1 GENERAL INTRODUCTION

Ground water encountered in hard rock tunnelling is one of the main risk factors for time and economy of projects. In the simpler cases, it is just a matter of practical working conditions inside the tunnel and how water ingress may influence the construction period. However, increasingly the tunnel influence on the surroundings is becoming dimensioning for what needs to be done to control ground water ingress, both during construction and during operation of the tunnel. Key words are ground settlements on surface and loss of ground water resources like lakes and drinking water wells.

In extreme cases, tunnels and equipment in the tunnel may be submerged in water due to excessive water inrush at the tunnel face, which may have serious effects regarding time and cost and especially if the method of excavation is by TBM. In such cases, the TBM itself becomes an obstacle regarding access to the face area for execution of mitigation works and the cost and time of repairing a TBM after such an event would be extreme.

In principle, it is quite simple to avoid serious problems caused by ground water ingress into tunnels:

1. Establish trigger values for PEG based on project specific requirements and risk
2. Execute systematic probe drilling ahead of the tunnel face
3. Execute grouting of the probed tunnel section, if triggered by water ingress into probe holes
4. Excavate to the end of treated ground, minus a suitable length of safety bulkhead

The devil sticks in the details regarding how to efficiently and successfully carry out the 4 above steps and the number of different opinions on the subject are certainly numerous. It is recommended to apply well proven strategies developed for tunnelling through 100's of km of practical experience, in clear preference to any highly theoretical and often quite complicated methods.

One very important aspect of PEG in tunnelling is frequently overlooked, or not given enough attention. If PEG must be executed for any reason, it becomes imperative to reach the required results with a minimum of time spent doing it. Some grouting specialists will focus on what is technically optimal for the grouting process as such, rather than what will deliver the best overall efficiency. Considering that the time-related cost of operating a tunnel face is extremely high, it is surprising how often focus on isolated cost elements leads to the wrong decisions. One example of this will be covered below, regarding the economic and technical effects of the type of cement that is chosen for PEG.

The development of techniques for PEG in tunnelling in Norway started already in the 70'ties due to urban tunnelling causing settlement- and environment problems in the Oslo area, as well as to control heavy water ingress in hydro power tunnels. Later, the extensive program of sub-sea road tunnelling in Norway became a driving force for further development. To cover all the steps leading up to today's best practice in PEG would be beyond the scope of this article. To present the current status of the technique, a summary of the HATS2A project in Hong Kong is used. This example is used to show what can be achieved when required, but less demanding circumstances will allow less elaborate approaches. The balance must be decided for each individual project.

2 INTRODUCTION TO HATS2A

In January 2006, Drainage Services Department (DSD) initiated the investigation, design and construction of the second stage of the planned Sewage Conveyance System. The HATS Stage 2A Sewage Conveyance System targeted collection and conveyance of pre-treated sewage from along the northern and south-western shoreline of Hong Kong Island, to the Stonecutters Island Sewage Treatment Works, for treatment before final disposal into the western harbour via an existing submarine outfall [1].

The project comprises about 20 km of tunnels at depths of about 70 m, 120 m and 150 m below sea level and a total of 14 vertical shafts from surface to tunnel level.

2.1 Project overview

Apart from the 4 km tunnel crossing underneath Victoria Harbour, the tunnel alignment is located beneath densely built up and populated areas (Figure 1). Since the tunnel follows the coastline of North and West side of Hong Kong Island, a fair share of buildings and infrastructure influenced by the tunnel are founded on reclaimed land on top of marine sediments. Some of these areas are sensitive to groundwater drawdown that may cause surface settlement.

The problem was well demonstrated during Stage 1 of the Project more than 10 years ago, when unusual settlement was experienced during tunnel excavation. Large water inflows through the generally highly fractured volcanic tuffs were encountered, despite significant pre-grouting and post-grouting efforts to control water ingress. Based on the available monitoring records, this resulted in extensive groundwater drawdown, and unexpected settlements that occurred at large distance from the excavation face. Drawdown at large distance was associated with the locations of faults and paleo-valleys.

