous
COMMERCIAL	Cost	0.01%	0.07%	1.00%	5.00%	10.00%	20.00%	30.00%
PROGRAMME	Time	0 to 1	1 to 7	7 to 29	30 to 89	90 to 364	365 to 1826	>1826
STAFF / CONTRACTOR SAFETY	Fatality					<5	5 or more	
	Major Injury				<5	5 or more		
	Minor Injury (>3ds)			<5	5 or more			
	Minor Injury (<3ds)		<5	5 or more				
ENVIRONMENTAL	Stop works			7 to 29	30 to 89	90 to 364	365 to 1826	>1826
	Press			Y	Y	Y	Y	Y
	Prosecution		1	3	5	10	>19	
	Complaints		1	5	15	25	>25	

FREQUENCY CATEGORY	FREQUENCY DESCRIPTION (QUALITATIVE)	PROBABILITY (%) OF OCCURRENCE
A	Almost Certain	99.999%
B	Likely	75.000%
C	Probable	55.000%
D	More than Occasional and less than probable	42.000%
E	Occasional	30.700%
F	Possible	3.070%
G	Very Unlikely to Occur	0.307%
H	Remote	0.0307%
I	Improbable	0.00307%
J	Incredible	0.00031%
K	Impossible	0.0%

R3	R1	R1	R1	R1	R1	R1
R4	R1	R1	R1	R1	R1	R1
R4	R2	R1	R1	R1	R1	R1
R4	R3	R2	R1	R1	R1	R1
R4	R3	R3	R2	R1	R1	R1
R4	R4	R3	R3	R2	R1	R1
R4	R4	R4	R3	R3	R2	R1
R4	R4	R4	R4	R3	R3	R2
R4	R4	R4	R4	R4	R3	R3
R4	R4	R4	R4	R4	R4	R3
R4	R4	R4	R4	R4	R4	R4

RISKS ANALYSIS FOR THREE MAJOR HONG KONG PROJECTS

The following section summarizes key risks for different types of tunnels in Hong Kong based upon the Risk Management System outlined above.

Cable Tunnel - Urban Area Rock Tunnelling

Table 3 provides a distribution of risks identified at tender for a design–construct cable tunnel in a partly rural, partly urbanized area in rock with anticipated sections of high water inflows and soft ground tunneling conditions. The distribution of risks in each category merits detailed study and whilst some risks are project specific several trends are evident, namely:

(a) about 17% of the anticipated total of 125 risks are related to design and approvals for this design and construct contract;

(b) 42% of the risks are concerned with site establishment and surface works (portal/shaft);

(c) in this case, a very high 35% are related to concerns regarding the tunneling works, with a notable 6 no. R1 level risks remaining in the high risk R2 category even after mitigation; and
(d) in general, and excepting these 6 no. cases, mitigation measures are considered to be effective.

Table 3. Distribution of Risks Identified before and after Mitigation for Cable Tunnel in Semi-Rural Area in Rock with Sections of Wet and Potentially Soft / Poor Rock Conditions

Structure	No. of Risks / Risk Level before Mitigation				Total No. of Risks
	R1	R2	R3	R4	
Design/Approvals	3	14	4	0	21
Mobilization / Set-Up	0	4	17	1	22
Portal	2	7	3	1	13
Shaft	1	9	7	0	17
TBM + Enlargements	11	14	18	1	44
E&M Testing etc	1	3	4	0	8
Total ∑	18	51	53	3	125
	No. of Risks / Risk Level after Mitigation				
Design/Approvals	0	3	14	4	21
Mobilization / Set-Up	0	0	7	15	22
Portal	0	1	9	3	13
Shaft	0	2	11	4	17
TBM + Enlargements	0	6	20	18	44
E&M Testing etc	1	0	2	5	8
Total ∑	1	12	63	49	125

Table 4 identifies 13 of the 18 no. R1 level risks (before mitigation) of concern for the project and for this type of tunnelling methodology and conditions. This provides a valuable opportunity checklist for Clients and Engineers to address such problems prior to tendering. Whilst an optimistic view of blasting mitigation measures in urban areas is expressed here concerns about interfaces with approving authorities are sustained in the following cases discussed herein.

Table 4. R1 Level Risks and Appropriate Mitigation Measures for Cable Tunnel in Rock with Sections of Adverse Geological Conditions

Risk	Possible Mitigation Measures
Objections from public and Government resulting in delays to works	Conduct PR, co-ordination with District Council, early consultation with all concerned, dust and noise protection
Delays for blasting submissions and permits	Early consultation with BD and GEO Mines Division, trials blasting, re-assess requirements, minimize blasting
Difficulties with major utilities at portal	Early co-ordination with utilities, effective and timely protection measures
Unforeseen ground conditions – long fault zones, highly fractured and weathered rock	Use double shielded TBM, employ IMS for TBM specification and review of tunneling operational requirements
Unforeseen ground conditions – high water inflows and flooding on downhill drives	On site management of water inflows, advance probing and grouting, pumping as required. Develop Risk Sharing option
Ineffective surface ground treatment.	Limit GT from shaft to seal junction at shaft and tunnel

Risk	Possible Mitigation Measures
High settlement affecting roads, railway crossing, utilities	Further SI, hydrogeological study, employ IMS for special grouting expertise.
Mechanical/electrical delays; low TBM utilization	Provision for learning curves, system analysis, breakdown logic check system, experienced crews and supervision
TBM programme delays for enlargement and other works to be carried out simultaneously	Non TBM works only undertaken during non TBM production periods. Saturdays, planned maintenance periods etc.
Crushing or other traumatic injuries	Specialist training, emergency courses, first aid stations, safety procedures, specialist staff
Fire/explosion in tunnel.	Restricted materials in tunnel, specialist training, procedures, adequate systems and installations
Delays to equipment supply e.g. fans for ventilation	Early approval of equipment by SO. Careful monitoring and supervision of supply programme.

Underground Railway Project; Urban Area Mixed Face Tunnelling

Table 5 summarizes risks identified prior to construction for an Underground Railway Project with associated station, cut and cover and TBM tunnel sections. From this the following observations are made:

(a) out of a very high total of 439 anticipated risks about one third of these are related to design management, design and approvals for this design and construct contract;
(b) of the three forms of structure required the TBM tunnel has marginally more risks and the station, situated on a green field site in this case, has relatively low risks;
(c) mitigation in all but a few programme related cases is expected to be effective;
(d) a few R2 risks remain for TBM tunnel that require a high level of control and mitigation during construction.

Table 5. Distribution of Risks Identified before and after Mitigation for Typical Underground Railway Project in Hong Kong's Urban Areas

Structure	No. of Risks / Risk Level before Mitigation				Total No. of Risks
	R1	R2	R3	R4	
Station	3	3	28	22	56
Cut and Cover	11	31	55	12	109
TBM	26	38	60	4	126
Design/Approvals	29	20	56	41	146
Total ∑	69	92	199	79	439
	No. of Risks / Risk Level after Mitigation				
Station	0	0	12	44	56
Cut and Cover	0	1	45	63	109
TBM	0	5	56	67	126
Design/Approvals	2	4	34	106	146
Total ∑	2	10	147	280	439

Table 6 identifies key R1 Risk Levels (prior to mitigation) for tunnelling works in such extreme high risk mixed face tunneling conditions together with typical risk mitigation measures based upon experience gained in both Hong Kong and overseas. It should be borne in mind that this table focuses only upon R1 risks and not R2 risks for example. Safety, Environmental and E&M risks, such as those for the cable tunnel, are not repeated in this case. The observation made at the bottom of the table regarding the trend for Contractors to avoid

blasting in urban areas reflects overly conservative and entrenched attitudes amongst Hong Kong's approving authorities that greatly impacts upon the economics of drill and blast tunnelling. This cost is ultimately borne by the taxpayer.

Table 6. Typical R1 Level Risks and Appropriate Mitigation Measures for TBM Tunnelling in Extreme Mixed Face Conditions in Urban Area

Risk	Possible Mitigation Measures
TBM to excavate in challenging and unusually risky conditions	Consider purpose designed TBM using good specifications, reliable manufacturer with adequate resources
Late order of TBM + segment molds	Early resolution and approval of tunnel profile
Late manufacture of TBM + molds	Review suppliers workload, close supervision
TBM assembly delays	Close liaison with supplier, scrutinize programme
Portal/launch shaft delays – delays due to inadequate ground treatment	Early submissions and approvals, timely access for ground treatment (GT).
Compressed air (CA) blow out	Good SI and interpretations, MS and controls for CA interventions, emergency and contingency plans (ECP) available including TTM and clean up crews
Slurry leakage e.g. to utilities or roads	Appropriate MS and calculations, operator training and close control of TBM operational facilities, ECP
Unforeseen obstructions and ground conditions	Good SI, easy access to cutterhead chamber, real time monitoring of ground movements, method statement (MS) for removing obstructions, experienced crews and supervision, ECP
Sinkholes at interfaces or cutter change interventions	Good SI and interpretations, appropriate MS and controls for interventions, real time monitoring, operator training and close control of TBM operational facilities, ECP
Excessive settlement of utilities, roads, buildings	Real time monitoring, GT, experienced crews and supervision, pre-fixed TBM operating parameters
Ground borne noise and vibration exceeding acceptable levels.	Monitoring, PR exercise, manage working hours to minimize impact
TBM mechanical / electrical delays	Training, monitoring, system analysis, planned maintenance, programme logic breakdown control, experienced crews and supervision
Approvals for drill and blasting enlargements/ shafts in urban areas	This method has almost been rendered impractical in Hong Kong's Urban Areas. Use non-blasting techniques.

Highway Tunnel in Rural Area

Table 7 summarizes risks identified prior to construction of a typical wide span highway tunnel in a rural area in Hong Kong prior to and after risk mitigation. From this data the following observations are made:

(a) in this Engineer's design contract only 78 risks were identified;
(b) of these some 53% were expected to be related to the tunneling works;
(c) in almost all cases mitigation measures were expected to be effective.

Table 8 summarizes R1 level risks (prior to mitigation) identified prior to construction. Based on feedback after construction it was found that risks concerning watertightness specification, water inflows, grouting and lining installation were all successfully mitigated. Similarly risks concerning the excavation of wide span tunnels in very poor tunneling conditions were closely controlled by good management and quality temporary support systems on site. However, risks concerning conservative attitudes amongst approving

authorities regarding blasting control served to create significant delays to the construction of the tunnels.

Table 7. Distribution of Risks Identified before and after Mitigation for Typical Drill and Blast Highway Tunnel Project in Hong Kong's Semi Rural Areas

Structure	No. of Risks / Risk Level before Mitigation				Total No. of Risks
	R1	R2	R3	R4	
West Portal	1	2	8	8	19
Main Tunnels	6	0	12	28	46
East Portal	1	1	3	8	13
Total ∑	8	3	23	44	78
	No. of Risks / Risk Level after Mitigation				
West Portal	0	0	4	15	19
Main Tunnels	0	0	7	39	46
East Portal	1	0	1	11	13
Total ∑	1	0	12	65	78

Table 8. Typical R1 Level Risks and Appropriate Mitigation Measures for Typical Drill and Blast Highway Tunnel in Hong Kong's Semi Rural Areas

Risk	Possible Mitigation Measures
Shared use of access with interfacing Contractor will impose restrictions on spoil disposal	Investigate truck options, co-operate with adjacent Contractor, make allowances in programme
Highly conservative blasting constraints	Early submission of blast assessment reports and CNP, close liaison with GEO/M&Q, WSD, further SI, trial blasting on site, re-assess blasting constants
Noise sensitive receivers at portals – CNP for 24 hr working not granted	Full compliance with authorities, early quality submissions, close liaison with all authorities/ parties involved, PT exercise.
Unforeseen ground conditions – water inflows in excess of stringent watertightness specification	Detailed predictions of water inflows and grouting at tender using IMS and grouting experts, further SI after tender award, special hydrogeological studies
Advance probing and grouting results in major delays to excavation	Drill horizontal boreholes for early detection of inflows, inflow decay rate and for advance grouting
Cannot bring up tunnel lining without achieving watertightness specification	VE sessions concerning timing of installation of lining, ineffectiveness of post excavation grouting, Partnering with Client, Engineer and WSD representative.
Unforeseen ground conditions – faulting, mixed face conditions, low rockhead at portals	Further SI, high level IMS engineering management team on site for temporary support design, flexibility towards design development and excavation sequencing, close monitoring

CONCLUSIONS

The growing need to implement quality Risk Assessment and Management systems is self evident from initiatives from the re-insurance industry (Anon, 2003) and from the Hong Kong Government's response to this and the disastrous outcome of Phase I of the SSDS. The system outlined has evidently been used for several major tenders and into the construction stage. Feedback indicates that it is more than a worthwhile exercise and can in fact prove fundamental for the successful, profitable outcome of tunnelling projects in Hong Kong.

Like all risk assessment systems, without the benefit of exhaustive data on the frequency

of occurrences of particular incidents it is at best semi-quantitative and therefore relies strongly upon the degree of engineering experience and judgment of the various engineering teams involved. Hence, in order to ensure a degree of objectivity an independent overview by a non-stakeholder is considered essential.

The analysis conducted using this system for three typical but nonetheless live tunneling projects is revealing and quantifies concerns that our local Construction Industry faces on a daily basis. It should be borne in mind that, in addition to these large number of risks many more apparently minor risks including issues of currency variations and supplier interfaces for example also have to be managed. The types and numbers of risks in each form of construction and associated works are clearly identified together with the mitigation measures typically employed. Their expected level of effectiveness are also indicated.

The introduction to this paper clearly identifies why and what types of changes are required in the Hong Kong Conditions of Contract, and in particular, for the need to avoid placing incalculable risks upon Contractors in terms of time and money. The time for action and improvement is now.

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DEVELOPMENT OF GEOTECHNICAL CONTROL AND RISK MANAGEMENT FOR TUNNEL WORKS IN HONG KONG

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Abstract

In Hong Kong, the unexpected difficulties encountered with tunnel construction under the Harbour Area Treatment Scheme Stage I Project in the late 1990's provided the catalyst for the Government of the Hong Kong Special Administrative Region (HKSAR) to examine seriously the need for geotechnical control of government projects involving tunnel works[2]. A review of the literature indicates that 'unexpected' ground conditions have seriously hampered the construction of many tunnel projects both in Hong Kong and elsewhere in the world. These have resulted variously in loss of life, damage to property, major delays to completion of the projects and major financial losses to all parties involved, including the insurance industry. The insurers have instigated change. This has culminated in the publication of an international Code of Practice for Risk Management of Tunnel Works in late 2005. This paper brings together the various events and changes within the tunnelling industry, both locally and internationally, which have led to introduction of a new Government policy on geotechnical control for government tunnel works and development of standards on geotechnical risk management for projects involving tunnel works in Hong Kong.

INTRODUCTION

Mountains and hills with steep slopes, many of which descend directly into the sea, dominate Hong Kong's landscape. The mountains consist primarily of volcanic rocks, with some of the lower hills formed of granite. Low-lying areas tend to be formed of granite or sedimentary rocks. In places, hill-slope debris forms a mantle over the bedrock and alluvium fills some of the valleys (Sewell *et al*, 2000).

With an ever expanding urban community, land suitable for development in Hong Kong is always in short supply. The demand for land and the limitation imposed by the natural landscape dictate that underground construction is often the only feasible option. Early examples include the Tai Tam waterworks tunnel, constructed between 1883-1887 (Surveyor General's Reports, 1884 & 1885) and the Beacon Hill railway tunnel, constructed between 1907-1910 (Eves, 1908; Phillips, 1990). In the past thirty years, in particular, many tunnels and associated underground facilities have been completed, both by the Government and the private sector, for road and rail transport as well as for utilities, including water supply,

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[2] In this paper, the term 'tunnel works' means tunnels, shafts, caverns and associated underground facilities, however constructed.

drainage, sewage disposal and cables. Drill and blast, tunnel boring machine, mechanical excavation and cut and cover methods have been used. Whilst tunnel works continue to be constructed successfully, some have encountered problems during construction, e.g. the Mass Transport Railway (MTR) Modified Initial System and the Island Line (Clay & Takacs, 1997; MP, 1983a&b; SCMP, 1983). Such problems, usually associated with either inadequate knowledge or a lack of appreciation of the ground risks, have had significant impacts on public life or property. Government, whether as a regulator or as the project client, often bears the brunt, financially and politically, of any major problems that impact the public from construction of tunnel works.

Currently, there are a significant number of new private and public sector tunnel projects at various stages of development and construction, with geotechnical control and other input from the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (Figure 1). Government has a strong interest to ensure that these projects proceed smoothly and safely.

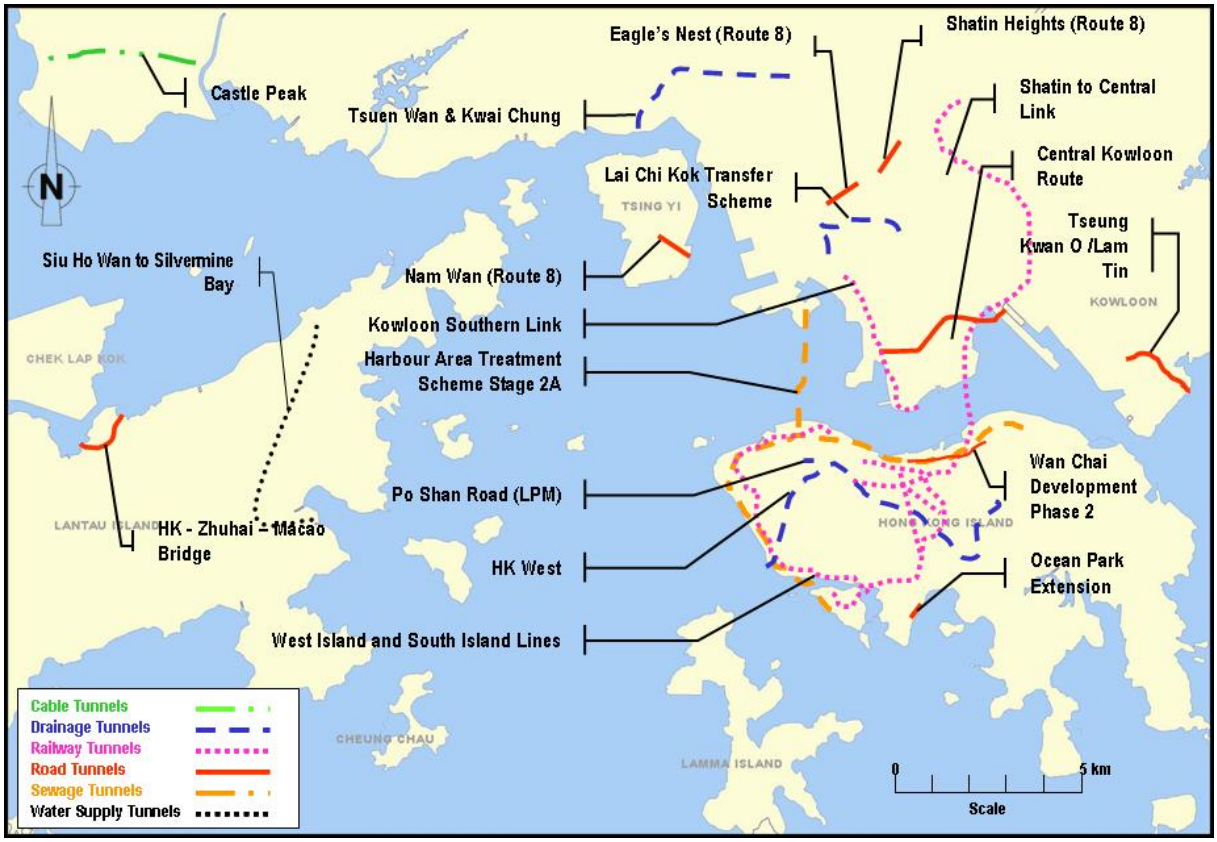

Figure 1. Recent Public and Private Sector Projects with GEO Input on Tunnel Works

REVIEW OF THE HATS STAGE I PROJECT

The Harbour Area Treatment Scheme (HATS), previously known as the Strategic Sewage Disposal Scheme (SSDS), is an overall sewage collection, treatment and disposal strategy, to deal with the water pollution of Victoria Harbour. Implementation of HATS was originally divided into four stages. Construction of the HATS Stage I project commenced in early 1995. It involved the provision of a system of deep tunnels 23.6km long for transporting sewage collected from a number of catchments around Victoria Harbour to a treatment works on Stonecutters Island. The tunnels were excavated largely beneath the sea at depths unprecedented in Hong Kong, ranging from 80m to 150m below sea level. They

were mainly in rock with a minimum cover of 30m in Grade III or better rock.

During construction, serious problems arose due to heavy water inflow into sections of the tunnels (McLearie, 2001). In addition to construction problems within sections of the tunnels, the water inflow led to extensive drawdown of the groundwater; in one section, significant ground settlements occurred up to 1.8km from the tunnel, resulting in major public concern. There was also significant delay in completion of the project and a substantial increase in cost. The extent and magnitude of these socio-economic impacts led to a review of the project by the Director of Audit (2004) and the Public Accounts Committee (LegCo, 2004a). In essence, the delay and cost increase were attributed to problems in constructing the sewage tunnel system: difficult ground conditions, major breakdowns in machinery and the forfeiture of two of the original tunnel contracts.

The Environment, Transport and Works Bureau (ETWB) also conducted a review of the project implementation issues of the HATS Stage I project with the assistance of relevant Government departments, including the Drainage Services Department (DSD) and the GEO. The findings and recommendations were reported to the LegCo Panel on Planning, Lands and Works in June 2004 (LegCo, 2004b). As a result of this review, a series of specific recommendations and associated implementation plans were developed, which involved improvements to policy, procedures and technical guidance documents and these were carried out by the ETWB, DSD and GEO. The specific recommendations relating to the geotechnical aspects of government tunnel projects were :

(a) in planning ground investigation for future tunnel projects, consideration should be given to the use of new technology.
(b) where consultants are selected for major tunnel projects, more weight should be placed on their geological and tunnelling knowledge and experience and their familiarity with the local ground conditions;
(c) GEO (in particular, the Hong Kong Geological Survey housed within GEO) should be consulted on planning and interpretation of ground investigations, ground and hydrogeological modelling, identification of problematic areas, and verification of geological features during construction;
(d) designers should be required to carry out detailed assessments at the design stage to estimate the magnitude and extent of ground settlement due to proposed tunnel works;
(e) the assessment of the magnitude and extent of ground settlement due to proposed tunnel works should be ‘checked’ and agreed by the GEO before tendering of the tunnel works;
(f) mandatory requirements on documentation of the geotechnical design should be imposed to facilitate verification of design assumptions through monitoring and, if necessary, modification of design or specifications during construction;
(g) all geological information revealed from future tunnel projects should be passed to GEO for use in updating published geological maps and preparing new maps.

Implementation of the above recommendations has been achieved through development and issue of a new policy document, viz. ETWB TC(W) No. 15/2005 on “Geotechnical Control for Tunnel Works”, and the associated GEO Technical Guidance Notes (TGN) on site investigation (GEO, 2005a) and geotechnical risk management for tunnel works (GEO, 2005b). The Project Administrative Handbook has also been updated to incorporate revised government procedures for public works projects involving tunnel works.

TUNNEL FAILURES

As part of the preparation work for the development of new policy, procedures and technical guidance documents, a review of the literature has been conducted. A list of the cases reviewed that involve tunnel failures with ground collapse or large groundwater inflow and major public consequences reported is given in Annex A. Based solely on the technical literature, it is difficult to assess fully the extent of the problems associated with tunnel works; many cases are likely to be unreported in the literature for contractual and legal reasons. Even for the cases where there are public reports of the major consequences, details, in particular the causes, are often not available. Concern from the insurance industry, however, indicates that worldwide, all too frequently large insurance claims have been made. A few investigations of forensic quality have been conducted, and the reports published give useful insights.

The Health and Safety Executive (HSE, 1996), in its review of safety of New Austrian Tunnelling Method (NATM) tunnels, presents data on 39 significant NATM incidents worldwide. It indicates that many projects have experienced multiple collapses. The incidents were not confined to countries with little prior experience of NATM. The consequences of tunnel collapse for the public, infrastructure and the built environment are consistently high in urban areas. HSE has classified the causes into five broad categories: unpredicted geological causes, planning and specification mistakes, calculation or numerical mistakes, construction mistakes, and management and control mistakes.

The report of the Committee of Inquiry into the Nicoll Highway cut and cover tunnel collapse in Singapore on 20 April 2004 (MoM, 2005) has identified several technical and administrative errors, which contributed to the collapse. It indicates that there were failures to demonstrate the necessary level of care, and serious human errors. It points to the need for robust design, risk management, design review and independent checking, purposeful back analysis, an effective instrumentation, monitoring and interpretation regime, an effective system of management of uncertainties and quality during construction, corporate competencies and safety management. It also recognizes that the safety of temporary works is as important as that of permanent works and should be designed according to established codes and checked by competent persons. The Building Control Authority of Singapore has subsequently published enhanced guidelines on design and construction control of deep excavations (BCA, 2005).

DEVELOPMENT OF GEOTECHNICAL RISK MANAGEMENT

Systematic identification and management of construction risks is becoming increasingly common practice in many areas of the construction industry and, in some jurisdictions, has become a regulatory requirement as part of construction site safety requirements, e.g. the United Kingdom's Construction (Design and Management) Regulations. Locally, the "Tang Report" (CIRC, 2001) on Construct for Excellence also points towards the need to adopt similar practices in Hong Kong's construction industry.

With regard to tunnel works, review of the technical literature indicates that many problems, particularly those that result in a direct impact on the public, are due to inadequacies in the management of geotechnical risks. These geotechnical risks are either, as Barton (2004) has suggested, often a result of an unexpected combination of factors or the unexpected magnitude of a single factor. Such incidents have resulted in significant losses

to clients, contractors, consultants and the insurance companies. The insurers' perception was that the tunnelling industry had an inconsistent approach to risk management, to the extent that it threatened the provision of insurance coverage to the tunnelling industry as a whole (Mellors et al, 2004). As a way forward, the insurers worked with the tunnelling industry to develop a "Joint Code of Practice for Risk Management of Tunnel Works", which was published by the Association of British Insurers and the British Tunnelling Society in 2003. This became the forerunner of the international code entitled "A Code of Practice for Risk Management of Tunnel Works" published by the International Tunnelling Insurance Group (ITIG, 2005) in late 2005.

The Code of Practice emphasizes the importance of risk management in all stages of a project, i.e. project development, construction and contract procurement, design, and construction stages. It promotes identification of hazards and the associated risks during all four stages, preparation of risk registers, cascading the registers throughout the project to ensure that all parties are aware of the previously identified hazards and risks, continuous review and updating of the registers throughout the project, and identification of a party to be responsible for managing each element of risk. It highlights the need for the project client to take proactive action and responsibility in risk management. It also requires the project client to carry out adequate ground investigation and to prepare (or have prepared on its behalf by a competent agent) "Ground Reference Conditions" for projects involving tunnel works.

GEO TECHNICAL GUIDANCE NOTES

Site Investigation

GEO Technical Guidance Note TGN24 (GEO, 2005a) was the first technical guidance document issued to address the recommendations of the HATS Stage I Project Implementation Review. It provides guidance on specific aspects of site investigation for tunnel works and supplements previous guidance given in Geoguide 2 (GEO, 1987) and Geoguide 4 (GEO, 1992). In essence, its purpose is to prompt project clients and their designers to consider carefully, at an early stage of the project, the scale and type of ground investigation (and the associated geotechnical assessment) required for a project involving tunnel works. This is to ensure that a proper early assessment is made of the level of geotechnical risks associated with the ground conditions, in order to optimize the choice of tunnel alignment and design and to minimize the risks to the programme, cost and the impact of the works on life and property.

The TGN promotes the following:

(a) for the benefit of planning, design and construction of tunnel projects, all factual geological data, particularly that revealed from tunnel faces, should be documented;
(b) a phased ground investigation approach should be adopted and careful consideration should be given to the use of new technology such as directional core drilling and new geophysical survey techniques with confirmatory boreholes;
(c) in view of the importance of planning of ground investigation and interpretation of the results, when consultants are selected for major tunnel projects, significant weight should be placed on their geological and geotechnical engineering knowledge and expertise, and on their knowledge and experience of local ground conditions.

For major tunnel projects in difficult ground, wrong interpretation of ground

investigation results by the contractor can lead to miscalculation of the risk and submission of an unrealistically low tender price. This may increase the risk to life and property if inadequate allowance is made for risk mitigation measures. It also may increase the probability of forfeiture of the contract. Therefore, timely availability of adequate ground investigation data, and ensuring that the tenderers make proper interpretation and use of this information in their tender submission, should be a key component of the overall risk mitigation strategy. The TGN recommends that interpretative geotechnical reports, in a form that will meet the needs of professional users, should be prepared using the data obtained during the various phases of the site investigation, and that these reports should update any previous relevant interpretative reports. The decision on whether to disclose all or selected interpretative geotechnical reports and/or Ground Reference Conditions to tenderers for construction of the tunnel works remains a sensitive contractual issue; doing so follows the spirit of the Code of Practice.

Project managers are advised to arrange to carry out all necessary site investigations and satisfy themselves that sufficient ground investigation information has been made available prior to the commencement of the contract and during the detailed design (see also ETWB TC(W) No. 17/2004). The TGN recognizes that in some cases access constraints limit the extent of pre-contract ground investigation and that, even with the best investigation, much in the way of ground information inevitably remains unknown until excavation of the tunnel works is underway. Therefore, where there remains significant uncertainty with the ground conditions prior to tender, the client (or the contractor) may need to undertake additional ground investigation during the construction stage. In addition, “probing ahead” (sufficiently far ahead of the tunnel face) during construction to identify the need for measures to mitigate potential ground hazards and risks is recommended.

Following issue of the TGN, the Geotechnical Projects Division of GEO, in providing assistance to DSD in the development of the next stage of the HATS project, has utilized the results of geophysical surveys (surface to borehole seismic and marine seismic reflection surveys) to assess alternative alignments for the tunnels under the project. In addition, it has completed a pre-construction ground investigation that includes the use of directional core drilling, with water absorption tests carried out at regular intervals, for the proposed drainage tunnels for stabilizing the hillside above Po Shan Road in a Landslip Preventive Measures project managed by the Planning Division. The Public Works Central Laboratory of the Standards and Testing Division have also carried out Cerchar Abrasiveness Index tests to provide data for the estimation of the TBM cutter wear rate for the latter project.

Geotechnical Risk Management

The second technical guidance document issued by GEO was TGN25 (GEO, 2005b). TGN25 refers to, and has incorporated the essential elements of, the International Code of Practice for Risk Management of Tunnel Works. It also refers to ETWB TC(W) No. 6/2005 on Systematic Risk Management, which applies to public works projects with cost estimates exceeding HK$200M; geotechnical risk management for tunnel works being an integral component of the systematic risk management for the overall project. A comparison of the risk management stages under ETWB TC(W) No. 6/2005, TGN 25 and effectively what is required under the Buildings Ordinance for private submissions is given in Figure 2.

From the Government’s perspective as a regulator, a key aim of geotechnical risk management in tunnel works is to ensure that the works do not affect adversely public life or property. To help achieve this, it becomes the responsibility of the project client, with due

advice from the project manager and an experienced geotechnical professional, to ensure that adequate resources are provided and an adequate system is in place for the management of geotechnical risks in the construction of such works. The implementation details should take into account the level of risk to life and property. The TGN advises that insurance of the risk does not remove the need, or reduce the responsibility of the client, to ensure safety is properly managed.

Some geotechnical risks are design or construction method-related. Consequently, evaluation of the tunnel alignment, layout design and construction methods is an important step in managing such risks, in that exclusion of particular layout designs or construction methods could avoid specific risks. In some cases, the contract may need to exclude designs or construction methods that are not acceptable based on risk management considerations.

GEOTECHNICAL CONTROL OF TUNNEL WORKS

In Hong Kong, private projects involving tunnel works are subject to geotechnical control under the Buildings Ordinance, Cap. 123, in which "building" is defined to include "cavern or any underground space adapted or constructed for occupation or use for any purpose including its associated access tunnels and access shafts".

GEO's role is to advise the Building Authority on the geotechnical aspects of the projects, including railway projects, constructed by the public authorities. The GEO District Divisions carry out technical audits and provide geotechnical advice to the Buildings Department accordingly. The scope of the audits covers the geotechnical aspects of designs, assessments of the effects of proposed tunnel works on adjacent land, services, buildings and other structures/facilities, as well as the standard of site supervision and implementation of geotechnical risk management.

For the railway projects, an Instruments of Exemption can be granted by the Building Authority subject to the applicant meeting certain conditions, including demonstrating the adequacy of the project's geotechnical design standards, and site supervision and geotechnical risk management provisions. The exemption is on certain administrative procedures and covers only some of the elements of the works under the project. The geotechnical control process for such projects is the same as that adopted for other private projects, using the same technical standards.

As part of their geotechnical control functions, the GEO District Divisions also provide advice to the Lands Department on the geotechnical conditions to be included in land documents where government land is to be sold, licensed or otherwise allocated, including land through which utility tunnels such as cable tunnels pass.

In a separate regulatory function to protect public safety, the GEO Mines Division, headed by the Superintendent of Mines (CGE/Mines) on behalf of the Commissioner of Mines (Director of Civil Engineering and Development), controls the manufacture, storage, transportation and use of explosives (including issue of blasting licences and blasting permits for rock blasting) under the Dangerous Goods Ordinance (DGO) Cap. 295. Projects involving tunnel works using the drill and blast method of construction are subject to regulatory control under the DGO.

For government projects, the ETWB issued ETWB TC(W) No. 15/2005 on "Geotechnical Control for Tunnel Works" in September 2005. Under this circular, the GEO audits the geotechnical aspects of design submissions on government tunnel works and the adequacy of the project's site supervision and geotechnical risk management provisions[3].

[3] The new policy does not cover immersed tube tunnels, box culverts, submarine outfalls and underground pipes for drains, sewers and water mains.

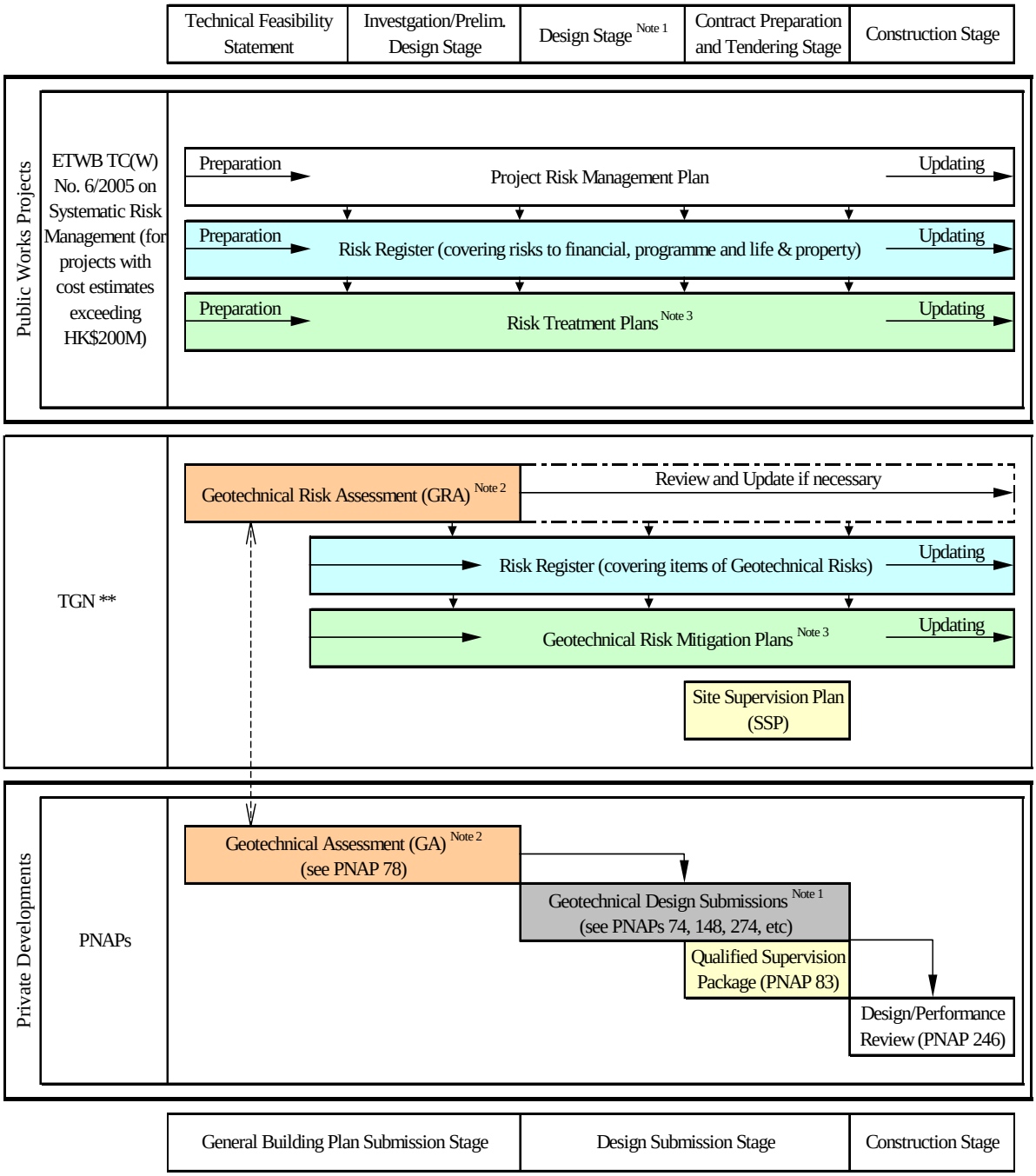

Figure 2. Processes and Risk Management Deliverables for Tunnel Works Projects

Notes:

1. *For Design & Build contracts, the Design Stage comes after Contract Preparation and Tendering Stage.*
2. *The results of the GRA in the project planning stage should be documented in the Preliminary Geotechnical Assessment (PGA) or GA. The GRA should be reviewed and where necessary updated in the Design Stage (when the tunnel alignment has been decided and the design of the works has been completed), and also in the Construction Stage (when the construction methods, equipment and procedures have been selected). The Geotechnical Design Submissions should contain the results of the updated GRA and Geotechnical Risk Mitigation Plans (GRMP).*
3. *The GRMP referred to in this TGN are detailed plans referred to in the Risk Treatment Plans for major public works projects (see ETWB TC(W) No. 6/2005).*

The GEO will also conduct site audits on the implementation aspects. The scope and focus of these audits are on risk to public life and property. This parallels the GEO's existing role to provide a geotechnical advisory service to the Buildings Department on private sector tunnel works controlled under the Buildings Ordinance.

In addition to GEO undertaking technical audits, project offices are required to consult the GEO at various project stages before tender under the ETWB circular. During the consultation and auditing processes, the GEO provides advice directly to the relevant government office managing the project. Table 1 shows the key requirements for submission to GEO (for consultation and for audit) for government projects.

In formulating its policy to enhance the standard of risk management for tunnel works, the Government recognizes the need for utilizing its in-house geotechnical expertise to provide advice to its project offices, independent of the parties directly involved in implementing and managing the project/contracts, and the need for advice throughout the project.

GEO Panel on Tunnels and GEO Services

To coordinate the GEO's consultation and auditing services for projects involving tunnel works, to ensure consistency, and to oversee the development of new technical guidance and procedures to support such services, GEO has set up a Panel on Tunnels. A Deputy Head chairs the Panel, which comprise members from various GEO Divisions. This brings together in-house blasting, geological, ground investigation and testing, analytical modelling, risk assessment and cavern and tunnel engineering expertise, as well as district geotechnical knowledge and experience. With the support of the Panel, the GEO District Divisions provide a 'one-stop shop' for GEO's services to government offices and private sector project clients, and their agents. In addition to ensuring public safety, the GEO's coordinated and integrated actions under the Panel help enable timely and cost-effective implementation of the projects.

Amongst GEO's services to government offices, other than geotechnical advice and technical audits by the District Divisions, the Hong Kong Geological Survey (HKGS) of Planning Division plays the important role of providing geological information and general geological advice, in particular on identification and verification of geological features. A good understanding of geology is essential for managing the ground risks inherent in tunnel works, and the ETWB circular requires project offices to consult the HKGS at an early stage of the project. The Geotechnical Projects Division provides advice on the standard of ground investigation and laboratory testing works. It procures such works for government offices through its term contracts, bringing in new techniques where appropriate. Where required, it can also provide a range of geotechnical advisory services including preliminary feasibility studies, engineering geological and risk assessments, and geotechnical design assistance for tunnel works. The Standards and Testing Division provides a wide range of laboratory testing services, including rock testing to provide data for the design of hard rock tunnels. It also assesses and can advise on the suitability of geotechnical engineering software. Where the drill and blast method of construction is to be adopted, the Mines Division advises on the requirements for delivery and site storage of explosives, and regulates their use to protect public safety and property.

Table 1. Summary of Geotechnical Submissions to the GEO District Division Required under ETWB TC(W) No. 15/2005

Project Stage (relevant para.)	**Consultation within GEO**	**Documents Required to be Submitted GEO**		
		Consultation	**Audit**	**Record**
Feasibility study (para. 12)	HKGS of the Planning Division for general geological advice	• engineering feasibility studies (EFS) • preliminary geotechnical appraisal (PGA) to be included in the Technical Feasibility Statement		• final EFS and PGA • initial risk assessment (including blasting risk) with respect to public life and property
Consultation during preparation of consultancy briefs (para. 13)		• consultancy brief (for design and construction)		• final consultancy brief
Engineer's designs - detailed design (paras 14 & 15)	Mines (see below)	• minimum site supervision personnel requirements (E's staff only)	• geotechnical design submission (on the permanent tunnel works and the associated temporary works)	• updated risk assessment where appropriate • final geotechnical design submission • final site supervision personnel requirements
Contract preparation and tendering (paras 16, 17, 18 & 19)	Mines (if blasting works are involved) for site storage, delivery and use of explosives	• Particular Specification (PS)/ Employer's Requirements (ER) clauses on tunnel works and the likely associated temporary works • minimum site supervision personnel requirements (Contractor's staff, and for a D&B contract the SO's staff) • whether GEO has major geotechnical concerns, before inviting tender		• final PS or ER clauses • final site supervision personnel requirements
Construction[1] (paras 20, 21, 22 & 23)			• Contractor's construction methods/procedures (method statements) • for Contractor's designs, geotechnical design submission and risk assessment by the Contractor (Appendix A) after detailed checking and certification required under the contract • for Engineer's designs, updated risk assessment and design	• Contract programme • details of site supervision arrangement and management structure (including details of key site supervision personnel) for the E/A/SO and the Contractor • final geotechnical design submission and risk assessment
Prior to consultancy /contract completion (paras 24 and 26)				• geotechnical performance review and monitoring/survey data • geological and hydrogeological data[2] • as-built survey plans • maintenance manuals • for sections of tunnel works exempted from GEO auditing, geotechnical design submission and risk assessment

Notes:

1. *Submission during the construction stage should be made prior to the commencement of construction of the related section of the tunnel works.*
2. *The PD shall submit the factual geological and hydrogeological data (such as field data collected from excavation faces) and monitoring data to the Public Section of the Geotechnical Information Unit, CEDD (para. 26), and confirm to the HKGS and the relevant District Division that this has been done prior to completion of the contract.*

CONCLUSIONS

The need to construct tunnels and other underground facilities in Hong Kong is an inevitable result of continuing urban development, combined with a short supply of suitable land and the limitations imposed by Hong Kong's landscape.

The HKSAR Government has a stake in tunnel works as a major client, a land owner and as a regulator. The problems encountered with the HATS Stage I project provided the catalyst required for the Government to re-examine the scope of geotechnical control of tunnel works, and bring its standards and procedures in line with international best practice. As a result, it has issued a new policy document relating to the geotechnical control of government tunnel works. Through this and the issue of associated technical guidance documents, it has provided guidance that facilitates safe and cost effective implementation of private and public sector tunnel works by ensuring that project clients build in the need to assess and manage geotechnical risks at all stages of a project. GEO will monitor the implementation of these new standards and procedures and provide appropriate advice to on-going and new projects involving tunnel works, with a view to adding value to such projects, partnering with the project clients, the profession and other key stakeholders.

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Annex A. List of Tunnel Failure Cases

Year	Project /Location	Reference
1964	Green Park, London, UK	Clay & Takacs (1997)
1965	Victoria Line Underground, London, UK	Clay & Takacs (1997)
1966	Southend-on-Sea Sewage Tunnel, UK	Clay & Takacs (1997)
1970	Orange-Fish Tunnel, South Africa	Clay & Takacs (1997)
1985	Gibei Railway Tunnel, Romania	Clay & Takacs (1997)
1989	Moda Collector Tunnel, Istanbul, Turkey	Clay & Takacs (1997)
1977	MTR Modified Initial System, HK	Clay & Takacs (1997)
1983	MTR Island, Hennessy Road, HK	GEO Records and SCMP (1983)
1983	MTR Island, Shing On Street, HK	MP (1983a)
1983	MTR Island, Shau Kei Wan, HK	MP (1983b)

Year	Project /Location	Reference
1991-98	Athens Metro, Greece	Kavvadas et al (1996)
1991-04	Hsuehshan Tunnel, Taiwan	Li (2006)
1993-95	Motorway Tunnels, Austria	Clay & Takacs (1997)
1994	Munich Underground, Germany	Ground Engng. (1994), ICE (1996), Boos et al (2004)
1994	Heathrow Express, London, UK	HSE (1996), ICE (1998b &c), ICE (1999), Ground Engng. (2000), HSE (2000), Boos et al (2004)
1998	Docklands Light Rail, London, UK	ICE (1998a), ICE (2004a)
1999	Sewage Tunnel, Hull, UK	Boos et al (2004)
2000	Taegu Metro, South Korea	Boos et al (2004)
2003	Channel Tunnel Rail Link. UK	ICE (2003)
2003	'Météor' Metro Tunnel, Paris, France	Dubois & Rat (2003)
2003	Shanghai Metro, China	Boos et al (2004)
2004	MRT Circle Line, Singapore	ICE (2004b &c), European Found. (2004), European Found (2005a), ICE (2005), MoM (2005)
2004	Kaoshiung Metro Tunnel, Taiwan	Taiwan Info (2004)
2005	Barcelona Metro, Spain	European Found. (2005b)
2005	Lane Cove Tunnel, Australia	SMH (2005), Ground Engng. (2006)
2005	Lausanne M2 Metro, Switzerland	Tunnels and Tunnelling (2005)

MANAGEMENT OF DESIGN PROCESS DURING ROCK TUNNEL CONSTRUCTION

Eric S. F. Li [1] and Steve S. M. Wong [2]

Abstract

Notwithstanding the great effort being spent on ground investigation, geological conditions as revealed during construction often deviate from those assumed in designs. This is particularly relevant for tunnelling works in soft and hard rock where detailed geological and/or hydrogeological information may not be available practically or economically. Many tunnelling contracts in Hong Kong require contractors to be responsible for the design of a temporary support system before a permanent lining is constructed. To minimise risks, it is of benefit to contractor to establish an experienced design team to derive practical and economical solutions, and to provide specialist advice to overcome geological difficulties as they arise. Experience shows that a close interface between design and construction teams is critical to the successful completion of tunnelling works as any delay will hamper the linear progression of the excavation.

The paper describes the authors' experience in developing a cost effective approach to managing designs associated with hard rock tunneling construction. The key considerations in the design of temporary support and permanent linings will be examined. The paper will also highlight with practical examples on how design changes can be implemented quickly when difficult ground conditions or non-standard conditions are encountered.

INTRODUCTION

The importance of ground investigation for a successful completion of construction works has been discussed by many authors, including ICE (1993) and Henderson & Pickles (2004). However, notwithstanding the great effort being spent on it, geological conditions as revealed during construction are often different from those assumed in design. This is particularly relevant for tunnelling works in soft and hard rock where detailed geological and/or hydrogeological information may not be available practically or economically in advance.

Many tunnelling contracts in Hong Kong require contractors to be responsible for the design of a temporary support system before a permanent lining is constructed. To minimise risks, it is of benefit to the Contractor to establish an experienced design team to derive practical and economical solutions, and to provide specialist advice to overcome geological difficulties as they arise.

This paper will discuss how the design process can be effectively managed during construction, making reference to a project involving rock tunnel construction in Hong Kong. The current designer-contractor interactions in the design-construction process will be

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illustrated with worked examples.

DESIGN AND CONSTRUCTION RESPONSIBILITIES IN TUNNELLING PROJECTS

Tunnel designers often prepare their designs based on limited site investigation data and generalized and simplified empirical models. Constrained by such limitations, conservative design parameters and assumptions are often adopted, sometimes resulting in uneconomical and impractical designs of the permanent works, and some refinements and optimizations to such designs may bring significant benefits to both contractors and clients. In addition, many tunnelling contracts in Hong Kong often place the responsibilities for the design of a temporary support system, before a permanent lining is constructed, to contractors. Against these backgrounds, contractors often have to establish a design team to manage the various design issues during construction.

In tunnelling projects, the primary objective of the Contractors' designers is to derive "standardized" temporary excavation support systems. Such systems should be cost-effective and practical to build, taking into account the confined working environment in a tunnel and linear progression of excavation. This then brings to a question "What is the standard system?". An all-encompassing generalized "standard" system will undoubtedly be too general and may not be economical. If too many "standardized" solutions are developed to address envisaged different conditions, the systems may become unnecessarily complex to follow by construction team. Hence, a balance must be struck. Another question then arises "What is the most cost and time effective approach to deal with the non-standards?" Both of these questions cannot be answered easily. It crucially relies on the experience and close coordination of the contractor's construction team and designer. The decisions can have profound effects on the cost and programme of works if not managed properly.

DESIGN METHODOLOGY FOR TUNNEL TEMPORARY SUPPORT

The interaction of design inputs to a tunnel construction, particularly for a drill-and-blast method, is illustrated in Figure 1. At the outset, a generalized standard support system is derived using available data and simplified empirical models. In Hong Kong, tunnel excavation support classification often follows that proposed by Barton et al (1974) and Grimstad & Barton (1993), which divides the support systems into 9 classes according to the rock mass quality (NGI Q-value), tunnel configuration (size or span) and excavation support ratio (ESR). The classification system is essentially a 2-dimensional model with limited considerations on 3-dimensional effect and construction method.

During the construction stage, a review of the site and geological conditions will then be carried out by a tunnelling engineer or geologist after each cycle of excavation. For typical drill-and-blast construction, the review will be carried out after each cycle of blasting and mucking out. The tunnel engineer will determine the most appropriate temporary excavation support according to the prescribed "standardized" system designed by his initial review. This is a process very much relied on the experience of the tunnelling engineer. He exercises an objective judgment on the application of support sequence and type of temporary support. It is important that the designer is open-minded and should not merely rely on the standardized details. On the other hand, he should examine whether adverse geological or hydrogeological features exist which merit a special review and the use of a "non-standardized" support system.

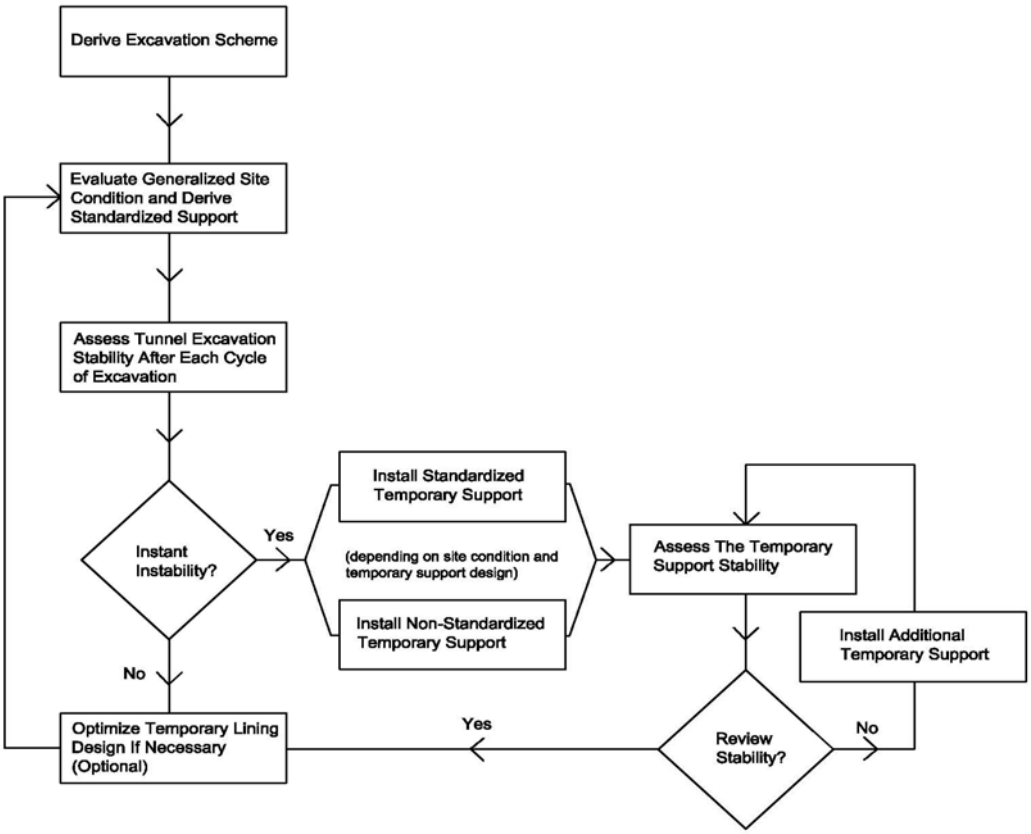

Figure 1. Tunnel Design Input to Hard Rock Tunnel Construction

As more information becomes available, the tunnelling engineer should review the entire approach to implementing the temporary support to excavation and determine whether improvements/refinements can be made.

Some of the design inputs commonly required for a drill-and-blast tunnel excavation project are discussed in the following sections. The authors' experience in managing such design inputs is described.

DISCUSSION CASE 1 - Temporary Support Designs to Non-Standardized Sections

The 2 tunnels referred to in the present discussion is of horse-shoed configuration, each of them are designed for a 3 lane vehicular traffic. The tunnel is 11m high and has a clear span of 17m generally but widened to 19 m near the entrances (see Figure 2). The length of each tunnel is approximately 900m. The tunnel excavation was carried out by means of either mechanical breaking or a drill-and-blast method. It was specified in the Contract that temporary support was to be designed by the Contractor. In addition, the Contract specified different types of permanent lining of different lining thickness and amount of steel reinforcement for different classes of temporary support conditions, which themselves were determined according to the rock mass quality Q-values (Hoek et al, 1995). For the given tunnel configuration, correlation between support classes and Q-value was specified in the Contract and is summarized in Table 1.

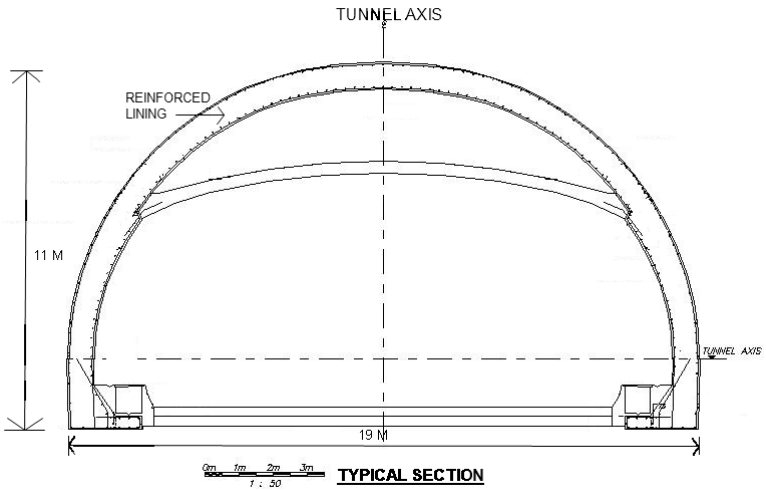

Figure 2. Typical Tunnel Cross Section

Table 1. Rock Mass and Support Class Classification for a 19 m Span Running Tunnel

Barton Support Class	Rock Mass Quality Q-value Q_{rock}
1 & 2	>5.4
3	2.2 – 5.4
4	0.9 – 2.2
5	0.34 – 0.9
6 & 7	<0.34
8 & 9	<0.035

Non-Standardized Sections – Portal & Transition Zones

According to a preliminary assessment, the tunnel excavation from the South Portal was envisaged to progress through a 20 m long "Initial Portal Zone" and then followed by a "Transition Zone" before reaching "Hard Rock Zone". It was anticipated that a mixed ground of soil and rock would be encountered in the Initial Portal Zone and very substantial temporary support would be required. The temporary support for the Initial Portal Zone was designed by an in-house team of the Contractor and comprised closely-spaced lattice girders, roof spiling and grouting, pattern bolting and shotcreting.

According to the preliminary estimations, the Transition Zone commences from where Grade IV/III decomposed rock was encountered fully over the crown of the tunnel. Based on available ground investigation data and the rock actually exposed at initial excavation, it was anticipated that the Transition Zone would extend approximately 60m to 90m from the portal before entering the "Hard Rock Zone" where a minimum 6m of hard rock cover to the crown was available. By then, the "standardized" temporary support design for hard rock excavation could be followed (see Figure 3).

The temporary support of the Transition Zone was originally designed based on soft ground assumptions. In order to provide an optimized design during construction, the exposed excavation faces of the initial 20 m from the Portal Zone were reviewed and considered in the prediction of rock conditions in the Transition Zones. It was noted that the tunnelling conditions were more favourable than originally envisaged and the rock quality

improved significantly soon after the initial drive. Grade III rock was encountered covering

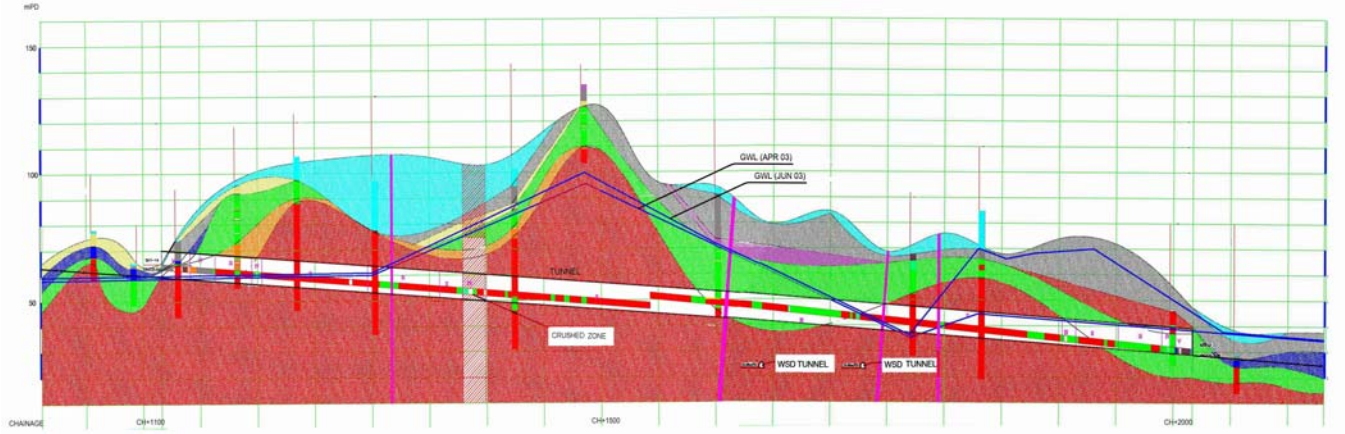

Figure 3. Geological Long Sections of Tunnel

the complete tunnel section at approximately 20 m from the portal instead of the originally assumed mixed ground and that the rockhead was at around the spring lines on both sides of the tunnel walls. No groundwater inflow was observed. Taking into account the observations and exercising engineering judgment, an alternative temporary design was derived, comprising a combination of a reinforced shotcrete with limited amount of lattice girders as the temporary lining support. This had achieved significant savings by omitting the use of lattice girders, grouting, spilling and pattern bolting.

Review of the convergence monitoring within the tunnel and surrounding ground movement instrumentation was continuously carried out by the authors. During the design review, the following tasks had been carried out to determine the most appropriate system of temporary support design:

(a) classifying the rock mass quality;
(b) determining Q-value and excavation support ratio (ESR), and making modification to temporary support;
(c) estimating support pressure;
(d) evaluating temporary tunnel support requirements;
(e) estimating allowable maximum unsupported span to expedite excavation cycle;
(f) estimating deformation in relation to Q-values; and
(g) assessing rock mass strength using the Hoek-Brown failure criterion

In addition to the above tasks, the following aspects, which were not known in detail prior to the construction, were also reviewed and adjustments to Q-values were made where appropriate based on the experience of the authors.

Blasting Vibration

The maximum allowable peak particle velocity (PPV) may be adjusted subject to the result of monitoring and tunnelling conditions exposed during excavation. This could result in higher charge rate for blasting being possible and achieving a higher production efficiency.

Stand-up Time

The stand-up time has direct impact on the selection of support systems and the time for their construction. Although some localized highly weathered bands were noted during excavation at the site, the tunnel was found to be capable to self-standing for a short period of time prior to the application of any support measures. It was therefore decided that the excavation length should be maximized with the temporary support to be completed within

24 hours of the excavation.

Shotcrete for Tunnel Crown and Wall Support

The shotcrete layer was specified to have a minimum compressive strength of 25 MPa at 28 days.

Water Table

Water table was assumed to be below the tunnel spring line based on the review of the initial drive, which was further verified during excavation.

Monitoring of tunnel convergence and ground deformation is an important element to tunnel construction. The measurements are valuable for validating the design as well as for making modifications/improvements in subsequent stages or in future. Convergence monitoring by means of tape extensometer and ring bolts fixed at the temporary lining was carried out.

Although favourable conditions were noted, the tunnel excavation still utilized a head and bench excavation to minimise tunnel convergence and ground deformation. Furthermore, additional support including inclined boltings or spiling were locally provided where adverse ground or loose blocks were encountered. The decision was made with close liaison between the site construction team and the designer.

Non-Standardized Sections – Crushed Zone

It was envisaged from initial geological assessment that a crushed (fault) zone might exist at around Chainage CH 1400 (Figure 3 refers). At this crushed zone or similar weak zones, a Support Category Class 8 was adopted. The use of Support Class 8 was also adopted to cater for other possible poor ground areas including the adjacent “down-dip” and “up-dip” sides of the crushed zone areas.

To optimize the design, the support class was further sub-divided into Class 8A and Class 8B. Support Class 8A would be applied in areas of tunnelling before reaching the weak zones (Down-Dip Zone) whereas Support Type Class 8B would be applied in areas after passing the weak zone (Up-Dip Zone). The extent of the down-dip and up-dip zone was originally estimated to be 9m in length on both sides considering that the crushed zone intersects the tunnel alignment at an oblique angle. This 9 m length was again adopted in the design carried out by the authors and was further reviewed during excavation.

The optimized design considered similar engineering factors as described before. Depending on the Q-values determined, plain shotcrete and lattice girders with or without pattern rock dowels were proposed as the temporary support for the Support Class 8 in the main tunnel excavation. Spiles were also proposed in the excavation to create a roof umbrella of reinforced rock mass ahead of the advancing face. Grouting was also allowed for the areas where significant water ingression was predicted, which might affect the stability of the tunnel structure.

In the down-dip sides and up-dip sides of the predicted crushed zone areas (minimum 9m in length on both sides), excavation in heading and benching was also proposed.

The design support pressure was originally carried out with conservative assumptions. The results of the improved analyses showed that 300 mm plain shotcrete with lattice girders would have adequate capacity to carry the load caused by potential ground deformation prior

to the permanent lining installation. During the excavation, the type of temporary support in the weak zones was verified from probing and the exposed rock surfaces as logged and assessed in accordance with the Q-system by an experienced engineering geologist or geotechnical engineer, and reviewed by the designer. The amount of the aforementioned support measures had been extensively reduced after careful review on the actual rock mass conditions.

Non-Standardized Sections – Cross Passages

Cross passages were required under the Contract to connect the north and south bound tunnels at predominantly 100m interval. These cross passages required an opening to be formed at the main tunnels which could become a weak point in terms of stability and deformation. In the temporary support design, the tunnel span and the excavation support ratio were adjusted accordingly.

The design carried out on the above basis resulted in heavily reinforced temporary lining being required at zones of highly fractured rock or fault zones. In such locations, a rapid and significant reduction in rock quality and an increased potential of water inflow were envisaged. It was therefore proposed that the cross passages be relocated to avoid construction in the fault zones, subject to a minimum distance between such passages being maintained for safety evacuation reason.

Some water courses exist over the alignment of the tunnels. Water seepage along the jointings or weathered zones was envisaged in view of shallow cover in many areas. However, the conditions of individual location were variable and individual assessments were therefore required. The actual ground water conditions were verified and confirmed during excavation by observation through the sign of seepage over the exposed rock faces and information from probing ahead.

DISCUSSION CASE 2 - Managing Underbreak in Tunnel Excavation

Another example of requiring close interaction between the construction team and the designer can be illustrated by how an underbreak non-conformity in tunnel excavation could be mitigated.

For tunnel excavation carried out by drill-and-blast method, the blasting pattern is commonly designed by a shot-firer after considering all the site constraints, geological features and advice from the tunnelling engineer. The blasting design aims to cut a clear hole that will be of sufficient dimension just to accommodate the permanent tunnel lining and the installation of temporary support measures. Some tolerance on the completed tunnel excavation will be allowed. For the Contract referred to in this paper, the tolerances for excavation were 100mm over-break (excavation exceeding the specified size of the tunnel) and 0 mm underbreak (excavation smaller than the specified size of the tunnel). In practice, overbreak and underbreak commonly occur in tunnel excavation and the extent varies across the different areas on a cross section. If overbreak occurs, the over-sized void is normally required to be filled by costly shotcreting or a thicker permanent lining is to be constructed. If underbreak occurs, one of the mitigating measures is to locally trim the area being under-sized. An alternative to the costly trimming will be to re-assess the structural strength of the permanent lining and to determine whether the underbreak section still has adequate structural capacity to withstand the imposed loads. If found to be inadequate, the designer can consider to provide additional reinforcement or enhance concrete grade to increase the structural capacity.

It had been reported by the construction team of the Contract that underbreaks were noted at widespread locations but mostly concentrated in the tunnel section where a type E1 lining would be constructed. This Type E1 lining was lightly reinforced with 0.15% steel ratio and had a nominal thickness of 450mm. Although the measured underbreak was generally limited to 50mm, trimming back to the required profile would be difficult and a very slow process (considering that a steel fibre reinforced shotcrete had already applied at the time). The authors therefore carried out an assessment of the impact of such underbreak sections with a reduction in the thickness at different radial locations of the lining.

The assessment followed similar approach commonly used in the design of permanent concrete lining. The ground-lining interaction was analysed using a computer programme SAP 2000. The tunnel lining was input as a chain of discrete beam elements, with the cross sectional characteristics representing the lining and the elastic modulus equivalent to that of concrete. The discretized beams were modeled to be supported by springs with stiffness equivalent to that of the rock mass and tension was not allowed. Different load combination cases acting on the tunnel lining had been studied, including:

(a) Self Weight Loads (lining itself plus the M&E loadings);
(b) Hydrostatic Load;
(c) Grouting Pressure Load;
(d) Rock Pressure Load;
(e) Rock Wedge Load;
(f) Earthquake Load; and
(g) any Unbalanced Surcharges Load

Many of the loadings considered in the assessment were similar to that used in the original design. However, with the benefit of knowing the actual geological conditions during excavation, some loading parameters were able to be refined. Some examples are discussed below.

Hydrostatic Load

As the tunnel was drained and had no structural invert for this type of lining structure, it was unlikely that hydrostatic loading would develop around the tunnel. The grouting criteria for groundwater inflows and drainage system provided behind the permanent lining had been designed to remove all groundwater that reaches the tunnel lining, thus groundwater pressures could be assumed to be zero. However, to allow for potentially localized high groundwater flows and possible local blockage in the drainage system, a nominal groundwater pressure equivalent to a 10 m head (100 kPa) had been assumed.

Rock Pressure Load

The rock loadings were calculated using the formula suggested by Grimstad & Barton (1993). The ranges of Q values for each lining type have been determined from the support chart assuming a span of 19 m and ESR of 1. Various combinations of Jn and Jr had been considered when determining the rock loading, and values of Jr = 1.5 and Jn = 12 were adopted for the Type E1 lining. These values were adopted with specific knowledge that they would be representative of the ground conditions concerned. The calculated Q-value on this basis for the Class 5 temporary support was 2.5 and the corresponding vertical and horizontal loads to the lining being approximately 110 and 50 kPa (unfactored) respectively.

Rock Wedge Load

Rock joint orientations were not known in detail at the time of preparing the original

design. Three hypothetical wedges were therefore considered where one joint was parallel to the tunnel axis, and two joints were oblique. Joint dips were adjusted to form wedges that would introduce an asymmetric loading on one side of the tunnel lining. The maximum possible wedge load was then estimated. From the preliminary analysis, this would require joint trace lengths of up to approximately 60m to form the rock wedges, which was considered unlikely. From the site tunnel excavation mapping records and site inspections, the joint persistence appeared to be less prominent and it would be unlikely that a wedge would exceed the tunnel size itself in real situations. A modification of the wedge size was therefore made.

For a lightly reinforced concrete lining, the most critical aspect of the lining design had been the flexural stress caused by bending moment. A typical bending moment diagram of the SAP 2000 analysis is presented in Figure 4 for an idealized tunnel lining. It can be seen that for an idealized lining subject to symmetrical loadings, the resulting bending moments would be small and limited to areas at roof (sagging), 2 upper corners above the spring line (hogging) and near the toes (hogging).

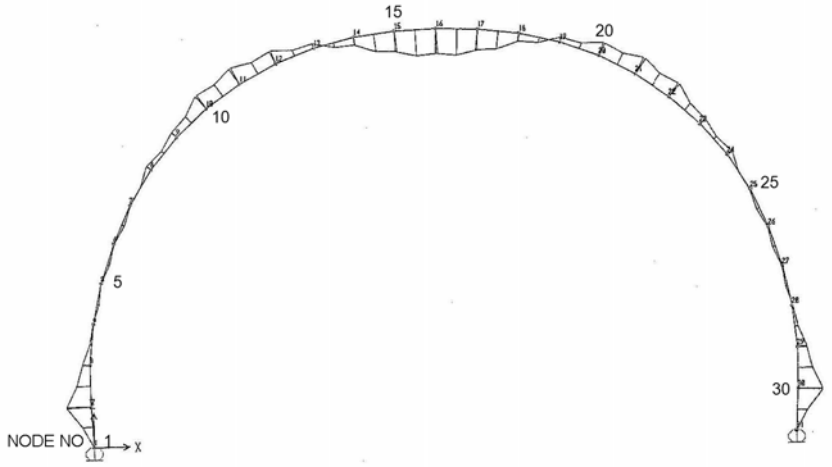

Figure 4. Typical Bending Moment Diagram of Tunnel Lining Structure

After establishing the model, the local reduction in lining thickness due to underbreak could then be input as a local reduction to the beam dimension. As the model was essentially a ground-structure interaction analysis, a local reduction in thickness would attract less bending moment but at the expense of larger deformation, depending on the location of the underbreak and the loading combination. The calculated bending moments for one particular area of lining underbreak under different load combinations are shown in Figure 5. These bending moments were then compared with the lining structural capacity. Should none of the calculated bending moments exceed the capacity envelop, the underbreak would be considered to be structurally acceptable. The assessment of the lining shear capacity was carried out in a similar manner.

The initial plan of the assessment was to carry out specific analysis at each cross section of the tunnels where underbreak was noted. This soon became impractical as there were significant numbers of matrix combinations of underbreak. A more systematic assessment was then derived, which separated a lining cross section into 7 principal segments, as determined by the areas where bending moments would be most significant (see Figure 4). If any spot of the lining was measured to have incurred underbreak, the complete segment was input into the computer program as a reduce-sized element. This brought the number of

analysis combinations down to approximately 30, which were manageable. Results templates were then developed, giving clear guidelines on underbreak combinations that would have negligible effect to the overall lining behaviour or be structurally unacceptable. For example, an underbreak of 50mm in segments A, C and D might be acceptable whereas underbreaks at segments A and B might not. Such templates were given to the construction team for ready reference and quick decisions could be made accordingly.

Having established the templates, any underbreaks not considered by the standard combinations would be assessed separately.

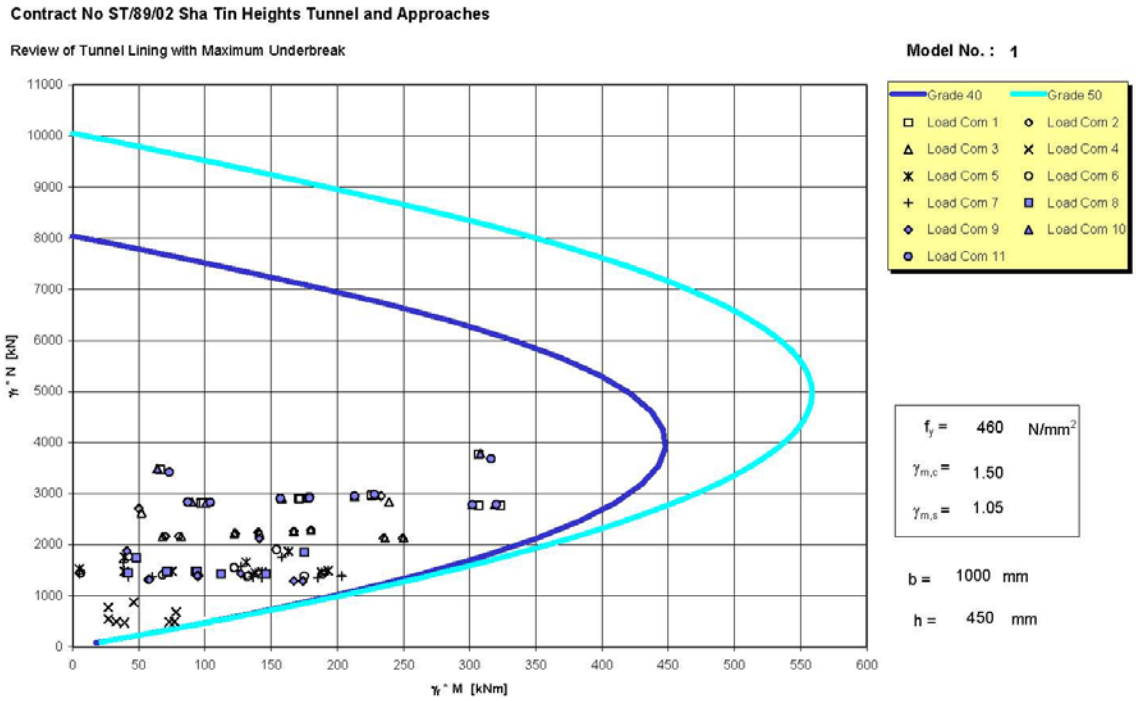

Figure 5. Typical M/N Interaction Diagram

CONCLUSIONS

In this paper, the importance of carrying timely review of the ground and geological conditions as exposed by tunnel excavation is examined. The benefit of close interaction between the construction team and the designer during the tunnel excavation has been discussed. It highlights the importance of integrating the excavation scheme with the design process. Different examples of application of this design process are also discussed.

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MONITORING DURING EXCAVATION OF ROUTE 8 EAGLE'S NEST TUNNEL

Tim Leung[1], Peter Poon[2], Bruce Cunningham[3] and Joseph Lo[4]

Abstract

Route 8 Eagle's Nest Tunnels have been constructed to provide dual three-lane carriageway tunnels. The main tunnels are 2.1 km long and of 19 metres span with emergency pedestrian cross-passages of approximate 4 metres span at every 100 metres. A secondary 400 metres long adit tunnel connects to the main tunnel overhead ventilation galleries with a fan building adjacent to Tai Po Road. The main tunnels also have ventilation buildings at the North and South portals. The tunnels are bored in the rock mass of granitic origin by drill and blast method. There are existing Water Services Department facilities, and other existing structures and slopes that are close to the tunnel alignment and within the zone of influence of the blast vibration. In this case, the instruments have been installed both within the tunnels to monitor the ground behavior, and above the tunnels to monitor the effects of tunnel construction at the ground surface and the sensitive areas. An accurate, systematic and continuous monitoring is absolutely necessary during the excavation to ensure stability of the tunnels. Extensive convergence monitoring and water flow monitoring have been carried out within the tunnels. There are over 400 instruments that require regular monitoring outside the tunnel. These include settlement points, piezometers, vibration monitoring points as well as inclinometers and multiple point extensometers. Overall managing the instruments and monitoring of the excavation posted a difficult and challenging job for excavation of the large span tunnels, with an excavation section of over 180 m^2.

INTRODUCTION

Route 8 in Hong Kong was the proposed major trunk road between Chep Lap Kok and Sha Tin, via Tsing Yi and Lai Chi Kok. The section between Chek Lap Kok and Tsing Yi has already been opened to traffic. When completed the Engle's Nest Tunnel will provide relief to the other heavily loaded links between the Northeast New Territories and urban Kowloon. The location and layout of Lai Chi Kok and Sha Tin section of Route 8 is shown in **Figure 1**. The Kowloon section includes the slip roads from Lai Chi Kok leading to the Eagle's Nest Tunnel approach road to the Eagle's Nest South Portal and the Eagle's Nest Tunnel itself which extents up to the location of the Toll Plaza. Northwards of the toll plaza is the Sha Tin Heights Tunnels.

The slip road from Lai Chi Kok is mainly on viaducts connecting to a fill embankment at Butterfly Valley. The embankments stretch towards extensive cut slope formation for the approach road leading to a two 2.1 kilometres section of tunnel. Eagle's Nest Tunnel is designed as a dual three-lane carriageway.

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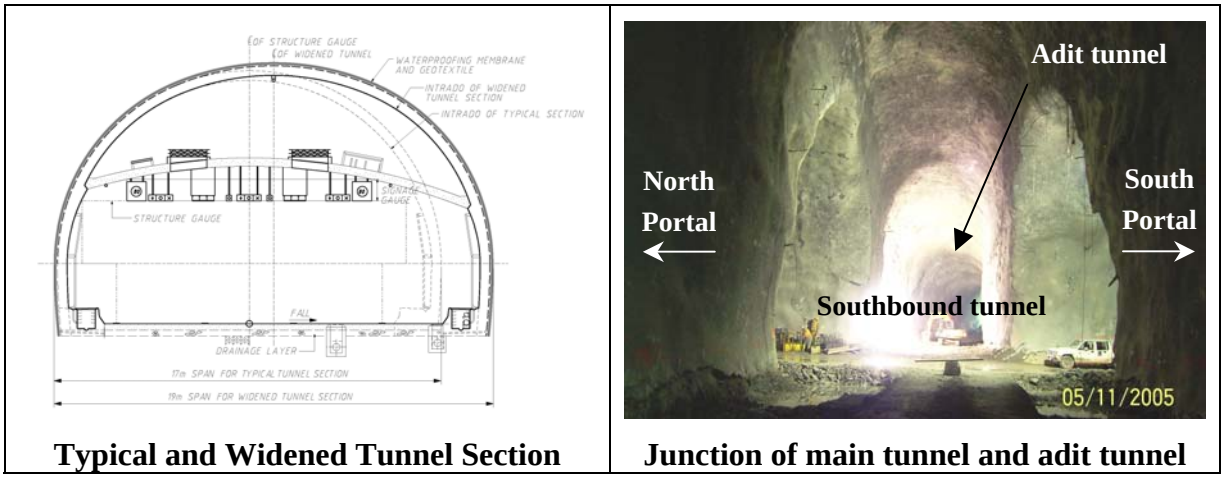
Figure 1. Route 8 Eagle's Nest Tunnel

The HK$1.83 billion contract for the Route 8 - Eagle's Nest Tunnel and Associated Works was awarded by Highways Department (HyD) of Hong Kong Special Administrative Region of the People's Republic of China (HKSAR) to Leighton-Kumagai Joint Venture (LKJV) in September 2003. Maunsell Hyder Joint Venture (MHJV) is the design and construction consultant.

The North Portal tunnel section and South Portal tunnel widened section are of 17 m and 19 m span respectively and with a secondary 400 metres long adit tunnel for ventilation (see **Figure 2**). The excavation cross-sectional areas are about 140 m^2, 180m^2 and 45m^2 for normal, widened and typical adit tunnels respectively. Two tunnels were driven curved and parallel on plan from the South Portal widened tunnel at road level of +67 mPD towards the mid-point of the tunnel at road level of +73 mPD and then towards the North Portal at road level of +67 mPD, with a minimum ground cover to the tunnel about 10m and a maximum ground cover to the tunnel about 180m.

A significant portion of the main tunnels are close to Kowloon Reservoir group which comprises the Kowloon Reservoir and the adjoining Kowloon Byewash Reservoir. The tunnel South Portal is under the Water Services Department (WSD) Cheung Sha Wan water treatment plant. The tunnel North Portal is bored above WSD Bend B tunnel. The tunnel sections are bored in rock mass of granitic origin. **Figure 3** shows the WSD facilities and other existing structures close to the tunnel alignment and within the zone of influence of the blast vibration.

This paper describes the groundwater measurement, monitoring results of ground response and vibrations from blasting during tunnel construction. The roles of specialists in monitoring of the tunnel excavation are also described.



Typical and Widened Tunnel Section | **Junction of main tunnel and adit tunnel**

Figure 2. Typical Tunnel Section and Excavation Photo of Junction of Main Tunnel and Adit Tunnel

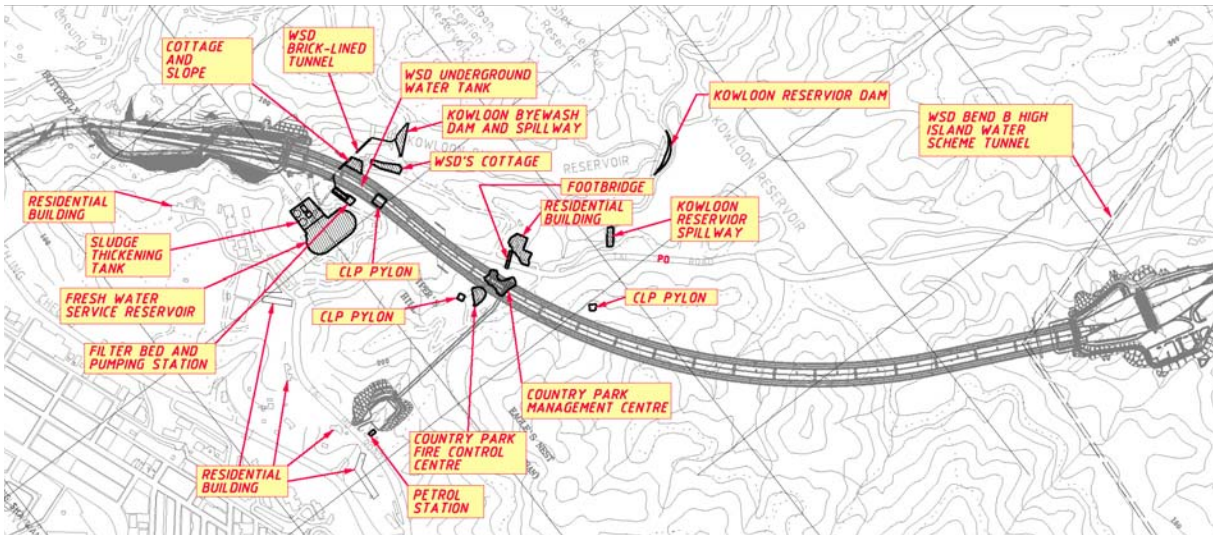


Figure 3. WSD Facilities and Other Structures Close to the Tunnel

GEOLOGICAL AND HYDROLOGEOLOGICAL CONDITIONS

The tunnels are located in an area of mainly in slightly/moderately granitic rocks with some faults through the tunnel sections. The general geological sections of the tunneling area as inferred from the results of the ground investigation are shown in **Figure 4**. The geological conditions found during excavation of the tunnels were as expected and prediction by site investigation. A particular tunnel section near the south portal is overlain by a mantle of saprolitic soils (completely/highly decomposed granite) weathered from the parent rock. The saprolite is covered for the most part by a thin moderate layer of residual soils/colluvium. The band of weathered granite was found approximately 8 m to 14 m thick and the weathering pattern closely followed the inferred rockhead profile. The residual soils and colluvium were found to be between 1 m to 15 m thick. This posted a difficult and challenging job for the excavation and initial ground support as well as monitoring for the tunnels incorporating a 19m span in an area of minimum ground cover of about 10m of saprolitic soils consisting completely decomposed granite and historic fill at chainage about Ch. 1123 of southbound tunnel (see **Figures 5a and 5b**).

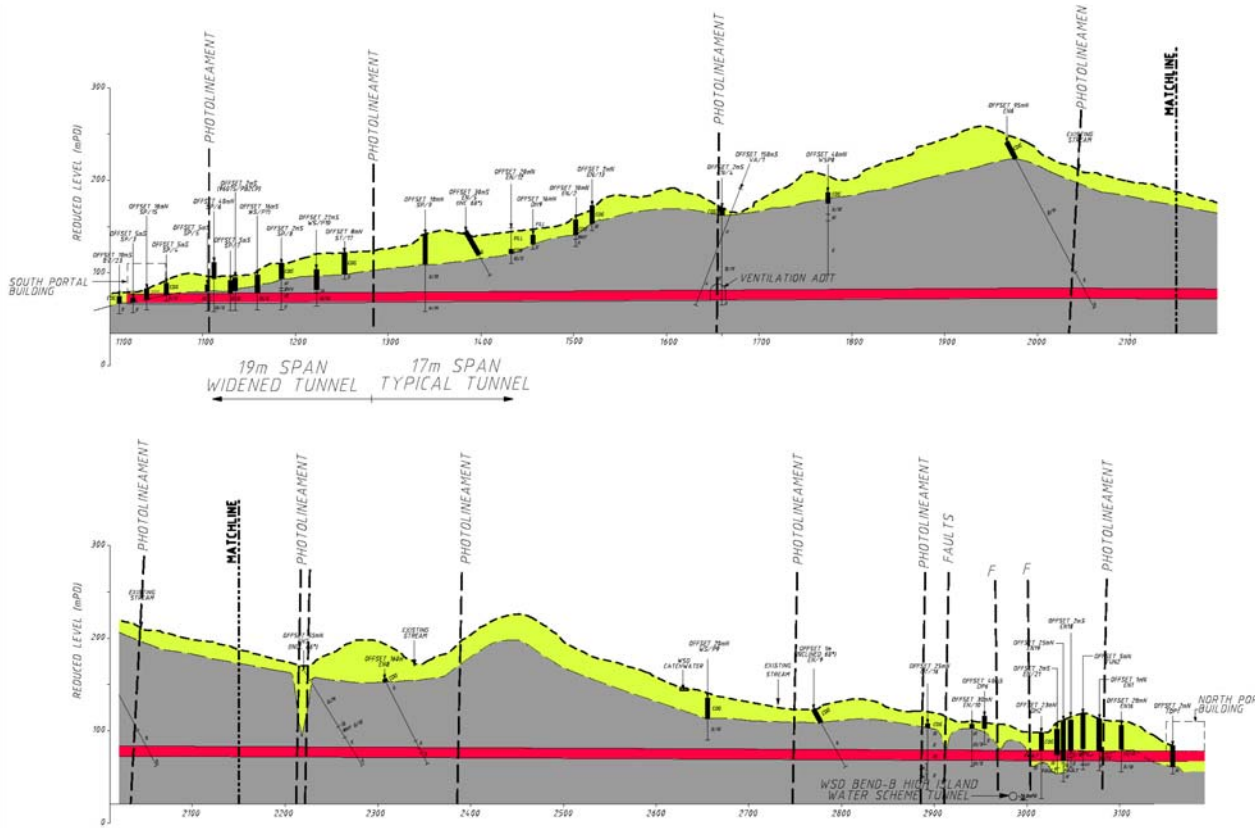


Figure 4. Longitudinal Geological Section of Main Tunnel

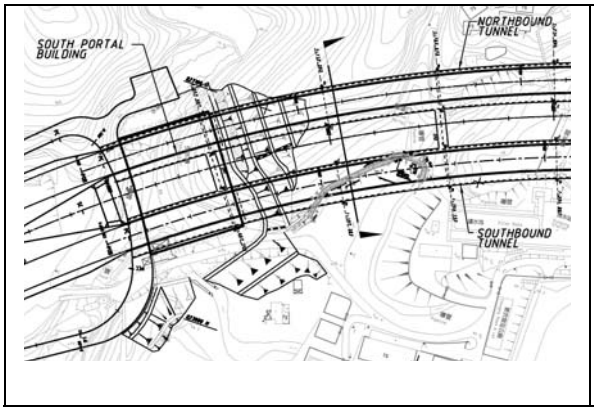


Figure 5a. 19 m Span Widened Tunnel

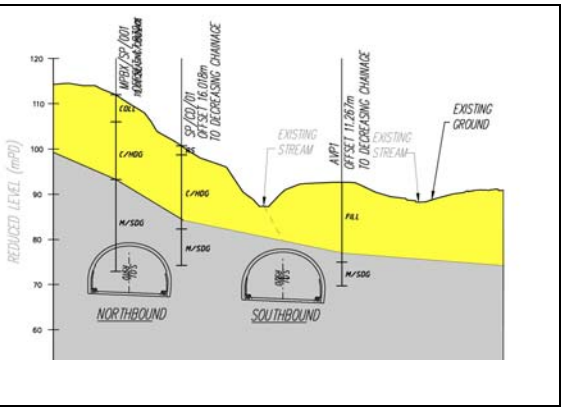


Figure 5b. Cross-section of Low Ground Cover to Widened Tunnel Section

Site Hydrology

As mentioned, a significant portion of the region serves as catchment for the existing Kowloon Reservoir group which comprises the Kowloon Reservoir and the adjoining Byewash Reservoir. It is considered that most of the reservoir capacity is fed by the intercepted run-off from the uphill tributary catchment via the existing lined catchwater and the direct catchment uphill of the basin, see **Figure 1**. The existing catchwater is one of the main feeders for the reservoir and treatment stations. The site is currently covered by dense vegetation, which would normally mean considerable infiltration rates during precipitation. However, the gradients of the existing terrain and the streamcourses would suggest high surface flow velocities, which would result in relatively larger run-off hence reduced infiltration potential.

The groundwater contours were developed using "Surfer" software and adopting the "Kriging" method in generating the plots using piezometric data. The plots of piezometric data from September 2000 and July 2001 are shown in **Figures 6a** and **6b** respectively.

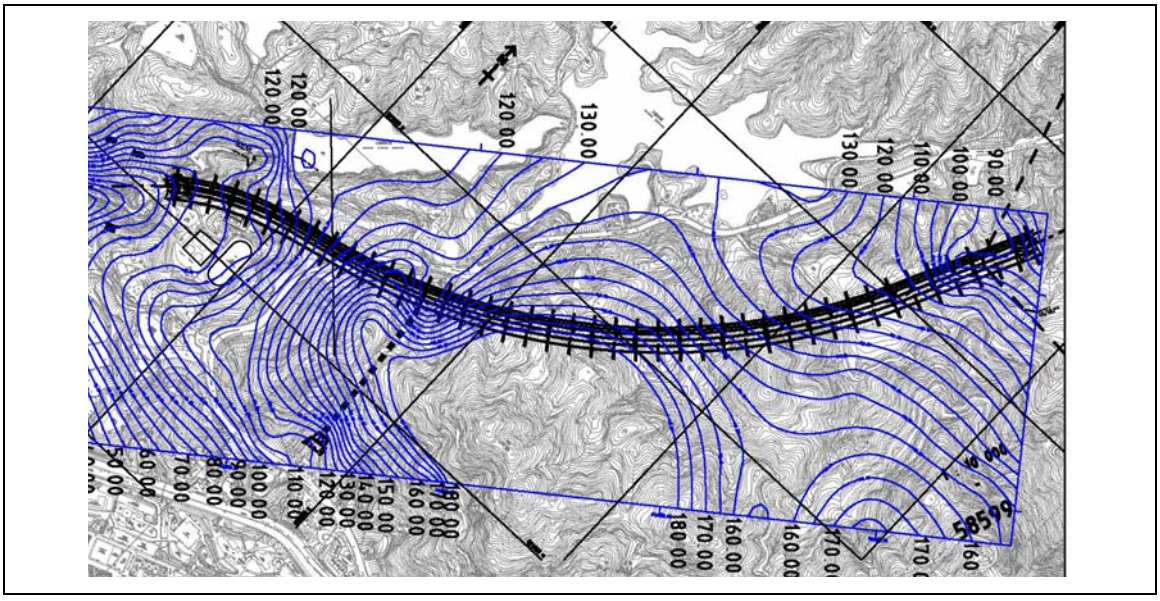


Figure 6a. Plan showing developed groundwater contours (September 2000)

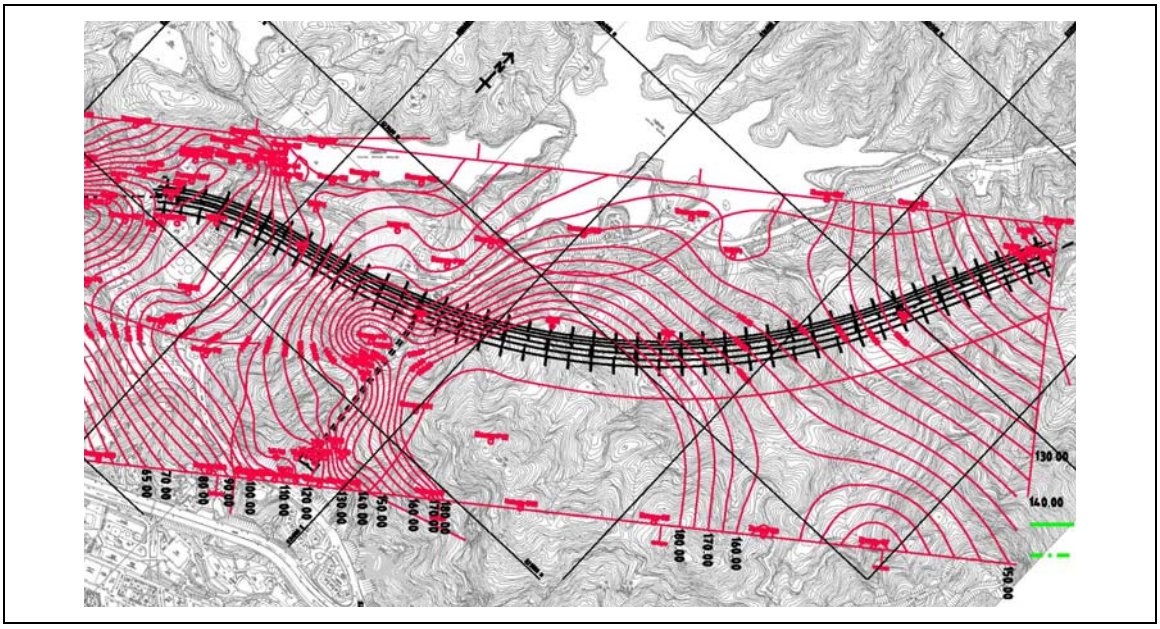


Figure 6b. Plan showing developed groundwater contours (July 2001)

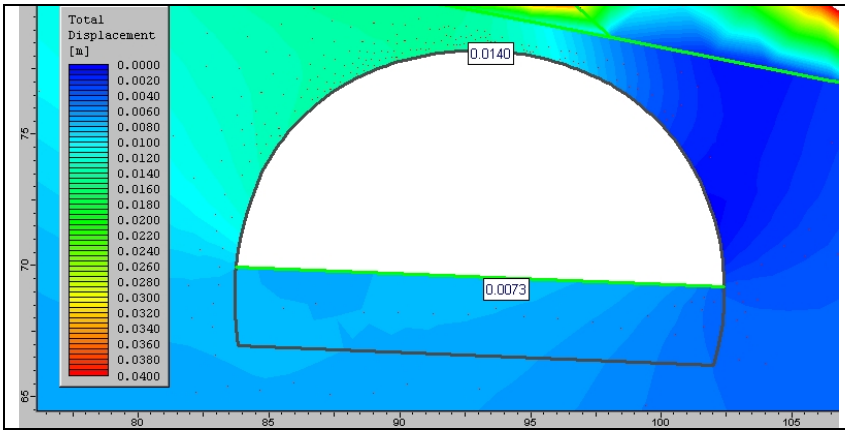
In the design stage, the groundwater contours plotted in **Figures 6a** and **6b** indicated the general gradient to be dipping northeasterly and west northerly and the groundwater mainly occurred within the rock mass. The groundwater contours developed from the piezometric data shows the groundwater gradients generally conform with the topography. Dry season water levels were considered to pose a more critical condition since the relatively lower groundwater tables had less recharge potential from the ground around the reservoir hence increasing the likelihood of water being drawn from the reservoir. The models presented were therefore based on a dry season baseline.

Tunnel Excavation

The principal objective in the design of tunnel support during excavation is to help the rock mass to support itself. Systematic temporary supports consisting of an integrated systematic rock reinforcement of fully grouted dowels and shotcrete that effectively acts together with the rock mass to limit rock deformation, and therefore bring the surrounding rock mass to a self-supporting and stable state with an adequate safety during construction until the permanent tunnel lining is installed. One of the most useful aspects of the Q-system is the support selection chart in which the tunnel support requirements are presented in a function of rock mass quality and ratio of the tunnel span to ESR (Excavation Support Ratio) that represents the safety of an excavation. Analytical methods were used to assess the stability of tunnel excavation and determine the requirement of all support classes, which were encountered during excavation of the varying conditions of hard, weak and mixed ground.

An assessment of the internal stability of the tunnel excavation in weak or mixed ground was made using a finite element program, Phase^2 (Roscience, 2001). A simplified model in two dimensions was established to simulate the complicated three dimensional behaviour of staging excavation and ground relaxation. The Mohr-Coulomb constitutive model was used to present the plastic behavior of rock mass and roof canopy. Four elements of temporary support have been included in the analyses such as, rock dowels, pile roof canopy, steel arch ribs and fibre reinforced shotcrete.

Due to the relatively large spans of the tunnels, the excavations were undertaken by the heading and bench method, so that supports was installed immediately after exposure thereby not allowing significant convergence to occur. For example a two stage excavation strategy of South Portal southbound tunnel in a thin rock cover condition is firstly a top heading (maximum 8m height) with one row pile roof canopy, lower bench staging excavation and steel ribs of UC 203mm x 203mm x 46 kg/m with 200mm thick fibre reinforced shotcrete. The predicted ground responses at chainage Ch. 1123 of southbound tunnel are shown in **Figure 7**.



- Vertical deflection at the tunnel crown in the order of 14 mm
- Invert heave of above 8 mm at stage 1 top heading excavation

Figure 7. Numerical simulation of staging excavation

INSTRUMENTATION MONITORING

The rock tunnel sections were excavated by conventional blasting method using cartridge and bulk emulsion explosives and non-electric detonators. In the poor and extremely poor rock masses, and where blasting and bulk explosive was not practical on the basis of high levels of blasting vibration in the poor ground, mechanical excavation by stages was carried out, with

supports installed close to the face for limiting ground deformations and convergence, and ensuring that ground movements would not exceed the preset limits.

Monitoring and the facility to upgrade support in the light of any adverse or unexpected ground conditions were also essential parts of the design and construction process. Monitoring and instrumentation along the alignment of the tunnel were established on the basis of the detailed design. Site monitoring during the whole construction process was controlled by relating the derived figures to alert, action and alarm levels. The instruments were installed both within the tunnels to monitor the ground behavior, and above the tunnels, particularly near the portals, in close proximity to the WSD facilities and the existing buildings and slopes, in order to monitor the effects of tunnel excavation by blasting. Prior to the commencement of construction, instrumentation at some critical locations was installed to establish baseline readings to monitor the ground movements and vibration during tunnelling. Over 400 instruments were installed outside the tunnel and over 2000 monitoring points were installed inside the tunnel for monitoring of tunnel construction. The total number of each type of instruments installed is shown in **Table 1**. Instruments installed outside the tunnel are shown in **Figure 8**.

Table 1. Number of instruments installed for monitoring of tunnel construction

Instrument type	Total number
Standpipe piezometers	35
Vibrating wire piezometers	10
Vertical inclinometers	1
Horizontal inclinometers	2
Multiple-points borehole extensometers	5
Ground settlement markers	224
Structure settlement marker	99
Structure tiltmeter	5
Utility monitoring points	3
Telltale	5
Vibrograph in borehole	7
Vibrograph	36
Air overpressure monitoring	3
Flow meter with V-notch weirs	43
Convergence monitoring	2000+ *

* Multiple arrays of convergence monitoring points at each blasted profile after advance by blasting.

Standpipe piezometers and vibrating wire piezometers at selected locations were installed to monitor the groundwater levels and to prove the performance of the temporary drawdown. The major underground monitoring consists of convergence measurements, groundwater measurements and vibrations from blasting operations.

As the instrumentation monitoring was numerous in quantity, a database was developed for storage all the instrumentation monitoring data. This also allowed data trends for individual instruments to be reviewed. This assisted in the operation of the alert, action, alarm system and also demonstrated when the instrument response was stabilized. The monitoring frequency could then be reduced and bring focus on the sensitive areas.

Groundwater Measurements

Groundwater levels were obtained over two seasonal durations from the standpipes and piezometers and have been analysed in conjunction with meteorological conditions over the seasonal durations. Hence, the groundwater contours were developed.

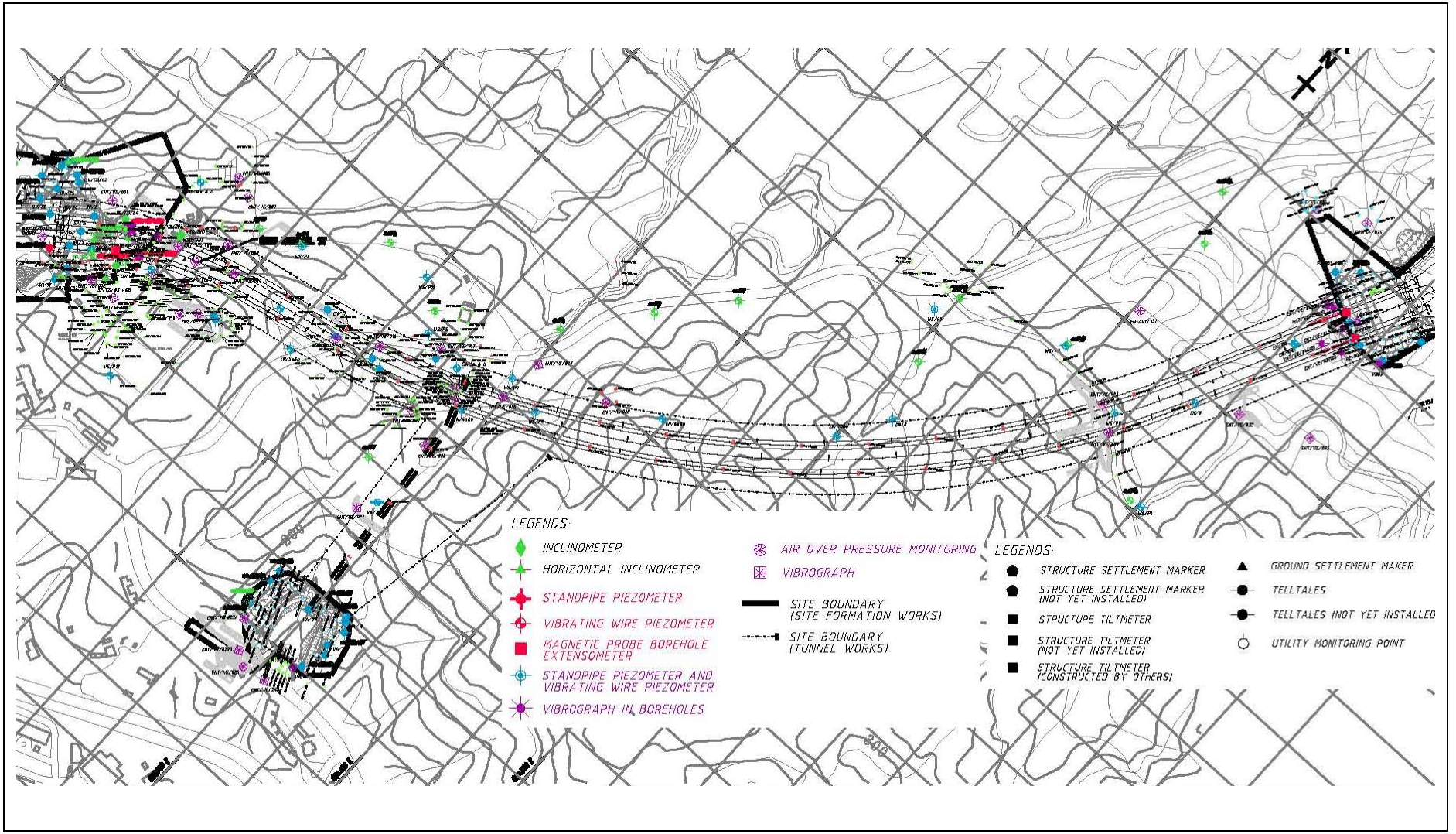


Figure 8. Instrumentation plan

All significant ground water ingress was monitored and the inflows have been measured in segments of 100m in length over the full length of the tunnels. Measurements obtained are expressed in litres/min/100m for each designated segment. Groundwater levels have been obtained from standpipes/piezometers installed in boreholes. Initial readings and past records have formed the basis of a baseline data set. If any monitored groundwater levels for any geotechnical feature should exceed its 'action level' for any of the three days immediately preceding a proposed blast, then appropriate action at the tunnel face was taken, such as grouting of the rock mass.

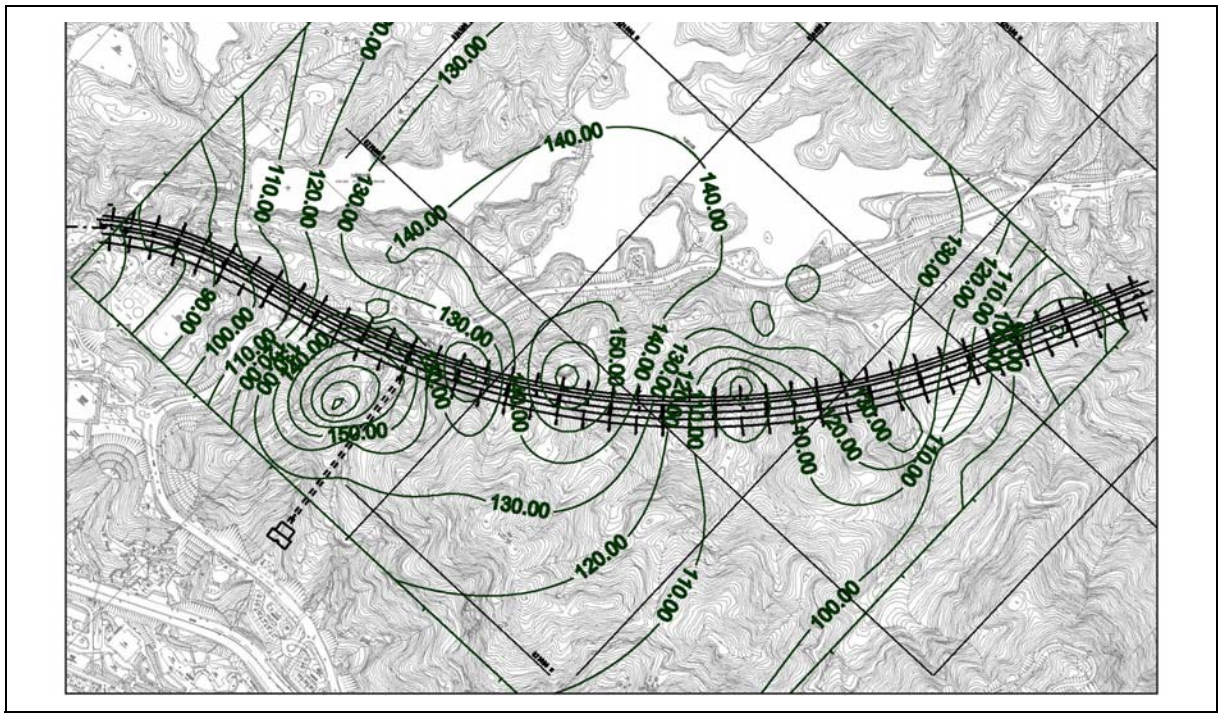


Figure 9. Plan showing developed groundwater contours (during excavation and construction of tunnel)

Figure 9 plots the piezometic data obtained when the permanent tunnel concrete linings are almost in place. The phreatic profile is found slightly altered with the tendency for the gradient to lower towards the toe of the tunnel portal formations.