Figure 1. HATS2A Layout for 20 km of tunnelling

Hydro-geological numerical modelling was used to analyse the maximum allowable residual ingress to the tunnel (after PEG) to avoid surface damage by settlement. The selected ingress classes range from 2.5 L/min/100 m of tunnel for the most sensitive areas to 50 L/min/100 m for the least sensitive sections. In between, there are requirements for 5, 15 or 30 L/min/100 m. It may be noted

that the two strictest ingress classes were considered not realistically achievable by PEG at the planned depth of tunnels. The design sent out for bidding, therefore contained a specified method for further reduction of water ingress down to the two strictest classes. This method was termed First Pass Lining (FPL) and consisted of an extra concrete lining with the sole purpose of water ingress reduction. The Contractors were required to design and describe this approach in detail. FPL for water ingress reduction was not implemented because of successful PEG works as well as serious problems of designing an approach offering reasonable chance of success.

2.2 Brief description of geology

The geology along the northern shore of Hong Kong Island to Stonecutter's Island comprises generally medium grained biotite granite of the Kowloon Pluton, which was expected to host the majority of the tunnels. Several HATS Stage 1 Tunnels were also driven in the Kowloon Pluton, as were sections of the MTRC Island Line near Admiralty Station, Causeway Bay Station and between Fortress Hill and Quarry Bay Stations. Consequently, a large amount of direct experience to draw upon in driving both deep and shallow tunnels in the Kowloon Pluton existed.

The Kowloon Pluton has been intruded into predominantly volcanic rocks of the Repulse Bay Volcanic Group which includes the Mount Davis and Ap Lei Chau Formations which underlie large areas of the western and southern parts of Hong Kong Island.

Several sections of HATS Stage 1 were driven in volcanic rocks which are similar in lithology, block size and strength to the volcanic rocks encountered in the HATS2A tunnels. The Hong Kong Electric cable tunnels between Aberdeen and Cyberport and through Ap Lei Chau Island also provide some additional experience of tunnelling in similar rock types.

All the rocks contain pegmatite and dyke rock intrusions as well as quartz and calcite veins to some degree. These secondary dykes and veins tend to be more common near the volcanic/granite contacts and near faults. Micro-fracturing can also be extensive in the volcanic rocks near the granite contacts. Three main geological units are recognised:

- Kowloon granite (Klk)
- Mount Davis Formation (Krd) consisting of predominantly volcanic coarse ash crystal tuff with subordinate fine ash tuff and metasedimentary rocks
- Ap Lei Chau Formation (Kra) consisting of predominantly volcanic fine ash vitric tuff with eutaxitic layers, coarse ash tuff, thin lava and epiclastic layers

2.3 Groundwater control at Stage 1 (SSDS)

The first Contractor was not prepared for and did not execute pre-excavation grouting (PEG). Because of this, he was only able to excavate a little over one km of the 24.5 km contract before the water ingress problems stopped further progress. Re-bidding was carried out and 3 new Contractors took over the TBMs already in place to continue the works. The TBMs had to be modified underground with drilling equipment for probing and PEG, which was an improvised and difficult task. In effect, only a limited number of drilling positions could be installed and the holes had to be started meters behind the actual face and at fixed look-out angle. The strictly limited adaptability regarding layout of drilling ahead patterns therefore became a significant problem.

PEG was started with ordinary Portland cement (OPC) and Bentonite grouts, but gradually changed to micro fine cement (MFC) or a combination with OPC. No chemical grout was used. The ground

water control works were not very successful and despite attempts at post grouting, the water ingress to the tunnels created significant practical problems and delays for the tunnel works as well as serious surface settlement and damage in the Tseung Kwan O area.

Even more serious were the face collapses in shear zones with extremely poor ground where the water triggered and contributed to the problems encountered. Face collapses caused months of construction schedule delays.

2.4 PEG, method of payment

In Hong Kong, different methods of measurements for grouting have been adopted in different tunnel contracts. This include:

- Grouting for ground water control is included in the payment for tunnel excavation.
- Grouting is covered by a separate lump sum irrespective of actual quantities.
- The BQ contains a provisional sum to cover cost of grouting when grouting is needed, on a cost-plus basis.
- The BQ contains a fixed number of tunnel meters that are assumed to need grouting, to be paid per meter of tunnel treated.