During the actual excavation the ground water inflows encountered were with only two exceptions within the specified limits. Those two exceptions were treated by pre-grouting at the excavated face to reduce the inflows below 36 litres/min on any 100-metre length of tunnel. Drawdown from the Kowloon and Kowloon Byewash reservoirs was not encountered during the excavation or concrete permanent lining operation, and water inflows to the tunnel ground water drainage system are low (see **Figure 10a** and **10b** respectively).

Convergence and Deformation Measurements

The tunnel excavation profile tends to close under the geostatic stresses of the surrounding rock mass and, depending on the ground characteristics, this effect may cause either reduction of the excavated area or collapse. Thus, a basic requirement for the excavation of a successful tunnel is the control of section closure or of its instabilities, with the establishment of a detailed monitoring system to record tunnel deformation during the excavation and indicate the need for additional measures if convergence exceeds acceptable preset levels.

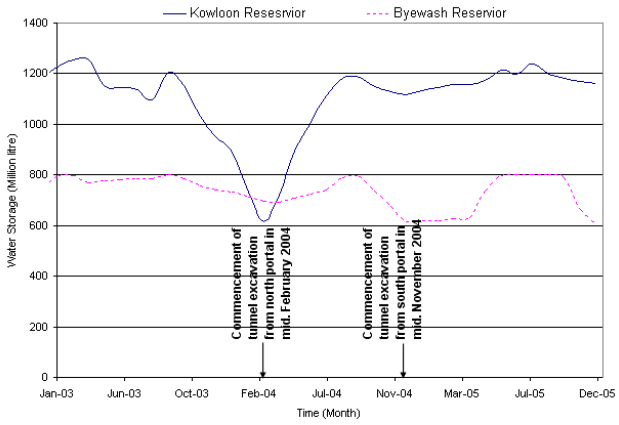


Figure 10a. Charges of water storage in reservoirs

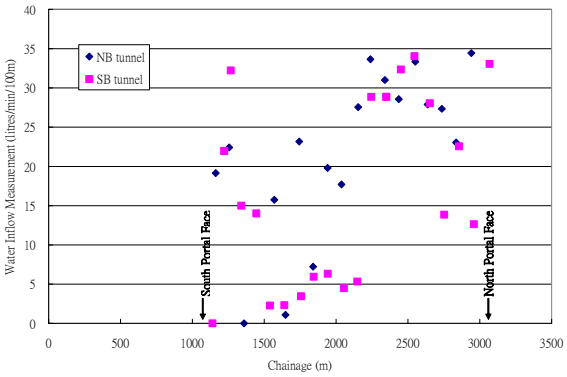


Figure 10b. Groundwater inflow measurement

The tunnel support shells react to the stresses of the surrounding rock mass tending to reduce the excavated opening and this reaction induces a time-dependent deformation of the various sections. An attempt to explain the relationship between ground and lining requires the detailed recording of their interaction and the analysis of the monitoring data in terms of time-dependency, tunneling method adjustments and local conditions. The geodetic observation of tunnel deformation provides data concerning the behaviour of the excavated area and permits the understanding of the kinematics of the support shell.

Within the tunnels, convergence monitoring during excavation has been carried out using 3 or 5 point arrays, and the monitoring locations and frequency were determined according to the excavation stages and the support class. The least ground cover section at chainage about Ch. 1123 of southbound tunnel, the crown settlement after top heading excavation was increased up to about 15mm and converged after bench excavation (see **Figure 11**). Comparison measured ground response showed a reasonable correlation with predictions. In addition, two horizontal inclinometers were installed above each shoulder of southbound tunnel at a distance of 2.5 metres in length of 40 metres from the south portal face and the multiple-points borehole extensometers were installed immediately about the tunnel crown to monitor the ground deformation particularly for this tunnel section with low ground cover.

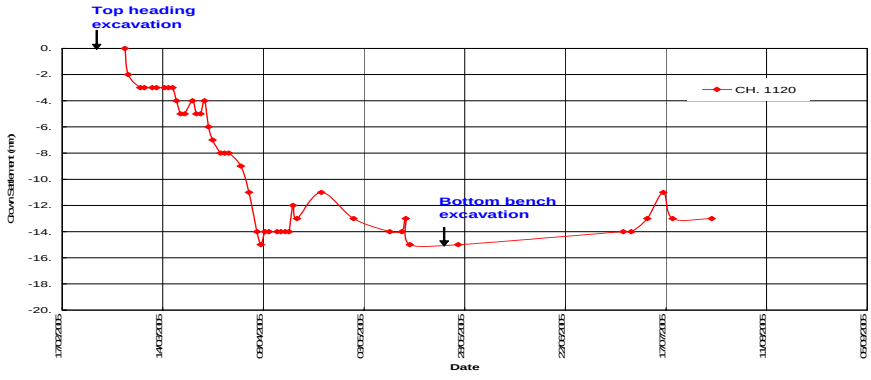


Figure 11. Measured deformation of tunnel crown at Ch. 1120 of southbound tunnel

Vibrations from Blasting in Tunnel

The blast-induced ground vibrations were monitored using vibrographs at the sensitive receivers in the vicinity of the blasts. Preliminary instrument locations are shown on site wide

monitoring and instrumentation layout plans. The locations and numbers of instruments shown on the site wide layout plans were a general regime of instrumentation, and during the actual blasting operations, the positioning of the surface mounted instruments was reviewed on a "blast by blast" basis to ensure that vibration monitoring was applied directly between the blast location and the adjacent to respective sensitive receiver on the surface.

For sensitive receivers that were underground such as the High Island Water Tunnel bend 'B', which was 30 meters under the ENT North Portal, blast monitoring instrumentation was installed in boreholes sunk to the same depth as the deep sensitive receiver being monitored.

Instruments on particularly sensitive receivers such as water retaining structures were commissioned before the blasting face was less than 50 meters from the sensitive structure and maintained operational until the blasting face had moved 50 meters from the structure. Vibration from each blast was reviewed and propagation constants for the rock matrix were constantly reviewed in conjunction with vibration levels sustained. Air overpressure measurements were also recorded and hence overhead and airblast was kept limited.

In addition to the blast vibration monitoring, pre-construction condition surveys, and regular weekly surveys were carried out by the Independent Blast Consultant (IBC) employed by the Contractor in order to check for damage due to blast-induced vibrations. The regular surveys included the monitoring of tell-tales and crack-meters installed on the sensitive receivers. Particular attention was placed on the identification of new cracks, the opening up of old cracks, or the initiation of tension cracks. A record of each subsequent survey was prepared and submitted to the Engineer. Pre-construction and subsequent condition surveys for features within the WSD Special Protection Zone were carried out in WSD's presence and were also submitted to the WSD's Consultants. In addition the scope of the IBC included:-

(a) recommendation for continuance or halt to blasting works at that particular development face;
(b) recommendation for amendment/adherence of the propagation constants for determination of allowable charge weights; and
(c) recommendation for the amendment/adherence of the current blasting methodology.

The review discussed was certified by the IBC prior to submission to the Commissioner of Mines and the Engineer for approval. Blasting at that particular development face proceeded after the Engineer's approval was given to the Contractor's blasting designs.

Action levels due to blast vibrations assigned following the IBC's assessment were subject to the approval of Commissioner of Mines and the Engineer. Additional approval was required from the WSD's Consultants for blasting works within the WSD Special Protection Zone.

For any portion of the blasting works falling within 50 metres of the WSD Special Protection Zone, the Contractor submitted Special Blasting Designs that make full allowance to comply with the Contract requirements. Special Blasting Designs were fully supported by calculations, dimensioned sketches, documentary evidence and all means necessary to demonstrate that there was no risk of damage to the Water Supplies Department facilities. Special Blasting Designs provided the following as a minimum:

(a) layout, diameters and lengths of loaded and unloaded holes (including details of opening cut);
(b) quantity and type of explosive and stemming (including initiators);
(c) explosives ratio;
(d) firing sequence including delay types and timing;

(e) charge weight per delay – distance – anticipated PPV and similar relationships, developed for each of the WSD facilities; and
(f) information on rock strata

SPECIALISTS

Specialists play an important role in monitoring of the tunnel excavation. These specialists include engineering geology and geotechnical engineering construction staff, blasting engineer, Registered Shotfirer, Independent Checking Engineer (ICE) and Independent Blasting Consultant (IBC) appointed by the Contractor, Resident Site Staff (RSS) and the specialists in the design office appointed by the Employer and the specialists consultants appointed by the WSD.

Different Types of Specialists and Their Roles

The above specialists are categorized below in terms of the level of technical skill and experience required for them to perform their duties.

ICE shall check the Contractor's works and then submit to the Engineer for the following areas of the works prior to execution on site:-

(a) initial supports and ground stabilization for tunnel excavation and corresponding instrumentation scheme;
(b) tunnel excavation and groundwater control proposal;
(c) certify face mapping, grouting and day to day monitoring of the works before submission of these documents to the Engineer;
(d) any other remedial work proposals as instructed by the Engineer or any adjustment in the Contractor's method of working to suit the probing water inflow and monitoring results; and
(e) pre-construction condition surveys and subsequent periodic surveys for the duration of the Works. The ICE shall certify the results of the surveys before submission to the Engineer.

IBC whose role is the same as that of an ICE to undertake the following:-

(a) review and certify blasting designs including trial blast programmes and other special measures that may be required for sensitive areas;
(b) attend initial and all subsequent conditions surveys and certify the results;
(c) participate in supervision of drilling and charging and give final approval to blast;
(d) review and certify blast remedial and contingency measures;
(e) review and certify protection works; and
(f) review pre-blast and post-blast records and monitoring instruments to independently monitor the blasting

The Contractor submitted a list of the personnel nominated for the tunnelling works as required in the contract and employed qualified personnel and carry out all the certification and recurrent training in accordance with the Factories and Industrial Undertakings (Confined Spaces) Regulation.

RSS are essential in any large civil engineering contract to ensure that the construction procedures and quality are in accordance with that specified in the contract drawings and specifications.

WSD's Consultants to ensure the tunnelling works to be carried out have complied with

the requirements for any part of the works falling within the WSD Special Protection Zones (WSDSPZ).

DISCUSSION

Two main tunnels of approximate 2.1 km long and of maximum 19 m span, pedestrian cross-passages of approximate 4 m span at every 100 m centres and a 400 m long ventilation adit tunnel were successfully excavated. A tightly controlled excavation and comprehensive monitoring scheme was implemented during tunnel construction. Ground movements and groundwater measurement during the excavation showed reasonable correlation to the estimation.

Through out the tunneling works, all parties especially the Contractor take on a significant role of protecting the WSD facilities and all other structures close to the tunnel from many risks and threats posed by drill and blast method for the large span tunnels excavation. It was therefore considered that the specialists played an important role in monitoring of tunnel excavation.

The management of such comprehensive monitoring data and continuously reviewing the data from time to time in the tight time schedule of tunnel construction work is an extremely critical event. Experienced that extensive resources and time input are essential, in order to ensure that there are adequate safety margin or the need for mitigation measures. If unstable ground condition encountered, the people would always question that is it moving? what is the direction of movement? what about the magnitude of movement? any groundwater drawdown? etc. It is therefore advisable to enforce in the contract to develop a database in order to process data and produce graphical plots daily, desirably after each production blast, for checking against the predicted.

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MONITORING OF GROUND RESPONSE ASSOCIATED WITH PIPE JACKING WORKS - RECENT EXPERIENCE IN HONG KONG

Jian Bing Wang[1], Leslie Swann[2] and Boyd Merrett[3]

Abstract

Currently, the prediction of ground settlements and jacking forces developed with pipe jacking are generally based on empirical methods derived from past observations. With the continuous development of jacking techniques and practice in different regions, it is necessary to carry out studies on local practices and gain a better understanding of the techniques.

This paper documents studies of the ground responses observed in several recent pipe-jacking projects carried out on the Hong Kong Island. The behaviour of the ground and jacking pipes during the tunnelling and jacking are discussed and compared to the predicted values. The propagation characteristics of the ground movements ahead of the tunnelling face are also investigated. Based on the observed data, factors affecting the pipe jacking works are addressed with recommendations made for pipe-jacking design and practices.

INTRODUCTION

Currently, pipe-jacking is widely adopted in Hong Kong for the installation of underground utilities, such as drainage systems and water mains with typical diameters of 800 mm to over 2000 mm.

It is not unusual that significant ground settlements occurred and adjacent properties damaged as a result of pipe-jacking works. Therefore it is very important for engineers to assess, at design stage, the likely impacts of the proposed pipe jacking work on the adjacent structures and utilities. Currently, the prediction of the ground settlements and jacking forces developed with pipe jacking are generally based on empirical methods derived from past observations. With the continuous development of jacking techniques and practice in different regions, it is necessary to carry out studies on the local practices and gain better understanding of the techniques.

This paper presents case studies of the ground responses observed in several recent pipe-jacking projects carried out on the Hong Kong Island. The pipe jacking projects presented in this paper covered deep and shallow tunnels at different ground conditions with either TBM or hand-dug with open shield. The development of the ground settlements and

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jacking forces recorded are presented with respect to ground conditions encountered and on site activities. The behaviour of the ground and jacking pipes are discussed and compared to the predicted values. The propagation characteristics of the ground movements ahead of the tunnelling face are also investigated. Problems arising from the prediction of the ground response by current design practice are highlighted. Factors affecting the pipe jacking works are addressed. Finally recommendations are made for pipe-jacking design and practices on the basis of the lessons and experiences learnt from the studies.

BRIEF DESCRIPTION OF THE PIPE-JACKING WORK

Pipe-Jacking Project Across Gloucester Road

The project involved installation of a watermain under Water Supplies Department (WSD) Contract 17/WSD/97. Pipe-jacking work was carried out under Gloucester Road, one of the busiest road located on Hong Kong Island. The project was discussed by Swann & Wang (2003).

The pipe-jacking work comprised installation of a DN1400 mild steel sleeve pipe (19 m in length) by an open shield pipe-jacking method with subsequent DN1000 M.S. pipes placed inside the sleeve pipe. The pipe-jacking work was carried out under only 3.5 m below the ground.

The jacking pit was located within the central divider of Gloucester Road and was 1 m away from Pier No.4 of the Arsenal Street Flyover which is founded on shallow footings. The jacking pit had dimensions of 8 m x 5.5 m x 6.3 m deep. The receiving pit was 6 m deep and 4 m x 4 m on plan. To overcome obstructions during installation of the supporting wall, steel pipe piles were installed and propped by three and two layers of walings and struts at the jacking and receiving shafts respectively. At an early stage of the construction works, prior to any excavation and pipe-jacking works, settlements of the ground and adjacent structures were recorded. Subsequently, an extensive programme of precautionary ground treatment works was implemented.

To minimise groundwater ingress during tunnelling and pipe-jacking and to strengthen the subsoil, horizontal grouting work was carried out along the jacking alignment prior to pipe-jacking work. The grout take was large (up to 500 litres of grout per metre was required in some locations) as the soil being grouted was loose with voids.

After a full technical review, it was decided to excavate the tunnel face manually with an open shield, given the existence of old sea walls and boulders along the jacking alignment.

Pipe-jacking work was carried out between 20 August 2002 and 28 September 2002. A DN1400 mild steel sleeve pipe was jacked from the jacking pit towards the receiving pit by two 250 ton hydraulic main jacks having a stroke of 1000 mm. To minimise the disturbance to the ground due to the excavation, the maximum clearance between the shield and the excavated working face was limited to 300 mm. Ground and groundwater conditions ahead of excavation were assessed based on the probing holes drilled 3 m ahead of the working face and the drilling record of the horizontal grouting holes. The working face was sealed with steel channels and sand bags during non-working hours.

The working face was observed to be in a stable and dry condition throughout the whole

period of pipe-jacking as a result of the good horizontal grouting, which was identified as layers and blocks of grout combined with grout veins. In addition to cobbles and boulders, concrete blocks (seawall), 2.5 m thick, were encountered between Chainage 8.5 m and 11 m.

Pipe Jacking Works For DSD Contract No. DC/2000/11-Wan Chai East and North Point Sewerage -Trunk Sewers

The project involved installation of sewerage trunk sewers between Wan Chai East and North Point under DSD Contract DC/2000/11. The project consists of 3.8 km pipe jacking work with outside diameter (OD) of 1400 mm to 2200 mm under 3 m to 17 m soft ground cover along the busy commercial and residential district of Hong Kong island. To facilitate the pipe jacking work, a total of 27 jacking and receiving shafts were sunk along the jacking alignment. Concrete pipes were jacked by slurry balanced TBM with an ability to cut hard rock and to bore in soft ground. Four mix-shield TBMs were supplied by mts Perforator and Herrrenknecht specially designed to drill through rock and soil and within groundwater.

Both circular and rectangular shaped jacking/receiving shafts were adopted to facilitate the proposed pipe-jacking work. The shafts were supported either by pipe pile walls or sheet pile walls. Grout curtains were installed below the temporary retaining wall to provide water cut-off. Unsurprisingly, various types of constraints/obstructions were encountered during the construction of the shafts in such an urban environment and variable ground conditions. Grouting and lagging often had to be considered to deal with any gaps/windows due to the existence of the major utilities. Shafts as large as 20 m long by 10 m wide were sunk with depths up to 16 m deep. However detailed shaft excavation is not discussed as the paper aims to study the ground behaviour associated with the tunnelling.

An automatic Pipe-jack Lubrication System was adopted using an automatic bentonite injection system which is fully programmable to allow the operator to inject a pre-determined amount of bentonite at all, or specific, areas of the tunnel without man entry to the tunnel. Every fifth pipe installed in the pipeline will have three bentonite injection ports. Three bentonite lines from the injection station distribute bentonite to the injection ports as in any conventional or manual Pipe-jack Lubrication System. Whilst the TBM is driven forward the line is manually topped up with bentonite by progressively opening and closing the valves in sequence along the full length of the installed pipeline.

DESCRIPTION OF SITE CONDITIONS

Gloucester Road Site

The pipe-jacking work was carried out under Gloucester Road, with the Arsenal Street Flyover directly above. In particular, the Arsenal Street Flyover was supported on shallow spread footings (2 m below the existing ground), and one of the pier footings was 1 m away from the jacking shaft. Unsurprisingly utilities, including gas mains, water mains, telephone and power cables, were also present immediately adjacent to the excavation.

The ground investigation indicated that Marine and Alluvium deposits were present underlying the reclamation Fill. Layers of cobbles and boulders were identified in the Fill stratum. The observed ground water level was approximately 2.6 m below existing ground level, typically standing at around +1.3 mPD.

Sewerage Trunk Sewers – Wan Chai East to North Point

Along the jacking alignment, there are numerous existing buildings/structures together

with various kinds of utilities above including gas mains, water mains, drainage pipes, telephone and power cables. Some of the buildings were built over 50 years ago, unsurprisingly being at poor conditions sitting on shallow footings.

The majority of the land along the tunnel alignment is old reclaimed land, reclaimed last century. The ground investigation indicated that, under most of the jacking drives, above Completely Decomposed Granite (CDG), Marine and Alluvium deposits were present underlying the reclamation Fill. The Fill was variable, described as very loose to dense clayey to silty Sand, Sand and Gravel. Cobbles, Boulders and construction debris also exist in the Fill. The Marine Deposits range from loose to dense Sand and clayey Sand, locally very soft to soft Silt/Clay. The Alluvium generally consists of medium dense, clayey Sand with some gravel.

The observed ground water level was generally 2 to 4 m below existing ground level, typically standing at around +1 mPD to +3 mPD, though exhibiting tidal fluctuations. Most of the shallow pipe-jacking portion of the alignment was excavated within Fill. The deep pipe jacking portion of the alignment, generally encounter superficial deposits of Alluvium and Marine deposits and locally Fill or CDG.

Plate 1. Pipe-jacking under Busy Road

Plate 2. Jacking Frame Setup at Shaft NP12

Plate 3. Removal of TBM Head

Plate 4. Large Boulder Encountered

PREDICTION OF JACKING FORCES AND GROUND MOVEMENTS ASSOCIATED WITH PIPE-JACKING

Predicted Jacking Forces

The jacking force comprises penetration resistance of the shield and frictional resistance to the pipe string. In the pipe-jacking design, prediction of the jacking load is important to select the jacking pipes, tunnelling method, design of jacking pit and thrust wall, and to determine the need for and the location of intermediate jacking stations, and lubrication requirements.

Penetration resistance to the advance of the shield or boring head is mainly related to the shield head type and excavation method involved. In open face operation, the penetration resistance is only the load required to overcome the cutting edge resistance while a TBM with pressure balance method consists of a combination of the contact pressure of the cutting face and hydraulic pressure supporting the working face.

The friction resistance to the pipe-jacking is mainly determined by following factors which is either construction related or site related :

(a) ground and groundwater conditions along the tunnel alignment;
(b) surface loading and stability of the tunnel bore and face;
(c) depth of tunnel and the alignment shape both vertically and horizontally;
(d) amount of overbreak during the pipe-jacking;
(e) pipeline misalignment;
(f) amount and type of lubricants used; and
(g) frequency and duration of jacking stoppages.

In particular, the friction resistance on pipes varies significantly depending on the effectiveness of applying lubrication, ground conditions, overcut ratio, stoppage time and the pipe jacking length. Based on the published case studies, Roe (1995) recommended a friction resistance of 5 to 25 kN/m^2 for design purposes. With an effective lubrication during jacking, the reaction resistance may be largely reduced. On many projects the use of the correct lubrication materials and techniques can bring about a considerable reduction in jacking loads and ground support problems. It may also allow the use of a smaller jacking frame, thus minimizing the size of the drive shaft and helping to reduce the overall cost of the project.

Predicted Ground Movements

Inward soil movement due to the stress release and overcut of the ground by tunnelling inevitably causes ground movement around a tunnel. If these movements become excessive, they can damage adjacent buildings and overlying facilities. While many factors contribute, ground loss is often caused by :

(a) overcutting between the diameter of the tunnelling machines and that of the lining;
(b) inward soil movements of the tunnel face;
(c) overcutting due to curved alignment and any significant deviation from the tunnel alignment occurred when steering the shield by workmen lacking in experience; and
(d) the effectiveness of tail void grouting.

In pipe-jacking, ground loss due to movements at the tunnel face can be negligible if the tunnelling face is continuously supported by slurry pressure or the excavation is carried out in

a stable ground. In such case, the overbreak of the ground during the pipe-jacking contributes most to the ground loss. Lake et al (1992) indicates settlement occurs at the ground surface at a distance of 1 to 2 times of tunnel depth ahead of the tunnel face and the initial settlement is typically 80% to 90% complete at a similar distance behind the face.

Peck (1969) established an empirical method of representing ground surface settlements due to the tunnelling. He also established a correlation between the relative depth of tunnel and the point of inflection of the transverse surface settlement trough for various soil types. The extent and shape of transverse ground surface settlement due to the pipe-jacking work can be described by an error function curve of the form shown in the following:

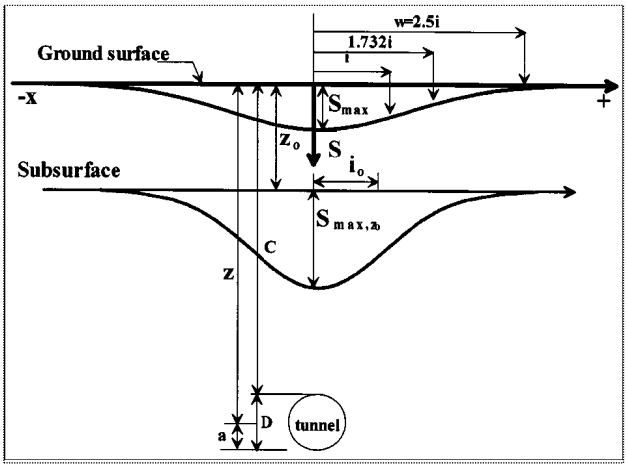


Figure 1. Transverse Ground Settlement Trough (after Peck, 1969)

Surface settlement, S_v, is defined as

$$S_v=S_{max}\exp(-y^2/2i^2) \qquad \text{Eq (1)}$$

The width of the settlement profile is conveniently described by i, which is approximately an linear function of depth H

$$i = K.H \qquad \text{Eq (2)}$$

where K is dependent on the soil type of the ground and

for Sands K=0.25 – 0.45
for Clay K=0.4 – 0.6

S_{max} may be calculated based on the assumed ground loss v_s. Provided adequate slurry pressure is provided and the pipe annulus is fully grouted on completion, settlement above the tunnel should be small. If inadequate slurry pressure or grouting is applied, for full closure of overbreak on the long term, then maximum ground settlement is

$$S_{max} = v_s /(i(2\pi)^{1/2}) \qquad \text{Eq (3)}$$

At a distance 2i from the central line of the tunnel (x=0), the ground surface settlements decrease to less than 15% of the settlement at x=0. The ground on the site comprises generally Sandy soil, a K value of 0.4 was adopted. Therefore the ground settlement

decreases quickly with distance away from the tunnel, and the settlement beyond 2i (0.8H) is relatively very small.

The maximum ground settlement and distribution of the settlement trough above the tunnel were predicted using equation (1) and (3). The predicted settlements are presented and compared with the observed values in the following sections.

OBSERVED JACKING FORCE AND GROUND SETTLEMENTS

Development of Jacking Force

Based on the recorded jacking force for each drive, the development of the jacking load during the pipe jacking for drive WC2-WC4, WC5-WC4, NP12-NP11, NP12-NP13 and NP15-NP16 is given in Figures 2 to 6. Brief description on the observed data for each drive is presented below.

Figure 2. Jacking Force Vs Chainage (WC2-WC4)

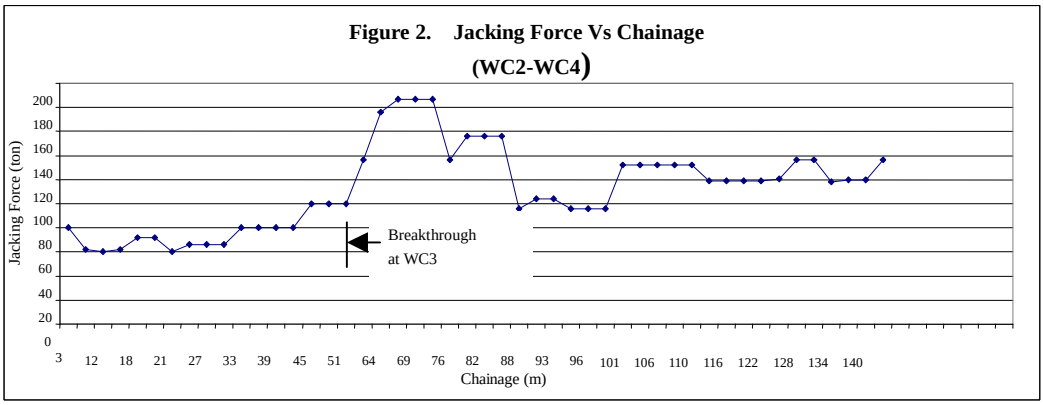
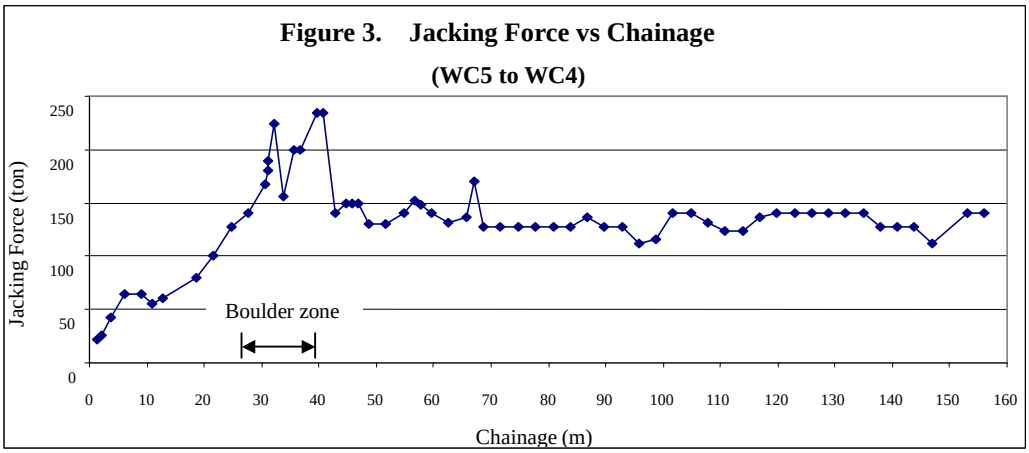
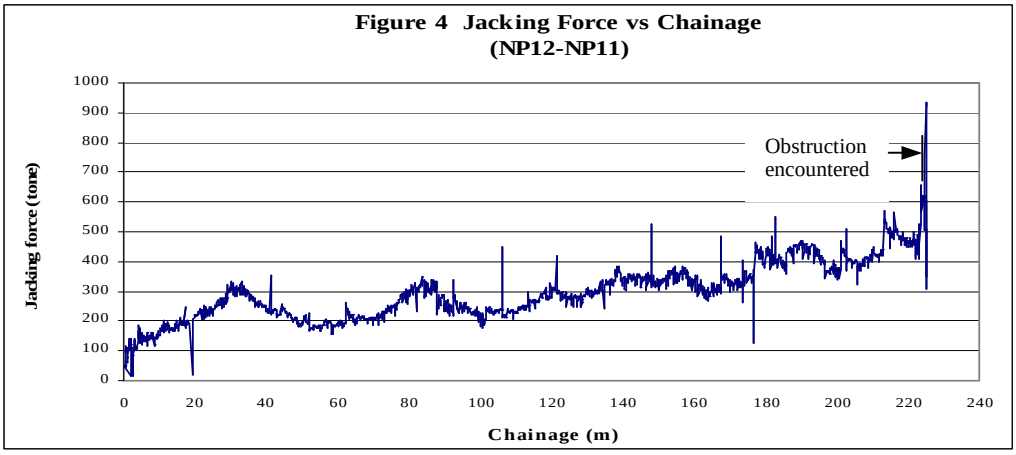
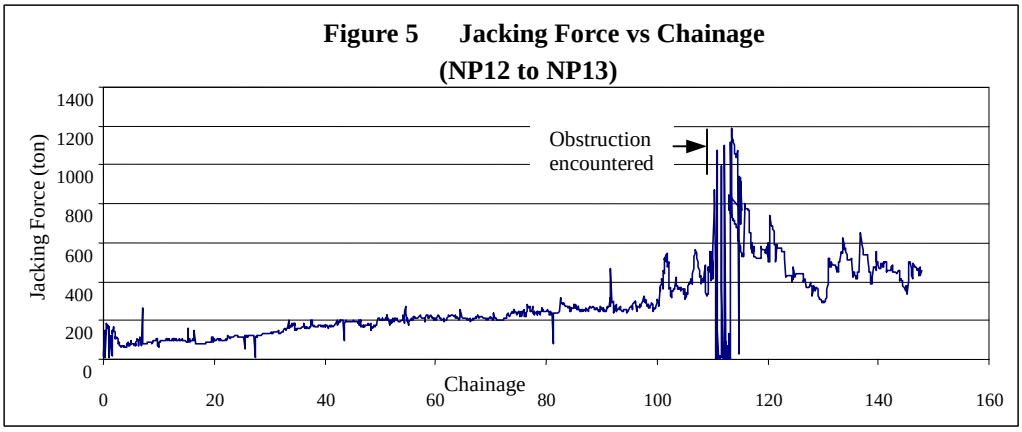
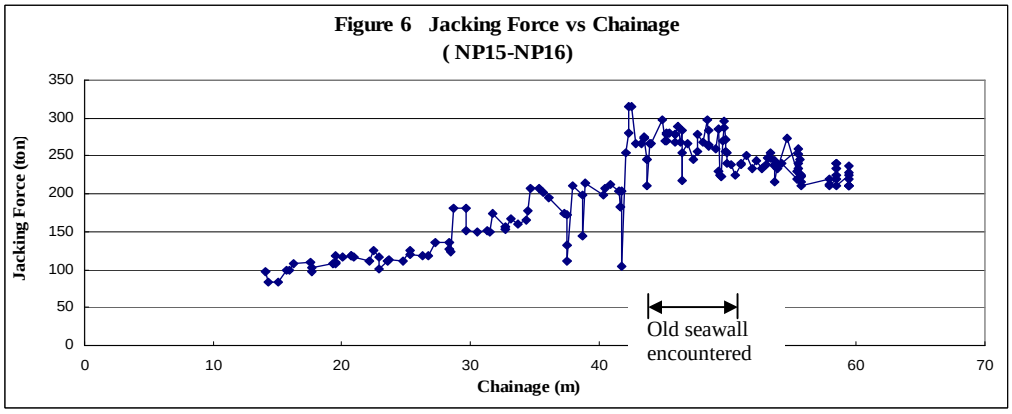


Figure 3. Jacking Force vs Chainage (WC5 to WC4)









Jacking Force Development during Pipe-jacking between WC2-WC4

The drive is approximately 142.7 m with a curved section of 400m radius, involving a diameter of OD 2200 mm pipe jacked under a depth of 4.5m to 5.5m below Percival Street. The pipe-jacking was carried out generally within the Fill and partly in Marine Deposit. The ground water is 2.1 m to 2.3 m below the existing ground level, at +1.2 mPD to +1.5 mPD. SPT 'N' values around the pipe jacking level are 2 – 17. From Figure 2, it can be seen that the jacking force remained very small, at around 80 tonnes till a chainage of 58 m where the jacking resistance raised sharply and reached 185 tonnes at Chainage of 66 m. Thereafter the jacking loads dropped and was maintained at around 120 tonnes from Chainage 88 m till completion of the jacking drive.

On 7 June 2004, the pipe-jacking successfully broke through shaft WC3 and re-entered the entrance eye of the WC3. The rise of the jacking force is likely due to the loss of lubricants during the process of the breakthrough and re-entering the ground at WC3. However, by examining Figure 2, the average friction resistance derived from the recorded jacking force is only about 0.84 kN/m^2 compared with the 8 kN/m^2 assumed in the design. The drive curvature did not cause any significant increase of the jacking force.

Jacking Force Development during Pipe-jacking between WC5-WC4

The drive between WC5-WC4 is approximately 158.4 m having an OD 1450 mm pipe jacked under a ground cover of 4.5 m only. The jacking-drive had a curvature of 750 m in radius. The majority of the pipe-jacking was carried out in a generally loose to medium dense sandy Marine Deposit immediate below the Fill, and in CDG around jacking shaft WC5. The recorded ground water is 1.6 m to 2.5 m below the existing ground level at +1.2 mPD to +2.5 mPD. SPT 'N' values are 8 - 22 around the pipe jacking level. Figure 3 indicates that the jacking force increased quickly before chainage 32 m reached 240 tonnes and dropped to 150 tonnes afterwards and remained constant for the rest of the jacking drive. An inter-jack was installed at chainage 57 m but was unused for the rest of the drive as the jacking force remained small.

Significant amount of corestones of granite were encountered between Chainage 0 and 32 m, which increased jacking loads steadily. Thereafter, the jacking loads remained generally constant showing the lubrication with bentonite being fully effective. The friction resistance developed at the first 30 m jacking length was estimated at about 9.8 kN/m^2 compared with the predicted 10 kN/m^2.

Jacking Force Development during Pipe-jacking between NP12-NP11

The drive has a total length of 290 m between NP12 and NP11 under Java Road. The tunnel is approximately 15.4 m below the ground having an OD of 2200 mm. The pipe-jacking was carried out in Alluvium Deposits for the first 130 m and in CDG for the remaining drive towards NP11. A thick layer of very soft to medium dense Marine Deposit exists immediate above the tunnel bore. Groundwater level is about 3.6m below the existing ground. SPT "N" values around the pipe jacking level are 4 – 62 towards NP12 and 10 - 76 near NP11. As Figure 4 shows, the jacking force increased steadily between Chainage 0 and 32 m reached about 340 tonnes without adding bentonite lubrication. Thereafter, both benonite and polymer were regularly applied, and the jacking resistance remained between 200 tonnes and 350 tonnes till Chainage 175 m. The jacking force rose to approximately 500 tonnes when the pipe-jacking was carried out to the Chainage of 223 m. However, a sudden rise of the resistance was recorded at change 225 m with a maximum value of 950 tonnes. The obstruction caused eight months delay to the pipe-jacking work. A rescue shaft was finally sunk at the location of the obstruction. It was observed that the cutters of the TBM were severely damaged. Finally the jacking broke through at shaft NP11.

The average friction resistance derived from the jacking loads for the first 35 m is about 9 kN/m^2, compared with the predicted 8 kN/m^2 for the whole drive. Therefore, small increase only of the jacking force was recorded for the remaining drive till obstruction encountered. The average friction on the jacking pipes between Chainage 45 and 223 m was only 2.3 kN/m^2.

Jacking Force Development during Pipe-jacking between NP12-NP13

Pipes of OD 2200mm were jacked between NP12-NP13 at a depth of 15.5 m below the

Java Road. The drive has a total length of 157.3 m. The pipe-jacking was carried out generally in Alluvium Deposit. Groundwater level is 2.5 m to 2.8 m below the existing ground level, standing at +1.4 mPD to +1.8 mPD. SPT 'N' values around the pipe jacking level are 4 – 22 around NP12 and 11 - 33 near NP13. As Figure 5 shows, the jacking force increased slowly between Chainage 0 and 110 m at which only about 400 tones was recorded. However, sudden rise of the resistance was recorded at change 110 m on 06/09/2003 with a maximum value of 1200 tones recorded. Steel shavings were observed by examining the excavated materials. The obstruction caused one month stoppage though the TBM head keep moving regularly to avoid stuck in the ground. The jacking loads varied generally between 400 tonnes and 600 tonnes after the TBM passed the obstruction.

Bentonite and conditioning foam were regularly applied throughout the jacking work. The average friction resistance derived from the jacking loads between Chainage 0 and 110 m is about 3.7 kN/m^2, compared with the predicted 12 kN/m^2.

Jacking Force Development during Pipe-jacking between NP15-NP16

The drive is 63 m in length involving jacking OD 2400 mm pipes at a depth of approximately 15.6m under Hoi Yu Street. The pipe-jacking was carried out in Fill near NP15 and in very soft to soft Marine Deposit towards NP16. The recorded groundwater is 2.4 m to 3.5 m below the existing ground at +1.2 mPD to +1.9 mPD. SPT values around the pipe jacking level are 11 - 40 at NP15 and 0 - 28 around NP16. From Figure 6, it can be seen that the jacking force increased steadily until Chainage 43 m. Significant amount of hard materials suspected as an existing old seawall were encountered between Chainage 43 m and 49.5 m. The jacking resistance jumped to 320 tonnes and remained around 230 tonnes after passing through the obstructions till break through. Frequently, metal and wood pieces were also recovered from the cutting materials along the jacking drive. Slurry pipes were also blocked several times by the cut metal and woods materials.

The average friction resistance derived from the jacking loads between Chainage 0 and 43 m is about 5.6 kN/m^2, compared with the predicted 12 kN/m^2.

Pipe-jacking at Gloucester Road

Given the existence of old sea walls and boulders along the jacking alignment, the tunnel was excavated manually with an open shield. The pipe-jacking work was carried out under only 3.5 m below the ground with a high groundwater level. In addition to cobbles and boulders, concrete blocks (seawall), 2.5 m thick, were encountered between Chainage 8.5m and 11m.

During pipe-jacking, inspection of the working face was carried out frequently. The working face was observed to be in a stable and dry condition throughout the whole period of pipe-jacking as a result of the good horizontal grouting. The recorded jacking force was relatively low, typically 10 bars to 20 bars, which was equivalent to 20 tonnes to 40 tonnes of jacking force or less than 5 kN/m^2 of pipe frictional resistance. No bentonite or other lubricants was provided as the jacking force was low. The alignment of the sleeve pipe was well controlled with a maximum deviation of 16mm recorded during whole jacking process.

Monitoring of the Ground Movements

Four types of deformation monitoring devices were installed along each jacking drive to monitor the settlements of the soil ground surface, rigid pavement, flexible pavement and building structures. Manual monitoring techniques were adopted. Grid of ground

settlement monitoring stations consists of three rows of settlement monitoring points at 5 m spacing along the jacking alignment. Settlement markers and tilting meters were also installed on adjacent buildings and structures. The movements of ground, underground utilities and adjacent structures were monitored 4 times a day at monitoring points within 'active working zone'.

Ground Settlement Along the Tunneling Alignment

Ground settlements during the pipe-jacking and after completion of the work were observed at the monitoring points along the jacking drive. Ground settlement contours generated based on the observed data are presented in Figures 7 to 11. To investigate the settlements caused by the tunneling, the ground settlements due to the shaft excavation are not included in the contour. It was observed that the cumulated total settlements due to the shaft excavation and tunneling are generally more than other areas of the drive as expected.

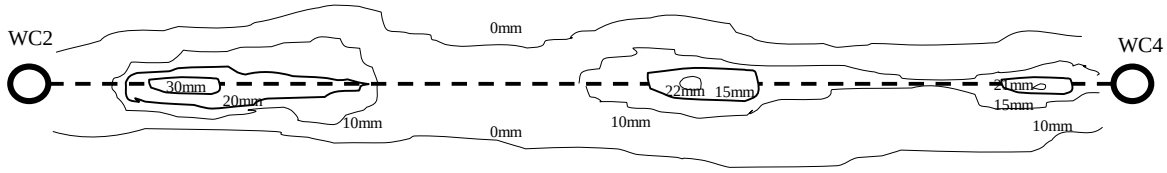


Figure 7. Surface Settlement Contour Along Drive WC2-WC4

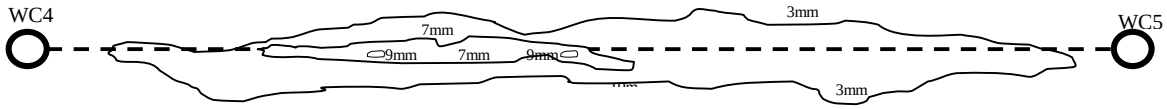


Figure 8. Surface Settlement Contour Along Drive WC4-WC5

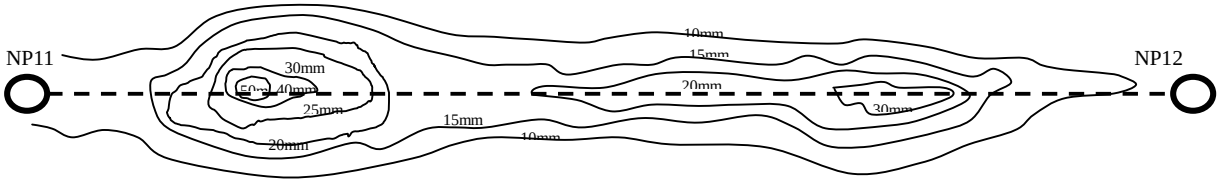


Figure 9. Surface Settlement Contour Along Drive NP12-NP11

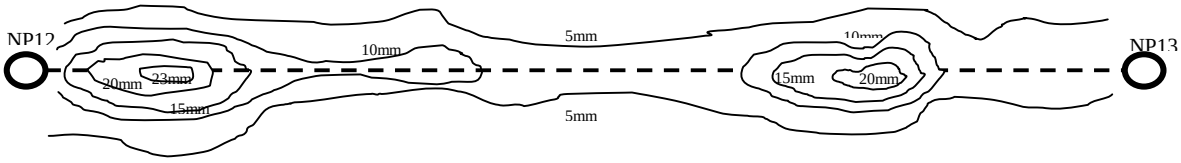


Figure 10. Surface Settlement Contour Along Drive NP12-NP13

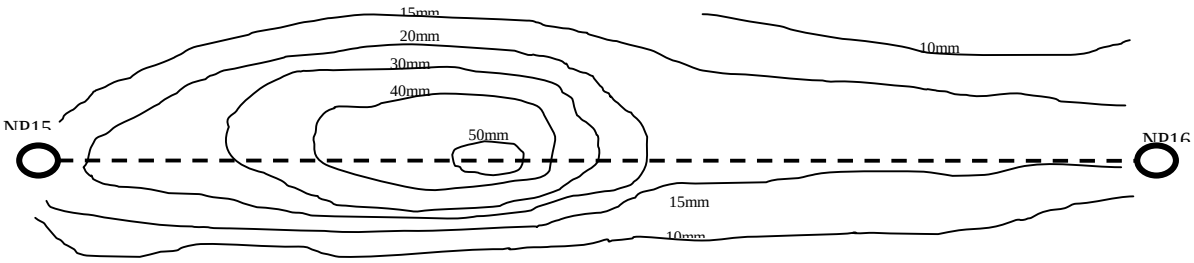


Figure 11. Surface Settlement Contour Along Drive NP15-NP16

Ground surface settlement contours shown in the Figures 7 to 11 indicate the settlements are generally less than 25 mm for majority of the drive. However, over 50 mm of maximum ground settlement was recorded along the jacking drive NP15 to NP16, and approximately half of the drive NP15-NP16 recorded settlements of 20 mm. Over 50 mm of ground settlement was also recorded near NP11 for the completed jacking between NP12 and NP11. The pipe-jacking between WC4 and WC5 caused only small ground settlement, being less than 10 mm.

By inspecting the settlement contours, it is observed the surface settlements distributed irregularly along the drive though maximum settlement always occurred immediate above the tunnel. The variation of the settlements is likely due to the variation of the ground conditions, workmanship and surface loadings, and is further discussed below.

Transverse ground settlements

The distributions of the surface settlements in a transverse section are presented in this section. As the settlements vary along the jacking drive as shown in the settlement contours, mean values of the recorded settlements at monitoring points along the central line, left and right hand side of the tunneling are adopted in the establishing the transverse settlement trough. The distribution of the transverse settlements is presented in Figures 12 to 16. To study the time effects, the development of the settlement immediate after completion of the drive and some time after is also presented in the Figures.

The predicted surface settlements based on the equation (1) is also included in the figures for comparison. It can be seen that, even after over one year, the recorded settlements along the central line of the drive are much smaller than the predicted values for shallow tunnels, only 30.7% and 30.3% of the predicted respectively at WC2-WC4 and WC5-WC4. However, for deep tunnels, the recorded settlements at drive NP12-NP13 are close to the predicted values (88% of the predicted) while that exceeded the predicted for drives NP15-NP16 and NP12-NP11.

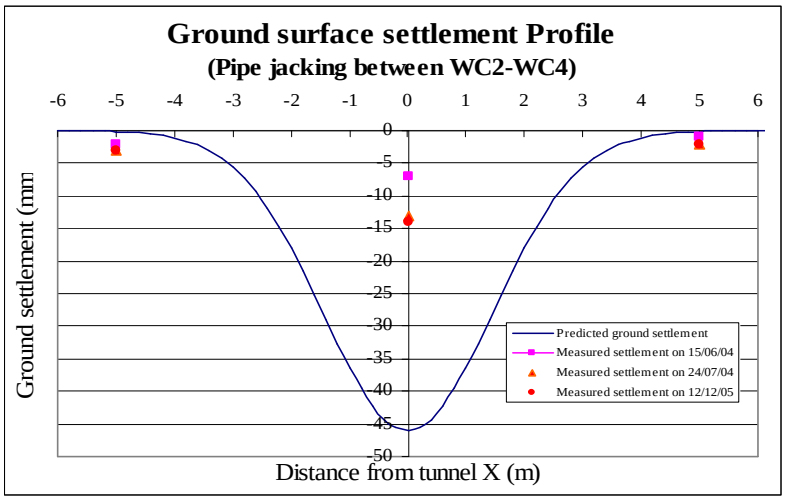


Figure 12. Observed and predicted transverse settlements trough for drive WC2-WC4

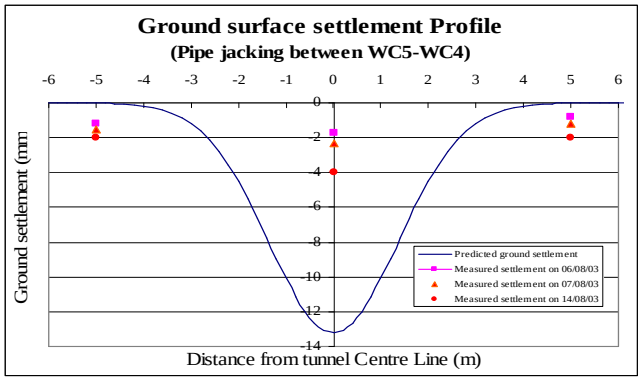


Figure 13. Observed and predicted transverse settlements trough for drive WC5-WC4

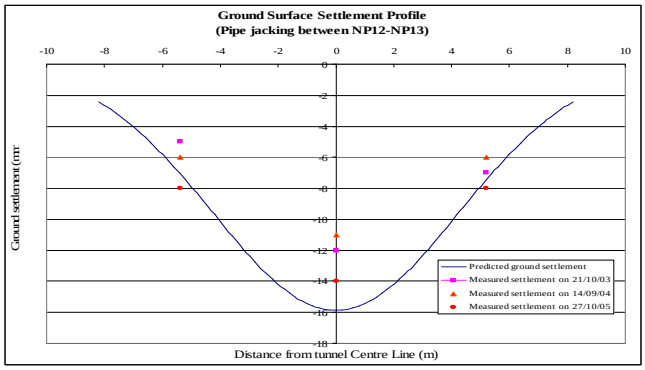


Figure 14. Observed and predicted transverse settlements trough for drive NP12-NP13

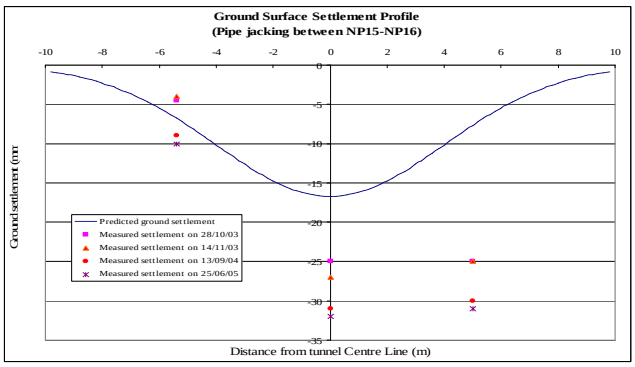


Figure 15. Observed and predicted transverse settlements trough for drive NP15-NP16

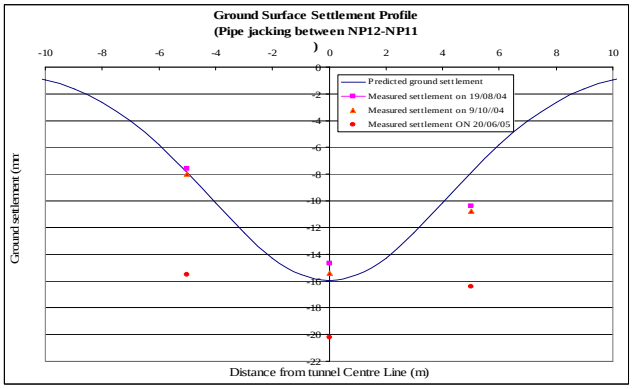


Figure 16. Observed and predicted transverse settlements trough for drive NP12-NP11

Table 1. Summary of Observed and Predicted Settlement (WC2-WC4)

Location	Predicted (mm)	Observed settlement (mm)			
		15/06/2004	24/07/2004	16/08/2004	12/12/2005
Central	45.6	7	13	14	14
Left	0.2	2	3	3	3
Right	0.2	1	2	2	2

Table 2. Summary of Observed and Predicted Settlement (WC5-WC4)

Location	Predicted (mm)	Observed settlement (mm)		
		15/06/2003	07/08/2003	14/08/2003
Central	13.2	1.7	2.3	4
Left	0	1.2	1.5	2
Right	0	0.8	1.2	2

Table 3. Summary of Observed and Predicted Settlement (NP12-NP11)

Location	Predicted (mm)	Observed settlement (mm)		
		19/08/04	09/10/04	20/06/05
Central	16.0	14.7	15.4	20.2
Left	8.0	7.6	8.0	15.5
Right	8.0	10.4	10.8	16.4

Table 4. Summary of Observed and Predicted Settlement (NP12-NP13)

Location	Predicted (mm)	Observed settlement (mm)			
		21/10/2003	14/09/2004	27/06/2005	27/10/2005
Central	15.9	12	11	14	14
Left	7.9	5	4	7	8
Right	7.9	7	6	8	8

Table 5. Summary of Observed and Predicted Settlement (NP15-NP16)

Location	Predicted (mm)	Observed settlement (mm)			
		28/10/2003	14/11/2003	13/09/2004	25/06/2005
Central	16.7	25	27	31	32
Left	7.7	4.5	4	9	10
Right	7.7	25	25	31	31

The development of the recorded settlements is also summarized in the Tables 1 to 5, indicting a 17% increase to more than double of the surface settlements after 1 to 2 years after completion of the pipe-jacking. It is also interesting to note that big increases occurred at the drive which recorded small settlement immediate after completion of the drive.

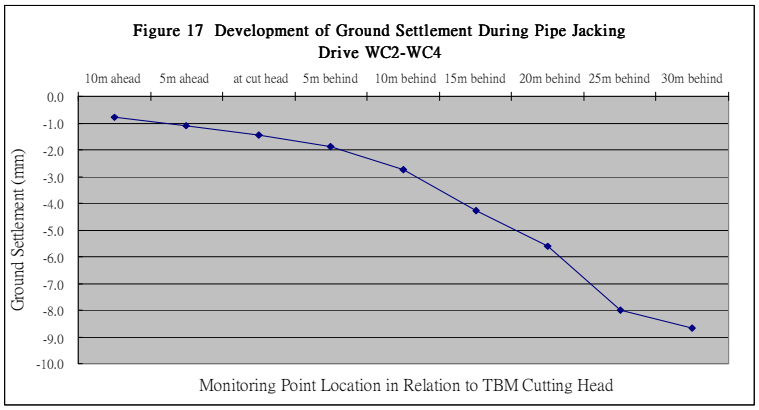
The pipe-jacking under Gloucester Road caused 4 mm ground settlements at monitoring

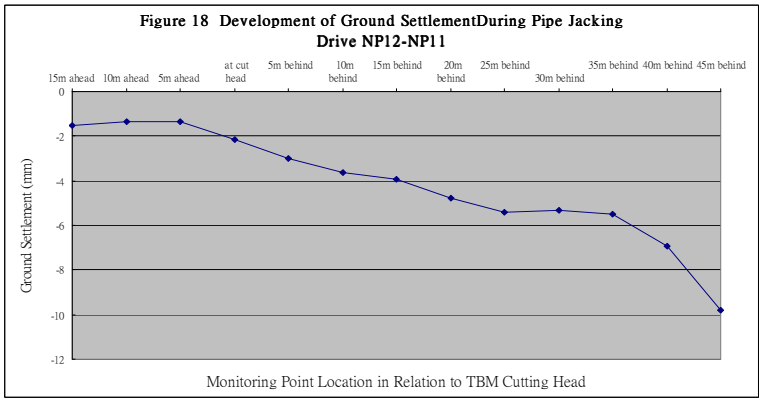
points located immediately above the pipe-jacking work, being only half of the predicted. This magnitude of ground settlement is considered to be minimal given that the pipe was being jacked under a small ground cover with heavy traffic above. The contractor was instructed to minimise the excavation by carefully trimming the ground and limiting the distance between the shield and the excavation face to a maximum of 300 mm during jacking. The working face remained stable and dry throughout the pipe-jacking work as a result of good horizontal grouting and pipe-jacking workmanship.

Development of Ground Settlement During Pipe-Jacking

The development of the ground settlement along the central line during the tunneling is presented in Figures 17 to 20. The average settlements recorded at the monitoring points along the tunnel central line ahead and behind the TBM cut head is summarized in the figures. It should be noted that the development and magnitude of the settlements may vary at each monitoring point, but the mean values of the settlements and their development along the jacking alignment can provide sufficient information showing a typical trend and magnitude of the settlements.

From Figures 10 to 12, it can be seen that the ground settlement at a distance beyond 0.5 depth of the tunnel (0.5H) ahead the tunneling face is generally negligible, and the settlement above the TBM is also small, being less than 8 mm in all four cases. However, the settlements developed quickly after the cutting head passed the monitoring points.





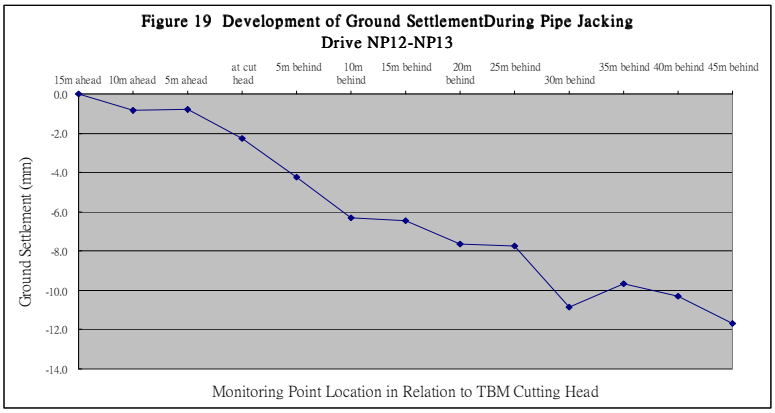
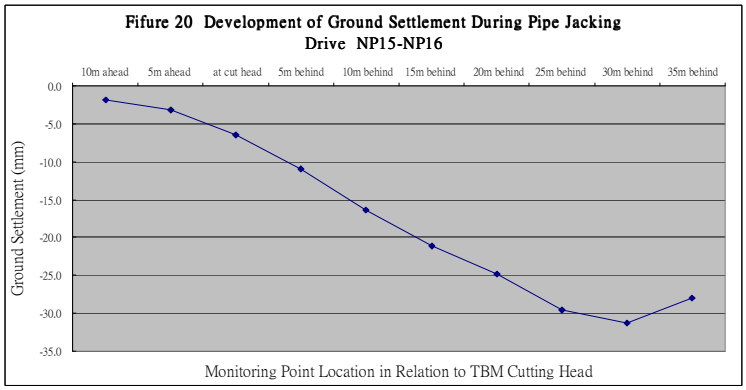


Figure 19 Development of Ground SettlementDuring Pipe Jacking Drive NP12-NP13



Fifure 20 Development of Ground Settlement During Pipe Jacking Drive NP15-NP16

DISCUSSION ON MONITORING DATA

Jacking Resistance

The recorded jacking forces suggest that the TBM cutting head resistance ranged from 30 t to 80 t. Up to total jacking resistance of 560 t was recorded for a pipe-jacking length upto 225 m. However, the jacking resistance increased significantly if a boulder or corestone zone was encountered during pipe-jacking. The interpreted average friction resistance on the jacking pipe was generally less than 5 kN/m^2, being at lower bound of the empirical value commonly adopted in the pipe-jacking design practice.

With lubricants, the jacking resistance can be greatly reduced. The total jacking force recorded during the pipe-jacking at drives WC2-WC4 and WC5-WC4 remained unchanged after the bentonite lubrication became fully effective. Jacking in a curved alignment (450 m radius) did not cause any significant increase of the jacking resistance.

It has also been observed that higher friction resistance was often associated with larger ground settlements. Friction resistance on the jacking pipes is developed quickly once the ground moved/collapsed on to the pipe during jacking. It is therefore important to maintain sufficient slurry pressure at both tunnel face and the tunnel bore throughout the pipe-jacking work. It is equally important to prevent any long stoppages during pipe-jacking to avoid any jacking difficulties.

Ground Settlements

From the above case studies, it was found that the ground settlements associated with

the pipe-jacking work in a general loose to medium dense sandy ground at both shallow and relatively deep depth are less than 25 mm for majority of the drives. Local larger settlements were also recorded, probably due to ground variations and workmanship involved during jacking work. Large settlement recorded around the rescue shaft near NP11 was clearly due to the hard and prolonged driving at that area as a result of an obstruction being encountered.

Comparison of the Observed and Predicted Ground Settlements

Empirical methods adopting an error function curve generally predict a narrow but relatively deep ground settlement trough in the case of shallow tunnelling. However, small ground settlements but having a wider settlement trough is often predicted for a deep tunnel using such method. By comparing the predicted with the observed ground settlements associated with the pipe-jacking carried out here, current empirical method overestimated the settlements under the shallow tunnel (drive WC2-WC4 and WC5-WC4) while they may underestimate the associated settlements under deep tunnels. The recorded ground settlements immediately after completion of the pipe-jacking work were all smaller than the predicted values except for drive NP15-NP16. After one year the settlement increased to 90%, 125% and 200% of the predicted respectively for deep tunnels NP12-NP13, NP12-NP11 and NP15-NP16. However the observed ground settlement associated with the shallow drives were approximately 30% only of the predicted values.

The estimated transverse distribution of the ground settlements matches with the observed reasonably well though significantly overestimating the settlements directly above the settlements for a shallow tunnel (Figures 12 to16). The recorded settlements at the left and right side monitoring points generally exceeded the predicted values suggesting a slightly flatter settlement trough. The observed ground settlement trough along the drive NP12-NP13 matches best with the predicted transverse distribution of the settlements. Settlement of 17 mm was predicted for jacking drive NP15-NP16 while over 50 mm of ground settlement was recorded, probably as a result of TBM tunnelling through an existing old seawall.

Observed Long Term Ground Settlements

Few data on the “long term” ground settlements associated with pipe-jacking have been reported in the literature. The project reported here provides a valuable opportunity for investigating the time effects. By comparing the recorded ground settlements along the jacking drives that occurred immediate after completion of the jacking with the observed values after 1 to 2 years after jacking, the development of the ground settlements with the time can be studied. The observed settlements suggesting a quick development of the settlements under a small overburden (shallow tunnel) with a 100% or more increase of settlements recorded (tables 1 & 2). The heavy traffic loadings caused full closure of overcut of the tunnelling bore onto the jacked pipes in a short period of time under a shallow tunnel.

However, the ground settlements continued over a much longer time, generally taking over a year to stabilize for tunneling under a relatively deep depth even in general sandy ground conditions. The further ground settlements along the central line of the drive are small, being less than 35% of the initial settlements. Longer time observed for developing settlements in deep pipe-jacking is considered to be mainly due to the gradually dissipation of the excess pore pressure after pipe-jacking.

Development of Longitudinal Ground Settlements

Figures 17 to 20 show the development of the observed longitudinal ground settlements

during the pipe-jacking for drives WC2-WC4, NP12-NP11, NP12-NP13 and NP15-NP16. It can be seen that the ground settlements occurred at 0.5 H ahead the TBM head were negligible. The settlements recorded immediate above the tunnelling face were also small, being less than 8 mm in all cases. However, the settlements developed quickly after the TBM pass the monitoring point, and still continue at a distance of 3 - 4 times of the tunnel depth behind the face. It is also interesting to note that the observed settlement development here is different from what was reported by Lake (1992) in which the settlements are expected at the ground surface at a distance of 1 - 2 times of the tunnel depth ahead of the tunnel face and 80% to 90% to be completed at a similar distance behind the face.

CONCLUSION AND RECOMMENDATION

This paper documented case studies on 6 numbers of pipe-jacking works. Jacking resistance developed and the associated ground settlements during the pipe-jacking and upto 2 years after tunnelling have been discussed. The reported pipe-jacking projects involved shallow and deep jacking works with different jacking-pipe diameters within either general Fill, loose Marine and Alluvium or a dense CDG ground conditions. Both manual excavation using an open shield and bentonite slurry TBM were adopted.

Based on the observed ground response to the pipe-jacking works, following observations may be established :

(a) With a reasonable workmanship, ground settlements associated with the pipe-jacking works can be controlled within 25 mm for a typical ground conditions in Hong Kong. However, tunnelling through obstructions such as buried seawall using TBM could cause significant ground settlements.

(b) Empirical prediction methods such as the error function curve (Peck, 1969) likely lead to overestimates the ground settlements under a shallow tunnel while giving a more reasonable prediction for a deep tunnel.

(c) Compared with deep tunnelling, more further ground settlements should be expected under a shallow tunnel and developed quickly in a generally sandy ground. The settlements associated with shallow pipe-jacking should be largely completed within one month.

(d) Settlements ahead of the tunnelling face are small, and the settlements developed quickly after the cutting head pass the monitoring point in a generally sandy ground.

(e) The jacking resistance developed during the pipe-jacking mainly depends on the addition of lubricants and ground conditions. An effective lubrication would reduce the friction to a negligible value, though obstructions such as boulders/corestones increase the cutting resistance significantly.

(f) Empirical values for estimation of the friction resistance are conservative, and a lower bound value of the recommended values (5 kN/m^2) should be sufficient for commonly encountered ground conditions in Hong Kong.

(g) Higher friction resistance is often associated with lager ground settlements, hence it is important to maintain sufficient slurry pressure around the tunnel bore to avoid early closure of the designed over cutting.

Finally, to avoid long delays and very cost remedial works, it cannot be over emphasized that a thorough understanding of the ground conditions and investigation of possible artificial obstructions along the drive are required. Contingency plans should be carefully proposed to cater for possible existence of such obstructions, such as buried piles or existing old seawalls

often encountered in a well developed urban environment. Sensible engineering judgment, practical and quick decisions must be made based on monitoring data, site conditions, and excavation technique adopted.

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GROUND INVESTIGATION FOR TUNNEL WORKS - A CAUTIOUS APPROACH

Pak-cheung Patrick Wong[1]

Abstract

Detailed technical guidance on ground investigation for tunnel works is readily accessible. This Paper provides an alternative thought of tackling this issue by linking it with the wording in the contract documents. The need to conduct comprehensive ground investigation during the design stage of tunnelling projects is not something to be queried. Given the inherent uncertainties in the ground conditions, it is very often hard to draw a very distinct line on what amounts to ″adequate″ ground investigation, especially for deep and long tunnels located in difficult ground conditions. If the actual ground conditions encountered during tunnel excavations are substantially different from those assumed in the design, a different scheme of stabilization measures, possibly with a higher cost and longer construction time, will be necessary. The Paper discusses the appropriateness of relying on the ″Site Inspection Clause″, the ″Disclaimer Clause″ and the ″Physical Impossibility Clause″ in the contract documents when unexpected ground conditions are encountered. No matter how the contract documents have already been specifically drafted in allocating the risk associated with the ground conditions, the ″foreseeability″ at the date of contract is always contentious whenever arguments arise during construction. The Paper focuses on how the courts and legislature interpret the ″adequacy″ of ground investigation in light of some commonly adopted contractual provisions in tunnelling projects. By construing the clauses in relation to ground conditions and understanding relevant technical factors attributed to in a contractual claim, we could have another interesting picture from a project administrator′s perspective of the required ground investigation for tunnelling works.

INTRODUCTION

There are inherent uncertainties in the ground conditions for tunnelling projects prior to construction, regardless of the nature and extent of ground investigation (GEO, 2005). Detailed technical guidance on ground investigation for tunnelling works is readily accessible. This Paper provides an alternative thought of the required extent of ground investigation to be conducted during the design stage of tunnelling projects by linking the issue with the ″Site Inspection Clause″, the ″Disclaimer Clause″ and the ″Physical Impossibility Clause″ in the contract documents. Such consideration is a slight departure from the established practice for a designer, but it is to be hoped that a fresh insight of this matter from a project administrator′s perspective will facilitate a more thorough appreciation of all the relevant

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technical guidance on ground investigations for tunnelling works.

SITE INSPECTION CLAUSE

It is common to include a ″Site Inspection Clause″ in the Conditions of Contract of a tunnelling contract such that the Contractor shall be deemed to have examined and inspected the site before entering into the contract. Could a properly drafted ″Site Inspection Clause″ in the contract documents be relied on when unexpected ground conditions are encountered?

Even though the ″Site Inspection Clause″ itself is a deeming provision, the tenderers might not be able to afford to take the necessary time, obtain access and deploy resources to arrange their ground investigation works to verify the accuracy of the records of those investigations carried out by the designer. Reasonably experienced tenderers have good reasons to rely on the results of ground investigations conducted by the designer in determining the appropriate tunnel excavation method and working sequence, identifying potentially problematic areas along the tunnel alignment and assessing the geotechnical risks.

Based on the ″Site Inspection Clause″ alone, it appears to be difficult to reasonably expect the Contractor to have adequately carried out his own exanimations and inspections. The ground investigation records would normally be largely relied on by the aggrieved party (probably the Contractor). As the subsequent arguments regarding extension of time and monetary claims put forward by the aggrieved party would have to depend on the wording in the relevant clauses in the Conditions of Contract when unexpected ground conditions are encountered, there is a direct relationship between what amounts to adequate ground investigation records and the methodology for assessing the claims. This reinforces the critical need for a designer to understand the Conditions of Contract of the tender documents and take a project administrator′s perspective in the planning of the extent and details of the ground investigation works.

There are no fast and straightforward guidelines detailing how a designer should proceed so as to discharge his responsibility properly and avoid any possible contractual disputes. More discussion with the project administrator should preferably be made. Moreover, a designer is strongly encouraged to go through the tender documents so as to appreciate the procedure and criteria for handling of claims once unexpected ground conditions are encountered.

Usually, the ″Site Inspection Clause″ is followed by a ″Disclaimer Clause″, under which the liability of the party preparing the Contract is attempted to be restricted.

DISCLAIMER CLAUSE

A ″Disclaimer Clause″ may be included in the Conditions of Contract of a tunnelling contract such that the Contractor shall not be allowed to be relieved from any risk or obligations, or to claim for additional payment when the ground investigation records available at the date of the contract are subsequently found to be insufficient during construction. Could a properly drafted ″Disclaimer Clause″ in the contract documents be relied on when unexpected ground conditions are encountered?

This kind of ″Disclaimer Clause″ excluding the liability for misrepresentation of the

ground conditions may be subject to the statutory requirement of reasonableness in accordance with the Misrepresentation Ordinance (Cap 284, Laws of Hong Kong), hereinafter referred as the ″MO″, and the Control of Exemption Clauses Ordinance (Cap 71, Laws of Hong Kong), hereinafter referred as the ″CECO″. The provisions in the MO are especially important for the construction industry because the burden of proof of misrepresentation does not lie with the aggrieved party (probably the Contractor); it is the offending party (probably the project administrator and/or the designer), pursuant to the MO, that is obliged to justify that the ″Disclaimer Clause″ in the contract documents is reasonable (Section 3(1) of the MO).

The offending party has to prove that he had reasonable grounds to believe, and did believe up to the time the contract was made, that the represented ′facts′ were true (Section 3(1) of the MO). A ″Disclaimer Clause″ in the contract documents of a tunnelling contract excluding or restricting liability for misrepresentation is of no effect unless it satisfies the requirement of reasonableness, as stated in Section 3(1) of the CECO (Section 4 of the MO).

What should be the reasonable extent of ground investigation records available at the date of a tunnelling contract so that the offending party could rely on the ″Disclaimer Clause″ when unexpected ground conditions are encountered?

The requirement of reasonableness for the purposes of the CECO and Section 4 of the MO is satisfied only if the court or the arbitrator determines that the term ″was a fair and reasonable one to be included having regard to the circumstances which were, or ought reasonably to have been, known to or in the contemplation of the parties when the contract was made″ (Section 3(1) of the CECO).

In view of the above statutory provisions, in order to rely on the ″Disclaimer Clause″ in a tunnelling contract, a key factor to be taken into consideration is whether or not the ground investigation records available when the contract was made are fair and reasonable. A test of fairness or reasonableness is an objective test, under which individual differences in handling the same matter would unlikely be considered. The work by the offending party needs not necessarily meet the highest international standards, but one is not entitled to make some kinds of discretionary or subjective decisions in determining the extent of the ground investigation works, and he should follow what other reasonable professionals would have done in handling the same task. Differences in working experiences and educational and technical background, etc. with respect to that of a reasonable counterpart in the industry would not be considered in the test of reasonableness. Hence, in examining the sufficiency of the ground investigation records once a dispute on the appropriateness of reliance on the ″Disclaimer Clause″ arises, a cautious approach that should be considered is not only to focus on the technical elements of the ground investigation records, but also to apply a test of reasonableness as laid down in the CECO. The fundamental and standard question to be asked is whether a reasonable veteran professional in the respective field would carry out a similar extent and nature of the ground investigation works for the same project before entering into a contract.

As the requirement of reasonableness in the CECO is similar to that in the Unfair Contract Terms Act 1977, an English case casts new light on how the court would deal with the exclusion clause in the contract and apply the test of reasonableness. In *The Chester Grosvenor Hotel Co. Ltd. v Alfred McAlpine Management Ltd. and others [1991] 56 BLR 115*, a UK Court listed the following six factors as the test of reasonableness, as stipulated in

Section 11(1) of the Unfair Contract Terms Act 1977:

(a) Were the parties of equal bargaining power?
(b) Was the contract ″take it or leave it″?
(c) Was the division of risks reasonable?
(d) Did the aggrieved party have a proper opportunity to consider the terms before accepting them?
(e) Was it right that the offending party should contract out of liability for his expertise?
(f) Was the existence of insurance relevant?

The factors mentioned in (c) and (d) above are directly relevant to our discussion on the adequacy of the ground investigation records available at the time of entering into a tunnelling contract.

In considering the aspect of ″proper opportunity″, one could review whether the aggrieved party has the necessary time and access before entering into the contract to examine the ground investigation records provided by the offending party. Perhaps one could also check whether the aggrieved party has obtained professional advice either from independent sources or from other companies within the group of companies of which the aggrieved party is a member, when affirming the aggrieved party has a ″proper opportunity″ to study the ground investigation records before entering into the contract.

Furthermore, we have to study whether the ground investigation records made available before entering into the contract is adequate enough to ensure a ″reasonable division of risks″. As the assessment of the reasonableness of the division of risks is again an objective test, individual differences in handling the same matter would unlikely be entertained. The standard question of whether a reasonable professional in the respective field would allow such division of risks for the same project is of relevance.

A UK court pointed out that, in determining the reasonableness of an exclusion clause in a contract, one has to consider the reasonableness for the offending party as well as the reasonableness for the aggrieved party (*British Fermentation Products Ltd. v Compair Reavell Ltd. [1999] BLR 352*). It is always easy and natural to put forward the argument that an exclusion clause in the contract is unreasonable. In the commercial world and especially in the construction industry where standard forms of Conditions of Contract are commonly used, however, the court seems to consider that it would be difficult to exclude the insertion of an exclusion clause, otherwise infinite or unlimited causes of action or remedies might then likely be available to the aggrieved party against the offending party. The court took the view that the inclusion of an exclusion clause in the contract provided an element of reasonableness in the contract. To tie in with this judicial viewpoint, it would seem that when one is dealing with the ″Disclaimer Clause″ in a tunnelling contract, the prejudiced presumption that this clause must be an unreasonable one should firstly be abandoned.

PHYSICAL IMPOSSIBILITY CLAUSE

A ″Physical Impossibility Clause″ may be included in the Conditions of Contract of a tunnelling contract such that the Contractor is required to execute the works in strict accordance with the contract unless it is physically impossible. To date, a number of cases regarding the enforceability of the ″Physical Impossibility Clause″ in construction contracts

have already been dealt with in Hong Kong, the results of which, however, are rarely published.

A well-known English case in the late 1970s on the problem of what would amount to a physical impossibility in construction contracts (*Turriff Ltd. v Welsh National Water Development Authority [1979]*) is the starting point of our discussion. The case involved the construction of rectangular pre-cast concrete segments of a sewer to specific tolerances. The ruling came somewhat as a surprise. The court ruled that a strict interpretation of this term as an absolute impossibility might not be appropriate and it should be construed in a commercial and practical sense. The court′s judgment was ground-breaking.

The above judgment deserves specific attention, although the logic may be hard to fathom at a first glance. It turns out that there are at least two implications to tunnelling projects when extreme difficulties are encountered or extraordinarily unexpected and special circumstances occur. Firstly, the Contractor in a tunnelling contract should be relieved of carrying out those works that are actually physically impossible to be performed in a commercial sense. Secondly, the Contractor in a tunnelling contract should also not be required to carry out those additional works which are instructed by the project administrators through variation orders if those works instructed are physically impossible to be carried out in a commercial sense.

The Contractors might perhaps still be willing to take up those works in certain circumstances, for example, upon mutual agreement amongst all parties with the works being paid on a *quantum meruit* basis, which is based on a fair commercial price.

The court had, regrettably, not elaborated on the issue further. It may, however, be reasonably presumed that the interpretation of a physical impossibility depends very much on the locality of the construction works involved. An extreme difficulty encountered or an extraordinarily unexpected and special circumstance arisen locally which is commercially impossible to be dealt with by the Contractors might, as expected, *prima facie* not be so when the same scenario is encountered in other countries. Technology and methods of working normally adopted and the expertise level commonly available in the construction industry in a particular place in dealing with the construction works in that locality are factors to be considered in the interpretation of the above term. In Hong Kong, which is renowned worldwide for the successful construction of various large-scale tunnelling works, the above factors would not be as simple and straightforward as one might expect and must be treated and applied with due care.

What should be the reasonable extent of ground investigation records available at the date of a tunnelling contract so that the parties of the contract could rely on the ″Physical Impossibility Clause″ when unexpected ground conditions are encountered? To address the root of this problem, one should re-examine how the tentative equipment/processes, construction methods, working sequence, etc. originally contemplated actually depended on the ground investigation records made available at the date of a tunnelling contract.

After all, in order to rely on the ″Physical Impossibility Clause″ in a tunnelling contract, the heart of the matter is whether any additional works incurred, which arise out of the discrepancy between the underground conditions actually encountered and those interpreted based on the ground investigation records available at the date of a contract, are physically possible to be carried out in a commercial sense. In scrutinizing the adequacy of the ground

investigation records, a cautious approach is to go an extra mile in thinking through fully and frankly as to whether the alleged ″inadequate″ ground investigation records have any significant adverse effects on the adopted construction methods, working sequence, etc.

AN EXAMPLE

In the famous Hong Kong case *Mitsui Construction Co. Ltd. v The Attorney General of Hong Kong [1984] 26 BLR 113*, the claimant, Mitsui Construction Co. Ltd., was engaged by the Government of Hong Kong in the late 1970s to construct a pipeline and tunnel system from the west of the New Territories to Plover Cove to facilitate the fresh water supply from East River to Hong Kong. During the tunnel excavation, there were substantial changes in quantities of the length of the tunnel lining and the excavation works and the claimant therefore needed extra time to ″cope with ground conditions in executing the tunnel excavation and lining works″. For example,

Item	As Billed	Rate	As Executed
E108 (lining type 1A with support)	275 m	HK$ 4,050	2,337.105 m
E301 (provide and fix as temporary support steel ribs, etc.)	40 tons	HK$ 4,000	2,943.681 tons
E325 (lining)	275 m	HK$ 1,538	2,448.20 m

The claim was first considered in arbitration and three main questions regarding the compensation and variation orders to cope with the ground conditions were disputed. The claimant appealed against the arbitrator′s decision to the Court of Appeal of Hong Kong. The claimant then appealed against the Court of Appeal′s judgment to the Judicial Committee of the Privy Council in England, which was the final appellate court under Hong Kong law before 1997.

In the Judicial Committee of the Privy Council, Lord Bridge of Harwich stated that ″it is common ground that, however thorough the pre-contract site ground investigations, the nature of the ground through which the tunnel was to pass could not be predicted accurately in advance but would only be discovered as excavation proceeded″ (*Mitsui Construction Co. Ltd. v The Attorney General of Hong Kong [1986] 33 BLR 1*). He went on to point out that ″tenderers gambling on favourable ground conditions (in this contract) would risk a large loss, while conversely, if all tenderers anticipated the worst case, but in the event reasonable conditions were encountered, the Government would be the losers ….. if the Government are right, there is a large element of wagering inherent in this contract″. He said that ″it seems to their lordships somewhat improbable that a responsible public authority on the one hand and responsible engineering contractors on the other hand, contracting for the execution of public works worth many millions of dollars, should deliberately embark on a substantial gamble″.

In the above case, the appeal was allowed. It is noteworthy that the Privy Council had use the wording such as ″gambling″and ″wagering″ in their judgment in discussing the

inherent uncertainties in the ground conditions in a tunnelling project. The court understands the importance of pre-contract ground investigation for tunnelling works. A designer must be cautious in determining the nature and extent of ground investigation in avoiding the recurrence of ″a substantial gamble″.

CONCLUSIONS

In the delivery of a tunnelling project, the wording in the contract documents is paramount. Designers of the ground investigation work of tunnelling projects need to be involved in the contentious contractual issue when the actual ground conditions encountered during construction are substantially different from those assumed in the design. In dealing with the argument of the adequacy of the ground investigation records, a cautious approach is to examine the matter also from a project administrator′s perspective and to go though the wording of some relevant clauses in the contract documents.

In the author′s opinion, the Contractor should not be blamed for not carrying out their investigation to testify the ground investigation records prior to entering into a tunnelling contract. Whether the amount of the ground investigation records is adequate or not depends on the methodology adopted to handle and assess claims as laid down in the Conditions of Contract. There is limited scope for a designer to change the Conditions of Contract of a tunnelling contract to limit his liability. Day-to-day handling of contractual claims arising from unexpected ground conditions is normally the duty of the project administrator, but it is about time that preconceptions have to be changed. A designer should have a sufficiently clear picture of how a project administrator delivers the project upon arguments arise when unexpected ground conditions are encountered.

To rely on the ″Disclaimer Clause″, one has to consider whether the ground investigation records available at the date of the contract are reasonable or not. A test of reasonableness is an objective one and individual differences in handling the same matter should not be taken into account. Other than re-visiting all the relevant technical guidance on ground investigations for tunnelling works, it would be prudent to also adopt the test of reasonableness as laid down in the CECO.

On the face of it, the ″Physical Impossibility Clause″ could be adopted once something deemed to be physically impossible has to be executed. The court has construed ″physical impossibility″ in a commercial and practical manner. When the expected ground conditions interpreted from the ground investigation records are so different from the actual ground conditions that it affects the parties in a commercial sense, it would have a reasonable ground to trigger an argument on physical impossibility.

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GROUND FREEZING FOR CROSS PASSAGE CONSTRUCTION BENEATH AN ENVIRONMENTALLY SENSITIVE AREA

Roger B. Storry[1], Benjamin Kitzis[1], Olivier Martin[2], David Harris[3] and Andrew Stenning[4]

Abstract

The Kowloon Canton Railway Corporation Sheung Shui to Chau Tau tunnels form part of the 7.4km Spurline connecting Hong Kong with a new crossing to Mainland China at Lok Ma Chau. The project comprises the design and construction of 3.2km of 8.75m excavated diameter twin bored tunnels using a shield Tunnel Boring Machine (TBM) and approximately 2km of cut and cover tunnels.

Cross passages were required to be constructed between the main running tunnels, with three of these located beneath an environmentally sensitive wetlands area called "Long Valley". Access from the surface for ground treatment was prohibited in the wetlands and as a result ground freezing was adopted to stabilise the ground and enable safe construction of the cross passages to be carried out through residual soils of volcanic rock 20m below the ground surface and the water table.

This was the first time that such works had been carried out in Hong Kong and a rigorous approach to both design and construction was implemented throughout to ensure that all risks were identified and appropriately mitigated.

INTRODUCTION

The Kowloon Canton Railway Corporation (KCRC) is constructing the 7.4km Lok Ma Chau Spurline in the northern New Territories of Hong Kong to form a second border crossing to Mainland China (Figure 1). The Spurline was originally envisaged as an elevated viaduct, however in order to protect an environmentally sensitive wetland area, the first 5.2km was changed to bored tunnels with lengths of cut and cover approaches at either end of the alignment and a station box in the central section.

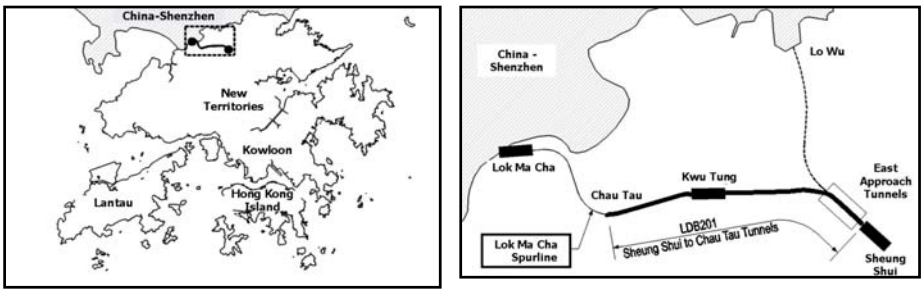

Figure 1. Location Plan

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[1] Dragages (HK) Limited, [2]Bouygues Travaux Publics, Paris, [3]Geotechnical Consulting Group, London, [4]Atkins China Limited

Dragages HK Joint Venture (DJV) comprising Dragages (HK) Ltd and Bouygues Travaux Publics, was commissioned in October 2002, under contract, 'LDB201 – Sheung Shui to Chau Tau Tunnels', to design and construct 3.2 km of 8.75 m excavated diameter (7.625 m i.d.) twin bored tunnels and approximately 2km of cut-and-cover tunnels.

The project's fire evacuation strategy required cross passages every 240 m between the two running tunnels to allow evacuation in the event of an incident. A total of 14 connection adits were formed in ground conditions varying from fresh rock to completely decomposed rock. Ground treatment was required for safe excavation. For 11 of these adits ground treatment was carried out from the surface and comprised jet grouting, tube-à-manchette injection grouting, full ground substitution and dewatering as applicable to the prevailing ground conditions. At the remaining 3 adit locations, namely CP2, CP3 and CP4, the Project Environmental Permit precluded any works from being carried out from the ground surface in the environmentally sensitive Long Valley wetlands area, hence ground freezing was selected as a suitable ground treatment technique and was carried out from within the tunnels.

GROUND CONDITIONS

The ground conditions along the tunnel alignment comprised superficial deposits overlying Volcanic Tuff, in various stages of weathering. A simplified geological profile for the project is given in Figure 2. A detailed study of the ground profile at each cross passage was carried out to accurately define the strata present, together with identifying any variations in the geotechnical properties of the decomposed volcanics. Macroscopic and microscopic analysis, in-situ testing, mineralogy, grading, plasticity and tri-axial testing were carried out to classify the soils and define the construction risks and efficient treatments.

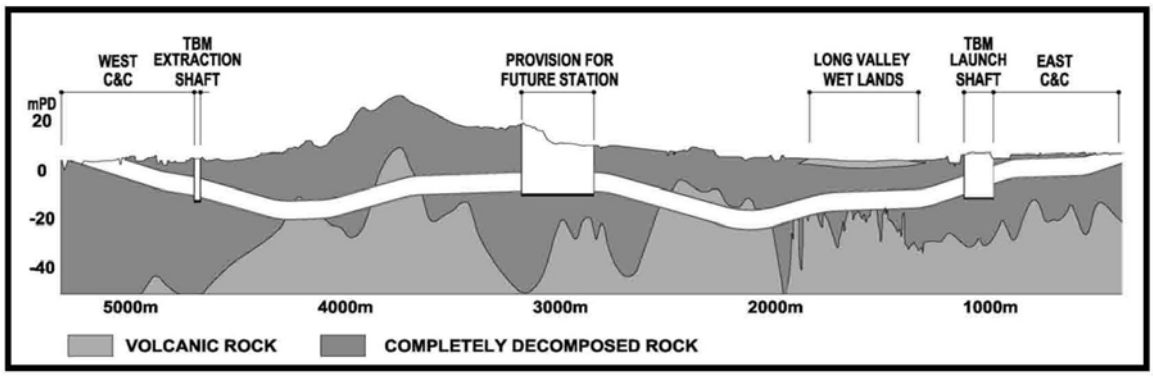

Figure 2. Simplified Geological Profile

DESIGN AND PREPARATORY WORKS

The detailed design of the ground freezing was carried out by the design department of Bouygues Civil Works with expert advice and technical review provided by Geotechnical Consulting Group of London and Bachy-Soletanche in France.

The ground freezing was to be carried out during the construction of the second running tunnel, which imposed significant constraints on methods of working. An iterative approach to the design had to be adopted, involving the designer, the subcontractor (a joint

venture between Intrafor HK and Bachy-Soletanche), the design checker and the production teams, to achieve a robust solution which was acceptable to all parties involved. The design phase included testing of frozen soil samples, undertaking sensitivity analyses of the design to optimise the freeze pipe layout and the definition of the instrumentation and monitoring regime. In addition a detailed risk assessment was undertaken for the ground freezing works to ensure all the key risks were identified early in the process and appropriate mitigation measures were planned and implemented in each case to manage these risks.

Some of the critical aspects to be addressed during design stage were:

(a) limiting the duration of the freeze - targeted to be 4 weeks for programme reasons;
(b) accommodating the requirement to install freeze pipes only from one tunnel;
(c) ensuring efficient connection of the freeze envelope to the running tunnels;
(d) minimising the number of freeze pipes;
(e) allowing for potential deviation of drilling from the theoretical design spacing;
(f) minimising the impact on the adjacent ground and sensitive receivers including the running tunnel lining;
(g) minimising the impact on activities on-going in both tunnels, in particular tunnel boring; and
(h) defining design loadings for temporary and permanent cross passage support.

There are no known case records of freezing of residual volcanic soils so to assist in the selection of design parameters laboratory testing was carried out on a number of representative samples. The specialist frozen soil testing was carried out at the University of Alberta in Canada and included the determination of strength, stiffness, salinity, mineralogy, creep, frost heave and thaw strain. Testing followed procedures developed by the University and were in line with the ASTM specification for testing frozen soils. A design instantaneous strength of 2.4MPa with an initial secant modulus of 200MPa was adopted. Very low creep and heave were recorded under the prevailing overburden pressures.

A number of numerical simulations were carried out to define both the thermal and mechanical behaviour of the frozen ground. From this the time to form the frozen annulus, the deviatoric stresses in the frozen soil, the stresses in the temporary support and the loading applied to the main tunnels were determined.

Based on the results of the numerical analyses the freezing pipe layout was defined with a 32 pipe pattern to be installed from the first running tunnel and surface cooling of lining in second running tunnel. The thermal instrumentation requirements, used to monitor the development of the freeze and its status during the excavation period, were also defined together with “Hold Points” where the design team, the site team and the independent checker were to review the monitoring data to confirm its consistency with the design predictions. To cater for an unexpected reaction of the ground to freezing and as an additional security to protect the running tunnel segment circular support frames were erected in the main tunnels at each cross passage location.

Given the specialist nature of this work, comprehensive independent checking of the ground freezing design was undertaken by Atkins. The aim of the independent checking was to ensure that the ice ring formed during the ground freezing was capable of acting as temporary support for the protection of the works during the cross passage construction, that the integrity of the running tunnels was not compromised during the ground freezing and that impacts to the adjacent ground and any existing facilities were minimised. For the design

checking a 2D plane strain model was created using FLAC Version 4.0 with thermal and mechanical simulation capabilities.

THE FREEZING PHASE

Installation of the freeze pipes was on the critical path as drilling was only able to commence once the TBM had passed each cross-passage location in the second, parallel running tunnel. Drilling and installation of the "access casing" was carried out from a specially fabricated self-elevating drilling platform (Figure 3) under the protection of a blow-out preventor (Figure 4). Each casing was installed to touch the grout surrounding the second running tunnel to minimise the risk of a poor connection between the frozen ground and the tunnel lining. The precision of drilling taken into account during the design phase was 1% of the drilled length which proved to be consistent with the as built installation achieved.

Figure 3. Self-elevating Drilling Platform

Figure 4. Blow-out Preventor

Three 94kW freeze units, using ammonia as the primary refrigerant, were installed inside the tunnel midway between CP2 and CP3. The three freeze units operated in parallel cooling the secondary refrigerant, a solution of calcium chloride, and feeding it to a central cold brine tank (target brine temperatures -30 to -35 ºC). The use of a centralised cold brine tank ensured that an uninterrupted supply of cold brine was available in the event that one or even 2 freeze units malfunctioned. This system addressed one of the major safety risks identified: interruption of freezing during excavation. Each cross-passage was supplied with cold brine by independent piping which was insulated to minimise temperature losses. The circulation of the brine within the ground was via freeze tubes composed of an inner tube of 40mm diameter and outer tube of 60mm diameter which were grouted into the access casing.

The primary freeze of the cross-passages lasted 5 to 7 weeks, the maintenance phase (excavation and construction phase) lasted 6 weeks and the thaw period was approximately 8 weeks. These times were generally consistent with those estimated by the numerical simulations.

INSTRUMENTATION AND MONITORING DURING FREEZING

The freezing process was controlled using a comprehensive instrumentation scheme monitoring temperatures, stresses, pressures and movements as part of the observational

approach. Over 500,000 data points were recorded and interpreted with respect to the numerical simulations during the works.

At each cross passage thermocouples were installed in tubes drilled between the two running tunnels. Two tubes were installed parallel to the freeze pipes at the design position of the design 0ºC isotherm and two were inclined across the ice wall. Temperature measurements were also made in the tunnel linings since the connection between the ice ring wall and the tunnel lining was expected to be the most critical location. In addition, monitoring of the supply and return brine temperatures, plant operating temperatures and ambient air temperatures was carried out.

Piezometers were installed both inside and outside the ice ring wall and deformations and induced stresses in the tunnel support frames were also monitored. The ground surface above the treatment zone was also closely monitored using survey techniques to record any settlement / heave associated with the ground freezing works.

Figure 5 illustrates the temperature measurements taken along one of the tubes installed at the location of the design 0ºC isotherm and compares these measurements with the range predicted by the "best-estimate" plane strain analysis, based on the as built location of the thermocouples. Also shown on the Figure is the change in pressure recorded in two piezometers, one inside and one outside the ice wall. A rise in pressure is shown on the instrument inside the ice wall about 7 days after commencement of freezing, whereas the instrument outside the ice wall shows no change. This is indicative of the closure of the ice wall trapping water inside and as the ice wall continues to grow and water expands during the phase change into ice there is compression of the trapped water resulting in an increase in pressure. To avoid damaging the developing ice wall, the pressure was controlled to between 0.5 and 1.0 bar above ambient pressure by draining water from inside the closed ice wall ring. Once the ice ring had reached its design thickness the valve was opened and the remaining trapped water was allowed to drain.

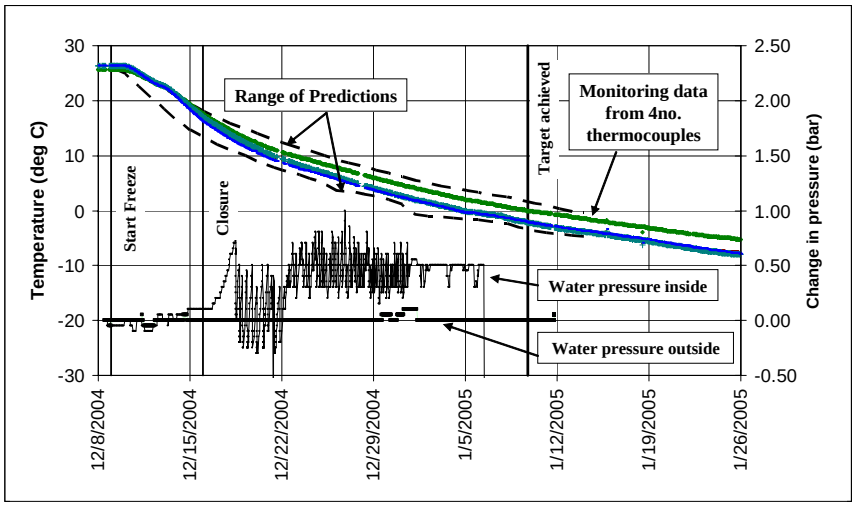

Figure 5. Temperature Measurements from Senors

The temperature measurements shown of Figure 5 are from the sensors located within the main body of the freeze. The observations from these instruments lie within the relatively narrow range of predictions obtained from the plane strain analyses and achieved the target of about 0ºC after between 4 and 4½ weeks of freezing. The two instruments at either end of

the tube are significantly influenced by heat flow from the running tunnels and were not expected to follow the same trend as the more remote sensors.

Careful consideration of the data obtained from the instruments close to the running tunnels was required in conjunction with temperature data obtained from the running tunnel linings to ensure that an adequate thickness of ice wall was formed at the connection between the ice ring and the main tunnel lining.

Figure 6 shows the distribution of temperature along a tube with distance from the first running tunnel at selected dates during the development of the freeze. The influence of proximity to the main tunnels on the end instruments is evident. The target temperature shown was determined from the plane strain analysis as a function of the distance of the tube from the cross passage axis based on as built location.

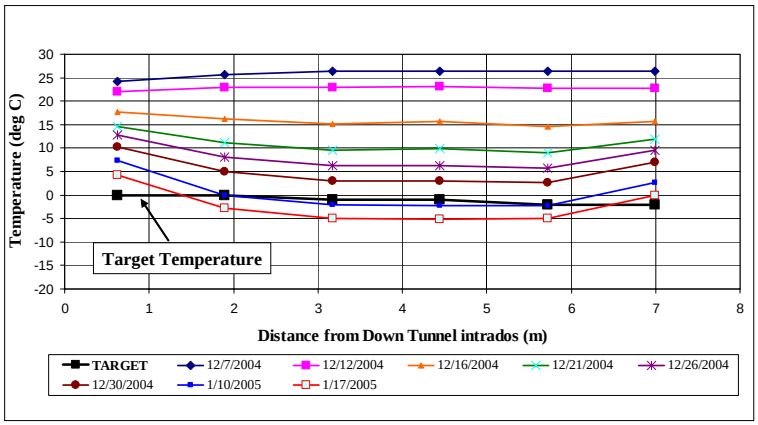

Figure 6. Distribution of Temperature Along a Tube

The observed deformations of the running tunnels were minimal and in accordance with the design predictions, based on low frost heave and creep potential. Observable movements commenced after ice ring wall closure had been attained but the rate of increase slowed steadily during the primary freeze period and stabilised to an ovalisation well within the lining capacity. Surface movements recorded above the ground freezing were negligible.

The flexibility of the ground freezing technique is demonstrated by the experience at CP4 where the conditions differed from those encountered at CP2 and CP3 in that most of the lower half of the ice wall was within intact rock rather than completely decomposed material. Drilling was rendered more difficult due to the mixed ground conditions and a far larger quantity of grout was used during installation of the access casing and the control on drilling direction was more difficult.

Figure 7 shows the divergence in behaviour for CP4 from that encountered at CP2 and CP3. A revised numerical simulation was undertaken taking into account the different thermal properties of the rock and the grout and the measured deviations of the access casing. It predicted an asymmetrical behaviour of the frozen ground and large differences between the top and bottom corners, as illustrated in Figure 8. The revised analysis together with the comprehensive monitoring allowed construction of CP4 to be completed successfully.

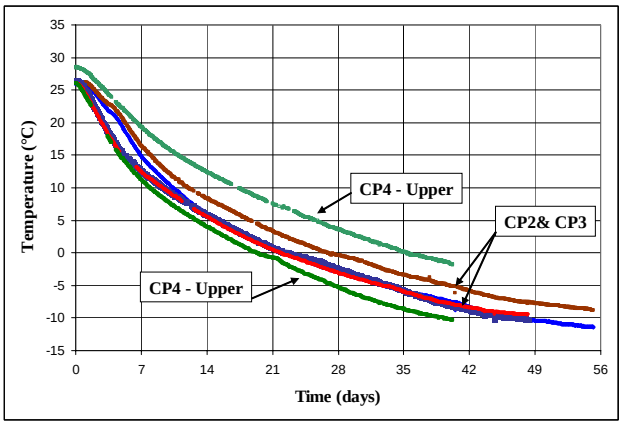

Figure 7. Temperature Variations at Adits CP2, CP3 and CP4

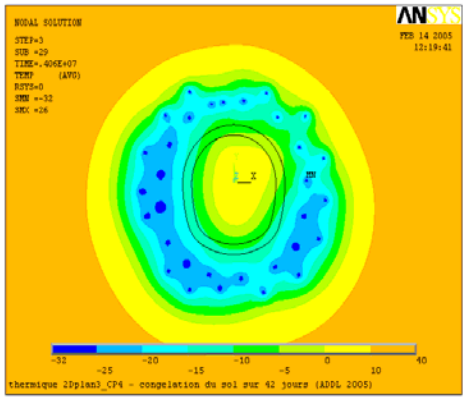

Figure 8. Asymmetric Behaviour the Frozen Ground

Overall, the instrumentation operated reliably throughout the construction phase and gave the necessary information such that the behaviour of the ground could be closely monitored and interpreted and related back to the design predictions. Detailed day-to-day interpretation by the designer, design checker and the contractor's teams allowed adjustments to be made in a timely manner.

EXCAVATION

Prior to opening the segmental lining and commencing excavation of the cross passage a steel door, which could be closed and sealed promptly, was erected over the future opening as a "final fail safe" in case of ground collapse or loss of freeze. Commencement of the excavation was formally controlled by "Hold Points" which were required to be released by a "Review Panel" comprising the designer, the independent checking engineer and the specialist sub-contractors, the contractor and the expert advisors. These "Hold Points" were released once a detailed review of all the monitoring data obtained from each treatment location had concluded that the data did not deviate significantly from the design predictions.

The cross-passage excavation was carried out 24 hours per day, 7 days a week in order to meet the programme and also to help ensure that the works area remained under permanent close supervision.

Excavation was carried out using handheld pneumatic tools (the ground had an instantaneous strength varying from 5 to 10MPa) with temporary support provided by circular arch ribs and sprayed shotcrete. Elevated temperatures within the excavation were foreseen as a critical safety issue in terms of the integrity of the frozen ground. The temperature was controlled by the erection of a plastic curtain at the entrance to the excavation to prevent ingress of warm air and frequent ambient temperature monitoring was carried out. No instability was encountered.

The permanent cross passage lining was 300mm thick sprayed shotcrete over a waterproofing membrane.

CONCLUSION

The use of artificial ground freezing on LDB201 Sheung Shui to Chau Tau Tunnels demonstrated that freezing techniques, if undertaken in a systematic and professional manner, can be a very efficient ground treatment technique adaptable to difficult ground conditions with minimal impact to the environment.

Due to the specialist nature of this type of ground treatment risks were always present and a rigorous study was required to identify all possible risks and devise adequate mitigation measures. Incidents could occur which could degrade the ice rapidly if not identified immediately. The frozen ground is water-tight and strong only during the time it is frozen and any disruption to the freezing effort can raise the temperature by as much as 10° C within a day.

Extensive instrumentation and the adoption of an "observational approach", involving suitably experienced team members, are considered essential to track the progression of the freeze in the ground and compare with the model predicted during the design phase. Such an approach enables any unexpected development of the freeze to be identified early before it could impact on the overall construction programme or the safety of the excavation.

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WATER INFLOW CHARACTERISTICS IN ROCK TUNNELS

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Abstract

Water flow in rock is dominated by rock joints or fractured zones. This represents a real challenge for design engineers to predict the amount of water inflow during tunnel construction. In this paper, the water flow measurements collected during the construction of the Eagle's Nest Tunnel were collated to shed light on the water inflow characteristics. The data together with some quantitative analyses suggest that the inflow measured during horizontal directional drilling in ground investigations is a good indicator of the actual inflow during tunnel construction. Their potential use to assess the impact on the surrounding environment, such as ground settlement and drawdown of groundwater table, is strongly encouraged.

INTRODUCTION

Water ingress during tunnel construction is one of the major concerns for tunnel designers, not only in soft ground, but also in rock. Although high quality rock is known to have very low mass permeability, the presence of faults and fissured zones with spatial variations imposes sufficient uncertainty for accurate prediction of actual water inflow into the tunnel during and after construction.

A common design approach is to assess the *in-situ* hydraulic conductivity through a Packer test (also known as Lugeon test). The measured coefficients of permeability at various locations along the proposed tunnel alignment are used to estimate the water inflow velocity assuming that Darcy's law applies. The advent of the technique of horizontal directional drilling (HDD) offers a better prediction by actually measuring the amount of water ingress in a pilot "tunnel" with a very small diameter along the tunnel alignment. This technique was adopted in the construction of the Eagle's Nest Tunnel which forms part of the Route 8. The potential loss of yield of a nearby unlined reservoir group had been a major concern. In addition to HDD, an extensive monitoring scheme was implemented, including advance probing, which identified highly permeable zones ahead of the advancing tunnel face. The actual water inflow along the tunnels during construction was also measured.

This paper details the water flow measurements prior to and after construction of the Eagle's Nest Tunnel. The intriguing relationship between water inflow in a miniature "tunnel" formed by HDD or advance probing and that in the prototype is discussed. Particular emphasis is placed on the time effect between initial and long-term water seepage into the tunnel. The merit of HDD for predicting the water inflow during construction is highlighted.

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THE EAGLE'S NEST TUNNEL

A comprehensive review of the Eagle's Nest Tunnel project, including details of the monitoring scheme, is given by Poon et al. (2006). The major focus of the present paper is placed on the water inflow measurements. On this background, the introduction of the project given below is confined to information relevant to the discussion put forward in this paper.

General Layout

The Eagle's Nest Tunnel forms part of the Route 8 which links Chep Lap Kok and Sha Tin via Tsing Yi, Kwai Ching and Lai Chi Kok. The sections comprising the North Lantau Highway and Lantau Link were opened to traffic in 1997. The Eagle's Nest Tunnel connecting Cheung Sha Wan and Sha Tin is one of the remaining sections to complete the entire high-capacity highway (Figure 1). The 2.1 km twin tunnels are designed to provide dual three-lane carriageway, with an aim to ease prospective traffic problems in other road links between the northeast New Territories and Kowloon. The South Portal of the tunnels leads to the slip roads in Cheung Sha Wan, whilst the North Portal is connected to the Sha Tin Heights Tunnel through a toll plaza. The drilled and blast tunnels are also equipped with a 400 m long ventilation adit terminating at a ventilation building off Tai Po Road.

Figure 1. Plan View of the Eagle's Nest Tunnel

Geological and Hydrogeological Conditions

The tunnels were constructed mainly in GradeII/III granitic rocks with a number of faults running across the tunnel sections. Near the south portal where the rock cover is minimal, the bedrock roughly follows the topography and is covered a layer of saprolitic soils (C-HDG). The saprolite with corestones is overlain by a mantle of completely weathered granite and a thin moderate layer of residual soil extending up to 8 – 14 m above the rockhead.

Piezometer data taken prior to construction indicated that the groundwater mainly occurred within the rock mass, with gradients generally conformed to the topography. The

water level near the south portal was located between 5 – 22 m below ground mainly within the M-SDG material, whilst groundwater near the north portal occurred within the CDG stratum.

The Eagle's Nest Mountain is characterized by heavy vegetated, undeveloped countryside. This infers that the infiltration rates during precipitation would normally be high. However, this is counter-balanced by large run-off due to the extreme gradients of the natural terrain. The Eagle's Nest Tunnel is situated in the proximity of several major Water Supplies Department's (WSD) facilities, including the Kowloon Reservoir group, which comprises the Kowloon Reservoir and the adjoining Byewash Reservoir. The reservoirs are located about 90 m above the tunnels, with a shortest horizontal distance on plan of only 90 m. These facilitates are fed by the tributary catchment area in the environs of the Eagle's Nest alignment, in addition to a concrete lined catchwater. The hydraulic connectivity between these unlined reservoirs and the drained Eagle's Nest tunnels had been a major concern.

Monitoring Scheme and Mitigation Measures

An extensive monitoring scheme was implemented in the Eagle's Nest Tunnel construction, which covered structural stability, groundwater variation, blast-induced ground vibrations and water ingress. As far as water inflow is concerned, three key pieces of information were gathered spanning from ground investigations to post-construction monitoring.

Ground investigations of the project comprised drilling of two horizontal holes with a diameter of about 75 mm, setting off from the North and South tunnel portals. The aim of horizontal directional drilling is two-fold. By positioning the drillholes along the tunnel alignment, precise ground conditions can be examined. In addition, any permeable zones with high water flux can be identified prior to excavation.

Advance probing is a complementary measure to examine water flow characteristics ahead of the tunnel face. During construction of the Eagle's Nest Tunnel, this approach was adopted and the probing distance was chosen to be 50 m alternating between left and right shoulders of the tunnels, with a short overlap. The grouting threshold was determined by seepage analysis, which corresponded to the inflow value above which loss of yield of the nearby reservoirs became an issue. A trigger level of 9 litres/min from a 25 m long probe hole persisting for 24 hours after completion was adopted. The water inflow from the tunnel face was also monitored using V-notch weirs. This measurement also formed one of the trigger criteria for grouting works. Pre-grouting needed to be applied if an inflow of greater than 36 litres/min, which persisted 24 hours for the excavated section within 25 m of the current face, was recorded.

COMPARISON OF PRE- AND POST-CONSTRUCTION WATER INFLOW

The three sets of data collected from HDD, advance probing and V-notch weirs portray a picture of the water inflow characteristics along the Eagle's Nest Tunnel at different stages of construction. In this section, results of each dataset are described and compared.

Horizontal Directional Drilling

Water flow measurements are only available in one of the two horizontal drillholes (HDD1), which sets off from the North tunnel portal. Figure 2 plots the flow rates measured

in each ~20 m section along the hole. The recorded water inflow rates are generally below 20 litres/min except for a short section near the middle of the tunnel alignment, where the horizontal drilling terminates. The results imply the very dry conditions in the rock prior to excavation. The exceptionally high water inflow near the middle of the alignment was an indicator for the presence of a permeable zone.

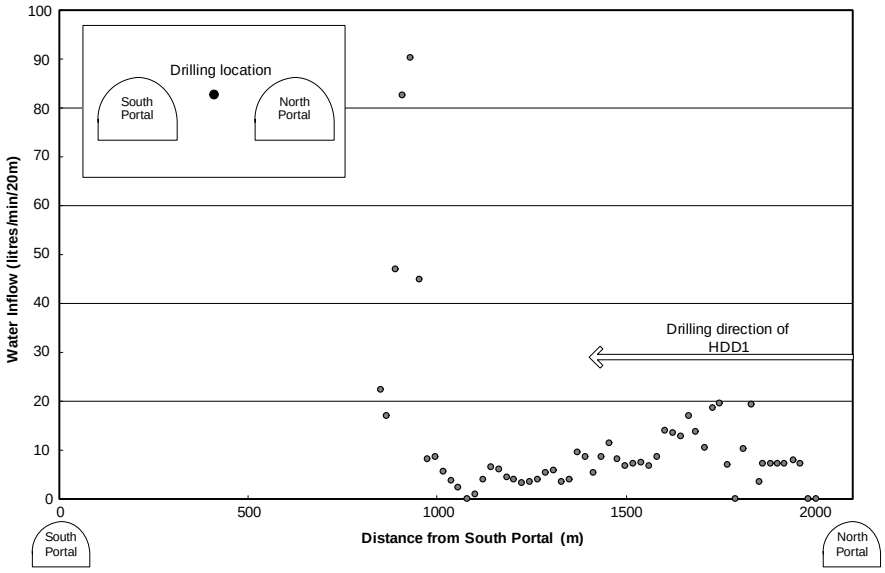

Figure 2. Measured Water Inflow During Horizontal Directional Drilling (HDD)

Probing and V-notch Weir Measurements

The rates of water inflow measured during advance probing are plotted in Figure 3. The flow rates were measured at several different locations at the tunnel face (indicated in the legend) for both the south and north bound tunnels. A glance at the data reveals that a majority of the measured water flow velocities fall below 50 litres/min, except for several locations where an unusually high inflow is encountered. These high flow rates are indicative of the presence of localized permeable zones. A comparison between Figures 2 and 3 suggests that the probing results are in good agreement with those obtained from HDD. A local zone with a high flow velocity near the middle of the tunnels was identified by both approaches. The reason for the additional permeable zones detected by advance probing, but not by HDD, could be attributed to the fact that several holes were probed at the tunnel face during advance probing which increased the chance of intercepting very contained fissured zones.

Also shown in Figure 3 are the rates of water inflow through the tunnel face interpreted from V-notch weirs which were placed at intervals of about 100 m. This data represents the water inflow patterns inside the tunnels at some point after the excavation. In order to match the dimension of the probing records, the data was scaled down by a factor corresponding to the ratio between the separations of V-notch weirs and the probing length, which is about 2 (100 m/50 m). A clear reduction can be seen with regard to the amount of water ingress before and after excavation, notwithstanding that the size of the tunnels are significantly larger than that of a probe hole. This reduction phenomenon was observed in other rock tunnels (McFeat-Smith et al, 1998 and Best & Parker, 2005). The reduction of water inflow rate can be attributed to the localization nature of the fissured zones in the rock mass. The extent of these highly permeable zones is normally limited. If an area linking the tunnel and

the water source at the far field has a lower permeability, the variation in hydraulic conductivity along the water inflow path will cause the inflow velocity to drop as the flow pattern changes from a transient state to a steady state condition. This hypothetical explanation is illustrated in detail in the next section.

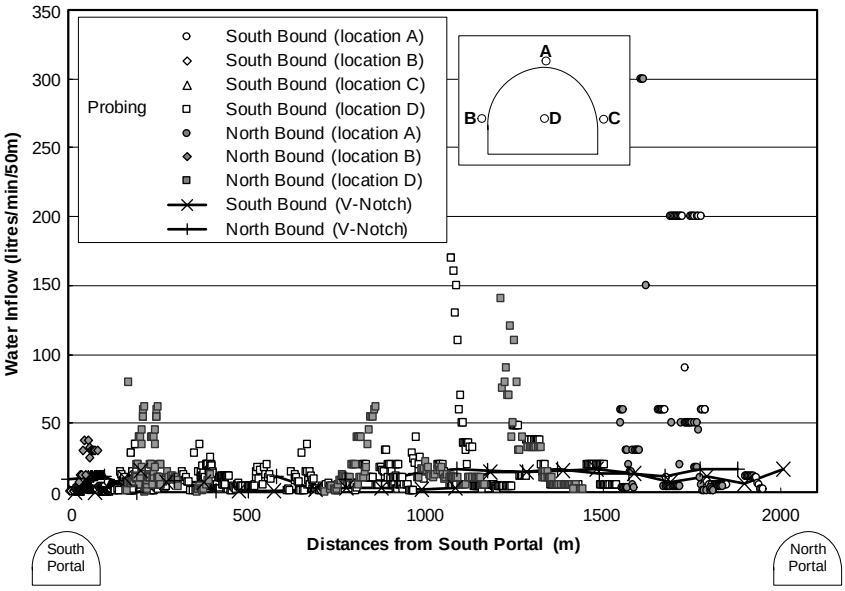

Figure 3. Comparison Between Water Inflow Measurements During Advance Probing and Within the Excavated Tunnel soon after Construction

Water Inflow Ratio, α

An attempt is now made to explore the correlation between water inflow from probing and that measured inside the tunnel after construction, via the introduction of a water inflow ratio (α) which is given by:

$$\alpha = \frac{Q_{tunnel}}{q_{probe}} \qquad \text{Eq (1)}$$

where Q_{tunnel} is the water inflow rate in the tunnel measured by V-notch weirs, and
q_{probe} is the water inflow rate measured by advance probing.

The α value bears two physical significances. The inherent difference between the sizes of a probe hole and a tunnel implies a scale factor relating to size effect being greater than unity. This is, however, counter-balanced by the reduction of water inflow rate due to time effect as described in the previous section. The rate of water ingress measured from a probe hole is representative of the initial flow rate for that particular embedded length. To highlight the time effect, the probe hole with the highest water inflow among the 4 locations (denoted as A, B, C and D in Figure 3) is chosen for the calculation of the α value. On the other hand, the V-notch weir data is a conservative measure of the long-term water inflow along the tunnel; the entire process from transient to steady state will presumably take a longer period of time.

The water inflow characteristics of the Eagle's Nest Tunnel quantified by the α value is depicted in Figure 4. The trends for the two tunnels are in good agreement, which are

characterized by a zone of low α values located at the middle portion. This substantiates the hypothesis that the high water flows measured during HDD and probing near the middle of the tunnel was a transient process, which decayed rapidly with time. The relatively high α values near the two ends of the tunnels are the results of the small inflow rates measured in the probe holes, implying a dry condition in the rock. The average α values are 0.62 and 0.94 for the south and north bound tunnels respectively.

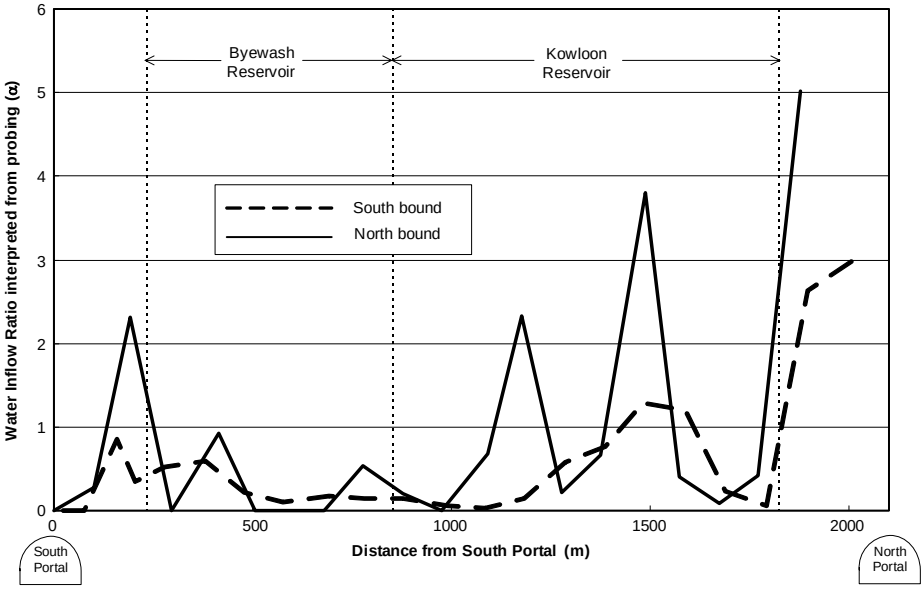

Figure 4. Reduction Factor Interpreted from Advance Probing

The above analysis hinges on the close relevancy between the probing inflows and the V-notch weir measurements. It is worthwhile to explore the correlation between inflows in HDD, which are normally available in the design stage, and water inflow in the tunnels after construction (V-notch weir measurements). The water inflow ratio interpreted from HDD (α_{HDD}) is plotted in Figure 5. The close agreement in Figures 4 and 5 is demonstrated by a

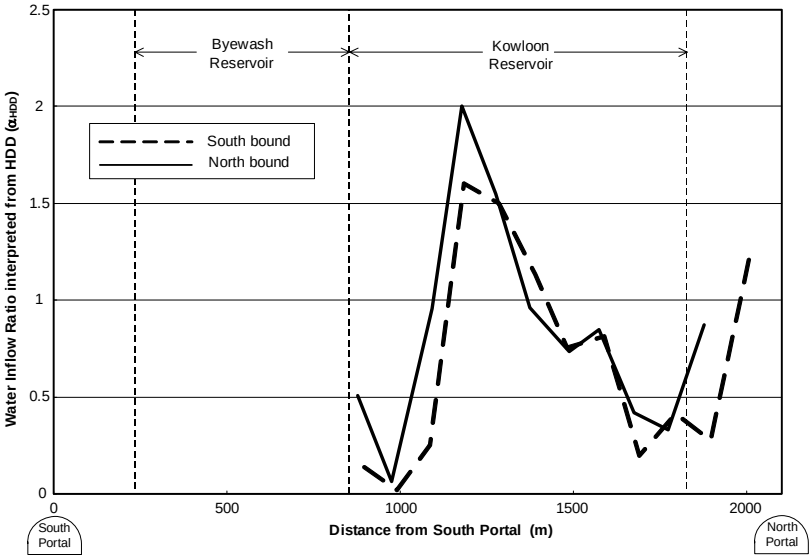

Figure 5. Reduction Factor Interpreted from HDD

small α_{HDD} value near the middle portion of the tunnels, and a relatively high magnitude of α_{HDD} value closer to the north tunnel portal. The average α_{HDD} values are 0.694 and 0.84 for the south and north bound tunnels, which are comparable with the average values calculated from probing. These conformities suggest that, for the chosen conditions, the water inflow information obtained during HDD is a good indicator of the actual water flow characteristics in the tunnel during construction.

DISCUSSION

Relative Importance Between Size Effect and Time Effect

The back analyses presented in the previous sections suggest an average water inflow ratio (α) of 0.78 from probing and 0.767 from HDD. These values being smaller than unity implies that time effect surpasses size effect for this particular case.

Goodman et al. (1965) propose that the water inflow to a circular tunnel can be estimated by:

$$q = \frac{2\pi k(H+h)}{\ln\left(\frac{4H}{D}\right)} \qquad \text{Eq (2)}$$

where q is the inflow rate per unit length;
k is permeability of the medium;
H is the cover depth above the centerline of the tunnel;
h is the pressure head from groundwater, and
D is the tunnel diameter.

This classical theory implies that the water flow rate in a cylindrical cavity increases with its diameter, albeit not in a linear fashion. The sizes of a drillhole and a tunnel normally differ from each other by at least 2 orders of magnitude. With a nominal cover depth of 50m, Equation 2 suggests that the water inflow rate in a 10 m diameter tunnel is about 2.5 times larger than that in a 0.1 m diameter drillhole.

The size and time effects may be separated from the α value into a size scale factor (S) and a time reduction factor (T):

$$\alpha = \frac{Q_{initial}}{q_{initial}} \times \frac{Q_{final}}{Q_{initial}} = S \times T \qquad \text{Eq (3)}$$

where $Q_{initial}$ is the initial water inflow in a large cylindrical hole (i.e. tunnel);
Q_{final} is the final water inflow in a large cylindrical hole (i.e. tunnel), and
$q_{initial}$ is the initial water inflow in a small cylindrical hole (i.e. probe/HDD hole).

With an assumed S value of 2.5, the inferred T factor for the Eagle's Nest Tunnel would be 0.312 and 0.307 for the cases of probing and HDD respectively. These genuinely small values demonstrate the prominent time effect in the water inflow characteristics during construction.

The Role of HDD

The field monitoring data together with the quantitative analyses serve as solid evidence to support the use of HDD as a reliable method to assess water inflow during rock tunnel

construction. The calculated α values suggest that the inflow during HDD (or probing) can be as significant as, if not higher than, the long-term water flow in the tunnel. If the impact on the surrounding environment caused by water inflows during tunnel construction is an issue, field monitoring should also be implemented during HDD considering the similar magnitude of water ingress in the two activities. These may include the potential drawdown of groundwater table as well as the associated ground settlement. The HDD has an overwhelming advantage of being small in size, which makes it possible to quickly cease any loss of groundwater if the drillhole hits a highly permeable zone.

At the design stage, the potential threats to the surrounding environment due to water inflows were normally determined by seepage analyses assuming a steady-state condition. The results were subsequently used to determine the grouting trigger level. During construction, it is impractical and unaffordable to allow water inflow to attain a steady state condition. Decision as to whether pre-grouting is needed has therefore to be made shortly after excavation based on a short-time transient flow. The long-term flow characteristics can actually be more easily determined during ground investigations through HDD. The obtained information can potentially be used to relax the grouting criteria based on more realistic long-term conditions.

CONCLUSIONS

The water inflow characteristics of the Eagle's Nest Tunnel were studied using the field measurements collected before and after construction. A prominent transient flow effect was observed. The correlation between pre- and post- construction water inflow was quantified by a water inflow ratio (α), which was calculated to be 0.78 and 0.767 from probing and HDD data respectively. These values together with simple quantitative analyses demonstrated that the water inflow rates in HDD or advance probing were of the same order of magnitude as those exhibited in the tunnel after construction due to a combination of size and time effects. This conclusion promotes the use of HDD for assessing the water inflow, as well as the associated groundwater drawdown and surface settlements, before the actual tunnel construction.

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GEOTECHNICAL ASPECTS OF THE SHATIN TO CENTRAL LINK SCHEME DESIGN, TAI WAI TO DIAMOND HILL

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Abstract

The Scheme Design for the Tai Wai to Diamond Hill section of the Shatin to Central Link railway includes tunnels approximately 3.5km in length under the Wong Tai Sin District initially, then under Lion Rock, emerging at Hin Keng. Ground investigation undertaken along the alignment revealed a number of geotechnical hazards such as soft ground, variable weathered zones, faulting and potentially high in-situ horizontal stresses within the rock mass through which the tunnels are to pass.

This paper provides a case study describing the geological conditions anticipated along the Shatin to Central Link alignment from Tai Wai to Diamond Hill. Specific geotechnical issues and potential hazards are described and their impact on the future tunnelling works assessed.

INTRODUCTION

The Shatin to Central Link (SCL) is one of the strategic rail projects identified in the Government of HKSAR's Rail Strategy 2000. SCL will have approximately 17 km long of new railway in total and will form an extension to the Kowloon-Canton Railway Corporation's (KCRC) Ma On Shan Railway (MOSR) and an extension to the East Rail across the Victoria Harbour. The Ma On Shan Railway extension will commence at the overrun tracks of the Tai Wai Train Maintenance Centre (TMC), pass under Lion Rock and through South East Kowloon in tunnel, and connect to the existing West Rail at Hung Hom. The East Rail will commence at Hung Hom, cross under the Harbour in a new fourth Harbour Rail Crossing on to Hong Kong Island and terminate in Central District. This paper will describe the geological conditions and associated geotechnical issues of the proposed works between Diamond Hill SCL Station and the MOSR Station at Tai Wai.

The section of the proposed SCL between Diamond Hill and Tai Wai includes tunnels approximately 3.5 km in length from Diamond Hill, under the Wong Tai Sin District initially, then under Lion Rock, emerging through a tunnel portal at Hin Keng. Northwards from there a cut and cover tunnel is proposed. The alignment between Diamond Hill and Tai Wai is shown on Figure 1.

An extensive ground investigation was undertaken over the alignment to obtain information on the site conditions and geology through which the tunnels will pass (a detailed

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description of the site investigation undertaken over the whole SCL alignment is provided by Ng and Wardall, 2005). The ground investigation included a number of deep boreholes up to 375 m below ground level beneath Lion Rock and specialist in-situ testing, such as hydraulic fracture testing in granite rock. This paper will describe the site geology and present an interpretation of the geotechnical conditions, based on the ground investigation undertaken, together with a discussion of the associated geotechnical hazards with respect to the construction of the tunnels. In particular, the presence of existing Mass Transit Railway Corporation (MTRC) tunnels and a WSD tunnel located above the proposed SCL alignment at Diamond Hill and Hin Keng respectively is anticipated to pose a significant constraint.

SITE GEOLOGY

A geological plan over the SCL alignment between Diamond Hill and Tai Wai is shown on Figure 2. The following provides a description of the site conditions and geology for the tunnel sections between Diamond Hill and Wong Tai Sin and between Wong Tai Sin and the tunnel portal at Hin Keng through which the tunnel will emerge.

Tunnel Between Diamond Hill and Wong Tai Sin

Based on the Hong Kong Geological Survey (HKGS) 1:20,000 Geological Map, Sheet No.11 for Hong Kong & Kowloon, the solid geology of the proposed tunnel alignment between Diamond Hill Station and Wong Tai Sin is dominated by Mesozoic intrusive igneous rocks comprising medium grained Granite of Jurassic – Cretaceous age. The medium grained granite is generally described as moderately strong to strong, pinkish grey, mottled black and white, moderately to slightly decomposed medium grained granite.

Superficial deposits of the Quaternary age, shown on the HKGS Geological Map, Sheet No. 11, over the tunnel between Diamond Hill Station and Wong Tai Sin include Debris Flow Deposits (mainly occurring on the slopes or immediate adjacent areas) and Alluvium (near Diamond Hill Station). In addition, some recent Fill material (related to site formation between Diamond Hill and Wong Tai Sin) is present.

A basalt dyke, trending NE-SW, is shown on the HKGS 1:20,000 Geological Map Sheet 11 to the north of Diamond Hill station. This dyke is indicated to be about 1.3 km long and is obscured by Fill, Debris Flow Deposits and Alluvium about 100 m to the north of the tunnel alignment.

Tunnel Between Wong Tai Sin and Hin Keng Portal

This section of the tunnel will pass beneath Lion Rock. Based on the HKGS 1:20,000 Geological Map, Sheet Nos. 7 and 11, the solid geology of the tunnel section between Wong Tai Sin and the Hin Keng Portal is dominated by Hong Kong Granite. Based upon the geological maps, defined zones of fine, medium and coarse grained Granite are expected to intersect this section of the alignment. However, all three types of Hong Kong Granite generally behave similarly on weathering and also have similar structural characteristics (GCO, 1987). To the south of the Hin Keng Portal area the Hong Kong Granite has been intruded by some major Quartz Monzonite and occasionally Quartzphyric Rhyolite Dykes.

Superficial deposits of Quaternary age over the tunnel between Wong Tai Sin and the Hin Keng Portal include Debris Flow Deposits, mainly occurring on the slopes to the north and south of the tunnel section and/or Talus (rockfall) Deposits near the crest of Lion Rock.

The HKGS 1:20,000 Geological Map Sheet Nos. 7 and 11 show four faults intersecting the tunnel alignment between Wong Tai Sin and the Hin Keng Portal. One further fault is also identified within the vicinity of the northern portion of this tunnel section.

The four observed faults which intersect the tunnel can be divided into two groups, one to the north and one to the south of the crest of Lion Rock. The first two faults are roughly parallel and are located to the south of the crest of Lion Rock (approximately 1.4 km to the north of Diamond Hill station). These two faults are about 80 m to 100m apart and trend approximately NE-SW with an interpreted steep dip to the south east. The second two faults intersect over the tunnel alignment about 1 km south east of the Hin Keng portal. The major fault trends NW-SE (approximately sub-parallel to the tunnel alignment) and the second fault trends NE-SW and would appear to terminate at the intersection with the major fault (to the NE of the proposed tunnel alignment).

A further NW-SE trending fault is indicated approximately 100m to the south-south-east of the Hin Keng portal. Most of the fault trace is masked by superficial (debris flow) deposits but it does not appear to intersect the alignment of this section of the tunnel. However, the fault appears to extend for over 1Km roughly sub-parallel to the alignment.

Two large quartz monzonite intrusions, trending NE-SW, and located approximately 300m to 400m to the south of the Hin Keng Portal are indicated on the HKGS 1:20,000 Geological Map Sheet No.7 and these are indicated on Figure 2. These quart-Monzonite intrusions are approximately 100 m to 200 m wide and appear to be subvertical. A much smaller Quartzphyric Rhyolite dyke and a basalt dyke, both trending NE-SW, are also indicated approximately 100m to the south of the tunnel alignment, near the Hin Keng Portal.

GEOTECHNICAL CONDITIONS AND HAZARDS

An interpreted geological plan and profile, based on the ground investigation undertaken, along the proposed tunnel alignment is presented on Figures 3 to 5. The following presents a description of the geotechnical conditions and a discussion of the associated geotechnical hazards for the two tunnel sections.

Tunnel Between Diamond Hill and Wong Tai Sin

The section of tunnel from Diamond Hill Station and Wong Tai Sin will be constructed within mixed ground. Based on the available ground investigation information superficial deposits comprising Fill, Colluvium and Alluvium, up to 13 m thick, are present along the section of tunnel from Diamond Hill Station to Wong Tai Sin. Rockhead, of medium grained Granite, is inferred at between approximately 10m (+9 mPD) to over 60 m (-30 mPD to –60 mPD) below ground level. Details of the subsurface conditions between Diamond Hill and Wong Tai Sin are presented on Figure 3.

Ground investigation revealed a deep weathering profile of the underlying granite in two areas: immediately to the North of Diamond Hill station, where rock head is determined at approximately –60 mPD; and also towards Wong Tai Sin where rock head is determined at approximately –30 mPD. Boreholes undertaken during the ground investigation provided evidence of faulting towards Wong Tai Sin and, therefore, this deep weathering may be structurally controlled. The inferred presence of faulting could potentially lead to abrupt and unpredictable changes in ground conditions, from relatively good and stable ground to

poor and unstable ground, with possible enhanced ingress of groundwater.

The proposed SCL tunnel alignment is to pass immediately beneath (minimum vertical separation of around 7m) the existing MTRC Kwun Tong Line tunnels at Diamond Hill which will potentially pose a significant hazard. The geological profile projected along the downtrack and uptrack of the proposed SCL alignment at the MTRC crossing is shown on Figures 3 and 6 respectively.

An interpretation of the ground conditions along the downtrack of the SCL alignment at the MTRC crossing indicates that both the uptrack and downtrack of the existing MTRC alignment tunnels are within moderately to slightly decomposed rock (Grade III/II) mass and are located approximately 7m above the proposed SCL tunnel. An interpretation of the ground conditions along the uptrack of the SCL alignment at the MTRC crossing indicates rockhead to be dipping towards the North-East. The downtrack of the existing MTR tunnel is interpreted to be passing through completely decomposed rock (Grade V) with its invert founded on moderately decomposed rock and the uptrack tunnel is interpreted to be passing through the less weathered moderately to slightly decomposed rock (Grade III/II) mass and is located approximately 2 to 3 m above the proposed SCL tunnel.

Based on a three dimensional FLAC analysis carried out to investigate the effects on the MTRC tunnels from the SCL tunnel construction 7 m below using TBM tunnelling method, the anticipated settlement of the MTRC tunnels would be within the limit set by MTRC (Buildings Department, 1998).

Tunnel Between Wong Tai Sin and Hin Keng Portal

Based upon the ground investigation undertaken, variable thicknesses of superficial deposits (up to 22.5 m thick) comprising Fill, Colluvium, and Talus (rockfall deposits) are inferred above the tunnel between Wong Tai Sin and the Hin Keng Portal. In addition, the depth of completely to highly decomposed rock is anticipated to range from approximately 20 m near Hin Keng portal to over 50 m near Wong Tai Sin.

The proposed tunnel section between Wong Tai Sin and Hin Keng Portal is to be constructed predominantly within moderately to slightly decomposed medium grained Granite. However, it is anticipated that the tunnel will also pass through two areas of Granite, near the Hin Keng portal, which have been intruded by Quartz Monzonite dykes. Details of the subsurface conditions between Wong Tai Sin and Hin Keng are presented on Figure 4 and 5.

Zones of variable weathered Granite, with corestones of less weathered material, were inferred in some boreholes towards the Wong Tai Sin end of this section of the alignment. The thickness of these variably weathered zones ranges from approximately to over 30 m; in some, cases this variably weathered zone is inferred within the depth range of the proposed tunnel. The presence of potential corestones poses hazards with respect to the tunnel excavation and could also affect the effective rock cover. A potentially thin rock cover above or highly weathered area at tunnel level would necessitate the adoption of a method of construction similar to NATM prior to commencing drill and blast tunneling methods through the moderately to slightly decomposed rock within the tunnel section beneath Lion Rock.

The rock encountered within the tunnel section from Wong Tai Sin to Hin Keng Portal is generally of granitic origin. However, some areas of Quartz Monzonite have been intruded

into the Granite towards the northern end of this section of the tunnel. One Basalt dyke (approximately 1.8 m wide) was also intercepted. Although it is possible that other Basalt dykes may exist within this area, no other areas of Basalt were encountered. Based on existing information, Quartz Monzonite bodies and Basalt dykes commonly coincide with fault zones and are believed to be intruded along lines of structural weakness. Ground investigation undertaken within the vicinity of the proposed alignment, indicates deep weathering of the Quartz Monzonite up to a depth of approximately 50 m below existing ground level. However, the contact between the area of Quartz Monzonite and the Granite may not be clearly defined and the contact zone between the Granite and the Quartz Monzonite is prone to preferential weathering. The presence of numerous small Quartz Monzonite dykes, interspersed within the relatively more resistant granite could also potentially result in variable ground conditions being encountered within the tunnel as well as influence the flow of water into the excavation, which could potentially affect the stability and construction of the tunnel.

Based upon the published geological information for this section of the tunnel, the four faults inferred could potentially intersect the proposed alignment of the tunnel. Three further faults have also been inferred based upon the findings of Aerial Photograph Interpretation and also from the ground investigation undertaken. The approximate locations of these faults are indicated on Figure 4 and Figure 5. The presence of faulting can lead to abrupt and unpredictable changes in ground conditions, from relatively good and stable ground (grade III or better rock with moderately spaced joints) to poor and unstable ground (sand, gravel, and cobbles), with possible ingress of groundwater. Most of these faults have been inferred to be steeply dipping. However, it is possible that the inclination of the inferred faults may be less steep and the potential zone of influence on the tunnel may, therefore, be much greater. Greater emphasis will, therefore, be needed on probe holes in advance of the tunnel to accurately determine the nature and extent of the faulted zones in order to accurately determine the requirements for support and groundwater control prior to excavation.

Two boreholes located near to Wong Tai Sin, towards Lion Rock, showed evidence of core discing, which is indicative of high in situ stress regimes from where the rock cores are retrieved resulting in stress relief cracking (Obert & Stephenson, 1965). The presence of high insitu-stress conditions will potentially affect the stability of the tunnel excavation and could influence the construction of the tunnel in terms of shape and support. In particular, such conditions may result in rock bursts during excavation, which will pose a significant hazard. Appropriate tunnel lining design, tunnelling method and temporary support system will need to be adopted in this section. Hydraulic fracture tests (ISRM 1987) carried out suggests that the vertical principal stress is the minimum principal stress at depths less than 150 m; this is consistent with the findings reported by Free et al (2000) and Ng & Wardall (2005).

A WSD tunnel is within 8 m of the SCL alignment. Drill and blast construction for the tunnel excavation in the vicinity of the WSD tunnel beneath Lion Rock Country Park has, therefore, been identified as a geotechnical hazard (Water Supplies Department, 1998). In addition, groundwater and the interpreted deep weathering of the Quartz Monzonite have also been identified as major geotechnical hazards near the WSD tunnel interface which may cause a rapid recharge of groundwater and significant amount of infiltration of water from the large catchment area associated with the presence of the deep weathered zone. However, visual inspections of the WSD tunnel revealed the ground conditions to be feasible to adopt controlled blasting within the vicinity of the proposed SCL alignment for the tunnel

excavations. In addition, installing a steel liner within the WSD tunnel in this section was also considered.

CONCLUSION

This paper has presented a case study on the geotechnical aspects of the Shatin to Central Link, Diamond Hill to Tai Wai Section, as part of the Scheme Design undertaken. Based on the geological and ground conditions to be expected, interpreted from ground investigations carried out over the alignment, a number of geotechnical hazards have been identified. These hazards include :

(a) deep weathering profile locally, which may be structurally controlled due to the presence of faulting;
(b) the close proximity of the existing MTRC Kwun Tong Line tunnels, which will require carefully considered tunneling techniques to ensure movements are within those stipulated by MTRC;
(c) contact zones where different rock formations have intruded into the Granite, which commonly coincide with fault zones and are believed to be intruded along lines of structural weakness;
(d) the presence of high in-situ-stress conditions, which will be one of the key factors in considering the stability during tunnel excavation and could influence the construction of the tunnel in terms of shape and support; and
(e) the close proximity of a WSD tunnel, which will impose limitations with respect to blasting.

To mitigate the above hazards, appropriate tunnel excavation sequence and methods, and lining design will need to be employed. Additional site investigations will need to be carried out to support the detail design of the tunnels. A number of measures will have to be taken to overcome the hazards identified:

(a) Probing ahead of the tunnel excavation may be undertaken to determine the ground conditions within the vicinity of faults and contact zones and if such investigations reveal adverse ground conditions, advance grouting may be required to prevent water ingress and to stabilize the ground.
(b) In order to limit movement of the existing MTRC Kwun Tong Line tunnels, it is envisaged that the tunnelling for the SCL at this interface will require the use of a Tunnel Boring Machine, which will effectively control the ingress of material and hence the settlement during excavation. In addition, it will be necessary to backfill the annular space between the erected ring of segments and the excavated soil immediately behind the tail skin; ground treatment may also be considered as a contingency measure.
(c) With respect to the WSD tunnel, it is expected that if drill and blast construction is to be adopted near the WSD tunnel then it is envisaged that a steel liner will need to be grouted into the WSD tunnel to minimize construction risks. Furthermore, it is likely that the charge weights of explosive within the vicinity and face area of the heading will need to be reduced in order to minimize ground borne vibration to the WSD tunnel.

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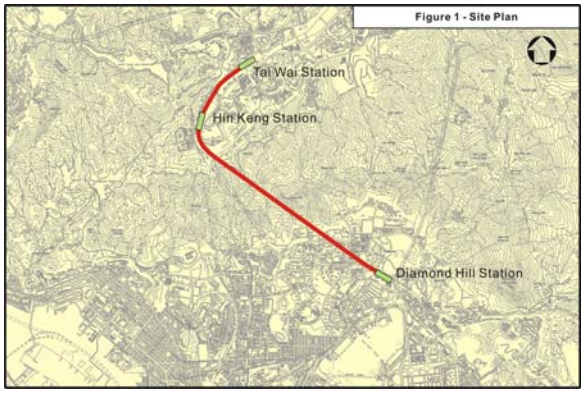

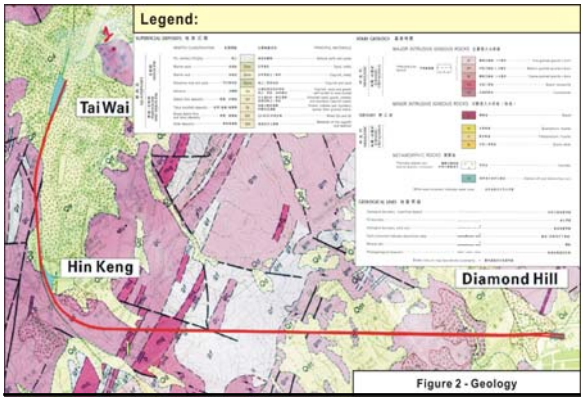

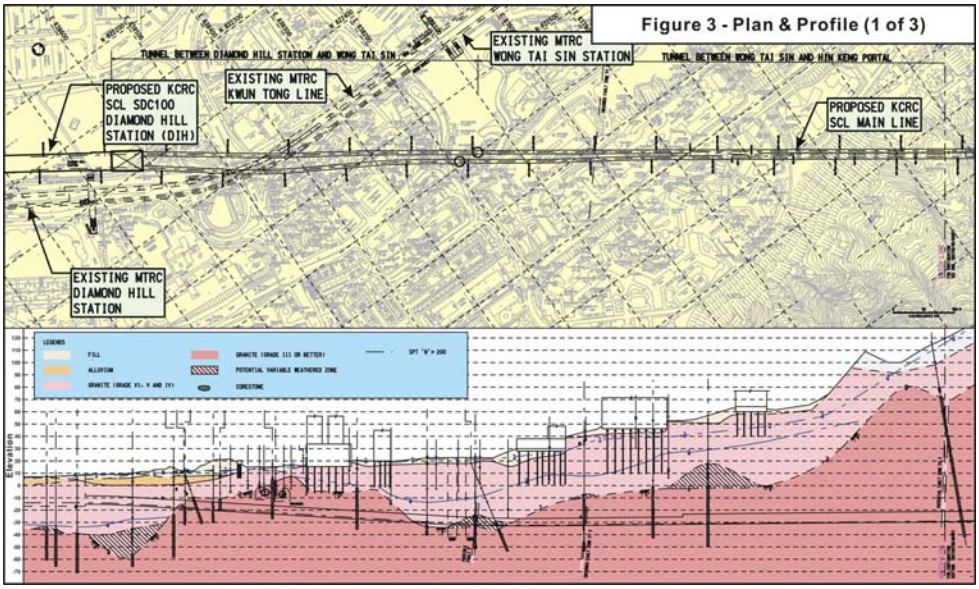

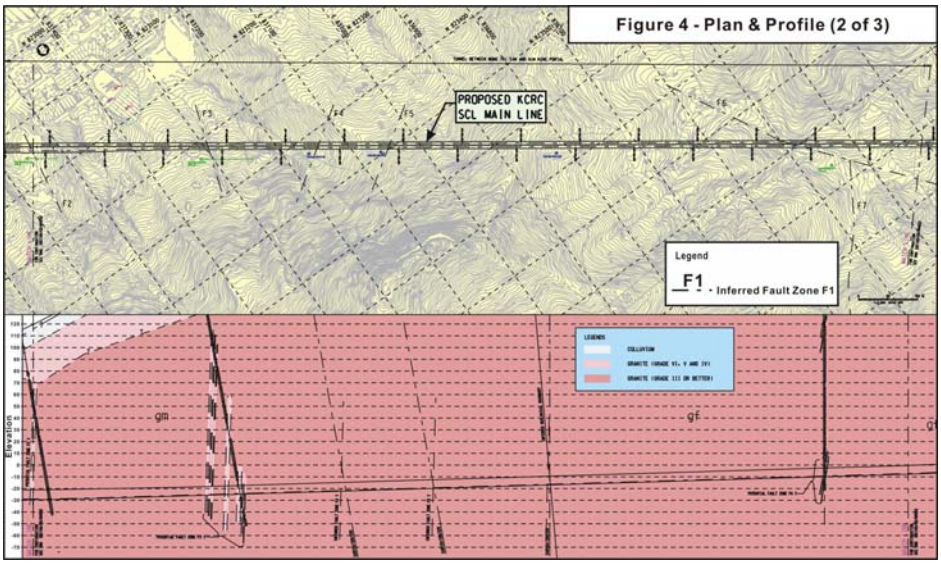

Figure 4 - Plan & Profile (2 of 3)

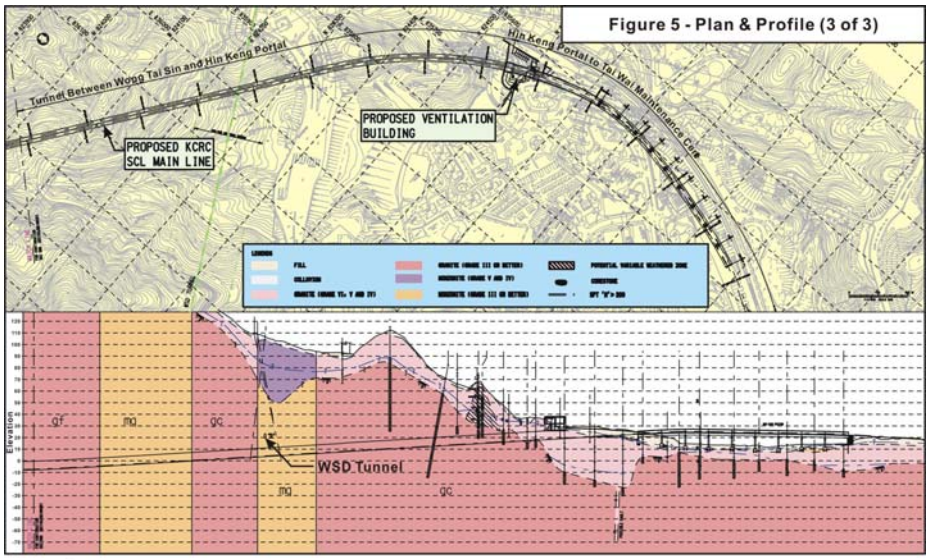

Figure 5 - Plan & Profile (3 of 3)

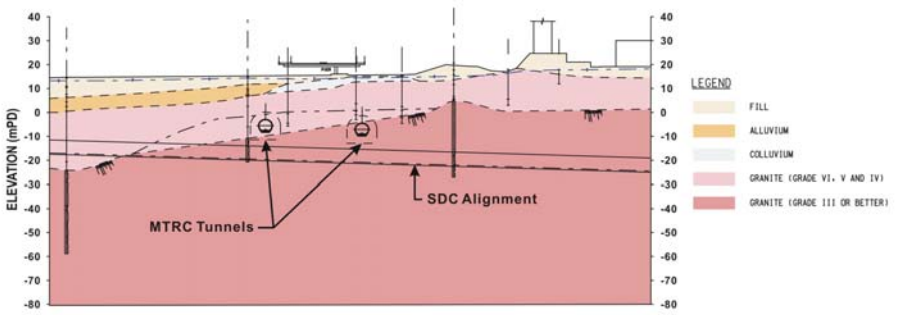

Figure 6 - Uptrack Profile at MTRC

Remarks on Steel Immersed Tube Tunnelling Techniques

W. W. Yang[1], Joseph Lo[2] and Ha King[2]

Abstract

The immersed tube method for underwater crossings involves prefabricating long tunnel units of either steel shell or concrete section, floating them to the site, sinking each in a previously dredged trench, and then covering with backfill. The steel immersed tunnel which has been widely used in North America, has some advantageous features of easy fabrication in a shipyard and are relatively light, with easy flotation of the units. The essential issues of the steel immersed tunnel are discussed in this paper in conjunction with a case of steel tunnel alternative for Metro Line 5 in Guangzhou China.

INTRODUCTION

An immersed tube tunnel is made up of a number of tunnel units fabricated in a dry-dock or a shipyard. The units with removable bulkhead are then floated out of the dry dock and towed to the site, immersed into a pre-dredged trench, connected one by one to form the final tunnel, and covered with backfill for final protection. An immersed tube tunnel is a feasible alternative to cross stretches of water at a comparable cost with a manageable construction risk. It offers a number of engineering advantages, including:

(a) the cross sections of immersed tunnels can be in single circular, binocular, or rectangular shape, to accommodate railway, highways, combined road/rail, or utility tunnels;
(b) immersed tunnels can be placed immediately beneath a waterway, which allows the tunnel approaches to be shorter and the approaching gradients to be flatter;
(c) immersed tunnels can be constructed in soft ground foundation, which is common in large river estuaries with alluvial deposits;
(d) immersed tunnels are relatively flexible structures which can effectively deal with the forces and movements in earthquake conditions; and
(e) construction programme risk can be minimized by possible concurrent construction operations of dredging, tunnel unit construction and tunnel installation.

With those advantages, the immersed tunnels have been widely built in many countries, since the first use of the immersed tunnel method to construct a twin track railway tunnel crossing the Detroit River in Michigan, USA in 1910. In the last century, more than one hundred immersed tunnels have been built world-wide for road or rail crossings.

Basically two main types of immersed tunnel have been developed around the world, known as steel and concrete immersed tunnels. The steel tunnels use a structural steel in the form of stiffened plates, working with the interior concrete in a composite action. The concrete tunnels use waterproofing structural concrete. Most immersed tunnels in North America are steel tunnels, whilst in Europe and Asia, most are concrete tunnels.

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[2] Maunsell Geotechnical Services Ltd.

One of the key features of the steel immersed tunnel is its ease of fabrication, usually in a shipyard with common welding facilities. Hence, there is no special fabrication site needed. Also there is no special site preparation required in contrast to the concrete tunnel construction method and the steel tunnel usually offers a shorter construction period and this might well offset the material cost difference to the concrete alternative. Another major feature is that the relatively light steel shell is fabricated first before launching to the outfitting site for adding the ballast. Therefore, the proximity of the fabrication yard to the tunnel site is a less important factor comparing to a concrete tunnel scheme. The key factor influencing the selection of a fabrication yard, besides its technical capability, is the economic factor by comparing the transportation cost versus the labour rate in the remote fabrication site, the workload and the overall general economy in the fabrication yard area. This issue is extremely important for tunnels in urban areas where the availability of land for casting basins has been getting scarce. This has proven to be the case in the construction of some tunnels in the United States.

The first steel immersed tunnel built in China is the cross-harbour road tunnel in Hong Kong, which used the single steel shell construction method in 1972. Since then, all other immersed tunnels built in China are concrete tunnels, including the MTR harbour tunnel in Hong Kong (1979), Keohsiung Harbour tunnel in Taiwan (1984), Eastern Harbour Crossing tunnel in Hong Kong (1989), Zhujiang tunnel in Guangzhou (1993), Western Harbour Crossing tunnel in Hong Kong (1997), Chang Hong tunnel in Nibo (2001), Wai Huan tunnel in Shanghai (2003) and LAR MTRC tunnel (1996). Recently in Guangzhou, more immersed tunnels are under study, design or construction, which include cross-river tunnel of Metro Line 5, Zhoutouzui road tunnel, Luntou road tunnel, Bio-island road tunnel and Yuzhu road tunnel as shown in Figure 1. Whilst most of the tunnels adopt the concrete immersed tunnel schemes, a steel tunnel alternative for Metro Line 5 was seriously considered for crossing river. Although a TBM tunnel scheme was finally selected mainly due to the change of navigation requirements, the key issues of the steel tunnel construction method are worth to be discussed in this paper.

Figure 1. Immersed tunnels in Guangzhou (1) Yuzhu Tunnel; (2) Bio-island tunnel; (3) Luntou Tunnel; (4) Zhoutouzui Tunnel; (5) Zhujiang tunnel; (6) cross-river tunnel of Metro Line 5 (steel tunnel alternative)

BRIEF DESCRIPTION OF STEEL IMMERSED TUNNEL

Steel immersed tunnels can be divided into single shell tube and double shell tube categories. The tunnel unit is usually fabricated in a shipyard using a modular technique, in which modular subassemblies are fabricated and fitted and welded together on the shipways to form a tunnel unit. Two ends of a unit are then closed by watertight bulkheads and the keel concrete is placed to provide the stability for launching and towing to the outfitting site. The interior concrete is placed while the steel shell unit is afloat in an outfitting site. The concrete has to be placed in a strict sequence to control the forces imposed on the partially completed unit. Upon completion of the outfitting, the unit is towed to the tunnel project site, lowered into its final place and jointed with the previously placed unit.

A single shell tunnel consists of an outer steel shell plate continuous over the unit length which acts as a permanent watertight membrane. The shell plate is usually 10mm thick and stiffened internally with internal transverse members at about 1.5m spacing and with longitudinal stiffeners when required. Inside the unit concrete lining is cast in-situ using the shell plate as an exterior form, which is also acting as the structural element to carry flexural forces along the exterior face of the tunnel before and after the placement of the concrete lining. The steel shell tube with the interior concrete lining will behave as a steel-concrete composite structure. For the protection of the outer steel shell against corrosion, a cathodic protection system is generally used. A convenient ballast box is placed at the top of the tube section permitting the use of economical stone ballast.

A double shell tunnel includes an interior steel shell plate continuous over the unit length that works compositely with reinforced concrete of minimum thickness placed within it. The shell plate is usually 8mm thick and stiffened by external diaphragms at about 4.5m spacing and external longitudinal stiffeners. An outer steel plate of 6mm thick encloses the exterior diaphragms and acts as an outer form for filling with tremie concrete into the space between the exterior and interior shells, which provides the ballast for immersion control and corrosion protection to the interior shell plate.

ESSENTIAL ISSUES OF STEEL IMMERSED TUNNEL

Connection Joint and Gasket

The units of an immersed tunnel are connected by intermediate joints under water. The joints between units can be rigid or flexible. A tremie concrete joint (see Figure 2) is usually used as intermediate joints for the steel immersed tunnel construction to make for continuity throughout the length of the tunnel and incorporates special joints at the terminal structures. Each tunnel unit is equipped with a set of rubber gaskets, alignment wedges, couplers, jacks, and a dewatering system. The rubber gasket set is composed of two continuous gaskets of trapezoidal shape. The so-called double layer gasket system as shown in Figure 3 is used in most immersed tunnels in North America.

The procedure of the under water connection of units is described as follows.

(a) while the new unit (being placed) is immersed under water and is still being held by the barge, it is moved to the outboard end of the previously placed unit so that it can be received by guide plates on the element in place;
(b) extend the couplers from one end of the unit and engage them with their mating

parts on the other unit;

(c) pull the new unit into contact with the previously placed unit by the hydraulic jacks and reach an initial seal by the deformation of the cantilever lip of the outer gasket; the function of the cantilever lip is similar to the soft rubber nose of the Gina gasket;

(d) dewater the chamber of the joint and allow the hydrostatic pressure on the far end of the new unit to compress the gasket thereby sealing the joint; and

(e) remove the bulkheads and weld the steel closure plates from within the tunnel to join the steel shells of the adjacent units, and cast the interior concrete.

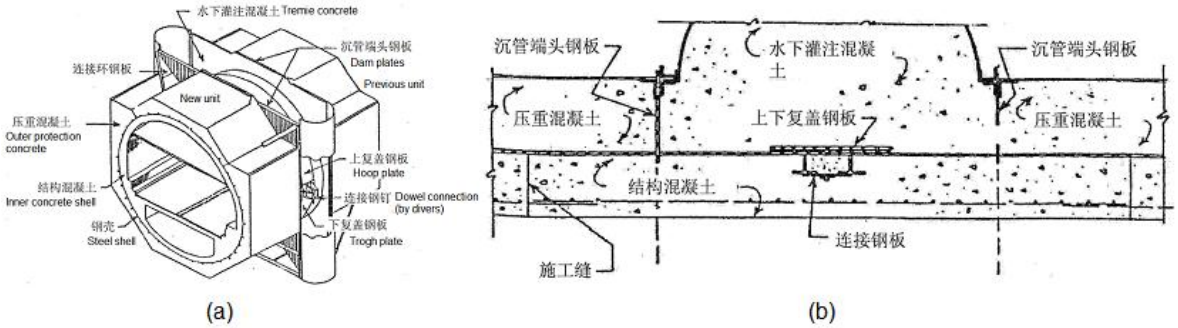

Figure 2. (a) Schematics of a Tremie Concrete Joint (b) Longitudinal Section of a Joint

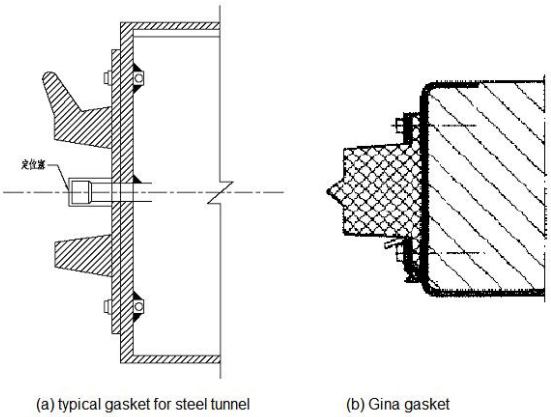

Figure 3. Schematics of a Rubber Gasket

Design and analysis of tunnel structure

Detailed design and analyses of the steel shell units are carried out for the loading cases at fabrication and launching, outfitting, immersion and in-situ stages.

During fabrication, the unit is analyzed for the type of fabrication to control the geometry, deformation and strength of the member structures. Once fabricated, the unit is towed to the outfitting site usually by a side launching method.

During outfitting, the interior concrete is placed in a predetermined sequence designed to keep the unit on an even trim and lift. At each outfitting stage, the following items are carefully checked :

(a) longitudinal stress analysis of the unit under self-weights, buoyancy forces and each additional staged concrete weight;

(b) cross-sectional stress analysis of the unit under hydrostatic pressure;

(c) strength analysis of the interaction between global and local stresses;

(d) the lateral and longitudinal stability analysis of the unit structures; and
(e) draft-displacement analysis of the unit.

During the immersion and dewatering of the joints, the buoyancy conditions are controlled. The minimum safety factors against flotation are usually specified to be between 1.02 and 1.05. The safety factors of the unit against uplift in the final position is usually about 1.2 to 1.25.

In the final in-situ position of the tunnel, a soil-structure interaction analysis is carried out, by taking into account the composite action among the shell plate, the stiffeners and the concrete lining.

STEEL SHELL SCHEME FOR METRO LINE 5 IN GUANGZHOU

Alignment and geological conditions

The proposed tunnel is required to cross the Zhujiang River and is part of the running tunnels between Tatansha Station (at the west bank) and Zhongshan Road Station (at the east bank) of the Metro Line 5 at Guangzhou. As there is limited land availability for a casting basin around the site area, a steel immersed tunnel scheme is considered to be an option, which can make use of the available shipyard facilities in Guangzhou.

The design water level (high) at the river site is 7.79mPD; and the existing riverbed level is about –1.7mPD. The river width is about 254m. The ground profile along the alignment of the tunnel is shown in Figure 5. The site investigations reveal that the ground stratification along the alignment can be summarized as follows :

(a) man-made fill: consisting of dumped or soil fill, slightly compacted to un-compacted. Thickness varies from 1.8 m to 8.1 m;
(b) marine-continental alternating sediments: consisting of loose, saturated, plastic and course sand layers with shell debris; thickness varies from 0.3 m to 18.9 m;.
(c) alluvial-colluvial sand: mainly consisting of course sand and cobbles, occasionally fine sand and medium sand; fully saturated, slightly dense to medium dense in most areas , loose in some areas; SPT N value ranges between 7 and 28; thickness varies from 1.0m to 16.3m;
(d) alluvial-colluvial Soil: consisting of silty clay, clay or silt; most of plastic clay and some harden plastic; SPT N value ranges between 5 and 20; thickness varies from 0.6 m to 5.5 m;
(e) river-lake deposit: consisting of saturated, highly plastic very thin layer; thickness varies from 0.6 m to 1.0 m;
(f) residual soil: decomposed from Cretaceous Era, sand stone; consisting of silty clay, occasionally with silt or clay, plastic, slightly dense to very dense. SPT N value ranges between 6 and 44. Thickness varies from 0.6m to 27m.
(g) completely decomposed rock: decomposed from Upper Cretaceous Era, Dalang Shan Formation, Huanghuagang section comprises mudstone, muddy siltstone, siltstone and corestone; SPT N value ranges between 30 and 43; thickness varies from 2.1 m to 11.3 m;
(h) highly decomposed rock; thickness varies from 0.8 m to 17 m.

The layout plan and longitudinal section for the tunnel scheme is shown in Figures 4 and 5 respectively. It is designed as a single shell immersed tunnel of three units each 80 m long,

with a total length of 240 m. The lowest dredging level is down –12.76 mPD.

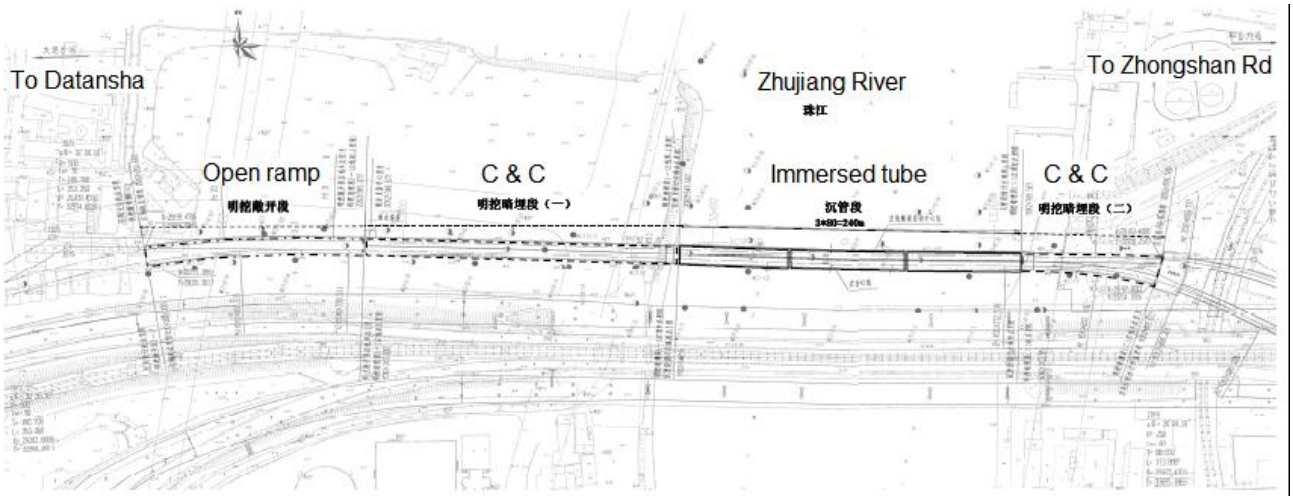

Figure 4. Cross-river Tunnel of Metro Line 5 – Layout Plan

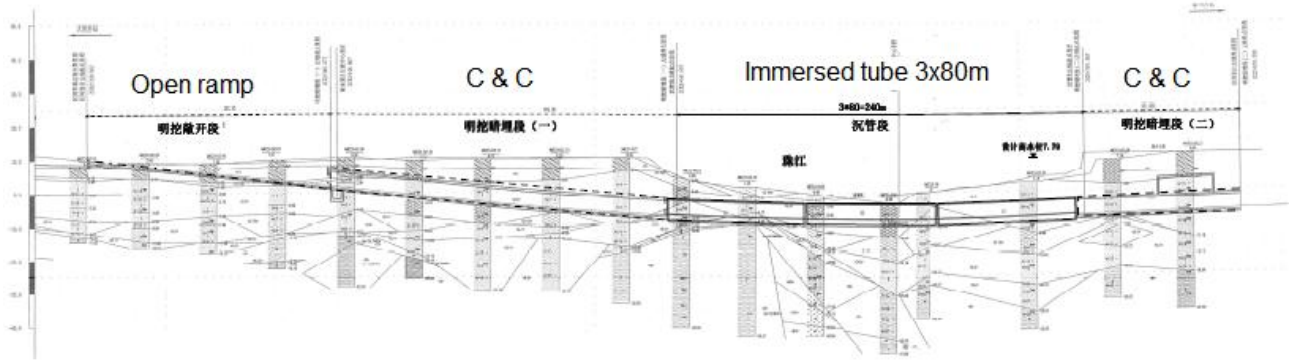

Figure 5. Cross-river Tunnel of Metro Line 5 – Longitudinal Section

Design considerations

The general arrangement of the cross section is shown in Figure 6. It is a twin circular shell tube. The two single shells of 5.9m internal diameter each, are connected with transverse steel diaphragms with horizontal stiffening trusses between diaphragms. The thickness of the outer steel shell is 8mm. The 300mm thick interior lining is of cast in-situ reinforced concrete of 40MPa design strength. The exterior keel concrete and the concrete

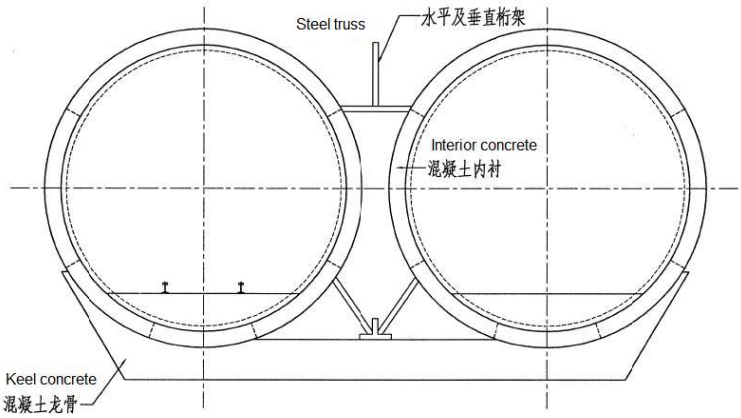

Figure 6. General Arrangement of Tunnel Section

Filling the space between two shells provide weight to resist the buoyancy and to protect the shell plate against corrosion. The other part of the shell plate will be protected by 75mm thick shotcrete.

Considering the strength and stability of the steel shell, the concrete placement sequence at the outfitting stage is determined as shown in Figure 7. There are five placement stages each with different pour sequence as follows :

(a) Stage I: keel concrete (placed at the fabrication stage);
(b) Stage II: interior concrete lining and invert concrete; and
(c) Stage III to V: ballast concrete.

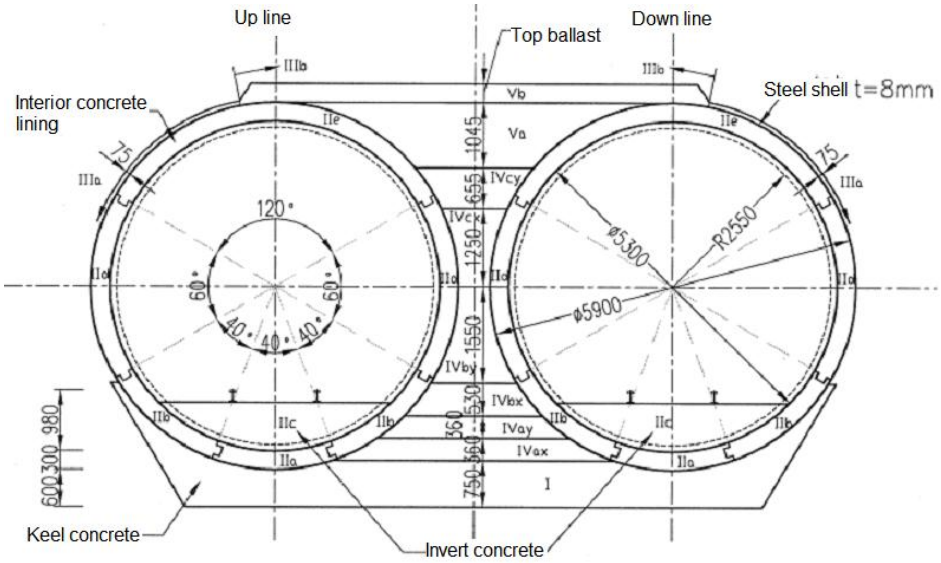

Figure 7. Concrete Placement Sequence in Section

Sensitivity analyses are carried out for the draft-displacement curves, one of which is shown in Figure 8.

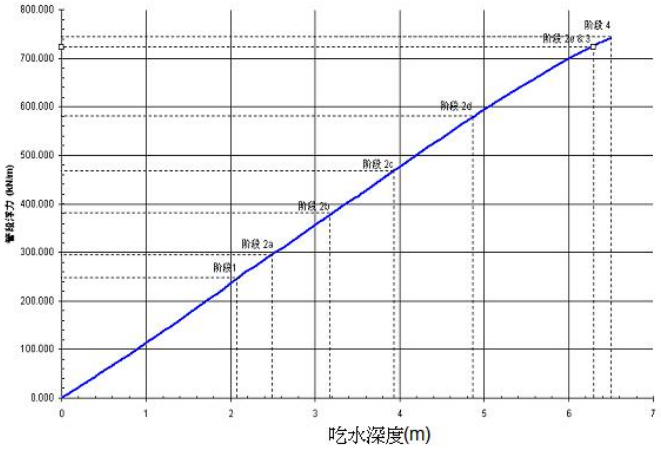

Figure 8. Draft-Displacement Analysis of a Unit

The response of the composite lining at the in-situ stage is obtained by performing a soil-structure interaction analysis and, some results at different loading combinations are presented in an axial force–bending moment interaction diagram with as shown in Figure 9.

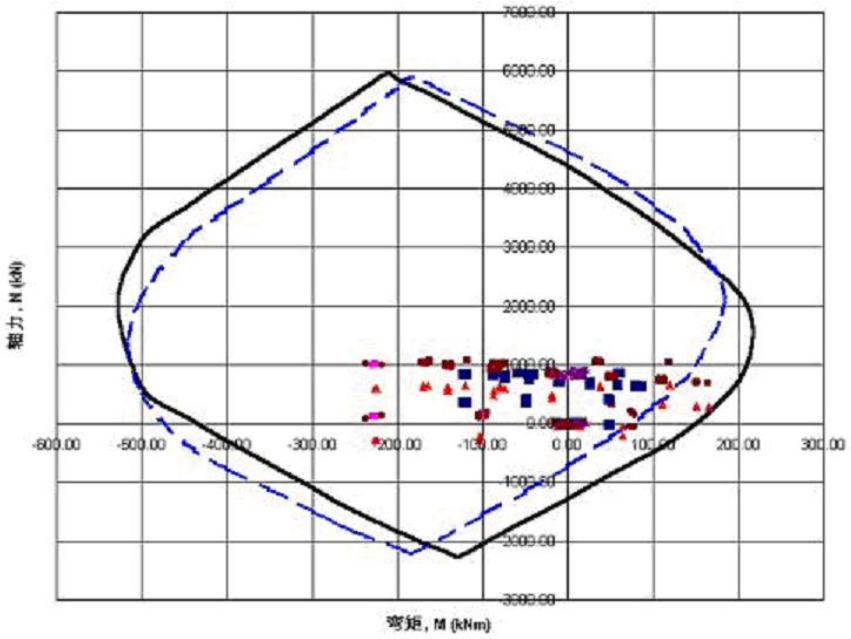

Figure 9. Response of the Composite Lining with two Different Reinforcement Schemes

Considering the composite action between the steel shell and cast in-situ concrete, a reinforcement detail for the interior concrete lining is proposed as shown in Figure 10.

All the joints between units are made under water. It is proposed to adopt intermediate joints as sleeved type as shown in Figure 11. The sleeved joint which was developed for Ted Williams Tunnel in Boston is used to absorb the longitudinal deformation due to temperature change and creep and shrinkage of concrete.

As the tunnel length is relatively short, an expansion joint connection between the immersed tube and the approach cut-and-cover tunnel at bank is proposed (see Figure 12).

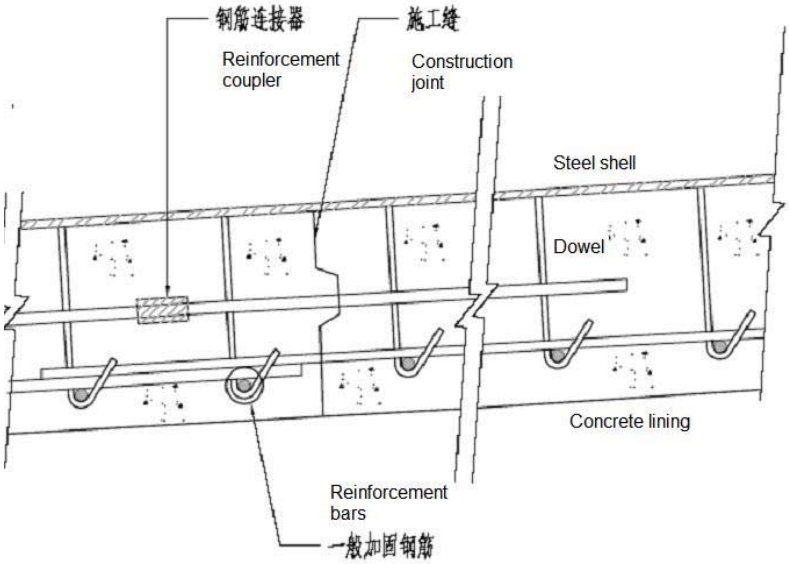

Figure 10. Reinforcement Detail for the Interior Concrete Lining

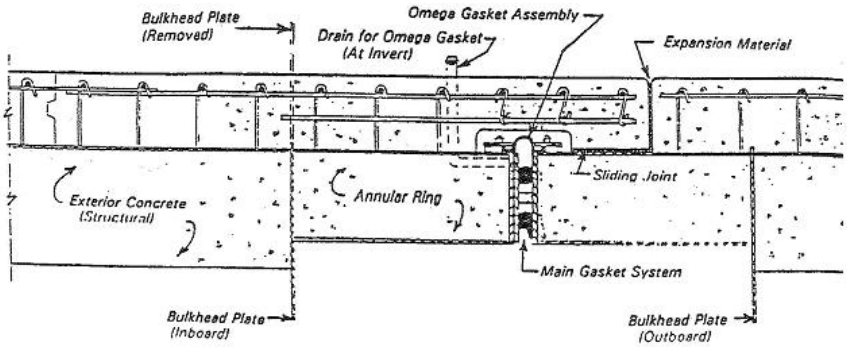

Figure 11. Schematics of a Sleeved Joint

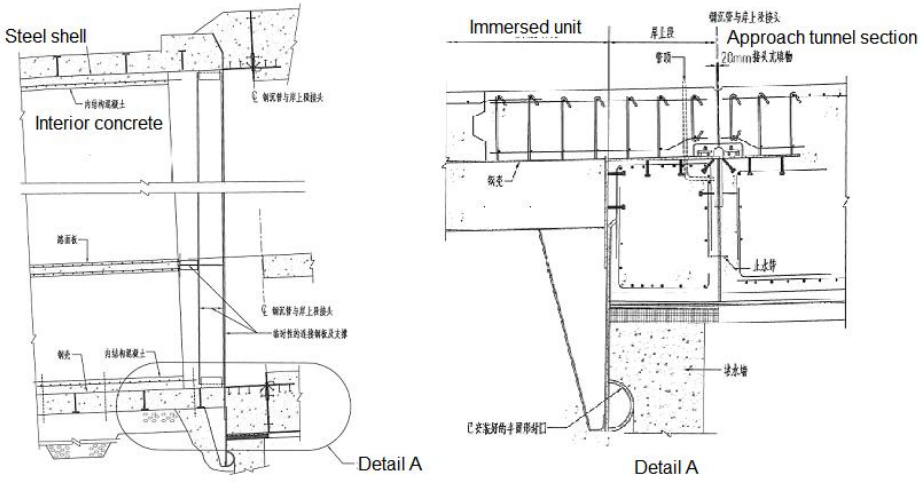

Figure 12. Joint between the Immersed Tube and the Approach Tunnel at Land

CONCLUDING REMARKS

There are two basic types of immersed tunnel which have been widely constructed since the early 1900s. In Europe a rectangular concrete cross section has been used exclusively, whilst in the America the steel shell tunnel lined with concrete have been preferred. Both forms have advantageous features in certain circumstances. Key advantageous features of the steel immersed tunnel are :

(a) ease of fabrication, usually in a shipyard with common welding facilities;
(b) no special fabrication site needed;
(c) no special site preparation required in contrast to the concrete tunnel construction method;
(d) relatively light steel shell unit and therefore easier towing to the site; and
(e) shorter construction period.

As there is the limited land availability for a casting basin around the site area of the proposed metro tunnel crossing of Zhujiang River at Guangzhou, a steel immersed tunnel scheme is considered to be a good alternative option, which can make use of the available shipyard facilities in Guangzhou. The geological conditions, the design and construction issues are discussed to show the feasibility of this construction method.

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VIBRATIONS FROM BLASTING FOR A ROAD TUNNEL - A STATISTICAL REVIEW OF SCALING LAWS

J.K. Murfitt [1] and Billy Siu [2]

Abstract

Current Hong Kong practice for monitoring of blast effects on structures and utilities, from both surface and subsurface construction blasting, is often limited to ensuring a defined peak particle velocity (PPV) value for each sensitive receiver affected by the blasting is not exceeded. Until frequency-related PPV monitoring and monitoring of strains resulting from temperature and humidity changes becomes more widely adopted within a regulatory framework, PPV limits will continue to be the default controlling factor over charge weight and thus rates of tunnel advancement. As charge weight control is currently governed by empirical formulae related to these limiting PPV's, this paper reviews the reliability of the statistical methods used in prediction of PPV's using blast data along a 1 km twin 3-lane road tunnel. Attenuation constants derived from blasts in a section of tunnel are reviewed, using square root scaling, cube root scaling and multiple linear regression techniques, along with statistical goodness of fit tests, to determine the reliability of empirical predictor equations. The paper also attempts to determine which tunnelling factors, including expected surface wave transmission at the Portals, close-in blasts, transmission medium, explosive type (emulsion vs. cartridge explosive), confinement as defined by full face or enlargement blasts and powder factor most affect resultant peak particle velocities.

INTRODUCTION

Excavation progress in any drill & blast tunnel is dependent on a number of interrelated parameters, some of which have a greater impact on progress than others, some of which are variable (e.g. charge weight) and some which are not (e.g. surrounding geology), but all of which combine to define the eventual rates of advance. Measures were put into place at the Shatin Heights Tunnels to review some of these factors, with those identified as having the greatest impact, and those with the greatest scope for improvement of progress being afforded the closest attention during construction. One of the issues reviewed was propagation equations for and the estimation of the magnitude of blast wave velocities (and amplitudes) at any one sensitive receiver.

The main focus of this paper is the accuracy of prediction of the magnitude of blast wave velocities that will arrive at a sensitive receiver, due to a blast using a known charge weight per delay, at a known radial distance from that receiver. What is universally accepted is that the attenuation constants should be derived as a site specific exercise as discussed by New (1986), Dowding (1996), Hustrulid (1999) and Siskind (2002).

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Using statistical analyses, this paper reviews :

(a) whether the blast data from this site follows a lognormal distribution and how well the residuals follow the assumption of normality i.e. the goodness of fit. Normality is the fundamental basis for prediction of the blast constants in the blast equation (Dowding 1996; Pegden et al. 2005).

(b) whether a relevant site specific blasting vibration attenuation relationship exists that more closely models the attenuation of seismic blast waves at various sections along this site, than that derived using the most-often used "square-root" scaling law and whether there are identifiable locations or ground conditions where it is more appropriate to use.

(c) the sensitivities of recorded blast vibrations to:
   (i) surface wave transmission at the portals;
   (ii) close-in blasts;
   (iii) transmission medium;
   (iv) explosive type: emulsion (vs. cartridge) explosive;
   (v) confinement as defined by full face or enlargement blasts; and
   (vi) powder factor (PF).

SOURCE OF BLASTING DATA

A detailed discussion of current blasting practice in Hong Kong, including an explanation of the derivation of the current baseline blasting constants adopted by the Mines Division of the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department, is presented in Li & Ng (1992). It recommends the square-root scaling method as being reliable enough for "local use", and adds that the "Mines Regression Graph" is a useful guide for calculating the maximum explosives charge weight per delay before a site specific blasting vibration attenuation relationship is obtained.

A review of the USBM RI8507 Report (Siskind et al. 1980) indicates the USBM Report conclusions were based largely on ground vibrations from surface coal mine and quarry blasts. The GEO Mines Division blast attenuation formula is itself developed from a data set of 520 combined blasts including quarrying, tunnelling, trench excavation, footing excavation and submarine blasting, the latter blast type for which industry practice is to use a cube root scaling law almost exclusively. This paper concentrates on blast attenuation solely from the current tunnel project based on a total data set of 1132 blasts, resulting in excess of 3580 recorded data points, and attempts to identify which, if any, specific scaling law is the most applicable along the tunnel alignment.

DETERMINATION OF THE ALLOWABLE CHARGE WEIGHT PER DELAY

The current general practice in Hong Kong for determining maximum charge weights per delay for use in blasting works is to employ a form of the energy equation based on "square-root scaling". This ground velocity-distance-energy relationship calculates the predicted peak ground velocity at a distance, R, from a source normalized by the input energy, as follows:

$$PPV = K * (R / W^{a})^{b}, \qquad \text{(Eq 1)}$$

where
PPV = peak particle velocity,

K = constant (intercept),
b = slope of regression line
a = charge weight exponent (0.5 for square-root scaling),
R = radial distance from the Blast Source and
W = charge weight per delay.

The term (R/W^{a}) is also referred to as the scaled distance (SD). The term "per delay" assumes an 8ms delay (industry standard practice for tunnel blasts) in this tunnel. In practice, there may still be cancellation, or reinforcement (overlap) of wave forms. Constants K and b in Eq. 1 above are normally derived on a site-specific basis.

THEORETICAL BASIS FOR SCALING LAWS

There are two main scaling laws based on Eq 1 in common use, square root scaling (exponent a = 0.5) and cube root scaling (exponent a = 0.33).

(a) Square root scaling : Dowding (1996) noted that square-root scaling is based on observations of the distribution of charge in a cylindrical hole. With density constant, the diameter of the charge hole (D_h) is proportional to the square root of the charge weight.

(b) Cube root scaling : Jimeno et al. (1995) observed that cube root scaling has as its theoretical basis, the supposition that the explosive column is a sphere and that any linear dimensions should thus be corrected by the cubic root of the explosive charge. New (1986) proposed the use of multiple linear regression techniques to determine the best fit.

A further detailed discussion on various scaling laws, including numerous variations of the exponent a, can be found in Bollinger (1980), Blair (1990), Singh et al (1993), Jimeno et al (1995), Dowding (1996) and Oriard (2002). Some attenuation laws do not take into account any particular charge symmetry, while others are based on assumed wave form type, e.g. Ghosh & Daemen (1983) related PPV to charge weight with different attenuation relations for body waves and Rayleigh waves and the shape of the charge. Singh et al (1993) even observed that square-root scaling laws are more appropriately used when blasting in softer rock types e.g. sandstones whereas the blast results for medium to hard rock masses, represented by basalt, dolomite and granite, are near optimum when explosive charge per delay is estimated using equations for both square root scaling and cube root scaling. Zhou et al. (2000) noted that at a "sufficiently large distance", the geometry effect should disappear and the exponent "a" should approach 1/3.

Some authors also noted cube root scaling is used for vibration prediction only in the extreme near field (Lucca, 2003; Yang et al, 2000). Further details specific to cube root scaling and case-histories where best-fit analyses resulted in cube-root scaling are discussed in NAVFAC (1986), Siskind et al (1977), Siskind et al (1980), New (1989), Olsen et al (1974), Olson et al (1972), Lucca (2003), Bacci et al (2002), Wu et al (2003) and Zhou et al (2000). Some of the above case histories determined cube root scaling as appropriate for their site and some did not. However, while these references indicate that square root scaling is still by far the most often used scaling law, other laws such as cube-root scaling continue to be checked for and used by practitioners to determine the applicability of attenuation results in various ground conditions.

MULTIPLE LINEAR REGRESSION ANALYSIS (MLR)

New (1989) noted that an attenuation equation derived from a MLR is a more appropriate predictor equation largely because it is site specific. This Paper discusses whether, using the statistical methods below and using the site specific data recorded, the energy exponent "a" derived from MLR, is close enough to 1/3 or 0.33, to justify use of cube root scaling in some areas of the tunnel, such as close-in blasting and/or when using very small (essentially spherical) charge weights. The multiple regression analyses in this paper were determined using a procedure outlined in Singh et al (1993).

ANALYSIS OF THE SHATIN HEIGHTS TUNNEL DATA

The Shatin Heights Data Set

The Shatin Heights Tunnel PPV data comprises records from 1132 blasts, with a total of 3580 data sets recorded. Charge weights varied between 0.066 and 3.841 kg/delay. Numerous "no-results" were also recorded, where the trigger level of the instruments was not met. This may be a function of both starting with very low charge weights (0.066 kg/delay) and instrumentation placement at a large radial distance from the blast.

The fundamental assumption behind the applicability of the blast vibration attenuation equation is that the data follows a lognormal distribution (Dowding 1996). In order to test how well the Shatin Heights data fits this assumption, analysis of the "Goodness of Fit" of the regressed distribution, in a similar fashion to Pegden et al (2005), was performed by conducting univariate normality tests on the residuals of the data and the results plotted against number of blasts (or results) used in the analysis. By plotting the data points in conjunction with tunnel chainage, it is possible to visually determine the number of data points required for normality and to also determine when the attenuation characteristics of the data set from some distance down the tunnel no longer matches the initial data. A series of normality tests has been carried out on several selected data sets, as described below.

Summary of Sensitivity Analyses Performed

In order to identify the sensitivity of contributing factors such as distance and charge weight in the predictor equations, residual plots, plots of PPV vs. range and plots of PPV vs. charge weight were shown for the entire data set.

Summary of Statistical Analysis Tests Performed

To ensure the data follows the assumptions of the statistical model, i.e. the lognormal distribution (for any form of scaling), selected data sets were checked using Frequency Histograms, Normal Probability Test Plots and Normality (Goodness of Fit) Tests on the residuals of the data.

SENSITIVITY ANALYSES

Residual Plots

Hunt et al (2003) concluded that the log/log approach to regression equation determination does not actually produce the best fitting power law equations connecting PPV, distance and charge weight. Hunt et al (2003) also noted that the construction of the 84% and 95% confidence limits assumes not only normality of the log(PPV) on log(SD) data, but also

normality of the distribution of the residuals i.e. the variance of the distribution from normality.

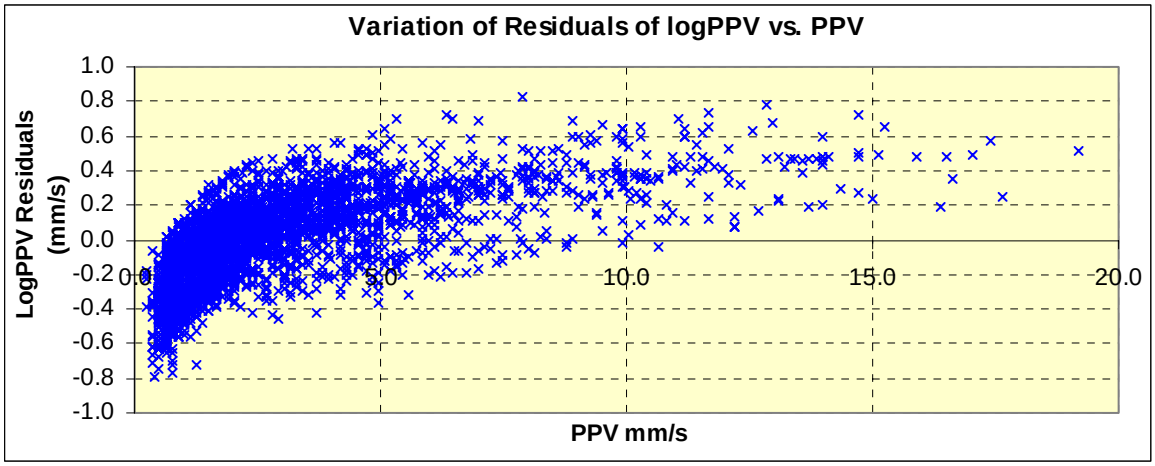

Figure 1. Scatter Plot of the Residuals

The residual plot in Figure 1 indicates both nonlinearity and heteroskedasticity over the range of PPV's recorded, for PPV results above and below approximately 1 mm/s. Heteroskedasticity (also commonly spelled heteroscedasticity) is the violation of the assumption that "the variance of the distribution from which the observations of the error term are drawn is constant" (Studenmund, 1992). While Studenmund (1992) noted a number of tests (e.g. White's test, the Breusch-Pagan test) can be performed to rigorously test for heteroskedasticity, only the scatter plot is shown above, to graphically demonstrate heteroskedastic variability. The variation in the residuals of the PPV's recorded from the SHT tunnel blasts, indicates that in general, the larger the PPV values (above 1 mm/s), the larger the error in the prediction of the PPV using the predictor equation of the form of Eq 1 above.

Sensitivity to Range and Charge Weight

Figure 2 show the expected variation of PPV with range and with charge weight, with range being the dominant factor as evidenced by the steeper curve. The charge weight plot rises up at a relatively flat gradient, indicating much lower sensitivity than the range.

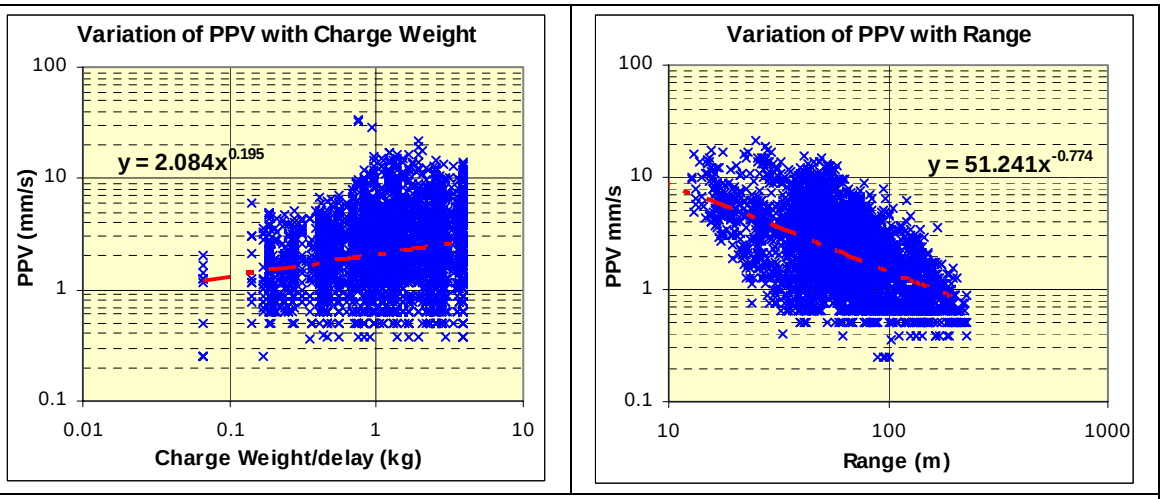

Figure 2. Sensitivity to Range and Charge Weight

STATISTICAL ANALYSES AND NORMALITY TESTING

Visual Inspection of Frequency Histograms & Normal Probability Plots

Inspection of a histogram of the residuals of the peak particle velocities, as shown below, indicates that the data generally follows a lognormal distribution; however the log-transposed distribution is in many cases some way from a symmetric Gaussian distribution, an observation borne out by

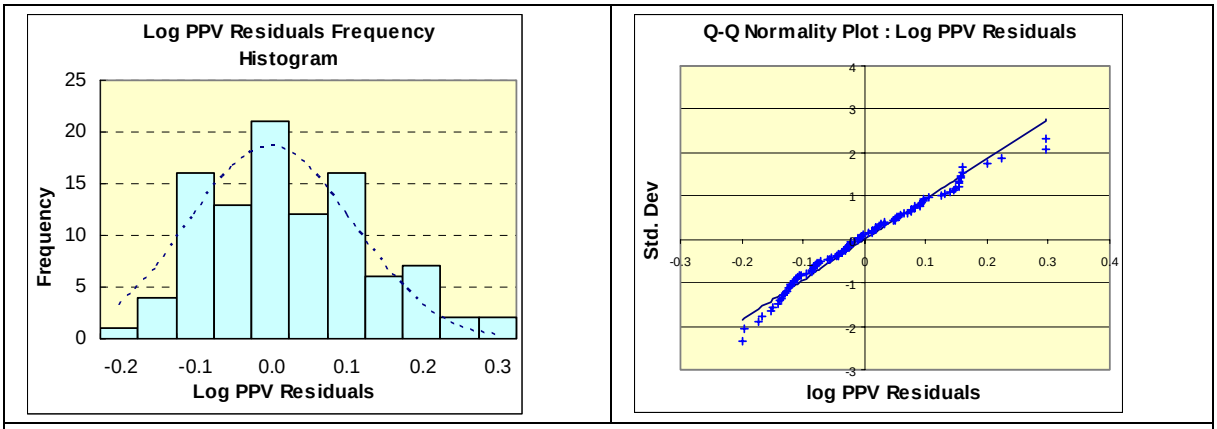

Figure 3. Blast data (100 data points) between Ch. 1477 and 1651

normality testing discussed below. This will influence any predictor equation, and the data set should be closely examined before using to determine allowable charge weights.

The histogram in Figure 3 clearly shows data both below and above the Gaussian curve and deviation in the tails of a Quantile-Quantile (Q-Q) normality plot is readily observable. While visual inspection of both the distribution of the residuals and the normal probability "Q-Q plot" in Figure 3 would likely reject the assumption of normality, surprisingly the data "passes" all the normality tests and three out of the four skewness and kurtosis tests.

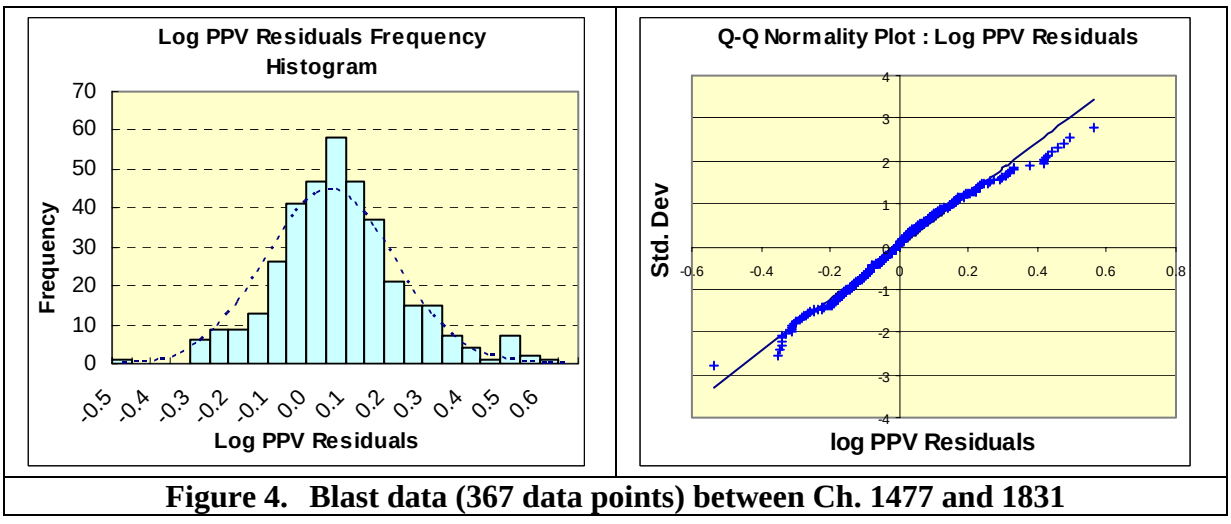

Figure 4. Blast data (367 data points) between Ch. 1477 and 1831

In comparison to Figure 3, the histogram in Figure 4 for a larger data set appears to be more normally distributed. Visual inspection of the distribution of the residuals would likely accept the assumption of normality, however all five normality test and all four skewness and kurtosis tests performed on the data set actually shows the data "fail". The results of Figures

3 and 4 agree with the conclusions reached by Pegden et al. (2005) i.e. visual inspection alone cannot satisfy the assumption of normality.

Continuously Increasing Data Sets and "Windowed" Data Sets

Pegden et al (2005) reviewed a series of quarry blast data, continuously adding new data to the existing data set and checking for normality at each addition of new data. Physically, this makes sense in a quarry situation where it is likely the ratio (A/R) of the advance of the quarry face (A) with respect to the distance (R) to surrounding sensitive receivers remains relatively small throughout the life of the quarry. Adding data to all preceding information is thus statistically viable as it is likely subsequently added data will not skew the original results, provided that scaled distances to the sensitive receivers remain in approximately the same range, and that geology does not change. The first analyses below conduct normality tests on a continuously increasing data set, similar to Pegden et al (2005) approach, with data added in a batch size as shown on the plot.

However, in the case of a tunnel, the location of sensitive receivers most affected changes with the location of the advancing face. The advancing face and different wave propagation paths and travel lengths through different media, mean that adding a set of vibration results from blasts e.g. 100 m apart, picked up by different sensitive receivers, is unlikely to yield results which are consistent. As such, a discrete "window" of a fixed data size, "moving" along the tunnel alignment and representative of the moving face, is considered a more appropriate method of analysing tunnel vibration data. For the analyses below, a window of a fixed size is defined and the window "moved" along the data set in a step size (generally smaller than the window size), with normality tests conducted at each increment.

Normality Tests

There are quite a number of Normality Tests available for checking "Goodness of Fit", some of which have greater power than others and some are more proficient at picking up deviations in the tails, or in the middle of the distribution. Thode (2002), Strom & Stansbury (2000) and Poitras (2006) give specific details of normality testing, particularly on the appropriateness of, and power of each type of normality tests for various sample sizes. The "Goodness of Fit" tests used on the Shatin Heights data set are the Shapiro Wilk, Kolmogorov Smirnov (with Lilliefors correction), Anderson Darling, Jarque Bera and D'Agostino Omnibus Tests. Note that these tests only provide statistical evidence for certain types of "non-normality", they do not guarantee "normality".

For this reason, although one test would be adequate, the above five tests have been selected and rather than tabulate results which show that a data set has "passed" or "failed" the normality test(s), a value of 1 is assigned to those passing the normality test and a value of 0 to those data sets for which the null hypothesis cannot be rejected. To pass a normality test the observed significance level, termed P value (Pval), should exceed a chosen significance level, which for the Shatin Heights Data, was set at 0.05, i.e. Pval>0.05. For the purposes of graphical presentation only and for ease of identification of patterns of normal distribution, a "Normality Index" is defined as the sum of these assigned (0 or 1) values, with 5 being the largest index value (normality likely) and 0 the smallest (normality highly unlikely).

Skewness/Kurtosis Tests

Skewness characterizes the degree of asymmetry of a distribution around its mean and Kurtosis characterizes the relative peakedness or flatness of a distribution compared with the

normal distribution. Similar to the normality index defined above, a value of 1 is assigned to tests "passing" the Skewness or Kurtosis tests, and for the purposes of graphical presentation only, we define a "SkewKurt Index" as the sum of the assigned values, with 4 being the largest index value (normality likely) and 0 the smallest (normality highly unlikely, i.e. significant Skewness and Kurtosis). The four tests are D'Agostino Skewness, D'Agostino Kurtosis and the Z Scores of Skewness and Kurtosis (2 tests).

Normality tests and skewness tests were performed on several data sets as shown in Table 1, however only the results of Test I, shown in Figures 5 and 6, are presented in this Paper. Normality tests were performed on both square root scaled data and on MLR-regressed data, using the same data set. The results include a comparison against the R-squared correlation, commonly used to determine whether a data set should be accepted as being appropriately valid for use in determination of charge weights for blast designs.

TABLE 1. Summary of Normality Tests

TUNNEL BLAST RESULTS : AREAS TESTED FOR NORMALITY	
Test A	Confined (Pilot) *
Test B	Unconfined (Benching, Pilot Enlargement) **
Test C	All Data with PPV > 1mm/s
Test D	All Data using Packaged Emulsion (Cartridge) Explosives
Test E	All Data using Gassed Emulsion Explosives
Test F	All Data within Range <20m
Test G	All Southbound Tunnel Data (lower overburden thickness)
Test H	All Northbound Tunnel Data (higher overburden thickness)
Test I	All In-Borehole Vibrographs (Rock-Rock Transmission) only ***

* assumed confined
** assumed lesser confinement
*** results presented in this Paper

Continuous Run Results

Figures 5A~5F shows the results of the continuous run on a data set of raw results. No outliers were removed. Even though R-squared values remained relatively high throughout the data set, the normality tests (Normality Index and Skewkurt results), while initially good, for the first 150 data points, thereafter indicated a poor fit to the normal distribution, for the remainder of the entire run. This may be due to the addition of data from blasts where the geology or topography was different than for the first 150 points or charge weights changed markedly, or where sensitive receivers changed. Between 50 and 150 points the b-value for the added data was fairly constant, but thereafter changed markedly. The K value remained fairly constant throughout the tests sequence. The "a" index indicated the selected data best fits a square root scaled distribution.

The following nomenclature is used in the following plots:
LR = Linear Regression using fixed exponent a
(labelled LR on plots)
MLR = Multi-linear Regression using fixed exponent a
(labelled MLR on plots)

Windowed Run Results

Figures 6A~6F show the results of the windowed run on a data set of raw results. Again, no outliers were removed. The R-squared values remained relatively high (>0.7) until point 250 of the data set, at which point the normality tests (Normality Index and Skewkurt results), while initially acceptable for the first 200 data points, drop to show poor normality. As the window moved along the data however and the old data is "removed" and new data points added to the data set, the fit to the normal distribution again improved, as did the R-squared results. While not passing the normality test for every single window in the entire data set, the results still indicate that analysing discrete sets of data specific to an area is likely to be more appropriate than simply continuously adding data points to an existing set. The "a" index variation shows the data best fit a square root distribution up to the point where normality was rejected, then a cube root distribution after the non-normal section of data had been passed.

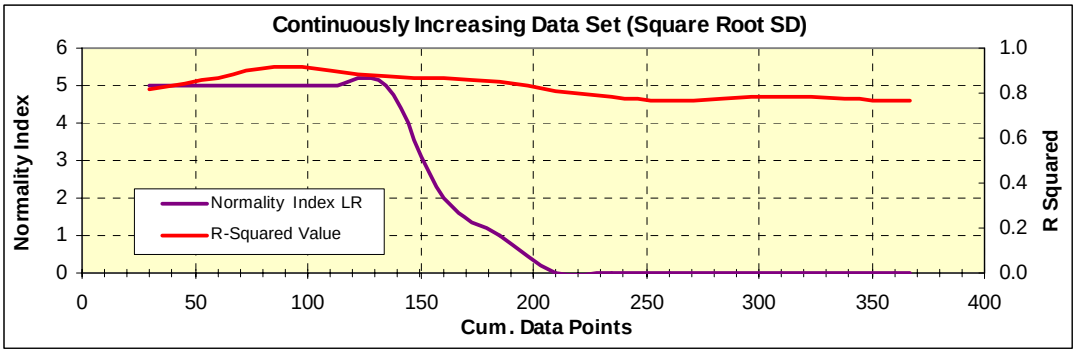

Figure 5A. Square Root Scaled Distance Regression Analysis (Southbound Tunnel in-borehole blasting data)

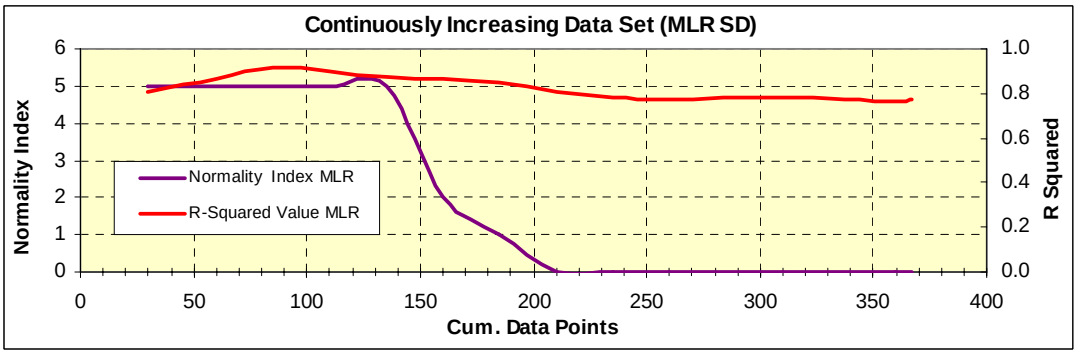

Figure 5B. Multiple Linear Scaled Distance Regression Analysis (Southbound Tunnel in-borehole blasting data)

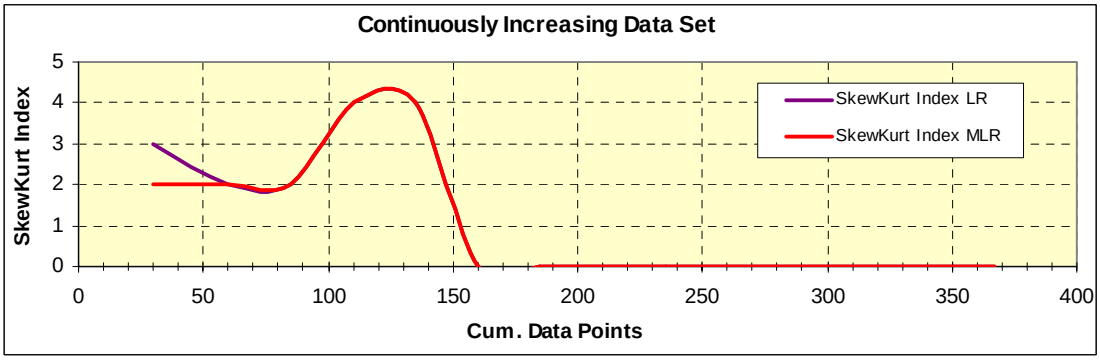

Figure 5C. Skewness/Kurtosis Scores from both Models: Continuous Run

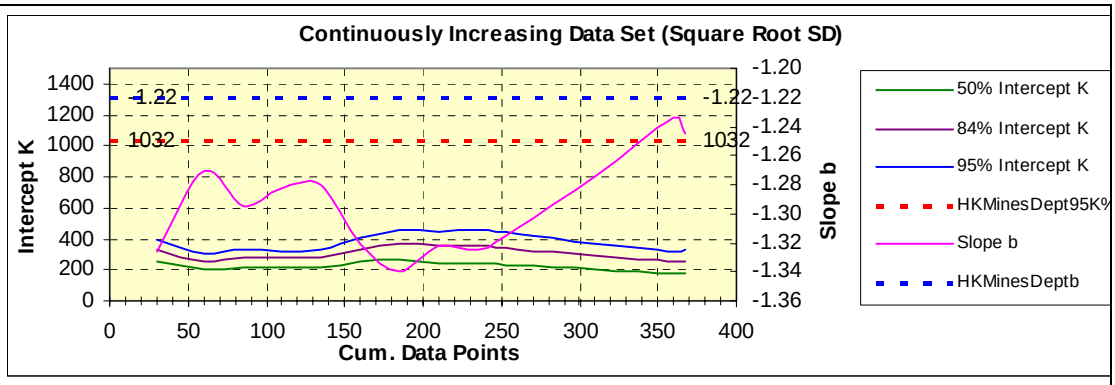

Figure 5D. Blast Constants from Square Root Regression Analysis

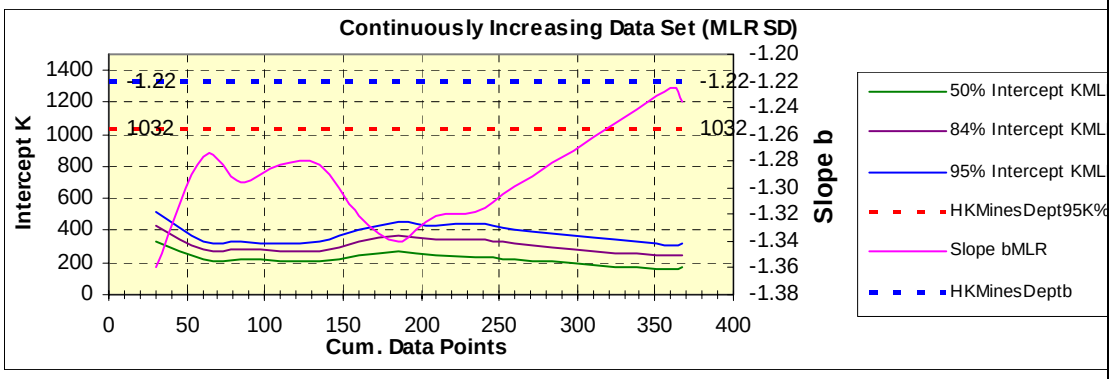

Figure 5E. Blast Constants from MLR Analysis

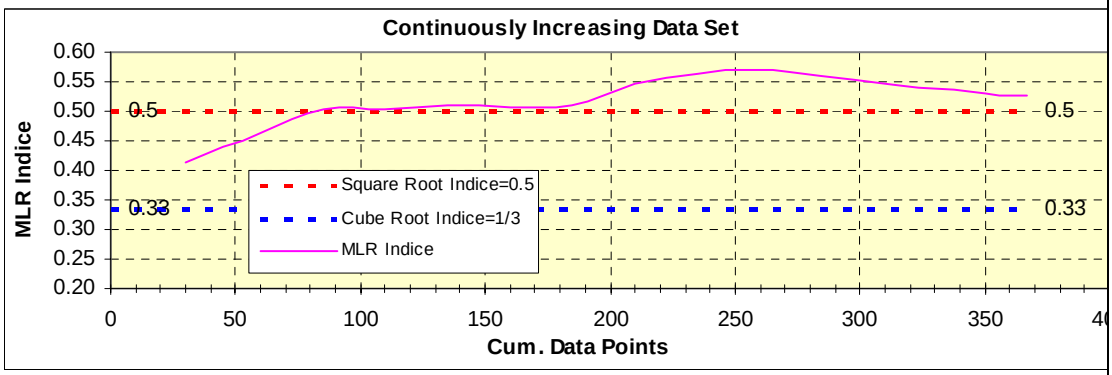

Figure 5F. Index "a" from MLR Analysis(0.5 implies square root scaling, 0.3 implies cube root scaling)

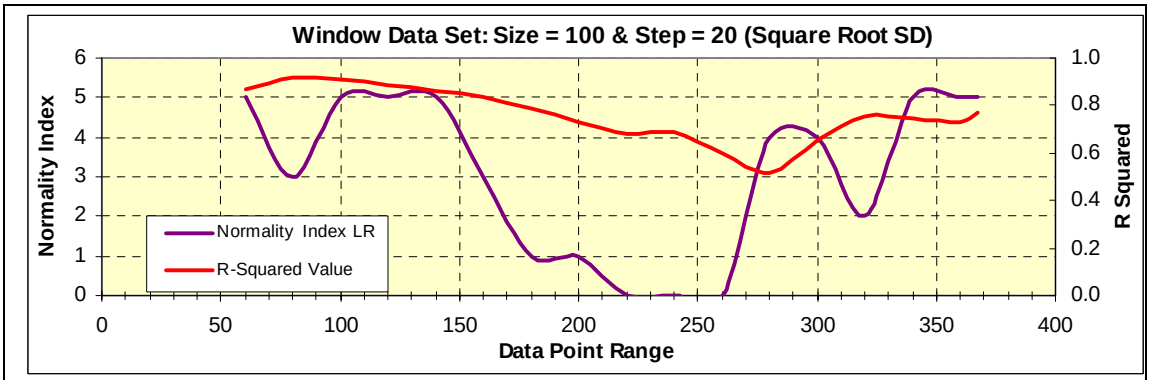

Figure 6A. Square Root Scaled Distance Regression Analysis (Southbound Tunnel in-borehole blast data)

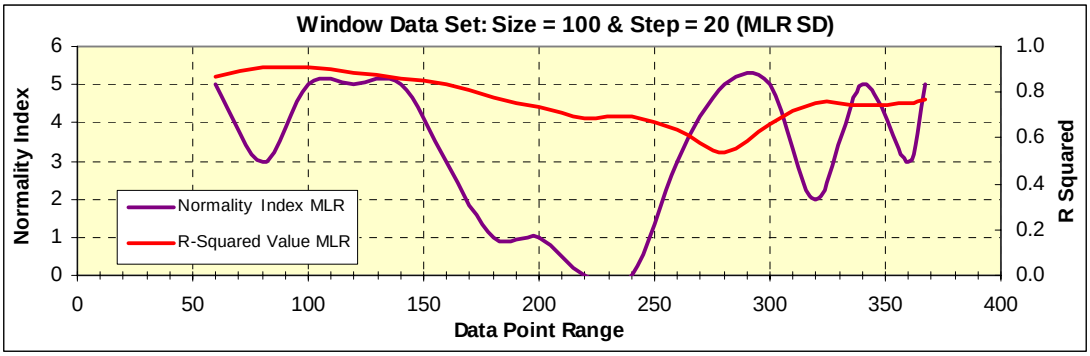

Figure 6B. ML Scaled Distance Regression Analysis (Southbound Tunnel in-borehole blast data)

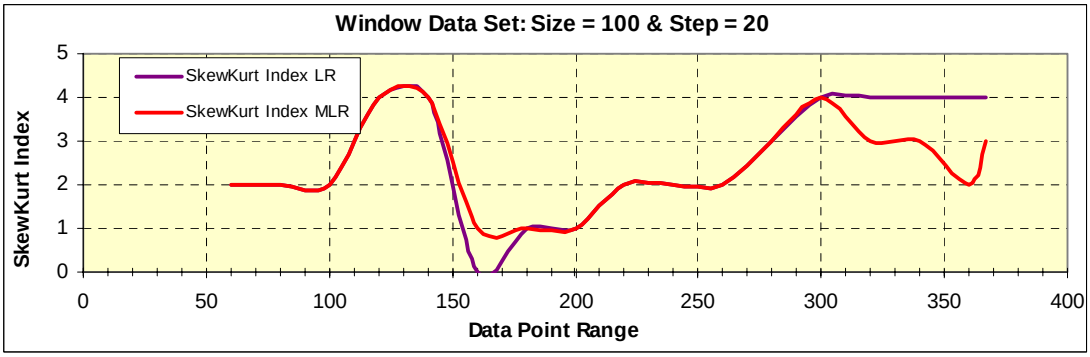

Figure 6C. Skewness/Kurtosis Scores from both Models: Windowed Run

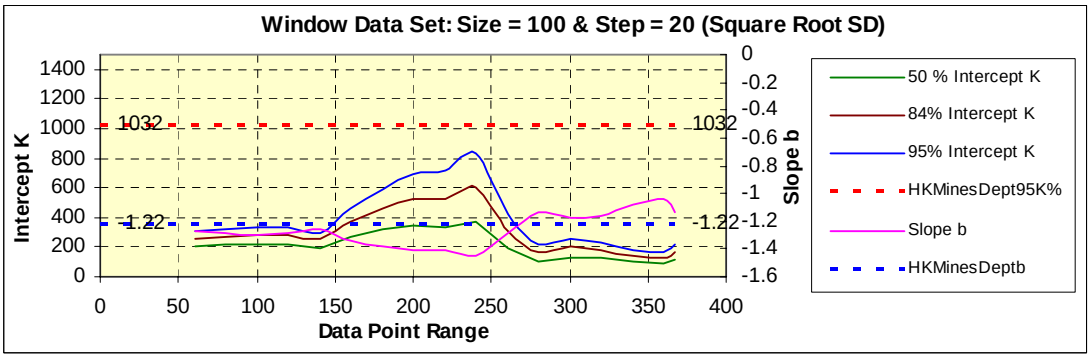

Figure 6D. Blast Constants from Square Root Regression Analysis

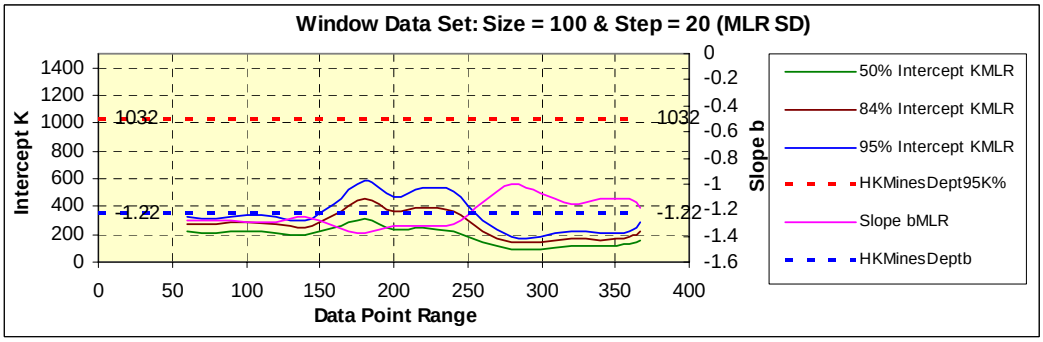

Figure 6E. Blast Constants from MLR Analysis

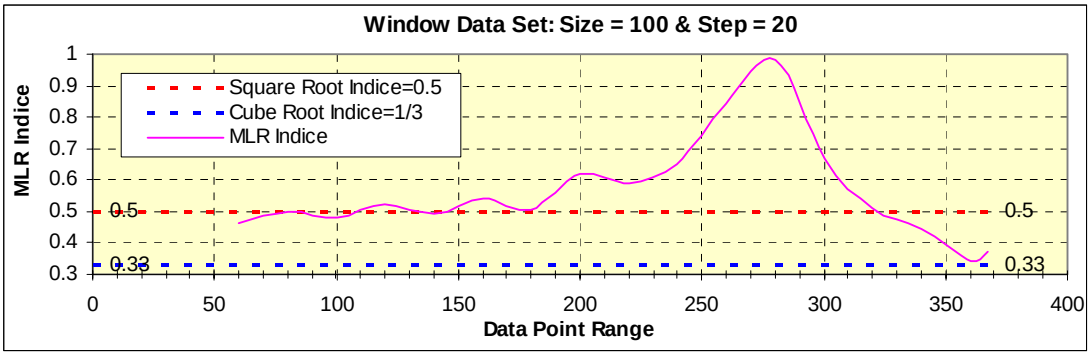

Figure 6F. Scaled Distance Index from the MLR Analysis

CORRELATIONS TO TUNNELLING AND BLASTING PARAMETERS

Chakraborty et al (2004) identified and proposed to link together several tunnel blasting parameters into an index, termed the Tunnel Blasting Index (TBI). While this Paper does not propose any correlations with this Index, an attempt to identify the most sensitive parameters which affect blast vibrations has been conducted. Chakraborty et al (2004) found that the 7 most influencing parameters (from 15 originally identified) on resultant blast vibrations can be divided into 3 groups comprising Rock Mass Parameters, Tunnel Configuration Parameters and Blast Design. The sensitivities of recorded blast vibrations to the following parameters were thus reviewed:

(a) surface wave transmission at the portals;
(b) close-in blasts;
(c) transmission medium;
(d) explosive type: Emulsion (vs. Cartridge) explosive;
(e) confinement as defined by full face or enlargement blasts; and
(f) powder factor.