Since grouting for HATS2A faced very strict limits at high water head, quantities were considered uncertain. It would consequently not be productive that the Contractor should alone bear the risk of this operation. Hence the contract provided re-measurable quantities for drilling and grouting for ground water control. The HATS2A sewage tunnels represented a unique challenge with no equivalent or even similar method of measurement from previous contracts in Hong Kong [2].

2.5 Execution of PEG

2.5.1 Choice of materials

The practical process of PEG execution for this project mainly consisted of well-known practical steps and has been described in detail in [3] and [4]. However, the overall combination of elements used for HATS2A is unique and this combination is the main reason for the successful results.

The most important elements of the implemented approach start with unconditional probe drilling ahead of the face for 100% of the tunnel length. Minimum number of probe holes was four. When the water inflow from probe holes triggered grouting, micro fine cement (MFC) was always the primary grouting material of choice. No ordinary Portland cement (OPC) was used. This choice may seem exaggerated to many, but the higher cost per kg paid off well in improved efficiency, less drilling of grout holes and improved execution control with better sealing effect.

A rough calculation comparing the use of OPC or MFC for PEG may illustrate this point:

- Common basis for the comparison
  - 1000 m of 25 m2 tunnel
  - PEG-stations every 20 m => 50 grouting stations per 1000 m tunnel
  - Average cement consumption = 300 kg/m tunnel
- Case I, grouting by OPC
  - 1 grouting round per station (not realistic, normally 2 or more)
  - Waiting time for cement setting = 5 hours (can easily be more)
    5h x 50 stations = 250h waiting time/1000 m tunnel
- Case II, grouting by MFC
  - Also 1 grouting round per station

  - Waiting time for cement setting = zero
  - Extra cost of MFC over OPC = 0.45 EUR/kg
    0.45 x 300 x 1000 m = 135.000 EUR/1000 m tunnel
- Comparison of I and II:
  - If 1h of face time cost EUR 540
    cost of OPC waiting time = 540 x 250h = 135.000 EUR/1000 m tunnel
    which would mean cost breakeven between I and II

However, reality is a different in many ways:

- The actual cost per h of face time is easily EUR 3000,
  which is 5-6 times more than the above breakeven point
- Normally, at least 2 stages of grouting are required per station, doubling the waiting time and cost of waiting given above
- Normally, OPC-consumption is higher than consumption of MFC for reaching the same result (not equal as used above)
- More time is needed for grout execution when using OPC
- More holes are needed with OPC due to poorer penetration
- More time is needed for drilling when using OPC due to more holes
- Results tend to be poorer using OPC
- OPC may therefore cause more use of post-grouting, which is notoriously time-consuming and extremely costly

The only reasonable conclusion is to use MFC only, also because of the benefits of simplified site operation when logistics only need to cover one cement type through 150 m vertical- and up to 2 km horizontal access.

When MFC does not sufficiently penetrate conductive features that still produce more water ingress than acceptable, to a degree this can be solved by closer hole-spacing and more grouting stages with MFC. However, for target ingress limits of 15 L/min/100 m or less, it quickly became more efficient to use colloidal silica (CS) as a supplement. In complicated ground conditions with clay gouge and other fines, CS was often necessary to achieve the targeted ingress limit.

Another important element consisted of the combination of non-bleeding and fast setting MFC grout, use of dual stop criteria (stop on pressure or quantity) and very high stop pressure (60-100 bar).

Furthermore, modern grouting equipment was crucial and nothing can beat the availability of a drill jumbo at the tunnel face to allow drilling of all necessary holes in whatever direction and length that is required. After every stage of grouting, the result had to be checked by drilling control holes to measure the water inflow to these holes. No excavation was allowed until the PEG result was verified to be acceptable by measured inflow through the control holes.

2.5.2 Observation design update

The PEG design was developed to target as closely as possible the different residual ingress limits along the tunnel. The final control was made by measuring the actual water ingress to excavated sections of the tunnel. If the result was far off (too good or too poor), there were two possible actions to consider:

1. If residual ingress was too high, it was possible that post grouting and/or water re-charge had to be employed to mitigate the problem.
2. The other consideration was about the not yet excavated tunnel length. Modifications to grouting trigger limits and other aspects of the execution could be made to achieve results closer to the target.