With reference to Figures 7 to 9, the results of PPV vs. scaled distance plots for cases (a) to (f) are discussed below:

Portal Zone (within 10m of Tunnel Portal)

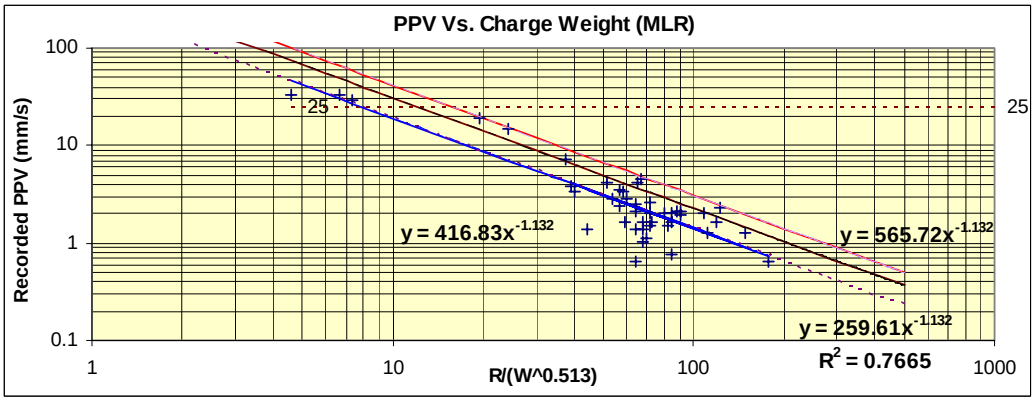

Figure 7. Multi-linear Regression Analysis of data within 10m of North Portal – Outliers Removed from Data Set

From Figure 7, square root scaling appears the most appropriate based on the limited data set, with a=0.513 close to 0.5.

Close-in Blasts

Figure 8 shows data extracted from the data set that falls within 20 m of a blast face within the tunnel. Outliers were identified in the full data set and have been removed for these regression analyses, as an R-squared value of 0.7 is considered the minimum acceptable value (Eltschlager, 2001). The multi-linear regression index "a" tends toward cube root scaling (0.382), which agrees both with Lucca (2003) and the close-in blasting equations published in USACE (1989).

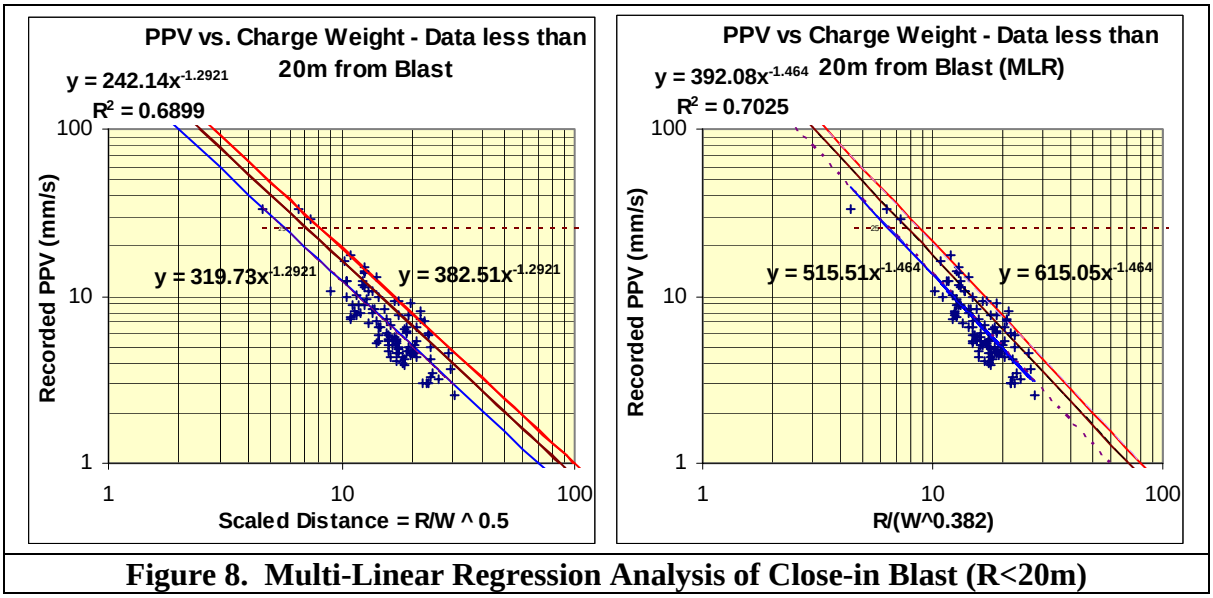

Figure 8. Multi-Linear Regression Analysis of Close-in Blast (R<20m)

Transmission Medium

The Shatin Heights Project offered the unique opportunity to review the amplification effects due to surface reflection and rock-soil transmission vs. rock-rock transmission, as the same blasts were monitored with both in-borehole transducers in rock and by surface instrumentation.

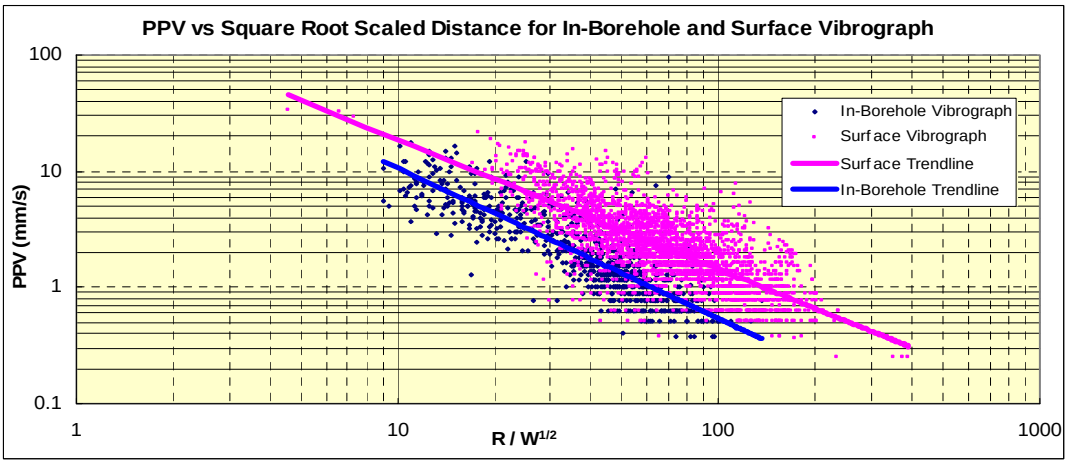

Figure 9. Vibration Monitoring Records by Surface/In-Borehole Vibrographs

Figure 9 shows plotted surface vibration levels in the order of twice that of the in-borehole results at equivalent scaled distances.

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Explosive Type: Bulk Emulsion vs. Packaged Cartridge Emulsion explosive.

Bulk emulsion is theoretically more efficient at energy transfer than (tamped) cartridge emulsion products (McKern et al, 2000), due to bulk emulsion providing improved coupling between the side walls of the blasthole. In comparison, (tamped) cartridge emulsion products often behave as decoupled charges, as energy is lost through gas expansion into the borehole cavity following detonation. Improper tamping reduces the efficiency of the explosive and the charge can act as a decoupled charge, actually reducing the vibration. McKern et al (2000) noted that insufficient tamping can lead also to low fragmentation and reduced performance. As noted by Jimeno et al (1995) if the energy imparted to the rock is not enough to break the rock due to too high confinement (stemming too long), or too high burden, higher vibrations may occur. To determine if there was any difference in vibration levels due to explosive type used, plots of vibrations due to both explosive types is shown in Figure 10, over a range of scaled distances.

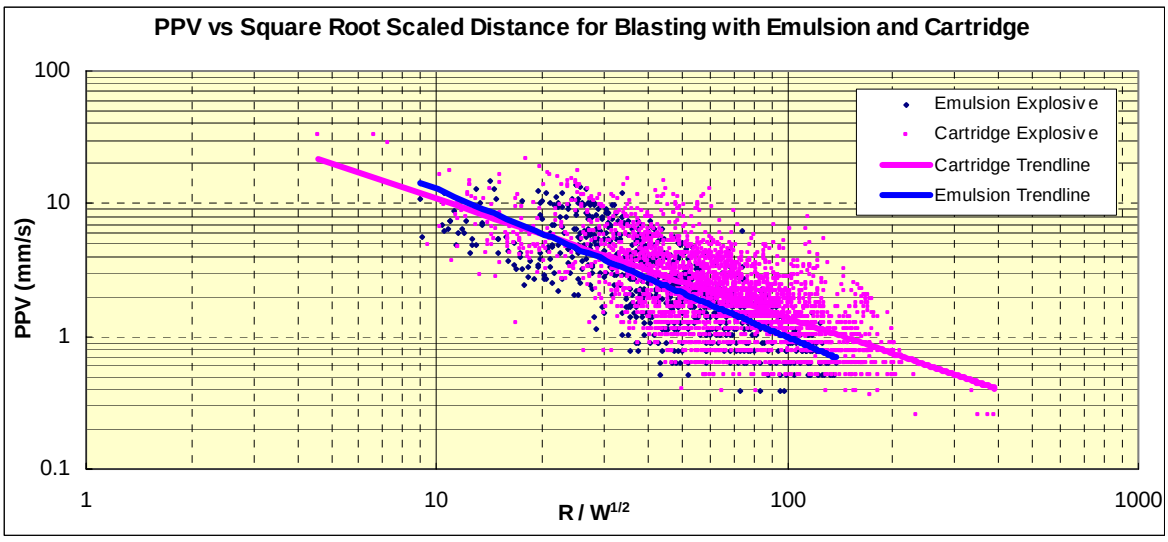


Figure 10. Vibration Monitoring Records (Blasting with Emulsion and Cartridge Explosives)

The vibration review shows a slight trend of lower PPV for bulk emulsion at (square root) SD above 20. Between SD = 10~20, the vibration levels are slightly higher, although there is little discernible difference between the two results. Note that a direct comparison is difficult to make as bulk emulsion was used almost exclusively at charge weights above 2.5 kg/delay, whereas cartridge (packaged) emulsions was used below 2.5 kg/delay.

Blast Confinement

Confinement was assumed, for purposes of reviewing this data set, as either the pilot, top-heading or full face blasts, whereas unconfined was defined as bench blasting and blasting for the enlargement where two free faces were available prior to blasting. High confinement should theoretically increase PPV levels as noted by Li & Ng (1992), Blair & Armstrong (2001) and the ISEE's Blasters Handbook (1998), which noted that "*if a charge is deeply buried with no free face nearby, the rock is not displaced (although damaged around the explosive) and more of the energy goes into seismic waves…lack of confinement has the opposite effect*".

It was initially expected that the pilot tunnel/full face blasts would behave as confined blasts in relation to the subsequent less confined stoping (enlargement) blasts. Surprisingly, the results of the confined vs. unconfined blasts shown in Figure 11 indicate a trend of lower PPV at (square root) SD above 15, but below SD = 20, the predicted PPV vibration levels being slightly higher. While bench blasting may be considered largely unconfined, the enlargement blasts are considered to be partially confined.

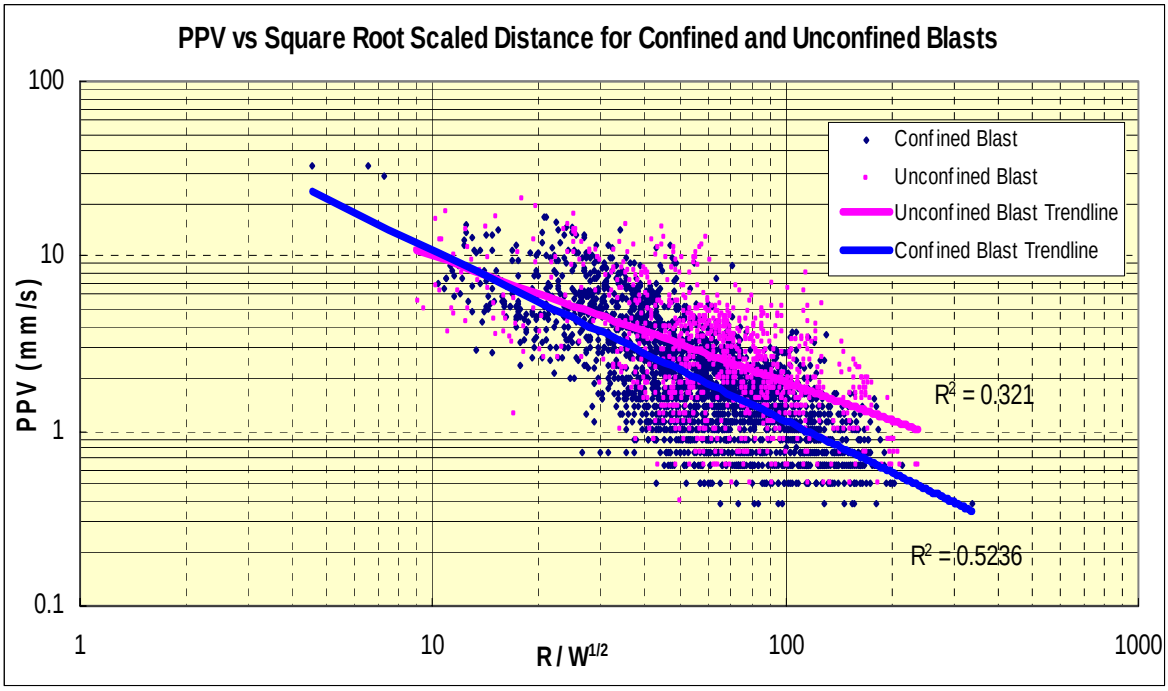
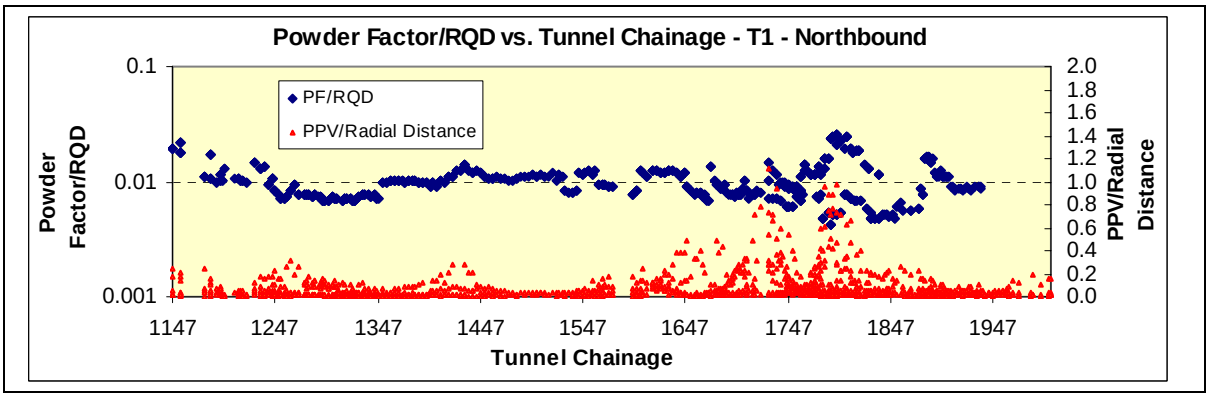


Figure 11. Vibration Monitoring Records for Confined and Unconfined Blasts

Powder Factor

Figure 12 shows the variation of powder factor (PF) normalised by RQD, and PPV normalised by radial distance. Normalisation of PF by RQD was chosen as it represents energy (PF) imparted into the rock, with poorer rock quality not requiring as high a PF for fragmentation as stronger rock. The PPV is normalised by the absolute distance R, as the charge weight time is indirectly included in PF. There is no discernible correlation between the two, indicating PPV is more strongly controlled by other factors, such as absolute distance. Note the scatter of PF used around Ch. 1700 to 1900 results from powder factors used first for pilot headings, then back over the same chainage for enlargement blasts



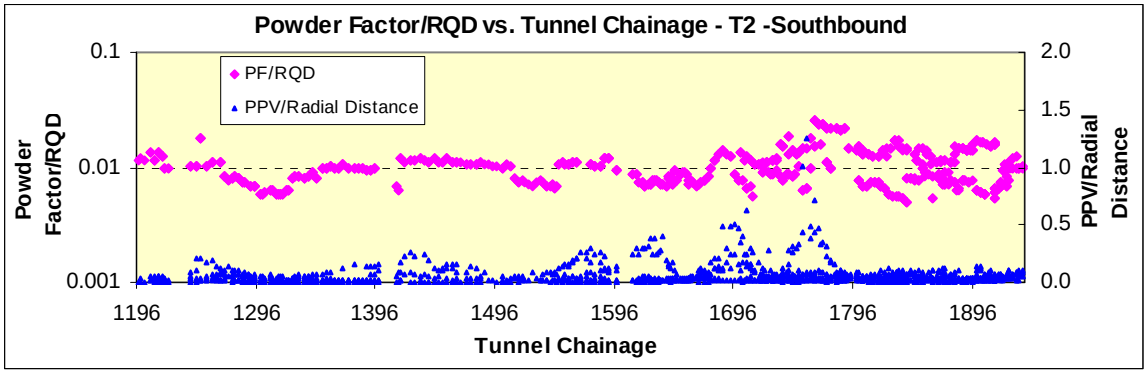


Figure 12. Variation of Powder Factor (normalised by RQD) and PPV (normalised by Radial Distance) along Tunnel Chainage

CONCLUSIONS

The blast data on this site follow a lognormal distribution only in specific sections of the tunnel. As a tunnel, unlike a quarry, cavern or deep excavation, always moves away from the plan location where blasting has previously taken place, the transmission media, its geometry and its properties change with each advancing face. It is thus considered appropriate for the attenuation constants and the scaling law itself to be checked as construction progresses. For a tunnel, this does not simply mean adding more data to the existing data set and producing K and b indices for the full set. A new set of independent data points should be checked at defined chainages or with changes in topography and geology, against the previous data set. Charge weights should thus be calculated from blast constants derived from data recorded reasonably close to the current blast face or, in the absence of previously recorded data, constants which are derived from blasts at locations which closely match the geology and topography of the current tunnel section.

The cube root scaling law, which has a strong theoretical basis, is still in common use in checking against actual blast results and is considered an appropriate expression for determining charge weights that maintain PPV within acceptable limits. However the above results show that it was not always appropriate for the Shatin Heights Tunnel blasts, with the exception of possibly close-in blasts.

The outliers in the results once again emphasise the need for proper geophone mounting. The International Society of Explosives Engineers (ISEE) publishes "good practice" guidelines (Eltschalger 2002).

Numerous parameters influence blast vibrations recorded at any sensitive receiver; however the parameters which had the largest controlling influence on the Shatin Heights Tunnels were absolute distance, charge weight per delay and transmission medium.

While statistics is regularly used as a basis for any review of blasting vibration results, it should not be used in isolation. Dowding (2004) observed that "*any statistical review should not exclude the practical issues affecting amplitudes such as geology, geometry, timing, mounting of sensors, variable detection with distance and amplitude and the relatively unimportant information obtained at very close distances (i.e. if the criterion is going to be 10 mm/s, and never closer than 100 m, the relevance of amplitudes of 100mm/s at 20 m is not great, nor is 0.3 mm/s at 3000 m). Statistics can never replace a careful design with accurate consideration of timing, burden, etc. Systematic changes such as a 10% miscalculation in the*

burden, differences in the propagation path and the use of pyrotechnic delays with their inherent timing errors, all affect the results."

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STRAIGHT OR CURVED SEWERS? PIPE-JACKING OPTIONS IN HONG KONG

Wilson W.S. Mok[1]

Abstract

The conventional open trench method has been commonly used in Hong Kong for construction of pipelines in the past. This may require the occupation of a works area in the carriageway/footway for a long time, causing disruption to traffic, necessitating major diversions of existing utilities, generating environmental problems and attracting public complaints. The deep excavated trenches also often require massive temporary works and dewatering, inducing significant ground movement. In order to minimize such effects, trenchless techniques have therefore been introduced, starting in a small scale (using simple a TBM or heading method for short sections) in late 1980s, to large contracts (using different operating modes for long sections) in recent years.

This presentation focuses on the works, in particular the problems encountered and the performance of works, carried out by the Drainage Services Department (DSD) of the Government of the Hong Kong SAR under Contract No. DC/2000/11 – Wan Chai East & North Point Sewerage – Trunk Sewers, which involved the construction of 4km of 600mm-, 1200mm- and 1800mm-diameter sewers, with a depth ranging from 3.9m to 18m, using slurry-operated TBMs in highly variable ground conditions in two busy urban areas. This was the first time that curved alignment sewers have been constructed in Hong Kong, using the pipe-jacking method, with one tunnel segment having a 404m-long S-curve alignment that also entailed tunnelling through and being received in, already constructed permanent shafts – another first in Hong Kong. Suggested areas for improvement in both design and construction are also discussed.

Successful completion of this Contract in Hong Kong gives the Government and Utility Undertakers additional proof that tunnel technology is moving forward and curved tunnels are more manageable than ever.

INTRODUCTION

Scope of Works

Contract No. DC/2000/11 was one of the five contracts under the Sewerage Project, managed by DSD, to improve the wastewater infrastructure and capacity in the Wan Chai East and North Point areas so as to meet current and anticipated future development needs. Black & Veatch (B&V) has been responsible for the design and supervision of construction of all of

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the contracts under this Project. This HK$427 million Trunk Sewer Contract – the main focus of this paper - started in May 2002 and was completed in December 2005. The Contractor was the Leighton – Kumagai Joint Venture.

The works, under the Contract, comprised the construction of two lengths of trunk sewers. One section is 1 km long which runs along Yee Wo Street, Hennessy Road, Percival Street and Gloucester Road in Causeway Bay. The other section is 2.9 km long and runs along Electric Road and Java Road in North Point. There were 27 nos. temporary shafts in between the trunk sewers, which were used as either a jacking shaft or a receiving shaft, to enable the tunnelling works. These shafts were also used for the construction of the permanent shafts for future maintenance, after the completion of the sewers.

The location of shafts and the tunnels in between are shown in Figure 1.

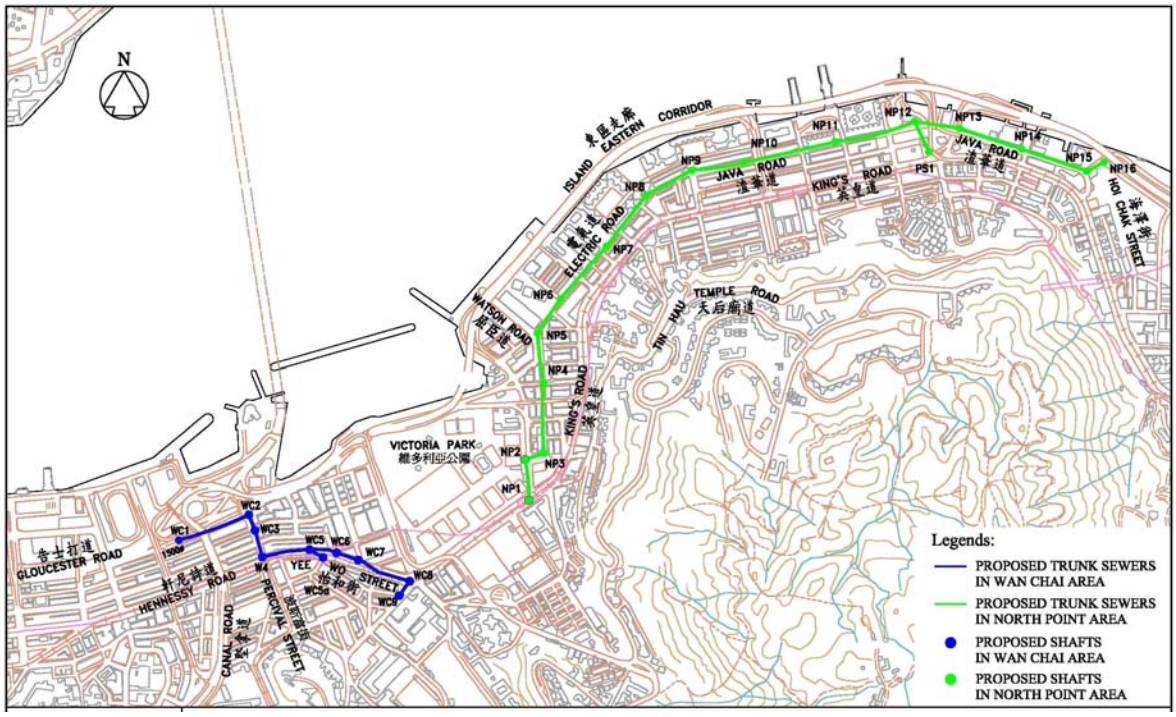

Figure 1. Layout Plan of Trunk Sewers

Planning

For many completed similar projects, most of the jacking and receiving shafts were designed to be located at heavily trafficked junctions with the alignment of pipeline in between confined to short but straight sections. Full-scale investigation and trials are seldom carried out during the design stage to explore all possible constraints, such as the presence of underground utilities and services, the traffic conditions and the underground obstructions and conditions. This always ends up with relocation of shaft and in changes to tunnel alignment during the construction stage. This would, of course, induce risks to the project due to uncertainty as only limited knowledge of sub-soil conditions and the potential obstruction of utilities and services at the new shaft locations was available. As a consequence, the works were often delayed and claims were received, incurring additional expenditure.

Learning from previous experience, B&V advised DSD to carry out an advance contract (named as Contract DC/99/11 – Advance Works) to carry out further investigation on various

aspects so that the results would be available for the tenderers to review during the tendering period. The aim of this arrangement was to provide as much relevant information as possible to the tenderers such that more reasonable tender prices would be formulated and the construction risks and the associated claims and disputes could be substantially reduced.

In the Advance Works Contract, full-scale trial runs were conducted for the tentatively selected shaft locations, to ensure that their implementation would not adversely affect the traffic. At certain locations, the shafts had to be shifted towards side streets. Therefore, some of the sewers were inevitably changed to curved alignments. Trial trenches were carried out to expose the existing utilities to facilitate advance co-ordination and planning for utilities diversions. Upon finalizing the optimum locations of the shafts, boreholes were sunk to determine the ground conditions, with soil samples obtained for laboratory testing.

Geological Conditions

The trunk sewers can be generally grouped into two depth ranges. Shallow sewers with depths in the range of 5.8 to 7.6m are mainly located in Causeway Bay. The ground conditions generally consist of Fill with boulders, with SPT'N' values ranging from 3 to 17, overlying thin layer(s) of marine deposits and/or alluvium.

Deep sewers, with depths between 10 to 18 m, are mostly located in the North Point area. The top layer (5 – 14 m thick) is mainly Fill, with SPT 'N' values ranging from 3 to 20 and boulders of different content, and is generally underlain by marine deposits, alluvium, completely decomposed granite, and moderately to slightly decomposed granitic bedrock at depth.

The inferred geological conditions along the two trunk sewers are shown in Figure 2.

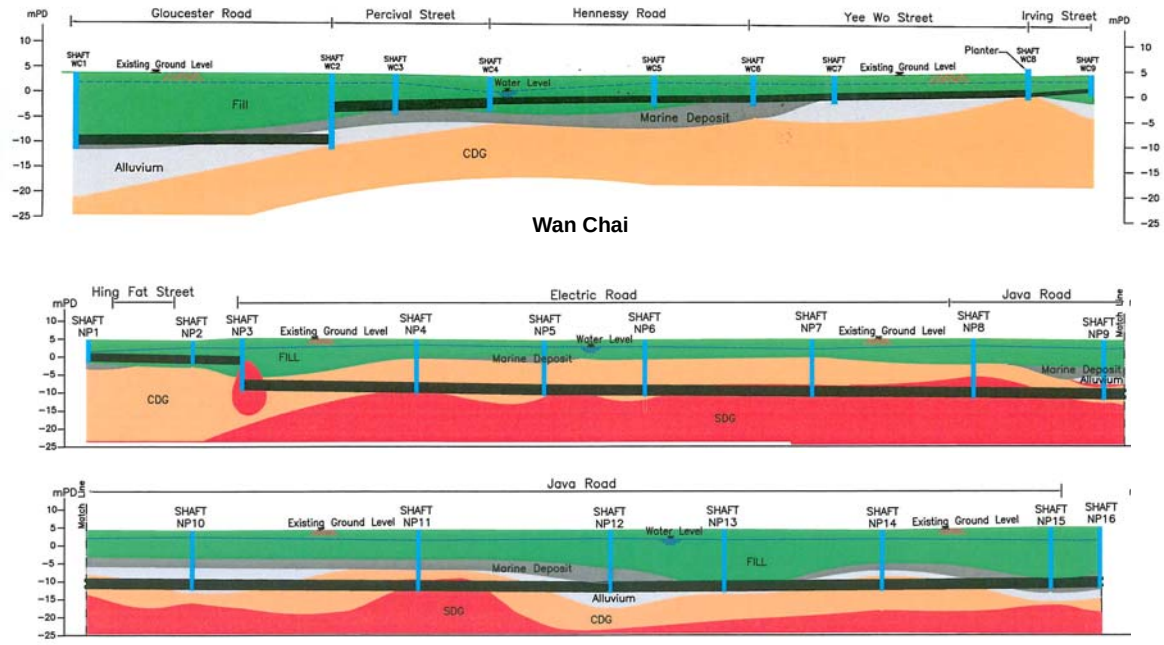

Figure 2. Inferred Geological Profile Along Trunk Sewer

TUNNEL DESIGN

Alignment of Pipeline

Straight sewers were adopted for the sections where no obstructions were present in between the jacking and receiving shafts. For those sections with site constraints such as lot boundaries of buildings, traffic, highways structures and major utilities/services etc, curved alignments were required. Due to operation and maintenance requirements, the curved alignment was not considered suitable for sewers with a diameter less than 1200 mm. After consulting with TBM manufacturers, the alignments of the curved sewers were fixed, with a curvature range from 340 m to 1965 m in radius, so as to ensure proper alignment control by the TBM, smooth loading transfer between pipes and watertightness at pipe joints.

Jacking Pipe

Jacking pipes manufactured by Doran were adopted. With a concrete strength of 50MPa, these pipes were checked to ensure that they had sufficient capacity to withstand the overburden pressure and the required jacking force for both straight and curved pipelines. In order to allow a more uniform loading distribution and have higher axial loading capacities, 18mm thick wooden packers were provided at the pipe joints for better dispersion of the eccentric loading, for pipelines with a minimum radius smaller than 1000 m. For other pipelines, 9 mm thick wooden packers were used.

Watertightness at pipe joints with an angular deflection of 0.5° was tested in the pipe manufacture factory under a simulated water pressure for the deepest pipeline.

TUNNEL CONSTRUCTION

Tunnelling Method and TBMs

As indicated by site investigations, the jacked pipelines in the Wanchai area would pass through a layer of filling materials with boulders at different locations, whereas those in the North Point area would tunnel through an alternating, highly variable ground made of marine deposits, alluvium, CDG and M/SDG bedrock, with non-homogeneous materials.

Figure 3. Appearance of TBMs Adopted

Based on the above, the records of underground artificial features such as old seawalls, disused piles, left-in sheetpiles, and the lengths and sizes of the pipelines, the earth pressure balance method was not considered suitable. Therefore, slurry pressure balance method, with the deployment of 4 nos. full-face slurry TBMs equipped with disc cutters for rock excavation (Figure 3), was selected to construct the two trunk sewers. The key information is summarized in Table 1.

Table 1 – Details of TBM

TBM	Herrenknecht AVN 1800T	Lovat 2000 mts	Herrenknecht AVN 1200TC	Herrenknecht AVN 600
Length	14m	9m	6m	5m
Outer Diameter	2150mm	2170mm	1450mm	780mm
Sizes of Pipeline to be Jacked	1800mm	1800mm	1200mm	600mm
Ground Conditions Likely to be Encountered	Hard rock and highly varied grounds	Soft grounds with occasional boulders	Filling areas with boulders	Filling areas with boulders
Allowable Jacking Load	8548kN	10000kN	5088kN	2600kN
No. of Steering Jacks	4	3	3	3
Cutting Wheel	Dome type with 11 nos. 305mm diameter single and double disc cutters	Semi-dome type with 11 nos. 305mm diameter single and double disc cutters	Dome type with 9 nos. 250mm diameter single and double disc cutters	Dome type with 6 nos. 250mm diameter single and double disc cutters

Operations

Both the Herrenknecht AVN1800T and Lovat 2000mts TBMs were equipped with an air-lock chamber, to enable inspecting the condition of disc cutters and making the necessary replacement. The Herrenknecht AVN1800T TBM had a telescopic section at the rear to provide sufficient capacity for smooth excavation in hard ground.

Before the commencement of pipejacking a drive, sub-surface settlement markers, in the form of steel rods, were installed at 5 m intervals along the centre line of the pipeline by coring through the concrete slab. Another 2 rows of markers at 5m offset were also installed at the two sides of the pipeline. Settlement monitoring was carried out 4 times daily during the course of works. The works would be suspended when the measured ground settlement suddenly increased or exceeded the preset limit. An investigation would be carried out to identify its cause, and any necessary remedial works would be executed, prior to resumption of the works.

For all TBMs, bentonite based slurry, with a viscosity of 40 - 50 Marsh sec. for sandy ground (equivalent to about 4% of bentonite by weight), was constantly pumped into the excavation chamber, through a slurry charge pipe. This forms a cake over the excavation surface, to stabilize the tunnel face and also prevent the ingress of groundwater (Figure 4).

The mix of slurry had to be adjusted from time to time to suit the ground conditions. Occasionally, polymer was required to be added to improve the performance. For clayey ground, no slurry was used but water under pressure was injected for such purposes. The slurry and water pressure was constantly monitored and adjusted to avoid blowing out the ground.

Lubricant, in the form of bentonite (with thinner mix) or polymer (under adverse ground condition), was injected to the overcut outside the pipeline, at a spacing of every 18 m. The injection was carried out through the 2 nos. of lifting holes at the crown of the jacking pipe and the two prefixed grout holes in its axis. It was controlled by an automatic system with pre-set timer and volume of flow. The lubricant can not only reduce the frictional force along the pipeline during the jacking operation, but also increase the shear strength of the surrounding soils, thus maintaining the stability of the overcut to prevent closing-up.

Intermediate jacking stations (Figure 5), each with 8 to 12 nos. 80T hydraulic jacks, were installed at pre-determined locations of the pipeline, usually at 80 - 100m intervals, to avoid excessive loading in the main jacking station. These stations were operated one by one, with the jacks in a station extended 500mm each time. Some of the stations were not used due to consistently low jacking force after the application of bentonite /polymer, and this also sped-up the jacking operation.

Water leakage was found in some of the stations during the driving of the curved sewers. It was later found that the rubber gasket provided at the rear part of the station was torn out after the repeated extension and retraction of the hydraulic jacks. The situation was rectified by reinforcing the steel rims at its two sides from the original 300 mm spacing single face spot weld to 100 mm spacing double spot weld.

High water pressure was present in some of the deep drives, pushing the TBM and the first few jacking pipes back when they were launched into the ground. This resulted in the necessity for a steel bracket temporarily fixed in the TBM/pipe and rigidly connected to the surrounding temporary works as support during the pipe connection.

Failure of the grout block was experienced at one location of receiving shaft, causing significant ingress of water and migration of soil, through the receiving eye. After this incident, the jacking and receiving eyes for the remaining drives were modified from an original single concrete structure to a complete concrete ring wall cast against the sheet/pipe piles, to increase their rigidity, with double layers of rubber seal provided at the eye locations.

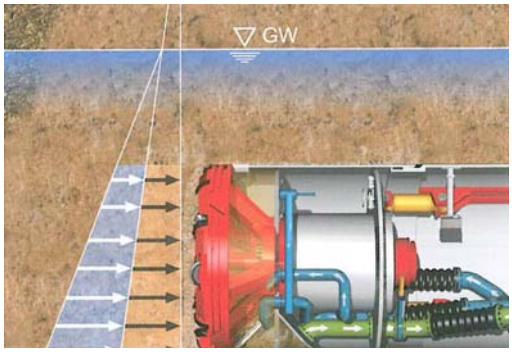

Figure 4. Mechanism of Slurry Pressure Balance

Figure 5. Intermediate Jacking Station in Operation

The tunnelling through clayey ground often caused slow transportation rate in the slurry discharge pipe due to the plastic material accumulated in the inlet of the slurry disposal chamber. High pressure water had to be used to disperse such material through the nozzles provided nearby. Pipe blockage was also found when obstructions such as timber, steel bars, etc., were encountered, resulting in the long time to dismantle the respective sections of the pipe for clearance.

A simplified diagram showing the lubrication and conditioning for slurry pipejacking works is given in Figure 6.

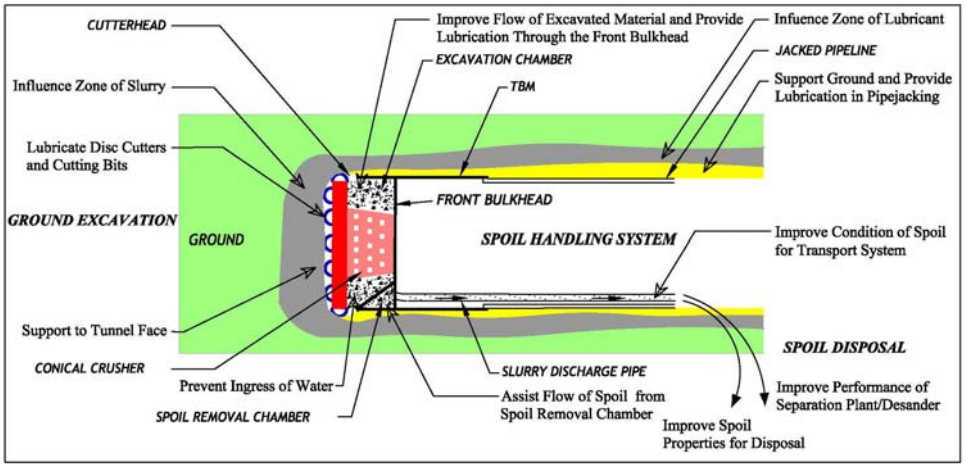

Figure 6. Typical Soil Lubrication and Conditioning for Slurry Pipe-jacking Works

Control of Line and Level of Pipeline

A laser guidance system consisted of a laser device installed at the rear of the temporary shaft and a target plate equipped in the machine can of the TBM was used for checking the line and level of a pipejacking drive. The relative position of the laser beam on the target plate was automatically transmitted to the control room for reference. For curved drives, a SLS-RV laser guidance system was used to control their alignment. Traditional survey check was carried out every 30m long of driving, using Gyro theodolite.

Inspection of Cutting Wheel and Disc Cutters

For sandy ground, activation of the TBM air-lock was required for the inspection and replacement of the disc cutters when little or no advancement achieved under consistent high torque pressure and jacking force, and/or metallic pieces were found in the desander. On top of the compressed air working environment, PU grout, a jelly form of chemical grout which swells rapidly with groundwater, was applied to the excavation face and the periphery of the rear of the TBM, to prevent ingress of water, before the access opening in the front bulkhead of the TBM was opened for such inspection and replacement. When disc cutters were replaced in rock pockets or clayey ground, activation of the air-lock was not required due to the low permeability of the materials in the excavation face.

Disc cutters had to be replaced due to normal wear and tear, or when they were damaged by obstructions. The damage could also occur by the dynamic impact of the TBM, resulting from the sudden change in ground condition from soft to hard, or the presence of unfavourable rock joints in the excavation face embedding the disc cutters. Typical conditions of worn-out and damaged disc cutters are shown in Figure 7.

Replacing disc cutters during the course of pipejacking works was time-consuming as for each replacement, the dismantling of the complete bearing system of the disc was required. Also, the rock mass in front of the cutting wheel had to be trimmed locally by hand tool to give sufficient room for the installation (Figure 8). On average, each replacement took 3 hours for completion.

Figure 7 – Conditions of Damaged Disc Cutters

Figure 8. Breaking of Rock Face for Installation of New Disc Cutter

THE CHALLENGE

Construction of a 404m Long S-shape Pipeline with Completion of R.C. Permanent Shaft at Intermediate Temporary Shaft and Receiving Shaft in Advance

Shaft NP4, located at the junction of Electric Road and Wing Hing Street, was originally designed as a jacking shaft for 2-direction driving. Since the implementation of the temporary traffic management scheme, a significant number of complaints, mainly related to public inconvenience, traffic congestion, loss of business and environmental issues, were received.

In view of the public's concern about these works, DSD requested B&V and the Contractor to review the construction programme and working methods to determine whether there was any way to expedite the works and minimize the disturbance to the public.

After several discussions amongst DSD, B&V, ER and the Contractor, it was established that the two drives between the shafts of NP3 to NP5 could be combined into one single drive (404 m long on a S-shape alignment), from NP5 directly to NP3 (through NP4), such that Shaft NP4 would not be used as either a jacking or receiving shaft.

It was also proposed to construct the complete permanent shaft of NP4 (intermediate shaft), except the benching, and the base slab and shaft wall of NP3 (receiving shaft), before the commencement of this long pipejacking drive NP5-NP3 (Figure 9).

The benefits of such proposal included eliminating the need for setting up the pipe-jacking equipment at Shaft NP4, thus reducing the disturbance to the public. It also allowed a much earlier completion of the shaft, as it would be constructed and reinstated

before the commencement of the pipe-jacking drive.

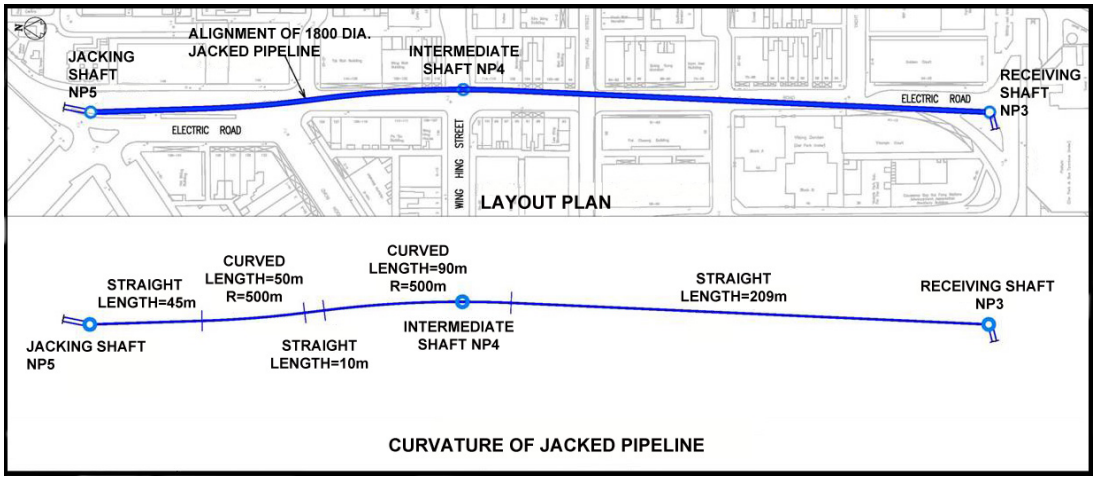

Figure 9 – Plan Showing the S-shape Pipeline

As the S-shape alignment using the pipe-jacking method and the completion of permanent shaft construction before the commencement of the respective pipe-jacking works were new concepts in Hong Kong, the technical feasibility of the proposal had to be assessed and reviewed carefully. This included the design of S-curve alignment to maintain a proper clearance to nearby buildings and existing structures, the proper load to be transferred between the two curves, methods and details on how the TBM could tunnel through a "completed" manhole structure without causing any damage (Figure 10), and the effect of vibration of TBM breakthrough on that permanent structure.

The clearance between the shaft wall of NP3 and the longest section of the TBM, at the narrowest point, with only 100mm, were checked carefully and repeatedly, to ensure that the TBM could be lifted up upon completion of the drive (Figure 11).

To account for the above, a mass concrete filling was provided across Shaft NP4 to confine the alignment of the jacked pipeline and prevent the escape of lubricant when the pipeline was passing through. This also enabled the inspection of the disc cutters and the necessary replacement under a watertight environment.

The gap between the temporary and permanent shafts at the "opening" locations was also sealed up by concrete to prevent ingress of water causing drawdown outside the temporary shaft, leading unnecessary ground settlement.

The speed of the TBM, when approaching the temporary shaft, was reduced to about 10mm/min and the position of the TBM was double checked and adjusted, as necessary, before the TBM was launching therein. The jacking force and the speed of launching was controlled in such manner that the vibration induced (with a maximum recorded level of 20mm/sec) would not damage the permanent shaft.

Upon the completion of the pipe-jacking works, the gap between the permanent shaft and the pipeline (with Hi-rib provided to increase bonding) was properly sealed up by non-shrinkage epoxy grout.

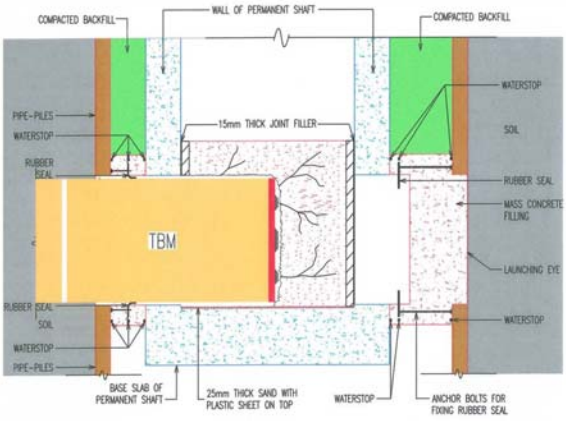

Figure 10. TBM Launching into Intermediate Permanent Shaft

Figure 11. Lifting-up of the Longest Section of TBM from Receiving Shaft

The shaft NP4 working site was completed and reinstated about 10 months earlier than that originally programmed and the 404m long, S-shape pipe-jacking drive NP5 - NP3 was satisfactorily completed in June 2005.

SUMMARY OF PERFORMANCE FOR PIPEJACKING WORKS

TBM Utilization (Expressed in % of Time Available)

Apart from the capacity in controlling the alignment and minimizing the ground settlement, the performance of a TBM is also reflected by the working time for excavation and advancement. For the 4 nos. TBM adopted, the average working time varied significantly from 18% to 55%. It is noted that the low working time for one of the TBMs had been greatly affected by the slow progress in the first drive due to the learning stage, and the stalled TBM problem in another drive.

TBMs also performed significantly different in working time even in similar ground conditions. The drives in soft ground usually had better working time than those in mixed or hard ground, with the highest 72% and the lowest 21%. The presence of high content hard materials sometimes caused damage to the TBM. Frequent repair and maintenance was often required which significantly affected the TBM utilization. However, there was no indication that the straight alignment had better working time and less downtime than the curved alignment.

Jacking Force and Rate of Penetration

The average jacking force required to tunnel through filling grounds for shallow sewers ranged from 70 to 220 T, with an average rate of penetration of 110 to 175 mm/min. This rate was reduced to 70 to 90 mm/min when hard materials were encountered.

For deep sewers in mixed to hard ground, the average jacking force was generally in the range of 290 to 480 T, giving an average rate of penetration of 30 to 90 mm/min. In soft ground such as marine clay or alluvial deposits, the rate of penetration could be as high as 350mm/min under a jacking force of 300 T. A tunnel through a full-face of bedrock had recorded a penetration of 18mm/min under a consistent jacking force of 300 to 330 T.

There was no indication that a higher average jacking force had been required for curved drives in soft grounds, however, a jacking force of more than 750 T had been used at certain

locations of the 404 m S-curve alignment and some of the curved alignments, due to high variations in ground conditions.

Tolerance in Alignment (Contract requirement: 50mm for line and 35mm for level)

In the 3.9 km long pipe-jacking drives, 21.3% of the line (horizontal alignment) and 19.4% of the level (vertical alignment) exceeded the specified tolerance, generally by 2 to 50mm. However, more than 100 mm had been recorded at some locations in a number of drives.

For the straight section (1.1 km long), both the constructed line and level exceeded the tolerance by 16% whereas for the curved section (2.8 km long), the constructed line and level showed an out-of-tolerance by 24% and 20% respectively.

For shallow drives with a depth up to 8 m, the constructed line and level are well within the tolerance as the ground contained no to very little hard material. However, in very loose ground, the control of such was always a problem as the TBM tended to move faster towards soft materials.

For deep drives, with a straight section, tunnelling through soft ground or a ground with little hard materials, their line and level could be generally controlled within the tolerance, except in marine clay and alluvial deposits that significant deviations always occurred.

The driving in hard ground or a ground with alternating soil and rock composition would easily cause the TBM to tilt up as a result of the likely trapping of rock debris below its bottom during the excavation process. The presence of rockhead in the invert of the TBM would also cause the pipeline to slant upward as the disc cutters were unable to cut rock effectively in such a small contact area, causing the TBM to tilt up as a result. This problem appears to be unavoidable but suitable combination of extension and retraction of the steering jacks in the TBM and slow advancement of the pipeline could minimize the excessive deviation.

TBM Daily Production Rate

The average TBM production rate for all the drives was 4.2 m/day. This rate included the time required for repair and maintenance during the course of work, such as extension of ventilation duct, disconnection and connection of power cables, extension of compressed air pipe, installation of jacking pipes, application of lubricant, change of disc cutters, survey check, and downtime such as mechanical fault, electrical fault, unblockage of slurry charge and discharge pipes, broken-down of hydraulic system, malfunctioning of computer system, and the worst, the stoppage of the TBM in ground.

For the Herrenknecht AVN 1200 TBM, the average production rate for the drives was generally in the range 6 – 9 m/day (with a minimum average of 3.3 m/day for the drive which needed to slow down to pass through the narrow gap between the MTRC pedestrian tunnel and the existing trunk utilities). A maximum length of 22 m was recorded in a single day (12-hour shift) when tunnelling through alluvial deposits.

The Herrenknecht AVN 1800T TBM had an average production rate of 5 m/day, ranging from 3.8 m/day for a curve tunnelling through a ground with high content of rock, to 9.7m/day for a curved drive in soft ground. A number of drives advanced more than 10m/day in soft ground, with a maximum length of 18 m, whereas 4m/day was achieved in a

drive through M/SDG bedrock, with strengths of 320 – 350 MPa.

The drives completed by the Lovat mts 2000 TBM showed the average production rate with significant fluctuations, from 1.2 m/day for a curved drive through a ground comprising soft, mixed and hard materials (in which the TBM had stuck in the ground for about 8 months due to obstruction), to 17 m/day for a curved drive in soft ground, with an average of 3.2m/day. The maximum length recorded in a single day was 36 m (15-hour shift) in this drive. In a number of drives, the centre disc cutters were found to be damaged but could not be replaced from the rear of the cutting wheel, affecting the excavation capacity.

Disc Excavation Rate

A total of 254 nos. disc cutters were replaced under different ground conditions for the 21 drives, averaging about 0.5 set of cutters for each shallow drive and 1.5 sets for each deep drive. A disc cutter had to be replaced when it was damaged or had a wear of about 12 – 15mm.

No discs had to be replaced for the Herrenknecht AVN 1200TC TBM during the course of driving, but a number of drives required the replacement of some disc cutters before commencement.

For the Lovat mts 2000 TBM, no replacement of disc cutters was required for the drives in soft ground, however, an average frequency of 0.4 - 3.3 times was recorded in highly variable ground made of soft, mixed and hard materials.

The Herrenknecht AVN 1800T TBM also did not require any replacement of disc cutters for the drives in soft ground. However, it showed an average frequency of 1 - 3 times in highly variable ground, and 1.9 times in hard ground.

The ratio of the need for replacing disc cutters ranged from 5 (for centre discs) to 12 (for gauge discs) for Herrenknecht AVN 1800T TBM, and from 2 (for centre discs) to 8 (for gauge discs) for Lovat mts 2000 TBM, under the mixed to hard and soft to mixed ground conditions respectively.

Depending on the ground conditions, particularly in the location with thick and hard materials, some of the disc cutters had to be replaced again after driving for a short length. This situation occurred in some long curved drives with highly variable ground conditions and the replacement of disc cutters in a 4 m drive length had been experienced in the worst case. Replacement of more than 30 nos. disc cutters during the course of works in three drives had been recorded.

The average service life of disc cutters for each TBM is given in Table 3.

Ground Settlement

No direct relationship between jacking force and surface ground settlement has been observed. However, it depends very much on the excavated material, and the type and density of material in the overburden. The presence of underground utilities and services above the jacked pipeline would also lead to under-estimate of the surface ground settlement. It is noted that settlement increased if unsuitable or no supporting medium was used to stabilize the excavated face. It was observed that the higher the jacking speed, the more the settlement, usually at a general flat gradient.

Table 3 – Service Life of Disc Cutters

TBM	Total No. of Disc Used	Average Service Distance	Ground Composition		
			Hard	Mixed	Soft
Herrenknecht AVN 600 TBM	5	53m	19%	38%	43%
Herrenknecht AVN 1200TC TBM	27	254m	6%	11%	83%
Herrenknecht AVN 1800T TBM	153	115m	30%	19%	51%
Lovat mts 2000 TBM	69	238m	15%	10%	75%

The average measured surface ground settlement, induced by the pipe-jacking work, for most of the drives, is generally less than that predicted. This could be because the overcut, formed by the TBM, was filled up by bentonite or polymer lubricant, and this also helped to stabilize the surrounding soils and prevent closing-up. A number of drives for which the measured ground settlement exceeding the predicted value can be attributed to no slurry having been injected to balance the water pressure in the excavated face as a result of sudden change in ground conditions which may not be immediately noticeable to the TBM operator. In some cases, the presence of hard materials in part of the excavation face also had some effect.

The summary of settlement caused by the pipejacking works is given in Table 4.

The measured settlement between the monitoring markers varied and this could be due to different ground conditions above the jacked pipeline throughout its length, different speed of TBM advancement and the presence of existing utilities and services at the respective locations.

About 74 to 77% of the settlement occurred during and immediately after the completion of pipe-jacking works. However, due to the time required for dismantling the fixtures inside the completed pipeline, such as slurry charge and discharge pipes, electricity cables, water hose, ventilation hose, and the automatic lubrication system, further settlement developed as a result of the tendency of the ground to converge on the pipeline before the grouting material could be applied.

SUGGESTED AREAS FOR IMPROVEMENT

Alignment Design

Notwithstanding maintenance issues for the permanent works, the minimum radius of a curved section is governed by the allowable angular distortion at the pipe joint for the respective pipe size. Therefore the larger the pipe size, the smaller would be the allowable curvature of the pipeline in order that its structural integrity and watertightness can be maintained.

Table 4. Summary of Settlement Caused by Pipe-jacking Works

Drive	Diameter (mm)	Depth to Invert (m)	Total Length (m)	Average Overburden SPT"N" Valve at Shaft Location	Ground Condition Encountered	Average Daily Production (m/day)	Average Measured Surface Settlement (mm)	Predicted Surface Settlement (mm)
Herrenknecht AVN 600 TBM								
WC8 - WC9	600	4.8 - 3.9	53	11	All soft	2.5	5	11.0
						Average Value	**5**	
Herrenknecht AVN 1200TC TBM								
NP2 - NP1	1200	5.9 - 5.4	150	16	All soft	5.8	21	14.7
NP2 - NP3	1200	5.9 - 6.6	60	5	30m soft, 10m mixed and 20m hard	6.6	9	13.7
WC5 - WC4	1200	5.6 - 6.6	160	12	135m soft and 25m mixed	6.0	4	13.2
WC5 - WC7	1200	5.6 - 5.0	165	11	All soft	6.2	15	16.2
WC5 - WC5A	1200	5.6 - 4.7	40	17	30m soft and 10m mixed	3.3	2	16.0
WC8 - WC7	1200	4.8 - 5.0	185	10	All soft	8.8	13	16.2
						Average Value	**12.2**	
Herrenknecht AVN 1800T TBM								
WC2 - WC1	1800	14.0 - 16.5	245	8	All soft	7.0	6	18.6
WC2 - WC4	1800	7.6 - 6.6	145	8	All soft	9.7	11	40.0
NP5 - NP3	1800	14.2 - 13.7	404	32	190m soft, 130m mixed and 84m hard	5.5	5	18.1
NP5 - NP7	1800	14.2 - 15.1	370	26	195m mixed and 175m hard	3.8	3	18.1
NP8 - NP7	1800	15.4 - 15.1	220	39	85m soft and 70m mixed and 65m hard	4.9	3	17.0
NP8 - NP9	1800	15.4 - 15.6	170	39	All hard	3.9	7	16.9
NP15 - NP16	1800	15.0 - 15.8	58	18	All soft	3.9	33	16.7
						Average Value	**6.2**	
Lovat mts 2000 TBM								
NP10 - NP9	1800	15.8 - 15.6	168	11	78m soft and 37m mixed and 53m hard	2.8	25	16.9
NP10 - NP11	1800	15.8 - 16.2	307	10	297m soft and 10m mixed	14.7	13	16.5
NP12 - NP11	1800	16.5 - 16.2	268	12	218m soft, 30m mixed and 20m hard	7.9	23	16.0
NP12 - PS1	1800	16.5 - 17.0	107	10	All soft	5.2	26	15.6
NP12 - NP13	1800	16.5 - 16.4	148	12	108m soft, 20m mixed and 20m hard	6.2	13	15.9
NP14 - NP13	1800	16.0 - 16.4	210	11	38m soft, 53m mixed and 119m hard	3.4	3	16.2
NP14 - NP15	1800	16.0 - 15.8	225	14	All soft	10.2	11	16.5
						Average Value	**15.5**	
					Average Value of All Drives		**11.0**	

Sufficient straight section, say 15m, should be allowed between any two curves for proper transfer of load. A straight section is also required outside the jacking shaft to enable the launching of TBM to be completed in ground before the steering jacks can be operated to change alignment.

Hydraulic Design

A marked difference in invert levels for incoming and outgoing pipes in permanent shafts, of say 200 mm, should be allowed to avoid the possible backflow situation due to the "out-of-tolerance" problem, if any, in the downstream pipeline.

Smaller sewers with steeper gradient should be used instead of large sewers with flatter slope to ensure flow capacity and self-cleansing velocity, due to the likely irregular profile after the completion of pipe-jacking works.

TBM Design

The performance of disc cutters affects the TBM's capacity of excavation and rate of advancement. The cutter change in the excavation chamber is labour-intensive, costly, and risky for the maintenance team, particularly in soft ground. It is important that rock cores are obtained for determination of rock strength before finalization of the design of the disc cutters. It is also important that all the disc cutters, particularly those in the centre, can be replaced at the rear of the cutting wheel, in the middle of the drive, at the right time. Recycled discs made by local shops should be used with care due to quality control.

It is understood that TBM manufacturers have developed a fully automatic hard and software system for detecting hard material in front of TBM by means of sonic soft-ground probing. This system probes ahead for density contrasts in the soil and visualizes the results. With these results, the TBM operator can plan the works in advance and adopt corresponding measures when the TBM is moving close to the "obstruction".

CONCLUSIONS

Certain observations and conclusions may be drawn from the experiences gained on this Contract .

(a) The works carried out under the Advance Works Contract have proved to be extremely useful to the success of the Trunk Sewer Contract, in that the shaft locations basically remained unchanged, except for a few numbers which needed to be slightly shifted to suit the existing conditions, and that less time was used for liaising with utility undertakers for the diversion works. It is suggested that other large scale pipejacking contracts should carry out similar works in the design stage.

(b) Although site investigations had been carried out at the shaft locations in the design stage, the ground conditions varied significantly along the tunnel alignment in some of the deep sewers, affecting the TBM performance. In one occasion, the TBM had to stop operation. It is considered that additional site investigation should be carried out as far as possible at suitable locations between any two shafts, when traffic permits. It could be carried out either during the design stage or as a contract requirements, in the construction stage, such that appropriate, planning, tunnelling method and TBM can be selected to minimize such risks. This could also enable better ground settlement prediction.

(c) For small size TBMs (less than 900mm in diameter), there is no access to the rear of the cutting wheel for replacing damaged disc cutters. It is therefore of paramount importance to determine the ground conditions and whether obstructions exist along the alignment of pipeline. Those obstructions shall be removed, if necessary, by horizontal drilling from shafts, before launching the TBM into ground.

(d) The Hong Kong decomposed soils usually contain corestones and boulders. It is preferable to have the TBM, apart from basic configuration, with the capacity of hard rock excavation, and equipped with an air-lock chamber, a telescopic section and a sensor located in the cutterhead for detection of obstructions ahead of the TBM, such that smooth driving can be achieved and the downtime minimized.

(e) The intermediate jacking stations in some of the drives were not used. However,

they did provide a measure of insurance when unexpected poor ground conditions were encountered or the TBM was not moving in ground for some reasons due to malfunction or replacement of disc cutters, resulting in a collapse of material onto part of the pipeline.

(f) The 404m long S-curve alignment had traveled into the 2 nos. constructed permanent shafts (intermediate shaft and receiving shaft), through the prefixed opening, in a correct position, without causing any damage or instability to the structure, by the Herrenknecht AVN 1800T TBM. The special design joint details ensured that the grouting of the pipeline annulus did not result in any grout influx to the permanent shaft, and the watertightness of the permanent shaft. The mass concrete filling inside the intermediate shaft can however be modified to prevent the leakage of water and slurry when the TBM is cutting through. The success of this drive and other curved drives gives more flexibility and confidence to clients and contractors in programming of the works, particularly in the event of delays.

(g) Jacking forces can be minimized by injecting lubricant and slurry/polymer. Additional intermediate jacking stations are noted to be effective for both the straight and curved drives.

(h) All 4 nos. TBMs had a different degree of deviation from the tolerated line and level in the completed pipelines. By virtue of the ground conditions encountered, a TBM with 4 nos. steering jacks appears to have better control in the curved alignment in most cases, particularly under mixed to hard ground conditions.

(i) The sub-surface settlement markers are an effective tool for monitoring surface ground movement in concrete carriageways during the pipe-jacking works. However, their locations should be aligned in such manner that they are distant from the road joints to avoid causing traffic problem during installation and monitoring, and, more importantly, the requirement of full panel reinstatement, as a result of damage, which would oppose the spirit of using the trenchless techniques.

(j) The skill, response and knowledge of the TBM operator plays an important role in the success of the works. Sometimes, problems arise as a result of human errors. These could be minimized by employment of well-experienced operator who is familiar with the operation of the TBM and the ground conditions likely to be encountered. It is vital to react quickly and stop the TBM immediately when obstructions are encountered, as chain reactions might occur and disc cutters or even parts of the cutterhead could be damaged.

ACKNOWLEDGEMENTS

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EVALUATION OF SITE-SPECIFIC PSEUDO-VELOCITY RESPONSE SPECTRA FROM TUNNEL BLASTS IN GRANITE

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Abstract

Seeking access to the residences of private citizens in Hong Kong for purposes of conducting pre-construction condition surveys often results in a low acceptance rate by the public in allowing contractors to perform this essential task. The ultimate goal must be to demonstrate to the public that it is in their best interest to allow not only pre-construction condition surveys to be conducted in their premises, but to also allow ongoing monitoring of cracks in their structures, and monitoring of environmental factors such as humidity and temperature. However until this goal is attained, non-intrusive preliminary estimates of limiting charge weights and detonator delay timing to minimise the probability of blast damage can be made by establishing pseudo-velocity response spectra (PVRS) from ground motion time-history traces of actual blasts. Current literature includes empirical methods of predicting velocity, amplitude and displacement bounds based on a series of both surface and subsurface blasts taking into account the density of rock and soil and the speed of propagation of the shock wave, in addition to the same charge weight and radial distance parameters that is used for conventional peak particle velocity (PPV) predictions of ground response. Using data collected from in-borehole seismographs monitoring tunnel blasts (rock-rock transmission) and surface instruments (rock-soil transmission), a site-specific series of indices have been determined for use in equations relating displacement, velocity and acceleration to charge weight, and the results compared against pseudo-velocity response spectra from actual blasts.

INTRODUCTION

Pre-construction condition surveys (PCCS) are generally acknowledged (Dowding, 1996; Siskind, 2000; Oriard, 1999) as of primary importance in the process of controlling, monitoring and militating against complaints from blast effects. However, gaining access to the residences of private citizens in Hong Kong for purposes of conducting PCCS often results in a low acceptance rate by the public in allowing contractors to perform this essential task. As an example, only approximately 10% of residents permitted entry into their mid-size residential complex comprising a total of approximately 128 units within range of blast activities on the Sha Tin Heights Tunnel project.

A literature review of blast case histories show there are a large number of cases resulting in site-specific attenuation formulae relating the predicted PPV at a receiver to distance from the blast and to charge weight. However, the drawback of these formulae is that they are site

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specific for either tunnels, mines or quarries, include no frequency component and are either determined based on square or cube-root scaling, scaling laws which are dependent on the geometry of the charge and on the distance (Dowding 1996). They also ignore the more rigorous parameters of wave propagation velocity, c, and density, ρ, of the transmission media.

The U.S Army Corps of Engineers (1989) observed that one of the reasons for the two main conventional scaling law variations are often due to regression analyses being performed on close-in and far-field data (>100 ft (33 m)) simultaneously, which have markedly different wave-forms. The peak particle velocity to the wave propagation velocity ratio (u/c) is proportional to a function of scaled distance (SD), ρ and c for point blasts and a function of SD, ρ, c and charge length for long charges). Thus for conventional blast attenuation formulae, ρ and c are neglected, as the variation in those parameters, within each of rock or soil media, is considered to be relatively small.

Response spectra analysis on the other hand, allows inclusion of ρ and c and more importantly, includes a frequency component, which can be used to determine susceptibility of any structure to the input ground motions produced by a particular blast. Response spectra analysis is not new. Much of the research is initially derived from and related to earthquake response spectra studies as discussed by Dowding (1996), to which it remains closely related, although the ground motion frequencies associated with earthquake motions are considerably closer to the natural frequencies of structures than typical predominant frequencies from e.g. tunnel blasts.

Frequency based Blasting Control

Wu et al (2003) observed that most of the focus of blasting regulations is on a fixed and limiting PPV, which is currently also true in Hong Kong. While this regulatory approach has the advantage that it is easy to use and monitor, little data exists of actual reported damage, whereas regulations in Europe and the USA follow PPV limits based on frequency ranges. A study conducted by Schneider (2001) summarised blasting regulations in the U. S. and worldwide which supports Wu et al. (2003) observations.

Frequency based PPV limits are detailed by Siskind et al. (1980) in the U. S. Bureau of Mines (USBM) report RI8507, which concluded that limiting PPV's *should be* frequency dependent. The current USBM guidelines are based on this frequency dependence, with an increasing range of acceptable PPV limits based on increasing frequency bands. At the time the USBM also recognized that the low frequency (4 ~ 12 Hz) part of any vibration record, even though this was not the frequency range with highest vibration amplitude, could be the area of most concern to structures, which generally have natural frequencies at the lower end of this range. The USBM also recommended that all response peaks within half an amplitude (or 6dB) of the peak frequency, be analysed. This appears as a footnote to Table 13 of the original USBM RI8507 report. Svinkin (1999) noted *"Spectra of the radial and transverse components of horizontal soil vibrations may have a few maxima and the one corresponding to the frequency of the source is not always the largest"*. As such, Dowding (1992) proposes as a limiting criteria inclusion of all frequencies within 70% of peak spectral frequency.

Clearly, the peak frequency is of importance not only by itself, but also as a measure of the relevant associated frequencies. Siskind (1996) outlines several methods of determination of the peak frequency and gives details of advantages and disadvantages of

each. These include zero-crossing (included in many software packages supporting Blasting Instruments), Spectral Analysis, Response Spectra Analysis (RSA) and conventional damage assessment. Fast Fourier Transformations are used to identify dominant frequencies. Siskind (1996) observed that the implications of exact frequency determination are more relevant to regulatory compliance than actual damage potential and concludes that RSA works well on larger (> 6 storey structures, prevalent in Hong Kong), but less well on low-rise (single storey) highly damped structures such as houses (e.g. wooden framed structures commonly found in the US). Siskind (1996) also observed that the determination of frequency levels to include in any analysis is discretionary.

The use of a more rigorous approach using Pseudo Velocity Response Spectra (PVRS) was not currently favoured by the USBM (Siskind 1996) due to the complexity and interpretation of the method and direct structure response (DSR) as discussed by Dowding (2001) is still proposed as the best method of checking of structural integrity.

Notwithstanding this, as a preliminary estimate and alternative in the absence of directly measured crack data (in difficult-to-access residences) which can be related directly to blasting, (rather than daily temperature and humidity changes and external vibration sources), PVRS continues to be used in reviews of structure response, both by researchers, Yang et al (2000) and in the industry, Aimone-Martin et al (2003). As shown below, peak frequencies are relatively well-defined for the 8ms delay blasts used in Tunnel Rounds for rock-rock wave transmission in particular. The large numbers of multi-storied buildings in Hong Kong, whose structural response can be considered to comply with a single degree of freedom (SDOF) system, also lend themselves well to this approach.

Hendron (1976) observed that knowledge of response spectrum of ground motions is valuable for making design decisions affecting specifications for those cases where precedent is lacking. With this in mind, this paper derives a set of site-specific blasting attenuation formulae and compares them to Hendron and Dowding (1974) original work.

Simplified approach to evaluation of potential for Structural Damage using PVRS

As strain (e.g. in a wall element) is simply relative displacement of the top and bottom of the wall divided by the wall height, overlaying the ground (or input motion) response over the structure response is required to determine strain levels. Ensuring the start and time scales match, this overlay will result in relative displacements, from which strain levels can be calculated.

A simplified approach to evaluation of damage potential can thus be outlined as follows :

(a) assume that the structure behaves as a single-degree-of-freedom system;
(b) calculate or estimate the natural frequency of the structure;
(c) estimate the percentage of critical damping;
(d) convert the shock pulse into a shock response spectrum, both in terms of acceleration and relative displacement responses;
(e) the peak responses of the structure are then derived from the shock response spectra, as a function of its natural frequency; and
(f) relate the peak acceleration and peak relative displacement values to some criteria limit, such as strain, ultimate stress, fatigue etc.

As no structures were directly instrumented on this Project, the results of the analyses in this paper are restricted to ground or input motions resulting from the blasting events within

the tunnel. Further research is required to instrument structures in order to determine structural response spectra from the ground input motions. Alternatively, frame models with simulations of structural response behaviour to ground-motion time histories could be used, similar to those for earthquake design. The ground input motions can thus be used to estimate structural response, including strain levels and cracking.

Pseudo Velocity Response Spectrum (PRVS) Plots

Dowding (1971) compiled radial velocity (u), acceleration ($\dot{u}$) and displacement ($\ddot{u}$) maxima from a series of blasts in quarries, mines and tunnels and plotted the results (normalised by propagation wave velocities and radial distance) vs. dimensionless ratios of charge weight, radial distance, transmission medium density and material wave propagation velocities.

Equations for each of these three maxima were then derived (Hendron and Dowding 1974) based on regression analyses of each of the 3 parameters.

$$u_{max} = 0.072\text{mm}\ (30.5/R)^{1.1}\ (3050/c)^{1.4}\ (W/4.54)^{0.7}\ (2.4/\rho)^{0.7} \qquad \text{Eq (1)}$$

$$\dot{u}_{max} = 18.3\text{mm/s}\ (30.5/R)^{1.46}\ (W/4.54)^{0.48}\ (2.4/\rho)^{0.48} \qquad \text{Eq (2)}$$

$$\ddot{u}_{max} = 0.81\text{g}\ (30.5/R)^{1.84}\ (c/3050)^{1.45}\ (W/4.54)^{0.28}\ (2.4/\rho)^{0.28} \qquad \text{Eq (3)}$$

Note that Hendron & Dowding (1974) omitted the propagation wave term from the velocity component as its resultant was very small, but it is included in the general equations derived in this paper. The 30.5 m, 3050 m/s and 4.54 kg figures are SI equivalents of 100 ft, 10000 m/s, and 10 lbs. Hendron & Dowding (1974) elected to arbitrarily select 100 ft, 10,000 ft. etc. as a reference distance, propagation wave velocity and weight, and the constant at the front of each expression, in his regression analysis, has been adjusted to take this into account.

As these equations are re-generated using site-specific data, a general form is written below allow identification of the coefficients and indices :

$$u_{max} = C_u\ (30.5/R)^{A1}\ (3050/c)^{B1}\ (W/4.54)^{C1}\ (2.4/\rho)^{D1}$$

$$\dot{u}_{max} = C_v\ (30.5/R)^{A2}\ (3050/c)^{B2}\ (W/4.54)^{C2}\ (2.4/\rho)^{D2}$$

$$\ddot{u}_{max} = C_a\ g(30.5/R)^{A}\ (c/3050)^{B3}\ (W/4.54)^{C3}\ (2.4/\rho)^{D3}$$

If blast vibration is approximated as sinusoidal motion then the 3 equations are related by

$\dot{u} = u\ (2\ \pi\ f)$ and $\ddot{u} = \dot{u}\ (2\ \pi\ f)$ (or $u\ (2\ \pi\ f)^2$)

Velocity, acceleration and displacement bounds can then be generated, which when viewed on a tripartite plot, from any combination of the input parameters, allows estimates of peak predicted ground input motions to be made. Dowding (1996) also tabulated amplification factors to apply to each of the 3 bounds, related to whether blasts were generated by single-delay timing or multiple-delay blasts, and whether they were from tunnel rounds or surface blasts. Both researchers recognised that the response spectrum based on the actual ground motion time-history trace, assuming a SDOF system was dependent on blast geometry, confinement and delay timing, with an amplified response extending beyond the 3 ground motion bounds.

The amplified bounds should thus encompass a SDOF response spectrum calculated from an actual ground motion-time history.

Details of the PVRS approach are discussed by Hendron and Dowding (1974). The 3 equations have been used in the explosives industry as predictor equations for estimation of PPV taking into account both the density of the transmission medium (ρ) and wave propagation velocity (c) and can also be used for two-layer media. Further discussion on generation of the PVRS from the actual ground motion time-history can be found in Dowding (1996).

In this Paper, the following tasks were conducted :

(a) performed the regression analysis on the radial component of the monitored peak velocity, displacement and acceleration of the ground motions to derive new expressions for u_{max}, $\dot{u}_{max}$ and $\ddot{u}_{max}$ which are for tunnel rounds only;
(b) compared the new expressions for u_{max}, $\dot{u}_{max}$ and $\ddot{u}_{max}$ to Hendron and Dowding (1974) published figures;
(c) estimated amplification factors for each of the three bounds by dividing the ground motion peak values from the average response spectra maxima (from actual blasts) on each of the three axes. The SDOF PVRS plots were calculated from actual ground-motion time histories recorded by seismographs on the Sha Tin Heights (SHT) Project;
(d) compared the amplification factors to Dowding (1971) figures; and
(e) plotted the predicted ground motion and predicted response spectrum using the SHT site specific and Dowding's (1996) ground motion attenuation formulas. The response spectrum calculated from the corresponded actual ground motion is also plotted for comparison.

This exercise has been performed twice, once for a series of 76 blasts monitored by in-borehole seismographs (rock-rock single media transmission) and again for surface monitoring instruments which recorded the ground surface response from blast waves that propagate first through rock then soil. The second analysis was conducted in order to gauge the magnitude of the difference in amplification factors between the two blast types, frequency attenuation characteristics and the propagation wave reflections at the free (ground) surface.

It should be noted, that these are site-specific results, even for rock-rock transmission, as differences not only in geology, but also the relatively shallow depth below Rockhead will cause wave propagation reflection and refraction. Surface, body and Raleigh wave overlap and interference may also occur, complicating the waveform. However, the PSVR spectrum predicted using the distance between the blast and the sensitive receiver, charge weight per delay, density and propagation velocity of the medium in which vibration is transmitting can be used to make initial predictions of structures most likely to be damaged by blasting.

In summary, PVRS analysis involves two steps :

(a) characterize the input motion; and
(b) characterize the structural response (the latter which is not part of this paper).

THE SHA TIN HEIGHTS TUNNEL DATA SET

The Sha Tin Heights Tunnel is a 1 km long twin-bored 19 m span road tunnel excavated by conventional drill and blast methods. Delay timing was typically 8 ms per delay using NONEL delays and charge weights ranging from 0.07 kg to approximately 4 kg/delay was used. Charge weights were gauged by a calibrated pump for weights above 2.5 kg/delay.

Blasts typically ranged from approximately 230 to 286 shot holes per blast, using both trunk line delays and short and long period (LP) down-the-hole NONEL delays.

Seismographs at receiver locations were placed at ground soil surface approx 10~30 m above rockhead. Rockhead is in turn typically 10 ~ 30 m above the blast locations. Transducers were placed in 5 boreholes located at the approximate level of the Sha Tin Heights tunnel crown, to monitor vibration levels adjacent existing water tunnels.

Distances between blasts and receivers were accurately gauged by surveying, using northing, easting and metres above principal datum (mPD) with coordinates of the receiver referenced to the Hong Kong Survey grid system. The closest of four surveyed points on the crown or invert was used as the tunnel blast location.

Assumptions for construction of Site Specific PRVS plots

The following assumptions were made in the construction of the Sha Tin Heights Tunnels PVRS plots:

(a) a seismic velocity of 4000m/s has been assumed based on a database of cross-hole tests in shallow HK Granites. This parameter is not as sensitive to velocity as much as absolute distance, in any case;

(b) a seismic velocity of 880m/s has been assumed for insitu soil;

(c) densities ρ of 2.65 t/m^3 and 1.95 t/m^3 have been assumed for rock and soil respectively;

(d) Dowding (1996) observed that (close to the blast) the peak frequency falls at the blast delay interval and the amplified PVRS velocity bound should be extended to include this frequency. 8ms delay interval is assumed as this was used in site blasting. This results in a frequency of 125Hz. The extension of the velocity bound has been performed for all blasts where the calculated extent of the amplified velocity bound does not reach 125Hz. The calculated extent of the amplified velocity bound spans between 0.5x and 1.5x the centre frequency. The centre frequency falls at the middle of the predicted peak ground motion bounds;

(e) only results from the radial direction (body waves) have been used, as in rock these produce the largest peak velocities. In soil (at ground surface), the ground response in the radial direction is not as dominant, however remains overall the largest response group; and

(f) damping of 3% of critical has been assumed. This compares with damping of 3~5% by Aimone-Martin et al. (2003) for low rise structures and was adopted for direct comparison with the equations for displacement velocity and acceleration in Dowding (1996), where 3% was also used.

The predicted PVRS based on the estimated ground motion attenuation formulas and structural response amplification factors are then compared with the PVRS plots generated using NUVIB, software developed by the Northwestern University, under Professor Charles Dowding. NUVIB processes the ground motion time-history traces recorded by the seismographs to solve for relative displacement, and hence the pseudovelocity of structures with different natural frequencies.

Dowding's original (1971) best fit lines to the original data set, extracted from Hendron (1976) and converted to SI units, is shown on the plots below for comparison. In the displacement and velocity domains there is reasonable agreement, however the acceleration

plot (Figure 1C) prompted derivation of a site-specific and blast type-specific (tunnelling) set of indices.

The plots in Figure 1 were used in regression analyses to derive equations (4a) to (6a) below, assuming c_{rock} = 4000 m/s and ρ_{rock} = 2.65 t/m^3. A similar set of graphs were plotted for data recorded by the surface seismographs to derive equations (4b) to (6b) assuming c_{rock} and ρ_{rock} as previous, and c_{soil} = 880 m/s and ρ_{soil} = 1.95 t/m^3. For the 2 layer rock-soil model the following relationships were used, after Dowding (1996).

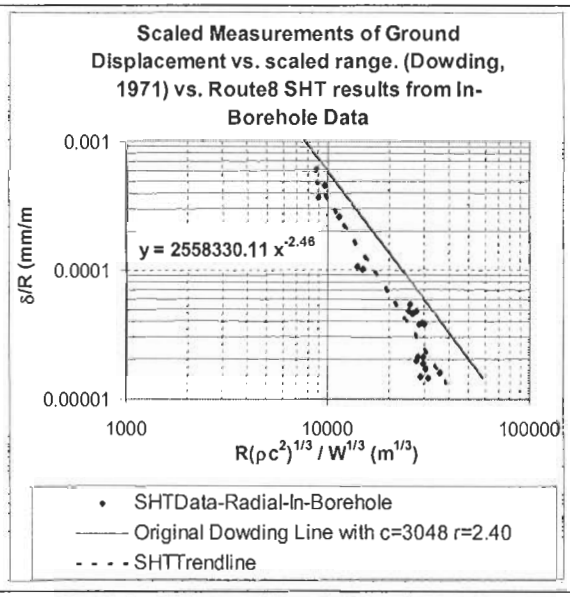


Figure 1A. Normalised Displacement vs. Scaled Distance

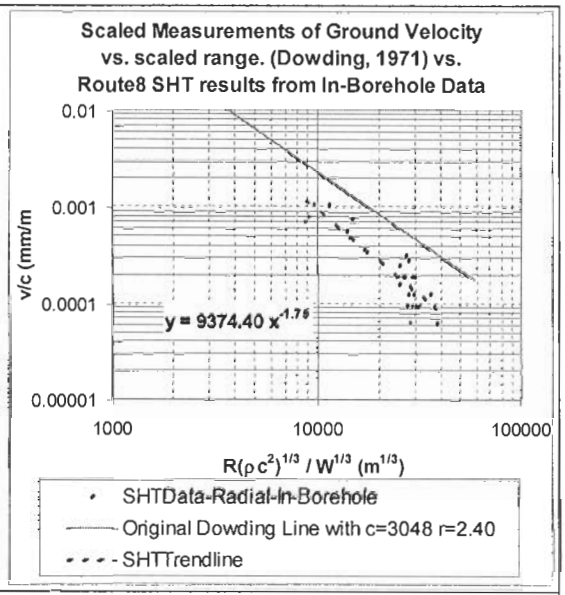


Figure 1B. Normalised Velocity vs. Scaled Distance

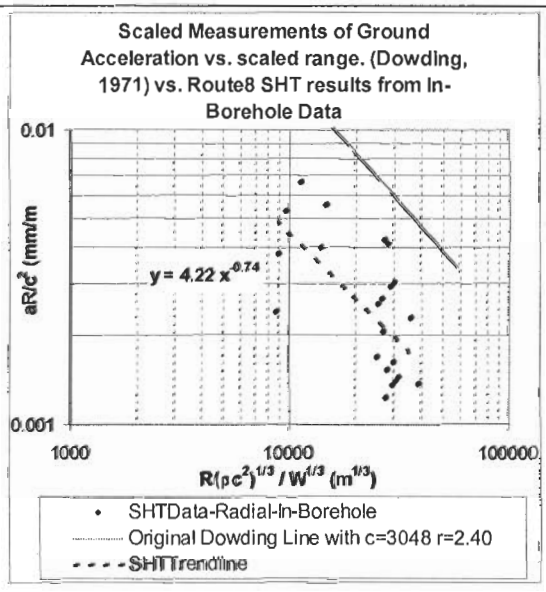


Figure 1C. Normalized Acceleration vs. Scaled Distance.

$$c_{avge} = R_1c_1 + R_2c_2 / (R_1 + R_2)$$

$$R(\rho c^2)^{1/3}/W^{1/3} = R(\rho_1 c_1^2)^{1/3}/W^{1/3} + R(\rho_1 c_1^2)^{1/3}/W^{1/3}$$

where

- c_{avge} = average weighted wave propagation velocity from Soil/Rock
- R_1 is transmission distance in Rock
- R_2 is transmission distance in Soil.

Transmission distances R_1 and R_2, through rock and soil respectively, were determined from co-ordinates of each blast (transmitter (blast) and receiver points), and interpolation of borehole geological data, to derive an intersection point at Rockhead.

Rock-Rock Transmission

$$u_{max} = 0.055\text{mm}\ (30.5/R)^{1.46}\ (3050/c)^{1.64}\ (W/4.54)^{0.82}\ (2.4/\rho)^{0.82} \quad \text{Eq (4a)}$$

$$\dot{u}_{max} = 8.95\text{mm/s}\ (30.5/R)^{1.75}\ (3050/c)^{0.17}\ (W/4.54)^{0.58}\ (2.4/\rho)^{0.58} \quad \text{Eq (5a)}$$

$$\ddot{u}_{max} = 0.23\text{g}\ (30.5/R)^{1.74}\ (c/3050)^{1.50}\ (W/4.54)^{0.25}\ (2.4/\rho)^{0.25} \quad \text{Eq (6a)}$$

Rock-Soil Transmission (2 layer model)

$$u_{max} = 0.017\text{mm}\ (30.5/R)^{0.40}\ (3050/c)^{0.93}\ (W/4.54)^{0.47}\ (2.4/\rho)^{0.47} \quad \text{Eq (4b)}$$

$$\dot{u}_{max} = 3.71\text{mm/s}\ (30.5/R)^{1.04}\ (c/3050)^{0.31}\ (W/4.54)^{0.35}\ (2.4/\rho)^{0.35} \quad \text{Eq (5b)}$$

$$\ddot{u}_{max} = 0.19\text{g}\ (30.5/R)^{1.54}\ (c/3050)^{1.64}\ (W/4.54)^{0.18}\ (2.4/\rho)^{0.18} \quad \text{Eq (6b)}$$

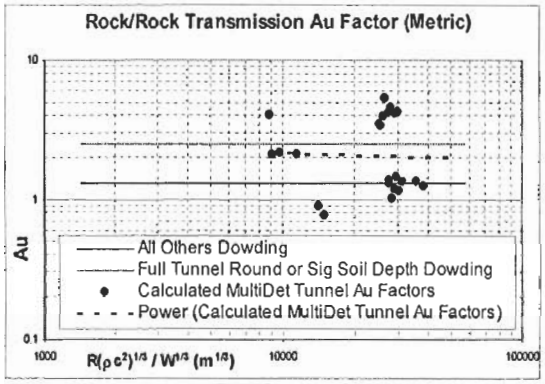
Equations (4 to 6) are the relationships for peak ground displacements, velocities and accelerations as in Figure 1. These equations enable an estimation of the peak ground motions to be conducted in a similar fashion to the widely used square root scaled distance formula.

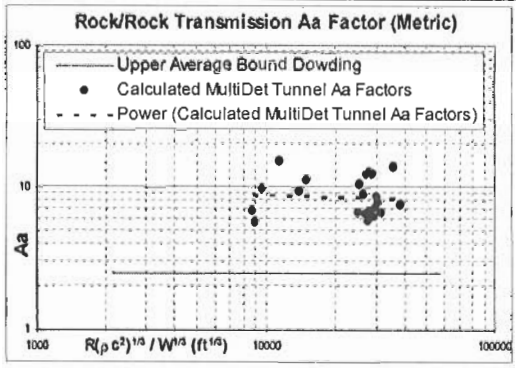
Estimation of the peak ground motions is performed knowing the distance between the blast and the sensitive receiver, charge weight per delay, material density (densities in the case of a 2 layer model) and propagation velocities of the transmitting media. The estimated peak ground motions are plotted on tripartite paper and the principal spectrum frequency, ω, is defined as the centre frequency of this ground motion spectrum (Dowding, 1996). The amplified PVRS can then be predicted by multiplying the peak motions with the appropriate amplification factors.

AMPLIFICATION FACTORS FOR APPLICATION TO PVRS SPECTRUM

Rock-Rock Transmission

Amplification factors which predict amplified response over input ground motions for each bound are assumed as Au = 2.5 and Aa = 9 based on the above plots. The SHT Av plot, as a function of $R(\rho c^2)^{1/3}/W^{1/3}$, agrees quite well with within Dowding (1971) upper average bound range. Dowding (1996) published Av value is thus used for rock-rock transmission.





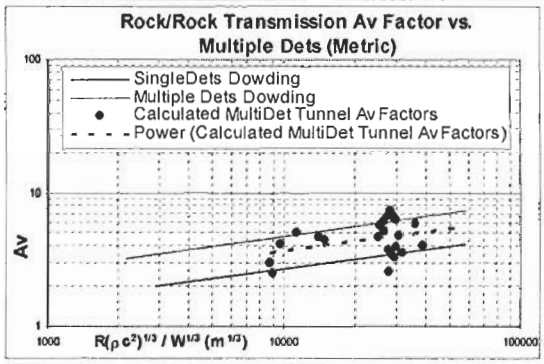
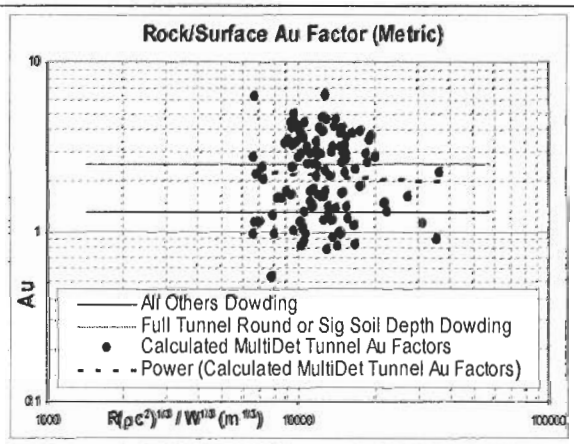
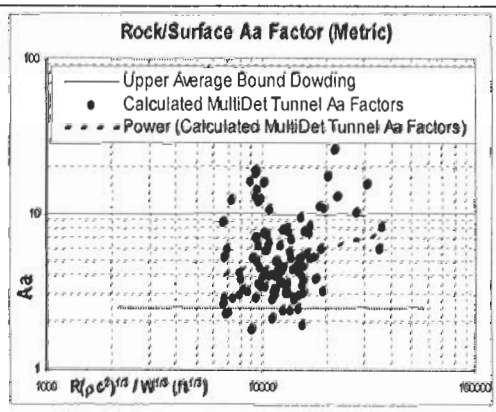


Figure 2. Amplification Factors Au, Av and Aa calculated from Site Specific Response Spectra within the rock from each blast. Dowding (1971) data (converted to SI Units) is shown for comparison. 3% damping is assumed.

Rock-Soil Transmission





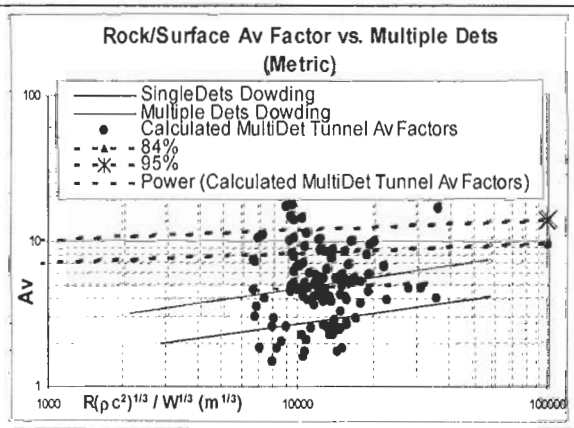


Figure 3. Amplification Factors Au, Av and Aa calculated from Site Specific Response Spectra at the ground surface, from each blast. Dowding (1971) data (converted to SI Units) is shown for comparison. 3% damping is assumed.

Amplification Factors Au = 2.5, Aa = 6 and Av as a function of $R(\rho c^2)^{1/3}/W^{1/3}$ were used for the Sha Tin Heights rock-soil 2-layers model, based on Figure 3. Note that an 84% bound is used to relate Av to $R(\rho c^2)^{1/3}/W^{1/3}$ to simulate the upper average of the relationship.

Figures 2 and 3 are generated based on the peak response of the PVRS processed by NUVIB with 3% critical damping. It is possible to carry out the same operation as Hendron and Dowding (1974) in order to estimate multipliers of the amplification factors for response spectra with various levels of damping, although this has not been performed in this study.

Dowding (1971) calculated multipliers of Au, Av and Aa for

- <3% damping (increases Au, Av and Aa) and
- >3% damping (decreases Au, Av and Aa)

However, as noted, these have not been included in these analyses. If required Dowding's fractions may be used. The construction of the PVRS plot outlined by Dowding (1996) is described briefly below :

(a) For a selected set of W, R, ρ and c, peak ground motions from equations (4-6) are plotted on tripartite paper. This forms the lower inverted V-Shaped Plot shown in Figures 4 and 5 below.

(b) Amplification factors Au, Av and Aa are applied to the peak ground motions and plotted on the same tripartite paper. Note that the applicable extent of each amplified bound depends on the centre frequency ω, as follows :

  (i) frequencies less than 0.25ω are governed by the amplified displacement bound;

  (ii) frequencies between 0.5ω and 1.5ω are governed by the amplified velocity bound; and

  (iii) frequencies exceeding 2ω will be governed by the amplified acceleration bound.

The 4 amplified velocities from 0.25ω to 2ω are then connected together to form the predicted PVRS. The extent of the velocity bound should be corrected if the frequency corresponding to the delay interval does not fall within the velocity bound. In this study, the delay interval of blasting is 8ms, which equates to a frequency of 125Hz, hence the upper limit of the velocity bound of the most of the predicted PVR spectra have been extended up to 125Hz.

A small departure from Hendron and Dowding (1974) procedure is also implemented at the limit of the acceleration bound, which is modified to the sum of the lower and the upper limits of the velocity bound. This results in a better fit to the actual data, for the entire data set.

Example PVRS plots for actual blasts are plotted in Figures 4 and 5 below for rock-rock and rock-soil transmission respectively using actual parameters W, R used in the blasts and the assumed ρ and c rock and soil properties.

Comparison of the response spectrum predicted from site specific ground motion attenuation formulas, spectral amplification factors and frequency ranges of the displacement, velocity and acceleration bound and response spectrum vs. the spectrum predicted with Hendron and Dowding (1974) ground motion attenuation formulas, spectral amplification factors and frequency ranges. The response spectrum calculated from the actual ground

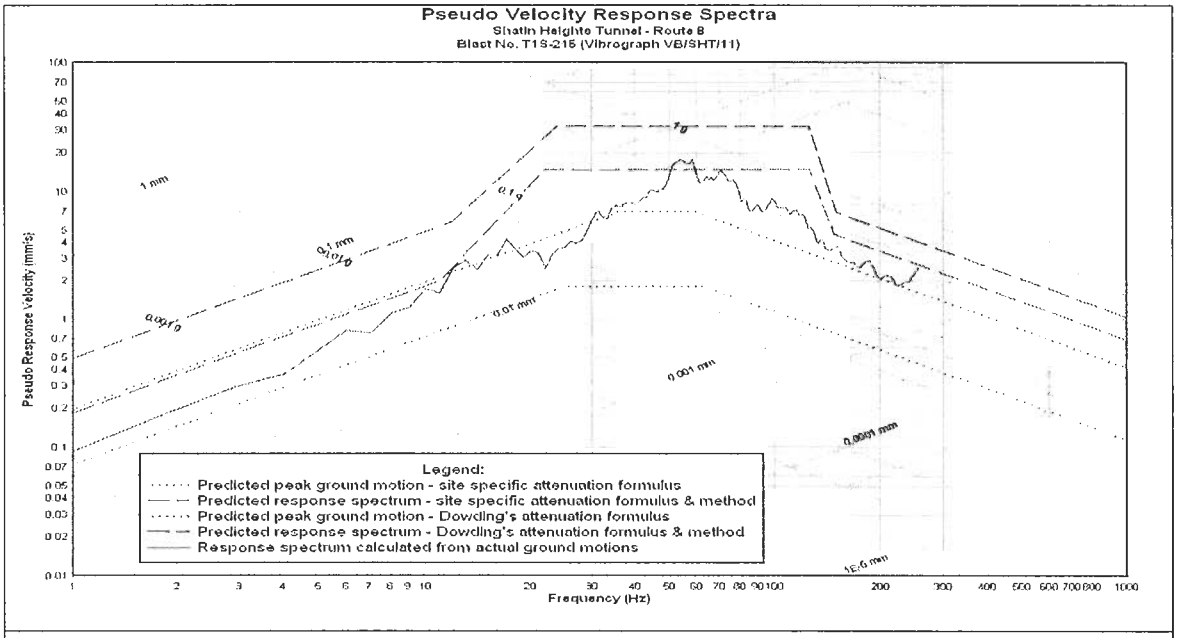


Figure 4. Rock-Rock Transmission

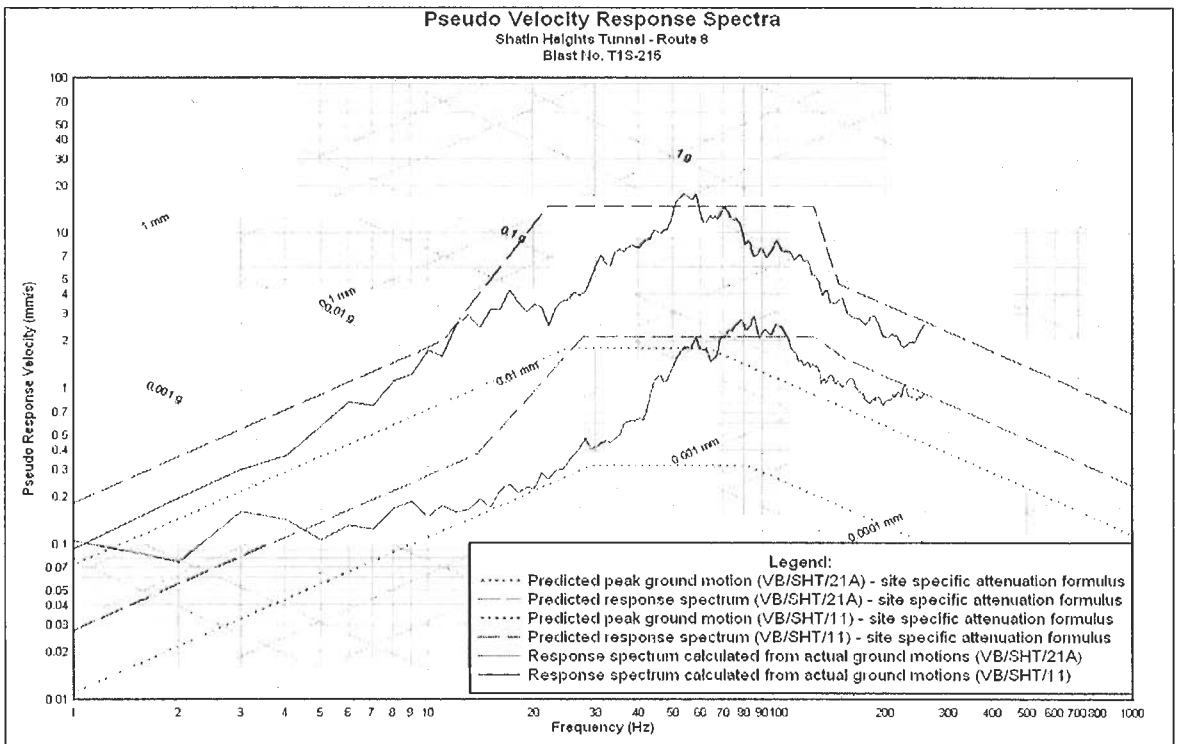


Figure 5. Comparison of the predicted peak ground motions and response spectrum for instruments at the ground surface (rock-soil transmission, upper curve) and for within the rock (rock-rock transmission, shown by the lower curve) from the same blast. The PVRS response spectrums calculated from the actual ground motions are also shown (uneven lines)

motions is also shown. The uneven line is the overlaid PVRS from the actual blast. Several observations can be made for both plots in Figures 4 and 5 as follows :

(a) the actual PVRS from the ground-motion time history fits quite well within the predicted amplified bounds;
(b) the actual PVRS from the ground-motion time history fits slightly better than the amplified bounds predicted by Hendron and Dowding (1974) formulae, which are based on several different blast types, including surface, quarry and underground tunnel blasts;
(c) the centre frequency is reasonably high, which implies that structures with a very high natural frequency are most at risk for blasting using this delay period. As discussed below this does not imply that lower frequency structures in the range 6~12 Hz will not be affected, but in general they will be governed by the displacement bounds, with lower predicted PPV's. Allowable charge weights can be calculated from the displacement ground motion equation;
(d) in Figure 5, which shows PVRS curves for the same blast recorded from ground-motion histories by in-borehole and ground surface instruments simultaneously, the peak velocity, amplitude and displacement from rock-rock transmission is lower than that for the surface instrument indicating amplification that may be attributed to ground (soil response) and surface reflection; and
(e) in Figure 5, the peak frequency corresponding to the rock-rock transmission is higher than for the soil, indicating more rapid frequency attenuation (decay) within the soil mass than through the rock.

Accurate Sampling Rates

To ensure high frequency components were captured, instrument sampling rates of 512 Hz were used so that FFT spectra up to 512/2 = 256 Hz could be derived, encompassing the 125kHz range associated with a 8ms delay interval. High sampling rates also ensured the Dowding (1996) recommendations for sampling were satisfied. Conditions for accurate sampling are as follows :

(a) With digitized traces the points making up the trace or graph are plotted only at the sample rate. The higher the frequency that is being captured the higher the sampling rate needs to be to ensure the peaks are not missed between two subsequent samples. Dowding (1996) recommends 7 to 10 samples per period (which results in 5~10% error, as opposed to the old-style analog pen graphs which shows all the peaks).
(b) To keep error sampling rate within sampling rates for the 125Hz peak frequencies the sample rates should be 7*125 ~ 10*125 or 875~1250 samples per second. At 512 samples/second, the maximum error is 512*(1/125) or 4 which corresponds to 27%.

Principal Frequencies

As principal frequencies are considered to be those which have the highest likelihood of occurrence (but noted these are not the only frequencies involved), then identification of these frequencies must first be made so that a comparison to the natural frequencies of structures in which they might cause amplification can be made.

Dowding (1992) describes several methods to determine the dominant frequency, which can be summarized as follows :

(a) visual interpretation of the ground-motion time history;
(b) plot the PVRS Spectrum and identify the centre (dominant) frequency from the plot;
(c) perform a Fast Fourier Transform (FFT) on the ground-motion time history and create a power spectrum; and

(d) use the Zero Point Crossing Method (used in much of the software supplied with Blast Measuring Instruments). This approach employs a count of twice time for two successive crossings of the zero line on a ground-motion time history plot (Dowding 1996). Some drawbacks of the zero-point crossing are discussed by Wheeler (2005).

The last three approaches are all methods used in current practice.

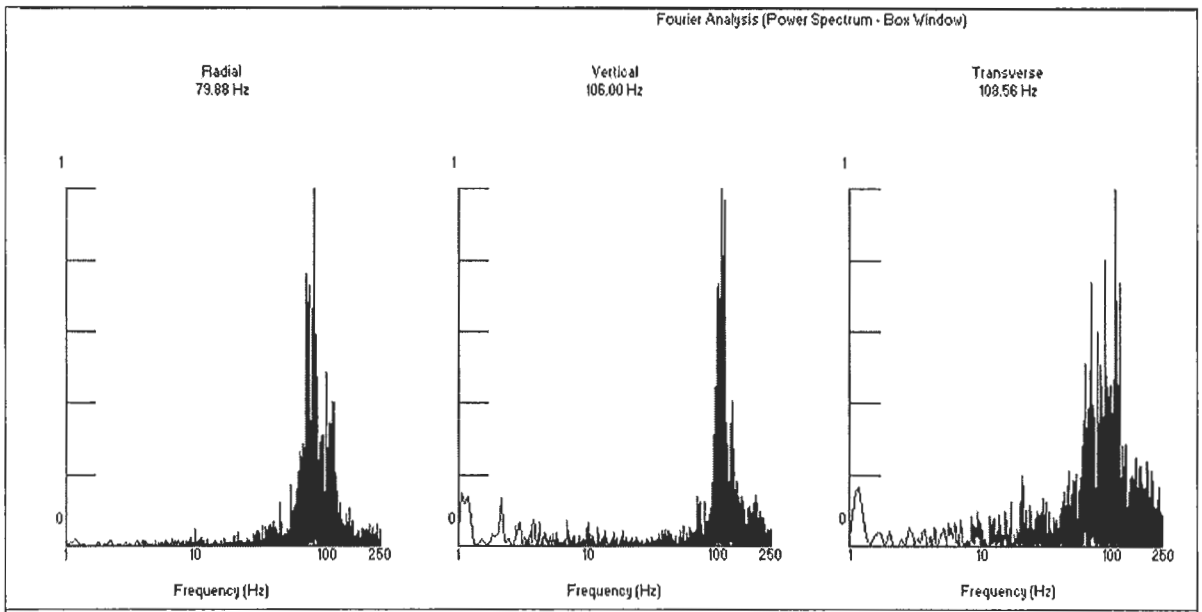


Figure 6. Power Spectrum Box Window for Blast T2S_146-20 showing clearly defined peaks at 90~125Hz

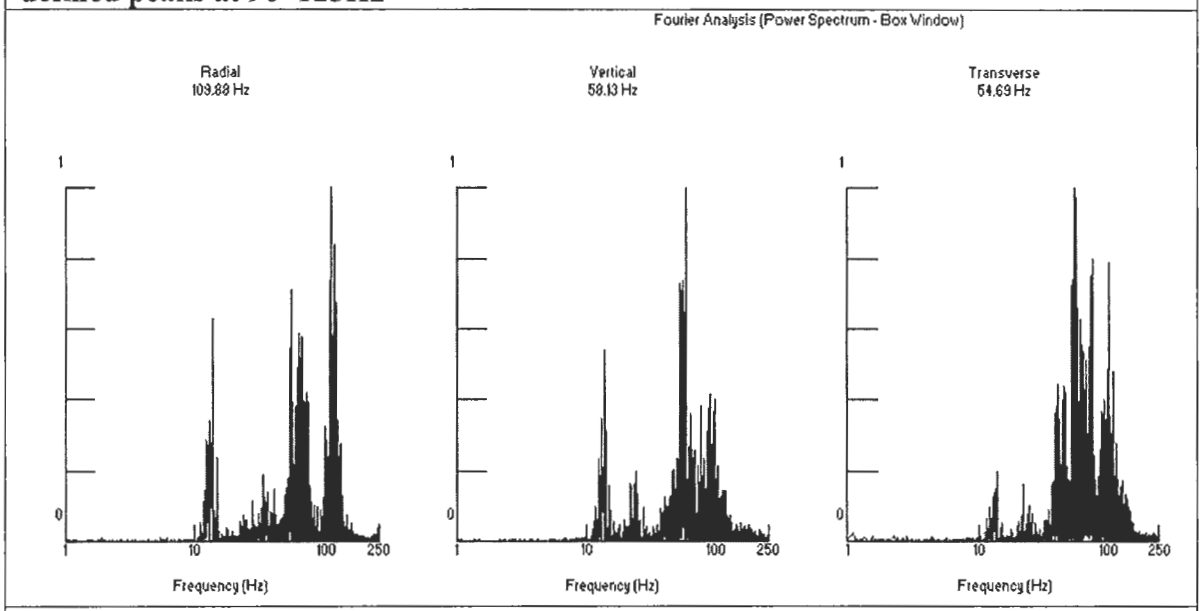


Figure 7. Power Spectrum Box Window for Blast T2S_150-22 showing clearly defined peaks at 90~125Hz, but also lower frequency components at around 20Hz.

Dowding (1992) also noted that it is not necessary to calculate Fourier or SDOF response spectra for all blasts when the PPV for most cases is < 20 mm/s and noted that

5~10% of monitored blasts is considered sufficient. Example Power Spectra Peak frequency distributions for two In-borehole receivers are graphed below.

A 8 ms delay interval results in a theoretical frequency of 1/0.008 Hz = 125 Hz. This is the frequency of the detonations, and higher frequencies are clearly evident in both the Fourier spectra (Figures 6 and 7) and the PSVR spectra (Figures 4 and 5).

However, as observed by Wheeler (2005) and evidenced by both plots above, the lower frequencies do not disappear entirely, although they are considerably less prevalent. Wheeler describes the reason for this can be traced to the signature waveform (where the signature waveform is the FFT plot from a single blast). If the FFT from the signature waveform does not contain much (or any) frequency energy within the range of the delay timing, then it is not possible to attempt to force a resultant dominant frequency equal to the delay timing.

Figure 8 below plots frequency histograms of principal frequencies as identified by FFT from the radial ground motion time-history traces of all 53 in-borehole and 115 surface results. Note that the frequencies are dominant frequencies generated by the FFT analysis and are not the frequencies commonly reported by instruments recorded at Peak particle velocity. The latter frequency is the frequency at which the PPV was recorded only, and does not represent the dominant frequency in the waveform, which is found by the techniques described above.

The plots in Figure 8 indicate that high frequencies in the 90~100Hz range dominate in rock (transmission) to rock (receiver) blasts, whereas much lower high frequencies in the 10~60Hz range dominate rock (transmission) to soil (receiver) blasts. For the same set of blasts, the PPV's (normalised by scaled distance (x100) so that comparisons can be made) indicate that PPV's are also generally higher for rock (transmission) to soil (receiver) blasts.

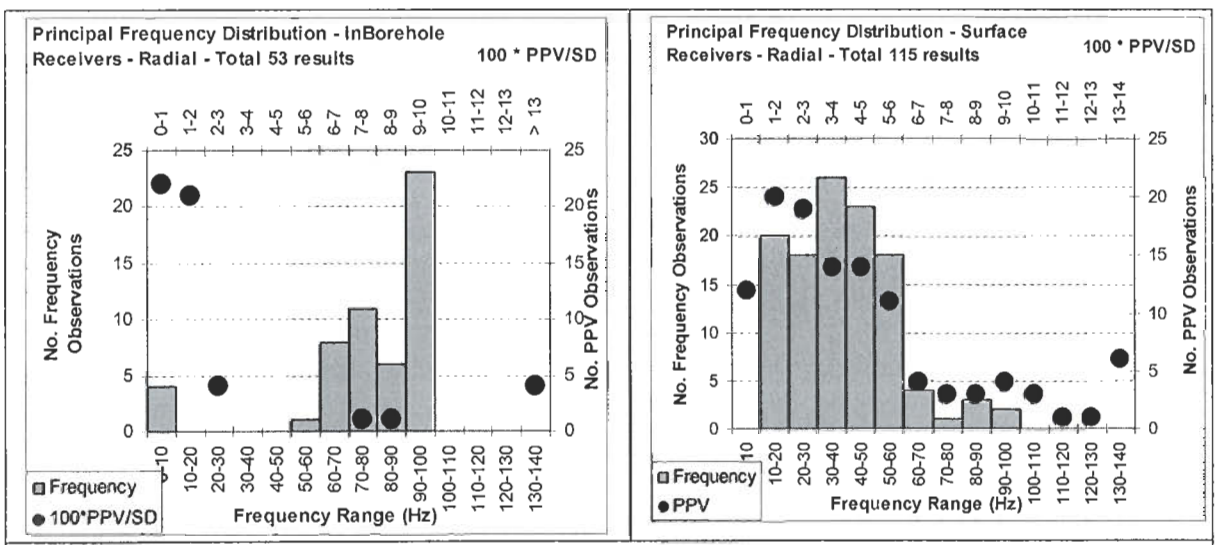


Figure 8. Frequency Histograms of Principal Frequency identified by FFT on Radial Ground-Motion Time-History (53 In-Borehole and 115 Surface Results).

Estimation of Natural Frequencies

A limited literature search for example magnitudes of natural frequencies of various structures are given in the following references. The references also included observations on the importance of frequency in blast vibration monitoring.

Author(s)	Frequency Observations
Aimone-Martin et al (2003)	Discusses whole-structure and mid-wall structure response of common single houses to surface coal mining blasts throughout the US and concludes whole structure frequencies average 6Hz, mid wall slightly higher at 8 to 14 Hz. Uses concept of strain developed in walls due to relative displacement divided by wall length (height). Notes that damage has never been observed for PPV < 25mm/s. Allowable limits are 50mm/s at 301 to 500ft. Makes sensible observation that as sensitivity of humans to blast vibrations is high, feeling and noting shaking or rattling as low as 1mm/s, oftentimes people start noticing cracking from normal changes, Temperature Humidity and attribute these to vibrations they feel. Report noted that the effect of confinement from pre-splitting resulted in far higher ground motion amplitudes than open cast mining.
Siskind et al (1980) USBM Report RI8507	Presents a well-defined database of frequencies for typical structures (1 or two story residences), built up over many years of vibration monitoring.
Aimone-Martin & Eltschalger (2003)	Observe that when the ground vibration frequency is considerably higher than that of the structure the motion is equal to that of the ground. Start taking free response of structures by overlying Ground Motion time history response on the structure motion-time-history, for various structures. This will give us some idea of amplification factors, but in absence of these, those from RI8507 at an average of 1.5Hz, ranging as high as 4Hz, can be used as an initial approximation.
Yang et al (2000)	Gives natural frequencies of walls in the range of 11~24 Hz, of roofs from 20~28Hz with damping ratios in the range 2.7~5.8%. The authors calculated amplification factors between 1~2.6, with greater for roof than walls. Amplification factor Av is the (Peak) PVRS/PPV.
Snider (2003)	Estimated natural frequency of superstructure for a single storey wooden house as 10~12hz, with walls around 17~18Hz.
Wheeler (2005)	Observes that a delay of 20ms doesn't necessarily equate to a 50Hz dominant frequency, that it depends on whether these frequencies have a significant presence on the waveform. Observations of a typical waveform from 8ms delay in a full face SHT Blast does show that 125Hz (= $1/2\pi f$) is indeed the dominant frequency, as shown by Figure 6 however, some blasts (e.g. Figure 7) also show other lower frequencies are present, with a reduced power spectrum value (thus a reduced amplitude), and should also be taken into account. Using the PVRS approach this can be performed by evaluating the design charge weight based

Author(s)	Frequency Observations
	on the natural frequency of structures which may respond at that frequency. From the PVRS spectrum produced, the displacements bound governs at these lower frequencies and PPV will be much lower (Hendron 1976). Allowable charge weights can be calculated from the equation for the displacement bound.
Crum (1997)	Estimated natural frequency of superstructure for a single storey wooden house as 5 ~ 7 Hz, and for single storey concrete block houses at around 11 Hz. Notes that the worst-case amplification was in the range of 0.9 to 2 times the structural natural frequency of the structure.

Fundamental periods for buildings are related to one or more of building height, length in direction of shaking, material composition and width. Numerous formulae have been proposed relating these parameters to the first natural frequency. Estimates of fundamental periods commonly used for earthquake design for both short squat (stiffer) structures and taller (more flexible) buildings can be found in national and international building codes or textbooks.

Leaving transportation, utilities and specialist structures such as water treatment plants and historical buildings aside, Hong Kong has three major types of residential structures namely 3-storey village houses, mid-size tower blocks (say 4 to 6 storeys) and multi-storey (>4 storey) tower blocks. Initial estimates can thus be made on potential damage i.e. to characterise the response assuming a known input ground-motion time history and to design blasts to avoid, as much as possible, frequencies close to the natural frequencies of the structures that may be affected.

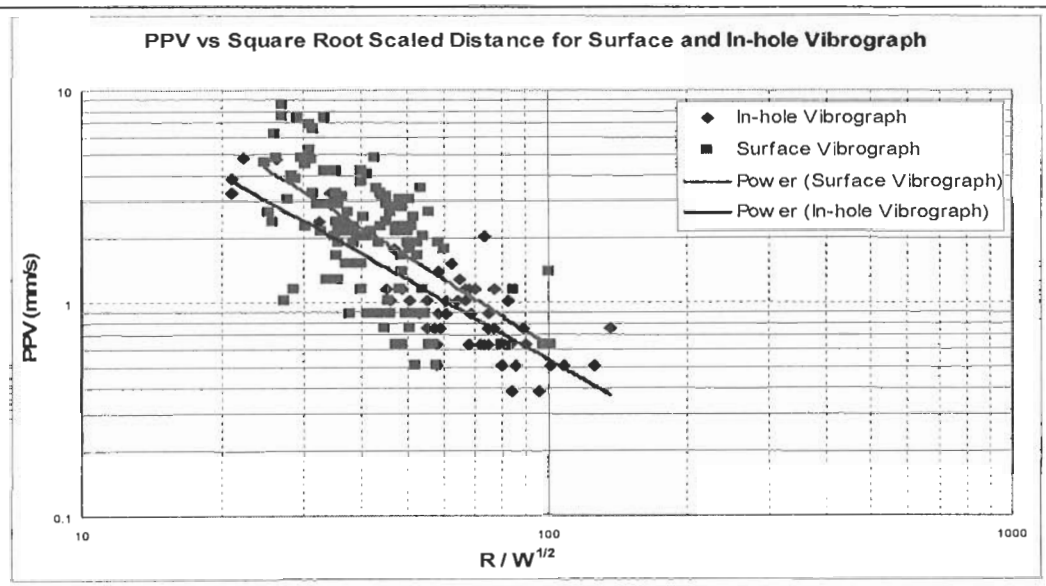


Figure 9. PPV vs. Square root scaled distance plots of vibrations measured by in-borehole instruments placed within the rock mass and surface instruments.

Observations of Blast Amplification between Rock Mass and Surface

The graph in Figure 9 shows the amplification at any fixed scaled distance, between vibrations measured by in-borehole instruments placed within the rock mass and surface instruments.

Using conventional square root scaling without any terms for propagation wave velocity or material density, at any fixed scaled distance, amplified PPV at ground surface compared to those within rock values for the same series of blasts are evident, with amplification factors in the order of approximately 2.

CONCLUSIONS

A site specific set of velocity, displacement and acceleration predictor equations for 8ms delay tunnel blasts in granite has been derived. These equations include terms for wave propagation velocities and material densities. The results agree reasonably well with published figures based on a number of different blasts types.

The PVRS approach can be used to estimate peak displacements, velocities and accelerations. Predicted relative displacements in particular, can be used, in conjunction with estimates of wall height, to calculate shear strains, from which initial crack estimates can be made.

Ground response at ground surface in a 2-layers media due to underground tunnel blasts in rock is amplified by a ratio of as much as twice that from rock-rock transmission, at similar scaled distances.

The PVRS approach itself does not quantify whether frequency of a particular blast will or will not occur; however as a tool for evaluation it does give some guidance on the likelihood of occurrence, and whether further detailed investigation, or specific monitoring, may be required.

For particularly large blasts, we should not be complacent in the belief that structures which are located beyond an arbitrary 60 m or 150 m limit are not affected by seismic vibrations. Low frequency vibrations can attenuate considerable distances and could affects structures with low natural frequencies.

While this paper focuses on the blast vibrations, it is clear that a single attenuation equation, while a good first indicator, is site-specific and effects of blast vibrations needs specific and ongoing monitoring during the course of blasting. For structures this includes comparison with temperature effects, and response coupled to vibration frequency and amplitude.

Signature blastholes at the start of a project, to determine dominant frequencies in the waveform at a particular site, could be useful in determination of optimum delay timing.

Further research to monitor structure response of local structures from ground input motions is needed.

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STATE-OF-THE-ART GROUTING REMEDY FOR GROUND IMPROVEMENT

Don Gupta[1] and Cary Hirner[2]

Abstract

Grouting is a frequently utilized remedy in tunnel construction and rehabilitation for ground improvement prior to mining and following construction to improve and decrease the permeability of rock. The current state-of-the-art grouting application used for Chicago's Tunnel and Reservoir Plan (TARP) McCook Reservoir offers a unique alternative to the tunneling industry for pre-excavation grouting as well as tunnel rehabilitation, rescue and post-excavation ground improvements. Upon completion of the McCook Reservoir, the grouting will be a significant component of a groundwater control system responsible for retaining over 26 billion liters of combined sewerage overflow (CSO) and preventing contamination of adjacent aquifers by CSO exfiltration.

The McCook Reservoir grouting project included implementing a computerized grouting control and data collection system for precision grout placement to construct a highly effective groundwater cutoff in fractured dolomites. By using multiple additives in properly formulated cement-based mixes, balanced and stabilized grout mixes were developed that exhibited zero bleed, high resistance to pressure filtration, low cohesion and high durability. The results of the grouting indicated that the effective permeability of the rock mass was reduced over two orders of magnitude to less than 0.1 Lugeon by installing a double line grout curtain. The stable grout mix designs allowed high pressure and effective grouting at depths up to 125 meters. The U.S. Army Corps of Engineers are currently revising their decades-old grouting engineering manual and guidance to incorporate the technological and material innovations and advancements achieved as a result of this project.

INTRODUCTION

Traditional neat cement-based grouts have long been used in the tunnel industry to improve and decrease the permeability of rock. These grouts typically consist of Portland cement and water and can be effective in achieving the desired results when mixed at a low water to cement ratio (2:1 or less). However, smaller open features in the rock that are capable of producing sizable volumes of water require thinner neat cement mixes to penetrate these fractures. These

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higher water to cement ratio mixes are unstable in the subsurface. During grout placement and until hardening, the cement particles in these mixes have a tendency to settle out of suspension creating an avenue for water to bleed through and are susceptible to pressure filtration. Microfine cement grouts and chemical grouts can be suitable to penetrate and satisfactorily grout these finer features. However, these specialty grouts are expensive and chemical grouting can be environmentally unsuitable for many applications.

Grouting finer open discontinuities in addition to larger features can be of particular importance to control the groundwater inflow during mining and infiltration and exfiltration of water during tunnel operation. Cement-based balanced and stabilized grouts are an effective alternative to grouting these features. These grouts are low water to cement ratio mixes that are formulated with multiple additives to produce a near zero bleed grout that has low cohesion and is resistant to pressure filtration. These grouts are capable of penetrating fine fractures while remaining stable and durable even when placed under high pressures. When used in conjunction with a computerized grouting control and data collection system, these grouts can be placed at precise locations with optimal control. The large scale grouting test conducted for the McCook Reservoir demonstrated the applicability of these grouts to effectively seal open features as well as finer discontinuities. In addition, data collected allowed for a comparison to be completed of the effectiveness of microfine cement to the lowest viscosity cement-based stable grout in penetrating finer features.

BACKGROUND

The Chicago Underflow Plan McCook Reservoir is one of three planned reservoirs in the Tunnel and Reservoir Plan (TARP), which incorporates approximately 175 kilometers of tunnels in the Chicago area. Upon completion the McCook Reservoir will store combined sewer overflow (CSO) conveyed from the TARP Mainstream and Des Plaines Deep Tunnel Systems until floodwaters recede and the CSO can be pumped to a nearby wastewater treatment plant. The reservoir is currently planned to be approximately 90 meters in depth with a perimeter of 4 km.

The McCook Reservoir will be located at the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) Lawndale Avenue Solids Management Area (LASMA) facility in Willow Springs, Illinois. The project location is shown in Figure 1.

The McCook Reservoir design includes tunnels, shafts, chambers, buildings, gates and valves. It also features an overburden cut-off wall and a grout curtain around the reservoir perimeter to control groundwater infiltration and CSO exfiltration. The purpose of the overburden cut-off wall is to reduce the seepage of groundwater through the overburden to minimize groundwater infiltration and to prevent overburden slope failure in the reservoir walls due to excessive flow. The purpose of the grout curtain is to minimize the infiltration and exfiltration from the reservoir. Excessive infiltration of groundwater would result in increased pumping and water treatment costs. Exfiltration of CSOs is an environmental concern that may result in groundwater contamination in the bedrock aquifer.

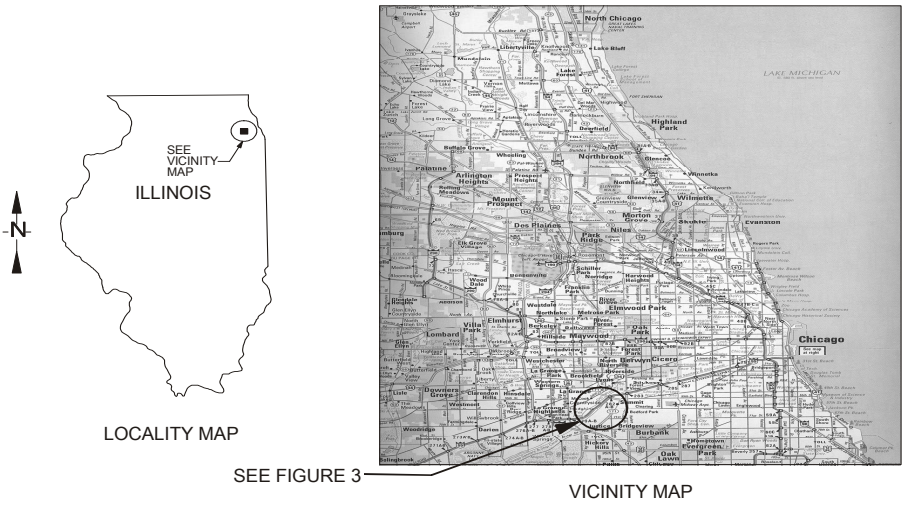

Figure 1. Project Location

Approximately 9 m to 21 m of glacial drift overlies Silurian, Ordovician, Cambrian, and Pre-Cambrian bedrock at the McCook Reservoir site. The reservoir is currently planned to be excavated into primarily the dolomitic rocks of the Silurian System including the Racine, Sugar Run, Joliet, Kankakee, Elwood, and Wilhelmi Formations (Figure 2). The Racine Formation directly underlies the Pleistocene overburden deposits at the site. Shale and dolomitic shale of the underlying Ordovician System Maquoketa Group including the Brainard, Fort Atkinson, and Scales Formations will underlie the floor of the reservoir.

Two near vertical, generally widely spaced joint sets with northeast and northwest strikes are present in the McCook Reservoir site bedrock. The joints were subject to solution from groundwater sometime after consolidation of the Silurian dolomite formations locally forming open fissures. Most of these fissures were later filled with mud that was the remnants of Devonian and Mississippian formations deposited and then later eroded from the area. Some are, however, open or partly open and can be conduits for significant groundwater infiltration into excavations. Near horizontal bedding planes in the rock can also account for large amounts of groundwater inflow, particularly in the Elwood Formation. In addition, fracture and solution zones are present locally in the rock.

The primary objective of the McCook Reservoir grouting test was to evaluate the effectiveness of grouting in this geologic environment while determining the most appropriate drilling method to complete the perimeter grouting of the proposed reservoir. This included grouting of the upper bedrock and integration with the overburden cut-off wall that was constructed previously. The project was also conducted to evaluate drilling methods and grouting parameters for construction of the reservoir perimeter grout curtain.

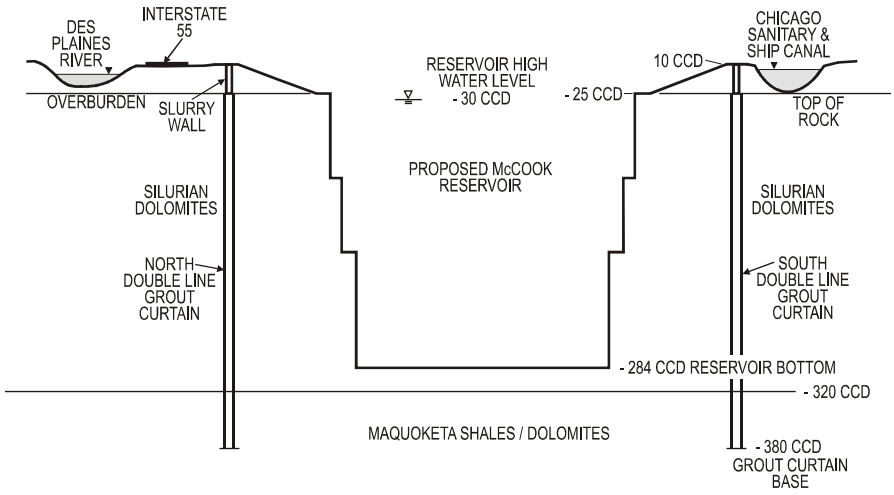

Figure 2. General profile of the planned McCook Reservoir

GROUTING PROGRAM

The McCook Reservoir Grout Test Project consisted of three grout curtain sections as illustrated in Figure 3 as follows :

(a) a 60 m double line section constructed along the north side of the planned reservoir using percussion drilling with water as the circulation fluid. This section is referred to as the North Test Section;

(b) a 60 m double line section constructed along the south side of the planned reservoir using rotary drilling. This section is referred to as the South Test Section;

(c) a 25 m section constructed adjacent to an overburden cut-off wall demonstration section using rotary drilling. This section is referred to as the Demonstration Section.

Each grout curtain section consisted of two lines of holes with each line drilled approximately 2.3 m from the centerline of the existing overburden cut-off wall. The holes in each section were drilled through steel casings, which were installed through the overburden and seated into rock. The holes and casings were inclined 15 degrees from vertical and directed parallel to the test section and overburden cut-off wall alignment, with the holes in the two lines in each section oriented in opposite directions.

The grout holes in the Demonstration Section were only drilled 3 m into rock to test grouting effectiveness and pressures at the rock/overburden contact. Each grout hole sequence in the Demonstration Section was grouted with a different color grout so that the Demonstration Section can be systematically investigated during the reservoir excavation to observe the grouting effectiveness near the contact between overburden and rock.

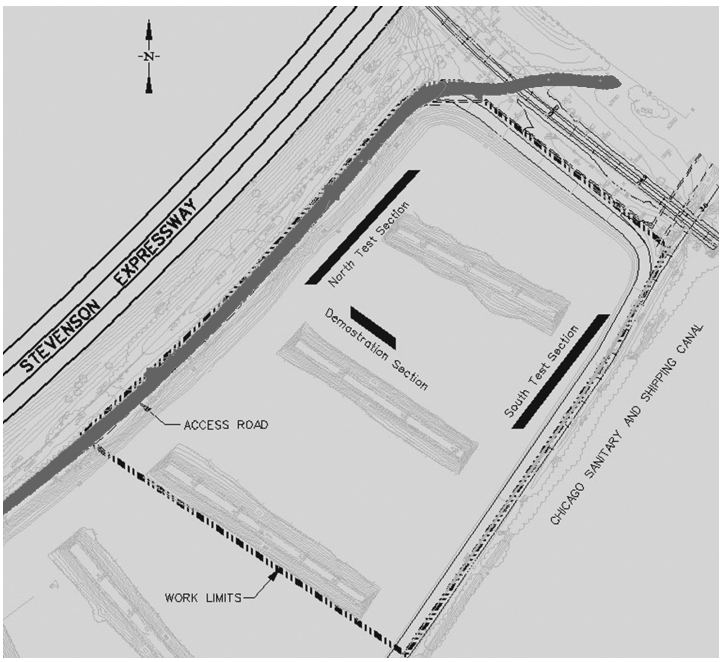

Figure 3. Plan View of Grout Test Curtains

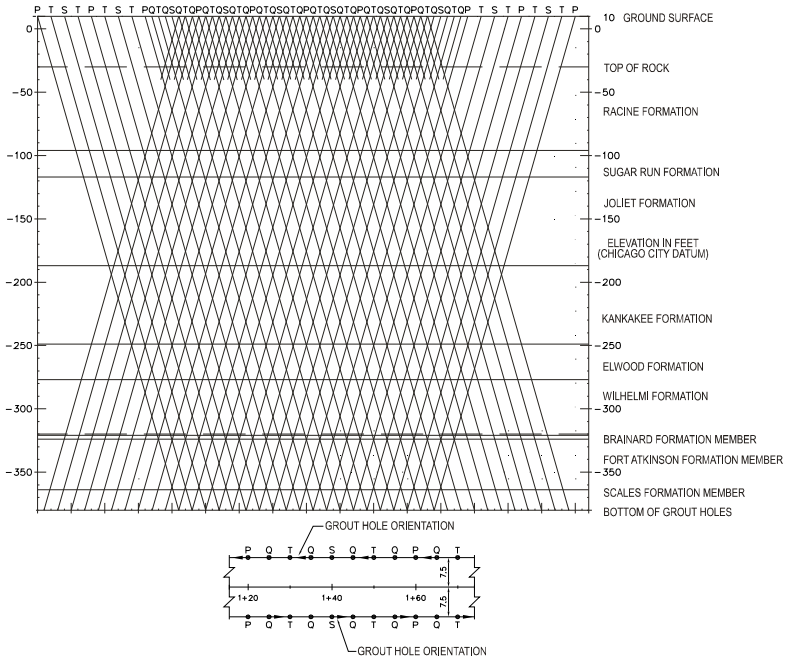

Figure 4. Grout Test Curtain Layout (English units)

The primary holes in the North and South Test Section curtains were 125 m deep to extend through the Silurian dolomitic formations into the Scales Formation (shale) of the underlying

Maquoketa Group. The North Test Section curtain consists of two grout hole lines designated as A line and B line, which were drilled using water percussion. In the South Test Section curtain the two grout hole lines are called C line and D line, which were rotary drilled. The inner grout hole lines in the North and South Test Section curtains are referred to as the A and C lines, respectively, and the outer lines are the B and D lines, respectively. The test section curtains were designed to be incorporated into the final perimeter grout curtain around the planned reservoir. Figure 4 is a sectional and plan view showing the grout hole orientation.

The drilling, washing, pressure testing, and grouting operations for all but the quaternary holes in the test sections were performed in two separate zones. The first zone extended from the top of rock to 3m into rock. In general, the second zone extended from the bottom of the first zone to the final depth of the hole. Downstage grouting of the top stage of rock was conducted to minimize the potential impacts to the overburden cutoff wall while drilling to final depth through the remaining bedrock. Successful grouting of upper rock zone was important to connect the overburden cutoff wall and grout curtain, and consequently provide a continuous barrier to minimize seepage.

After drilling and grouting of the first zone, the grout curtain was constructed using the upstage grouting method whereby each hole was drilled to a final depth and grouted through a packer that was set at successively shallower depths, except when unstable conditions were encountered or circulating water loss occurred while drilling. When these conditions were encountered, the zone of instability or water loss was grouted as a descending stage before completion of drilling to full depth. Occasionally, when the water loss zone extended over the majority of the drilled length, the entire portion of the hole completed was grouted in ascending stages before drilling was completed.

Each grout hole stage was pressure washed and water pressure tested before grouting. Pressure washing was conducted at each grout hole until clean water was circulated. In general, water pressure testing was performed in stages of either 3 m or 12.2 m lengths with a double (straddle) packer assembly. However, water pressure testing was also conducted using a single packer in some of the deeper stages. The permeability of the rock formations was calculated in Lugeons for each interval tested (1 Lugeon being roughly equal to 1.3×10^{-5} cm/sec).

GROUT MIXES

Seven cement/bentonite grout mixes and an ultrafine cement mix were designed and developed specifically for the McCook Reservoir test grouting project. Mixes A through G were formulated with 8% silica fume Type II cement, 8% (by weight of water) bentonite slurry, superplasticizer, and a viscosity modifier. Each additive served a purpose in improving the rheological properties of the grout mix (Table 1). With the exception of an increased amount of superplasticizer in the A mix, the A through G mixes varied only by the amount of water used. The water to cement ratio increased from 0.69 for mix G to 1.76 for mix A. The ultrafine mix (M mix) contained ultrafine cement, superplasticizer, viscosity modifier, and water.

Table 1. Purpose of Grout Mix Additive

Grout Mix Additive	Purpose of Additive
Silica fume	Reduce bleed, increase resistance to pressure filtration, enhance penetrability
Bentonite	Reduce bleed, increase resistance to pressure filtration, enhance penetrability
Superplasticizer	Reduce viscosity and cohesion
Viscosity modifier (Welan Gum)	Reduce bleed, increase resistance to pressure filtration

Bench and full scale mix testing programs were conducted to determine the mix design constituents and proportions (Table 2). Each grout mix was initially tested for Marsh funnel viscosity and mud balance specific gravity (API Method RP 13B-1); bleed (ASTM C 940); and pressure filtration coefficient (API Method RP 13B-1). In addition, the initial and final set time and cube strength at 3, 7, 14, and 28 days were tested for the five lowest viscosity grout mixes. During grouting operations each mix used was tested daily for Marsh funnel viscosity and mud balance specific gravity and weekly for bleed and pressure filtration coefficient. Grout temperature was monitored each time a different mix was batched, several times a day during grouting operations in hot weather, and at least once a day during cold weather.

Grout was batched from an entirely automated plant using bulk cement stored in an adjacent hopper. The plant contained two 300-liter (79.3 gal), high-shear, colloidal mixers one for mixing bentonite slurry and one for mixing grout; two 700-liter (184.9 gal) bentonite hydration and holding tanks; two 700-liter (189.4 gal) grout agitator and holding tanks; and three 3-stage, progressive cavity pumps for grout distribution through coiled lines to two header carts for injection.

Grouting on each stage generally started with three batches of mix A before thickening the grout, except where the ultrafine mix was used. A minimum of two batches of each subsequent mix was injected before thickening. The general criteria followed for thickening of the grout was no observed increase in pressure or no reduction in the apparent Lugeon value after achieving the maximum injection pressure during an injection period of 15 minutes. Grout was placed through single, inflatable packers lowered into the grout holes on the whips of the self-propelled header station carts. Injection pressure was supplied by the pumps in the grout plant and controlled with injection and return valves in the header carts.

The maximum grouting pressure used for each stage was equal to 6.9 kPa (1 psi) per foot of vertical cover over the center of the stage, except for the first stages that generally were grouted at 3.45 kPa (0.5 psi) per foot of cover. Grout placement was continued until the refusal criterion was reached at the maximum grouting pressure. The refusal criterion was adjusted during the program to balance the need for a tight curtain with the expense of injecting grout at very low flow rates for extended periods of time. In general, the refusal criteria when grouting the first stage near the overburden and bedrock interface was a take of 0.76 L/min (0.2 gpm) for 15 minutes, and below the first stage was 0.38 L/min (0.1 gpm) for 15 minutes.

Table 2. McCook Reservoir Perimeter Curtain Grout Mixes

Mix Type	A	B	C	D	E	F	G	Microfine
Mix Amounts								
Water (litres)	168.8	124.9	103.0	82.9	73.0	50.0	40.1	90.1
Bentonite Slurry Water (litres)	42.6	42.6	42.6	42.6	42.6	42.6	42.6	--
Dry Bentonite Weight (kg)	3.41	3.41	3.41	3.41	3.41	3.41	3.41	--
Silica Fume Cement (kg)	120.0	120.0	120.0	120.0	120.0	120.0	120.0	--
Welan Gum (kg)	0.09	0.1	0.1	0.1	0.1	0.1	0.1	0.08
Super Plasticizer (litres)	2.38	2.23	2.23	2.23	2.23	2.23	2.23	1.51
Ultrafine Cement (kg)	--	--	--	--	--	--	--	75.0
Calculated Ratios								
Water/Cement Ratio (weight)	1.76	1.4	1.21	1.05	0.96	0.77	0.69	1.2
Water/Cement Ratio (volume)	5.64	4.47	3.38	3.35	3.08	2.47	2.2	3.6
Grout Properties								
Batch Yield Volume (litres)	252.8	208.6	186.6	166.5	156.7	133.6	123.8	116.6
Specific Gravity	1.36	1.38	1.45	1.48	1.54	1.6	1.7	1.41
Marsh Viscosity (seconds)	33	38	43	49	55	73	90+	33
Bleed at 3 hours (%)	1.0	0.5	0.0	0.0	0.0	0.0	0.0	1.0
Pressure Filtration Coefficient ($min^{-1/2}$)	0.032	0.024	0.025	0.021	0.021	0.02	0.016	0.011
Initial Set Time (hours)	12	10	8	7	6	--	--	--
Final Set Time (hours)	15	13	10	9	8	--	--	--
3-Day Compressive Strength (kPa)	972	1441	3110	4013	6357	--	--	--
7-Day Compressive Strength (kPa)	1034	1572	3413	4337	6584	--	--	--
14-Day Compressive Strength (kPa)	2220	2730	6136	8466	12569	--	--	--
28-Day Compressive Strength (kPa)	2282	3857	7598	9515	16954	--	--	--

Drilling Results

There was no consistent pattern of deviation in the drilling of the grout holes, though more holes tended to veer right rather than left. Drilling deviation from the intended alignment was greater for the percussion drilled holes than for the rotary drilled holes even though the percussion rig was only operated at about 40 percent of its capacity in order to limit deviation. No stabilizers were used on the drill strings, but the percussion down-hole hammer was configured with stabilizing vanes along its length. The total deviation at the bottom of the primary holes averaged 3.7 m for percussion drilled holes and 3 m for rotary drilled holes (Figure 5). Five surveyed holes exceeded the deviation tolerance of 4 cm per meter drilled by minor amounts ranging from approximately 0.3 to 40 cm at the bottom of the hole.

During the course of the test grout project an increase in drilling efficiency with both the rotary and percussion rigs was achieved. Drilling rates for both types of rig increased in each successive sequence of grout holes. However, percussion drilling was consistently more than twice as fast as rotary drilling. As an example, in the tertiary holes the rotary drills averaged 5.5 m of hole per work hour while the percussion drill averaged 12 m of hole per work hour. These rates are inclusive of the time required to move between holes, setup, add rods and retrieve core from the rotary drills.

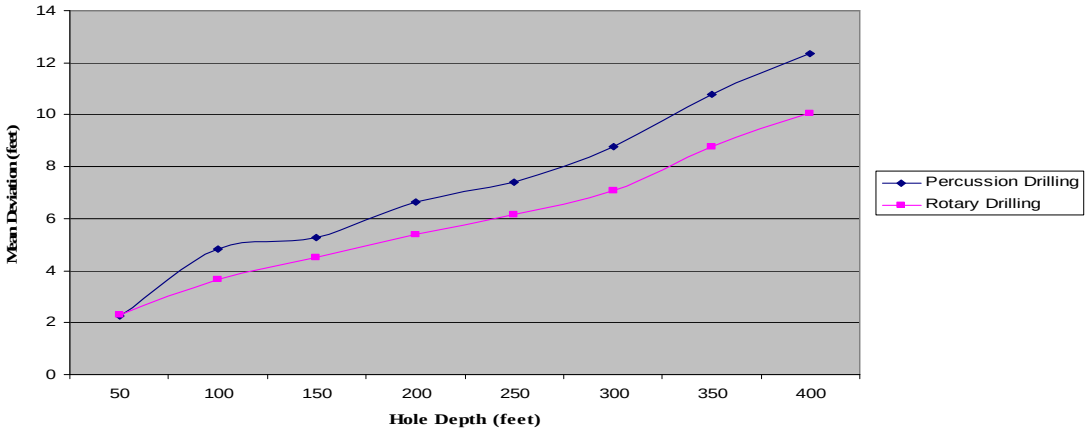

Figure 5. Borehole Deviation Results

The higher efficiency of the percussion drill was due both to higher penetration rates and reduced time in adding rods. Percussion drilling times for a 3 m rod averaged approximately 9 minutes in the Silurian dolomite formations while drilling the secondary holes. The drilling pause needed to add rods was approximately 1.5 minutes. Every third drill rod the operator ran the drill head slowly to the top of the mast and then to the bottom with the wash water running to flush the hole. This process took about 1.5 minutes. The time for a 3 m rotary run The time for a 3 m rotary run without complications averaged approximately 15 minutes. Between runs the

average time to retrieve, empty, reset the core barrel, and add a drill rod was 11 minutes.

The rotary and percussion rock drills produce cuttings with decidedly different grain size distributions as demonstrated by visual inspection as well as by sampling and testing (Figure 6). The rotary drill cuttings were almost entirely silt size with only a very small, fine sand sized fraction. The percussion drill cuttings were mostly well graded fine to coarse sand sized with about 35 percent silt sized particles. However, similar water pressure testing and grouting results in both test sections demonstrated that the larger size drill cuttings from the percussion drill did not clog the rock discontinuities and prevent effective grouting.

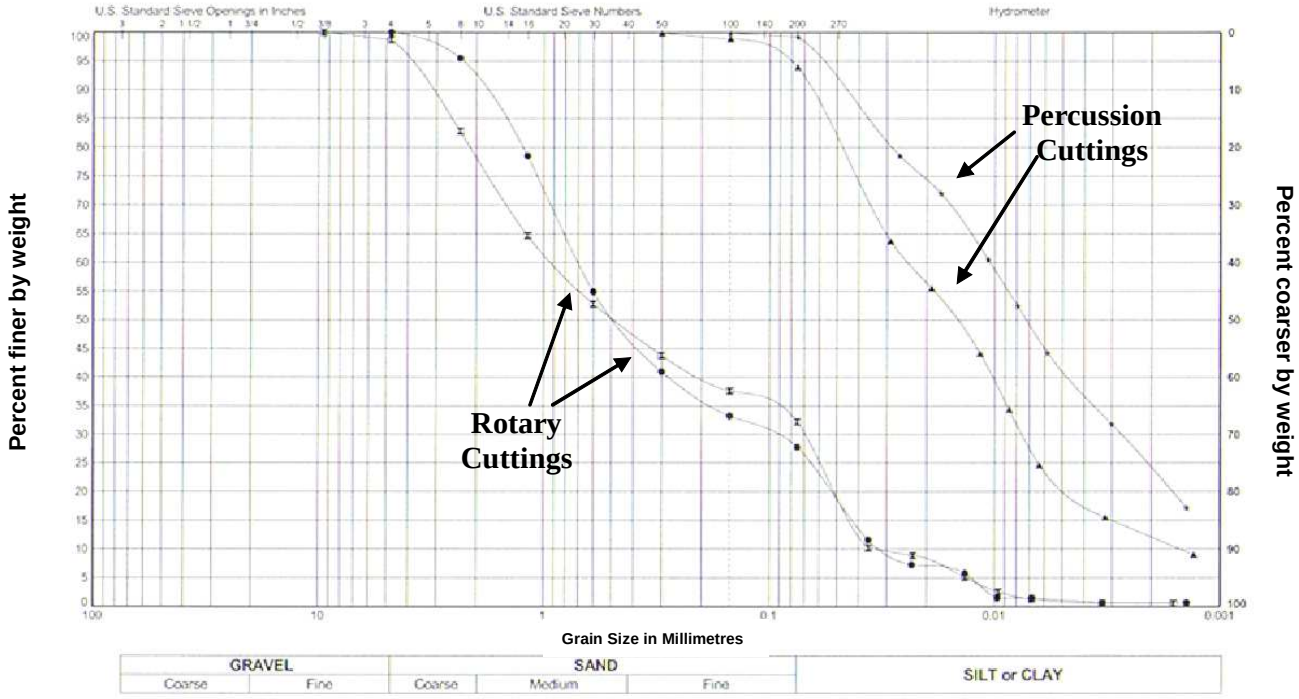

Figure 6. Drilling Cuttings Comparison

Judging from the permeability reduction achieved in both test sections, a high quality grout curtain can be achieved with either drilling method if there is appropriate quality control. The most important aspect to the drilling quality control is minimizing deviation. The amount of deviation experienced in the grouting test project with the percussion drill did not appear to adversely affect the quality of the grouting. If the drilling deviation is adequately controlled, the choice between rotary and water actuated down-hole hammer drilling is primarily dependent on cost. The water actuated down-hole drilling is clearly more cost effective due to its demonstrated higher production rates. It is expected that future programs will feature experimentation with stabilizers to further decrease deviation and allow drill rates to be increased.

Evaluation of Water Pressure Testing

European practitioners have questioned the value of water pressure testing in grouting for some time. They have noted that water pressure testing is generally not correlated to grout take,

so it is of no value in establishing starting grout mixes or other grouting parameters. They contend that the best test for the grout hole is with grout. Figure 7 is a plot of water pressure test results versus grout take for all the stages in the grouting test project below the first stage. The data for the first stage grouting is similar. The data confirmed that grout take cannot be directly related to water pressure test data in intervals that were highly permeable when water pressure tested or to grout takes.

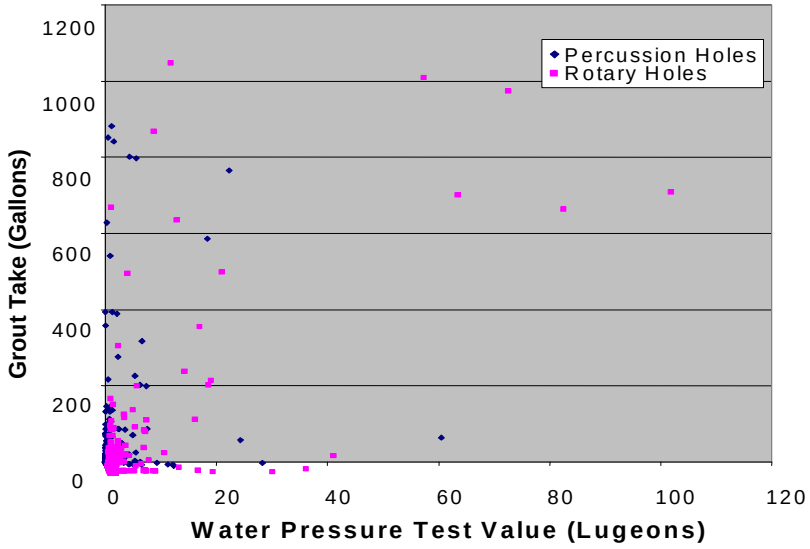

Figure 7. Water Pressure Testing and Grout Take for All Stages Below the First

During the McCook Reservoir grouting test, water pressure testing was used to identify high permeability horizons as well as to establish grouting stage lengths in the primary holes. The grouting effectiveness was measured based on the water pressure testing of successive sequences of grout holes. High permeability zones in the primary holes were separated from less permeable zones and grouted in relatively short stages. This ensured that the less permeable zones were not treated with the thicker mixes required in high permeability zones. Exposure of the less permeable zones to these thicker mixes could result in premature refusal of the grout before adequate penetration was achieved and could have resulted in a less effective curtain.

The water pressure testing in the secondary and tertiary holes allowed tracking of net reductions in overall permeability attained at each sequence. It was not used to establish stage lengths, initial mix, or other parameters used in grouting the holes. All stages were grouted without regard to the water pressure test results, even if the stage tested tight.

It was concluded that water pressure testing in the secondary and subsequent sequence grout holes could be curtailed to about 25 percent during the future construction of the perimeter grout curtain. This would lead to a more economical approach, as well as shorten the construction schedule.

AUTOMATED DATA COLLECTION AND CONTROL

A computerized automated recording, display, and control system was used for the water pressure testing and pressure grouting operations. This was an integrated system that used electronic equipment for data collection, monitoring, record-keeping and analysis (Figure 8). The availability of real-time data allowed for the accurate control of the grouting program by making the necessary adjustments to the flow rates and pressures.

Figure 8. Automated Batch Plant and Data Collection and Control

Remote sensors in the gages, flow meters, and scales allowed the automated control system to gather data and use the abilities of the computer. The system measured and displayed effective and gauge pressures, flow rate, and Lugeon and apparent Lugeon values in real-time during water pressure testing and grouting operations. The system incorporated static water level information and head loss curves developed for water and each grout mix to calculate and display effective pressure in the grout hole interval, Lugeon values for water or apparent Lugeon values for grout, and volume of grout injected. The data was stored in an SQL database. The system generated both tabular reports and plots in AutoCAD that could be displayed on monitors or printed and plotted in color. The plots include both traditional “stick” and three-dimensional plots to report and aid in the evaluation of the grouting progress.

RESULTS

The grouting program at McCook Reservoir approached the practical limits for cement-based grouting applications in reducing the ground permeability. The final permeability of the ground after grouting was substantially reduced from the initial permeability. Verification holes tested following the completion of the grouting program indicated a reduction in permeability greater than two orders of magnitude for the entire rock mass. The permeability of the rock decreased throughout the rock mass regardless if the zone’s initial permeability was high or low (Table 3).

The results also indicated the suitable applicability of these stabilized grout mixes to penetrate not only large openings but finer features as well.

- Primary holes produced a large improvement in the ground from the original conditions indicating that larger open and persistent smaller features were grouted.
- Continued grout takes in the secondary and tertiary holes and the small, inconsistent declines in permeability between the secondary and tertiary holes indicated that the tertiary holes continued to close the finer discontinuities in the rock.

Table 3. Grout Program Permeability Results

Rock Permeability Summary	North Grout Curtain (Lugeons)	South Grout Curtain (Lugeons)
Effective Initial Permeability	11.09	
Effective Final Permeability	0.06	0.3
Initial Mean Permeability First Stage – Interface	58.07	
Final Mean Permeability First Stage – Interface	0.68	1.27
Initial Mean Permeability Upper Racine	8.93	
Final Mean Permeability Upper Racine	0.14	0.19
Initial Mean Permeability Racine/Sugar Run	37.0	
Final Mean Permeability Racine/Sugar Run	0.03	0.09
Initial Mean Permeability Joliet/Kankakee	0.93	
Final Mean Permeability Joliet/Kankakee	0.04	0.10
Initial Mean Permeability Elwood	51.58	
Final Mean Permeability Elwood	0.09	0.05
Initial Mean Permeability Below Elwood	1.12	
Final Mean Permeability Below Elwood	0.02	0.02

Comparison of A Mix to Microfine Cement Mix

The automated data collection system allowed for a comparison of the performance of the microfine cement mix to the stable cement-based A mix by examining grouting results of the relatively tight stages of rock (defined as less than 3 Lugeons when water pressure tested). In particular, the amenability of the grout was evaluated and compared. Amenability is the ratio of the apparent Lugeon value during the initial application of grout to the water pressure test Lugeon value for a grout hole stage. A higher amenability indicates better penetration by the grout. The mean amenability of the microfine cement mix was 1.48 versus 2.25 for the A mix. These comparisons along with the water pressure testing and grouting trends observed during the project suggested that the microfine cement mix was not any more effective than the A mix in penetrating the finer discontinuities.

CONCLUSIONS

The grout test program for the McCook Reservoir demonstrated that properly designed and formulated cement-based balanced and stabilized grouts, coupled with fully computerized

grouting control and monitoring systems, significantly increases grouting efficiency. The U.S. Army Corps of Engineers (USACE) is currently revising their decades-old grouting engineering manual and guidance to incorporate the technological and material innovations and advancements achieved as a result of this project. It is the opinion of the authors that the tunneling industry can significantly benefit from similar precise grouting applications for tunnels, whether it is used ahead of a tunnel boring machine, from the surface, or to rescue and rehabilitate leaking tunnels as groundwater infiltration and exfiltration is increasingly becoming the most significant concern to tunnels.

A REVIEW OF THE PARAMETERS GOVERNING PREDICTED AND ACTUAL RQD EVALUATION IN SHATIN HEIGHTS TUNNELS

J K Murfitt[1], J W Tattersal[2] and Billy Siu[3]

Abstract

Hard Rock Tunnels in Hong Kong granite formations lend themselves well to temporary and permanent support application using the Norwegian Geotechnical Institute (NGI) 6-parameter Q-System. Initial prediction of one of the 6 parameters, RQD, is often performed from borehole logging and mapping of rock outcrops. Well-documented pitfalls of RQD include directional bias and lack of sensitivity. Several methods exist to improve the ease of computation, including modification of the standard 0.1m threshold level and use of Volumetric Joint Ratio (Jv) as an estimate of RQD, as proposed by Palmström (1982). The various methods are discussed in this paper and the impact of the use of Volumetric Joint Ratio (Jv) compared to pre-tender RQD estimates based on borehole results is evaluated.

INTRODUCTION

The Shatin Heights Tunnels are twin 3-lane 19m span tunnels excavated by conventional drill & blast methods. Tunnel temporary support was successfully applied following face mapping using the Q-System for tunnel support (Barton et al, 1974; Barton & Grimstad, 1994 and Barton, 2002).

One of the parameters used in the Q-System is Rock Quality Designation (RQD) first proposed by Deere (1963) and later clarified in more details by Deere (1989), a parameter which has been used in both rock mass classification system and even as single index for tunnel support design in the USA (Deere & Deere, 1988) prior to its inclusion as one of the parameters in a multi-parameter rock mass classification system. As observed by Marinos et al (2005), it has been used successfully in hundreds of kilometres of Tunnels.

In this paper, comparisons are made between the resultant percentiles (in terms of tunnel length) of Q-values estimated at tender stage from both vertical boreholes and horizontal boreholes. Support classes are not used as a reference as they both :

(a) encompass a range of Q-values; and
(b) are tunnel-span dependent, which means that any design estimates for a smaller span must be recalculated for larger spans, although Q-values remain constant.

RQD in its current form must continue to be used in the Q-System to ensure no departure from the case-histories that make up the Q-System charts, i.e. it cannot simply be replaced by another parameter which some authors argue better characterizes the overall rock mass properties, including block size and shape (Palmström, 2001; Sen & Eissa, 1991). As such

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the challenge is to determine the most consistent method of its determination in a large-span tunnel environment.

SENSITIVITY OF AND DIFFICULTIES IN RQD ESTIMATION

The sensitivity of and difficulties in RQD estimation are well established as discussed by Deere (1989), Palmström (2001; 2005) and Hack (2002). The main issues can be summarised as :

(a) directional bias i.e. RQD is a vector quantity;
(b) lack of sensitivity to large joint spacing (e.g. in large underground openings) due to abrupt threshold boundary at 0.1 m; and
(c) difficulties and influences in measurement.

Directional Bias

Palmström (2001 & 2005) highlights in detail the problems associated with directional bias and the limited range of RQD with respect to block sizes and shapes. However, while the extreme case of RQD estimates from a single directional joint (e.g. in a bedded sedimentary rock) can lead to severely skewed RQD estimates, estimates from boreholes in multi-jointed granites relate reasonably well to RQD estimates derived from tunnel face and sidewall mapping, using Volumetric Joint Ratio (J_v). It should be noted that raw RQD estimates from boreholes are tempered by a review of both the RQD profiles with depth and structural domains (faults, shear zones) and orientation of the tunnel with respect to horizontal cores along the tunnel alignment.

Directional bias can also be reasonably well taken into account by taking vertical and horizontal scanlines, such as those taken in the Yucca Mountain Tunnel Project in Nevada, USA (Sandia, 1995). In this project, fracture frequency was measured along vertical and horizontal scanlines taken on the walls, and a horizontal scanline on the face.

Peacock et al (2003) proposed a method of RQD mapping in a tunnel using single, but curved, scanlines along tunnel or borehole walls which allow RQD mapping to be performed directly, (or using Hudson & Priest (1979) fracture frequency definition) which mathematically removes directional bias. However there are practical difficulties in setting up scanlines in large openings in particular.

Sensitivity

One of the reasons for the sensitivity lies in the assumption of the threshold level of 0.1 m (pieces of intact rock > 4 inches or 100 mm) used in Deere's (1963) original formulation and presented mathematically by Hudson & Priest (1979). Harrison (1999) observed that RQD values tend to be either high (say, above 85%) or low (often 10% or below). In practice this means that certain values of RQD are less frequently encountered, for example, 40% through to 60%.

One method proposed by Hudson (1997) to increase the sensitivity, or provide a better correlation, of RQD is to revise the threshold level of 0.1 m to a larger value e.g. 1.0 m. Adu-Acheampong (2003) performed a comparison of resultant RQD values that were derived if the threshold values were revised from 0.1 m to 1.0 m and observed the resultant RQD values were reduced by half. The drawback is that any change in estimation of RQD would impact the empirical database if RQD (or its estimate J_v) is not derived in the same way as in the case-histories from the Q-System is comprised.

Sen & Eissa (1991) investigated RQD as a function of block shape and determined that :

(a) RQD was not related to J_v linearly, as per Palmström's (1982) relationships; and
(b) RQD is a function of block shape.

They noted however that for moderate J_v values, Palmström's (1982) approximation is adequate. Published charts of RQD based on J_v for various block shapes are presented in Sen and Eissa (1992).

RQD measurement on underground exposed faces

The procedure for RQD estimation (but not the definition), was reviewed by Deere and Deere (1988). This procedure improves repeatability and makes observations about mechanical fractures and other variables, such as core size.

Mitigation against difficulties in measurement on exposures (rather than from borehole core) can be achieved by a number of methods. The main methods include:

(a) recording the overall distribution of RQD values using a histogram to record a range of RQD values (Barton 1974; 1994). This is particularly useful in tunnels with large spans, or longer mapped lengths;
(b) using (multiple) scanlines measuring fracture frequency as proposed by Hudson & Priest (1979);
(c) using weighted values based on number of observations as discussed by Vardakos & Gutierrez (2005), similar to Barton's histograms. This principle can also be applied to J_v (easier to estimate); and
(d) using J_v which is generally a lower-bound fit, and thus slightly conservative.

Recent work by Palmström (2005) has resulted in a revised volumetric joint ratio relationship J_v intended to more closely fit observations of RQD in boreholes while remaining within the original overall range of common variations. It does not however address extreme limits of RQD. The old and new correlations are :

- $RQD = 115 – 3.3\ J_v$, Palmström (1982), for $J_v < 4.5$, $J_v > 35$
- $RQD = 110 – 2.5\ J_v$, Palmström (2005)

Alternatives to RQD within Rock Mass Classification Systems

Hack (2002) reviewed a number of rock mass classification systems for slope stability works, some of which have a background in tunnelling works, e.g. Bieniawski's (1989) Rock Mass Rating (RMR) and Barton's Q-System (1974). Hack (2002) discussed the most sensitive parameters in each system, of which RQD was highlighted as being one of the most subjective to evaluate. As a result, there have been a number of proposals for replacement of the parameter RQD, including within the framework of the Q-System, with alternative and more readily measurable rock mass quality parameters. These include :

(a) Palmström (1995), with recent improvements detailed in Palmström (2000), proposed the Rock Mass Index (RMI) system, in which the block volume (V_b) and joint orientations are measured and used in place of RQD; and
(b) Maerz & Germain (1996) observed that the rock mass classification process is highly subjective and requires experienced personnel. They propose a simulation technique for block size determination and noted that this could introduce the possibility of replacing the estimated parameter RQD/Jn completely, with a measurable parameter that is less subjective than RQD and with no limitation on block size.

While not an alternative to RQD, Marinos et al (2005) noted that the Geological Strength Index (GSI), which is a rock-mass characterization system and not a replacement for RQD, was proposed in part because RQD so poorly represents the degree of jointing, especially high degree of jointing in poor rock, and that it is open to different interpretations depending on the experience and level of conservatism of the observer.

COMMON PRACTICES IN RQD ESTIMATION FOR TUNNELS

The review of common practice for RQD estimation included personal communication with a number of authors of publications on RQD evaluation, the Q-System, rock-mass classification systems and tunnelling. Comments from these authors are summarised in the following table, and highlight some of the different approaches considered most appropriate.

Author	Comment
Prof. John A Hudson Imperial College and Rock Engineering Consultants, UK	"I recommend measuring the RQD directly in the tunnel along scanlines. Even the RQD is a derived property and so I prefer to start with fracture frequency of the fracture sets, then the fracture frequency in any direction can be calculated. I recommend measuring the RQD directly in the tunnel along scanlines … to imagine cores or lines in different directions through the rock mass. But even the RQD is a derived property and so I prefer to start with fracture frequency of the fracture sets, then the fracture frequency in any direction can be calculated. ("Engineering Rock Mechanics – Illustrated Worked Examples" by Harrison and Hudson, Elsevier, 2000). Because the fracture sets have specific directions, the overall fracture frequency through the rock mass will be a function of the orientation of the scanline being considered. Similarly for the RQD, which is also a directionally dependent parameter. If the RQD is explicitly required in certain directions, then that can be measured along scanlines. Alternatively, one can also estimate the RQD and hence its variation in all directions from the fracture frequency and its variation (see Harrison and Hudson). The problem with estimating RQD from Jv is that it will give the RQD as a single scalar value; whereas RQD is a type of vector (although always positive) because it is a function of direction. Think of the errors that could be made in estimating RQD from vertical boreholes when the information is required for a horizontal tunnel." Hudson (2005).
Dr. Nick Barton, Nick Barton & Associates	"I have never personally used volumetric joint count except when giving lectures. I have never set up a scanline either – in a tunnel situation. I just use my brain to do the scan – and come up with an approximate figure with never a decimal place, nor a '62', but in a range e.g. 55-65 etc – always on a histogram logging sheet for the last 17-18 years. I always give a range of opinions – the statistics takes care of the doubts – and real variations. I have never used the Hudson and Priest formula either. Your 'classic' core box picture 0 (10) or 100 dilemma sometimes nearly arises, or actually arises in mudrocks etc. But it doesn't cause me to doubt the validity of any 'data base' re. RQD, nor question the

Author	Comment
	general applicability of 'Q'. I have never experienced John Harrison's 'above 85, 10 or below' syndrome. RQD shows, invariably, the widest range of the 6 parameters. That is why it has survived so long – it is very sensitive – and could be used 'alone' for tunnel design in the USA for some years before RMR and Q arrived." Barton (2005).
Dr. Doug Milne, Assoc. Prof. of Civil Engineering, College of Engineering, University of Saskatchewan	"I would stick with Jv measurements when underground. I would not change the threshold level for RQD. It's true that large openings may be more sensitive to a larger threshold value, but you'd lose the empirical data base associated with RQD if you changed the threshold. I don't know a relationship between different RQD threshold values and block size, but I imagine it would be feasible." Milne (2005).
Dr. Arild Palmström (Norconsult)	"I am of the opinion that the block volume is theoretically the best measurement where it is possible to observe it." Palmström (2005b).

LITERATURE REVIEW

A limited literature review also shows several different approaches are used to determine RQD for tunnelling work.

(a) Barton et al (1974) gave an example of Q-estimation using RQD derived from $J_{v,}$ however as noted by Barton above, Barton has rarely used it rigorously in a tunnel. Barton (2001) in his letter to the December 2001 ISRM news journal also observed that J_v may be used in the absence of core data, and that the ratio RQD/J_n remains a sensitive indicator of block size. Note that Barton et al.'s (1974) paper quotes a private communication with Palmström in 1974 regarding Jv, however Palmström did not publish his correlation between RQD and Jv until 1982.

(b) Hudson & Priest (1979) show RQD can be derived from scanline joint frequency using :

$$RQD = 100e^{-0.1\lambda} * (0.1\lambda + L), \text{ which for } L = 6 \text{ -}16$$

can be simplified to

$$RQD = -3.68\lambda * 110.4.$$

(c) Milne et al (1998) observed that RQD in Tunnels can be estimated using the Volumetric Joint Ratio (J_v).

(d) Choi & Park (2004) conducted an analysis of the effect of RQD on scanline orientation for tunnelling and concluded it could be in error by as much as 24% where vertical drillholes were used for estimation of RQD in a sub-horizontal tunnel. Their review also determined that optimal RQD estimates for their data were achieved when the threshold level was increased from 0.1m to approx. 0.5 m for simulated RQD values based on scanline lengths ranging from 0.2 to 5.0 m.

(e) Edelbro (2004) observes that for large-sized tunnels RQD is of "questionable value" as it is unlikely that all defects found in boreholes would be significant to rock mass stability. Note however, that support of the large span 60m span Govik Cavern in Norway was actually based on RQD estimates.

(f) Loset (1999) observes that in weak competent rock, joints may be more or less missing i.e. according RQD = 10. Loset also observes that rock as weak as 1 to 3 MPa should be defined as incompetent and the RQD set to 0 (i.e. 10 in the Q-System)

whatever the jointing. This agrees with Deere and Deere's (1989) guidelines to use only pieces of "hard and sound" rock in RQD logging. However Loset also notes that if the stress level is low, weak competent rock may behave almost in the same way as strong rock (RQD=100) i.e. is stress-dependant, and related to SRF, also a Q-System parameter.

RQD AND ROCK STRENGTHS IN WEAK ZONES

Intact Rock Strength

Barton (1994) observed that the Q System in the Norwegian Method of Tunnelling (NMT) was essentially for rock with c = 3 MPa to 300 MPa, with clay bearing zones. Two of the Q-System parameters could be considered to be indirectly related to strength as follows :

(a) the parameter Jn : a high degree of jointing implies higher rates of water infiltration, leading to greater depths of weathering, leading ultimately to lower rock mass strength; and
(b) the parameter SRF : this parameter is related to observations of stress condition, and type and magnitude of deformations, which is also indirectly related to strength.

Weak Zones

Neither RQD (nor J_v) is a measurement of degree of weathering or strength, simply the degree of jointing. Loset (1999), in regard to the use of the Loset Formula, recommends that :

(a) in practice the formula should only be used for weakness zones with a width ranging from 0.5 m up to a few metres; and
(b) the maximum value of Qsb should be two times the boundary Q value for support/no support, where Qsb = the Q value of the surrounding rock.

THE SHATIN HEIGHTS RQD ESTIMATES

A comparison of RQD recorded from boreholes vs. tunnel mapping has been conducted for the Shatin Heights Tunnel. While as close a comparison as possible has been made, a direct comparison is not possible as :

(a) RQD derived from Jv was not used to estimate RQD for the entire tunnel length, however where RQD is estimated from scanlines, or visual interpretations, Jv has been back-calculated from the estimated RQD; and
(b) the original estimates for RQD were based on both vertical and horizontal boreholes. The horizontal boreholes were bored in a straight line (horizontal alignment) whereas the actual alignment is curved. However the horizontal RQD values have been correlated as closely as possible to tunnel chainage.

Pre-tender boreholes comprised two horizontally drilled boreholes, one from each portal, and a series of boreholes along the alignment, with a cluster of boreholes at each portal. The horizontally drilled cores (HDC) were straight (in plan) and while the initial part of the coring captured rock core within the tunnel domain, the ends of the HDC core runs departed from the tunnel alignment, as they were not drilled along the same (curved) alignment as the tunnel itself.

RQD PERCENTILES

The pie-charts (Figure 1) below shows the estimated expected percentiles of RQD, based on RQD values interpolated from:

(a) horizontal boreholes only, with proportions of RQD within each range weighted for length of that range cored;
(b) vertical plus horizontal boreholes, assuming discrete depth ranges of 0~6 m, 6~12 m, 12~20 m, 20~50 m and 50~100 m, for the section of borehole drilled in the depth range that the tunnel was excavated. The proportions are again weighted for the length of core, over the total length of core retrieved, within the depth ranges. This ensures any clusters of boreholes (e.g. at the tunnel portal) do not artificially skew RQD values.

From Figure 1, it is clear that in general, the results of the RQD ranges do not differ greatly from the results of the combined vertical and horizontal core.

The small graphic markers in the middle graph of Figures 2 and 3 show the "raw" logged Q values vs. tunnel chainage. These markers indicate :
(a) a large variation in RQD with tunnel chainage; and
(b) do not correlate particularly well against the mapped RQD values.

The large variations are likely to come about as a result of :
(a) possible mis-interpretation of drilling breakages during core-logging;
(b) not including only pieces of sound and hard rock;
(c) short sections of core with very low RQD; and
(d) natural pessimism in recording RQD values during site logging.

A slightly better distribution is derived on the same (middle graphs) in Figures 2 and 3 by weighting the RQD core percentages within the same tunnel lengths as was actually mapped. These are represented by the large diamond markers. Notwithstanding this manipulation of the RQD values, the distribution is still some way below actual mapped RQD values in the tunnel.

Some possible reasons for this may include :
(a) scale effects. Logging < 100mm core in 1.5m sections compared to mapping a 19m span tunnel in up to 5m sections;
(b) anisotropy of the core (essentially a vector quantity) vs. the more 3-dimensional tunnel space;
(c) natural visual weighting when mapping, resulting in an average RQD; and
(d) natural optimism in recording RQD values during tunnel mapping. Small variations in mapped parameters can maintain the Q values such that support remains within a desired support classes.

Figure 4 shows the same data as Figure 1, but includes the distribution of RQD within each depth range. In general the proportions of better rock quality improve with depth range, as expected of improved rock quality with depth below rockhead.

A significantly better correlation with mapped RQD can be derived as follows :
(a) split up all the available SI into lengths of core (incl. HDC) at various depths below rockhead. (0 – 6 m, 6 – 12 m, 12 – 20 m, 20 – 50 m, 50 – 100 m). This assumes that weathering and RQD is uniform to the same depth, provided all the SI is from the same general area (Shatin Heights Tunnel).;

(b) at a cross-section of the tunnel, measure the depth from rockhead to the tunnel crown, and assign one of those five depth ranges (or a fault, where RQD is assumed to = 10) to the tunnel chainage; and
(c) a single weighted **average** RQD is then chosen within each depth range. This value is presented on the lower graph in Figures 2 and 3 and clearly shows a better correlation with the mapped data.

The above approach should be used with caution; all structural domains identified from Geological Survey Maps, photo-lineaments and any known fault zones should first be added. The assignment of a single average RQD to a large chainage interval can miss short sections of poor ground within that interval.

A similar exercise with average Q values has been performed for comparison with the mapped Q values, which are shown in the top chart of each of Figures 2 and 3.

A comparison of mapped RQD (based on the volumetric joint ratio (J_v) or simply from scanline data) and RQD re-calculated from Palmström (2005) new J_v relationship is also shown in the middle charts of Figures 2 and 3. At the values of RQD mapped, there is little difference between the Palmström (2005) relationship and the original definition of J_v, albeit the new relationship results in slightly higher RQD values.

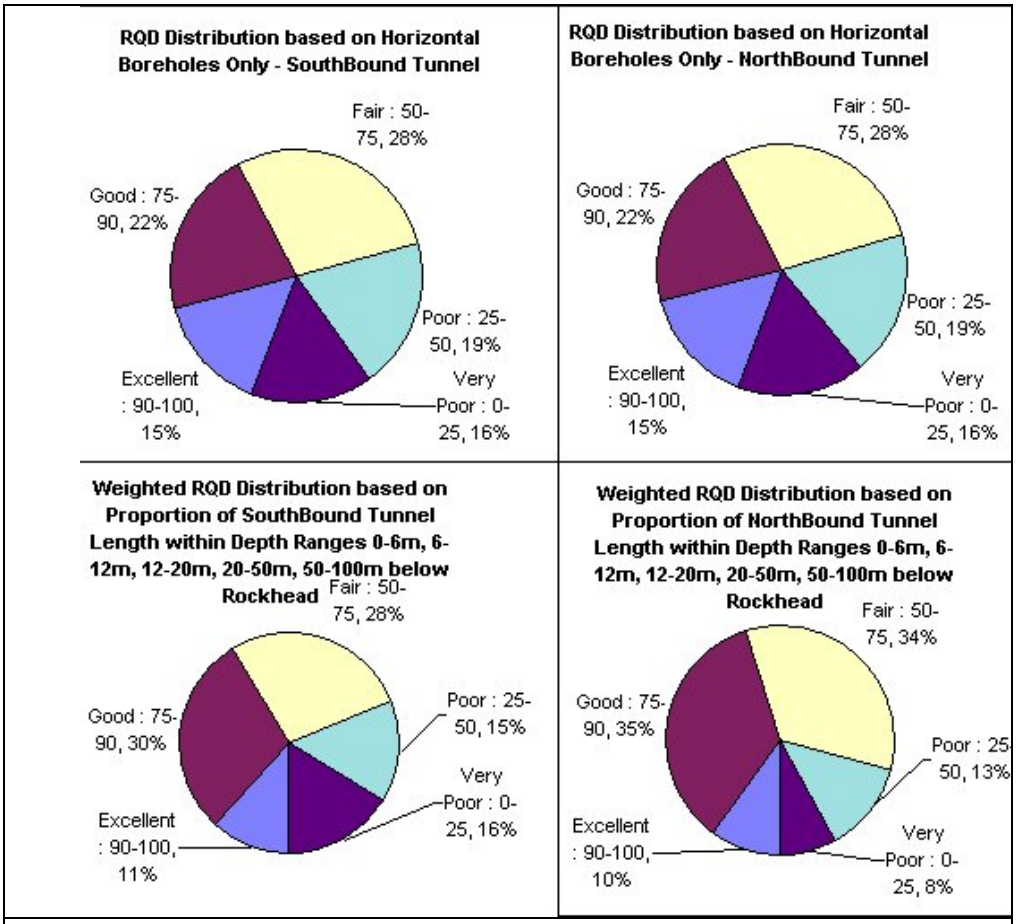

Figure 1. Weighted RQD-Range Percentages from
(a) Horizontal Core and
(b) Vertical and Horizontal Core divided into discrete depth ranges.

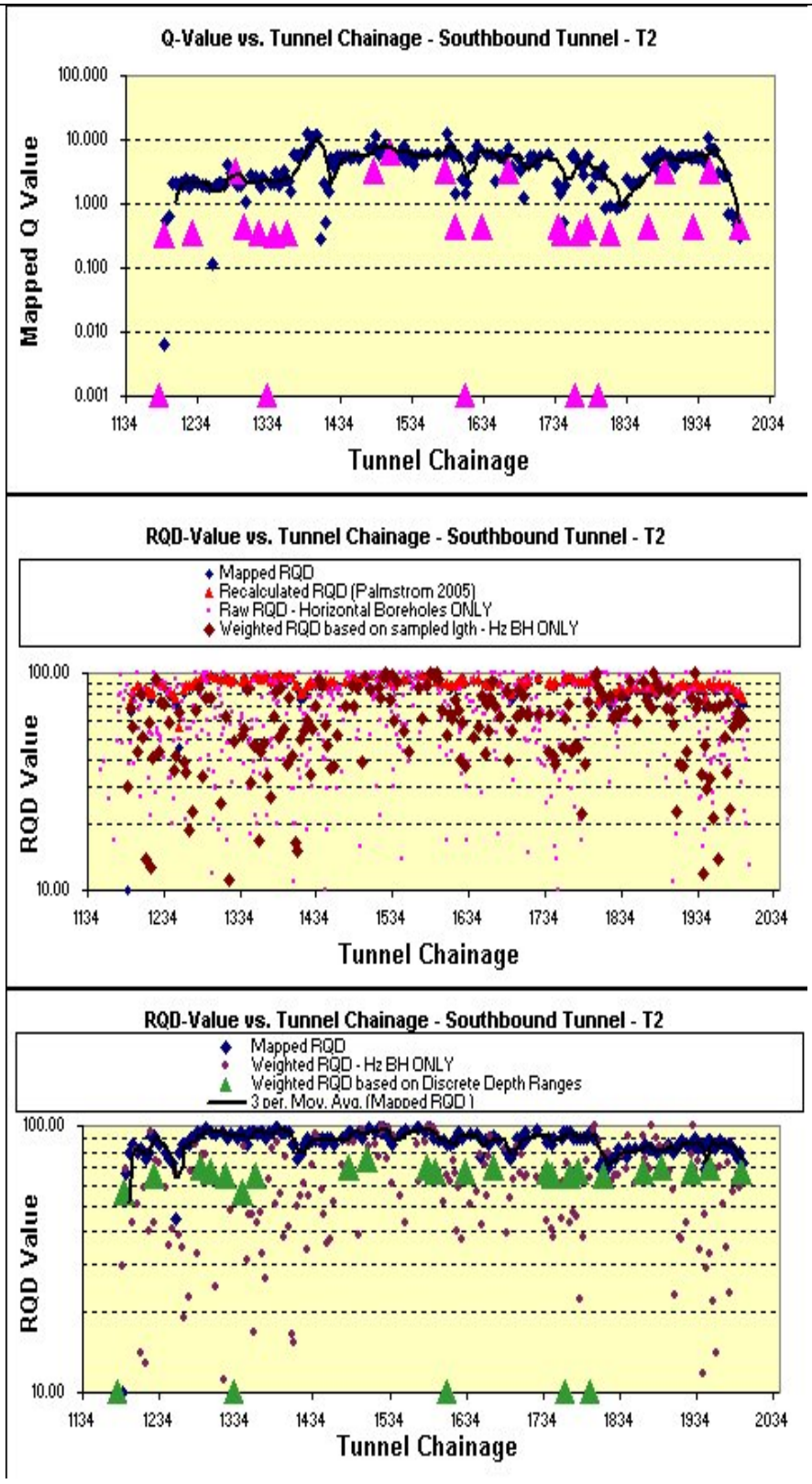

Figure 2. Variation of Q values and RQD values along Southbound Tunnel – T2

Pie chart results of the actual mapped RQD percentages are shown in the lower two charts in Figure 5. Clearly there is a significant departure from the proportions in Figure 1 above. The upper charts in Figure 5 represent the weighted average RQD.

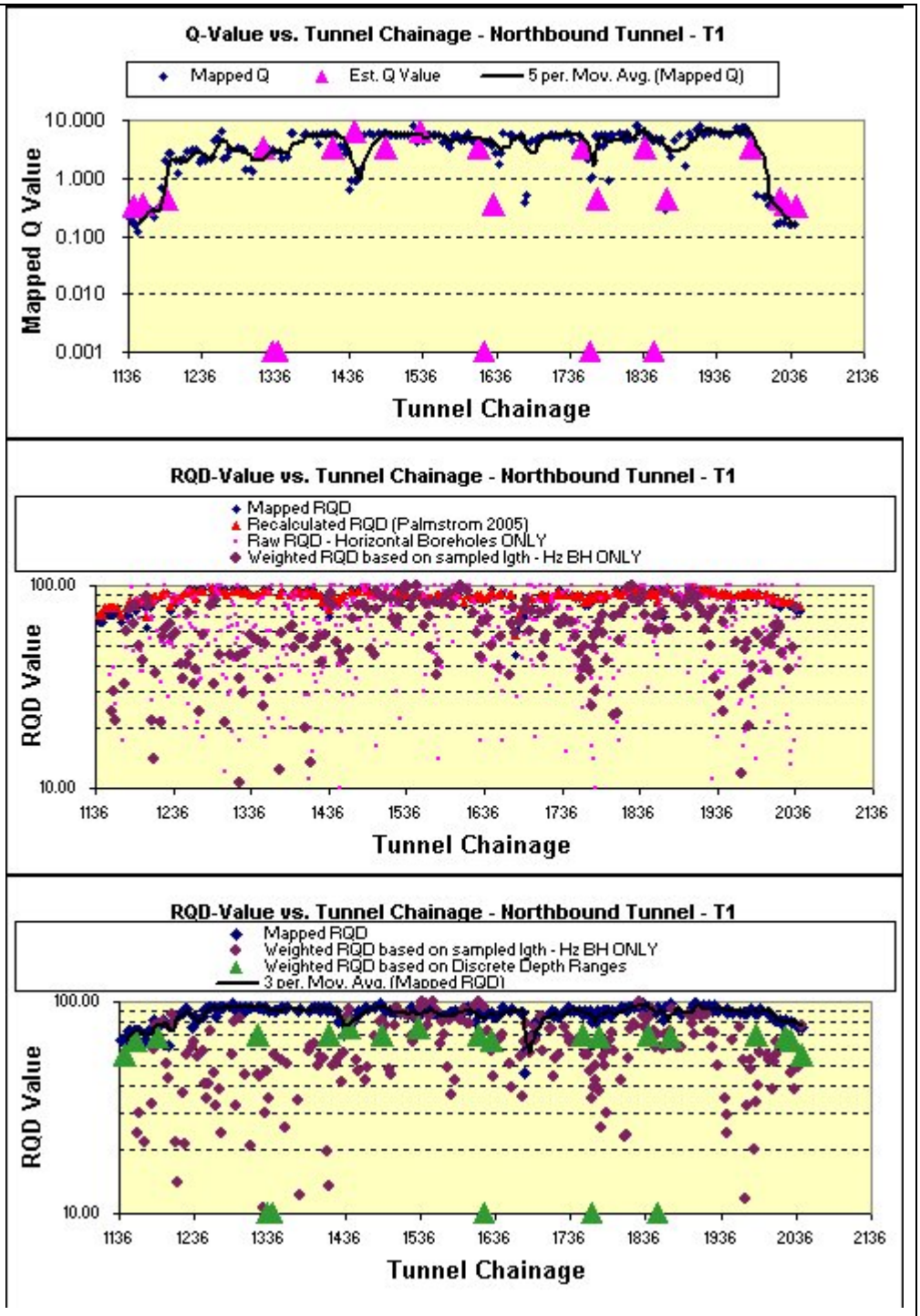

Figure 3. Variation of Q values and RQD values along Northbound Tunnel – T1.

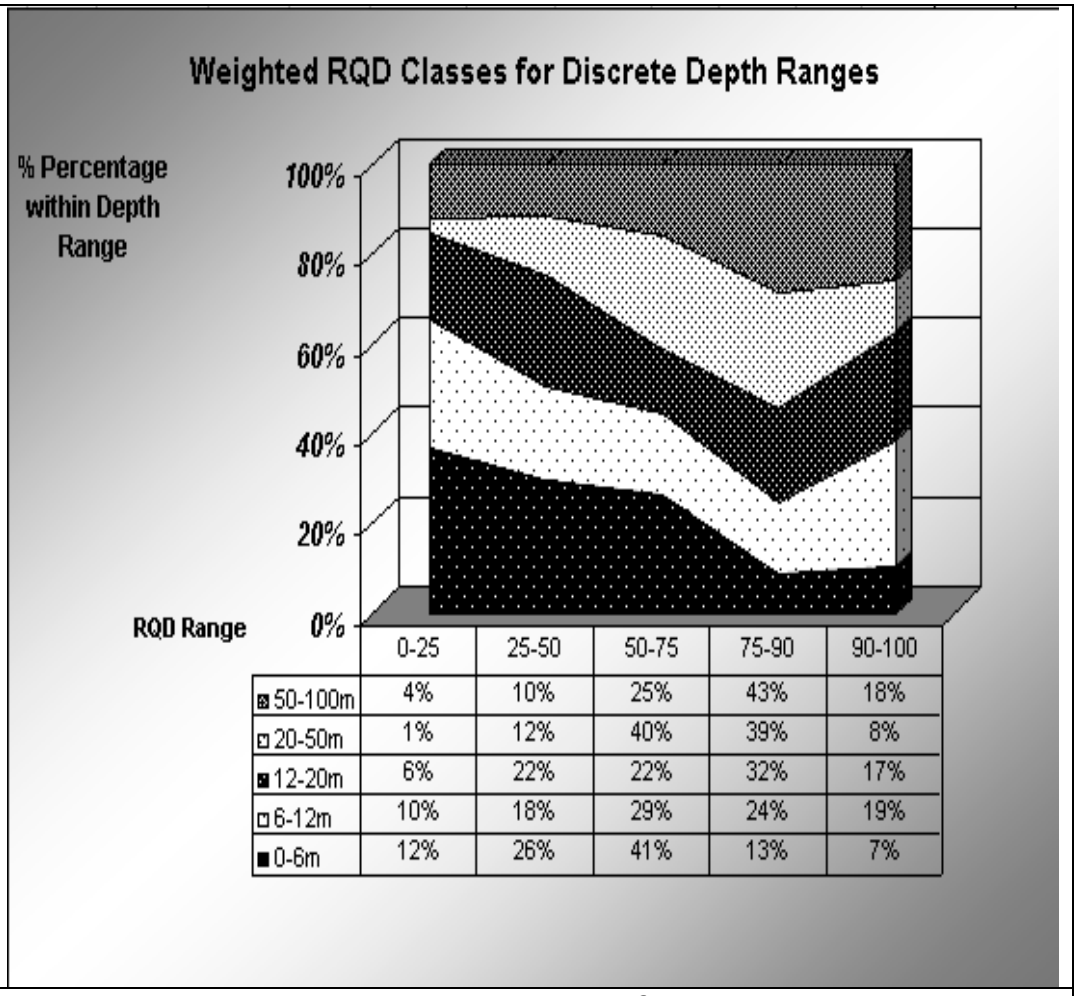

Figure 4. Weighted RQD-Range percentages from both Vertical and Horizontal Core for discrete Depth Ranges.

IMPROVEMENT OF RQD ESTIMATION IN TUNNELS

Improvement of RQD estimation in tunnels can be made by :

(a) Barton's (2003) suggested use of a histogram with a range of RQD values (and indeed all 6 Q-parameters) for estimation of the Q parameters is practical and sound advice;

(b) use of weighted RQD or weighted J_v values as suggested by Vardakos & Gutierrez (2005) are recommended for large tunnel faces/sidewalls based on a frequency count of observations, rather than a "visual assessment" resulting in a single parameter for RQD;

(c) longer (and more frequent) scanlines for large discontinuity spacing if scanline mapping is used. The greater the discontinuity spacing, the longer the scanline lengths required to reduce errors in RQD (Sen and Kazi 1984). Recent research has been conducted into performing this electronically (Lemy and Hadjigeorgiou 2004; Antony and Dove 2006), however as noted by Maerz and Germaine (1996), there is no simple replacement for a visual inspection by experienced personnel.

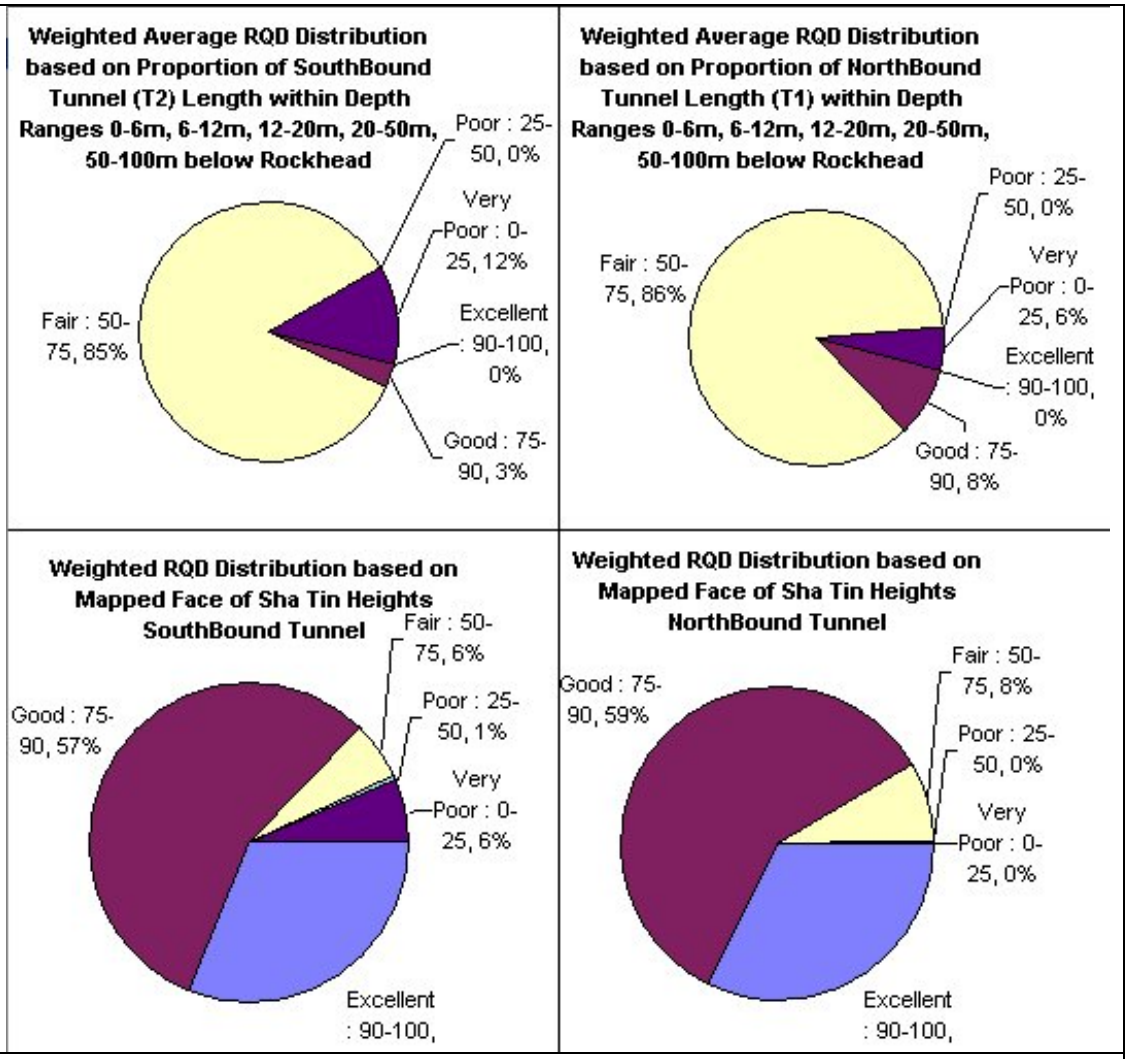

Figure 5. The lower charts show the actual Mapped RQD distribution.

CONCLUSIONS

RQD estimates from boreholes in the Shatin Heights Tunnel have tended to underestimate actual RQD values mapped in the tunnel itself. RQD distributions alone are not often identified as they generally become lost in the entire process of deriving Q values.

Further research would involve reviewing the core in the box against the mapped features in the tunnel as low RQD can represent simply a "minor shear" and not a regional fault.

Use of Volumetric Joint Ratio (J_v) is a quick method of checking RQD in large tunnels and has recently been revised to a slightly less conservative relationship than the earlier expression.

The orientation of mapped joints, if not adverse to stability during excavation (e.g. semi-perpendicular to alignment) and on a macro scale, may result in optimistic RQD values not entirely representative of the more micro-scale borehole core values. Horizontally drilled cores are along the tunnel alignment but the critical dimension for stability is often transverse the tunnel direction.

When reviewing core at pre-tender stage, care is required to follow the guidelines for RQD mapping and to temper the large variation in results with a review of structural features and joint orientations. A mathematical distribution can help define, but not replace, an experienced overall view.

Any misinterpretation of the RQD value, whether it be of the borehole core or in a tunnel itself, even as much as 50%, is not as major a contributor to errors in resultant Q values as some of the other more sensitive Q-System parameters, such as Ja or SRF which vary logarithmically with change in joint condition.

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GEOTECHNICAL ASSESSMENT OF PIPE JACKING THROUGH ALLUVIAL SOILS IN NORTHWEST NEW TERRITORIES

K.W. Mak[1], C.K. Wong[2] and P. K. Chan[3]

Abstract

This geotechnical assessment focuses on the estimation of tunnelling induced ground movements and the required pipe-jacking force. A qualitative risk assessment was also undertaken to estimate the risk rating of identified hazards and to prioritize risk treatment measures. A suite of geotechnical instrumentation is proposed to monitor ground movements, pore water pressure changes and pipe-soil interface friction.

INTRODUCTION

The scope of the sewerage project "Yuen Long & Kam Tin Sewerage Disposal, Kam Tin Trunk Sewerage Phase I & Au Tau Trunk Sewers" comprises construction of 5.8 km of gravity trunk sewers, 5.7 km of twin rising mains and three underground sewage pumping stations to serve the existing and upcoming developments and unsewered premises in the Nam Sang Wai and Yuen Long areas as shown in the site location plan in Figure 1. Since the proposed sewers traverse rivers and highways, it is proposed to utilize pipe jacking technique to facilitate sewer installation.

A geotechnical-cum-risk assessment of the proposed pipe jacking work was carried out to facilitate the project cost estimation, tender preparation and the checking in accordance with the Environment, Transport and Works Bureau Technical Circular (Works) No. 15/2005 "Geotechnical Control for Tunnel Works" ETWB (2005). The assessment focuses on the estimation of ground movements induced by tunnel boring machines (TBMs) and the required pipe jacking force. A qualitative risk assessment was also undertaken and a design-stage risk register prepared. It is revealed from the risk assessment that the most hazardous stage of the pipe jacking process is when the TBM is launching from or advancing into the tunnel eyes of the jacking and receiving pits respectively. In order to mitigate this risk, soft ground treatment work outside the tunnel eyes would be required to prevent ingress of soil and ground water. Site verification tests have been specified to ascertain the integrity and the effectiveness of the ground treatment works.

Based on the geotechnical assessment, field instrumentation has been specified to monitor the pipe jacking operation and ground response. The instrumentation includes strain gauges embedded in the concrete jacking pipes to measure the pipe-soil interface friction, surface settlement check points, extensometers and piezometers.

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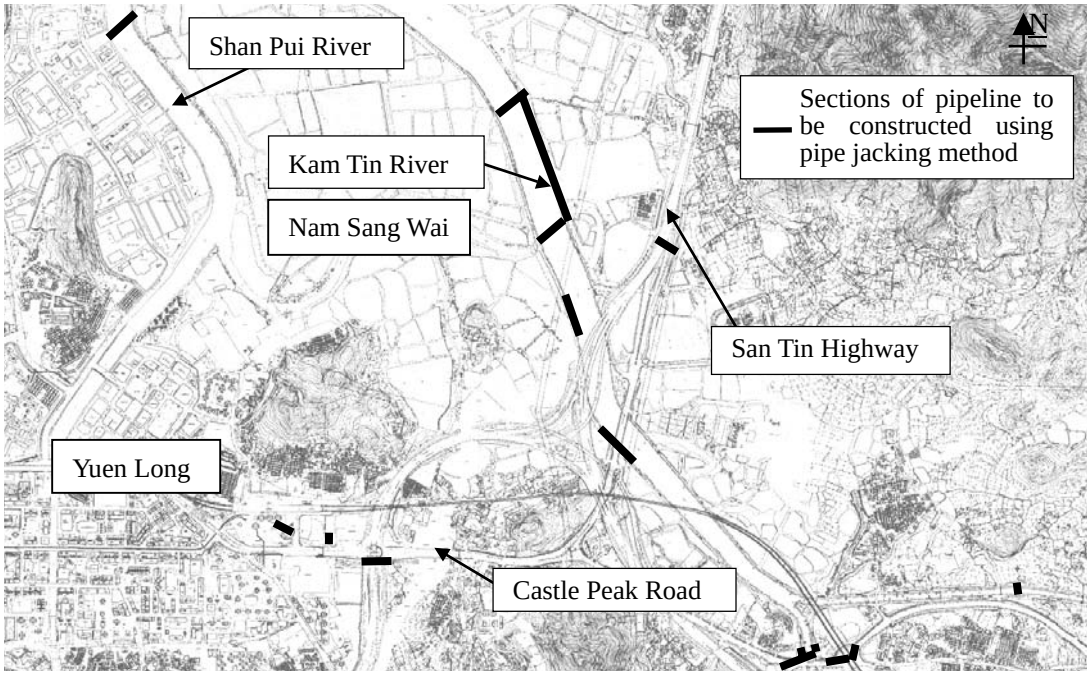

Figure 1. Locations of Proposed Sewers to be Constructed Using Pipe Jacking Method

PIPE JACKING

This project calls for the installation of gravity sewers and rising mains with internal diameters ranging from 250mm to 1350mm located approximately 3.5m to 11.0m below ground or river bed levels.

The geological sequence of the areas mainly consist of 0.5m thick fill, 0.5m to 7.0m thick clayey and silty marine deposits, 1.1m to 22.0m thick alluvial deposits comprising clayey to gravelly materials underlain by completely decomposed siltstone and sandstone. The groundwater table is about 3.5m below ground level. The typical geological profile of this site is shown in Figure 2.

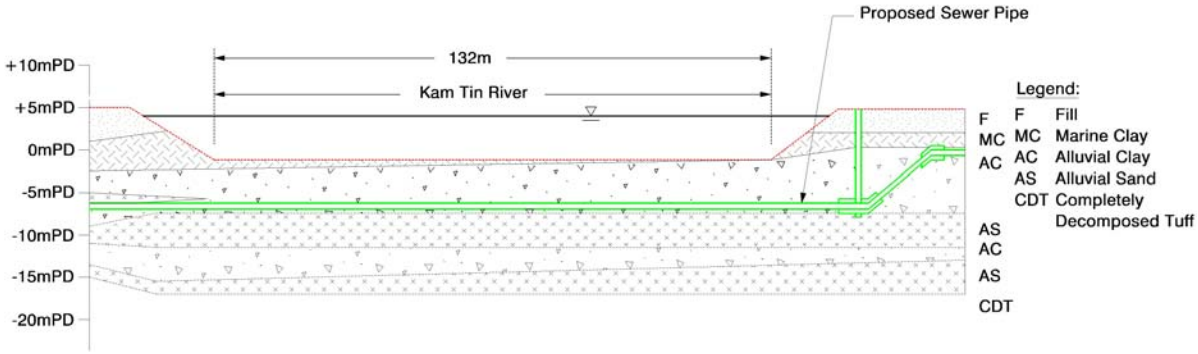

Figure 2. Typical Geological Profile of the Site (for Kam Tin River Section)

Closed mode slurry-pressure-balance (SPB) or earth-pressure-balance (EPB) TBMs are both suitable for the proposed pipe jacking work. In the SPB TBMs, bentonite or polymer slurry is employed to maintain the stability of the tunnel face. Whereas in the EPB TBMs, the excavated material mixed with foam or other soil conditioners is used to balance the external soil and hydrostatic pressures exerted on the tunnel face.

High strength spun concrete jacking pipes with a maximum length of 3.0m would be employed in this project. During pipe jacking, relatively compressible packing material distributes the load of the jacking force at joints between pipes to reduce the risk of excessive local contact stress and thus prevent spalling of concrete. Special pipe sections are also manufactured for intermediate jacking points.

Bentonite and/or polymer slurry would be used to lubricate the interface between the jacking pipes and the soil. The lubricating slurry would be injected into the overcut through openings (20mm diameter holes) in the jacking pipes. The same pipe openings would be used to inject cement grout into the pipe annulus space on completion of pipe jacking. Intermediate jacks may be placed at some points along the jacking pipeline just in case the jacking force exceeds the capacity of the main hydraulic jacks located in the jacking pit at the end of the pipeline.

TUNNELLING INDUCED GROUND MOVEMENTS

Mair (1998) listed the following principal components of ground movement associated with closed shield tunneling:

(a) Deformation of the ground towards the face resulting from any stress relief.
(b) Passage of the shield: the presence of an over-cutting edge combined with any tendency to deviate from the intended alignment i.e. over-cutting actions induced by the pitching and yawing motions of the TBM particularly in very soft ground or boulder zones.
(c) Tail voids: the existence of gap between the tail skin of the shield and the jacking pipes means that there will be a tendency for further ground movements into this annulus space.
(d) Deformation of the sewer pipes under the soil/water pressures.
(e) Consolidation: as pore water pressures in the ground change to their long-term equilibrium values the associated changes in effective stress may lead to additional ground movements.

Component (a) is of major importance in many cases. However, in the SPB or EPB mode, this component can be negligible if the face support load (P_s) is carefully controlled. Thus, if the face support pressure is less than the active earth pressure of the soil ($P_A > P_s$) ground loss will result which is transmitted to the surface as a settlement trough. Conversely, if the face support pressure is greater than the passive pressure of the soil ($P_P < P_s$), outward movement at the face will cause heave at the surface or leakage of slurry into the surrounding ground. The balanced situation is achieved where the face support pressure lies between the active and passive case ($P_P > P_s > P_A$). In addition, the external hydrostatic head will be balanced by the slurry pressure in the case of SPB TBMs. It is generally recognized that even in modern TBM control system, the principal control of face support pressure is in the hands of the TBM driver.

Component (b) can be appreciable if the over-cutting edge is of significant thickness, and if there are steering problems in maintaining the alignment of the TBMs.

Component (c) can be minimized by limiting the maximum size of tail void and immediate grouting of the tail void through the grout holes of the concrete pipes upon the completion of jacking operation.

Component (d) is considered negligible because of the rigidity of the high strength concrete pipes (grade 50 MPa concrete).

Component (e) can be of importance, particularly when tunnelling in soft clay. Therefore, slurry pressure will have to be carefully controlled in order to avoid overcompensation of soil/hydrostatic pressures which may result in the generation of excess pore water pressure and subsequent consolidation settlement.

Another important component which warrants special attention is over-excavation caused by the imbalance in the volume of excavation with the rate of TBM advance. A serious case of over-excavation may cause a funnel type face collapse resulting in surface depression. Over-excavation normally occurs when the TBM encounters mixed ground with boulders.

Surface Settlement

Peck (1969) and subsequently many other practitioners have shown that the transverse settlement trough immediately following tunnel construction is well described by an inverted Gaussian distribution curve (Figure 3) as:

$$S(x) = S_{max} \exp(-x^2/2i^2) \qquad \text{Eq (1)}$$

where

$S(x)$ = settlement at transverse direction x
S_{max} = maximum settlement on the sewer centre-line
X = horizontal distance from the sewer centre-line
I = horizontal distance from the sewer centre-line to the point of inflexion of the settlement trough, 3i is approximately equal to half the surface settlement trough width

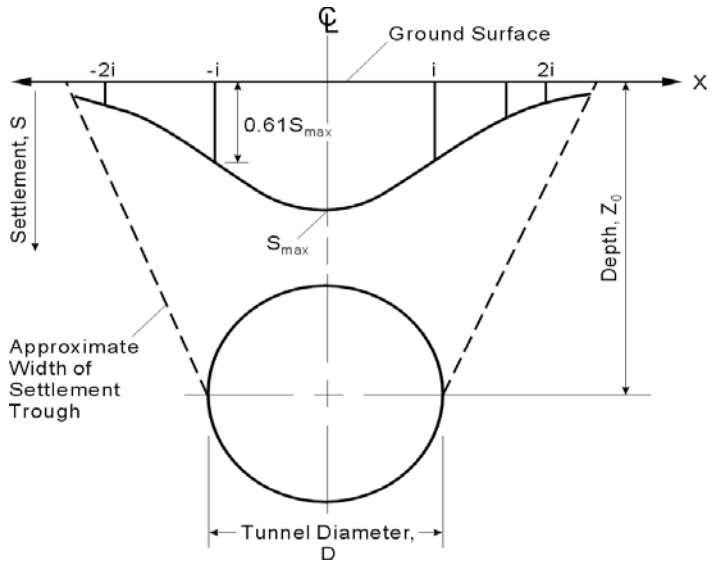

Figure 3. Surface Settlement Prediction using a Gaussian Distribution Curve

The volume of the surface settlement trough (per metre length of sewer), V_s, can be evaluated by integrating equation (1) to give

$$V_s = (2\pi)^{0.5} i S_{max} \qquad \text{Eq (2)}$$

The volume loss, V_t, is the amount of ground lost in the region close to the sewer, primarily due to one or more of the components (a) to (d) described above. When tunnelling under drained conditions, for example in dense sands, V_s is less than V_t because of dilation. In very loose (contracting) sands, movements, particularly in the vertical direction, will extend over a much larger distance and will probably reach the surface of the ground and therefore V_s is more than V_t. When tunnelling in clays, ground movements usually occur under undrained (constant volume) conditions, in which case $V_s = V_t$. Whatever the soil type, it is convenient to express the volume loss in terms of the volume of the surface settlement trough, expressed as a percentage fraction, V_l, of the excavated area of the sewer, i.e. for a circular tunnel of diameter D;

$$V_s = V_l \frac{\pi D^2}{4} \qquad \text{Eq (3)}$$

Peck (1969) suggested various relationships between the parameter i, tunnel depth and tunnel diameter, depending on the ground conditions. Many practitioners have investigated similar relationships e.g. Cater *et al.* (1984); Lake *et al.* (1992), and a simple relationship has been proposed:

$$i = K z_0 \qquad \text{Eq (4)}$$

where
K = trough width parameter
Z_0 = depth to tunnel axis

For clay/silt and sands/gravels, Mair (1998) suggested that K could be taken as 0.5 and 0.25 respectively.

Based on the cumulative probability function, and assuming deformations occur without volume change, the deformation at the sewer can be approximated as a linearly translating point source of loss. Since either SPB TBMs or EPB TBMs are to be employed, ground movements attributable to stress relief at the tunnel faces would be negligible. Therefore, the main cause of volume loss is the elastic deformations or partial/complete closure of the tail void created by the over-cutting actions of the TBMs. Bickel (1996) suggested that for fair practice in fast reveling ground, V_l could be taken as 2.5% with no volume change. For a perfectly aligned sewer, i.e. no wriggling along the intended alignment, it is assumed that the volume loss is equal to the volume of the tail void. Therefore, for a 1600mm external diameter concrete pipe, V_l = 2.5 % is equivalent to a 20mm tail void. Based on the foregoing, the estimated maximum ground settlement is about 17mm.

Mok (2002), reported that in the Central, Western and Wan Chai West Trunk Sewers Project, SPB TBMs were employed to install sewers (diameter up to 1800mm) in loose fill, marine and alluvial deposits. The measured ground settlement was generally less than 10mm but its magnitude increased rapidly when boulders were encountered.

PIPE JACKING FORCE

Jacking force (F_T) comprises two components: face resistance ($P_{s)}$ and frictional

resistance (F) along the circumference of the TBMs and concrete pipes. The face resistance is usually a small component of the overall jacking force. However, it can be significant for short drives, hard grounds or when the ground water table is high.

The following factors have a significant influence on jacking force:

(a) overcut;
(b) lubrication;
(c) steering; and
(d) initial misalignment

Frictional Resistance

In the estimation of frictional resistance (F), it is prudent to assume that the soil squeeze into the overcut and the concrete pipe is in full contact with the surrounding soil. The normal soil pressure acting on the concrete pipe can be estimated using the Terzaghi's "silo theory" (Stein *et al.*,1989). The theory assumes that the ground above the pipe is settling along two vertical planes (Figure 4). The reduced vertical soil pressure σ_{EV} due to the effect of arching is defined as:

$$\sigma_{EV} = \frac{b\,(\gamma - 2 \times C/b)}{2 \times K \times \tan\delta} \times (1 - e^{-2K(\tan\delta)H/b}) \quad (kPa) \qquad \text{Eq (5)}$$

where

H = height of soil cover from crown level (m)
γ = soil density (kN/m^3)
K = coefficient of soil pressure above the pipe = 1 (assumed)
b = influencing width of soil above the pipe (ideal silo width) (m)
= D(1+2tan(π /4 –ø/2))
D = pipe external diameter (m)
δ = angle of wall friction in plane of shear =ø (assumed)
C = cohesion (kPa)

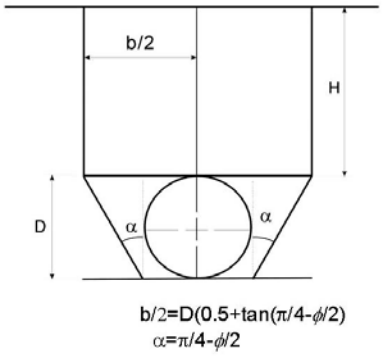

Figure 4. Boundary Planes of Wedges Failure.

The force N acting orthogonally to the pipe is defined as

$$N = L \times D \times \pi/2\{(\sigma_{EV} + \frac{\gamma \times D}{2}) + (K_o \times (\sigma_{EV} + \frac{\gamma \times D}{2}))\} \quad (kN) \qquad \text{Eq (6)}$$

where
L = drive length (m)
K_o = coefficient of soil pressure at-rest

The friction resistance (F) resulting from ground pressure is defined as

$F = \mu \times N$ (kN) Eq (7)

where μ : coefficient of friction between soil and pipe

Based on Stein *et al.* (1989), the commonly adopted μ values are tabulated as follows:

Type of Friction	μ
For static friction	
Concrete on gravel or sand	0.6
Concrete on clay	0.4
For dynamic friction	
Concrete on gravel or sand	0.4
Concrete on clay	0.3
For fluid friction	
When using bentonite suspension as supporting and lubricating fluid	0.3

Since the jacking pipes would be lubricated, it is proposed to adopt the static coefficient of friction $\mu = 0.3$ in the estimation of jacking force.

Face Resistance

The face resistance (P_s) is equal to a combination of the contact stress imposed by the soil and the ground water. It is considered appropriate to calculate the soil contact pressure by simple lateral earth formula.

$$P_s = \pi \times D^2/4 \times \{ K_o \gamma (H + D/2) + \gamma_w (h_w + D/2)\} \quad \text{(kN)} \quad \text{Eq (8)}$$

Pipe Jacking Force

The pipe jacking force $F_T = F + P_s$ (kN) Eq (9)

If the calculated jacking force exceeds the permissible axial strength of the concrete pipes and/or the capacity of the thrust wall, intermediate jacks would be installed to reduce the load on the main hydraulic jacks. Normally, intermediate jacks would be inserted into the pipe string when the jacking force reaches around 70% of the available jacking force or the concrete pipe working load.

Based on the above theoretical analysis, the estimated maximum jacking force for the 1.60m external diameter, 140m long jack pipe string is about 5800 kN which is within the 6240 kN design working load of the concrete pipes.

The pipe jacking force is transferred onto the soil by means of reinforced concrete thrust wall at the jacking pit. In order to prevent excessive wall movement, a factor of safety of 2

would be applied to the coefficient of passive earth pressure K_P adopted in the thrust wall analysis.

RISK ASSESSMENT

This project draws upon the guidance on risk assessment and management given in various reference documents (ETWB, 2005; GEO, 2005; Eskesen *et al.*, 2004). The selection of risk assessment techniques hinges on the availability of high quality statistical information on specific hazards. It is considered important not to "over-sophisticate" the risk assessment process beyond the level of accuracy and reliability of the statistical data.

It is considered adequate to employ relatively simple qualitative risk assessment technique for this pipe jacking project. During the design stage, experienced pipe jacking contractors were invited to participate in brainstorming sessions with a view to soliciting their opinions on project risks and options on mitigation measures. Before the brainstorming sessions, individual pipe jacking contractors were requested to fill in an abridged version of the risk register with a list of risk items pertinent to this project together with a questionnaire regarding the technical details of their TBMs and pipe jacking operations such as measures to overcome excessive jacking forces. Based on the documents returned from the pipe jacking contractors, the brainstorming sessions were focused on individual items listed in the risk register, and the capability and performance of their TBMs. The project team also consulted the site supervisory staff engaged in pipe jacking projects and their advice was also taken into consideration in the preparation of the risk register. Based on the inputs from the above parties, risks were mapped according to the likelihood and consequences of the event occurring, leading to an estimated level of risk of the event.