Generally, it was decided to maintain a high level of safety margin to ensure that the execution would satisfy the specified ingress limits and not cause settlement beyond local limits.

Complicated shear zones within sections of very strict ingress limit turned out to represent the most difficult challenge. The sections of tunnel that failed to satisfy the local ingress limit were primarily within such zones and the failed results occurred despite executing a high number of grouting stages for individual PEG stations, but settlement monitoring results were fortunately still within limits.

It should be noted that the specified First Pass Lining mentioned above was never implemented since the required ingress limits were satisfied by PEG only.

2.5.3 Groundwater control in granite

Some tunnel sections passed through excellent quality, almost massive granite where no grouting was required even for the strictest ingress limit. At the other end of the scale, open joints have produced full-pressure water jets from individual probe holes of > 4000 L/min as illustrated in Figure 3.

In basically fresh granite, intrusions of diabase were frequently crossing the tunnel alignment and sometimes exposed highly conductive channels with groundwater.

Figure 3. Probe hole hitting 15 bar water channel.

In a typical 20-30 m long borehole in virgin ground, the water conductivity contrast would normally be very high. This meant that along the length of the hole, there would typically be one or very few channels that generated most of the water. This had the effect that most of the grout would take the path of least resistance and frequently the grouting would stop on quantity limit at low grouting pressure. Even though such cases often lead to more than one grouting stage, it was relatively easy to bring large original ingress from fresh granite down to practically nothing.

Shear zones on the other hand, could also yield a lot of water, but clay and fines content would often cause a rapid increase of the grouting pressure to reach stop level already at very small grout quantities. When this happened, the hole was practically “wasted” since penetration and distribution of grout stopped prematurely. This is where the use of CS became very beneficial. With a viscosity of 5 cP, it penetrates practically like water and can significantly reduce the number of boreholes needed to reach the required grout distribution and sealing effect.

Another technique that was used very successfully in highly water conductive shear zones with clay gouge and crushed rock was double overlap grout fans. By drilling and grouting a fan length of 16 m and starting the next one after 8 m tunnel advance, all cross sections would be covered by two layers of grout holes.

2.5.4 Very wet and very dry sections through granites

The granite demonstrated a huge variation in ground water conductivity, even within pretty short distance. As an example, the first about 1500 m from Sai Ying Pun towards East (Drive 3, Tunnel K)

went through almost massive granite with very little water ingress. Long sections did not trigger any PEG and the overall consumption of MFC was about 50 kg/m of tunnel.

As a striking contrast, Tunnel L from Sai Ying Pun direction Stone Cutters Island (Drive 2) under Victoria Harbour hit extreme ground water ingress. A summary of a 413 m long section from about Ch 1150 showed MFC consumption of 1549 kg/m of tunnel. This was caused by wide open clean joints in otherwise fresh granite. Measurements of water ingress to probe holes and first stage grout holes (holes into virgin ground) over these 413 m of tunnel, translated into 9200 L/min/100 m of tunnel (or 38 m^3/min in total). It is obvious that mining ahead into this tunnel section without PEG could have yielded a lot more than what was measured from the boreholes and the consequences could have been catastrophic. See Figure 4, showing a probe hole hitting 1200 L/min at 15 bar pressure.

The execution of PEG in the wet section was carried out according to the normal procedures, but the stop quantity per hole was increased to 4000 L of grout and using lower w/c-ratio than the standard 1.0. Mostly, two stages of grouting would allow advance of the tunnel face.

Measurement of residual ingress covering the 413 m long wet section after excavation, showed ingress of roughly 1.0 L/min/100 m of tunnel, which is close to a 100% ingress cut-off (down from at least 38 m^3/min).

Figure 4. Probe hole hitting 1200 L/min flow rate.

When mining through the treated ground, open joints of more than 100 mm aperture were filled by MFC as mapped in the tunnel walls and face. The joints were 100% sealed by hardened MFC (see Figure 5).