The risks were evaluated against both the likelihood and consequence with the product of them giving a risk rating. Based on the above risk mapping scheme, a risk register was prepared. The event's risk level determines its priority for treatment.

Based on the result of the qualitative risk assessment, some of the high risk events associated with TBM tunnelling work are highlighted below:

(a) ingress of soil/water from the tunnel eyes of the jacking/receiving pits;
(b) blow out of bentonite slurry to the ground surface or into the river.; and
(c) over-excavation of soil causing ground settlement.

Particular specification on risk management is incorporated in the tender documents so that the contractor is obliged to participate in the risk management activities and prepare risk reports. These activities include attending risk management workshops, meeting with relevant stakeholders, and preparing the contractor's own risk treatment plans, risk management plans and risk register. Prior to the commencement of work, a comprehensive risk register and risk mitigation/treatment plan would be prepared jointly by the contractor and the project engineer. Through the above risk management process, the risk register is reviewed and updated regularly.

RISK TREATMENT

The ingress of soil/water from the tunnel eyes has been accorded the highest risk score, therefore it is of paramount importance to ensure that proper precautionary measures must be

undertaken before the TBMs are allowed to exit or enter the temporary cofferdams.

The provision of grout plugs outside each jacking and receiving pit is one of the feasible means with which to prevent the ingress of soil and groundwater. The tunnel eyes are mainly located within the soft alluvial clay and sand deposits, therefore jet grouting (suitable for soft clay materials) and two-stage bentonite-cement/chemical (suitable for sandy/silty materials) Tube-à-Manchette (TàM) grouting techniques may have to be employed to strengthen the weak soils outside the tunnel eyes.

Where jet grouting technique is employed, it is proposed that a cased borehole of at least 150mm diameter should be formed at each jet grout location to assist the escape and venting of the slurry flush material in order to prevent ground heave or lateral ground movement attributed to the blockage of jet grout holes. Trial test would be carried out to establish the key parameters for jet grouting work (e.g. grout hole spacing, injection pressure and rate, lift and rotation speed of the nozzles, grout mix, compressive strength of the jet grouted soil, etc).

Boos *et al.* (2004), reported that in recent years, a number major incidents that occurred during tunnelling with TBMs were caused by poor quality grout plugs. In order to verify the strength, integrity and the overlapping of the jet grout columns, horizontal probe holes would be carried out to penetrate the jet grouted zones. The water tightness of the jet grouted zones would be assessed based on the amount of water inflow collected from individual probe holes. The unconfined compressive strength of the collected soil-cement core samples would also be verified.

Similarly, where two-stage TàM grouting technique is employed, site trial would also be carried out in order to establish the optimum grout mix, grout hole spacing, injection pressure/rate, and grout volume. Site verification tests similar to those specified in jet grouting would also be carried out.

INSTRUMENTATION

Instrumentation employed for the monitoring work is divided into two categories: pipe jacking and ground response. The former includes instrumentation arrays in the jacking pits and pipe string for measurement of the jack loads and pipeline advance, and the latter arrays of geotechnical instruments are to measure ground deformation and pore water pressure changes.

Pipe jacking instruments: In the jacking pit, heavy-duty load cells would be installed on the main jacking rams to measure total jacking loads. A linear displacement unit would record the forward progress of the pipeline. Strain gauges are proposed to be embedded in the concrete pipes located directly behind the TBMs to measure the pipe-soil interface friction. Two types of high temperature strain gauges, electrical resistance (ER) gauges and vibrating wire (VW) gauges, are proposed to provide redundancy and an independent cross-checking of the results.

Ground Instrumentation: Instrumentation to monitor ground deformation and pore water pressure comprises, extensometers, piezometers and surface settlement check points. Instrument clusters would be positioned along the sewer alignments for settlement troughs measurement. Extensometers and piezometers would be installed in boreholes at 1.0m above the crown and at axis level. Ground monitoring would be continued for at least two

months after the completion of pipe jacking work with a view to verifying the effectiveness of tail void grouting and confirming the existence of any time-dependent ground settlements caused by the dissipation of excess pore water pressures generated during tunneling work.

CONCLUSIONS

A geotechnical-cum-risk assessment was undertaken to assess the effects of the pipe jacking work on the surrounding ground, evaluate the identified risks and prioritize risk treatment measures. During the preparation of the design-stage risk register, advice from pipe jacking contractors and site supervisory staff conversant with pipe jacking technique was considered invaluable. Ideally, high quality data on specific hazards or past incidents should be readily available to the practitioners for reference. One obstacle to learning from case histories is litigation or commercial confidentiality, which might prevent the full disclosure of information about "near miss" cases or failures to the extent that the practitioners might not be able to learn about them.

It is considered that more sophisticated risk assessment methods might be used when more accurate and reliable information is available. Moreover, most risk assessments are subjective and need to be recognized as such. Practitioners should be aware of the benefits and limitations of such assessments. Professional judgment is one of the essential elements in the risk assessment process. Formal risk assessment should only be used as an aid to judgment. Even the most sophisticated risk assessment techniques and comprehensive case history database should not substitute judgment based on adequate knowledge and experience.

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Sha Tin Heights Tunnel : A Review of Factors Affecting Excavation Cycle Times

J. K. Murfitt[1], Cameron Chin[2], Billy Siu[3] and C. H. Chan[4]

Abstract

Excavation progress for the Sha Tin Heights Tunnel was constrained by a combination of adjacent sensitive features, noise regulations, access restrictions, weather, construction equipment, blasting technology and programme. This Paper presents and discusses how the blast, mucking out, temporary support and excavation cycles were monitored during the course of the Works. Various proposals, both put forward and implemented, to minimise cycle times and increase excavation advance rates within the framework of the external constraints are also discussed.

INTRODUCTION

The Sha Tin Heights Tunnel (SHT) is a twin bored 19 m span and 11 m high road tunnel of around 1 km in length, excavated mainly through coarse-grained granite, with occasional basalt dykes. One portal drive (south portal, southbound tunnel) was excavated through approximately 50 m of soft and mixed ground before encountering rock with a design 6 m cover over the tunnel crown, and the remaining three portals commencing in Grade III rock. A number of possible faults along the proposed route were identified at tender stage. One of these was observed during the tunnel excavation.

Excavation progress in any drill and blast tunnel is dependent on a number of interrelated factors, some of which have a greater impact on progress than the others. Some of these factors are variable (e.g. charge weight) and some fixed (e.g. the surrounding geology), but all of the factors combine to define the eventual rates of advance.

Measures were put into place at the SHT to review some of these factors as works progressed, with those identified as having the greatest impact, and those with the greatest scope for improvement of progress being afforded the closest attention during construction.

Assuming that the size of the excavation, the surrounding geology and topography remains fixed, those factors identified as having the greatest scope for review on the SHT can be separated into the following broad categories :

(a) Working cycle times - this includes the number and type of elements in an excavation cycle and the resources required for minimising the time required for each cycle;

(b) allowable working hours using machinery, to comply with statutory noise regulations - working hours were agreed in consultation with the Environmental Protection Department (EPD);

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(c) efficiency of blasting - this category includes blasting technology, use of bulk emulsion versus cartridge emulsion charges, possible use of electronic detonators to cover larger face area with lower vibrations and more efficient blast patterns, blast vibration attenuation laws, and burden; and
(d) vibration limits on sensitive receivers - this includes the applicability and accuracy of both the parameters, and the approaches themselves, used to derive Peak Particle Velocity limits (PPVs) for sensitive receivers, such as slopes; a study of PPV limits for one particular slope is discussed.

Within each of the above categories there are in turn various approaches and methods of achieving the greatest efficiency in performing the works, in terms of progress. This Paper discusses examples from within each of the above four categories and presents monitoring results of working cycles and progress achieved.

SITE CONSTRAINTS FOR TUNNEL EXCAVATION

Site constraints were identified prior to tunnel commencement and categorized to determine their impact on the excavation. The following is a broad outline of site constraints on the SHT Project :

Site Constraint 1 : Vibration Limits of Adjacent Sensitive Receivers

When conducting any excavation works by blasting in Hong Kong, particularly in urban areas, most of the site constraints arise due to sensitive receivers adjacent to the blast zone. Flyrock, vibration, airblast, noise and fumes during blasting may individually, or in combination, cause adverse effects to public safety, and may cause human discomfort, cracking or in severe cases, structural distress.

Vibration limits for blasting currently set by commercial and residential building owners and by utility companies in Hong Kong are normally stated as single peak particle velocity (PPV) vibration levels, with no frequency component. In the case of Water Services Department (WSD) structures, both vibration and amplitude limits are defined by the WSD.

The main sensitive receivers for blast vibrations are shown in Figure 1. The sensitive receivers that are shown limited the allowable charge weights per delay that were able to be used in the excavation cycle, in turn, limiting the advance for each blast.

Most of the vibration limits of these features were fixed by owners and utility companies, however, as the slopes were assessed separately, there was scope for review of the PPV limits by revisiting the models used in the initial assessments. A more detailed discussion of the reassessment of these slopes is presented below. Slopes along Tai Po Road presented one of the major controls over allowable charge weights per delay.

The crowns of two approximately 3 to 3.2 m span WSD water tunnels were located less than 18m below the SHT invert level. In accordance with WSD's requirements, a zone within 60 m plan distance of each tunnel was designated as a WSD Special Protection Zone. These zones reduced charge weights to a minimum approaching the WSD Tunnels, due to concerns over vibration (causing rockfalls or leakage) and tunnel deformation. Figure 2 below plots charge weight and pull length along the northbound tunnel alignment and shows a marked drop in allowable charge weights around the WSD Tunnels. Pull length is the length the face advances per round i.e. face advance after each blast.

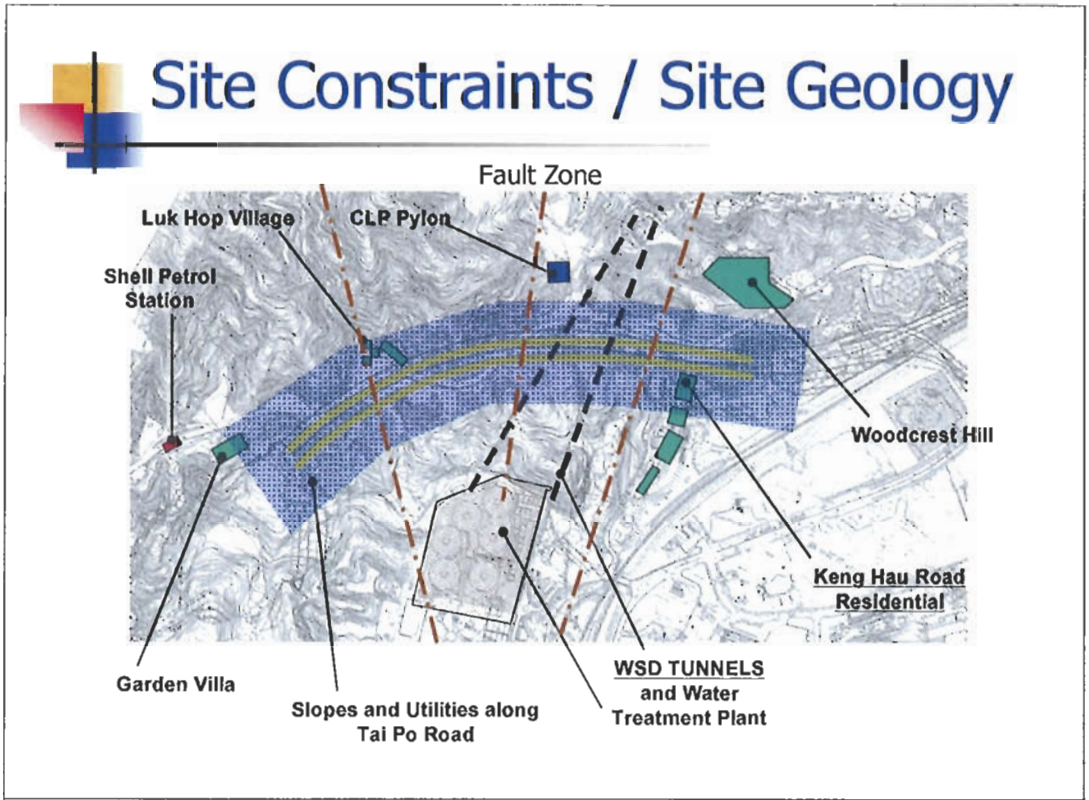

Figure 1. Excavation Progress Constraints

Site Constraint 2 : Noise

No blasting was conducted (nor permitted) at night time as the noise generated by the detonation of explosives may have caused discomfort to the residents of the properties in close proximity to the blasting zone (e.g. Garden Villa and the Shatin Heights residential complex), and had the potential to panic road users.

One of the major activities in the blasting cycle, mucking out, is also the most time consuming one. Working hours for mucking out were limited by the Noise Control Ordinance (NCO). As such, no mucking out was permitted after 19:00 hrs on normal working days.

Site Constraint 3 : Temporary Access Roads (TAR)

Tunnel spoil material removed after blasting was either transported off-site immediately or stored in the temporary stockpile near the portal areas prior to the removal off site via the temporary access roads (TARs) at the South Portal (TAR1) and North Portal (TAR3). Owing to the high traffic flow of Tai Po Road, which was the designated exit point of both TARs, a large number of dump trucks were used during the excavation cycle, with truck movements along TAR1 reaching a peak of 40 trucks/hr. The high traffic flows along Tai Po Road placed restrictions on the maximum combined numbers of trucks able to use TAR1 and TAR3 simultaneously.

Site Constraint 4 : Drill Length, Drilling & Blasting Plant and Equipment

The drill length was dependent on the maximum allowable charge weight per delay. This resulted in a drill length of, on average, 1000 mm plus the length of the charge column which ensures the explosive is placed an adequate depth beyond the free face. This provides

appropriate confinement for efficient blasting and ensures adequate stemming-plug depth can be provided.

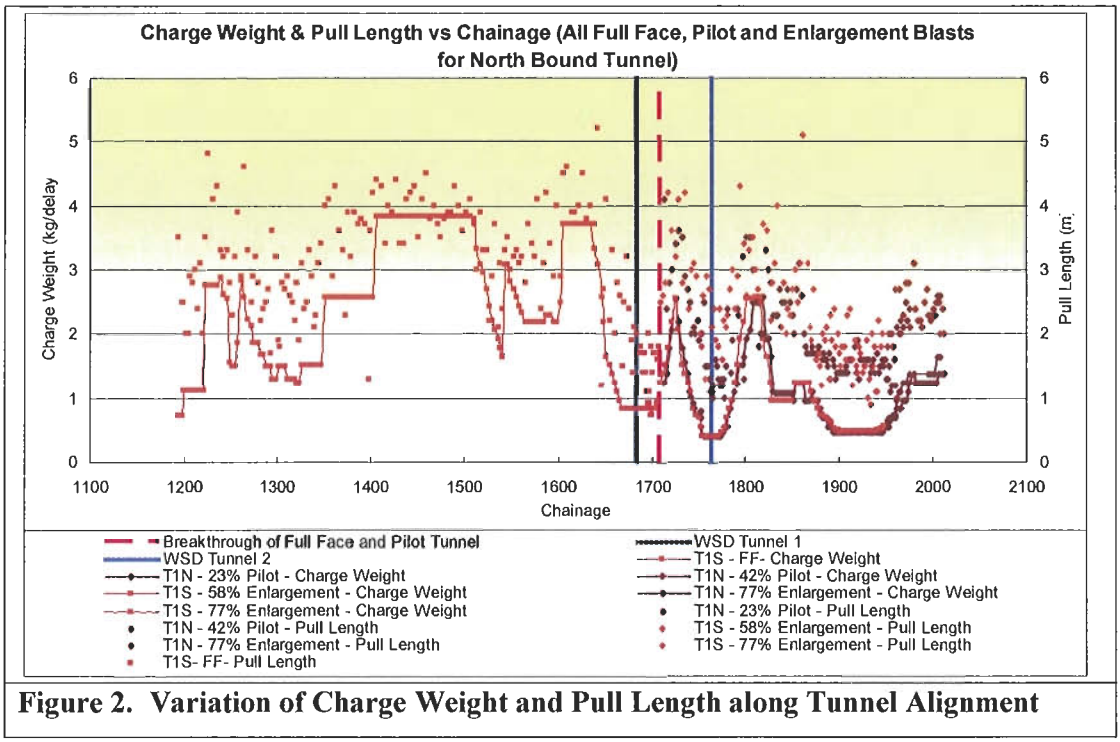

Figure 2. Variation of Charge Weight and Pull Length along Tunnel Alignment

Drill lengths varied from 0.5 m to 4.1 m and were dependent on the allowable charge weight per delay. While allowable charge weights per delay permitted longer drill lengths at some sections of the tunnel, the maximum length drilled was 4.1 m due to two factors :

(a) limitations of the tunnel drill rigs : a single drill rod was able to drill 4.1 m and longer lengths could be drilled if necessary, but would have required adding new drill rod by hand, as this process could only be automated for single drillholes, not for a full blast pattern. To drill deeper, a jumbo with longer feeds would be required. The feeds on the large 3-boom Jumbos used were 4.3 m (14') long and the 2-arm Jumbo 5.5 m (18'). These feeds were used with the hydraulic rock drill COP1838 which has the most effective output power for blasthole drilling. The full 165 m^2 face with a typical 238 drill holes could be drilled within the cycle times tabulated in Figure 5, with automatic positioning of the boomer arm, however, a change of drill rod would mean a considerably increased drilling cycle, possibly extending the cycle to the next day, delaying the blast. It was thus determined, even at chainages where charge weights would have allowed a longer pull length, to maintain the drill length at 4.1m. The concept of extension of drill rod is suitable for surface vertical drilling but not tunnelling. The automatic extension rod system used on a surface drill rig, for vertical holes, would be too heavy for a Jumbo feed which is cantilever supported. It is also dangerous to extend the drill rod alongside the other 2 operating booms on a three boom jumbo. For safety reasons, the Jumbo has sensors near the booms and the booms stop automatically if an object or person comes close to the booms during operation. This means all 3 booms stop during extension of the drill rods.

(b) mucking out and support cycle : restrictions on noise levels meant that even had a longer pull been designed, the resultant size of the muckpile would have required a

longer mucking out and support cycle period. In order to avoid unnecessary stockpiling and double-handling of the spoil, haulage of the excavated rock off-site was performed by trucks only during times permitted by the Environmental Protection Department. This limited the total volume of spoil able to be hauled off-site during the mucking out operation, hence longer drill lengths were not considered an advantage to the overall drill and blast cycle.

In addition to the tunnel drill rigs, the efficiency and periods of downtime, for maintenance purposes, of other tunnel plant (excavators and loaders) and equipment (e.g. the shotcreting rig) controlled the blasting cycle.

Site Constraint 5 : Explosive Delivery and Capacity of Site Magazine

A site magazine was approved by the CEDD Mines Division (representing the Commissioner of Mines) to be built on site under the SHT project. This magazine had a maximum capacity to store 800 kg of cartridge explosives and 200 kg of other DG-Cat I accessories such as detonators and detonating cords. With a maximum charge weight of 3.8 kg/delay x approximately 213 holes/face (1 delay/hole) = approximately 850 kg/blast, the capacity of the site magazine was insufficient for 4 blasts per day during the peak period of the project. Daily explosive deliveries by the Mines Division were thus required to supplement explosive supply.

The daily explosive delivery also affects the blasting cycle, as the timing of delivery is partially dependent on CEDD Mines Division and is also subject to both weather and supply-route constraints.

Site Constraint 6 : Adverse Weather

Delivery of explosives is not recommended when a thunder storm warning or a tropical cyclone warning signal. No 3 or higher is in force. The numbers of blasts affected by adverse weather is shown in Figure 3 below.

Site Constraint 7 : Program

An additional site constraint governing the tunnel excavation method was the key-date for possession of areas of site. For the two temporary access roads, TAR1 at the South Portal and TAR3 at the North Portal, the site area required to construct TAR1 was available at the commencement of the Contract while the site area required for TAR3 was only available 4 months after Contract commencement.

Apart from normal key dates for completing the tunnel lining for handover to third parties for electrical and mechanical (E&M) installations, the early removal date of TAR1 required a tunnel breakthrough on or before the TAR1 removal date, so that explosive delivery from the magazine at the south and in particular stockpiled spoil removal could continue unimpeded through the northern access.

Considering the removal date of TAR1 and the earliest start date to construct TAR3 was later than that for TAR1, the Contractor proposed, and implemented, pilot tunnel excavations from the North Portal to meet up with the full face drive extending from the South, followed by enlargement of the pilot tunnel from both north and south ends of the pilot drive. The tunnelling sequence is shown in Fig. 4, with the extent of the pilot tunnels shown at the right hand side of the figure.

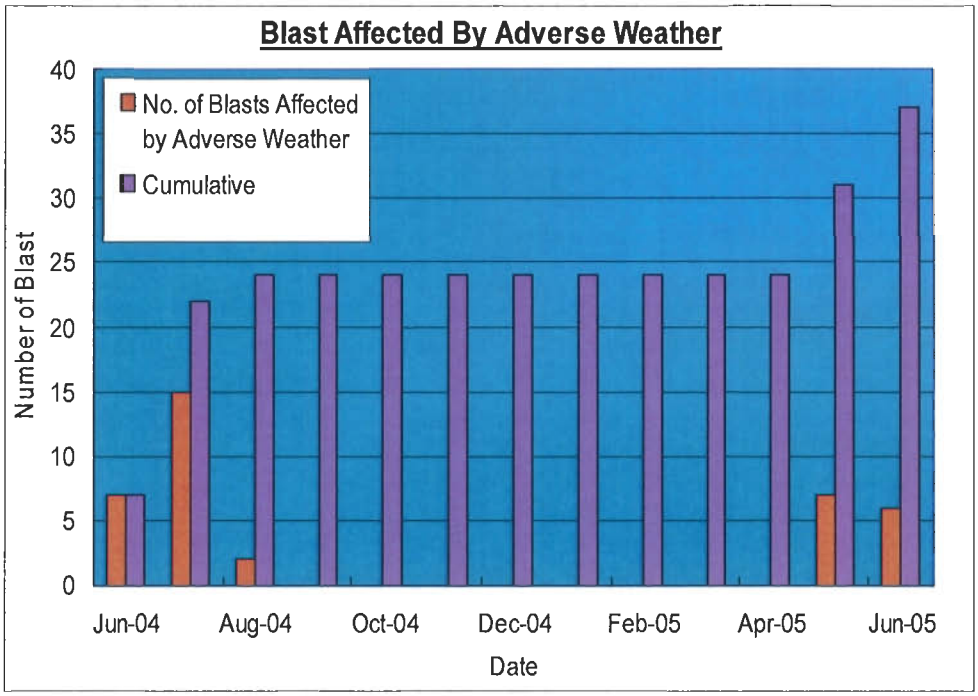

Figure 3. Blasts Affected by Adverse Weather

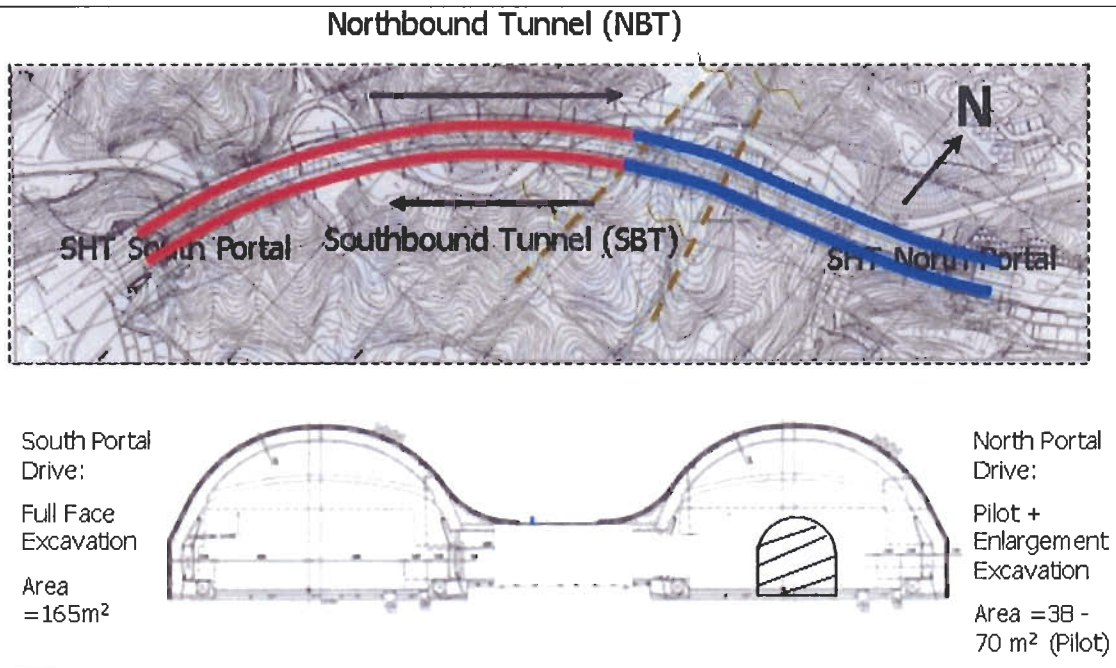

Figure 4. Tunnelling Sequence

TUNNELLING CYCLE

The sequence of a normal drill and blast cycle is listed below :

(a) *Profile survey and setting out :* Surveyors will check the tunnel profile and set out the alignment prior to any construction works.

(b) *Probing :* Probing is carried out to ensure probed material is always 20 m in advance of the tunnel face. When water inflow from probe holes exceeds the specified limits, grouting will be conducted in advance of the excavation. The maximum pull lengths for blasts in the SHT are approximately 5 m.
(c) *Charging and blasting :* The blasting engineer will design blasting patterns for every blast, which are checked and certified by the independent blasting consultant (IBC). Charging by the shotfirers will be carried out after the completion of blast hole drilling. After blasting, the tunnel ventilator will be switched on to provide fresh air inside the tunnel.
(d) *Mucking out and scaling :* Mucking out and scaling will be carried out by loader and hydraulic breaker respectively. Upon completion of mucking, loose rock at the exposed tunnel face will be scaled off by a hydraulic breaker.
(e) *Face Mapping, Surveying and Installation of Temporary Support :* The Contractor's geotechnical engineers will inspect and map the exposed tunnel face in order to determine the support class and the corresponding temporary support measures. After the completion of their inspection, the survey team will check the profile and the miners will prepare the installation of temporary support, if any. Face mapping sheets and all subsequent temporary support that is required are certified by an independent checking engineer (ICE).
(f) *Vibration and Convergence Monitoring :* Vibrations resulting from each blasting operation will be recorded after each blast and the results will be checked by the blasting engineer. Convergence monitoring pins will be installed when the rock class is determined to be lower than Class 5 (for the SHT Tunnels) in accordance with Barton's Q system.

The inspection procedures are implemented by the Contractor's Geotechnical Engineers, and the results of the inspections are checked by the Consultant's Geotechnical Engineers, following the completion of mucking out and scaling, in the sequence listed below :

(a) inspect the excavated roof, wall and tunnel face;
(b) inform the tunnel staff to continue further scaling if deemed necessary;
(c) inform the tunnel staff to install rock bolts at the required locations if deemed necessary;
(d) collect information for the determination of the Q-value; and
(e) design temporary support required, such as pattern rock bolts and thickness of shotcrete, as per the mapping record sheets.

Before the commencement of the subsequent drilling and charging cycle, a visual inspection will be carried out by the Geotechnical Engineers to verify if further scaling is required. The General Foreman and Shotfirer also visually check the excavated face for any loose material or blocks that require either support, or scaling.

Drill and blast excavation was carried out throughout the rock formation except for excavations through weak zones and the soft ground at the portals, where mechanical excavation methods were employed. Before excavation, monitoring instruments such as ground settlement markers, extensometers, tell-tales and piezometers were installed at the required locations on existing slopes, buildings, private and commercial facilities, utility installations and roadways). Initial readings were then recorded and checked by surveyors and the Contractor's geotechnical engineers. In weak ground, and in defined areas, such as the WSD Zone, convergence markers were installed in the tunnel periphery.

Details of the total length of the tunnels, total explosive used, rock excavated, shotcrete and other support applied, and number of rock bolts installed, are presented in Table 1.

Table 1. Tunnel Excavation Summary

Total Length of the tunnel				Explosive Usage	Weight (Kg)		
Tunnel	Chainage		Length (m)	Bulk Emulsion	84650		
	From	To		32mm Cartridge (Packaged) Emulsion	123418	Totals (kg)	Totals (t)
Northbound	1136.4	2041.22	905	25mm Cartridge (Packaged) Emulsion	9597	217665	218
Southbound	1134.7	1994.38	860	Tunnel Face Area	Full Face	166	m^2 Approx
Total Length =	1765	m			Pilot	38~70	m^2 Approx
Total Volume Rock Excavated:				Average overbreak after smoothing:			
Northbound Tunnel	161675	m^3 Approx.		Northbound	286	mm	
Southbound Tunnel	158581	m^3 Approx.		Southbound	297	mm	
Cross-passages (CP2-CP10)		2125	m^3				
Average Powder Factor =		0.89	kg/m^3	Volume of shotcrete applied:		7600	m^3
				Tunnel Periphery Length Approx.		27	m
				Calc. Average Shotcrete Thickness		160	mm

Temporary Support No. of Bars:		
Dowel Type	No.	Approx. bars/m^2
Swellex	4367	0.14
Resin Anchored	2393	Approx. m^2/bar
Totals	6760	7

No. of Drillholes for Full Face Blasts	
Max: 286	Drill Length:
Min: 213	Max: 4.1m
Average: 250	Min: 0.5m
	Average: 2.3m

Length of Softground support:			
Tunnel		Length (m)	Totals (m)
South Portal	Northbound	38	
	Southbound	36	74
North Portal	Northbound	25.5	
	Southbound	9	35
Totals		109	(m)

Tunnel Soft Ground Support UC/Lattice Girders		No. of UC Girders	No. of Lattice
North Portal	Northbound	17	0
	Southbound	6	0
South Portal	Northbound	6	44
	Southbound	11	35
	Totals	40	79

MONITORING OF PROGRESS

Tunnel progress was monitored against the Contractor's programme using activity durations recorded from the works activities in the excavation cycle. These activities are :

(a) charge hole drilling;
(b) charging and blasting;
(c) mucking out and scaling; and
(d) face mapping, surveying and temporary support installation.

A summary of the resultant average excavation cycle times for each of the full face and pilot face area is presented in Figure 5 below. Figures 6 shows the weekly advance rates achieved.

Note that for comparison with full face advance rates, at locations where a pilot, benching or enlargement drive was conducted, the percentage face area (compared to a full face tunnel) multiplied by the actual advance was used to calculate equivalent full-face drive progress.

Figure 7 graphically presents the typical relationship between pull length and charge weight per delay for the Shatin Heights Tunnel. Plotting pull length versus the charge weights per delay used permitted an assessment of future progress to be made, as charge

weights per delay were already known at each chainage interval along the alignment based on the Contractor's blast assessment report.

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Summary of drilling cycle time (excluding Heading and Benching section)

Type of Blasting	No of Blasts	Average Excavation Time per Blast (hr)				
		Drilling	Loading & Blasting	Mucking & Scaling	Support Installation	Total
Full face	324	2.4	2.0	9.5	3.9	17.8
Enlargement (77% to be enlarged)	217	1.8	1.5	7.4	1.7	12.2
Enlargement (58% to be enlarged)	43	1.5	1.3	7.9	2.0	12.8
Pilot (42% eq. FF)	63	1.8	1.1	5.6	0.7	9.2
Pilot (23% eq. FF)	255	1.5	1.1	3.0	0.5	6.1

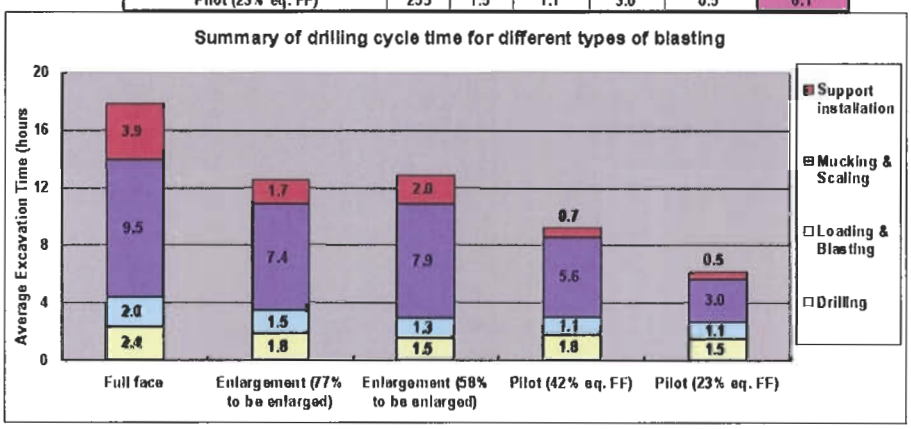
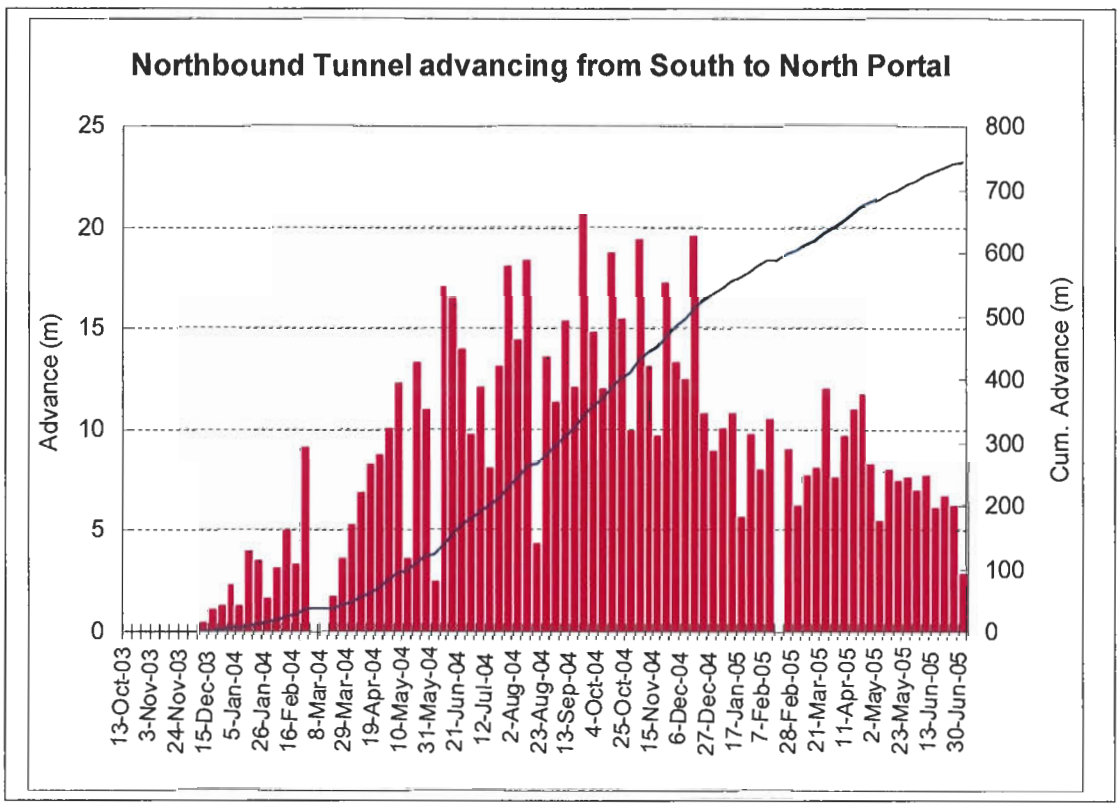


Figure 5. Summary of Cycle Times for Various Blast Faces (Full Face = 165m^2 approx.)



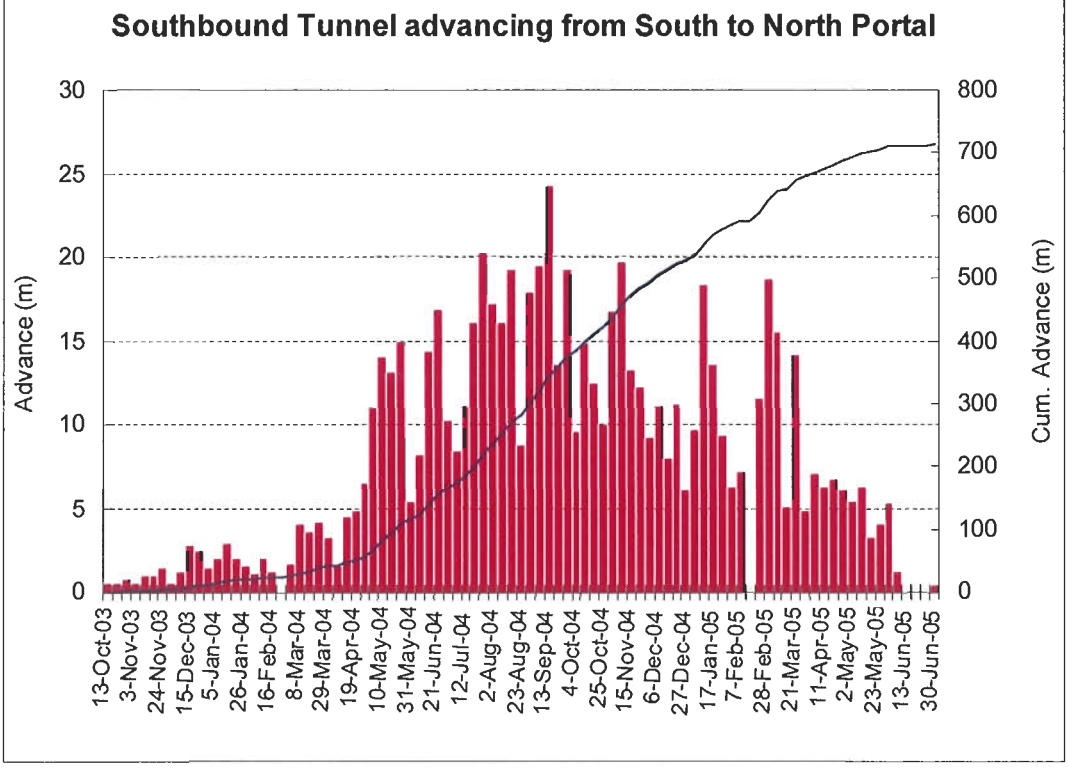


Figure 6. Full Face (or Equivalent Full advance where appropriate) Weekly Advance Rate Monitoring.

PRIMARY FACTORS AFFECTING EXCAVATION PROGRESS

Each of the following factors affecting progress were reviewed to determine opportunities for improving excavation progress :

(a) working cycle times;
(b) allowable working hours using machinery;
(c) efficiency of blasting; and
(d) vibration limits on sensitive receivers.

WORKING CYCLE TIMES

Working cycle times may be affected by down time of plant and equipment, supply of explosives and detonators, delays due to misfires, weather and weak zones within the tunnel.

Down Time of Plant and Equipment

To minimise the down time of the plant and equipment, the Contractor provided sufficient spare plant, spare parts for the plant and an effective service team to maintain the plant and equipment.

In conjunction with the monitoring of cycle times and constant updates of the predictions for advance as a function of charge weight as shown above, machine in-service and maintenance schedules were prepared, and additional plant brought into service for large blasts in particular, where increased mucking out and shotcreting times added significantly to the cycle.

During tunnelling, with the exception of a single event of a shotcreting machine breakdown causing disruption to one blast cycle, no stoppages were caused by down time of plant and equipment.

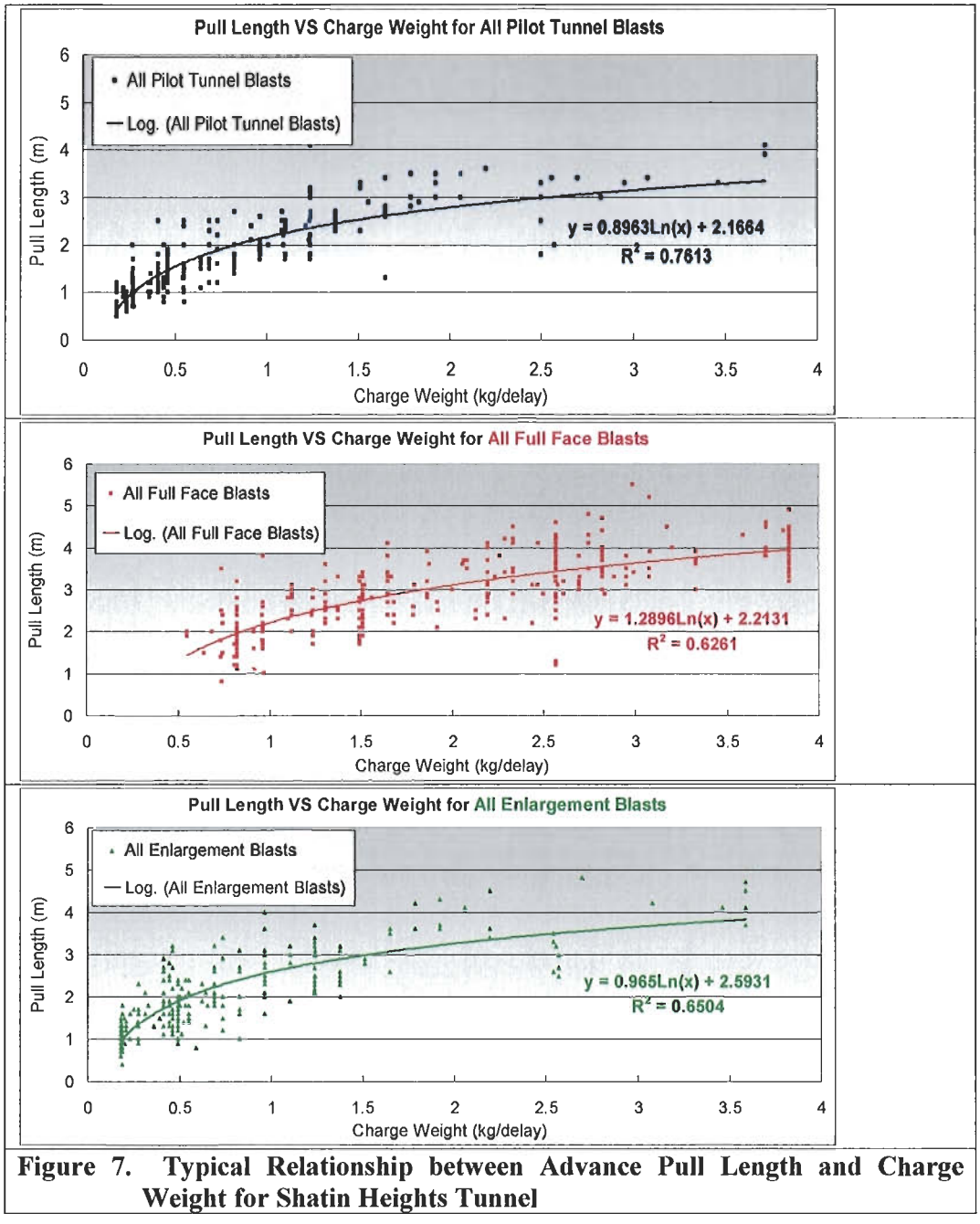


Figure 7. Typical Relationship between Advance Pull Length and Charge Weight for Shatin Heights Tunnel

Supply of Explosives and Detonators

At the peak period of the tunnelling works, the capacity of the site magazine was insufficient to conduct 4 blasts per day. As a result a daily explosive delivery was arranged with the CEDD Mines Division. The reassessment of 2 slopes (as discussed below) also increased the allowable charge weight along chainages affected by those features. This had the advantage of not only permitting more rapid advance due to the larger charge weights per delay, but also had the added advantage of opening up a longer tunnel section where bulk

emulsion explosives could be used. Supply of bulk emulsion explosives are not affected by the capacity of the site magazine.

Bulk emulsion explosives were used where charge weights more than 2.5 kg/delay could be used. An emulsion depot was built on site by the explosive supplier. Bulk emulsion has the advantage that while it is a lower class Dangerous Goods (DG7, flammable) material it is not classed as an explosive until gassed at the nozzle, and larger quantities could be stored on site than in the magazine.

The management of the explosives supply was crucial. The blasting engineer performed a daily review of the blast designs and the explosive balance in the site magazine, to prevent both lack of explosives or overloading, the latter which was not permitted.

Delays due to misfires

Misfires may be caused by :

(a) poor connection or crimping of plain detonators;
(b) damage of detonating cords or detonators;
(c) poor wiring; or
(d) cutting off of detonating cords due to rock fragments from the surface detonators immediately after initiation.

If a misfire occurs, the blasting site shall be closed for at least 30 minutes (*Dangerous Goods (General) Regulations, Chapter 295B, Regulation 57*). The shot firer will then carry out a detailed inspection and checking procedure to identify the area where the misfire occurred and determine the most appropriate method of reconnection prior to re-firing. The whole procedure to review a single misfire can take up to 2 hours subject to the difficulties of reconnection.

During the project, one case of misfire was reported due to damaged detonating cords. The blasting cycle was delayed by one day due to this incident.

To reduce the chance of a misfire, the Contractor used the "Surefire" system for initiation, which is generally accepted as a more reliable and safer initiating system than traditional electric detonating systems or those which use plain detonators (initiated by safety fuse). The "Surefire" system sends a pyrotechnically initiated shock wave down a NONEL shock tube, which in turn sets off the NONEL initiating detonator. All components downstream of the Surefire device are thus non-electric, and unaffected by stray currents. Traditional electric detonating systems send a current down electrical wires to an electric initiating detonator, which can then subsequently be used to set off NONEL shock tubes leading to NONEL detonators. All components downstream of the electric detonator up to and including the initiating detonator are thus electric and may be initiated by stray currents. Electric initiating systems must also undergo rigorous circuit checks, which are time-consuming and can thus affect blasting cycle times.

Stoppages caused by the problem of explosives supply during tunnel excavation were thus able to be kept to a minimum.

Weak Zones within Tunnel Excavation

Apart from soft ground at the south portal (Southbound Tunnel), pre-tender site investigation including two horizontal drill holes providing ground investigation data for the full tunnel length showed several faults or weak zones could be expected, one of which was

actually encountered during excavation. The slow advance rates at the south portal in soft ground can be seen in Figure 6 above, resulting in an average advance rate of just 2 m/week, due to requirements for application of both ribs and shotcrete. There is a significant difference between the advance rates in soft ground and hard rock (approximately 10.5 m/week).

The weak faulted zones away from the Portal were probed extensively prior to advancement for strength and particularly groundwater inflow determination. Mechanical breaking was used to remove spoil. Although the material was of sufficient strength that no ribs were required, with ground support being supplied by shotcrete alone, but it has considerably slowed down the progress.

Most of the tunnel length was dry, even at the weak zones, providing ideal tunnelling conditions, with maximum recorded water inflows of less than 3.2 l/min.

Plate 1 : Surefire (Left); Electric Blasting Machine (Right)

Figure 8 shows mapped Q value versus tunnel chainage, which gives an indication of the rock mass quality encountered. With one exception, a weak zone, the primary factor which determined tunnel advance was allowable charge weight per delay, not rock quality.

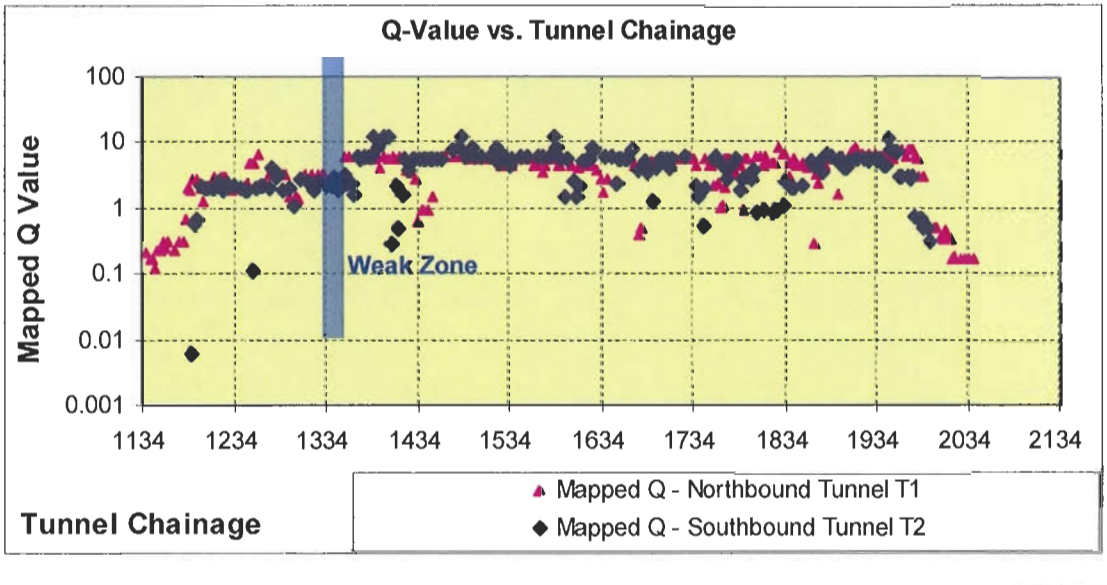


Figure 8. Mapped Q Value vs. Tunnel Chainage

NOISE LIMITS - PUBLIC COMPLAINTS

Blasting operations are always sensitive works with regards to the public and residents who live in the vicinity of a blast site. The concerns are not only flyrock, excessive vibration, noise and airblast, but also the blast fumes that may contribute to discomfort amongst residents or road users.

With the support from the CEDD, the Resident Site Staff (RSS) and the Contractor's blasting team, the Contractor created good communication channels with the public, with site personnel providing details of what could be expected during blasting. The blasting was carried out with the general support of residents in the surrounding area. This resulted in the actual numbers of complaints that were received being relatively few. Three cases of minor cracking were reported in residences. However, an examination of the available blast timing versus dates upon which cracking observations were made, vibration levels, distance, type and direction of cracks observed and review of settlement results all indicated that the blasting was extremely unlikely to have been the cause of the incidents. No structural damage was recorded.

The tunnelling works were never stopped due to public complaints or complaints by EPD, WSD or CEDD Mines Division.

EFFICIENCY OF BLASTING METHODS AND TECHNOLOGY

Several proposals were put forward to improve advance rates as follows :

A joint proposal was put forward to the CEDD Mines Division by the Contractor, Client and Consultant to use bulk emulsion explosives with charge weights less than 2.5 kg, and weight-trials to test the accuracy of the computer controlled delivery pump performed at Shek O Quarry. The reduced limit would not only have helped with explosives delivery and storage, but also theoretical improved coupling of explosive to charge hole walls could have helped in reducing vibration levels. The theoretical reduction in vibration levels was reviewed by plotting vibrations due to both explosive types at weights just below and just above 2.5 kg/delay, at similar scaled distances. However, this was not considered a sufficiently reliable review as no direct comparison with cartridge (packaged) explosive weights were possible and many other factors such as sensitive receiver locations were considered to have biased the data. The vibration review, however, showed little difference in the resultant vibration levels. Unfortunately, due to CEDD Mines Division concern over the suitability of the SHT site for trials of this type, and the sensitivity of adjacent receivers at this site, the proposal for use of a lowered bulk emulsion charge weight below 2.5 kg/delay was not accepted by the CEDD Mines Division.

An informal proposal was also put forward to the CEDD Mines Division by the Contractor for the possible use of electronic detonators in place of NONEL detonators. Electronic detonators provide superior accuracy in delay timing, which reduces overlapping blast shock wave pulses, theoretically reducing vibration levels (Hoshino et al, 2000; Zhang et al, 2004). Added advantages include improved fragmentation and preservation of the integrity of the insitu rock mass (Grobler, 2003). A further advantage of electronic detonators is that the higher number of delays available than conventional non-electric (NONEL) delays would have permitted a full face to be blasted in one shot, for very low allowable charge weights. Low allowable charge weights (e.g. 0.066 kg/delay as used at the Northbound Tunnel) results in a blast design with close blasthole spacing (low burden), which, with the limited NONEL delays available restricts the blast to a small portion of the face. The 0.066 kg/delay blasts e.g. required three separate blasts to advance the entire face a distance of just 500 mm. Due to the length of time required for testing and approval of new blasting equipment coupled with the cost of electronic detonators, the Contractor's proposal to use electronic detonators was not pursued further.

An internal review of blast vibration attenuation laws was conducted to determine if some of the more sensitive close-in blasts, which generally result in low allowable charge weights per delay, more closely followed cube-root scaling (Lucca, 2003; Yang et al, 2000) than square-root scaling (Dowding, 1996). A more appropriate scaling law, if justified, may have permitted higher allowable charge weights per delay, subject to CEDD Mines Division approval. However the results of the internal review indicated that while cube root scaling may have been appropriate at the portal areas, there was insufficient evidence to justify its use throughout the tunnel, due to the dynamic nature of the location of the advancing face with respect to the location of the sensitive receivers.

VIBRATION LIMITS ON SENSITIVE RECEIVERS

Reassessment of Slopes

A significant constraint on blast progress was due to the nearby slopes. There are many of them and they presented the greatest challenge due to the initially low assessed critical peak particle velocity (PPVc) limits. Current guidelines for determining peak particle velocities published by the GEO are presented in GEO Report 15, Wong & Pang (1991) and the GEO Report 15 approach was adopted for initial review of the limiting PPVc on these slopes.

GEO Report 15 adopts relatively conservative assumptions (as acknowledged in the report) with dynamic response analysis of slopes conducted at limiting equilibrium, assuming a pseudo-static response mechanism. A transfer function is used to determine resultant soil surface motions as a function of bedrock input motion, with values of the transfer function increasing towards what is termed (Kramer, 1996) as the fundamental frequency of the soil deposit, but also falling off as frequency increases further. The fundamental (first natural) frequency of the soil deposit is determined from the geometry and properties of the material, which are also subject to damping. The second, third and fourth natural frequencies are also included in GEO Report 15.

Conservative lower bound input frequency levels of 30 Hz are commonly adopted, and were used in this case, although these can be adjusted and were also reviewed, and the report methodology also assumes infinite durations of input motions. The duration of input motions from blasting are considered as commonly being of considerably shorter durations than earthquake input motions.

The limiting PPVc in GEO Report 15 applies to bedrock input motion PPV below the slope. However, on many soil slopes, particularly fill slopes, bedrock motion can be difficult to measure directly unless boreholes are drilled through the feature down to bedrock and in-borehole transducers are employed, or outcrops (not boulders) are identified on sites for instrumentation. Bedrock input motions below softer or weaker deposits commonly result in amplified displacements at ground surface (Dowding, 1996), although as Kramer (1996) observes, higher frequency (short wavelength) motions may cause portions of a slope above the failure surface to be moving in opposite directions.

As time constraints did not permit a more rigorous analysis of the failure mechanisms involved, the underlying assumptions for the original subsurface profile were re-visited, based on borehole information and a comprehensive site inspection. The pre and post-contract assumed subsurface profiles for one particular slope are presented in Figures 8a and 8b below.

Wong & Pang (1991) observed that the calculation for the critical PPVc had several levels of conservatism in the assumptions. However, they also noted the dynamic slope factor of safety (Fd) should not be allowed to drop below unity, in particular for saprolitic soils which exhibit a large post-peak drop in shear strength, to ensure any slope would not be stressed beyond the failure point. Given the slope was located immediately above a busy main road (Tai Po Rd) a dynamic factor of safety Fd of unity (1.0) was thus chosen as the basis for estimation of the acceleration factor, α.

As a result of the ground profile review, the acceleration factor (the factor which would theoretically cause the slope to fail in a pseudo-static slope stability analysis model) was increased from 0.075 to 0.095. This small change, in conjunction with the appropriate subsurface model, yielded a critical PPV of 13mm/s (which is higher that the PPV of 8mm/s initially adopted in the design). This resulted in an over 2-fold increase in allowable charge weight, pushing it up above 2.5 kg/delay, where bulk emulsion could be used.

In re-visiting the slope profile, the most important factor in increasing the acceleration factor, α, was the presence of the HDG layer in the subsurface profile, immediately above rockhead.

For the initial PPVc of 8 mm/s, the limiting charge weight (~Ch.1550) was1.38kg/delay (max.). After additional ground investigation (GI) with a revision of ground profile and GWL information :

Final PPVc = 13 mm/s
Charge Weight = 3.10 kg/delay (max.)

Charge Weight Ratio Increase = 1 : 2.2

Assumed frequency in analysis= 30 Hz
Actual measured frequency:

- Surface vibrograph 30-85 Hz
- In-hole vibrograph 85-170 Hz (In-hole to Surface).

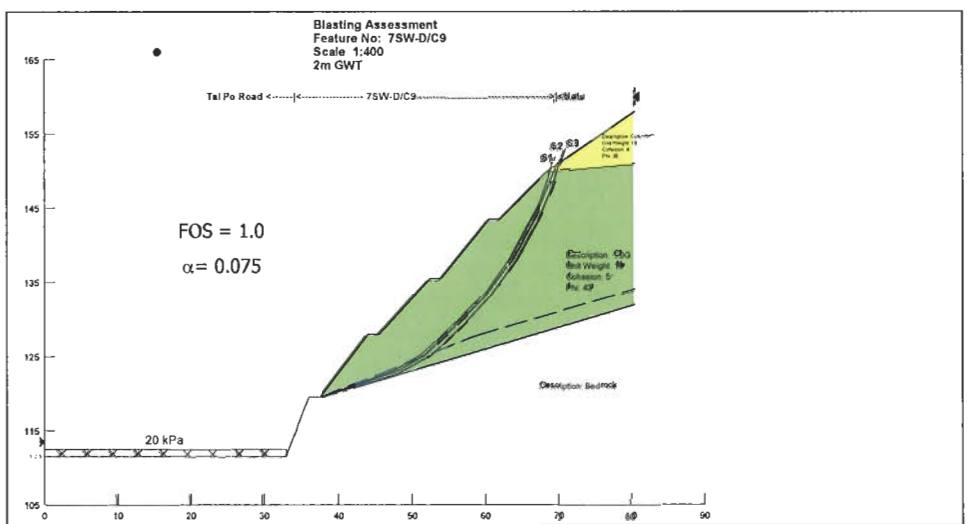


Figure 9a. Original Assumed Subsurface and Groundwater Profile

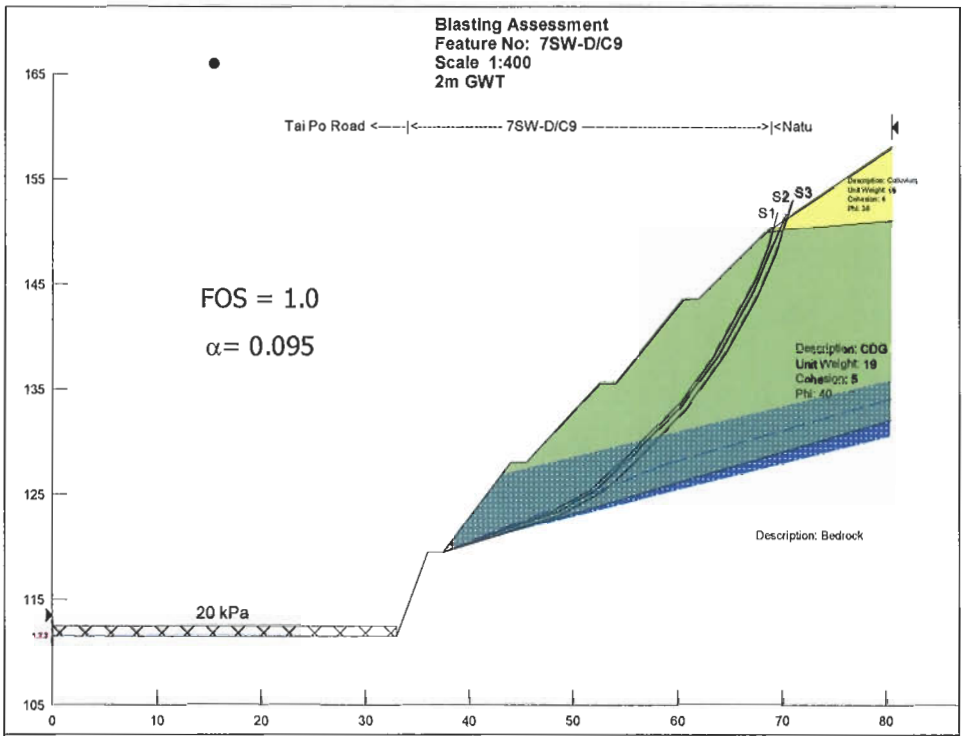


Figure 9b. Revised Assumed Subsurface and Groundwater Profile (HDG shown shaded)

CONCLUSIONS

The continuous monitoring of all the elements of the excavation cycle, reviews of areas where additional resources were needed, reviews of site constraints and joint efforts amongst the Client, the Engineer and the Contractor to not just accept the constraints, but pro-actively seek solutions to overcome them, all resulted in timely completion of the tunnel excavation of Sha Tin Heights Tunnel.

ACKNOWLEDGEMENTS

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COMMENTS ON THE USE OF SOIL MODELS WHEN ASSESSING THE EFFECTS OF TUNNELLING ON ADJACENT STRUCTURES

S. W. Lee[1], A. R. Pickles[1] and T. O. Henderson[1]

Abstract

The prediction of ground movements and the corresponding assessment of building damage are often significant parts of design and construction of urban tunnelling projects. Critical decisions relating to cost and programme are often made on the basis of numerical modelling of building response to ground movements induced by tunnel excavation. Practicing engineers are often unaware that the results from numerical modelling are sensitive to the choice of constitutive soil model chosen for the analyses and that the models need to be calibrated for site specific conditions before the analyses are undertaken. This paper takes two soil models, a standard Mohr Coulomb linear elastic perfectly plastic model and a more advanced non-linear model, and presents, for each model, simple calibration exercises using data from a tunneling case history that are recommended as a validation procedure before use in numerical modelling. The two soil models are then used to produce predictions of ground movement that can be compared with measurements taken from an actual tunnel excavation and commonly adopted empirical predictions. The comparison is then extended to simple soil-structure analyses commonly undertaken during design and construction of urban soft ground tunnel projects. The results show that the greenfield horizontal and vertical ground surface movement predictions from the advanced soil model are a better fit with actual measurements from a tunnel project than the results from analyses carried out using standard Mohr Coulomb model. The advanced soil model also provides a better fit to empirical predictions. There is subsequently more confidence in the assessment of ground movements, building response and potential damage from analyses using the advanced model than similar analyses carried out using the Mohr Coulomb model.

INTRODUCTION

The estimate of ground movements and the assessment of associated building response is a major element of soft ground tunnel design and construction in an urban environment. For most tunnelling projects the assessment of ground movements and building damage commences with Gaussian type calculations assuming greenfield conditions. The typical product of this exercise is surface settlement contours which are then used to initiate a potential damage classification system for adjacent buildings and structures using empirical criteria (Rankin, 1988; Boscardin & Cording, 1989).

Initial structural damage classification is usually carried out using simple assumptions

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relating to building geometry and stiffness based on the Gaussian settlement predictions and elementary principles of beam flexure (Mair et al, 1996). The requirement for advanced numerical modelling is normally only triggered if this initial assessment produces exceptional levels of anticipated damage, normally measured in terms of maximum tensile strain, or if the building under assessment is considered to be particularly sensitive. Numerical modelling is therefore introduced in the assessment process principally to understand in more detail the interaction between the ground deformations caused by the tunnel excavation and the structural response of a selected number of buildings.

The results of the numerical modelling exercise can therefore have a significant influence on the design and construction process for urban tunneling projects and in particular on decisions relating to costly and time consuming advance protective works or even route alignment selections. It is therefore important that practicing engineers are aware that the output from numerical modelling is sensitive to the choice of constitutive soil model and its suitability for the specific problem requiring analyses.

The most commonly used soil model is termed "Mohr Coulomb" which has a linear elastic perfectly plastic stiffness response. A more advanced soil model has non-linear and volume change characteristics. Most commercially available finite element packages offer the choice between the two and occasionally additional variations.

Using published case history data from an actual tunnelling project, it is first necessary to carry out a simple validation procedure for both standard and advanced models before embarking upon detailed analytical studies. Having validated the models, a comparison can then be made between the predictions of ground movements from the analyses with measurements taken during an actual tunnel excavation and the results from an empirical approach using the Gaussian curve fit approximation. This provides a greater level of confidence when one is required to examine the results of simple soil-structure interaction analyses commonly carried out as part of building damage assessment in urban tunnelling projects.

When considering the adoption of a soil model in a finite element analysis for the purposes of assessing building damage the following four steps should be followed :

(a) Calibration of the more advanced soil model against stress strain data for the soil. For advanced models this calibration would typically be against data obtained from triaxial tests which follow a variety of stress paths. For a Mohr-Coulomb model this might be limited to adjustment of the stiffness values to obtain a ground volume loss value that is similar to that measured in the field.
(b) Modelling tunnel construction to determine the correlation between ground relaxation before installation of the lining in the model and the predicted ground loss.
(c) Confirmation that the adoption of the soil model in an analysis of tunnel construction results in a predicted ground response which is comparable with observed behaviour at greenfield sites and/or empirical predictions based on a large number of case histories.
(d) Incorporation of the particular structure for which an assessment of damage is required into the finite element model and an assessment of the effect of tunneling on the structure.

An example of the process described above is presented in the following sections. In this example steps (2) and (3) have been combined. This process has been carried out using a

simple Mohr Coulomb soil model and a more advanced strain hardening soil model.

PROJECT CASE HISTORY REFERENCE DATA

The tunnel project that provides reference data for the modelling exercise is part of the Paulista line of the underground transport system in Sao Paulo, Brazil, constructed using the New Austrian Tunnelling Method (NATM). It has been selected because there is published data on the ground conditions, field and laboratory tests, excavation and construction sequence and measurements of ground response. Azevedo et al (2002) presented the background information for the project and the ground conditions at an instrumented section. Figure 1 shows the generalized subsurface conditions, tunnel construction sequence and instrumentation at the selected section for the FE analysis. The oval-shape NATM tunnel has a maximum height and width of 8.4 m and 11.4 m respectively, with a cross-sectional area of 82 m^2. At the section analysed, the soil cover is 7.6 m and the ground water table is located at about 12 m below the ground level close to the centerline of the tunnel. Excavation of the tunnel was carried out in three phases. The lining consists of a 0.2 m thick primary shotcrete and a 0.15 m thick secondary shotcrete liner.

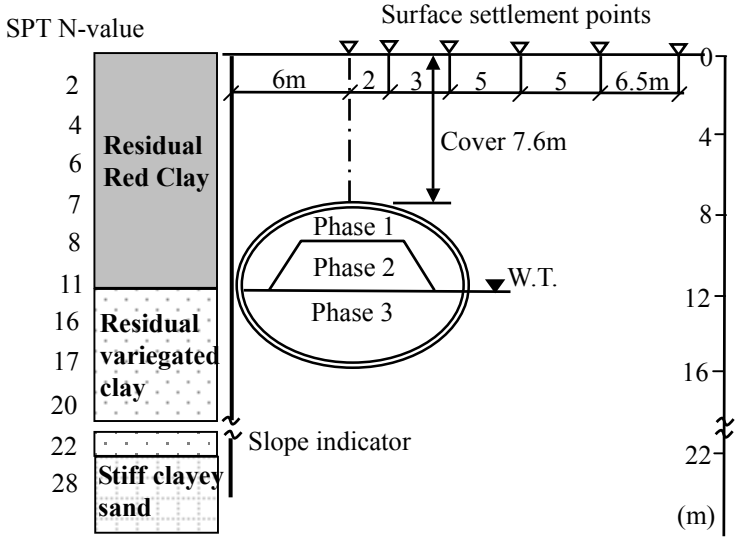


Figure 1. Subsurface Conditions, Tunnel Excavation Sequence and Instrumentation (after Azevedo et al, 2002).

The tunnel was excavated through residual soils of the Sao Paulo Basin (Azevedo et al, 2002; Zornberg et al, 1998). A typical geotechnical profile consists of an upper layer of residual Red Clay underlain by a layer of residual variegated clay. The Red Clay is porous and slightly over-consolidated, with a void ratio typically greater than 1.0 and an over-consolidation ratio (OCR) of 2 to 3. The Standard Penetration Test (SPT) N-value ranges from 4 to 11. The variegated clay is stiff, fissured, over-consolidated (OCR of 5 to 6), and has a SPT N-value ranging from 16 to 22. Figure 2 shows the effective strength parameters obtained from studies on the properties of these two lateritic clays (Kupper et al, 1985; Massad et al, 1985).

Azevedo et al (2002) carried out drained triaxial testing on unsaturated Red Clay samples at their in-situ moisture content, whereas for variegated clay the samples were saturated by back-pressure. The samples were initially isotropically consolidated, followed

by loading and unloading drained shearing. The tests included hydrostatic compression (HC), conventional triaxial compression (CTC), reduced triaxial compression (RTC), and reduced triaxial extension (RTE) tests. Figure 3 shows the different stress paths followed in the tests. These different tests are used to study the influence of stress path on the soil behaviour around the tunnel excavation, which generally follows vertical and horizontal unloading stress paths.

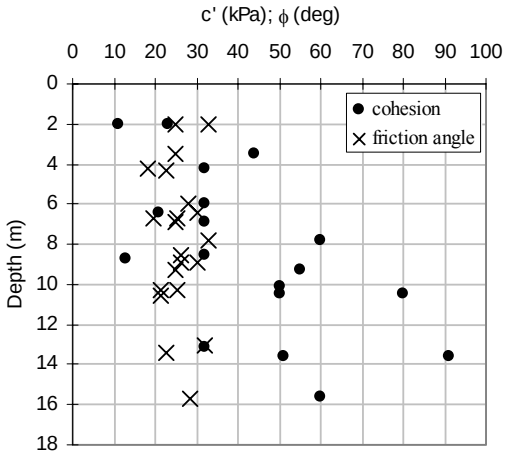


Figure 2. c' and ϕ for Red Clay in In Sao Paulo (after Kupper et al, 1985)

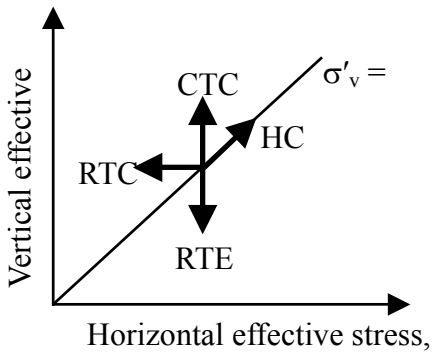


Figure 3. Different Stress Paths in Drained Triaxial Tests

CALIBRATION OF SOIL MODELS BEFORE DETAILED ANALYSIS

Calibration of Advanced Soil Model

A commercial FE software programme, Plaxis, has been used to model the NATM tunnel construction for the Paulista Line. The advanced soil model chosen for the purposes of the exercises described in this paper is the Hardening Soil (HS) constitutive model which is available in the Plaxis programme. Figure 4 shows the non-linear hyperbolic stress-strain relationship in the HS model and its total yield contour in principal stress space. The HS model is a development of the hyperbolic model of Duncan and Chang (1970) and incorporates plasticity, soil dilatancy and a yield cap. Further information about the suitability of the HS model for modelling both soft and stiff soils and the definition of its input parameters can be found in Plaxis (2004).

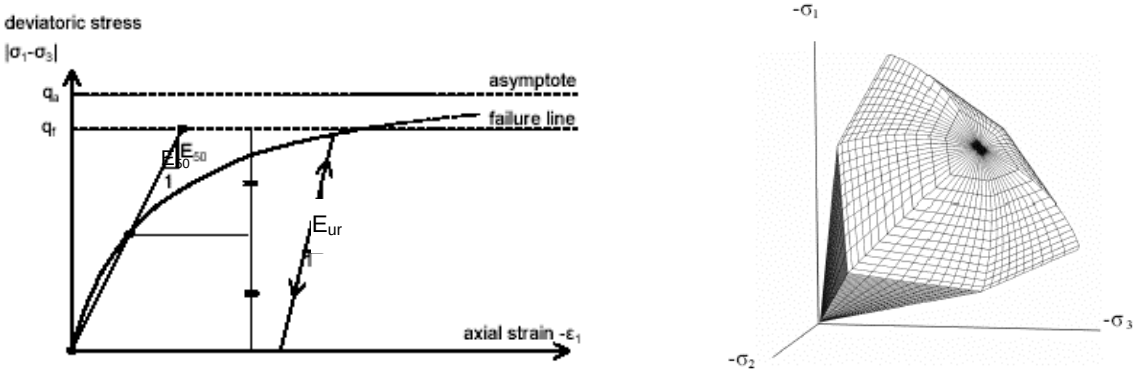


Figure 4. Hyperbolic Stress-Strain Relationship and Total Yield Contour of HS Model (Plaxis, 2004).

The results of the triaxial tests presented by Azevedo et al (2002) have been used to calibrate the HS model by carrying out single element triaxial modelling of HC, CTC, RTC and RTE tests. Based on comparisons with the laboratory triaxial results, it was found that the input parameters summarized in Table 1 can reasonably replicate the stress-strain-strength characteristics of Red Clay and variegated clay in different loading and unloading shearing modes. Figures 5 to 9 present the comparisons between the triaxial tests and the HS model predictions, in which the agreements between the two are reasonably good.

Table 1. Input parameters of the HS model for Red Clay and variegated clay

Soil	γ_{dry} (kN/m^3)	γ_{wet} (kN/m^3)	E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	E_{ur}^{ref} (kPa)	υ_{ur}	m	c' (kPa)	ϕ (deg)	p_{ref} (kPa)
Red Clay (above 3.5m)	14	16	2,000	2,000	6,000	0.2	0.75	15	21	100
Red Clay (below 3.5m)	15	17	3,500	2,250	10,500	0.2	0.75	15	21	100
Variegated clay	15.5	17.5	30,000	14,000	90,000	0.2	1.0	15	21	100

Calibration of Mohr Coulomb Soil Model

The stress-strain behaviour of MC soil is bilinear, i.e. linear elastic perfectly plastic, without considering the degradation of stiffness with increasing strain and the difference in loading and unloading stiffness. The tunnel excavation analysis is carried out as a conventional total stress analysis, for which the soil strength is input as undrained shear strength (c_u). The calibration exercise for the Mohr Coulomb model required that the Young's modulus values of the clays be adjusted so as to predict the same volume loss for the tunnel excavation as measured in the field, i.e. a settlement trough volume of 1.53 m^3/m (or volume loss of 1.87%). The Young's modulus derived from this calibration exercise is about 4,500 kPa and 50,000 kPa for Red Clay and variegated clay respectively.

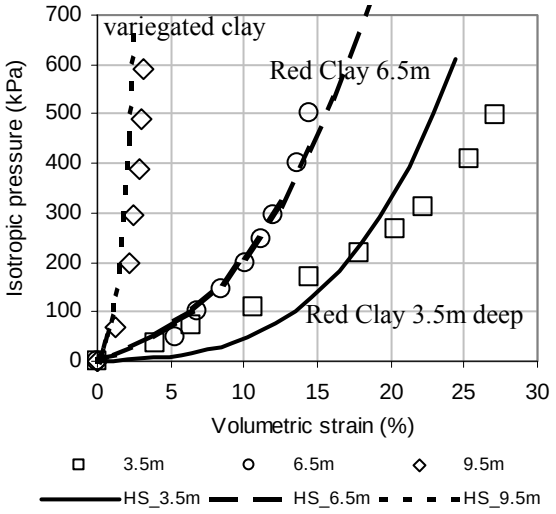


Figure 5. Comparison of HC Tests.

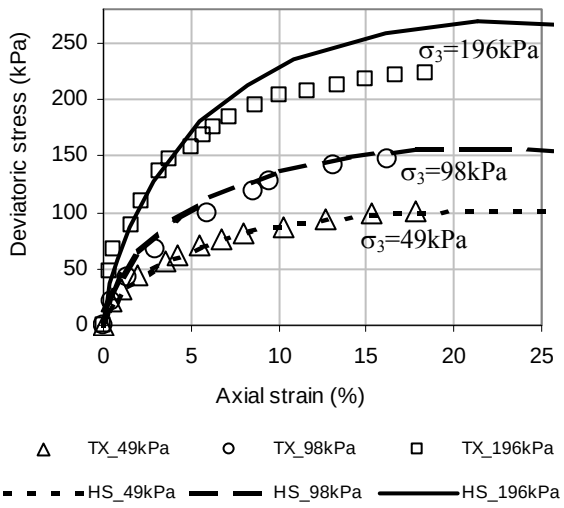


Figure 6. Comparison of CTC Tests for 6.5m Deep Red Clay.

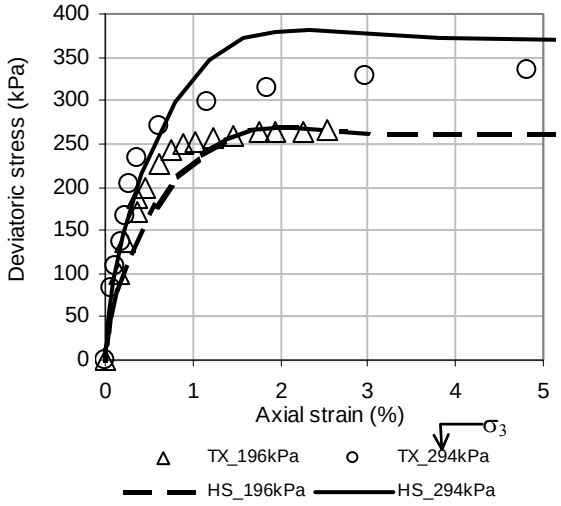
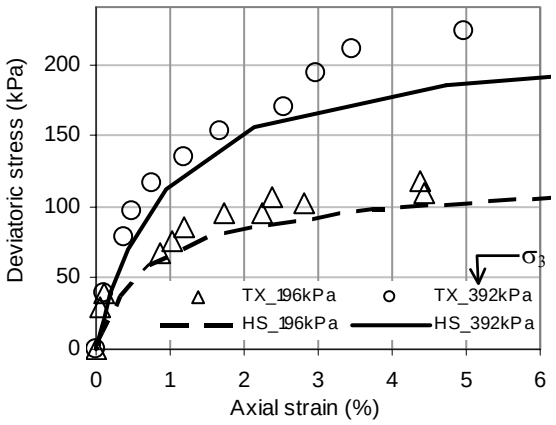


Figure 7. Comparison of CTC Tests for Variegated Clay.



Note: Deviatoric stress is defined as $q=\sigma_1-\sigma_3$ similar to that in Zornberg et al. (1998), rather than $q=(\sigma_1-\sigma_3)/2$ in Azevedo et al. (2002).

Figure 8. Comparison of RTC Tests for 3.5m Deep Red Clay.

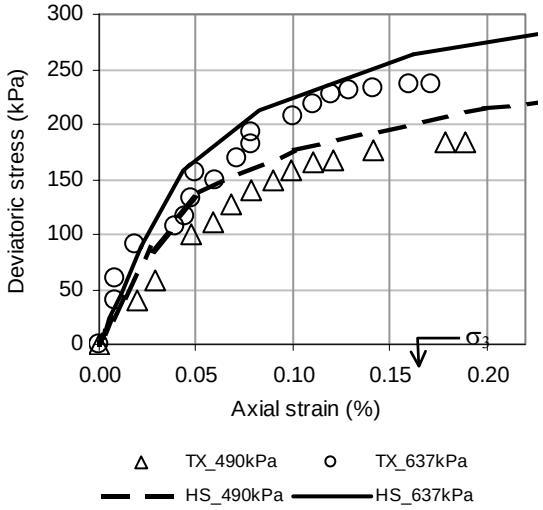


Figure 9. Comparison of RTE Tests for Variegated Clay

2D MODELLING OF NATM EXCAVATION

Background and Results of Analysis

Figure 10 presents the 2D FE mesh adopted for modelling the tunnel geometry and ground conditions at the instrumented section of the tunnel excavation. The total lining thickness is 0.35 m with a Young's modulus of 10 GPa (Azevedo et al, 2002). The water table is at 12 m below ground level. The analysis has been carried out with the generation of excess pore pressure for soil layer below the water table. The coefficient of earth pressure at rest (K_0) for Red Clay and variegated clay is specified as 1.0 and 1.2 respectively, which corresponds to an OCR of 3 and 6 respectively (Azevedo et al, 2002).

The sequential excavation of the NATM tunnel is a 3D construction activity. Plane

strain (2D) analysis of tunnel excavation would result in an over-prediction of surface settlements, as the 2D analysis has not considered the 3D stress redistribution in front of the tunnel face (Sharma et al, 2000). To approach a 3D analysis with an equivalent 2D analysis, Panet & Guenot (1982) proposed a percentage-unloading method or λ-value. In the λ-value method, a fraction of initial stresses around the tunnel inner perimeter is removed before the installation of tunnel lining (λ<1). For tunnelling in stiff London Clay, Stallebrass et al (1994) determined a λ-value of 0.34, which gave a volume loss of 2%. For the Paulista tunnel, Azevedo et al (2002) observed a λ-value of about 0.4, based on the measured variation of surface settlement at the instrumented section with respect to distance to excavation heading. The Plaxis programme includes a percentage-unloading function for modelling of tunnel excavation, which is referred to as the β-method.

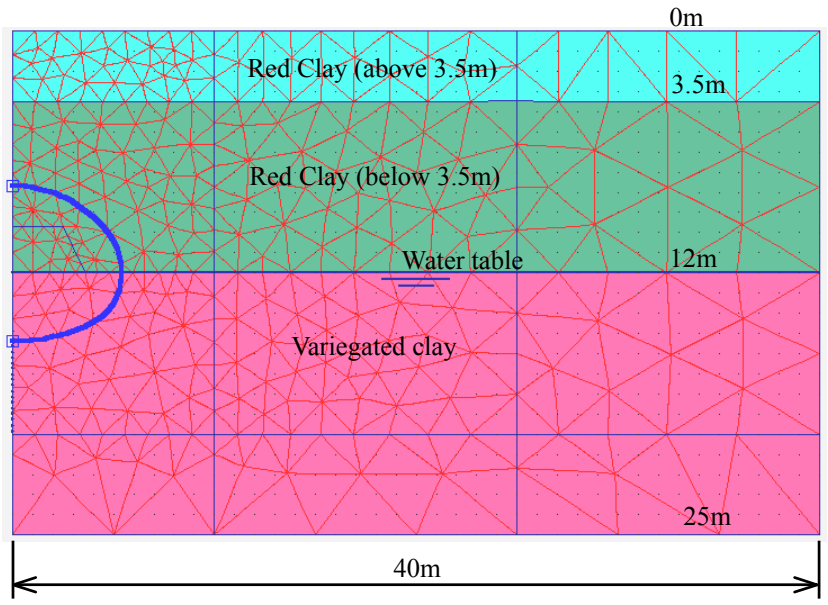


Figure 10. FE Mesh for the Paulista NATM Tunnel.

The first stage of the FE analysis is to achieve an initial stress equilibrium. The second stage models the tunnel excavation, which corresponds to a λ-value of 0.4. The third stage models the installation of the lining and the removal of remaining stresses around the tunnel inner perimeter supporting the soil. The FE analysis has been carried out using both the calibrated HS and Mohr Coulomb (MC) constitutive soil models for Red Clay and variegated clay. The λ-value assumed for both models was 0.4. The results of the analyses are presented in Figure 11.

Comparison of Results for Greenfield Site

The greenfield surface settlement profile caused by tunnelling can be approximated by a Gaussian distribution (Peck 1969), which is expressed by

$$S(x) = S_{max} \times \exp(-x^2 / 2i^2) \qquad \text{Eq (1)}$$

where S(x) is settlement at x-distance from the tunnel centreline; S_{max} is the maximum settlement above the tunnel centreline; and i is a parameter expressed by i=kH, where k value for clays ranges 0.4 to 0.6 (Mair and Taylor 1997) and H is the distance from the tunnel axis to ground level which is 12 m for the Paulista tunnel. The value of S_{max} for the Gaussian distribution is given by $S_{max}=V_S/2.5i$

The measured surface settlement profiles at the Paulista site presented in Figure 11 indicated that the volume of the surface settlement trough (V_S) is 1.53 m^3/m, which corresponds to a volume loss (V_L) of 1.87%, based on a tunnel volume (V_T) of 82 m^3/m. From the expression $S_{max}=V_S/2.5i$, the S_{max} is calculated as 102 mm. The surface settlement profile calculated by the Gaussian distribution is superimposed on Figure 11.

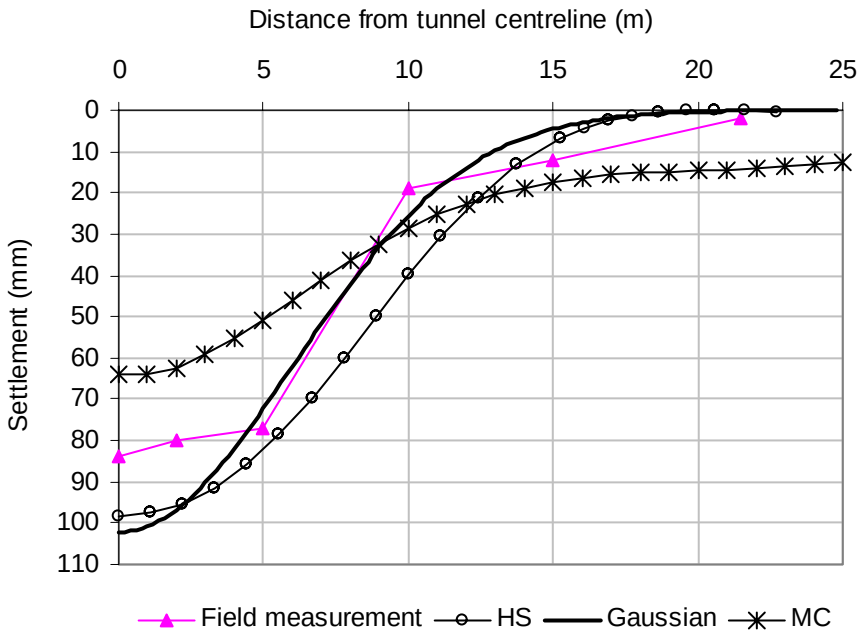


Figure 11. Comparison of Surface Settlements

It can be seen that there is a good agreement in the settlement profile between the measurements, predictions by the HS model and Gaussian curve. The MC model however predicts a much shallower and slightly wider surface settlement profile, which is different from the measurements, the results based on the Gaussian curve and the analysis by the HS model.

Comparison of Results for Surface Horizontal Movement

Tunnelling also induces horizontal movements at the ground surface which can cause damage to structures and utilities. The distribution of surface horizontal ground movement can be approximated as (Mair & Taylor, 1997)

$$\frac{S_h}{S_{h\max}} = 1.65\left(\frac{x}{i}\right)\exp\left(\frac{-x^2}{2i^2}\right) \qquad \text{Eq (2)}$$

where S_h is the surface horizontal movement and S_{hmax} the maximum surface horizontal movement. Equation (2) has been validated by data from field measurements (Mair & Taylor, 1997). Using an i-value of 6 m (i=0.5×12 m), the distribution of normalised surface horizontal movement in Equation (2) is shown in Figure 12. The predictions made by the HS and MC models are also superimposed on the figure. While the HS prediction shows a reasonable agreement with the Mair & Taylor's equation, the MC model over-predicts the lateral extent of surface horizontal movement. Unfortunately there are no reliable measurements of horizontal movement to make comparisons with field response.

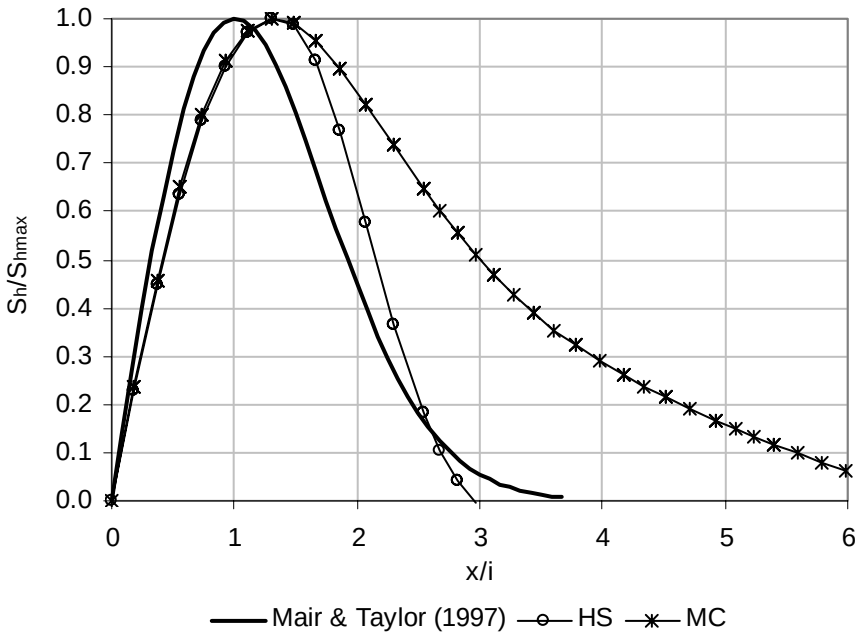


Figure 12. Comparison of Surface Horizontal Movements.

PREDICTING DAMAGE TO STRUCTURE USING HS AND MC MODEL

It can be seen from the above that the predictions of both the surface settlement profile and the horizontal movement at the ground surface using the HS model agree well with both field data and with the empirical relationships which have been developed on the basis of observed data, whilst the predictions using the commonly adopted Mohr Coulomb method are less consistent. These comparisons provide confidence that the HS model can be used to make assessments of the effect of tunnelling on adjacent structures which are more reliable than similar predictions using the simpler Mohr Coulomb model. An example of the difference in the predictions of structure response using the HS model and the MC model is presented for comparison purposes.

Two additional analyses have been carried out for the Paulista tunnel that have incorporated a building structure at the ground surface (see Figure 13). In both of the analyses the building has been assumed to be located between 7 m and 25 m from the centreline of the tunnel, which straddles the zone of maximum hogging for a greenfield site, see Figure 11. The analyses have modelled a single storey building and have been carried out using both the HS and MC models and have assumed that the building consists of a base slab and roof slab. The bending and axial stiffness of the building have been based on the stiffnesses presented in Franzius et al. (2006) and are summarised in Table 2

The predicted surface settlement profiles and bending moment distribution for the buildings are presented in Figure 14. The maximum bending moment and horizontal strain predicted for the building structures are presented in Table 2.

For this particular example the HS model predicts a bending moment which is approximately 25% higher than the MC model and a maximum horizontal strain at ground

surface which is approximately 10 times bigger. These could be significant when making critical decisions on the anticipated response of building to tunnelling induced ground movements and the need for advanced protective measures.

Table 2. Input parameters and predicted results for building structure.

Soil model	Bending stiffness, EI ($kN/m^2/m$ width)	Axial stiffness, EA (kN/m width)	Max. bending moment (kNm/m width)	Max. horizontal strain (%)
HS	2E+7	7E+6	170	0.0025
MC	Same as above	same as above	126	0.0002

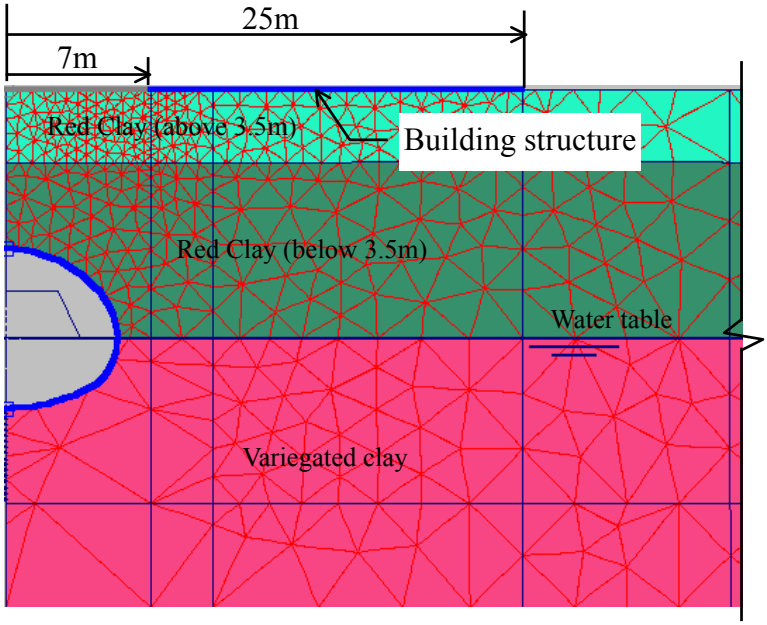


Figure 13. Mesh for Modelling a Building Structure on the Ground Surface

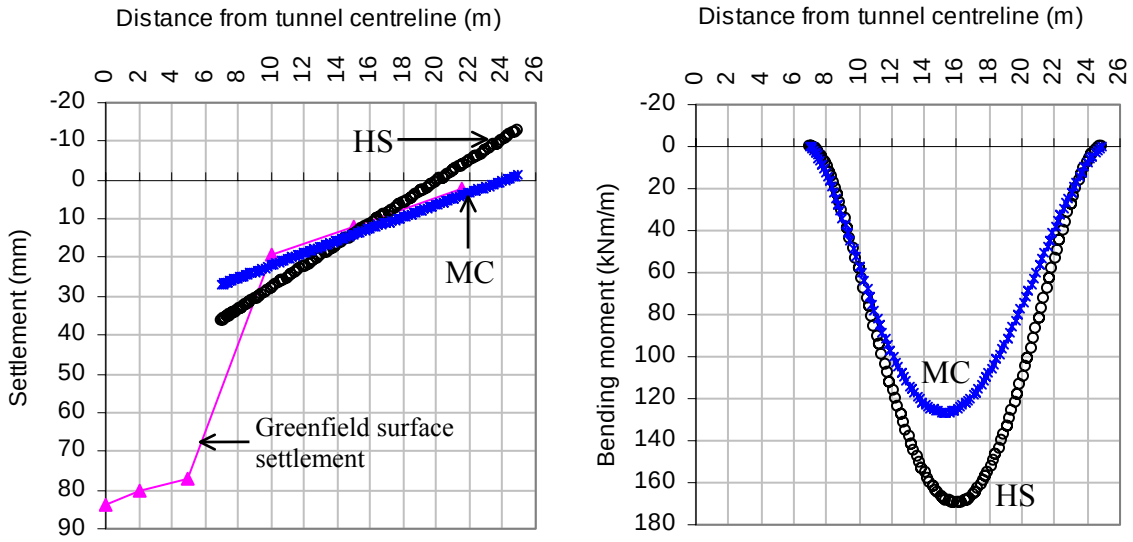


Figure 14. Comparison of Predicted Settlements and Bending Moment of the Building Structures using the HS and MC Soil Model

SUMMARY

An advanced soil model (Hardening Soil, HS) has been used to model the behaviour of residual clays for a NATM tunnel excavation in Sao Paulo. The input parameters of the HS model have been calibrated against the results of drained triaxial loading and unloading tests to replicate the stress-strain-strength relationship of residual Red Clay and variegated clay. The 3D tunnelling activity has been modelled by an equivalent 2D FE analysis by means of percentage-unloading method or λ-value. A similar exercise has been carried out using a Mohr Coulomb soil model where the calibration exercise involved adjusting the soil stiffness to match the volume loss measured during tunnel excavation. These calibration exercises, or something similar, are considered a necessary step in the numerical modelling of excavation induced ground movements and associated building response.

The advanced HS model predicts a surface settlement profile that is in good agreement with the measured settlement profile, though the maximum settlement above the tunnel centreline is over-predicted by about 15%. The surface settlement profile estimated using the commonly used Gaussian distribution curve also agrees well with the FE profile using the HS model. The FE model using the linear elastic perfectly plastic Mohr Coulomb (MC) constitutive model predicts a surface settlement profile which is shallower and wider than the measured profile and the maximum surface settlement is approximately 30% less than the measured value and 40% less than the values generated by the HS model and the Gaussian curve.

The distribution of surface horizontal movement obtained using the HS model shows a good agreement with the Mair & Taylor's (1997) empirical relationship. In comparison, the MC model over-predicts the lateral extent of surface horizontal movement. There are no reliable field data available for comparison with the predictions.

In simple assessments of building damage, the impact of the differences in predicted ground response using the HS and Mohr Coulomb soil models means that there are significant differences in the predicted level of horizontal strain in the building and bending moments. These differences may have a critical impact on decisions associated with costly and time consuming protective works.

For a greenfield site, the prediction of the surface settlement profile can be approximated by a Gaussian curve based on an assumed ground volume loss and there is no particular advantage using a more sophisticated soil model. However, in an urban environment other factors such as the presence of foundations need to be considered, the use of advanced soil models such as the HS model in Plaxis allows the tunnel designer to have more confidence in the prediction of the effect of tunnelling on adjacent structures. If more advanced soils models are to be adopted then they must be calibrated against laboratory test data for various stress paths and also against field results before they can be relied upon for more advanced predictions.

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TEMPORARY SUPPORT DESIGN FOR A SHALLOW DRIVEN TUNNEL ON HONG KONG ISLAND

Florent Desaintpaul[1] and Ian Askew[2]

Abstract

Queensway Subway will link the Three Pacific Place building to the existing subway leading to the MTRC Admiralty Station on Hong Kong Island. It is a 350 m long pedestrian subway under a major road serving as the main artery into the Central Business District. The alignment comes within 20 m of existing railway tunnels, and within 5m of the foundation of a highways bridge. The main 7.15 m span tunnel was partly constructed in soft/mixed ground. A widened section (17 m span) was excavated in rock with a ground cover of only 11 m. The subway was mined in free air below sea level with ground treatment by grouting for groundwater control. The temporary works design included detailed risk evaluations and measures to minimize surface settlement and maintain the integrity of existing buildings, structures and utilities. The variety of excavation and tunnel support techniques used are presented; this includes the use of pre-support pipe roofs, face support measures in soft/mixed ground and the works for breakthrough into the existing Admiralty Subway. Monitoring results of ground response during tunnel construction are also presented.

INTRODUCTION

Queensway Subway is a 350 m long pedestrian tunnel currently under construction on Hong Kong Island. It will provide entrances to Three Pacific Place, a new office building, and the existing Mass Transit Railway Corporation (MTRC) Admiralty Tunnel, which currently serves pedestrians between the Pacific Place shopping mall and the MTRC Admiralty Station (Figure 1). Gammon Construction was awarded the contracts for the construction of the Three Pacific Place building and the tunnel. Ove Arup & Partners Hong Kong (Arup) is the permanent works designer and Engineer for the project.

This paper presents an overview of the variety of tunnel excavation and initial support techniques used. The Contractor's in-house engineering design consultant, Lambeth Associates, carried out the temporary works design. It incorporated detailed risk evaluations and measures to minimise surface settlement and maintain the integrity of existing buildings, structures and utilities. Some monitoring results of ground response during tunnel construction are also presented.

SUBWAY LAYOUT

The site is located in the north of the Hong Kong Island along Queen's Road East and

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Queensway (Figure 1). Queensway is a major road serving as the main artery into the Central Business District. As shown on Figures 1 and 2, the subway comprises a main tunnel that connects, at its western end, to the existing Admiralty Tunnel and, at its eastern end, to the deepest basement of the new Three Pacific Place development. The main tunnel is widened locally into a rock cavern to allow for a direct escalator access to an intermediate basement level. There is one 'stub tunnel' leading off the main tunnel below Queensway to provide for access to an emergency escape route, via a shaft, and for a possible future subway link to the Pacific Place building.

The ground level along the proposed subway alignment is fairly uniform (+6 mPD). The tunnel was constructed below sea level with a variable cover/tunnel span ratio ranging from 1.9 to only 0.5 .

GROUND CONDITIONS

As shown on Figure 1, the interpreted geological profile along the subway alignment generally comprises a surface layer of fill, underlain by completely decomposed granite and bedrock, mainly described as strong to moderately strong, slightly to moderately decomposed granite. A rockhead depression was inferred near the middle section of the tunnel, following the alignment of a valley. The anticipated ground conditions at tunnel level included rock, mixed ground and soft ground. Although the alignment is close to the original shoreline, no superficial deposits (fill and colluvium) or marine deposits were found at the tunnel levels.

At least 2 or 3 joint sets with random joints were found in drillholes and during basement excavation works for the Three Pacific Place development. Rock Quality Designation (RQD) values were typically high with joints found to be mostly rough planar with surface staining and occasional thin coatings of manganese.

Based on groundwater monitoring data, the groundwater level assumed for temporary works design is typically 2.5 m below the ground surface.

TEMPORARY TUNNEL SUPPORT AND GROUND TREATMENT

Outline of Design Objectives and Construction Methods

The tunnel was excavated in a congested urban environment underneath a major operating road. The subway alignment comes within 20 m of the existing MTRC Island Line tunnels, and within 5 m of the foundation of a highways bridge, in soft ground. Key considerations to be addressed both by the design and construction site practice were, in addition to tunnel stability / deformations, the control of ground movements and groundwater drawdown to avoid disruption to activities on the surface or damage to existing buildings, structures and utilities.

This was achieved by designing temporary support for a wide range of ground conditions. The construction methods adopted were to ensure a timely installation of the support. Monitoring, precautionary and contingencies measures, site control measures and regular design reviews were specified in the design (see Section "Risk Management").

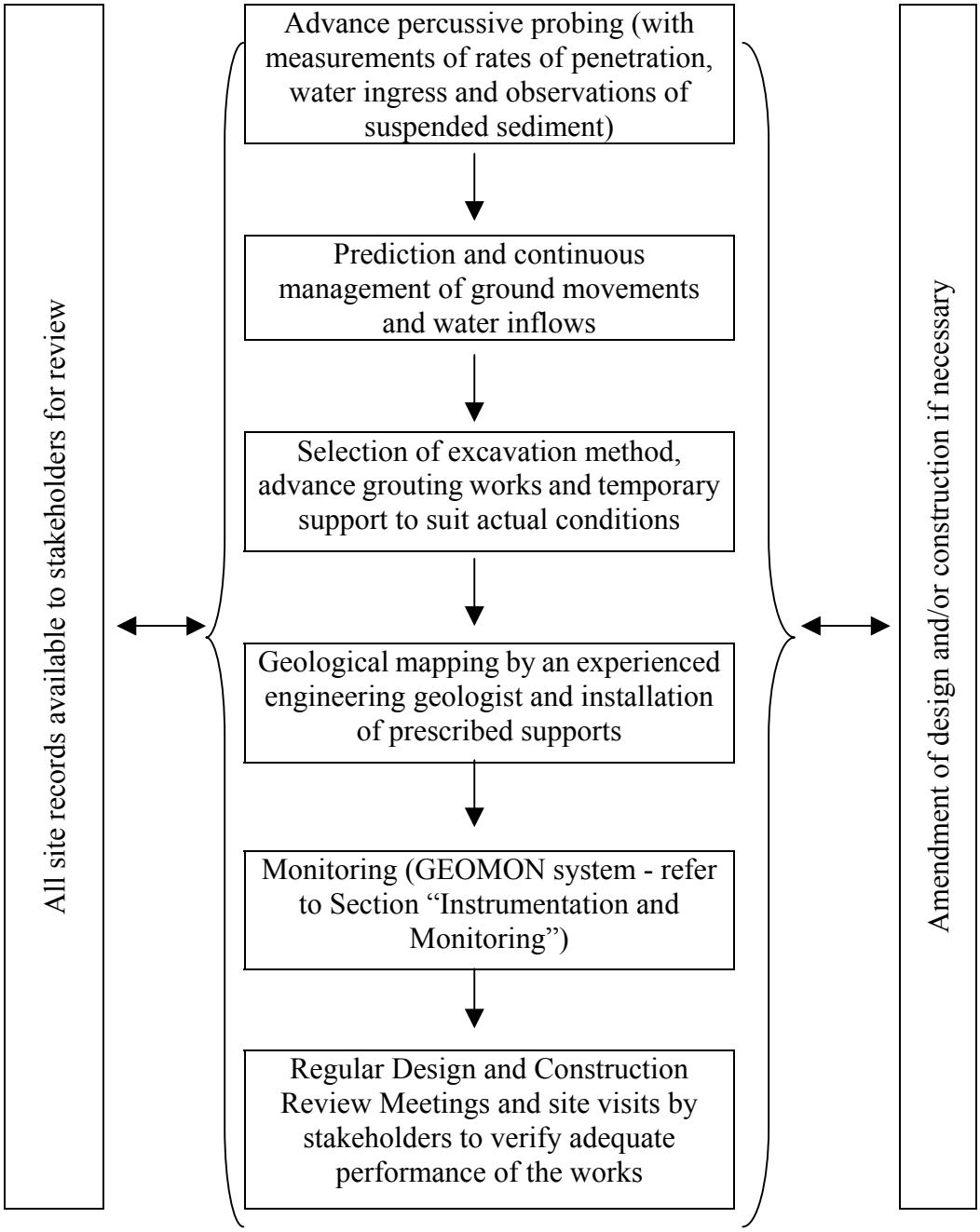
The subway was mined in free air. The main tunnel has a 7.15 m span and an excavation area of about 45 m^2. A 20 m long section was widened up to a 17 m span with 11 m ground cover and an excavation area of about 160 m^2 (staged excavation – see Figure 2). Excavation in

hard rock was by drill and blast methods and/or mechanical excavation techniques. In areas of soft ground, a mechanical excavation technique together with forward ground treatment and pre-support was used. Excavation of sections in mixed ground was carried using a combination of these techniques. The advance rates were adjusted to suit the site conditions.

Risk Management

A comprehensive review of potential design and construction issues was carried out at an early stage of the project, with participation from Arup, MTRC and the Independent Checking Engineer. In accordance with MTRC requirements, a Risk Register was prepared and updated during construction.

Key aspects for geotechnical risks are as follows :



Support Classes

(1) General

The anticipated ground conditions included rock and soft/mixed ground. Therefore, temporary support catered for a wide range of ground conditions, from rock tunnel to full-face soil tunnel. The temporary support was not incorporated into the permanent support, which consists of a concrete or shotcrete lining.

In accordance with the MTRC design standards, the excavation was treated as in rock if the effective rock cover was greater than half the span of the opening. This formed the "full rock cover" design case. For the purposes of temporary works design, a "limited rock cover" case was introduced. It catered for rock cover less than half the tunnel span.

The engineering properties of soils and rocks encountered along the alignment were determined based on the in-situ and laboratory test data from a site-specific ground investigation.

During construction, probing was carried out by percussive drilling to verify the ground conditions ahead of the face, with measurements of rates of penetration and water ingress. Geological logging of some of the drillholes for ground treatment / pre-support also provided additional information on ground and ground water conditions.

(2) Full or Limited Rock Cover

For the full rock cover case, support was to be provided in various configurations depending on the tunnel size, rock cover above the tunnel crown and rock mass quality (Figure 2). This comprises cement-grouted rock dowels or Swellex bolts and shotcrete. The support was based on a well-established empirical design method, the Q-system of the Norwegian Geotechnical Institute (Grimstad & Barton, 1993), with allowance for the temporary nature of the support in accordance with the MTRC Design Standard Manual.

A specific geotechnical assessment was carried out for the sequential excavation of the 17 m span section ("cavern" – Figure 2). Considering the generally competent nature of the rock mass at that location, geological structural features were the main control in terms of stability. The potentially unstable rock wedges that could form around the excavation were analysed using the computer programme Unwedge Version 2.3 (Rocscience 2002). This confirmed the adequacy of the empirical tunnel support using a combination of Swellex bolts and shotcrete.

In areas of the main tunnel with limited rock cover, pre-support by spiling (prescriptive design) or pipe roofing was used depending on the actual rock cover and rock mass quality. Steel supports were provided as necessary.

The effect of rock blasting on the stability of existing slopes was analysed in accordance with GCO Report No. 15. Utility owners and the MTRC provided blasting limits (for existing buildings and structures). These determined the critical peak particle velocities along the alignment for the Contractor's detailed blast design. The allowable peak particle velocities (PPV) ranged from 10 mm/s to 25 mm/s depending on the receivers.

(3) Soft / Mixed Ground

(a) Internal Tunnel Stability

Soft/mixed ground support typically comprised steel frames embedded in a shotcrete lining. Three-bar lattice girders were generally used for steel support. Typical support is shown on Figure 3.

During construction, after due consideration of the actual site conditions in the soft ground area, tunnelling proceeded by full-face excavation with provision of face support, which includes a temporary support buttress ("ring-cut" - see Figure 3).

(b) Pre-support

Pipe roofs were used to ensure tunnel stability near the face, prior to installation of shotcrete and steel supports. Steel pipes with an outer diameter of 48.3 mm or 60.3 mm and 6 m length were used, with a double row canopy in soft/mixed ground. The pipes were perforated to enable cement grouting of the installation drillholes. Pipes of this size could be installed by means of a conventional drilling jumbo, without the need for specialized equipment. A typical layout is shown on Figure 3.

Due to the shallow depth of the tunnel, a robust design was adopted to minimise ground movements. The loading calculation approach and the pipe design structural model after John & Mattle (2002) (corrected) was adopted. It is a closed-form solution based on silo theory, finite element analysis calculations and validations from case studies. The vertical stress for structural design of the pipe roof was calculated as:

$$\sigma_v(z) = \frac{(\gamma R - 2c').[1 - e^{(-2K_o.\tan\phi'.z/R)}]}{2K_o\tan\phi'} + \sigma_s.e^{(-2K_o.\tan\phi'.z/R)}$$

where:

$\sigma_v(z)$	=	Silo pressure (kPa) at depth z (m)
γ	=	Soil unit weight (kN/m^3)
c', ϕ'	=	Soil shear strength parameters (cohesion and friction angle)
K_o	=	Coefficient of earth pressure at rest (= 1 – sin ϕ')
σ_s	=	Surface surcharge (kPa)
R	=	silo radius = $0.5x\sqrt{(D.s)}$ where D = tunnel "diameter" and s = n x unsupported length with n= 1.5 in soft ground (after John & Mattle (2000) and n taken as 1.0 for limited rock cover

This method provided the necessary pipe spacing for a given unsupported length, which was generally taken as 1.5 m. The overlap between successive pipe roofs was based on face stability calculations, assuming a localised failure model. The structural capacity was calculated or the ultimate limit state in accordance with BS 5950:Part 1:1990.

In the finite element analysis, pre-support was modelled using a material of equivalent properties, as recommended by Rocscience (2005).

(4) Face Stability

The minimum support pressure required for face stability was evaluated using the method described in Lee and Nam (2000), which had been used for shallow urban tunnels in Korea. This is an upper bound limit analysis solution for a 3D face collapse mechanism, with allowance for the destabilising effect of seepage forces at the face. It is an extension of the solution by Leca and Dormieux (1990).

Seepage forces were calculated from the average horizontal hydraulic gradient near the face, obtained from seepage analysis (vertical gradients are small near the face and can be neglected). The analysis was carried out using the programme OASYS SEEP (1997) Version 3.10. It included a model represented by a vertical section along the tunnel axis, i.e. it modelled a "horizontal slot". The limitations due to the 2D nature of the model were mitigated by the use of an appropriate factor of safety on the calculated required face support pressure. The seepage analysis also helped to estimate the piezometric drawdown caused by the inflows into the tunnel and to predict a part of the ground movements (see Section "Ground Movements and Monitoring").

Face support was provided by a combination of shotcrete and face reinforcement by cement grouted, 12 m long T25 rebars. An average density of about 0.4 No./m^2 was used, with a 5 m overlap between successive patterns of face reinforcement.

It is worthy of note that reinforcing the face generally results in a reduction of surface settlement (Lunardi 2000). Indeed, by reinforcing the face and reducing the ground deformation, ground loss can be reduced since the deformation at the tunnel face is a major component of ground loss in open face tunnelling in mixed or soft ground.

Ground Treatment

The subway was mined in free air with ground treatment by grouting primarily for groundwater control. Grouting was carried out from the tunnel face as ground treatment could not be carried out from the ground surface, due to a lack of adequate working space and the presence of numerous utilities. A 2 m wide annulus around the tunnel periphery was grouted together with a 5m long grout "plug" formed ahead of the face (Figure 3). The grout hole layout was finalised on site by the Contractor depending on the actual ground water inflows and prevailing ground conditions ahead.

Ground treatment works were initially designed as a 2-stage tube-à-manchette (TAM) grouting operation (cement-bentonite grout followed by sodium silicate chemical grouting). After completion of the first section of grouting, single-stage grouting with micro-fine cement grouting was successfully adopted as an alternative, which offered time and cost benefits.

Back-grouting was also carried out behind the face on a case-by case basis to further control water inflows and ground surface settlement.

Breakthrough Works

The Engineer design required the construction of a permanent reinforced concrete portal structure prior to breakthrough. One of the movement joints of the existing Admiralty Tunnel was located some 2 m from the breakthrough location. As the excavation would create an out-of balance horizontal force on the existing tunnel, maintaining its lateral stability was a key consideration. The construction sequence was derived accordingly and is outlined on Figure 4.

Lateral stability was achieved by carrying out the excavation and construction of the new subway permanent base slab and portal frame section by section (central heading followed by two side headings). Temporary tunnel support and temporary inclined struts (contingency measure) were introduced in stages, followed by casting of the main portal frame. Temporary megashore props were also provided inside the Admiralty Tunnel as a precautionary measure.

The junction was formed by saw cutting of the lining of the Admiralty Tunnel.

GROUND MOVEMENTS AND MONITORING

Prediction of Ground Movements

In the rock portion of the alignment, ground movements were expected to be minimal. However, in soft/mixed ground portions, ground movements occur due to volume loss and de-watering near the face. The methods detailed in the following publications were used for assessment purposes: Peck (1969); O'Reilly & New (1982); Rankin (1988); Attewell & Hurrell (1985); and Attewell et al (1986) (for full references refer to Brand, 1996).

The volume loss is controlled by several factors, most notably the soil conditions, position of the water table and tunnelling method. Based on case history data from the Hong Kong Institution of Engineers (2001), Attewell et al (1986) and Burland et al (2001), a magnitude of 1% volume loss was considered to be appropriate for assessment of potential ground movements.

The shape of the settlement trough may be approximated to an inverted Gaussian distribution curve (Peck, 1969). i_y is the parameter that can be used as an indicator of trough width is named is the distance of the point of inflexion from the tunnel centre line on this curve. O'Reilly & New (1982) suggest that for most purposes an empirical equation relating the parameter i_y to the depth to the tunnel axis z_o may be adopted, namely $i_y = K z_o$, where the value of K depending upon the soil type and varying between 0.4 and 0.5 for fill and granular soils below the water table. For this project, a value of K = 0.5 was adopted.

To assess the effect of ground movements on surrounding buildings and infrastructure, the approach presented by Rankin (1988) was adopted. Rankin categorised the likely 'risk' to buildings based on the settlement of the building and the angular distortion that it experiences (the term 'risk' is used herewith in the sense of Rankin's paper, i.e. degree of damage).

Potential damage to utilities is attributable to differential ground movements, rather than absolute movements. Damage to existing buildings, structures and utilities due to ground movement were anticipated to be 'negligible'. The predicted ground movements were therefore considered to be acceptable.

Instrumentation and Monitoring

(1) Layout

Instrumentation and monitoring enabled the verification of the tunnel behaviour and construction effects on the existing buildings, structures and utilities. This comprised monitoring with surface instruments and tunnel convergence arrays, and monitoring of the adjacent existing MTRC running tunnels of the Island Line. Owing to access difficulties, it was not always possible to physically monitor directly above the subway

alignment. Monitoring was completed by regular visual inspections of the existing ground, roads and infrastructures in the vicinity of the Works.

An important aspect is the timely acquisition and evaluation of the data, to be compared with pre-defined trigger levels (**alert, action, alarm**). The levels are based on the design calculations and on values defined by third parties. In accordance with MTRC requirements, the trigger levels were set at 50%, 80% and 100% of the allowable movements. If any of these levels were exceeded, pre-defined actions were implemented.

The Gammon GEOMON system was used to manage the ground and groundwater monitoring records. GEOMON is a web-based data monitoring and reporting system that captures data on a regular basis and presents it in a graphical or tabular format. The results can be viewed at any time and by anyone with a web browser and security clearance. E-mail warnings are automatically generated by the system if readings pass pre-set trigger levels. Details of GEOMON are provided by Frame (2005).

(2) Summary of Measurements

No significant movements were recorded at convergence arrays installed on the temporary support.

Figure 5 provides a comparison of measured and predicted surface settlement. This relates to the area of lowest rockhead i.e. at approximately invert level. The maximum measured settlement is about 15 mm. The predicted values were based on a more conservative assumption of soft ground below the invert.

CONCLUSIONS

The challenging excavation of the shallow Queensway Subway was successfully completed underneath a busy major road on Hong Kong Island. The subway was mined in free air with ground treatment by micro-fine cement grouting for groundwater control. The NGI Q-system was used to estimate support requirements for excavations in rock. For tunnelling in soft/mixed ground, temporary supports of steel arches and shotcrete, pre-support (pipe roofs) and tunnel face support were designed with the aid of finite element analyses and closed-form solutions.

The temporary works design included detailed risk evaluations and measures to minimise surface settlement and maintain the integrity of existing buildings, structures and utilities. Ground movements during the excavation were less than predicted, with a maximum ground surface settlement of about 15 mm.

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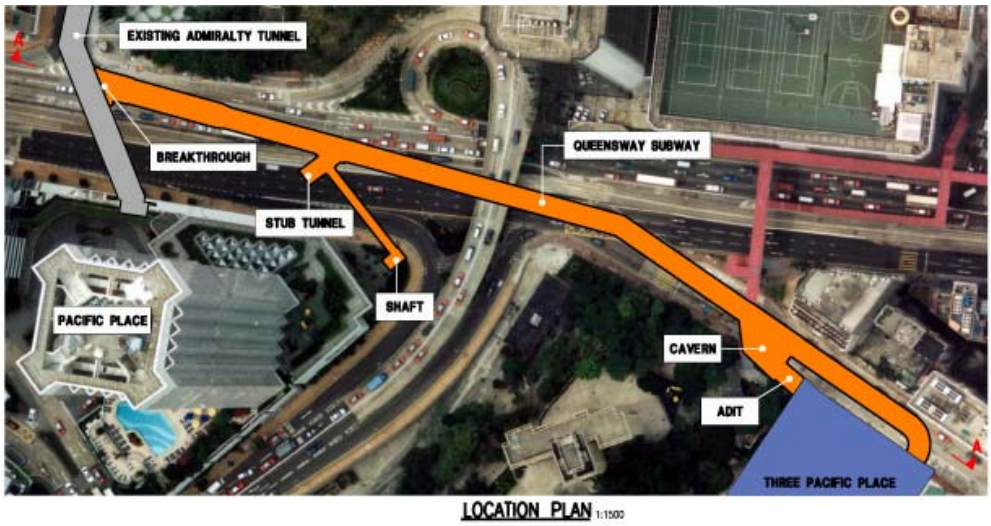
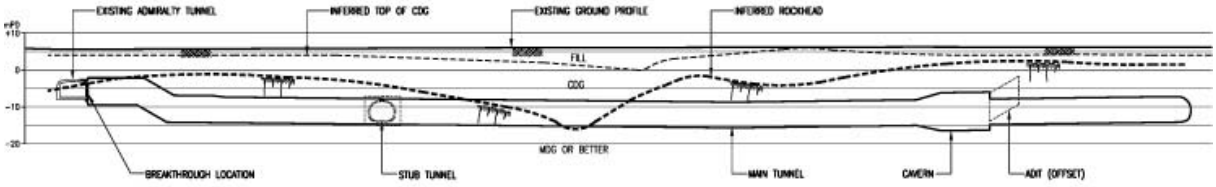
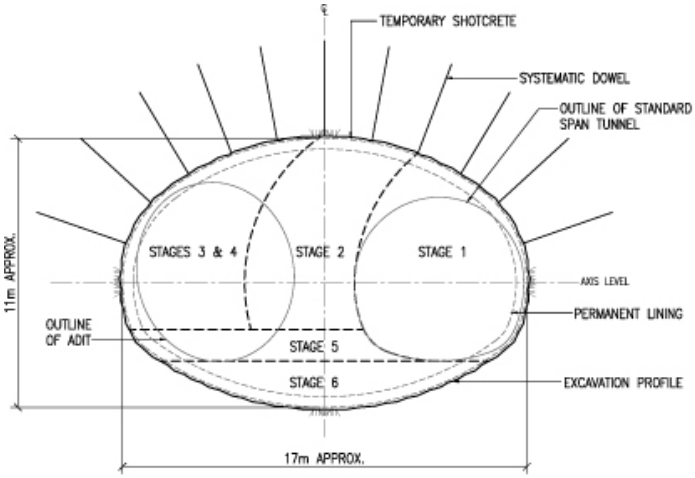


LOCATION PLAN 1:1500

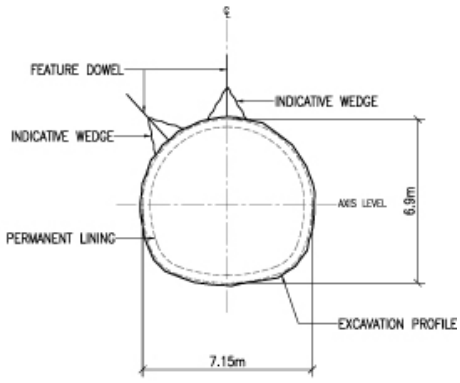


LONGITUDINAL SECTION A-A 1:1250

FIGURE 1 - LOCATION PLAN & LONGITUDINAL SECTION

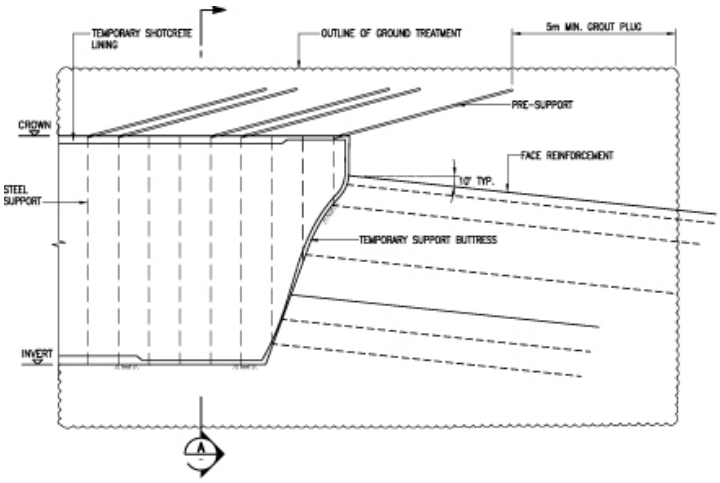


CAVERN 1:200
(SYSTEMATIC ROCK BOLTING (SWELLEX BOLTS) & SHOTCRETE)

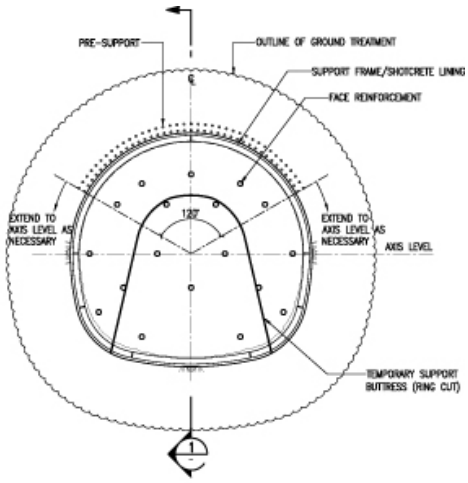


STANDARD SPAN TUNNEL 1:200
(MOSTLY SPOT BOLTING)

FIGURE 2 - TYPICAL TEMPORARY TUNNEL SUPPORT FOR FULL ROCK COVER

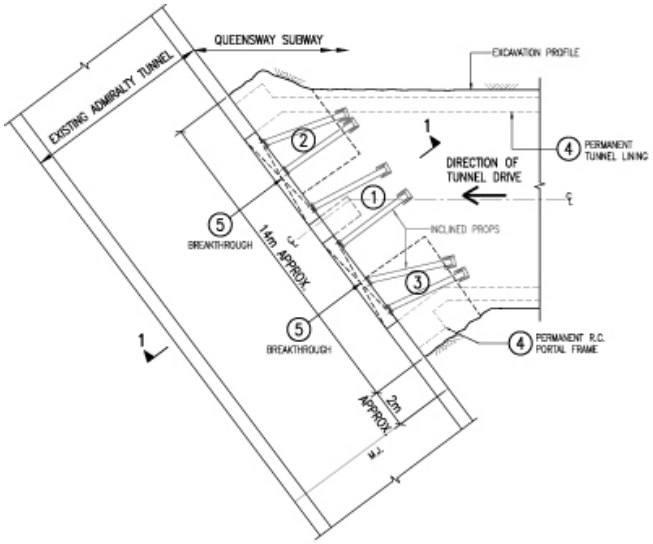


SECTION 1–1 1:150
TYPICAL LONGITUDINAL SECTION

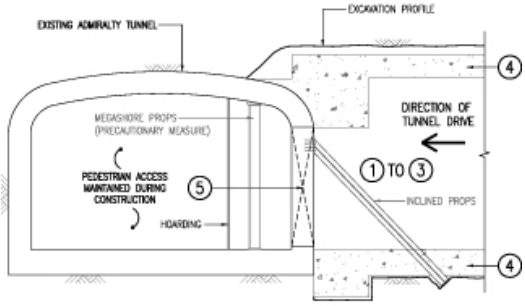


SECTION A–A 1:150
TYPICAL SECTION – SOFT GROUND

FIGURE 3 - TYPICAL TEMPORARY TUNNEL SUPPORT FOR SOFT/MIXED GROUND

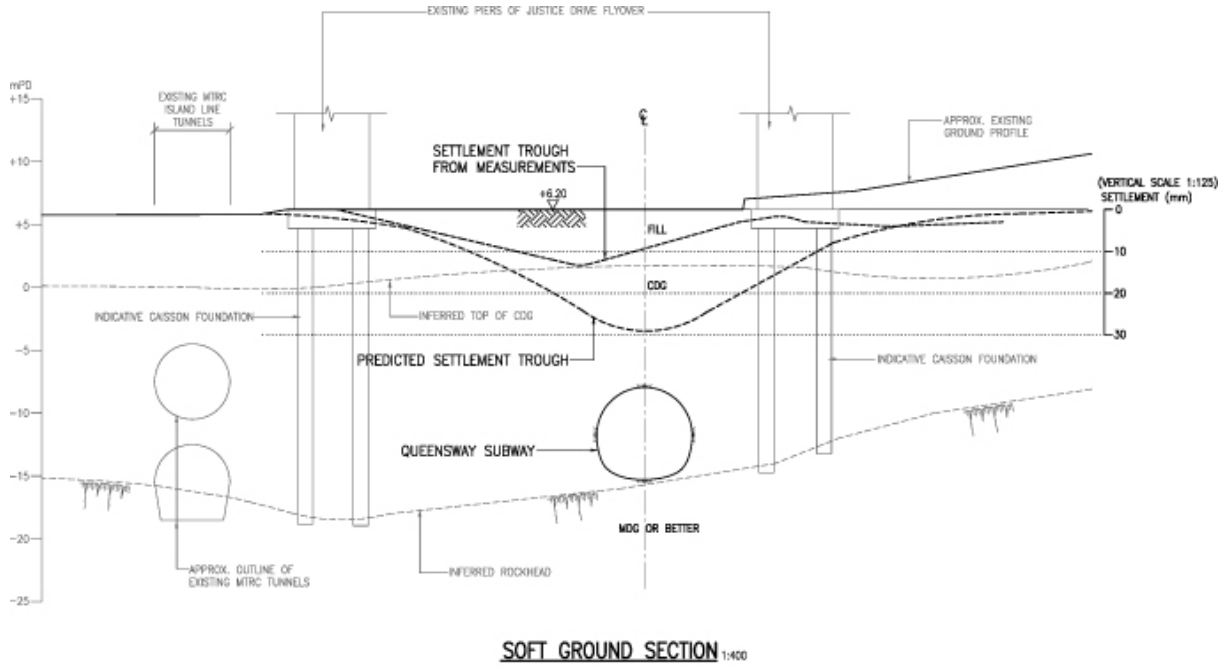


PLAN AT SUBWAY LEVEL – KEY CONSTRUCTION STAGES 1:200
NOTE: TEMPORARY SUPPORT OMITTED FOR CLARITY



SECTION 1–1 1:125
NOTE: TEMPORARY SUPPORT OMITTED FOR CLARITY

FIGURE 4 - OUTLINE OF CONSTRUCTION SEQUENCE FOR BREAKTHROUGH WORKS



SOFT GROUND SECTION 1:400

FIGURE 5 - COMPARISON OF MEASURED & PREDICATED SURFACE SETTLEMENT IN SOFT GROUND

CONSTRUCTION OF A CUT AND COVER TUNNEL ADJACENT TO AN OPERATING RAILWAY IN HONG KONG

Roger B. Storry[1], Richard Scott[1], Daniel Altier[1], Jack Pan[2], Glen Plumbridge[2],

Abstract

The Kowloon Canton Railway Corporation (KCRC) is constructing the Lok Ma Chau Spurline - a new 7.4 km railway connecting Hong Kong with a new border crossing to Mainland China at Lok Ma Chau. The contract, 'LBD201 - Sheung Shui to Chau Tau Tunnels' forms part of the Spurline and comprises the design and construction of 3.2 km of 8.75 m excavated diameter twin bored tunnels and approximately 2 km of cut-and-cover approach tunnels.

Construction of the cut-and-cover tunnels at the east was particularly challenging as it involved working within very restricted land boundaries at a distance of 1.5m from the live KCRC East Rail mainlines and 1m from the strategic Dongjiang Watermains. The choice of the construction techniques and sequencing was critical to ensuring that the works could be achieved without impacting the railway and watermains.

The railway operator imposed stringent criteria on the allowable rail movements and automated monitoring was implemented to ensure that the works were carried out without influencing the operation of the railway. Back analysis was undertaken based on the observed wall deflections to determine the actual ground stiffness. The results of the back analysis showed that the ground stiffness values for the coarse alluvium and completely decomposed granite could be up to 4 times the SPT 'N'.

INTRODUCTION

The Kowloon Canton Railway Corporation (KCRC) is constructing the 7.4km Lok Ma Chau Spurline, a second border crossing between Hong Kong and Mainland China, in the northern New Territories of Hong Kong SAR, refer to Figure 1. KCRC commissioned Dragages HK Joint Venture (DJV), under contract 'LBD201 - Sheung Shui to Chau Tau Tunnels', to design and construct 3.2km of 8.75m excavated diameter twin bored tunnels and approximately 2km of cut-and-cover approach tunnels and associated structures. Ove Arup and Partners (HK) Limited were employed by DJV to carry out the detailed design with Atkins China fulfilling the role of Independent Design Checker.

[1] Dragages HK Joint Venture,
[2] Ove Arup & Partners (HK) Ltd

One particularly challenging aspect of the Project was the construction of the tie-in with the existing East Rail mainlines north of the existing Sheung Shui station. This required the construction of 700 m of cut-and-cover tunnel in excavations of up to 14 m deep and 20 m wide within a very narrow strip of land, between the adjacent KCRC East Rail mainlines and Dongjiang Watermains, supplying potable water from China.

Ground movements were of prime concern to KCRC and to the Water Supplies Department (WSD) and a strict sequence of works, rigorous selection of plant and stringent monitoring controls were necessary to ensure that the live railway and the watermains were unaffected by the works.

Instrumentation was used to provide information on the performance of the temporary-works and to routinely verify the design. A back-analysis of the ground and wall movements demonstrated that the soil stiffness was much higher than that assumed in the design.

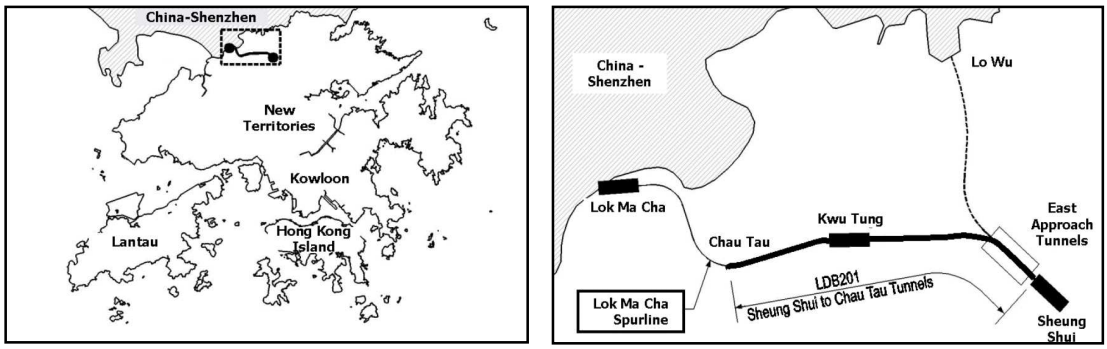


Figure 1. Location Plan

SITE DESCRIPTION

The cut-and-cover tunnels were to be constructed between the Dongjiang Watermains (Five pipes ranging in diameter from 1200 to 2400 mm) and the adjacent operating KCRC East Rail mainlines by diverting the twin-track railway to temporary “interim” alignments. Track realignment was required over a length of 950 m and was undertaken in several stages without interruption to the operating railway. This provided a narrow strip of land 13 m wide in which the construction of the tie-in and cut-and-cover approach tunnels could be carried out 1.5 m from the operating railway and 1m from the pipelines.

GROUND CONDITIONS

The ground conditions along the cut-and-cover tunnels comprised of up to 5 m of fill overlying between 4 and 8 m of alluvial deposits, which in turn overlies completely decomposed volcanics. Rockhead was encountered between 19 and 35 m below ground level. The ground water level was typically between 2 and 3 m below ground level.

DESIGN

The controlling factor in the design of the excavation was the sensitivity of the adjacent facilities to ground movements. Ground movements resulting from wall deformations and ground water drawdown are attributable to a combination of :

(a) installation of the sheet-piles walls;
(b) lateral movement of the retaining walls during the excavation and strutting; and
(c) groundwater draw-down associated with dewatering within the excavation.

Ground movement resulting from sheet-pile installation is largely dependent upon the installation method. The design was based upon negligible movements, which was later verified during installation of the piles.

The design of the retaining walls and the strutting sequence was undertaken using the FREW software. The recommendations of CIRIA Report C580 (2003) on the soil deformation analysis and radius of influence were adopted to convert the output into predicted vertical and lateral movements behind the wall. The effect on the groundwater was estimated using seepage analysis. The geotechnical parameters adopted for the design are summarised in Table 1.

Table 1. Design Soil Parameters

Material	γ (kN/m^3)	ϕ' (deg)	c' (kPa)	K_o	k (m/sec)	E* (MPa)
Man-made Fill	20	33	0	0.46	1×10^{-4}	8
Alluvial Sand and Gravel	20	34	0	0.44	5×10^{-5}	8-10
Completely Decomposed Volcanics (CDV)	19.5	31	6	0.48	1.2×10^{-5} to 2.0×10^{-6}	8-30

* A correlation of E=SPT N (MPa) was used

For the deeper excavations between 8 m and 14 m below ground, the design depth of sheet-pile wall was approximately 15 m to 23 m. With preloading of the temporary struts, the corresponding maximum predicted wall deflections ranged from 25 mm to 45 mm.

METHODS OF CONSTRUCTION

The final design required that two single-cell cut-and-cover tunnels be constructed side-by-side, both at different levels, ramping down to interface with the bored tunnels. Temporary sheet-piled excavations were adopted with the deepest formation level being approximately 14m below ground.

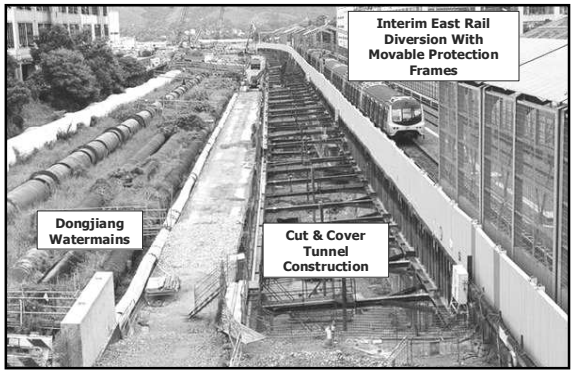


Figure 2. Cut & Cover Tunnel Construction

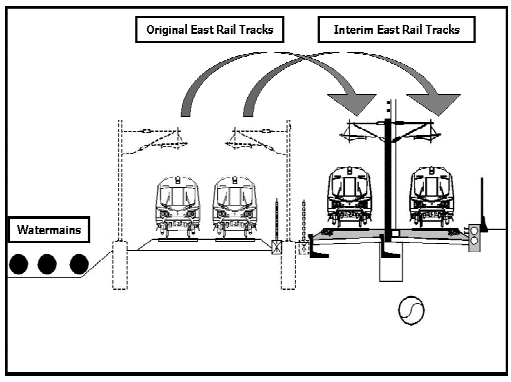


Figure 3a. Step 1 – Diversion of Existing Mainlines

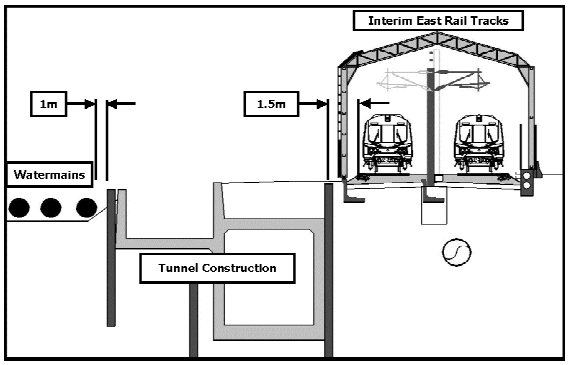


Figure 3b. Step 2 - Construction of Cut & Cover Tunnels

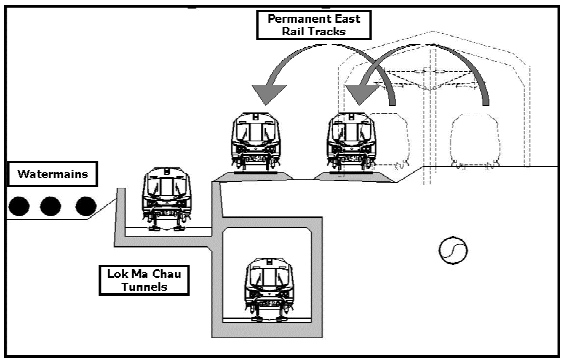


Figure 3c. Step 3 - Diversion of Mainlines to Permanent Position

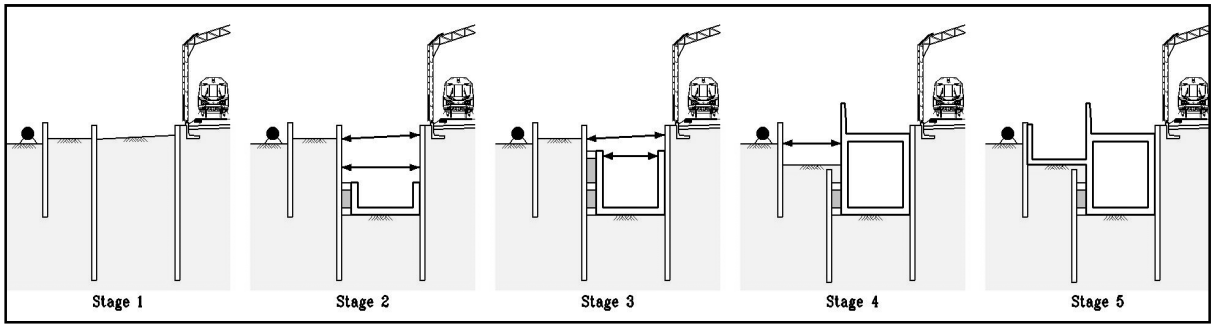


Figure 4 Typical Construction Staging

The method used to install the cofferdam walls was a significant consideration of the site with regard to vibration limits imposed for both the railway facilities and the watermains. A further requirement imposed by the railway operating company was to provide robust and "fail-safe" protection systems between the works and the live railway.

Figure 2 shows one of the excavations under construction sandwiched between the live railway and the watermains with the overall sequence of works shown in Figures 3a, 3b, and 3c. The construction staging adopted is presented schematically in Figure 4. In the event of a

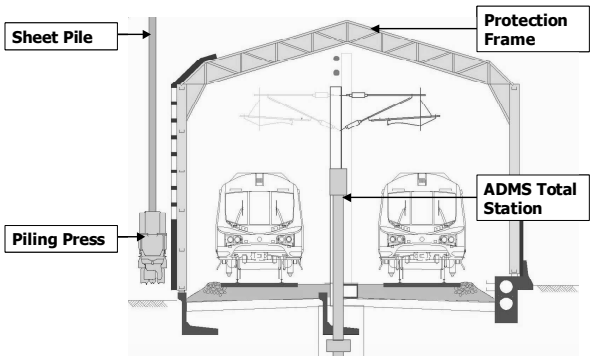


Figure 5. Press-in Piling Method Adjacent to Live Railway

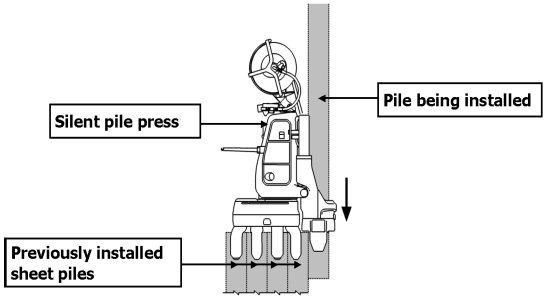


Figure 6. Press-in Piling Machine

failure of a crane boom or item of mechanical plant, measures needed to be in place to ensure nothing could conflict with the railway facilities. As the works had to be carried out close to the track this proved a particular challenge. This requirement was addressed using movable steel frames placed over both mainline tracks; these are shown in Figure 5. The protection frames could be moved during the night when railway operations were suspended. The movable frames provided adequate capacity to restrain against falling or swinging equipment, so the rotation and positioning of crawler cranes had to be restricted by electronic alarms and cut-off devices to ensure they could not move into the railway area.

The installation of the sheet-piles commenced using conventional vibratory techniques but following the use of several types of high frequency hammers it was concluded that the vibration limits could not be complied with where driving was directly adjacent to the railway and the watermains. The reaction-based press-in method was therefore employed to install the sheet-piles in locations directly adjacent to the railway and the watermains (Figure 6).

The press-in method utilises a reaction force derived from the previously installed piles and provides a solution that is more sensitive with respect to vibration and reduced risk to the adjacent facilities as it removes the need for large vibratory equipment being suspended at high-level in very close proximity to the railway. The unique way in which the system holds the sheet pile during installation was a further advantage in minimising risk to the railway. Crane movements close to the railway were minimised by adopting this system with only one service crane per area being used to supply the sheet piles to the pile-press from a fixed location.

Despite the clear safety advantages of this system ground conditions played an important role in the success of the technique. Sheet-pile installation was complicated by the presence of a layer of rock and boulder fill below the railway tracks. The press-in method could not penetrate the sheet-piles through such a layer, necessitating advanced continuous predrilling along the line of the sheet-pile wall. This was time consuming and difficult to achieve working so close to the live railway in terms of sequencing, locating the protection frames and the resulting ground movements.

OBSERVATIONAL APPROACH

The “Observational Approach” to construction is widely accepted in Hong Kong as a practical means of controlling construction works in urban areas whilst allowing the adoption of an optimal design. For this project, the difficult access constraints dictated the methods to be adopted and the proximity of the works to sensitive facilities meant that frequent and accurate monitoring control was essential.

The success of the approach relied on the establishment of clear lines of communication so that decisions could be made and appropriate actions taken in a timely manner. Monitoring data from survey and geotechnical instrumentation was reviewed against the pre-determined Alert, Action, Alarm (AAA) and Stop Work levels by a dedicated team of engineers. Prompt action was taken where data indicated movements approaching the AAA levels.

MONITORING

The monitoring scheme included :

(a) automated monitoring of the existing and diverted East Rail tracks at 4 m centres;
(b) ground deformation monitoring on adjacent existing structures and utilities;
(c) piezometers to monitor ground water levels around the excavation; and
(d) inclinometers to monitor deflection of the temporary retaining walls.

The scope of monitoring for the live East Rail "interim" railway included monitoring 800m of twin track, all electrical overhead line equipment (OHL) in addition to sensitive trackside signalling equipment. This monitoring was implemented for the full duration of the construction works along the cut-and-cover tunnels, using an automated system with results displayed in semi real-time on screens in the contractors and clients offices. The system was configured to alert key staff by SMS text message if predetermined trigger values were reached.

The automated system provided the following advantages:
(a) accurate monitoring with no possession of live tracks required for the regular monitoring activities;
(b) displacement data displayed at 4-hour intervals;
(c) graphical visualisation of the latest measurements and of the historical time graphs; and
(d) automatic notification of trigger values being exceeded 24 hours/day.

A full description of the monitoring results is presented in Storry et. al. (2005).

The movable protection frames posed a key challenge to the development of the automated monitoring system as they introduced blockages in sightlines. To overcome this, eight of the total stations were mounted on steel masts between the two interim tracks and one was located adjacent to the tracks (Figures 5 and 7).

The monitoring system was designed to cater for the eventuality that 1 or 2 total stations within a group could be blind to their reference prisms at any time due to the protection frames. Each group of total stations had a minimum number of reference targets mounted on adjacent buildings and shared targets, which enabled the blind total stations to locate themselves in space so assuring the integrity and accuracy of the overall monitoring system.

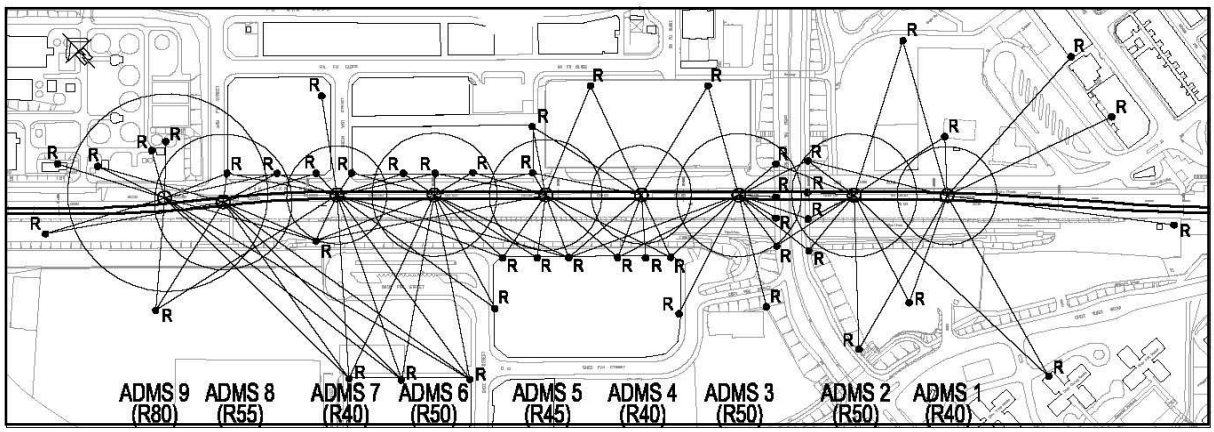


Figure 7. Layout of Total Stations and Reference Prisms

RESULTS

An example of the recorded movements is shown in Figure 8. Initial settlements can be seen to have occurred during the predrilling phase to break up the layer of boulders prior to the installation of the sheet piles. This data was used to alert the construction team and the railway operational team as to the scale and trend of the displacements and was used to facilitate decisions regarding the need for remedial works. When the railway operator considered that displacements could impact on the safe running of the railway, localised track tamping was carried out to maintain the railway tracks within defined operational limits with regard to twist and cant. Similar horizontal monitoring was used to ensure that the operational gauge of the railway was not impinged by track displacements.

This approach proved to be highly successful and no disruption to the operation of the railway occurred during the works.

Conventional optical monitoring of the watermains confirmed that the displacement resulting from the excavations were negligible. Inclinometer monitoring of the sheetpile wall confirmed that the displacements of the retaining walls were within those predicted by the design.

BACK ANALYSIS

Back analysis of the observed wall deflections was carried out to identify the primary factors leading to differences between the initial design model and the actual behaviour recorded. Near inclinometer INC103, the maximum wall deflection recorded after removal of the top temporary strut was 12.5 mm compared to a maximum predicted wall deflection of 27 mm at that stage as indicated in Figure 9. The actual wall deflections were less than predicted. This is considered to be due to a number of factors; actual ground water levels being 0.5 m below the design water levels; live loads from the trains being transient; and adoption of conservative soil stiffness parameters in the design.

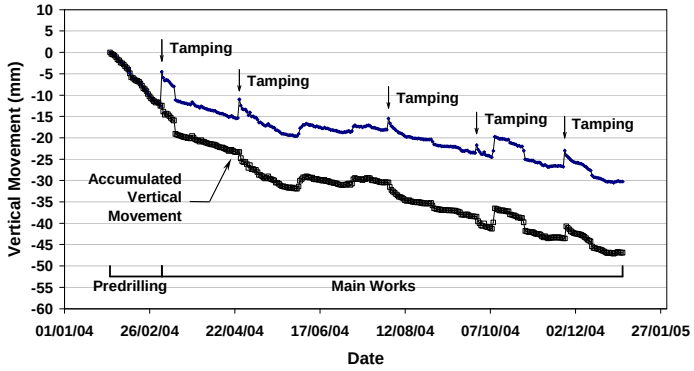


Figure 8. Vertical Movement (mm) of East Rail tracks

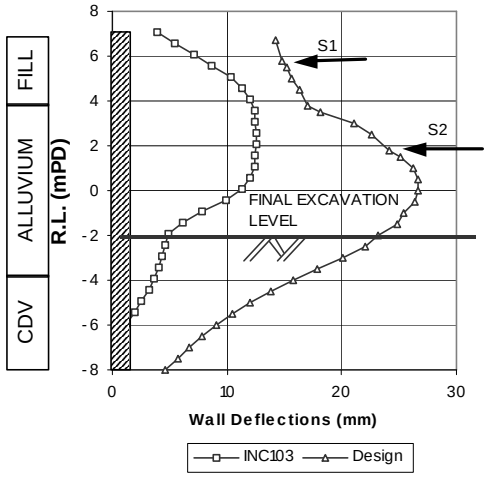


Figure 9. Comparison of Predicted and Actual Wall Deflections

The stiffness values used in the design were generally between 1.0 and 1.7N. The back-analysis for these cut-and-cover approach tunnels found values were more likely 1.5N for fill and 4N for course alluvium and CDV/CDG. Further details on the back analysis are included in Pan et. al. (2005).

CONCLUSION

The tie-in of the new KCRC Spurline to the East Rail mainlines has been successfully constructed within a narrow strip of land in close proximity to a live railway and primary water supply pipelines.

Special methods of work were adopted to meet the stringent requirements of the railway company (KCRC) and water supply provider (WSD) in terms of operational safety, ground movements and vibration. An automated monitoring system allowed an Observational Approach to be adopted to control the movements of the East Rail tracks and to adopt more flexible and efficient construction methods.

Construction of the 700 m of cut and cover tunnels commenced in September 2003 and was completed in March 2005, allowing the reinstatement of the permanent mainline tracks to start more than 5 months ahead of the envisaged programme.

Monitoring of the sheet-pile walls confirmed that the displacements of the retaining walls were well within those predicted by the design. Back analysis was carried out based on the observed wall deflections to determine the actual ground stiffness. The results of the back analysis show that the ground stiffness values of the coarse alluvium and CDG could be up to 4N, which is consistent with other published data in Hong Kong.

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CONSTRUCTION OF TWO ROAD UNDERPASSES AT MA ON SHAN

Sam H.Y. Ho[1] **and Eric S.F. Li**[2]

Abstract

Two road underpasses each some 200 m long, 12 m/16 m wide and 10m high were constructed in tunnel. The tunnel excavation encountered various types of weak, altered and metamorphosed sedimentary rock and strong granite rock in close proximity. The paper describes the methods employed for the tunnel excavation and discusses the temporary support system and also examines the typical fault zone materials encountered. Their effects on mechanical and blasting excavation, and the methods of determining the temporary support for closely spaced large span tunnels are described. The commonly adopted blasting design and vibration control are reviewed with the trial blasting results fed back into the design and procedures to optimise the production blasting. The successful and early completion of the tunnelling works highlights the importance and benefit of the regular input from an experienced engineering geologist. This helps improve the efficiency of the construction sequence and the excavation for the underpasses.

GENERAL DESCRIPTION OF THE PROJECT

Two road underpasses were constructed in tunnel in 2002-2003 to serve as slip roads as part of a new major trunk road T7 in Ma On Shan. Road T7 is a major road to serve the expanding new town urban development in Hong Kong. The road has a 3.2 kilometres stretch of dual 2-lane carriageway at the outskirts of Ma On Shan New Town. The Contract Value of the entire Road T7 project was approximately HK$1.4 billion, which comprised a key element for the construction of two road underpasses in tunnel (Underpasses C and D). The underpasses, which have tightly curved horizontal alignment, are gently sloping. Each of the underpasses was approximately 200 m in length and is required to link up the new Road T7 with the existing Sai Sha Road at Lee On Estate (Figure 1). The adoption of a tunnelled construction has the benefit of minimising extensive land take, avoiding roadside sloping cuttings and enhancing visual impact. The basic data associated with the project are listed below:

	Underpass C	**Underpass D**
Tunnel Size (approx. clear exc. span/height)	12 m / 10 m	16 m / 10 m
Tunnel Length, approx.	180 m	200 m
Excavation Section Area, approx.	117 m^2	148 m^2
Ground Cover	5 m (minimum) to 35m (maximum)	
Spacing of Tunnels	Minimum separation is about 25 m centre to centre. Minimum rock pillar width between tunnels is about 9 m	
Permanent Lining	400 mm to 550 mm thick	
Construction Period	Tunnel Excavation: April 2002 to Nov. 2002	

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DESCRIPTION OF THE TUNNEL CONSTRUCTION

General

The tunnels were formed initially by mechanical excavation using backhoe in soft or mixed ground, followed by a drill-and-blast rock excavation method in partially weak and competent rock. The layout of the Underpass tunnels is shown in Figure 1. Typical tunnel sections for the respective Underpasses C and D are shown in Figure 2.

As a common practice in tunnel construction, the Contractor of the project carried out a geological assessment at the outset. The principal objective of such geological assessment was to determine the likely extent of soil and rock distribution in excavation so that he could plan for the optimised method of excavation, types of temporary support and the programme of works. The detailed geology, rock mass classification and groundwater condition, which all have impacts to temporary excavation support and blasting design, were then reviewed by experienced tunnelling engineer/geologist during construction with reference to advance probing and the rock surface exposed.

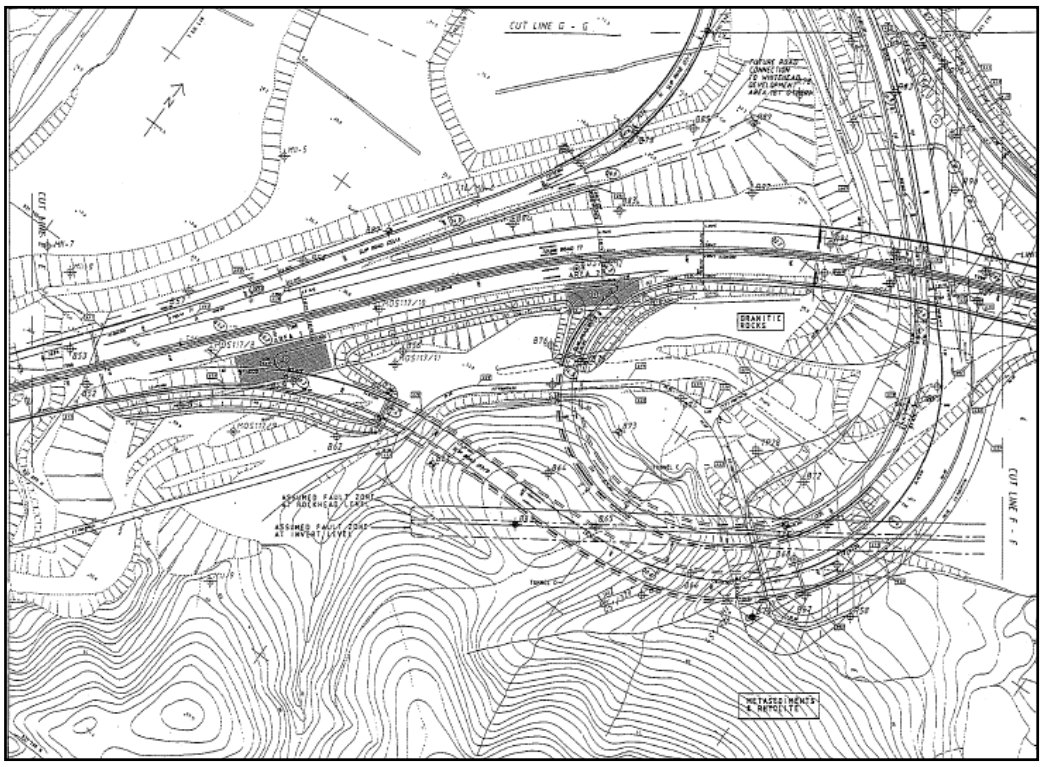
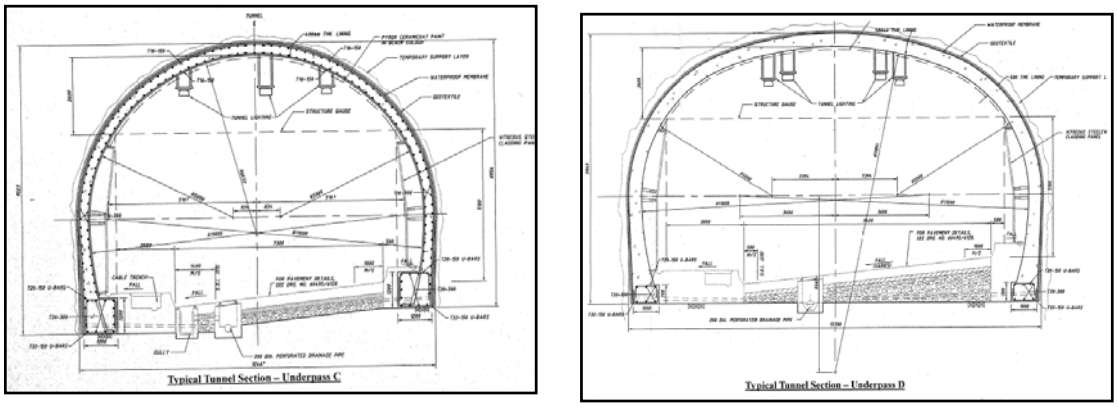


Figure 1. Site Plan

Engineering Geologist's Input in Tunnel Excavation

The cost-effectiveness of the selected temporary excavation support and smooth execution of each work cycle to a very large extent rely on the experience of the contractor's tunnelling engineer and his close interfacing with the construction team. For the tunnel construction at Road T7, the contractor opted to employ a highly experienced engineering geologist to work full time on site during the tunnel excavation. The majority of technical consideration and temporary work design for tunnel excavation was then entrusted to the geologist. Without the opportunity to review the detailed site investigation

and design data kept by the original designer, the geologist was required to carry out detailed study of the working drawings and to carry out his own geological and hydrogeological assessment prior to excavation based on the available information. This provided the basis for geological sections and profiles, which then could be updated as the excavation progressed.



Underpass C **Underpass D**

Figure 2. Typical Tunnel Sections

Tunnel rock mapping, carried out soon after each cycle of excavation almost on a daily basis, provided valuable knowledge about the structure, strength and behaviour of rock mass to be excavated. A continuous geological record and rock mass classification based on the NGI – Q System (Barton et al, 1977 and Grimstad & Barton, 1993) of the strata actually encountered during the excavation was also prepared. This helped to determine the form and extent of the temporary support system to the excavation. Although outline details of the temporary supports were provided by the Contract, the Contractor was allowed the flexibility to modify them, subject to the favourable Q values derived from rock mapping work and satisfactory ground deformation measured during construction. The temporary support class was then proposed to the Engineer for approval and implementation within 12 hours in order to meet the tight working cycle.

Tunnel Geology

The general geology of the site as revealed by the excavation is indicated in Figure 3. Based on available drillhole data prior to construction, the tunnel alignment was anticipated to pass through a series of originally yellowish brown to dark grey mudstone/siltstone strata, and massive, pinkish light grey, medium grained moderately weathered to fresh granite. The former had been metamorphosed by the igneous granite rock and associated faulting effect resulting in a locally heavily disturbed, altered, mylonitic weak rock which the designer generally named it as a metasediment unit.

This weak rock was associated with regional fault zone and was characterised by dark greenish grey, highly chloritized, fine grained, highly to moderately weathered altered rock with slickensided, closely spaced joints and wavy cleavages, which was less common in previous rock tunnelling works in Hong Kong. The rock materials were waxy

in appearance with polished surfaces and could easily be crumbled by hand which reflected their low strength. The joints or cleavages were usually inclined at 40° to 60° to the tunnel floor. There were also laminated thin layers of altered dyke rocks, which were rich in kaolin and locally sheared, usually completely to highly decomposed in nature. The rock was classified as weak and was associated with fault zones in this area, particularly when groundwater seepage was occurred.

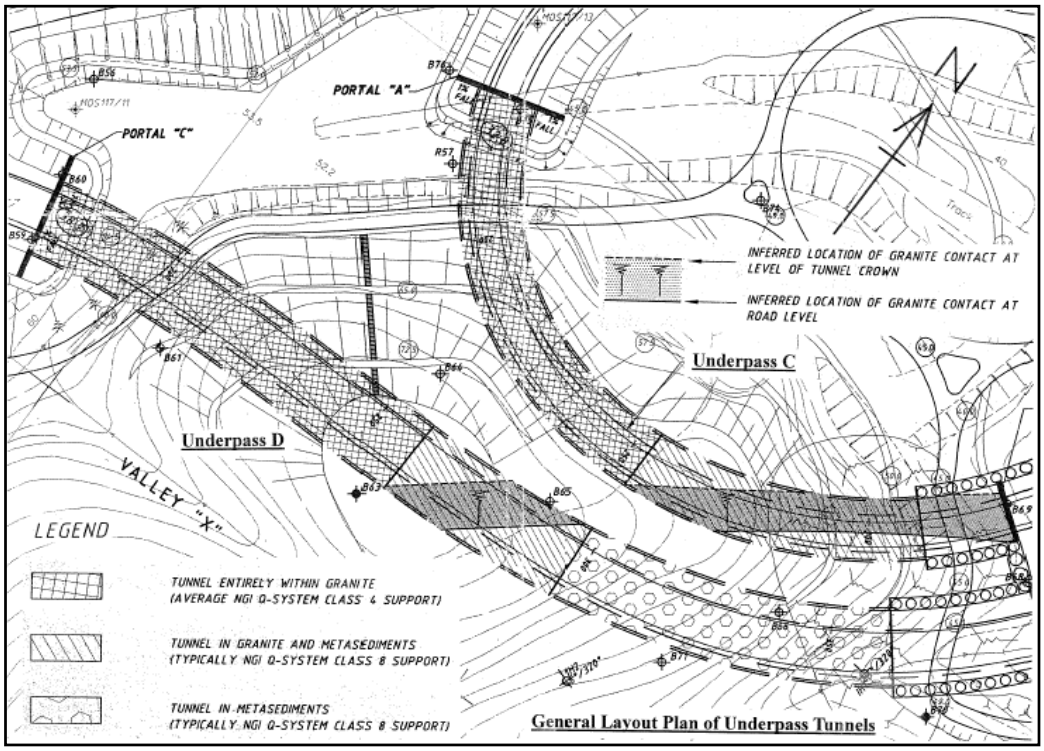


Figure 3. General Layout and Tunnel Geology

Geotechnical difficulties encountered in the fault zones during excavation included instability of the face, excessive overbreak, deformation by stress relief of sheared altered rocks, and intermittent water inflow frequently associated with natural stream courses at slope surface.

Fault zones between 0.5 m to over 10 m thickness, with thin dykes or minor rock intrusions, deformed and mylonitic structures, were all identified during the tunnel excavation. These geological structures were mainly occurred obliquely (some 50°) or were parallel or sub-parallel to the tunnel alignment. Zones of completely to highly decomposed rock associated with faults along the rock boundaries were recorded in thickness up to 8 m. Local minor water seepage was observed along the inclined contact plane between two major rock types and the softening effect of water infiltration was often noted. Due to the many cleavages and the preferred orientation of structural joints in the metasediments, there was a concern that the integrity of the rock structure would be impaired by groundwater seepage at the excavation face. As the contact zone between the rock masses was inclined at relatively steep angles, the associated joint pattern displayed adverse joints, which had the potential for local instability and block sliding failure. All of

the above features were identified as key factors in the determination of the temporary support type for the tunnel excavation.

Where the granite had intruded into the sedimentary rock, it was found to be only slightly altered near the rock interface and was usually massive with limited and moderately to widely spaced joints. The granite was very strong and, in some places, prolonged time was required to break or split by power tools where the ground condition was not suitable for drill-and-blast operation. There were also cut-and-cover sections at the eastern entrance to the tunnels where the portals were formed in soil and extremely weak completely decomposed rock.

ADOPTED METHOD OF UNDERPASS CONSTRUCTION

Excavation Sequence

Probe holes were drilled using a Jumbo Drill to ensure that at least one probe hole was maintained at a distance of at least 20 m ahead of the tunnel excavation face. The recorded penetration rate and resistance and the flushed returned rock chips or washed fines were reviewed by the geologist, which helped to assess the continuity and resistance of rock likely to be encountered. This information also helped to identify the potential for any heavy groundwater inflow that may require advanced ground treatment.

Plate 1. Steel Ribs and Fibre Reinforced Shotcrete Temporary Support for Excavation in Weak Rock

Excavation in the weak metasediment rock was carried out in both Underpasses C and D from the eastern portal using hydraulic breakers/hammers and backhoe excavators until the hard granite rock was encountered. The daily rate of progress was limited to 1.5m initially, which was controlled by the installation of a full steel rib set (see Plate 1),

shotcreting and support canopy. The latter was formed by drilling and installation of 6 m long inclined upward sub-horizontal steel pipes from the portal face into the rock. The 50mm diameter steel pipes were subsequently pressure grouted and plugged. In order to accelerate the grout setting time, an admixture was used to enable a grout strength of 30MPa in 24 hours could be attained. Using this approach, the normal excavation could be re-commenced 24 hours after grouting.

A 25 m length of mixed face conditions was encountered during the tunnel excavation, which comprised poor quality weak metasediment rock closely interfaced with strong competent granite (see Plate 2). For this length of excavation, a large number of closely spaced holes were drilled into the face to facilitate splitting of the granite and excavation using hydraulic breakers. Where the excavation consisted of a full face of strong relatively massive granite, the excavation was carried out using a drill-and-blast technique. The excavation for both tunnels employed a two-bench system with a top heading and bottom bench excavation.

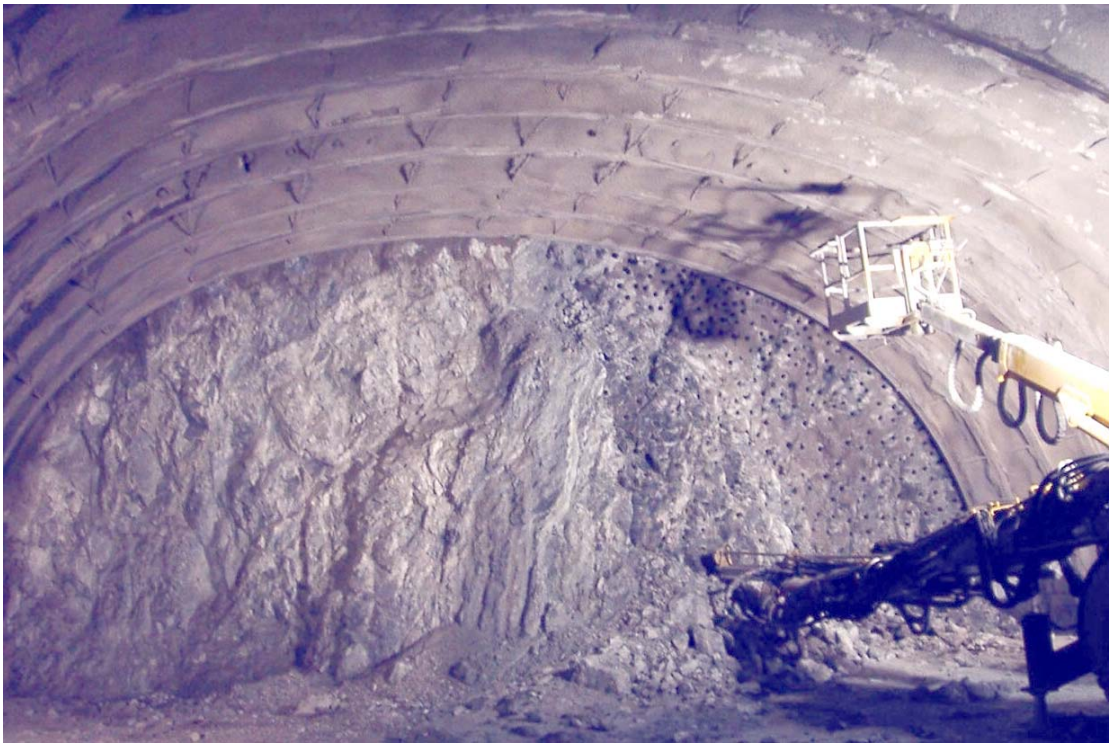


Plate 2. Tunnel Excavation at Interface between Weak and Very Strong Rock

Top Heading Excavation

The top heading was initially excavated using the ring cutting method with a backhoe excavator and a hydraulic breaker utilised as required. The excavation profile had allowed clearance to accommodate the installation of the lining, geotextile, waterproof membrane, road structure and pavement. The excavation proceeded for slightly in excess of one metre, which allowed a first layer of shotcrete to be applied for initial support. The upper portion of a temporary arch steel rib was then lifted into the end of the excavation and braced to an existing temporary arch steel rib.

The arch rib temporary support was then strengthened by the application of a second layer of shotcrete to ensure that the gaps between the first layer of shotcrete surface and the surface behind the arch ribs were adequately filled. The excavation of the top bench was extended in this manner at 1-metre increment until relatively competent rock was encountered or a change in the temporary support class was required.

The shotcrete had a minimum compressive strength of 30 MPa and the steel fibre complied with ASTM A820-90. The minimum quantity of steel fibre to be added was 40 kg/m^3 and the as-placed water cement ratio was less than 0.5. The shotcrete was placed in a circular fashion by a truck mounted spraying machine to build up to the required thickness, working from the bottom upwards so that rebound does not accumulate on the area that still has to be covered. The nozzle was held so that the sprayed concrete struck the receiving surface at approximately right angles and at a distance of 1 m – 1.5 m from the nozzle. The shotcrete was applied in uniform layers to minimise rebound and segregation and to prevent slumping or sagging. Weepholes were installed using a Netlon drain mat where persistent heavy seepage or significant flow rate was recorded. The fine trimming of the excavation was carried out using hand tools or backhoe to achieve the required profile and to minimise overbreak and ground movement.

Plate 3. Full Tunnel Face Hard Granitic Rock after the Lower Bench Excavation

Bottom bench excavation

Upon completion of a portion or full length of the top heading in weak rock, excavation of the bottom bench commenced. Access ramps were established to suit the site condition and maintained as the excavation progressed. Alternating excavation using a pocket (or “hit-and-miss”) approach was adopted to minimise excessive rock face exposure without temporary support. When a full depth of the bottom bench (see Plate 3)

was exposed the temporary support was installed following a sequence similar to that adopted for the top heading.

BLASTING DESIGN AND VIBRATION CONTROL

General

A drill-and-blast excavation technique could only be adopted effectively in tunnel sections where a full face of strong competent rock was exposed. Cartridge emulsion explosives Powergel 32 x 300 and 25 x 300 were used together with detonating cord Cordtex 5. A non-electrical detonator (NONEL) was used for initiation. According to the HK regulations, all cartridge explosives and related accessories were delivered to the site by the Mines Division of the Geotechnical Engineering Office immediately prior to each blast, as no site magazine for storage could be practically provided and also due to the limited quantity. Installation of explosive sticks was closely supervised by inspectors of the Mines Division.

Typical Blasting Design

One of the principal concerns associated with the blasting design for Road T7 was the potential adverse effects of ground vibration on adjacent slopes, buildings, utilities, and the adjacent tunnel excavation. This was addressed by imposing a conservative maximum allowable peak particle velocity (PPV) value induced by the blasting. This limit was usually set significantly below the tolerable maximum for the structures at risk of damage.

These values were verified by trial blasting on site using regression analysis of data of measured ground vibrations (GEO, 1992). For the public and private residential buildings and roads adjacent to the site, the allowable ppv was taken to be 25 mm/s, which was reduced to 13 mm/s for water carrying services and adjacent tunnels.

For both tunnels, the drillhole depth and explosive charge weight per hole were 0.55kg (1 m drillhole) to 3.85 kg (5 m drillhole). The estimated volume of production per blast ranged from 88 m^3 to 440 m^3 for Underpass C and 119 m^3 to 590 m^3 for Underpass D. The charged drillhole spacing was a function of estimated rock mass strength and was generally between 500 mm and 700 mm. The charge concentration was 0.395-0.220 kg/m. Optimum blasting patterns were determined following detailed face mapping of the exposed rock surface by the geologist and discussion with the shot-firer.

TEMPORARY TUNNEL SUPPORT

Temporary Support Class

The temporary support classes for the 16 m span tunnel excavation that were initially adopted for the construction of the underpass tunnels are presented in Table 1. These were based upon prescribed classes indicated by the Engineer in the Contract documents. There were only minor differences between the support requirement between Underpasses C and D. At commencement of construction it was envisaged that between 40% to 55% of the

tunnel excavation would require either Class 8 or 9 temporary support. This included the provision of steel ribs at 1200m spacing braced by a full span horizontal strut inserted across the base of the excavation.

Table 1. Prescribed Temporary Support Classes for Underpass Tunnels

SUPPORT CLASS	ROCK MASS Q VALUE (Q (ROCK))	MINIMUM INITIAL SYSTEMATIC SUPPORT REQUIREMENTS (BASED ON Q(ROOF) & Q(WALL)	MINIMUM MONITORING REQUIREMENTS
1 & 2	>4	Feature dowels : Types A or B in Crown & Wall as necessary Systematic dowels : None Shotcrete : Min. 25mm applied as necessary to local fracture zone	Convergence monitoring unlikely to be necessary
3	2 – 4	Feature dowels : Types A or B in Crown & Wall as necessary Systematic dowels : 4m long on 2.4m grid in crown, none in walls Shotcrete : Min. 25mm in crown, none in walls	Convergence monitoring unlikely to be necessary
4	0.6 – 2	Feature dowels : Types A or B in Crown & Wall as necessary Systematic dowels : 4m long on 2.2m grid in crown & walls with faceplates Shotcrete : 70mm in crown, none in wall	Convergence monitoring at maximum 36m intervals
5	0.3 – 0.6	Feature dowels : Types A or B in Crown & Wall as necessary Systematic dowels : 3m long in 1.8m grid in crown & wall with faceplates Shotcrete : 70mm fibre shotcrete in crown, 40mm fibre reinforced in walls	Convergence monitoring at maximum 24m intervals
6 & 7	0.1 – 0.3	Feature dowels : Types A or B in Crown & Wall as necessary Systematic dowels : 4m long on 1.5m grid in crown & walls Shotcrete : 120mm fibre reinforced in crown & walls	Convergence monitoring at maximum 12m intervals
8	0.03 – 0.1	Steel ribs, at 1200mm c/c, 152x152 30 UC, min with full span strut. 200mm thick steel fire reinforced shotcrete lagging. Support canopy, tubes at 500mm c/c in one layers	Convergence monitoring at maximum 12m intervals
9	< 0.03	Steel ribs 152x152 30 UC at 750mm c/c, min with full span strut. 200mm thick steel fibre reinforced shotcrete lagging. Support canopy, tube at 500mm c/c in two layers staggered.	Convergence monitoring at maximum 12m intervals

The final extent of temporary support to Underpasses C and D are shown in Table 2.

Table 2. Summary of Temporary Support Classes Adopted in Tunnels

Underpass /Tunnel	Total Length of Excavation	Length (%) in Support Class 8 (Q – Values)	Length (%) in support Classes 1 & 2 (Q – Values)	Remarks
Underpass C	136m	59m (43%) (Q = 0.02 to 0.16)	77m (57%) (Q = 8.3 to 15.0)	43% of tunnel excavation by mechanical breaking
Underpass D	187m	103m (55%) (Q = 0.06 to 0.22)	84m (45%) (Q = 8.3 to 15.0)	58% of tunnel excavation by mechanical breaking

Rock Mass Classification for Temporary Tunnel Support Design

Rock mass properties observed in the excavation of tunnel were classified using the NGI Q system. Upon removal of the fill, colluvium and in-situ deeply weathered rock or soil in the initial excavation, bedrock as subsequently exposed in the first half of tunnel was mainly weak, highly fractured, foliated and sheared metasedimentary rock associated with contact metamorphism of the region (see Plate 4). The remaining half of the tunnel was in massive and blocky moderately weathered to fresh coarse grained granite. A change in rock characters from weak rock, mixed weak and strong rock and massive strong rock was therefore experienced during the two tunnel excavations (see Plate 2).

Plate 4. Sheared and Altered Weak Rock of Metasediments with Inclined Intrusion at Underpass C

For temporary tunnel support determination, the areas where metasediment and granite met and affected by faults were of great concern. The standing time, failure modes, extent of failure and the anticipated tunnel performance upon blasting to the adjacent tunnel were considered whenever signs of such weak rock bands were revealed. These rock bands were characterized by the formation of blocky, brecciated (disintegrated) and clayey (foliated/laminated/sheared) bands of weak rock. NGI Q values of this type of fault-influenced weak rocks ranged from 0.04 to 0.11. Brief descriptions of the various weak rock zones are given below.

(a) Blocky fault zones are formed by many intersecting discontinuity sets, mainly within the granitic bedrock with preferred joint orientations and friable rock fragments. Clay or brecciated infill is generally rare in this type of fault zone.

(b) Brecciated fault rocks are poorly interlocked and heavily broken rock mass with a mixture of fine to medium sized angular and sub-rounded rock pieces. This type of fault zone was generally found at Road T7 in alternations of dykes

or intrusions, which occurred at the interface of granite and metasediment. The alteration produced a high chlorite and mica content, which exhibited deformed texture and slickensided coating. Ravelling in brecciated zones and flowing conditions in areas affected by groundwater were encountered.

(c) Clayey (foliated/laminated/sheared) fault zones are thinly laminated, tectonically sheared, slickensided and highly weathered bands of weak rock. Clayey fault zones are generally associated with dykes and intrusions with rich feldspar and mica content. Mineralization and subsequently deep weathering created a pattern of white and dark grey to black colour foliated soil seams within the contact zone of metasediment and granitic rocks.

Weak rock resulted from fault or shear zones has not been addressed in details in the NGI Q-System. At Road T7, there were apparently large variations in characteristics of the parameters such as the rock mass strength, drill core quality, uniaxial compressive strength, spacing of discontinuities, separation, roughness, weathering, groundwater and strike and orientations of major discontinuities, etc. Such parameters all have impacts on the use of temporary support types and optimization of the temporary support system could be carried out taking into account of the as-exposed geological conditions. The decision however must be made together with the need for speedy progress.

MODIFICATION OF TEMPORARY SUPPORT MEASURES IN WEAK ROCK

Rock mass classifications normally consider only the homogeneous units and so adjustment of the classifications adjacent to shear zones may be difficult. It is envisaged that the rock mass affected by a shear zone could be much larger than the shear zone itself exposed. Hence, this rock mass must be scaled down to the quality of the shear zone so that a heavier support system than a standard section will be required.

The initial support class adopted by the contractor was predominantly Class 8. This involved the installation of close-loop steel rib sets and the application of steel fibre reinforced shotcrete as a temporary lining. Convergence measurements, precise level and optical target surveys were all carried out. The monitoring results indicated the excavated tunnel could be in more stable conditions than that assumed by empirical method. During the initial phase of excavation, the contractor and the geologist investigated the potential of a more efficient and effective construction and initial support cycle. These investigations suggested that the full width invert strut, which originally was a part of the steel set design to form a closed system, could be deleted. This was checked using the survey and monitoring data together with rigorous numerical analyses using the FLAC (Fast Lagrangian Analysis of Continua) computer modelling program. The numerical analyses were valuable in carrying out sensitivity studies for several permutations of excavation sequence and temporary support systems for the tunnels, together with varying estimates of load transfer to the temporary support due to ground relaxation and the presence of groundwater.

The results were used to develop an optimized excavation sequence for Class 8 or 9 temporary support system. This involved a heading and bench excavation, steel ribs and

shotcrete, but without full width horizontal invert strut. The analyses indicated that the magnitude of the bending moments and axial forces in the steel ribs and shotcrete for the optimized scheme were similar to those associated with the designer's originally envisaged form of construction. The invert steel strut was subsequently deleted and replaced by inclined rock dowels installed at both ends of the steel set. Convergence monitoring of the steel rib set and shotcrete cover in more frequent time interval demonstrated that the alternative support system was adequate. This change in temporary support design increased the rate of excavation in weak rock from 4.5 m/week using the original design to 7.5 m/week using the optimized support scheme. This change and the associated increased productivity contributed to achieve an early completion of the tunnel excavation by one month, which represents a time saving of some 15% against the original plan.

INFLUENCE OF GROUNDWATER DURING CONSTRUCTION

During the excavation period, it was observed that all the installed temporary support measures were adequate despite the frequent rock blasting and relatively short distance or separation between the two large span tunnels. The water seepage through opening of joints and discontinuities in the exposed rock was lower than anticipated. It was also noted that the metasedimentary rock formation had been altered by local contact metamorphism to form a mylonitic and highly chloritized, deformed rock. This had produced a heavily fractured but tightly compacted rock mass with sealed joints that were non-uniform and resulted in low mass permeability. This meant that the effect of groundwater infiltration in altered rock with a preferred orientated jointing system would be quite different from what was normal assumed in design. It is possible that these particular rock mass characteristics lie outside the conditions covered by the NGI Q–System. The current application of a relatively light steel rib support system combined with fibre-reinforced shotcrete was proven to be adequate.

INSTRUMENTATION AND DEFORMATION MONITORING

Ground movement instrumentation was installed prior to the tunnel excavation and rock blasting works. This comprised inclinometers, magnetic extensometer and ground settlement plates installed at shallow depths. This helped to assess the performance and adequacy of the temporary support during excavation, particularly in areas of poor quality weak rock. Although the monitoring system was designed with intervention (alert, alarm and action) levels, it was not necessary to exercise any of the actions associated with these limits even when blasting works were undertaken with limited soil and rock cover. It was noted that the measurement devices, such as reflective prism targets, for convergence monitoring and precise surveying were more practical and reliable in monitoring ground deformations within the tunnel. However experience from the two underpasses construction showed that any interpretation of monitoring data must be accompanied by regular and detailed inspection of the as-sprayed shotcrete protective layer where crack generation may be a useful supplementary evidence of ground movement.

DISCUSSIONS

In comparison to typical granitic rock zones, due to their highly fractured nature, the metasediments in Road T7 tunnels were more sensitive to the vibration induced by rock blasting. The present Q-System rock mass classification and support systems for the fault zones developed by NGI appears to be insufficient for describing deformation, failure mechanisms and other phenomenon encountered during the drill-and-blast rock tunnel excavation. In addition, there were a few incidents that dyke rocks and inclined rock boundary collapsed soon after excavation, which required immediate supporting measures.

Another potential hazard of tunnel instability is related to rock blasting at large ground openings in close proximity. It was however observed through convergence monitoring and close examination of the integrity of temporary shotcreted linning that the tunnels were not apparently affected by regular blasting operation, which had lasted for some four months. It is considered that the applied temporary support measures, i.e. quick set shotcrete with steel fibre reinforcement and the low primary stress distribution, had been effective to safeguard any adverse effects to the closely spaced tunnels.

The presence of fault zones in tunnel excavation may lead to a completely different approach to the temporary support requirement. In this case, the drilling data could possibly indicate a poor to fair rock qualities and the advance rate of excavation and the provision erection of steel set would be significantly affected.

In determining an appropriate temporary support class, the routine rock mapping exercise could not be totally relied on. It is recommended that the geologist and shot-firer should work together to figure out the potential standing time of excavated faces, charge weight and blast hole arrangement, likely performance of rock upon blasting and the need to reduce the pull out length etc. It is therefore important to have experienced field personnel who possess in-depth knowledge of rock mass properties, behaviour of rock upon blasting, groundwater seepage handling, and skill in engineering design of temporary support.

In ground investigation for tunnel construction, exploration of fault zone and dyke rock is often difficult and requires drilling equipments that are capable of retrieving high core recovery samples. If the faults are not detected in advance, it may have an adverse effect on time and construction costs.

It appears that new rock classification parameters are required to describe the deformation and behaviour in fault zone environment. In this respect, development of new applicable classification of sedimentary and metamorphic rock will be beneficial, though it may be of limited application for specific types of rocks in Hong Kong.

CONCLUSIONS

Two large span tunnels, each of approximately 200m in length, have been constructed to form road underpasses for urban development at Ma On Shan. The construction of these tunnels involved excavation in rock of variable quality ranging from

heavily fractured, highly decomposed, very weak metasediments and intrusions, to very strong, relatively massive granite. Prior to and during construction there was significant interaction between the construction team and the tunnelling engineering geologist who was deployed to the tunnel excavation and temporary support design. This arrangement

Plate 5. Completed Underpasses C and D Showing East Portal

was proven to be highly effective and cost effective leading to optimisation in temporary support systems in poor quality rock, and blasting patterns in more competent rock. The geotechnical input from a proficient and practical professional on site was vital to a smooth and effective tunnel excavation, particularly in complex and unusual ground condition. It also contributed to a highly effective, productive, problem solving approach working relationship with the Engineer's site staff, design office and main contractor. The outcome of a smooth progress of works which ended up with an early completion of tunnel excavation and cost saving could be envisaged. This brought a happy partnering ending for the Employer, the Engineer and the Contractor (see Plate 5 for the completed Underpasses).

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NORTHWEST SIDE RELIEF SEWER TUNNEL CHALLENGES BECAME SUCCESS STORIES

Don Gupta[1] and Clay Haynes[2]

Abstract

The Milwaukee Metropolitan Sewerage District's (MMSD)'s challenges of basement flooding and wet weather sewer overflows were creatively addressed with the Northwest Side Relief Sewer Tunnel. The US$120 million, 6.1 m (20-foot) diameter, and 11,500m (37,740 feet) long conveyance and storage tunnel excavation in dolomite at depths reaching 50 m (165 feet) below ground surface was successfully completed in the fourth quarter of 2005. The design and construction of the tunnel is a success story for Milwaukee area tunneling practice where state-of-art pre-excavation grouting techniques were successfully employed for shaft and tunnel excavations to manage high groundwater inflows. Other tunneling challenges included dealing with contaminated sites, tunneling in densely populated urban setting and along the environmentally protected pristine Menomonee River parkway. The project contract documents included a comprehensive Geotechnical Data Report and the Geotechnical Baseline Report (GBR).

This paper also presents the design details including calculations of groundwater inflows using Heuer's and Goodman's methods, addressing constructability considerations in an urban environment, tunnel connections to existing large interceptors and a deep shaft, consolidation and contact grouting design to minimize groundwater inflows upon completion, and how to establish appropriate baselines in the GBR.

INTRODUCTION

The Milwaukee Metropolitan Sewerage District's (MMSD)'s challenges of basement flooding and wet weather sewer overflows will be significantly reduced with a US$132 million (including design, construction and contract administration) storage and conveyance tunnel. The Northwest Side Relief Sewer (NWSRS) tunnel provides relief to surcharging interceptors and introduces an additional 88 million gallons or 20 percent more storage to the MMSD's collection system. The tunnel is 6.1 metre (20 feet) in finished diameter and 11,500 m (37,740 ft) long, and excavated in dolomite

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rock at depths ranging from 40 to 50 m (130 to 165 ft) below surface in Milwaukee, Wisconsin. The tunnel boring machine (TBM) mining started in March 2003 and completed in April 2004. The final concrete lining was completed in May 2005 and the project facilities were operational by the end of 2005.

MMSD is the project owner, Black & Veatch Corporation is the designer, and the J.F. Shea-Kenny JV is the construction contractor. The NWSRS tunnel is a good model for tunneling industry where multiple design and construction challenges have been turned into success stories and ultimately resulted in happy project partners.

DESIGN CONSIDERATIONS

The NWSRS tunnel alignment generally follows pristine rivers and parkways along the Little Menomonee and Menomonee Rivers and extends from 91st Street and Denver Avenue to an existing drop shaft (CT-1) at the Milwaukee County Grounds for connection to the existing Inline Storage System (ISS) tunnel (Figure 1).

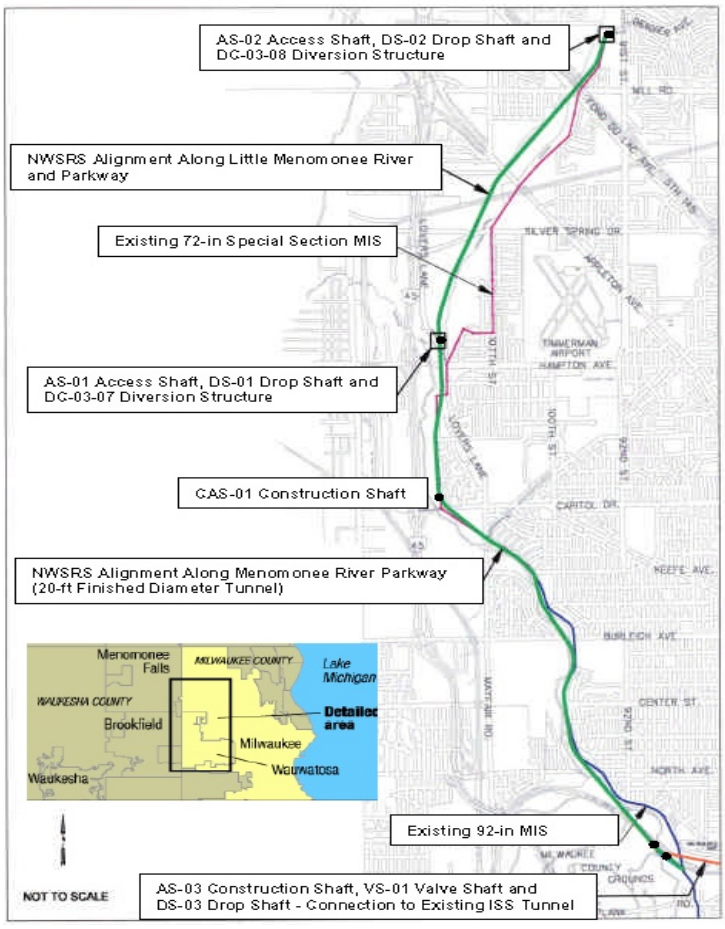

Figure 1. NWSRS Tunnel Alignment and Structure Locations

The tunnel will normally be empty and will function as an off-line storage sewer during certain wet weather conditions, initially on the order of two to five times annually and increasing in frequency as flows increase over time. The tunnel fills and empties by gravity and slopes downward from its northern terminus to its lowest point near the ISS drop shaft connection. After the wet-weather events, the NWSRS tunnel is drained into approximately 45 m (150 ft) deeper ISS tunnel through a series of cone valves, and the wastewater flows are ultimately transferred to treatment facilities via the ISS. Another operating scheme includes closing the cone valves and operating the system in an inverted-siphon relief sewer mode where surcharges are directed to existing shallower Metropolitan Interceptor Sewer (MIS) at the downstream end. This mode of operation saves costs involved with pumping the flows to the treatment facilities from the deeper ISS tunnel.

The tunnel slope was established at 0.2 percent to be self-cleaning and to accommodate topographic and geologic conditions. The tunnel invert depth was selected based on a low point in the top-of-rock subsurface elevation just south of Hampton Avenue. This low point was a pivot point controlling the vertical alignment to ensure the entire tunnel remained in rock. Another factor determining the grade was the potential future tunnel extension to 107th Street and County Line Road, roughly 7,300 m (24,000 ft) northwest of the project limits. The project is located in seismic Zone 1, in an area of low seismic activity. Based on available information, the tunnel was not required to be designated gassy.

NWSRS PROJECT FACILITIES

In addition to the tunnel, the NWSRS project included a total of 11 shafts for construction and operation of the facilities and three diversion structures with sluice gate and stop log controls. Five larger diameter shafts were used for construction access and valve housing, and six smaller shafts consisted of vortex drop and air vent shafts. A 6.1-m (20-ft) diameter Access Shaft AS-03, 7.3-m (24-ft) diameter wet-and dry pit Valve Shaft VS-01, 3.7-m (12-ft) diameter Drop Shaft DS-03, and air vent shaft (AV-03) are located within Milwaukee County Grounds at the tunnel starting area. A 1.8-m (6-ft) diameter temporary shaft (CAS-01) is located at about mid point of tunnel, just north of Capitol Drive and used for ventilation and concrete placement. Two additional access shafts (AS-01 and AS-02), two vortex drop shafts (DS-01 and DS-02), and two air vent shafts (VS-01 and VS-02) are for construction access and diversion of sewer flows, and located at Hampton and Denver Avenue, respectively. The shaft depths range from 40 to 50 m (130 to 165 ft) and were constructed using both dewatering and water-tight construction techniques.

GEOLOGIC SETTING AND GEOTECHNICAL INVESTIGATIONS

The shaft locations and tunnel alignment are within an area containing post-glacial river-valley deposits (silty clay tills, out-wash sand and gravel, and lacustrine silt and clay soils) and glacial deposits overlying the Silurian-age bedrock (Racine and Waukesha dolomite formations). The two principal joint sets are the northwest set

with a mean strike of N49°W and northeast set with a mean strike of N52°E. Most of the joint sets are steeply dipping (generally greater than 60°) and the joint fillings are described as clay-filled, shale-filled, or open. The tunnel intersects primary joints in a perpendicular fashion with exception of a segment north of Appleton Avenue that is aligned parallel to joint sets striking northeast. Most faults observed in the area are relatively narrow and often have widths less than 3 cm (0.1 ft). The resultant vertical displacement is typically no more than 15 cm (0.5 ft).

Bedding planes are often described as thin (less than 0.08 cm or 1/32-inch wide) shale-filled or clay-filled partings. Some of the bedding planes, particularly near the top of rock, appeared to have solution openings which historically had significant impacts on groundwater inflows into area tunnels. The lateral extent of these solution features are generally not known, however, in some cases, they were observed to extend at least 3 to 6 m (10 to 20 ft) beyond the tunnel wall and a few meters to tens of meters along the tunnel wall in the area

During excavation of shafts in overburden, boulders and cobbles were encountered within the glacial and post-glacial till materials. There was no specific reference in number and frequency of boulders encountered during previous excavations in the area, however boulders as large as 6 feet were reported during excavation of the ISS tunnel shafts. In addition to boulders, shaft excavations required managing and controlling excessive groundwater inflows at the soil and rock interface.

Geotechnical Investigation Program

The geotechnical design considerations were based on a review and evaluation of existing information and the supplemental field explorations performed for the project in a phased approach. Phase 1 investigations consisted of the work performed during the preliminary studies in 1999. A total of nine borings were drilled during Phase 1 and laboratory tests were performed on select soil and rock samples. The Phase 2 investigations were performed by Black & Veatch during the detailed design. A total of 26 (24 vertical and two inclined) borings were included in Phase 2. One vertical boring was advanced at each of the proposed structure locations. The other vertical borings were advanced at intermediate locations along the entire tunnel alignment. The average spacing between borings was approximately 412 m (1,350 ft). One inclined boring was advanced in the vicinity of the shear zone and another inclined boring was drilled in an area of shallow rock cover noted during Phase 1 investigations. Piezometers were installed in all Phase 2 borings drilled in rock at the tunnel alignment and within overburden at major structure locations. Water pressure (double-packer) tests were performed in each boring drilled in rock using the Houlsby's five-step test approach. Highly sensitive pressure transducers were installed at the pressurized test intervals for accurate pressure readings. No pump test was planned or performed. Upon completion of Phase 2, the Geotechnical Data Report (GDR) and Geotechnical Baseline Report (GBR) were prepared for bidding and construction.

Rock Support Design Parameters

Rock support considerations for shafts included installation of temporary support systems to provide and maintain stable openings during excavations. The allowable shaft support systems were listed based on material properties, loading and estimated range of groundwater infiltration. The support systems also allowed for installation of the final shaft lining as soon as practical. Rock support considerations for the tunnel were based on an evaluation of in-situ vertical and horizontal stresses. Rock support design parameters were developed based on unconfined compressive strength (UCS), elasticity, and rock quality designation (RQD) values. An evaluation of the rock strength parameters based on field measurements and laboratory test results indicated that overstressing of the intact rock was not likely to occur under typical shaft sinking and tunneling conditions. Other failure modes evaluated were possible stress relief movement along faults or joints and wedge and block formations as a result of orientation and characteristics of discontinuities observed in the rock mass. The GBR included an interpretation of the ground conditions for tunnel excavations and types and quantity of the rock support to be installed.

Groundwater Exfiltration and Infiltration Analysis

The groundwater inflow into the tunnel excavations was estimated based on the hydraulic conductivity or permeability of the surrounding bedrock using Heuer's and Goodman's methods, and was predominantly defined by the secondary hydraulic conductivity resulting from observed discontinuities such as joints, faults, and open bedding planes. The occurrence, type, and extent of solution openings along the discontinuities significantly influence the hydraulic conductivity or groundwater inflows experienced in the area tunnels. The GBR prepared for the project included detailed and extensive discussions and interpretation of the groundwater inflow estimates and analysis. Potential groundwater infiltration impacts were summarized under three categories:

(a) drawdown or depletion of potable water wells in the vicinity of the tunnel alignment;
(b) potential migration of contaminants in groundwater; and
(c) adverse impacts on tunnel excavation and lining operations.

A well record search was performed within approximately 760 m (2,500 ft) of the tunnel alignment and identified several specific water supply wells as potential concern for excessive drawdown or depletion. The project included contingency measures to service these wells in case of excessive drawdown. The specified 30.5-cm (12-in) thick concrete liner, cutoff and contact grouting program behind the liner were intended to significantly reduce groundwater infiltration into and wastewater exfiltration out of tunnel. The entire tunnel is required to have 31.5 liters per second (500 gpm) or less groundwater infiltration upon completion of lining installation and cutoff and contact grouting.

SHAFT CONSTRUCTION

As stated above, various methods were used to excavate shafts on this project. The 7.3-m (24-ft) finished diameter Valve Shaft and the 6.1-m (20-ft) finished diameter Access Shaft at the main site were excavated with pre-drilled soldier piles and lagging and supported in the 16.8 m (55 ft) of glacial overburden. Instead of bracing, the soldier piles were supported by installing the finished concrete lining from the top down in 3.0-m (10-ft) stages as the excavation progressed. The water table was lowered by dewatering wells located around the shafts. The remaining 36.6 m (120 ft) of bedrock was excavated by drill and blast in 3.0-m (10-ft) benches to the tunnel invert. The concrete lining in the rock was also poured from top-down as the shaft excavation advanced.

Secant piles installed by the Gillen Company provided support for the overburden at two vortex drop shaft (3.66 m or 12 ft finish diameter) locations. The overburden was excavated out from inside the secant piles and the permanent concrete lining was constructed. The shafts were then advanced using drill and blast methods in rock to the crown of the tunnel and lined with concrete. The shafts were completed prior to the tunnel excavation and were used for access to the deaeration chamber drill and blast excavations for the diversion facilities.

The construction access shaft located approximately mid way in the tunnel was used for ventilation. Gillen used blind shaft drilling to install a 1.23-m (7-ft) diameter steel can liner for the overburden support. Gillen then excavated the rock with a reverse circulation drill with a 2-m (6.5-ft) diameter head. Gillen also used blind shaft drilling to excavate the three drop shafts and three air vent shafts. The drop shaft at the main site was over-excavated to accommodate the vertical belt conveyor used to transport the tunnel muck to the surface. The 1.22 and 1.52 m (4 and 5 ft) diameter drop shafts were lined using cast in place concrete. The air vent shafts were lined using 91 cm (36 in) diameter pre-stressed concrete cylinder pipe. A 91 cm (36 in) diameter horizontal air recirculation pipe was installed between the drop and air vent shafts at each site.

The project also has six subsurface structures and manholes in soils. The support for these excavations also consisted of soldier piles and lagging. Circular ring beams were used to support three larger shafts. The other three smaller excavations were excavated in rectangular sections and supported with conventional walers and struts for bracing. Excavators and clam buckets were used for the overburden removal with hand excavation using clay spades for trimming near the soldier piles and for lagging installation.

The layout of the main site was unique in that there was no excess excavation needed to conduct the TBM mining operation. The Access Shaft was located at 30.5m (100 ft) away from the Construction Shaft allowing the drill-blast section in between to be used as the TBM tail tunnel. The Valve Shaft is located 16.2 m (53 ft) from the centerline of the tunnel and Access Shaft, and was used as sump and to run down the utilities into tunnel. This left the Access Shaft free of obstructions with the vertical belt

in the drop shaft. The future air vent shaft is located between the Construction and Access Shaft and was used for ventilation of the first half of the tunnel.

TBM DETAILS AND TUNNEL EXCAVATION

The minimum TBM specifications in the design were the following:

(a) ability to probe and grout ahead of the TBM to reduce groundwater inflow into the tunnel;
(b) ability to install roof support/rock reinforcement within 7.6 m (25 ft) of advancing tunnel face;
(c) ability to be backed out of tunnel heading through 10 cm (4-in) steel sets and final lining; and
(d) have torque and thrust consistent with conditions described in the GBR.

The TBM selected for the project was a Robbins Model No. 222-183-3. The 6.77 m (22.2 ft) diameter TBM was rebuilt by J.F. Shea-Kenny JV in Milwaukee. This TBM was first used on the Weller Creek project in 1977 and then used on several other projects; Rodgers Pass, Dallas DART Tunnel, and Fermilab physics tunnel near Chicago. The TBM was retrofitted with eight variable frequency drive (VFD) 261 kW (350 hp) motors and two hydraulic drills for probing and grouting. The VFD motors allowed the TBM operator to vary the cutterhead revolution from zero to 10 revolutions per minute. The cutterhead was equipped with 45 x 43.2-cm (17-in) front-loading cutters. A 91.4 cm (36 in) wide conveyor transported the muck from the head to the back of the trailing gear and filled into Muhlhauser muck cars. The four 1.83 m (6 ft) stroke and 38 cm (15 in) diameter propel cylinders provided up to 15,702 KN (3,530 kips) of thrust for TBM advance. The 76.2 cm (30 in) diameter gripper cylinders could produce 31,426 KN (7,065 kips) of force.

The tunnel excavation started from the AS-03 Access Shaft at the main site on Milwaukee County property. The tail tunnel was blasted out taking the bottom heading first and then blasting the top down. At completion, it was lined and a bulkhead was added turning this section into a deaeration chamber. The starter tunnel was blasted for 99 m (325 ft) using the same bottom heading first then the top shot down. The 9.1 m (30 ft) between the Access and Vent shaft was shot out and used as the sump for the tunnel. A muck pit was shot out in the starter tunnel and a hopper and slope belt were installed to feed the vertical belt. The components of the muck haulage system included a radial stacker belt on the surface, a vertical belt rated at 680 t/hr (750 ton/hr), a 91.4 cm (36 in) slope belt, muck hopper with grizzly, muck trains, and a 91.4 cm (36 in) belt from the cutter head to load the trains. The trains consisted of a 25 t (27.6 ton) Schoma diesel locomotive and eight 15.3 m3 (20 cubic yards) Muhlhauser side dump muck cars. In full production, five muck trains were used. They passed each other on two switches that were installed in the tunnel and the third one pulled behind the TBM. The TBM was installed in the starter tunnel and mined for 610 m (2,000 ft) with eight pieces of trailing gear, then it was stopped and the remaining five pieces of trailing gear were installed along with the traveling switch. Once the full set of trailing gear was installed the remaining 10,879 m (35,692 ft) of tunnel was driven.

The TBM advance rate was influenced by a number of factors including ground conditions, maintenance, probing and preexcavation grouting activities, and shut-downs due to holidays, and averaged approximately 29 m (95 ft) per day, for a total of 367 calendar days and 11,500 m (37,740 ft) of tunnel. The tunnel water (groundwater infiltrating and removed) was also measured during the excavation phase. Figure 2 shows the cumulative tunnel advance and tunnel water inflow.

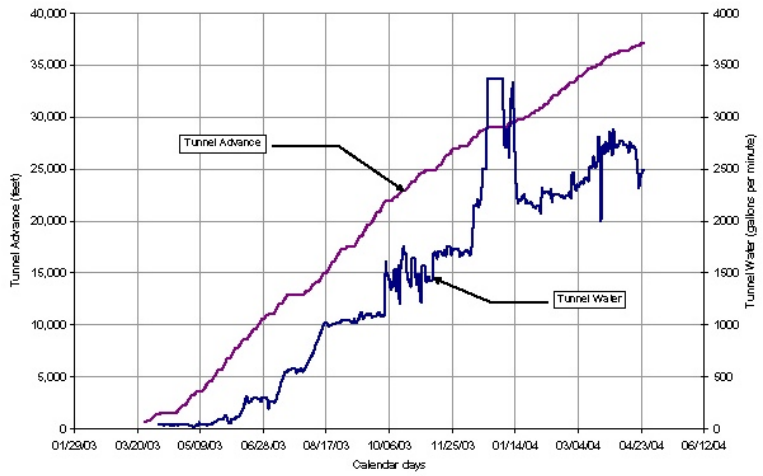

Figure 2. Cumulative Tunnel Advance and Tunnel Water Inflows

PRE-EXCAVATION GROUTING

Certain portions of the tunnel utilized probe holes to determine presence of significant groundwater inflows ahead of the TBM that could impact mining operations. A total of 10 ports were provided through the cutter head for probe and grout hole locations. Figure 3 shows the locations of probe holes at the periphery of the TBM. Two hydraulic drills were used to drill out approximately 4.5 cm (1.75-in) diameter probe and grout holes. In areas designated by the resident engineer and based on an interpretation of the geotechnical reports, two 91.4 m (300 ft) probe holes were advanced into the face of the tunnel. The water inflow was measured from these two probe holes. If the inflow exceeded 1.9 litres per second (30 gallons per minute) for one hole or 2.5 l/sec (40 gpm) between the two, two more holes were drilled to grout the face. With the four holes drilled, packers were installed and the grouting commenced from one of the holes. Initially, a grout mixture of 4:1 water to cement ratio was used. Once there was a communication between holes, all packers were closed. Grouting on the first hole continued until refusal was reached with pressures not to exceed 3,450 kPa (500 psi). If the grouting pressure failed to build up in 20 minutes of pumping, the mix was thickened to a 3:1 mix, then 2:1 and so on down to a 0.5:1 ratio as necessary to reach refusal. Following grouting of the first hole to refusal, the other three holes were hooked up and grouted to refusal as well. Once all four holes were grouted and the grout was given sufficient time to set, two additional verification holes were drilled and the water inflows were measured again. If the water inflow was below the trigger levels, then mining resumed to a 15 m (50 ft) distance from the end of grout holes. Two probe holes were drilled to 91.4 m (300 ft) and the

process was repeated. During the course of pre-excavation grouting, the starting grout mix was subsequently modified to a 1:1 water/cement ratio. A total of 7,700 bags of Type III Portland cement were used for the project for pre-excavation grouting.

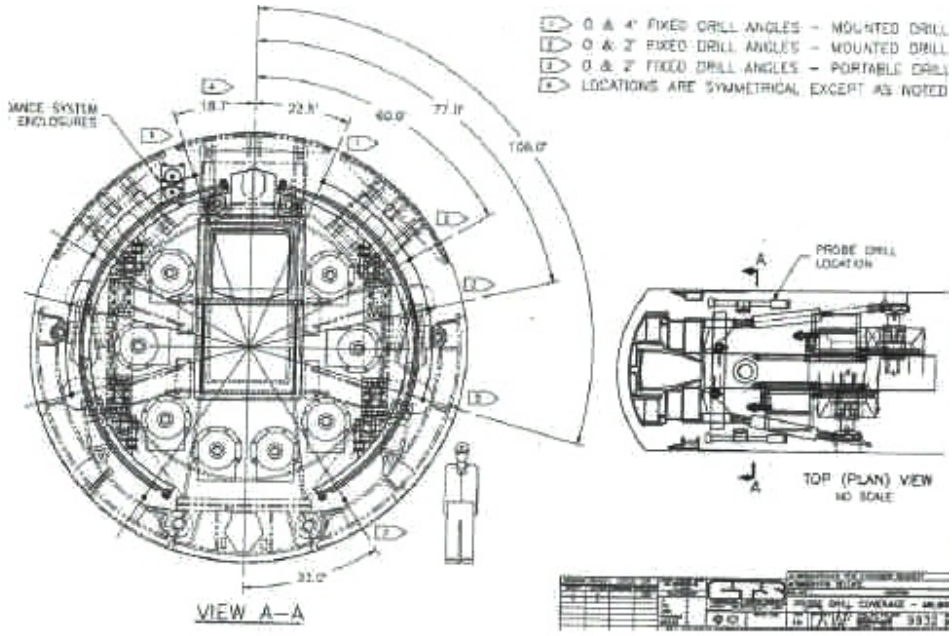

Figure 3. Probe Hole Locations

TUNNEL LINING

The full length of the tunnel is currently lined with a nominal 30.5 cm (12 in) thick concrete. The lining is not reinforced except around the shafts and in areas of low modulus rock that cannot support the pipe under maximum internal pressure conditions. The 27 MPa (4,000 psi) concrete mix with a 0.40 water to cement ratio was specified for the project. The concrete was pumped from the surface with a Putzmeister concrete pump. A 12.7 cm (5 in) slick line was used from the surface, down a drop hole and along the tunnel to a Putzmeister placer car and placed into the forms. The slump for the mix ranged from 21.5 to 27.5 cm (8.5 to 11 in) at the pump depending on the pumping distance. A total of four down holes were drilled along the tunnel alignment at four existing access shafts locations to keep the maximum pumping distance less than 1,097 m (3,600 ft). The forms were designed for full hydrostatic head for monolithic pour. Total length of the forms was 74.7 m (245 ft) and moved in 10.7 m (35 ft) sections. The capacity of the concrete pump was roughly 76.5 m3 per hour (100 cyd/hr) totaling 512 m3 (670 cyd) for a 74.7 m (245 ft) pour. One pour cycle was completed each day. The concrete was poured on day-shift, the tunnel was cleaned out on second shift, and the forms were moved and set on the third shift. Additionally, three crews were engaged in the cut-off and contact grouting behind the lined sections and patching and finishing of the liner.

OTHER DESIGN AND CONSTRUCTION CHALLENGES

Contamination Concerns

Areas of known contamination existed with the potential for contaminant migration to the project area due to an increased hydraulic gradient in soil overburden. The primary concern for contaminated spoils and groundwater management was in areas where excavations were located inside the limits of the Moss- American Superfund site. The Little Menomonee River floodplain along the northern third of the tunnel alignment was considered part of this Superfund site. The mitigation measures consisted of provisions included for management (i.e., handling and treatment) of potentially contaminated soils and groundwater in the contract documents.

Treatment and Discharge of Shaft and Tunnel Water

Due to high volume of groundwater infiltration, the tunnel water generated during construction was discharged to surface waters after treatment. A general Wisconsin Pollution Discharge Elimination System (WPDES) permit was required and obtained from the WDNR. This permit limited the concentration of total suspended solids in the discharge to 30 or 40 parts per million (ppm) depending on flow rate. The contractor installed a settling basin and used lamella plate clarifiers and press filters near the point of discharge at the construction shaft area to meet the stringent effluent permit limits.

Permits

A number of federal, state, and local permits were required for the project, including the U.S. Army Corps of Engineers Section 404 permit for work at surface structure locations within wetlands, the WPDES pit/trench dewatering permit for tunnel construction related discharges to waterways, WDNR Chapter 30 permit, Wisconsin Department of Transportation permit, Milwaukee County permits, cities of Milwaukee and Wauwatosa permits, local and state permit for blasting. Blasting hours were restricted near the Milwaukee County Medical Center and Eye Surgery Institute due to potential adverse vibration impacts.

The design also addressed the tunnel and shaft spoil handling, land acquisition and easements, noise and dust control, erosion control plan, traffic control, power requirements for TBM, pre-construction quality control surveys of structure locations and tunnel alignment, and scheduling and cost control measures.

RISK MANAGEMENT APPROACH

The NWSRS contract documents included the GBR prepared by Black & Veatch. The GBR established a contractual statement quantifying the baseline for the geotechnical conditions anticipated to be encountered during construction. A separate geotechnical data report (GDR) was provided to the bidders with all data on which both the project design and the GBR were founded. The use of this contract approach required modification of several key clauses in the front end of the MMSD's standard construction contract. The most notable changes were to clauses for handling differing

site conditions, installation of a Disputes Review Board (DRB) and continuation of the work during a dispute. The DRB Board members were mutually selected and included James W. Mahar, Ph.D., Anthony Stewart, P.E., and Edward S. Plotkin, P.E.

The MMSD required escrowing a copy of the bidder's bid documents. This contract requirement was based on the concept that cost sharing on a particular dispute could be settled more fairly if the contractor's prebid understanding, evaluation, and actual bid tabulation were available.

Design Review Board

It is Black & Veatch's practice to employ an independent tunneling Design Review Board of three to four experts with specialized expertise in the areas of rock mechanics, subsurface investigations, underground construction, tunnel boring machine, cost estimates, and significant sewer design to review major project elements at critical stages. The Black & Veatch Design Review Board members for the NWSRS project were P.E. (Joe) Sperry, Roger Ilsley, Stan Hattrup, and John Schilling.

MMSD's Board of Review

The MMSD engaged an independent Board of Review to examine and comment on major work products. The Board of Review members consisted of Andrew Merritt, P.E., Ph.D., Skip Hendron, Ph.D., and Ron Heuer, P.E., Ph.D. Additionally, MMSD staff provided valuable comments for the work products submitted based on their past experience in the area tunnels and contracting practices. Periodic meetings and timely input and feedback helped tremendously to continue the work in an efficient manner.

SUCCESS STORY

The NWSRS project had significant design concerns regarding excessive groundwater inflow into tunnel excavations, potential impacts on local and groundwater resources and potable water wells, blasting related vibrations on nearby sensitive structures, and handling of contaminated soil and groundwater. Additionally, potential presence of geologic discontinuities, specifically solution openings that could complicate the construction were among chief concerns considered during the design process. The plans, specifications, and GBR and GDR included appropriate measures to address these potential adverse conditions and the bid documents included fair and equitable compensation. The bids were very closely spaced and three of the bids received were within less than one percent of each other. The construction management (CM) team and the contractor have been pleased with the contract documents, specifically with the GDR and GBR. The contractor's experience in regional tunnels and willingness to partner and work with the CM team and the MMSD has turned this project into one true "success story." The facilities also offer a great operational flexibility to the MMSD and the citizens of Milwaukee and six of the surrounding communities to further maximize the project benefits.

The NWSRS tunnel project was awarded for construction in December 2001. The construction began in April 2002. Tunnel excavation started approximately one year later in April 2003 and was successfully completed in April 2004, safely and without a significant delay or claim. The facility was placed in service in late 2005.

ACKNOWLEDGMENTS

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LARGE SPAN TUNNEL CONSTRUCTION WITH LOW COVER: EAGLE'S NEST TUNNEL, HONG KONG

Tristan Green[1], Angus Maxwell[2] and Dale Pugh[3]

Abstract

The Eagle's Nest Tunnel, located between Cheung Sha Wan and Shatin Heights, forms part of the Route 8 Highway within Hong Kong. The tunnel comprises twin bores up to 18.5 m in span separated by some 20 m of rock. The tunnels are situated close to and sub-parallel to the Tolo Channel Fault which is of regional significance and close to water service reservoirs and slopes with history of movement. The southern portals were known to have limited cover and in some places soil conditions were expected less than 10 m above the tunnel crown. The paper describes the design of the temporary support measures in the southern section and describes how these were installed as the ground was excavated. Up to 35 mm of crown movement was predicted in the design worst case resulting in approximately 10 mm of surface settlement directly over the tunnel. Extensive instrumentation was installed and the results were monitored. Up to 10 mm of crown movement was observed in the general South Portal area with 35 mm of crown movement observed where the rock head was lowest. This was within 25% of that predicted by design. In this area, very localised low points had been infilled with completely decomposed rock backfill which was exposed in the crown during the excavation. Pipe pile canopies were installed in the portal area and advances in weak ground were limited to allow timely installation of steel arches and fibre-reinforced shotcrete adapted to meet the localised conditions. While movement of the tunnel crown in the low cover area was quickly arrested, the loss of water into the excavation resulted in some subsequent consolidation of poorly consolidated/compacted saturated superficial materials.

INTRODUCTION

The South Portal of the Eagle's Nest Tunnel in Hong Kong is situated approximately 50 m to the south of the Tolo Channel Fault, immediately downstream from the Kowloon Reservoirs (Figure 1). The project requires two parallel uni-directional road tunnels with three lanes of traffic in each direction. Due to the horizontal curve in the road alignment, the tunnel profile is widened to 19 m at the south portal to allow sufficient line of sight for inbound and outbound traffic.

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[2] Director, Maxwell Geosystems Ltd
[3] Senior Site Agent, Leighton/Kumagai Joint Venture

GROUND CONDITIONS

The site investigation revealed that the level of the rockhead was variable along the southbound tunnel at the South Portal (Figure 2); ranging from 10 to 22 m below the existing ground surface. In relation to the tunnel alignment this meant that the rockhead was typically less than 10 m above the tunnel crown within the first 140 m of the southbound tunnel alignment.

The regional structural geology indicated that an inferred fault was likely to run parallel to the tunnel alignment starting from a location intersecting the portal excavations. The presence of this fault was likely to impact on the extent of the highly decomposed rock mass and further deepen the zone of weathered material and locally reduce the level of rockhead within this area. In addition, a feature of this significance, when combined with the limited depth of cover close to the portal, was likely to have an adverse impact on the development of a natural arch for the full tunnel span. The tunnel support system had to be developed to consider this special case geology at the South Portal location.

In the event, this lineament was revealed on site to consist of a rhyolite dyke with sheared contact margins to the granite that ran sub-parallel with the alignment and intersected both the southbound and northbound tunnel drives as they progressed away from the south portal. Fortunately, the presence of this dyke allowed the excavation to proceed without excessive additional support needing to be installed.

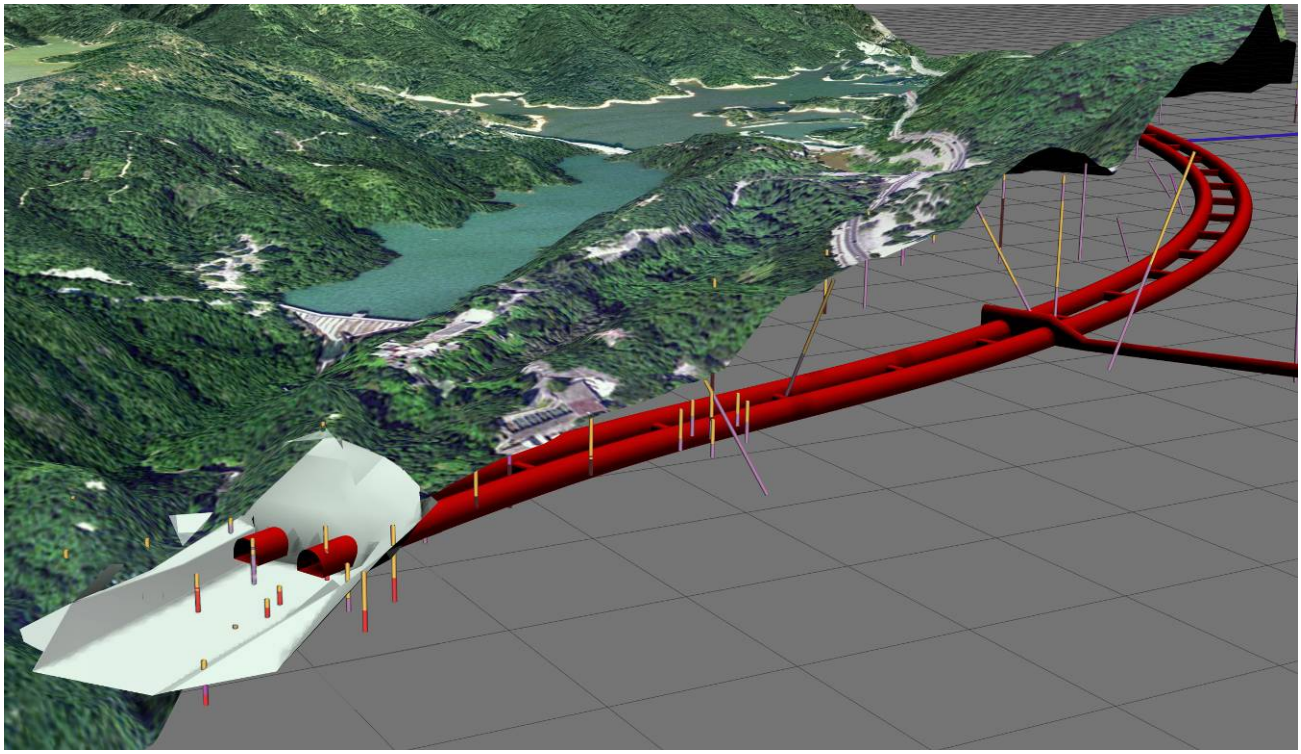

Figure 1. Alignment of the Eagle's Nest Tunnel viewed from the southern end

South Portal Ground Conditions

A detailed ground survey was undertaken for the project that revealed a deeply incised stream course near the south portal with a stream bed level of between +87.0 and +88.0 mPD (Figures 2, 3 and 4). With a ground level of approximately +87.0mPD in the bottom of the stream course this left a depth of cover of only 9 m to the crown of the 19 m span excavation for the widened tunnel section. Combined with the low ground cover, the nature of the stream bed material and the ground type through which the stream ran were also important factors. It was evident that the original stream course had been modified by the placement of a quantity of fill

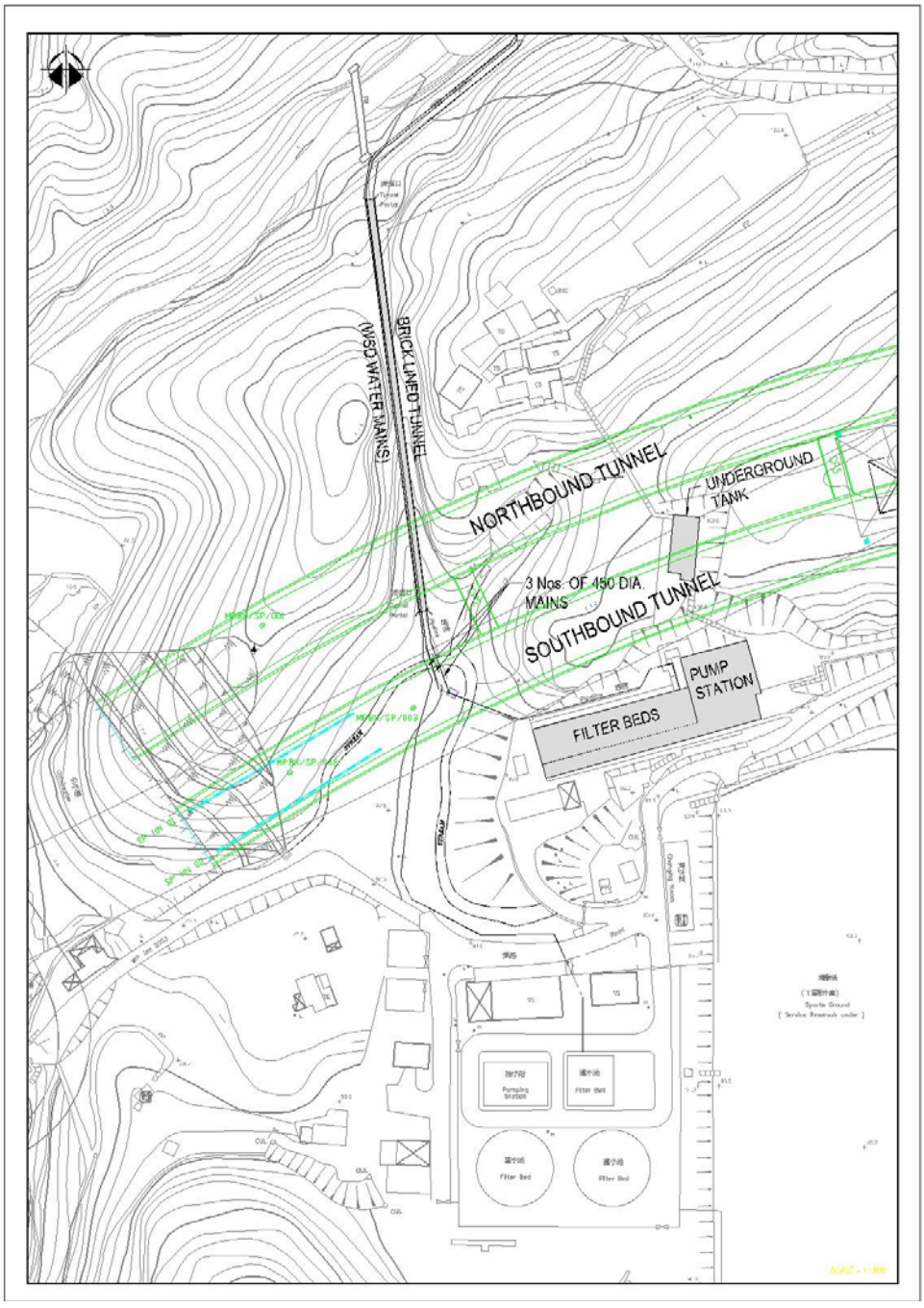

Figure 2. General Layout at the Southern Portal

material (completely decomposed granite) which was used to fill the old valley floor. The stream had been trained to the side of the original valley and the central part of the valley was now filled to a level higher than that of the present stream course.

The original stream bed had been identified from the surface site investigation drill holes in this area and it appeared that little, if any, of the original alluvial deposits in the stream bed had been removed before the fill was placed within the valley.

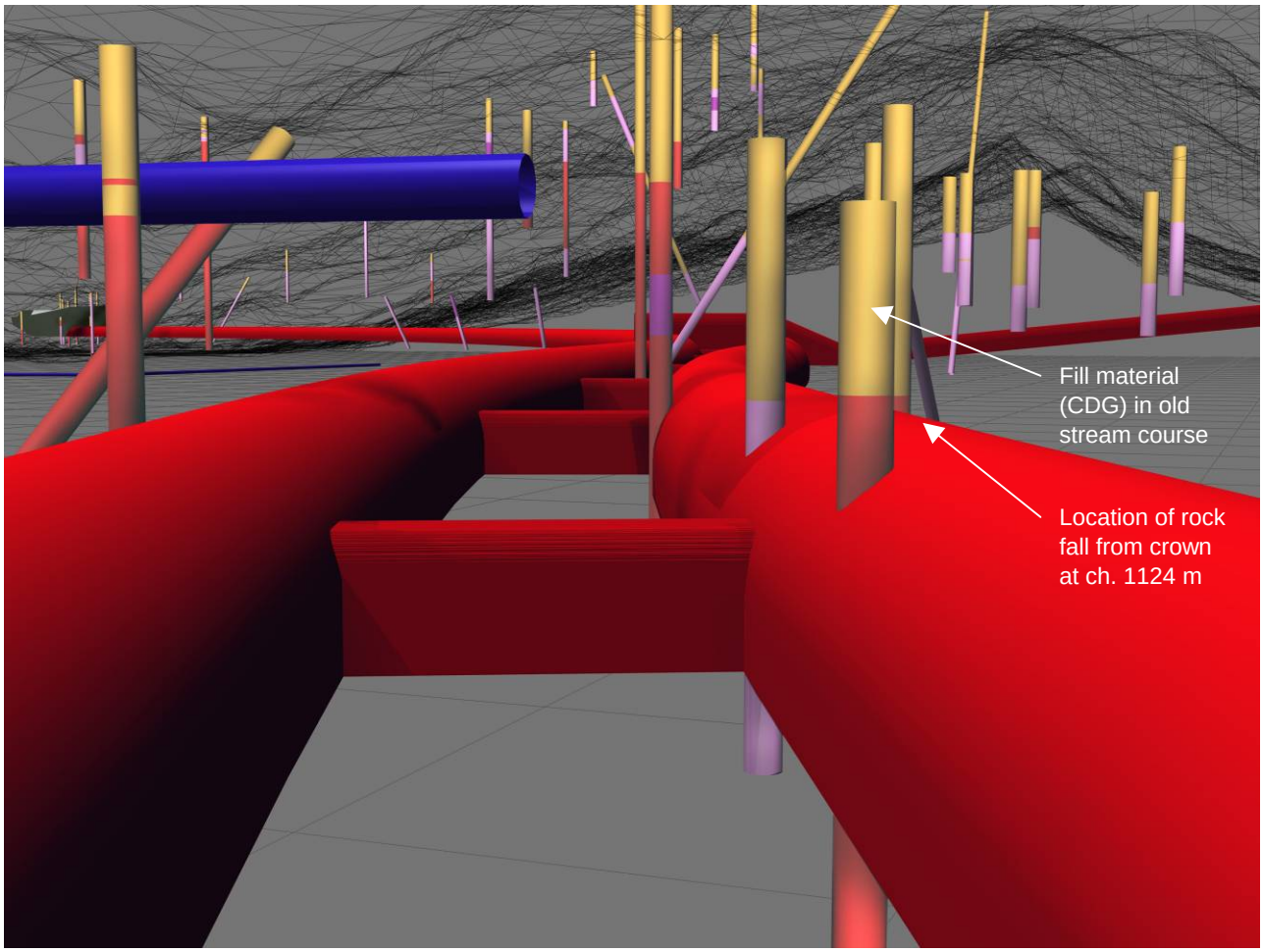

Figure 3. Tunnel alignment at the South Portal below the filled stream course

Additional Investigations at the South Portal

The limited cover above the southbound tunnel drive and the potential for ground movement was recognised at the design stage and two horizontally installed inclinometers, SP_HN_01 and SP_HN_02 were specified, located above each shoulder of the excavation and extending for 40 m along the tunnel drive from the portal slope. A simplified section is shown in Figure 4. The horizontal inclinometers were installed to a depth of 40 m to measure vertical deflections along the holes.

Prior to commencement of excavation an array of three 40 m long probe holes were drilled from the portal slope face to check for poor ground and assist in draining the initial section of tunnel.

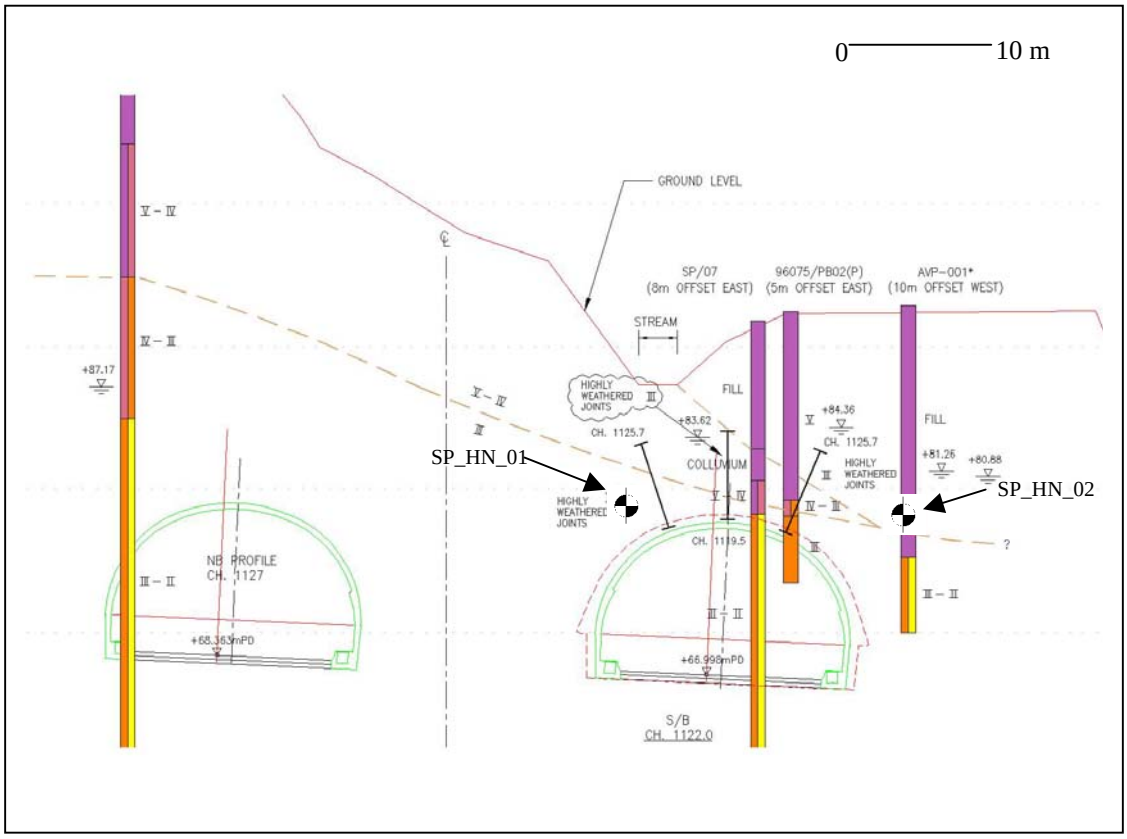

Figure 4. Simplified section showing the geological units in the zone of low cover

The holes were positioned with one on the centreline at the crown and one at each side of the tunnel at axis level. Water outflow was measured at intervals throughout the drilling of the holes and the responses of adjacent piezometers were monitored. A gradual reduction in water outflow from these holes was observed over time as the water table was locally lowered and water pressure reduced above this section of the southbound tunnel drive. Probe hole arrays were repeated every 20 m throughout the initial 100 m of tunnel drive.

TEMPORARY SUPPORT DESIGN AND EXCAVATION METHODS

The geological model and detailed land survey revealed that the initial excavation in the southbound tunnel drive would encounter a very low overburden depth compared to tunnel span (approximately half-span cover to the crown) and likely rockhead at or even below the crown level. This high risk section was therefore assessed using an FEM analysis (PLAXIS).

The temporary support system was developed by the construction and the design teams to achieve the required lining capacity determined by the FEM modelling.

The temporary support system included the following elements:

(a) excavation in heading and bench;
(b) pipe pile canopy installed in advance of the heading excavation with 76 mm diameter pipes which were fully grouted;
(c) in sections of tunnel with moderate or high water inflows from the crown the pipe pile canopy was preceded by a round of grouting in an overlapping fan arrangement to allow for the successful installation of the pipe pile canopy without bringing in excessive water and destabilising the tunnel roof;
(d) heading advance was initially limited to 1.0 m and gradually lengthened to 3.0 m;
(e) primary support was achieved with 4-part steel arches that were assembled in situ;
(f) arches were then encased in fibre-reinforced shotcrete which was extended to fill the space between adjacent arches forming a composite steel-shotcrete arch lining;
(g) rock bolts were installed between the steel arches within the crown where rock was present of a suitable quality to ensure the bolts had sufficient anchorage capacity; and
(h) bench excavation followed behind and side walls were stabilised with rock bolts and fibre-reinforced shotcrete.

INSTRUMENTATION SETUP

Figures 2 and 5 show the arrangement of instruments installed on the project. Surface instruments (including convergence targets within the tunnel and ground surface settlement markers) are represented as circles; instruments in boreholes are represented by squares. A summary of these instruments is given in Table 1. Instruments referred to in the text have been labelled.

CONSTRUCTION PROGRESS AND OBSERVATIONS

Immediately from the portal slope the top heading face of the southbound tunnel drive was dominantly composed of moderately to slightly decomposed granite with minor amounts of more highly decomposed granite confined to discrete joint planes and associated with near vertical shear zones that ran sub-parallel with the tunnel. However, the rockhead-soil interface was found to be only a few metres above the crown level and potential for instability was considered high.

Prior to any excavation works a full canopy of 12 m long pipe piles was drilled and grouted across 120 degrees over the crown of the tunnel to reduce the risk of major roof instability during the portal excavation. Subsequent rounds of overlapping canopies were installed using either 6 m long grouted pipe piles or spile bars, selected on-site depending on the ground conditions revealed by face mapping and results of probing. Given the hard rock conditions in the face and risk of instability in the crown drill and blast tunnelling commenced from the South Portal with initial advances

limited to 1 m. Each blasting round was divided into a pilot blast followed by a ring stripping blast developed to allow blasting to take place within strict vibration and air-blast limitations imposed by the proximity to WSD structures above and adjacent to the portal site.

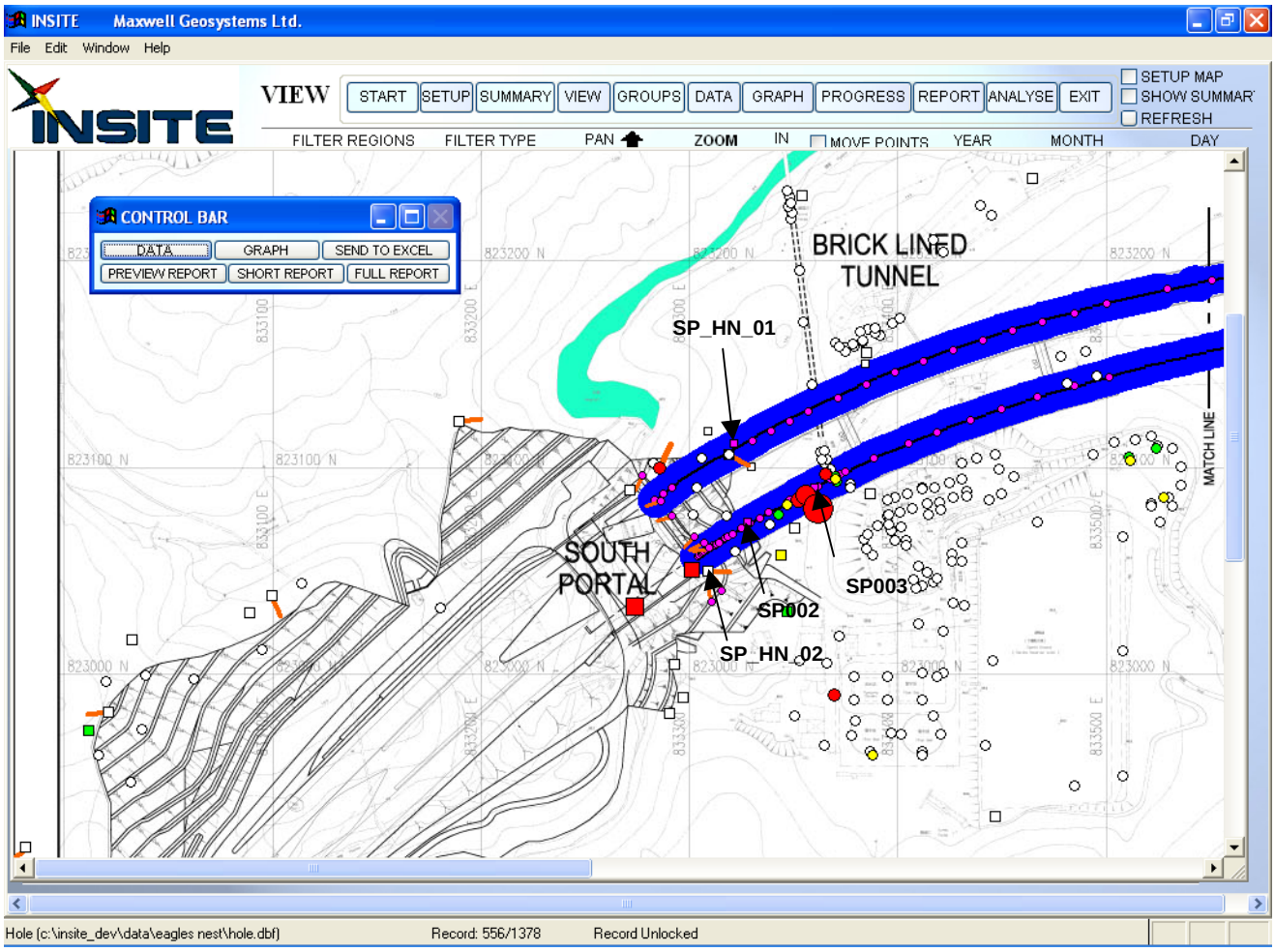

Figure 5. Layout Plan Showing Arrangement of Instrumentation at the South Portal

Temporary support, comprising a composite steel rib and fibre-reinforced composite arch, was installed after every round of excavation to limit ground movements.

A minor rock fall from the crown was experienced after approximately 60 m of tunnelling of the southbound heading as shown in Figure 6. This rock fall occurred immediately after the blast and prior to the commencement of mucking out at chainage 1024 m exposing the rockhead and revealing the overlying stream bed deposits and fill materials. This showed the extent to which the original valley had been filled and demonstrated the highly irregular profile of the rockhead in this area. This rock fall allowed the overlying material (generally unconsolidated fill of completely decomposed granite) to unravel and a small area of overbreak was formed before the ground could be successfully stabilised with shotcrete.

Table 1. Summary of South Portal instrumentation.

Parameter to be Measured	Instrumentation Installed
Vibration	Vibrographs at sensitive receivers
Convergence within the tunnel	Prisms in 7 point convergence network monitored by total station
Vertical movement within the ground profile	Magnetic extensometers (3 nos.) Horizontal Inclinometer (1 no. near left and right shoulder)
Movement of portal slopes	3D monitoring points
Total water inflow into the tunnel	V-notch weir measurements
Water drawdown in compressible deposits above the tunnel.	Piezometers (casagrande and vibrating wire types)
Ground and Structure Settlement	Levelling

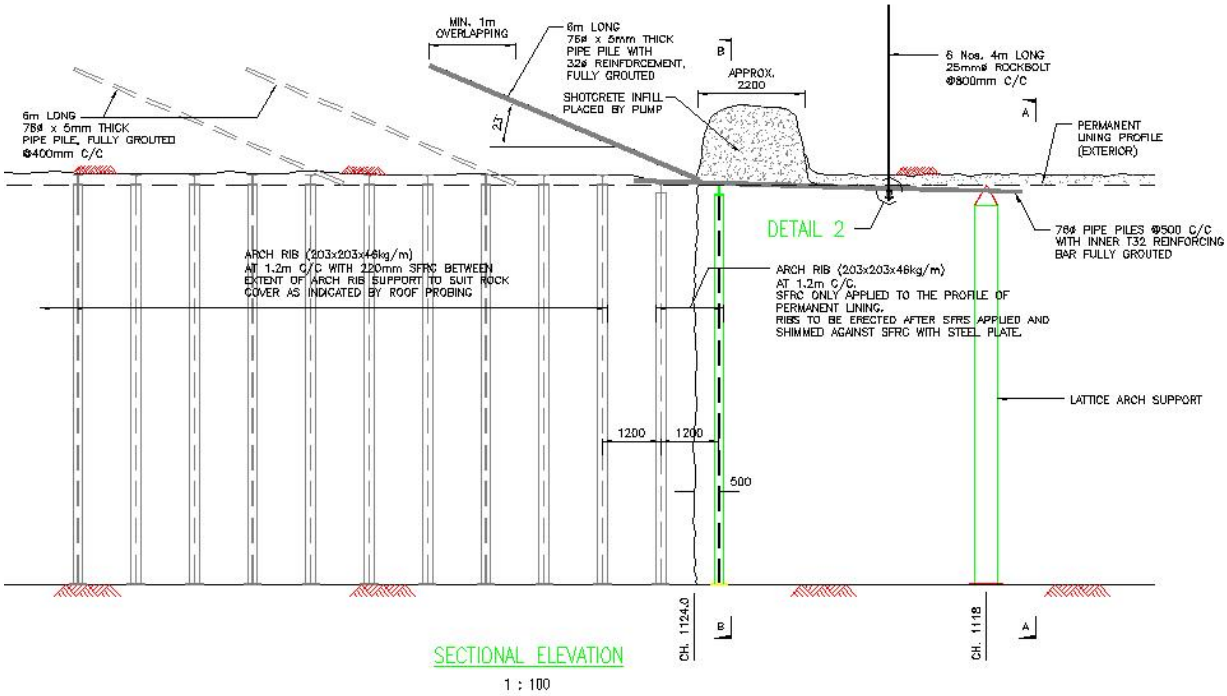

Figure 6. Arrangement of Support Elements in the Zone of Low Cover and Rockfall

The stream bed and alluvial deposits were subsequently exposed in the tunnel crown and upper part of the face during the excavation of further rounds between Chainages 1127 m and 1136 m. In total this poor ground extended for approximately 14 m of the southbound tunnel drive and required the installation of a substantial temporary support system incorporating 15 m long grout fans in advance of 6 m long fully grouted pipe piles at 250 mm centres in fans with a 3 m overlap, steel arch ribs at 1.0 m centres, 5 m long rock bolts on a 1.0 by 1.5 m grid, and up to 250 mm thick fibre reinforced shotcrete.

It was observed that there were layers of clays, loose sands and gravels with vegetation present immediately above the rock surface and below the overlying fill material. It is believed that these unconsolidated materials were those deposits that were laid down in the original stream bed. It was evident that there was no removal of these alluvial deposits prior to the placement of the fill in this area. The presence of these layers of sand and gravel provided an easy flow path for water to migrate from the present day stream course to the top of rockhead.

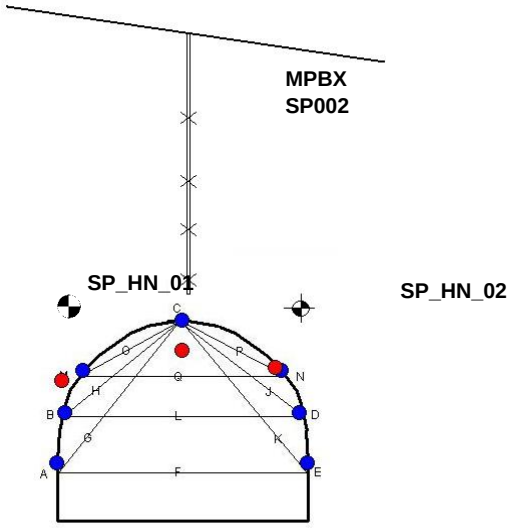

Figure 7. General arrangement of instruments for convergence monitoring

ANALYSIS

Convergence of the tunnel perimeter was measured using two horizontal inclinometers positioned over the tunnel shoulders, two vertical magnetic extensometer arrays located above the tunnel crown, and convergence measurements using optical survey, as shown in Figure 7. The end of the inclinometer hole SP_HN_02 was at Chainage 1101 m; some 37 m from the tunnel portal at Chainage 1064 m. The inclinometer data is shown in Figures 8a and 8b where measured vertical displacement is plotted relative to the end of the holes at Chainage 1101 m i.e.; to measure the progressive strain as the tunnel face approaches and passes the position of the instrument. The plots show the movement on the inclinometer at Chainages 1078 m and 1083 m. The relative position of the face to this point is given by the thick solid line such that the face passed the inclinometer position at A. The inclinometers show total vertical movements in the right crown areas of between 10 and 15 mm which ceased within 25 m of the tunnel face.

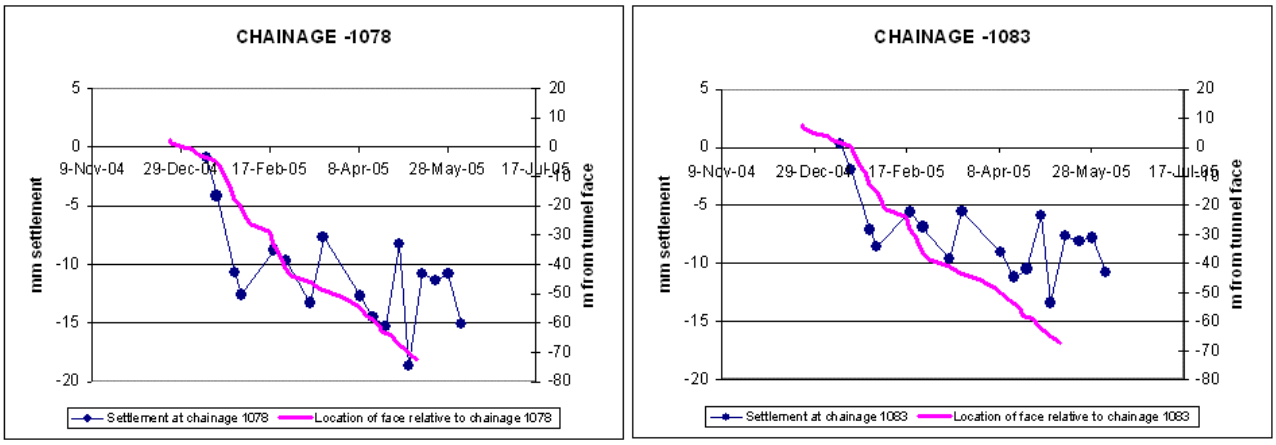

Figure 8a and 8b. Vertical Movement as Recorded in the Horizontal Inclinometers at Chainages 1078 and 1083 as the Tunnel Face Passes

Magnetic extensometer SP002 at chainage 1093 m was installed with 11 magnets at 1.0 m intervals between 2 m below ground level and approximately 1 m above tunnel crown level.

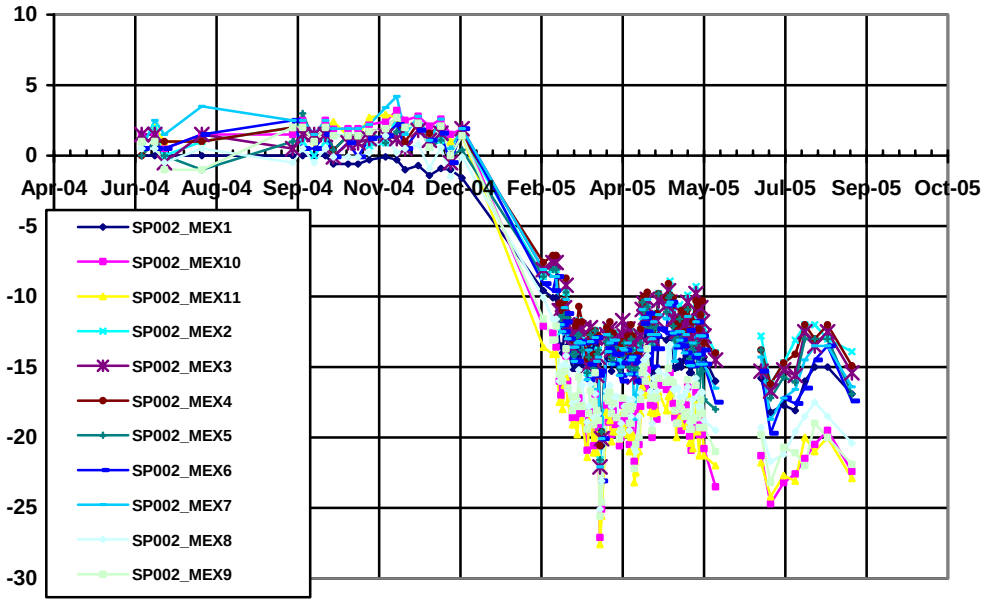

Figure 9. Magnetic Extensometer SP002 at Chainage 1093 m

The magnetic extensometer showed a total ground settlement and bodily movement of the extensometer of 16 mm with a further 5 mm of movement recorded in the last four magnets. The movement of the last four magnets in rock indicates that localised rock strains contributed 5 mm of the total movement as the rock expanded towards the void. The remaining 15 mm of downward movement represents deflection of the rock/soil arch. Convergence monitoring at Chainage 1084 m is shown in Figure 10 below. The final converged shape after heading excavation only reveals some 5 mm of downward crown movement, 1 mm of inward movement of the right shoulder and a 3mm outward movement of the left shoulder.

There is considerable uncertainty about the interpretation of the convergence results. Due to the necessity for the installation of relatively heavy temporary support in these sections twenty further metres were excavated before the section was first monitored for convergence. Survey prisms or extensometer hooks would have been destroyed by the blasts had they been installed earlier. Additionally, rather than monitoring ground movements the convergence array actually recorded movements of the temporary support system. At this location the temporary support system comprised an initial sealing layer of shotcrete, followed by the erection of the steel arch ribs and finally fibre-reinforced shotcrete encasement of the steel ribs. Shotcrete was sprayed with a Meyco Suprema shotcrete machine and it was virtually impossible to ensure that any air gaps between the steel ribs and the initial shotcrete were completely filled for the whole perimeter of the arch. With this in mind, the outward

movement of the left hand shoulder most probably represents the hogging of the steel arch as it took up load in response to the crown settlement.

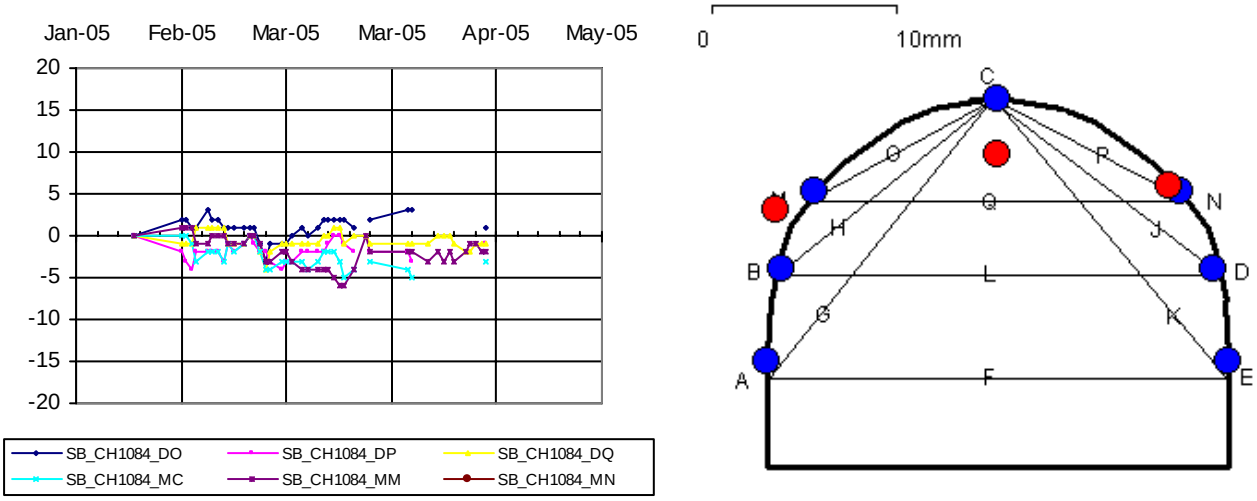

Figure 10. Convergence Monitoring at Chainage 1084

The chart on the left shows the change in level or length of each measured point or distance. The diagram on the right shows the deformed shape.

Design Predictions

Design of temporary supports in the low cover zone was carried out for two numerical models constructed using Plaxis 3D. Two materials were modelled: Intact slightly/moderately decomposed granite SDG/MDG and completely decomposed rock CDG forming the soil layer. The parameters adopted are shown in Table 2.

Table 2. Summary of Design Parameters

Material Type	E MPa	ν	C (kPa)	ϕ (deg)	γ_t (kN/m^3)	K_0
CDG	5+2*depth	0.25	2	38	19	0.5
SDG/MDG	1000	0.2	50	45	26	2.0

Two models were investigated each with only 1 m of rock cover but one having 15 to 18 m of total overburden and the latter 25 m of overburden. The planned sequence of excavation was followed within the modelling and various structural supports were installed as detailed in Table 3.

The 3D model focussed on a 10 metre section of the tunnel in the region where rock cover was expected to be of the order of 1 m only. In this zone the installation and gradual strengthening of the support elements was investigated. Soil and rock were modelled for some 20 m in front of and behind this zone to provide restraint in the longitudinal direction. The segment of excavated tunnel was supported purely by the arch ribs and fibre reinforced shotcrete at 120 hour strength.

Table 3. Structural support types

	Axial Stiffness EA (kN/m)	Flexural Stiffness, EI (kNm^2/m)	w (kN/m^2)	ν
Design Case 1:				
Pipe Pile OD114.3mm-6.3mm@450 mm cc	0.48e6	0.7e3	0.2	0.15
Arch Ribs[1] with 200 mm FRS (12hrs)	3.4e6	17.0e3	5.3	0.15
Arch Ribs[1] with 200 mm FRS (36hrs)	4.4e6	20.3e3	5.3	0.15
Arch Ribs[1] with 200 mm FRS (60hrs)	5.1e6	22.6e3	5.3	0.15
Arch Ribs[1] with 200 mm FRS (84hrs)	5.6e6	24.1e3	5.3	0.15
Arch Ribs[1] with 200 mm FRS (5 days)	5.9e6	25.3e3	5.3	0.15
Design Case 1:				
Pipe Pile OD114.3mm-6.3mm@450 mm cc	0.48e6	0.7e3	0.2	0.15
Arch Ribs[2] with 250 mm FRS (12hrs)	4.7e6	38.1e3	5.3	0.15
Arch Ribs[2] with 250 mm FRS (36hrs)	5.9e6	44.5e3	5.3	0.15
Arch Ribs[2] with 250 mm FRS (60hrs)	6.8e6	49.0e3	5.3	0.15
Arch Ribs[2] with 250 mm FRS (84hrs)	7.3e6	51.9e3	5.3	0.15
Arch Ribs[2] with 250 mm FRS (5 days)	7.8e6	54.3e3	5.3	0.15

FRS = Fibre Reinforced Shotcrete, 1. Arch ribs= UC 203*203*46kg/m at 1 m centres. 2. Arch ribs= UC 254*254*73kg/m at 1 m centres

It was assumed that there would be no groundwater pressure in the model since drainage close to the portal should have drawn the local ground water table to invert level. Piezometers adjacent to the tunnels revealed groundwater table within the rock at some 8 m above the invert and a perched water table within the superficial deposits.

Comparison with Design.

Table 4 below summarises the predicted movements at the locations measured by the instrumentation programme. Table 5 summarises the measured movement using the three measurement methods.

Measurements

The measurements recorded by the instruments showed generally good agreement with that predicted by design. Convergence monitoring from within the tunnel was only able to record 50% of the deformation at best. It was possible to record the full ground movements using both the horizontal inclinometers and the vertical magnetic extensometers.

Table 4. Predicted Movement from Design Models

	Rock Cover	Total Cover	Vertical Movement	Location
Design Case 1:	1 m	18 m	-16 mm	Shoulder
	1 m	18 m	-24 mm	Crown
Design Case 2:	1 m	25 m	-10 mm	Shoulder
	1 m	25 m	-36 mm	Crown

Table 5. Actual Movement from Instrumentation

	Rock Cover	Total Cover	Vertical Movement	Location	Variance from Design
Observed Inclinometer deflection at Chainage 1078 m	1 m	10 m	-15 mm	Shoulder	-3%
Observed Inclinometer deflection at Chainage 1083 m	2 m	15 m	-10 mm	Shoulder	-30%
Observed Vertical Movement in Extensometer SP002 at Chainage 1094 m	1 m	13 m	-23 mm	Crown	-2%
Observed Vertical Movement in Extensometer SP003 at Chainage 1129 m	1 m	13 m	-35 mm	Crown	+50%
Convergence monitoring at Chainage 1083 m	2 m	15 m	-4 mm	Crown	-75%

Monitoring in the rock fall zone

Magnetic extensometers and surface settlement points around Chainage 1120 m on the southbound tunnel recorded significant settlements related to the minor rock fall on 28 March 2005.

The extensometer movement was referenced to the top magnet to measure crown movement. However, survey revealed that the reference magnet had moved by some 80 mm. Analysis of the extensometer data revealed that the surface settlement could be divided into some 35 mm of crown movement (Part A in Figure 11) with an additional 45 mm of settlement within the superficial deposits which comprised unconsolidated fill (Part B in Figure 11). The form of the settlement profile in the fill and the settlement period is consistent with fill consolidation due to underdrainage.

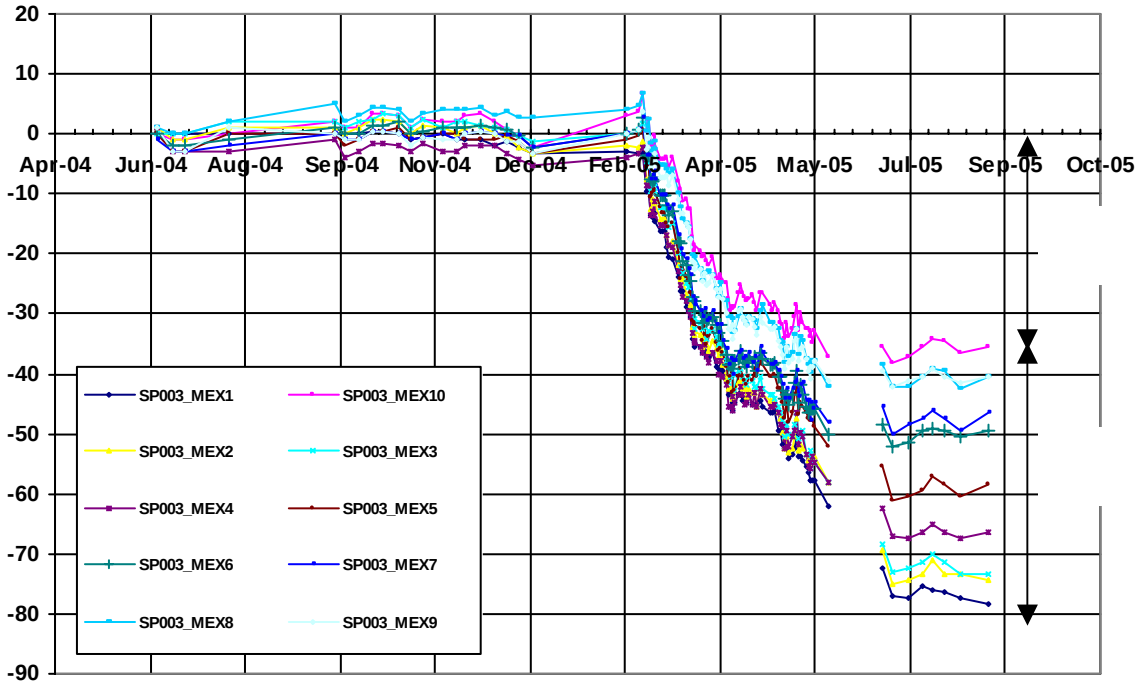

Figure 11. Magnetic Extensometer SP003 Readings at Chainage 1129 m

CONCLUSIONS AND RECOMMENDATION FOR FUTURE SIMILAR CONSTRUCTIONS

This paper has shown that in favourable rock conditions large span tunnelling can be accomplished with very low cover provided that the tunnel can be drained and that effective pre-support is utilised. Monitoring of the excavations showed movements which were in line with design assumptions.

The analysis of multipoint borehole extensometers needs careful consideration when used in the tunnelling setting where neither the top nor the base of the instrument is referenced to an immovable reference point. The absolute movement of the lowest spider is of the greatest concern during the initial excavation phase as this gives an indication of the convergence of the ground and deformation of the excavation and lining. However, as observed in this project, the upper spiders experienced a greater magnitude of movement due to the longer term consolidation of the overburden. It is important that the reference level of the top of the instrument is regularly monitored to convert the relative movement of the spiders into absolute movement where both the upper and lower parts of the instrument are expected to experience movement.

The use of systematic arrays of convergence targets when using a drill and blast excavation method needs thorough research before it is specified in hard rock tunnels. From experience on this project there is little chance of a complete array being installed close enough to the face and lasting long enough to produce enough

meaningful readings before the array is partially or completely damaged by fly rock from blasting, disturbed by mechanical means during removal of muck and scaling operations, or obscured by shotcrete for temporary support or smoothing purposes. At best the convergence can confirm that movements have ceased but is ineffective in monitoring tunnel movements against design estimates.

ACKNOWLEDGEMENTS

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DESIGN AND CONSTRUCTION OF NO-DIG CABLE DUCT CROSSINGS UNDERNEATH A VEHICULAR BRIDGE ABUTMENT AND A PILE SUPPORTED DECKED NULLAH

W.K. Lau[1] and H.W. Leung[2]

Abstract

CLP Power Hong Kong Limited (CLPP) is presently reinforcing the 132kV power transmission network to ensure continuing supply security for our customers. The network reinforcement comprises, inter-alia, the installation of no-dig cable duct crossings for laying the new 132 kV cables under heavily trafficked highways and existing buried structures. CLPP has recently completed two groups of twin cable duct crossings at Kwai Chung Road and Canton Road. These 1.2 m diameter cable duct crossings were installed underneath the pile supported decked nullah at Kwai Chung Road, and the northern abutment of Kowloon Park Drive Flyover at Canton Road. The difficult site conditions and adverse engineering constraints necessitated engineering feasibility studies and unconventional design of these cable duct crossings. With proper project planning, project management and stakeholder management, CLPP successfully implemented the construction work with minimum disturbance to the public and the environment. This paper describes the planning, design, construction and project management aspects of the successful installation of 1.2 m diameter cable duct crossings up to 80m long underneath the northern abutment footing of Kowloon Park Drive Flyover and in between the driven pile foundations of the road decking of Kwai Chung Nullah.

INTRODUCTION

Project Needs and Site Constraints

CLPP has been striving to improve the supply reliability to our customers to meet their increasing demands. To meet this increasing demand it was identified that new 132kV cable circuits in Kwai Chung and Tsim Sha Tsui would be required in the early 2000's. At the cable route selection stage, various options have been studied to enable the project to be completed at the lowest possible cost while minimising disturbance to the environment and to the public. The optimum routes of these cable circuits were subsequently identified as involving no-dig cable crossings underneath the heavily trafficked Canton and Kwai Chung Roads. The design and construction of these cable duct crossings had to take into account the special engineering and site constraints imposed by the Kowloon Park Drive Flyover at Canton Road and the Kwai Chung Decked Nullah under Kwai Chung Road. CLPP then formed a project team to conduct the feasibility studies of these two no-dig cable duct crossing projects. The project team identified that the design and construction of these cable

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duct crossings was technically feasible. However, an appropriate no-dig construction technique had to be adopted and the project required a conscientious and committed team.

This paper describes the organizations of the respective project teams and the contract arrangement for the execution of the two no-dig construction projects. The paper also highlights the planning, design, and construction supervision aspects that have been taken into consideration to fulfil the statutory requirements.

PROJECT TEAM ORGANIZATIONS AND CONTRACT ARRANGEMENT OF THE NO-DIG CONSTRUCTION PROJECTS

The planning, design and implementation of the no-dig construction projects was under the project management of CLPP. CLPP appointed a consultant to complete the feasibility study of the construction of the two cable duct crossings at Canton Road and Kwai Chung Road. The orders for the two no-dig construction projects were then issued to the Term Contractor under an Outline Agreement. Under the Agreement, the Contractor undertook the design and construction of the required no-dig cable duct crossings commissioned by CLPP. The Contractor appointed Black & Veatch Hong Kong Limited, the Consultant who conducted the feasibility study, as the Designer. Another consultant firm was appointed as the Design Checker for the detailed design of the temporary and permanent works and independent checking of the detailed design submissions respectively. CLPP appointed the Responsible Officers to administrate and supervise the two no-dig construction projects.

PLANNING, DESIGN, CONSTRUCTION AND PROJECT MANAGEMENT OF NO-DIG CABLE DUCT CROSSINGS AT CANTON ROAD

Planning and Feasibility Study

The new 132kV circuits coming from the direction of Yaumatei needed to cross Canton Road to enter into Canton Road Substation. The high daily traffic volume at Canton Road and the presence of existing flyover abutment structures, together with large diameter underground drainage pipes and water mains necessitated a technical feasibility study of the proposed no-dig cable duct crossings at Canton Road. As the crossings were situated at a critical location, ducts for the 11kV cable was also added to the no-dig work, and the similar requirement was included in the Kwai Chung Road crossings. The study needed to identify the possible site and engineering contraints, recommend the conceptual design and the preferred construction method. Hui, Hui & Lau (2002) compared different no-dig methods available in Hong Kong as shown in Table 1. After a comprehensive appraisal of various no-dig methods and taking into account the identified site and engineering constraints, pipe jacking was recommended for the installation of the cable duct crossings.

Pipe jacking is a no-dig technique for the installation of pipes and ducts. This technique requires a jacking pit for accommodating jacking equipment and jacking a shield head and then sleeve pipe sections into the ground. A receiving pit is required at the other end for retrieval of the shield. Hydraulic jacks push sleeve pipe sections through the ground. Excavation is carried out at the work face in the front end of the pipe. After pushing a full sleeve pipe section into the ground, a consecutive sleeve pipe section is then placed into the pit and connected to the end of the previous jacked pipe section and the jacking process is repeated. Selection of suitable locations of the working pits depends principally on site and traffic conditions, and accessibility and availability of storage areas. Accessibility of the sites

is essential for transportation of plant and materials to site and for disposal of excavated spoil. The major disadvantage of pipe jacking is that it can only be used for installing a pipe along a straight line. It is very difficult and expensive to rectify any misalignment once the jacking operation has started.

Table 1. Comparison of Various Trenchless Techniques

	Horizontal Directional Drilling (HDD)	**Pipe Jacking**	**Horizontal Core Drilling**	**Pipe Ramming**	**Rod Pushers**	**Auger Boring Machine**
Distance	Up to 2km	Up to 200m	Up to 20m	Up to 60m	Up to 100m	Up to 200m
Cost	High	Medium	Low	Medium	Low	High
Main Risks	- Geology - Stuck of down hole equipment - Pulling cable thru' long duct	- Set tlement of road - De-watering	- Mis-alignmen t	- Vibration - Noise pollution - Humping - Mis-alignment	- Damage adjacent services	- Damage adjacent services - Lost the machine due to stuck by obstacle
Limitations	Not suitable for minor crossings	Suitable working pits to be available	Short distance and small pipes	Same as pipe jacking and unable to pass boulder	Not suitable for rock base	Unable to break through large boulder
Applicable Areas	- Straight or curved alignment - sea / major crossings	Road / Railway / Stream Crossings	Minor crossings	Road / Railway / Stream Crossings	Road / Railway / Stream Crossings	Road / Railway / Stream Crossings

Detailed Design and Risk Management

Under the specifications of the Outline Agreement, an Approval in Principle (AIP) submission and a Detailed Design Approval (DDA) submission had to be forwarded to relevant government departments and interested parties for comment or approval.

In the course of seeking comments on the AIP submission, the proposed jacking and receiving pits for installing the cable duct crossings at Canton Road were originally situated at an open space outside the southern corner of Tsim Sha Tsui Fire Station and the southbound slow lane adjacent to Canton Road Substation respectively. The excavation and lateral support works of the pits comprised strutted sheet piles and soldier piles and shoring. Laggings were provided between soldier piles to support the excavated surfaces underneath the existing utilities. However, the AIP submission led to a critical design review and amendment due to a number of issues outlined below.

Interface with West Kowloon Cultural Development Project (WKCD)

It was identified that the proposed jacking pit would encroach upon the revised boundary of WKCD. As a result, the proposed sleeve pipe crossings and cable circuits had to be installed outside the WKCD boundary limit. This condition necessitated relocation of the jacking pit from the open space to the western footpath and northbound slow lane of Canton Road. However, the proposed relocation of the jacking pit was then objected due to the adverse impact on the traffic at the Canton Road northbound lanes. To resolve the potential encroachment upon WKCD and the adverse traffic impact issues, the proposed jacking pit was finally sited at the open space outside the northern shutter doors of Tsim Sha Tsui Fire

Station and the western footpath of Canton Road. Since the alternative jacking pit location inevitably obstructed the normal operation of the fire station, the Fire Services Department's permission for temporary closure of one shutter door of the fire station at a time was sought in order to facilitate the complicated four-staged temporary traffic management scheme required for the construction of the jacking pit. The diameter and number of the no-dig cable duct crossings were modified to further minimize the disturbance to the public and the operation of the fire station (see Figure 1).

Interface with a future Railway Project

At the project planning and feasibility study stage, a major railway operator advised that a planned railway development might have interface issues with the proposed cable duct crossings at Canton Road. To minimize potential risks of the proposed no-dig construction works, CLPP actively participated in a series of critical liaison meetings with the railway operator and lobbied the operator to grant permission for the construction of the proposed cable duct crossings. To address the operator's serious concerns over the risks to the railway project arising from the construction of the proposed no-dig cable duct crossings and their potential interface with the proposed railway tunnelling works and subway linking a future railway station, CLPP issued a justification paper to the operator. This provided a substantiation of the need for the proposed cable duct crossings at Canton Road, a detailed assessment of the feasibility of alternative options, and a recommendation of the preferred scheme for the construction of the proposed cable duct crossings at Canton Road.

Impact on Kowloon Park Drive Flyover North Abutment

The Canton Road no-dig cable duct crossings were installed underneath the existing northern abutment of the Kowloon Park Drive flyover (see Figure 1). The abutment structure comprises a series of soil retaining structures with shallow spread footing foundations. There was a risk that the no-dig crossings might induce ground settlement and impact on the structural integrity of the abutment. About 3 m vertical clearance between the spread footing of the abutment structure and the crown of the sleeve pipes is therefore provided. The predicted ground settlements and the differential settlements of the abutment structure due to the pipe jacking work were assessed based on the recommendations of Mair, Taylor & Burland (1996). An Alert, Action and Alarm trigger levels settlement and tilting monitoring scheme was then established to closely monitor any induced settlements and tilting of the abutment structure. These trigger levels corresponded to 0.5, 0.8 and 1.0 of the predicted maximum vertical settlement and angular distortion of the abutment structure. A contingency action plan was proposed to effect immediate actions when these trigger levels were reached. The Highways Department (HyD) Bridges and Structures Division commented on the revised AIP Submission that the risk of damage to the abutment structure of the proposed pipe jacking work needed to be assessed. To address HyD's concern, the induced strains and the degree of structural damage to the abutment structure were assessed based on the recommendations of Mair, Taylor & Burland (1996). The predicted degree of structural damage to the abutment structure was within acceptable limits and no underpinning of the abutment structure was required.

Conflict with Existing Medium Pressure Gas Pipe and Trunk Fresh Water Main

The construction of working pits was inevitably going to expose some major existing utility services, e.g. gas pipes, fresh water mains, etc. Lengthy discussions and special arrangements have been made with relevant parties to mitigate the potential risks of damaging the existing services and to enable public safety during the course of construction. Owing to the restrictive space available for the construction of these pits and the presence of

high grade IV rock head, a non-conventional design of the excavation and lateral support (ELS) works was adopted. The ELS works comprised strutted sheet piles (for earth support) and rectangular steel I-beam rings (for Grade IV rock support).

Construction and Supervision

The construction works were carried out by CLPP's Term Contractor and the Responsible Officer undertook the contract administration and site supervision. Qualified site safety supervision under the Categories I and III qualified supervision plan was provided throughout the construction period to ensure a safe execution of the works and compliance with the statutory requirements.

The construction of the cable duct crossings at Canton Road commenced in the fourth quarter of 2003. The construction of the jacking pit outside Tsim Sha Tsui Fire Station was executed with a well planned four staged temporary traffic arrangement scheme, approved by the relevant Government Departments, for pedestrian diversion and manoeuvring fire engines. The construction of the jacking pit comprised installation of strutted sheet piles and soldier piles at a spacing of up to 4.7 m, and temporary utility supports. Owing to the high ground water table, a grout curtain was installed behind the soldier piles before commencement of pit excavation to stop seepage inflow. To limit the induced ground settlements around the pit, groundwater table draw down during dewatering in the pit was limited to 1 m. Openings were formed in the sheet pile wall opposite to the concrete thrust wall for thrusting shield heads and sleeve pipe sections into the ground.

The construction of the alternative receiving pit inside Canton Road Substation involved temporary demolition and reconstruction of the existing concrete cable trough and temporary drainage diversion work to facilitate cable circuit installation work. The construction of the alternative receiving pit on the southbound slow lane of Canton Road was executed with the implementation of an approved temporary traffic management scheme.

Mixed ground conditions were encountered during the pipe jacking operation and any hard materials obstructing the shield heads were excavated with a pneumatic breaker and then removed. Particular measures were required for stabilization of the work face cut into the mixed ground.

Project Management and Stakeholder Management

Monthly Project Control Meetings between CLPP, the Responsible Officer, the Contractor, and the Designer were held to monitor works progress and to review and resolve all the critical issues of the project. The project team worked collaboratively and were committed to managing all project risks. CLPP management and engineers also conducted joint site inspections with the Contractor and ad-hoc site checks to ensure safety, health, environmental, and quality aspects of the work sites were properly managed.

There were consultations and close liaison with all relevant government departments and interested parties throughout the planning, design and construction phases of the project. These stakeholders provided valuable comments and advice on the study and design submissions, all of which facilitated the successful implementation of the project. Any objections and concerns raised by the stakeholders were properly addressed in a timely manner by the project team. With continuous and arduous stakeholder engagement, the project team persuaded the concerned stakeholders to lift their objections to the project.

PLANNING, DESIGN, CONSTRUCTION AND PROJECT MANAGEMENT OF NO-DIG CABLE DUCT CROSSINGS AT KWAI CHUNG ROAD

Planning and Feasibility Study

An existing box culvert and the Kwai Chung Nullah run underneath and along Kwai Chung Road. These drainage structures restrict the installation of the proposed cable crossings above the culvert and nullah deck due to inadequate soil cover and heavy traffic on Kwai Chung Road. To pursue the installation of the proposed cable circuits and cable ducts across this road, a no-dig construction method was considered to be the most promising solution to resolve this particular engineering problem. Twin cable duct crossings of 1.2 m diameter and about 35 m length needed to be installed below the base slab of the decked nullah in between its existing piled foundation and below the box culvert (see Figure 2).

Kwai Chung Nullah was built in 1967. Its concrete deck was subsequently built in the 1980s. The structural form of the deck is a concrete beam and slab system supported by precast concrete piles. There are two types of precast concrete pile supports, namely 381 mm octagonal piles and 406 mm square piles. These piles are arranged in a rectangular grid pattern of 4.5 m x 3 m. The two side walls of the nullah are actually part of the precast reinforced concrete sheet piles. However, there is no record of the founding levels of the concrete piles or the toe levels of the concrete sheet piles.

CLPP appointed Black & Veatch to conduct a feasibility study of the proposed no-dig cable duct crossings in 2002. The feasibility study needed to recommend the most preferred no-dig construction method and to identify any potential project risks. After taking into account all the site constraints and difficulties, pipe jacking was recommended.

Site Investigation and Survey

Based on incomplete record plans of Kwai Chung Nullah, it appeared that the exterior walls of the Kwai Chung Nullah were formed by contiguous interlocking precast concrete sheet piles. To ensure that the cable duct installation did not clash with the concrete sheet pile walls, site investigation works, comprising a series of inclined boreholes to detect the possible toe levels of the concrete sheet pile walls and two vertical boreholes to collect subsurface information, were conducted.

The toe levels of the concrete sheet pile wall were determined by drilling three inclined boreholes (see Figure 3). The first of these, drilled at an angle, did not intercept the eastern concrete sheet piles and was presumed to have passed beneath them. The second was then drilled at a slacker inclination angle but this one hit the sheet pile wall. A CCTV inspection of the borehole confirmed that the concrete sheet pile wall was encountered. The third borehole was set to an angle between the first two and this too hit the sheet pile wall. The toe level of the sheet pile walls was then calculated from the data.

The concrete piles supporting the nullah deck in the vicinity of the proposed cable duct crossings were surveyed inside the nullah to ascertain their exact positions against those shown in the as-built record plan. No substantial discrepancies were found.

Detailed Design and Risk Management

To mitigate the risks associated with the uncertainty of toe levels of concrete sheet pile walls and unforeseen ground conditions, it was recommended that there should be a clearance of 2 m between the crown of the sleeve pipes and the inferred toe level of concrete sheet pile

walls and that a horizontal pilot hole be drilled along the proposed alignment of each cable duct crossing to collect subsoil information.

It was also proposed that the twin cable duct crossings be installed at about 9 m below the carriageway and in between two rows of concrete piles. A clearance of about 1.2 m between the sleeve pipes and the adjacent concrete piles was maintained in order to minimize the impact on the existing piles. The jacking and receiving pits were sited at the eastern verge area and the western footpath of Kwai Chung Road respectively.

Assessment of Potential Impact on Concrete Piles supporting the Nullah Deck

The potential impact of the installation of the cable crossings on the concrete piles was assessed using relevant available information including as-built record plans, site investigations and surveys, and ground water table monitoring data. The founding levels of the concrete piles were predicted on the basis of the practical driven pile length and inferred soil strata providing pile skin friction and end bearing capacity. This assessment made reference to relevant geotechnical research reports and papers. The induced ground settlement and impact on existing piles of the decked nullah and the box culvert due to pipe jacking were assessed on the basis of the recommendations of Mair, Taylor & Bracegirdle (1993); Mair, Taylor & Burland (1996); Loganathan & Poulos (1998); Chen, Poulos & Loganathan (1999); Grant & Taylor (2000); and GEO Publication 1/96. The assessment indicated that all induced effects of possible concern were within acceptable limits.

Interface with a Trunk Fresh Water Main

The alignment of the proposed cable duct crossings intercepts that of a 1200 mm diameter water main along the kerb of the western footpath of Kwai Chung Road, and also the proposed receiving pit would have to be close to it. To relieve WSD's concern over potential damage to the integrity of the water main, an Alert, Action and Alarm trigger levels settlement monitoring scheme was adopted to closely monitor any induced settlement and angular distortion of this trunk water main.

Construction and Supervision

In addition to the site practice adopted for the construction of Canton Road cable duct crossings, an Alert, Action and Alarm trigger levels settlement monitoring plan was implemented to closely monitor any adverse effects on this critical decked nullah, piled foundation, and services due to pit and trench excavation, dewatering, and pipe jacking operations. A contingency plan set out the required actions to be taken at different trigger levels in order to control further aggravation of potential impact on the affected features. An automatic deformation monitoring system (ADMS) was also adopted for monitoring any excessive movements of the nullah decking and the roof slab of the box culvert above the cable duct crossings. The ADMS was a 24 hour alert system connected to the responsible persons' mobile phones and could inform them in the first instance when any trigger levels were reached during installation of the proposed cable duct crossings. Upon receiving any automatic notification of the ADMS, the responsible person should take immediate action as laid down in the contingency plan.

A horizontal pilot hole was drilled through the centreline of each cable duct crossing to determine any presence of obstructions and hence minimise risks. As the two horizontal drill holes encountered soft ground materials along the pipe jacking alignment, the pipe jacking operation then proceeded and was successfully completed on schedule.

Project Management and Stakeholder Management

The project management team closely liaised with relevant government departments and interested parties during the planning, design and construction phases of the project. The feasibility study report and detailed design reports of the proposed cable duct crossings were submitted to relevant government departments and interested parties for comment. Comments on the feasibility study and detailed design of the cable duct crossings were sought from major stakeholders, including the Geotechnical Engineering Office of the Civil Engineering Department, Highways Department, Drainage Department, Water Supplies Department, Kwai Chung District Council Member and representatives of local residents of nearby residential building. All comments received were addressed.

To relieve the environmental and safety concerns of local residents, the project team committed to providing proper environmental mitigation measures; temporary hoarding surrounding the receiving pit; a temporary steel deck covering the pit after working hours; and 24-hour security guard service.

CONCLUSION

CLPP has been striving to improve its supply reliability and is committed to executing construction works in a manner that is at the lowest cost while causing the least possible disturbance to the public and to road and pedestrian traffic. CLPP has put effort into the preservation of the environment and public safety in the vicinity of the construction sites.

The two no-dig cable duct crossing construction projects described in this paper demonstrate the pillars of successful project execution comprising prudent engineering decisions, planning, design, project management, and stakeholder management.

With the experiences gained from these two no-dig cable duct crossing projects, it is believed that installation of sleeve pipe crossings underneath bridge abutment structures and in between piling foundations can be pursued for further improvement in the future.

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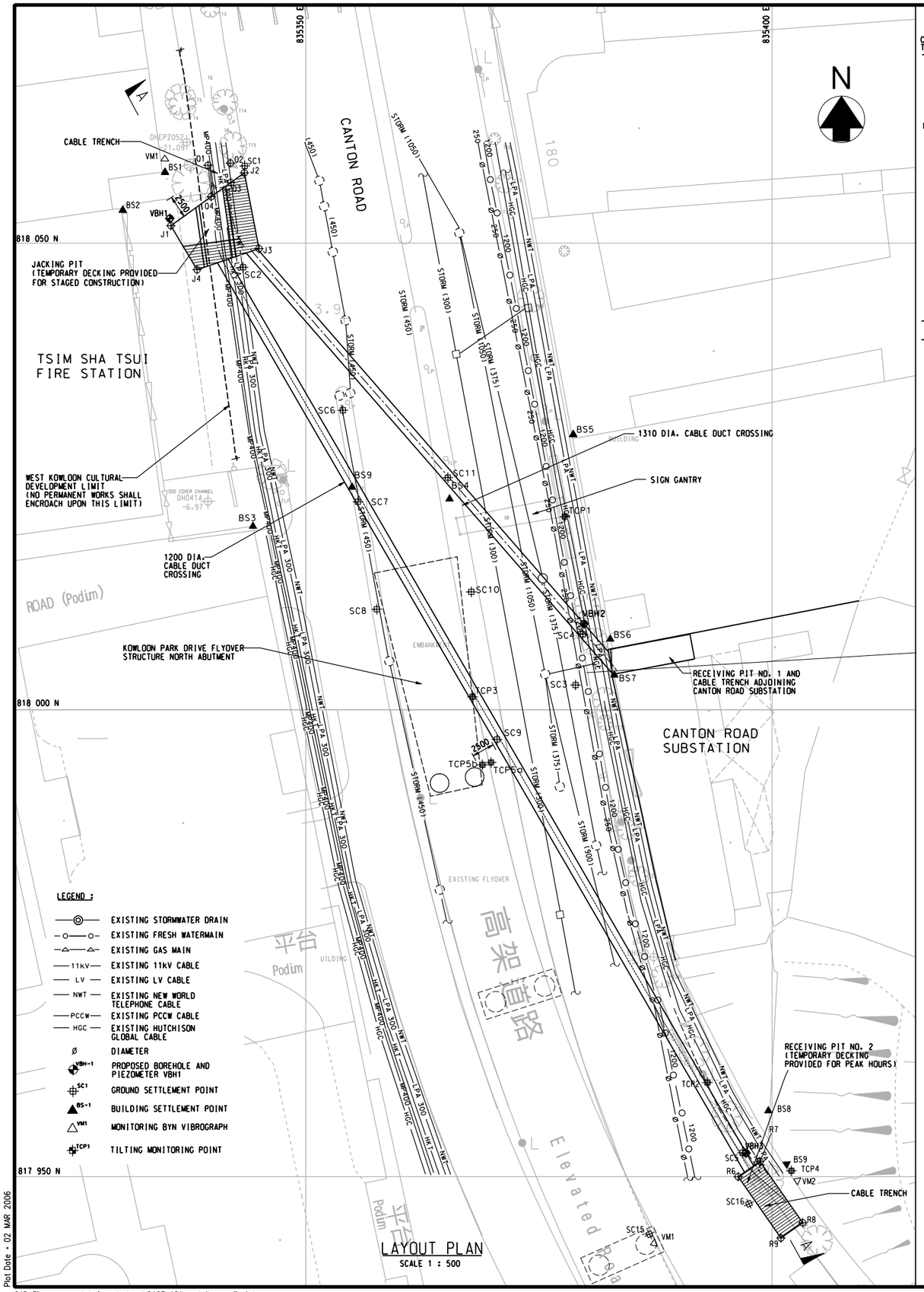

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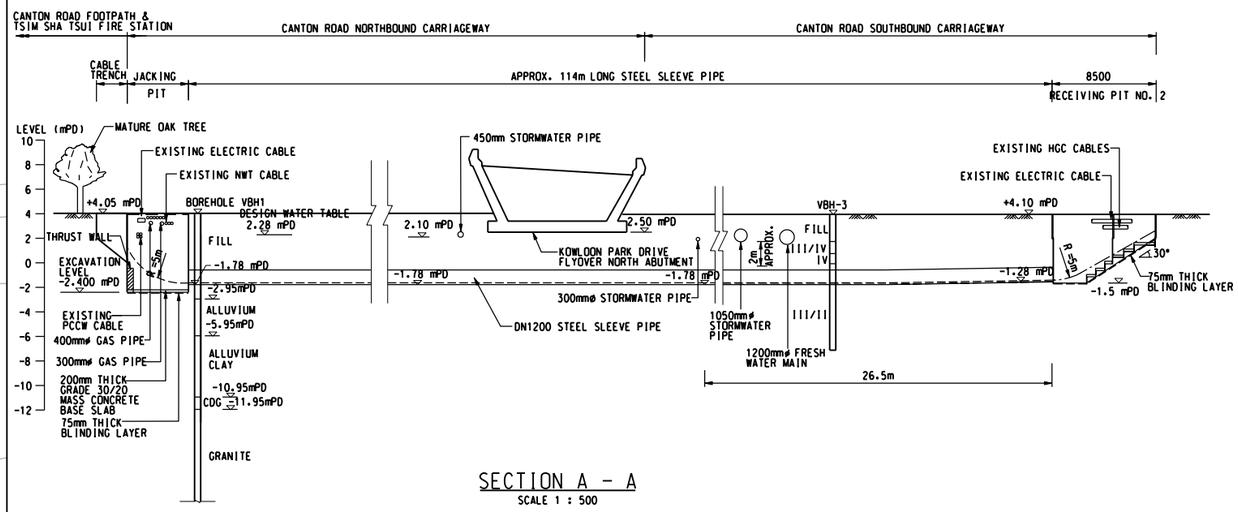

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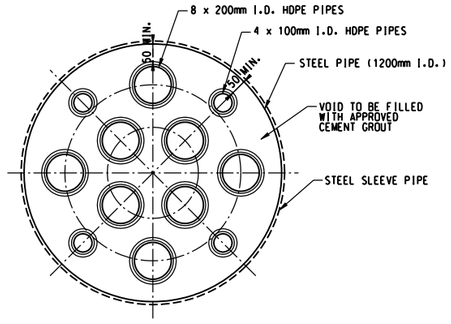

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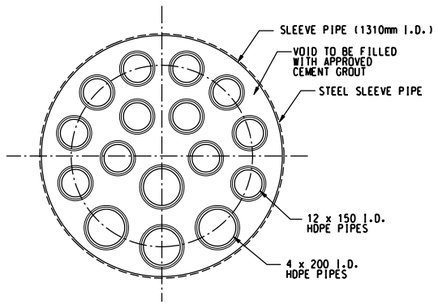

1310 DIA. CABLE DUCT CROSSING
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FIGURE 1 - LAYOUT PLAN AND SECTION OF CANTON ROAD CABLE DUCT CROSSINGS

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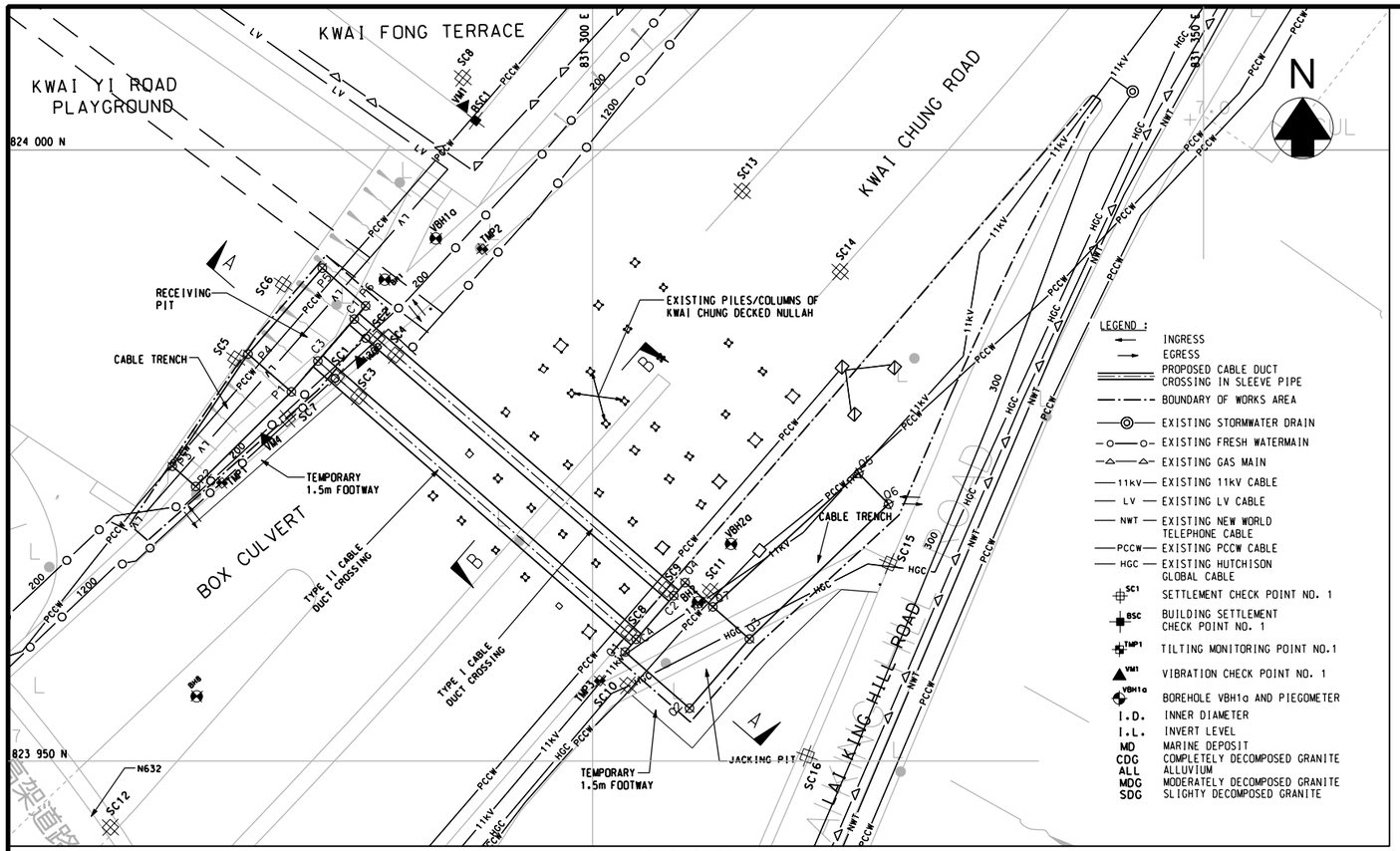

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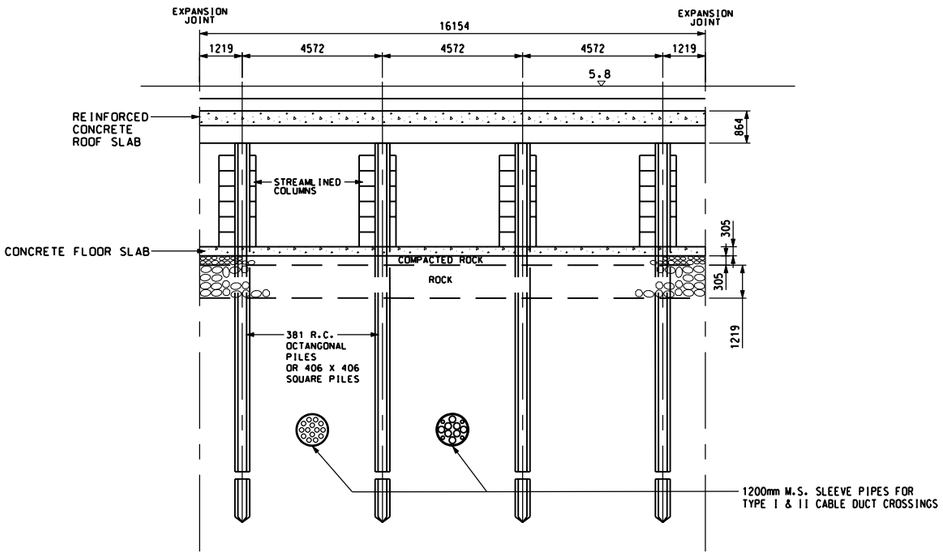

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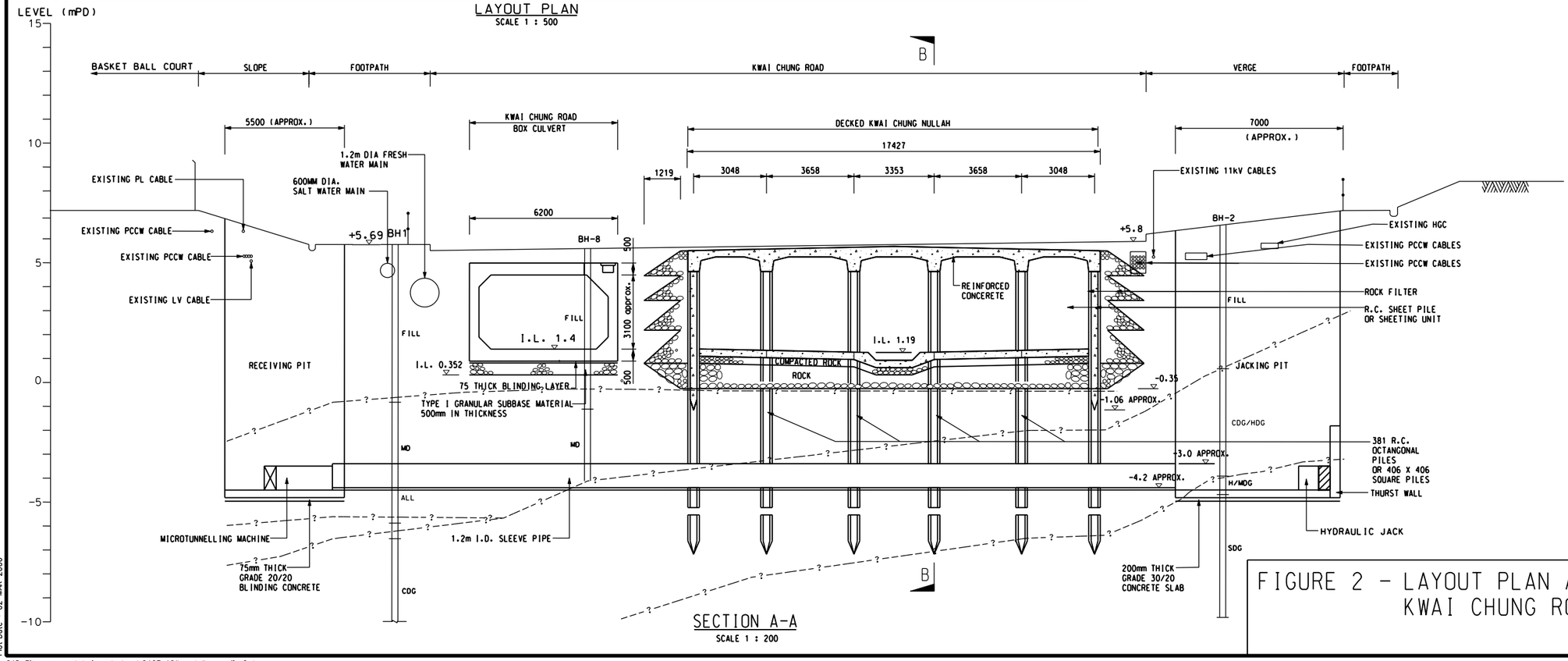

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FIGURE 2 - LAYOUT PLAN AND SECTIONS OF KWAI CHUNG ROAD CABLE DUCT CROSSINGS

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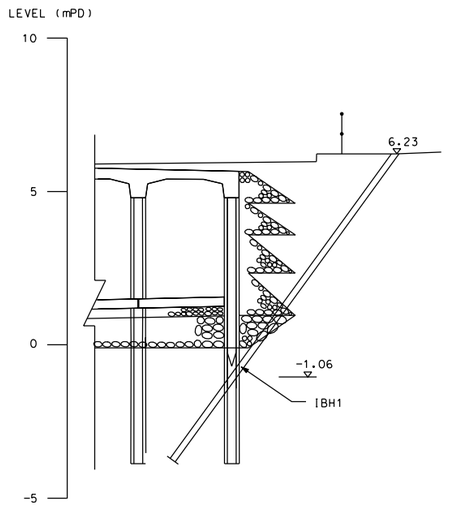

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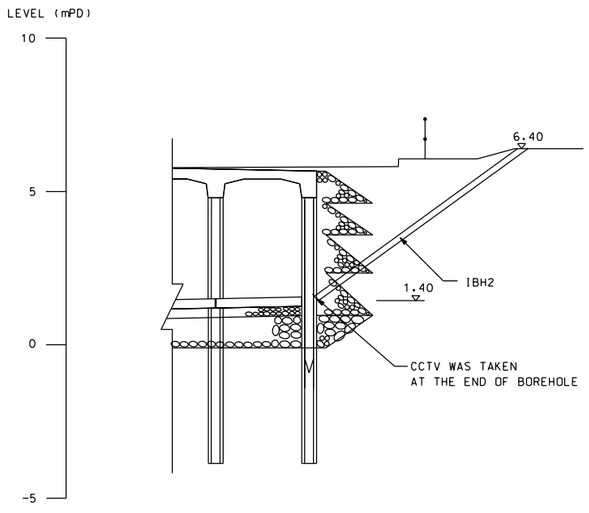

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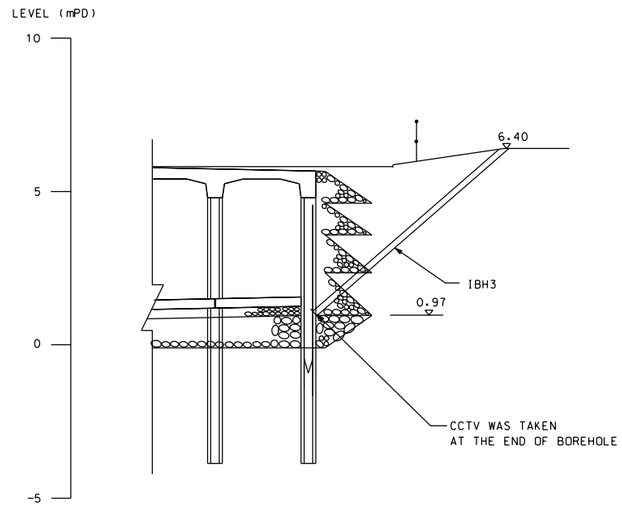

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FIGURE 3 - SITE INVESTIGATION FOR DETERMINATION OF TOE LEVEL OF CONCRETE SHEET PILE WALLS OF KWAI CHUNG NULLAH

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Design and Construction of the Disneyland Resort Line Tai Yam Teng Tunnel

David Salisbury[1], Justin Taylor[2] and Oscar Wong[3]

Abstract

MTR Corporation Limited (MTRCL) awarded the design and construct contract, 581-Tai Yam Teng Tunnel, to Gammon Skanska Limited (GSL), who in turn appointed Ove Arup & Partners Hong Kong Limited (Arup) as their main design consultant for the permanent and temporary works.

This paper will discuss the successful design and construction of the tunnel including the excavation, lining, portal structures and related earthworks. Whilst the majority of the works could be considered relatively straightforward and uneventful, a number of design, construction and contractual initiatives were undertaken on the project which led to significant cost and programme savings. These include; risks sharing for measurement and payment for primary support, precast top slabs to the cut & cover box section, spray on waterproofing membrane, preformed cavity invert drainage, and the use of shotcrete for permanent tunnel lining.

INTRODUCTION

The 3.8 km long Disneyland Resort Line (DRL) on Lantau Island provides a shuttle service between a new Sunny Bay Station at Sunny Bay, where the DRL interchanges with the Tung Chung Line, and a new Disneyland Resort Station at the Penny's Bay reclamation providing the main public access to the Hong Kong Disneyland Theme Park. The two stations are linked by a 3.6km long single line track. The line consists of 280m of open section at Sunny Bay, 870m of tunnel at Tai Yam Teng and 2.6 km of open section in the railway reserve area on the new reclamation in Penny's Bay.

The DRL four-car trains can carry up to 10,800 passengers per hour per direction utilizing a single track configuration with a passing loop. This capacity can be increased to eight-car trains to suit future demand. If required a second tunnel could be added enabling DRL to have a dual track configuration and further increasing the line's capacity.

A planning study was done between May 2000 and May 2001 which confirmed the alignment and station locations to enable the gazetting of the scheme under the Railways Ordinance. The scheme designs were completed by September 2001 with the Design and Construct tunnel contract being awarded to GSL in June 2002. The tunnel contract had a total value of HK$ 103 million including a section of Engineers designed emergency access road.

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[2] Engineering Manager, Gammon Construction Limited
[3] Construction Engineer, MTR Corporation Limited

LOCATION AND DESCRIPTION OF THE TUNNEL

The contract comprises a North Portal in cut and fill slopes at Sunny Bay, beneath the North Lantau Expressway, followed by section of 120m long soil cut and cover tunnel which connects to the 750m long rock tunnel. The tunnel swings through approximately 90 degrees from east to south along its length and emerges at the South Portal at the old Choy Lee Shipyard location in Penny's Bay. From here it continues in a cutting, swinging back from south to east, to the Disneyland Resort Station. The alignment is designed to operate at 80km/hr with a 3.5 minute headway using a passing loop on the open area between the southern portal and Disneyland Resort Station.

The location of the Tai Yam Teng Tunnel is shown in Figure 1, and the longitudinal section is shown in Figure 2.

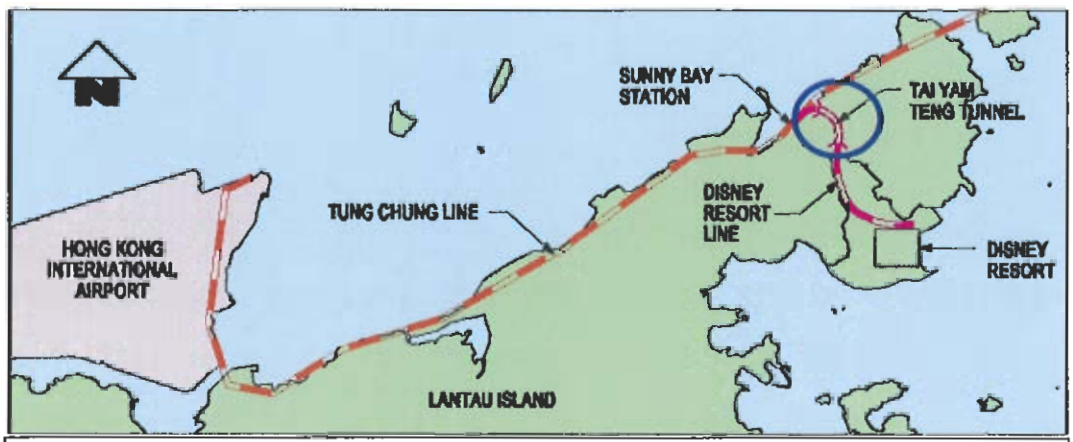

Figure 1. Location Plan

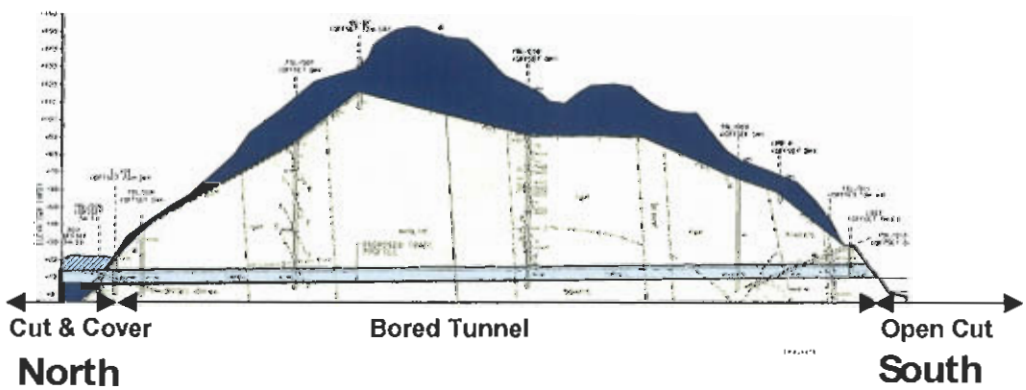

Figure 2. Longitndinal Section of Tai Yam Teng Tunnel

DETAILED DESIGN

MTRCL employed Arup to assist in the design of the DRL in early 2001. The two stations and the open track section were developed to detailed design by. MTRCL and Arup, while the bored tunnel section was developed to a scheme design stage before being tendered in September 2001. GSL were awarded the Design and Construct contract with Arup's tunnel team as their designer for all the permanent works and the bored tunnel temporary works.

The temporary works design for the cut and cover section was done by GSL's in-house design team Lambeth Associates.

As part of the early detailed design process a number of value engineering workshops were held from which several innovative design initiatives were taken forward, several eventually were developed into full detailed designs for construction. Among these were the use of 16 tonnes pre-cast reinforced concrete roof slabs for the cut and cover box section soffit, a preformed cavity high density polyethylene (HDPE) drainage system for the drained tunnel invert and the use of a sprayed waterproofing membrane in conjunction with the permanent works shotcrete for non-standard sections of tunnel.

The design submissions programme for the works was set by MTRCL with fixed milestone payments based in design submission approvals. This effectively required that any value engineering or design alternatives had to become fixed at a very early stage to allow the detailed design to be submitted in accordance with the design programme. By allocating sufficient design resources and providing regular design co-ordination meetings between all parties the majority of the detailed design was completed in just four months.

PARTNERING INITIATIVES

During the early stages of construction, GSL approached MTRCL with a risk sharing proposal for the re-measurement of the temporary support required during the excavation of the rock tunnel.

Following detailed negotiations an agreement was drawn up between MTRCL and GSL. The agreement was a simple 50/50 gain/pain share model which applied to any cost saving/overrun on agreed base quantities of the anticipated temporary support.

In return, the contractor agreed that all overbreak of the excavated profile would be absorbed under their responsibility including all effects on cost and programme. This approach has clear benefits in reducing the potential for claims for additional payment or extension of time relating to overbreak thus providing greater programme certainty, which was considered a priority by MTRCL.

A further initiative was introduced to the design agreement between GSL and Arup relating to two areas of the detailed design which were not clearly defined at the start of the contract, namely the scope of the System Assurance and Traffic Management input requirements from the design team. To cover the uncertainty of scope on these two items a mechanism for payment based on hourly rates with staged changes in rates up to a ceiling limit was agreed. Beyond this point a supplementary agreement would be required. The result of this mechanism allowed a more efficient use of design resources and resulted in a design agreement which led to no claims and an eventual discount on the design fee to GSL.

CUT AND COVER TUNNEL

The 110m long cut and over tunnel is an undrained reinforced concrete box structure with a wall thickness ranging from 500-700mm. From the North Portal, the permanent work structure, approximate 120m long, passes through the existing fill slope beneath the North Lantau Expressway, Cheung Tung Road and the Penny's Bay slip Road P2.

The permanent box tunnel structure is a combination of cast in situ and pre-cast concrete. The invert slab and the wall is cast in situ whilst the roof is made of 16tonne pre-cast concrete slab with cast in situ stitch joints between panels and box wall.

The box structure was formed in a strutted cut and cover temporary work excavation. Interconnected sheet piles or soldier piles with lagging formed the walls of the excavation with 305 mm x 305 mm heavy struts and composite beam section walers providing the temporary support, see Figure 3.

Figure 3. Strutting in cut & cover excavation

To ensure program milestones were met the phasing for the excavation and lining of the cut and cover works and portals was carried out on multiple work fronts. This allowed the completion of the box culvert invert and walls first, then commencement of the Drill and Blast tunnel works whist the pre-cast lids for the cut and cover box were being installed, see Figure 4. The lids were designed with exposed rebar to form cast in-situ stitch joints at the joints between each beam and at the connection to the walls.

Figure 4. Installing pre-cast lids

At the same time as the cut and cover works the excavation of the south portal works was commenced using the tunnel team while they awaited access to the bored tunnel section at the north portal. Blasting operations were only stopped at the North portal to allow the construction of the transition section between box and horse-shoe tunnel.

Figure 5. Finished cut and cover tunnel

Traffic diversion was arranged for Cheung Tung Road and P2 Slip Road with steel road decking and beam supports provided to maintain traffic access. The existing utilities were supported in situ across the excavation by hanging from the traffic deck supports or providing independent beam supports.

DRILL AND BLAST TUNNEL EXCAVATION

The main tunnel section, with a length of approximate 720m, was excavated in hard rock primarily by the drill and blast method. Local mechanical excavation was carried out at the portals and at one location or poor rock approximately 250m from the North Portal.

The rock tunnel is a drained, cast in-situ, horse shoe shape tunnel with an unreinforced structural lining. The excavated span is approximately 6.1m with a wall thickness between 250 and 350mm. The section of poor rock noted above was constructed as a 4m long fully reinforced section, see Figure 6.

The primary support design used during excavation consisted of a combination of "Swellex" rock dowels, shotcrete both plain and fibre reinforced and steel ribs in accordance with the rock quality encountered. The Q-system was used to determine the prescriptive primary support requirements.

The rock type and quality encountered varied significantly throughout the course of the excavation, with Grade I-III porphyritic Rhyolite, Grade I-IV Granite and Grade I-IV Tuff featuring at various stages. Some local excavation difficulties were encountered with the widely spaced jointing in the Rhyolite causing structural and excavation shape controlling issues in terms of excavation profile and temporary support of blocks, though fortunately these were limited, see Figure 7.

Rock quality prediction extrapolated from the very limited ground investigation data available at the time of tender proved very accurate and allowed the Contractor to be confident on the overall program. Added insurance through the use of partnering and the risk sharing on the ground conditions noted above gave all parties concerned an improved confidence level allowing engineering and timely delivery of the finished tunnel to take priority over commercial interests.

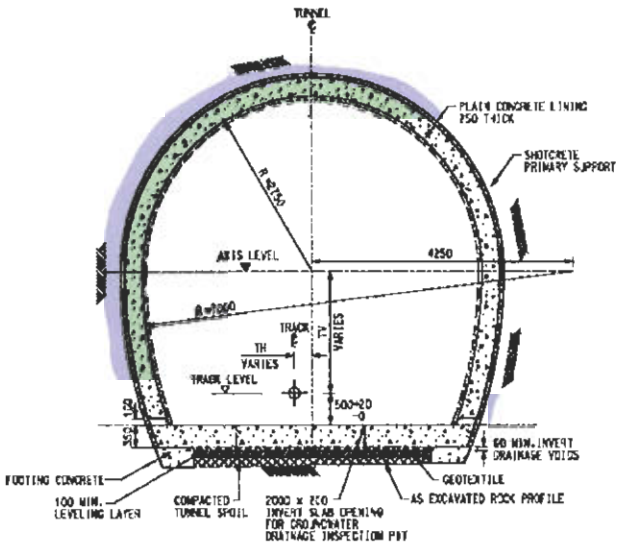

Figure 6. Typical Cross Section of Tunnel

Probing ahead was carried out, typically to 40m ahead of the face, providing a minimum 5m overlap. Groundwater inflows were predominantly low (5-10l/min), and generally pre-grouting was not used ahead of the face.

At the south portal ground conditions were significantly poorer. The construction required heading and bench excavation methods in conjunction with steel rib reinforcement and horizontal forepoling, grouting was used to stabilize the decomposed granite and colluvium with moderate success.

A Blast Risk Assessment was prepared and submitted to CEDD Mines Division and GEO four months ahead of the planned first blast. Mines Division/GEO expressed concern about the perceived noise impact, particularly from the use of 40g detonation cord (used in the perimeter holes), and the proximity of the North Lantau Expressway. To mitigate and quantify this, extensive day and night time noise monitoring and blast protection measures were implemented from the commencement of blasting works, no allowable levels were exceeded or complaints recorded. Blast doors were constructed at the North Portal together with separate independent mesh screens and sound proofing within the cut & cover sections of the tunnel. This was intended to reduce the noise impact of the blasting works. Separate screens were also constructed at the South Portal, for the final blast, prior to mechanically breaking the remaining 5m of rock for the tunnel breakthrough.

Figure 7. Geological influence on shape

Although a magazine for the storage of packaged explosive was approved, this was mainly used for detonators, 25g explosive primers and ancillary explosives. Bulk emulsion was used throughout the majority of the works, and a combination of cartridge and emulsion used for the majority of blasting. The explosives magazine was constructed off site, on an adjacent plot of open land, which allowed continuous blasting operations without reliance upon the timing of daily deliveries. Blasting operations, upon receipt of the 24-hour license, allowed up to 3 blasts per day.

The overall blasting works consisted of approximately 200 rounds of typically 80-85no. 48 mm diameter production holes, with 3no. 100mm diameter uncharged holes over a standard face area of 32 m^2. The two 30m long fan-niches had a face area of 55 m^2.

Using an Atlas-Copco Rocket Boomer L2C drill jumbo (Figure 8), a typical round length of 4.2 m was drilled for a typical pull of 3.8 m.

Approximately 38,000 kg of bulk emulsion, with 6,000 kg of cartridge explosives, and 17,000 detonators were used during the course of the excavation. A typical round used 250-275 kg of explosive (95% emulsion), with 92 detonators, including phase delay detonators and 200 m of 40g detonating cord.

Figure 8. Atlas-Copco L2C drill jumbo

At peak production the average weekly excavation was 44 m, with a maximum achieved weekly rate of 56 m.

Overall excavation took from May to October 2003, a period of 6 months, including changes of interface and operational set-up to allow the construction of the cut & cover sections and portals and slower production rates until the 24-hour permit was approved.

Q-values ranges from 0.01 at the South Portal to typically >1.0, with a maximum of 5.4. 17 m of excavation was carried out using heading and bench methods, using a mechanical breaker at the South Portal.

WATERPROOFING SYSTEMS

Smoothing shotcrete was sprayed over the rock surface to regulate the blasted tunnel to an acceptable profile prior to installing the waterproofing membrane.

The majority of the waterproofing system is a 2 mm thick PVC membrane placed against the crown and walls and held in place by hot welding to PVC washers nailed into the smoothing shotcrete. There is also a geotextile fabric between the membrane and the smoothing shotcrete layer, see Figure 9. This provides an effective 'umbrella' to the main tunnel structure discharging to the drained invert. The PVC sheets are double seam hot welded together, with the welds tested to an air pressure of 2.0 bar.

In the fan niches a permanent shotcrete lining was proposed as opposed to a conventional in-situ concrete as the complex geometry would make the formwork for the fan niches time consuming and expensive. Experience from other tunnelling projects raised concerns that traditional sheet waterproofing membrane had created problems in terms of voids at the crown due to poor bonding of shotcrete to the membrane and poor sealing details of the short rock anchors used to support mesh pinning the membrane to the shotcrete surface. This had resulted in water leaking, even with good quality control and workmanship. GSL requested Arup to investigate alternative solutions for the waterproofing of the fan niches. Alternative waterproofing systems were extensively reviewed in terms of safety, applicability, costs and ease of installation.

Figure 9. Waterproofing sheet membrane installation at interface with Fan Niche

Although several sprayed waterproofing systems were available, these were typically solvent based, and therefore unacceptable for the confined space environment. An alternative sprayed membrane based on polymer modified cement utilising shotcrete technology to spray the material was selected. Since this was a new material, extensive trials and tests were performed prior to the final application in the tunnel.

As the rock tunnel was designed to be drained, the niches were first lined with a geotextile fleece membrane to provide a drainage path before being coated in the sprayable membrane, see Figure 10.

Figure 10. Preparation for sprayed waterproofing membrane in fan niche

The costs associated with the spray applied membrane were found to be comparable with sheet membranes, with the real savings to be found by the lack of extensive remedial works, that had been know to occur where penetrations are required in sheet membrane systems. The sprayed waterproofing membrane material dried in several hours and could then be over sprayed with the permanent lining shotcrete, without the need for drilling anchors through it, and therefore creating areas of potential leakage.

CAVIDRAIN DRAINAGE MEMBRANE

The permanent lining is a drained structure with a geotextile fleece and 2 mm thick waterproofing membrane around the crown and side walls of the tunnel lining. The drainage system beneath the structural invert was envisaged in the scheme design as a central perforated drain pipe with feeder pipes, surrounded by a single size graded material.

On a recently completed tunnel section of the MTR Tseung Kwan O Extension (TKE) an alternative cavity drainage system was adopted. There had been some difficulties with the fixing of the drainage membrane on the TKE project leading to blockages of the drainage paths following completion of the in-situ concrete works. However, following design meetings with the supplier and MTRCL, it was felt that these issues could be designed out or overcome by modifications to the installation and construction sequence. The 'Cavidrain 60' membrane is an injection moulded, castellated, HDPE mat, which is laid on top of a highly permeable sub-base of 20-40mm crushed stone covered by a geotextile membrane. The mats interlock and overlap the drainage membrane coming down from the walls on a concrete strip plinth down each side of the tunnel, see Figure 11. The tunnel invert concrete is cast directly onto the mats which provide sufficient bearing capacity onto the sub-base while also producing an extremely efficient drainage path. A catch pit and 200 mm drain pipes

Figure 11. Installation of cavity drainage mats

transfer the drained groundwater through the undrained cut and cover section to discharge in to the storm water drainage system at the North portal.

CAST IN-SITU TUNNEL LINING

The typical tunnel lining layout is shown in Figure 6 above. The majority of the tunnel permanent lining is a typical MTRCL horseshoe lining consisting of an unreinforced base slab and arch cast in 10 m long bays using a steel expanding form shutter, See Figure 12. The minimum designed thickness was 250 mm, increasing to 350 mm for low rock quality and two reinforced sections, a 20 m length at the South Portal and a 4 m length at a faulted zone 250m from the North Portal. Lining was undertaken following completion of the excavation works. A key programme constraint of a relatively small size MTRCL tunnel is that it is impractical to construct the permanent lining while the excavation is ongoing. Lining started from the South Portal working downhill to the North Portal in a staged production cycle with concrete delivery from the South Portal.

Figure 12. Steel Arch Lining Form

SPRAYED CONCRETE LINING

At the two 30 m long Fan Niches and a number of other minor non-standard tunnel sections the crown of the tunnel was completed using fibre reinforced sprayed concrete.

As can be seen from Figure 13, the side walls were cast in-situ using the same profile as the main tunnel form, this is more efficient and has the added benefit of keeping the majority of the installed surface fixings and brackets to be mounted on the walls a standard design. The final layer of the sprayed lining was plain shotcrete to avoid steel fibres sticking out of the surface, which would be a safety hazard to the maintenance crews and a possible durability concern. The use of sprayed concrete for permanent works has now become well established in Hong Kong tunnels with the technique becoming more detailed and refined on each new project.

Figure 13. Sprayed concrete crown in Fan Niche

CONCLUSIONS

The tunnel contract was completed in June 2004 on programme and handed over to other system-wide contractors for trackwork laying and E&M installation. This in turn led to the opening of DRL in advance of the Disneyland theme park in 2005.

The success of this project was the result of close collaboration and partnerships. Whist the Tai Yam Teng Tunnel project appeared to be very straightforward in its overall design and construction this was not achieved without the combined effort of a highly motivated and professional team of engineers and skilled workers. Teamwork among MTRCL, Arup and GSL, with a strong partnering spirit and innovative ideas were vital in overcoming the challenges and achieving the objectives to deliver the project within time, budge as well as meeting the quality requirements.

Figure 14. Completed Southern Portal

The commitment to partnering repeatedly manifested itself in a desire from all parties to find the right engineering solution to all problems. Above and beyond this was a willingness to consider all innovative and value engineering proposals with an open mind allowing win/win solutions to dominate and benefit the overall solution. Not all the proposals put forward made it to the construction stage, however, they did provided a project that was interesting to work on and a body of useful knowledge for use in future tunnel projects.

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