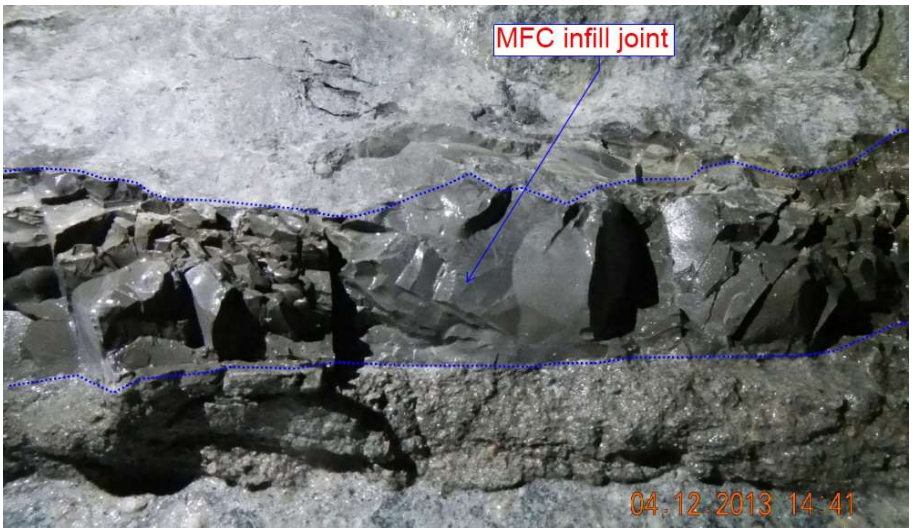

Figure 5. Open joint filled by MFC.

2.5.5 Ground water control in volcanic tuff

The volcanic tuff could locally be practically massive, as demonstrated by core samples of one meter length. Since this rock type shows very high compressive strength and is very brittle, most of the tunnelling therefore went through highly fractured and jointed rock.

The typical joint spacing was very close and most joints were of short extension and with a fair share of random orientation. The resulting joint apertures were in average just fractions of a mm. The water ingress to a borehole could still be significant because of the high number of narrow joints and cracks. In areas of shear zones, the presence of widely extending channels and joints of very high conductivity sometimes yielded water inflow of 100 L/min and more from single boreholes.

Typically, there was a significant tendency for holes grouted with MFC to rapidly go to maximum pressure shortly after having filled the borehole volume. Especially in areas of strictest ingress limit (5 L/min/100 m), this required significantly closer borehole spacing, need for more grouting stages and much more use of CS than originally expected or specified. The flip side of this situation was that the consumption of MFC was much lower than expected and specified. Compared with the granite, consumption of MFC was 30% lower per meter of drilled boreholes.

Like in granite, the main problem was not the extreme leakage locations, but rather to sufficiently seal off all the finer cracks and joints with the use of a limited number of grouting stages. When the limited penetration achievable by MFC became a problem, CS turned out to be an invaluable supplement.

Also, in tuff disturbed by faulting, the use of double overlap grout fans was implemented locally and especially in the shear zone close to the foot of the Cyberport shaft, it was also required to extensively place standpipes rather than packers.

2.6 Grout materials used

2.6.1 Micro-fine cement (MFC)

The primary grout material for PEG was specified as micro-fine cement with Blaine value not less than 625 m^2/kg. Selected cements had maximum particle size < 30 µm, initial set of between 60 and 120 minutes and final set of between 120 and 150 minutes (ASTM C-191: Vicat Needle). Furthermore, it was required to use a non-bleed grout that still offered a very low viscosity (less than 2% bleed after 2 h and less than 35 s Marsh Cone time measured on 1 L of grout). To satisfy this combination of requirements, the MFC had to be of pure Portland cement type and any of the many blended micro-fine cements available for soil grouting and foundation grouting were ruled out. The

minimum compressive strength of the grout was specified as 5 MPa, but the selected cement types gave typically double or even quadruple strength depending on actual w/c-ratio used.

The requirement of non-bleed property is important to allow the use of stop criterion on quantity, without having reached any significant pressure. When stopping on quantity, the channel volume filled will remain filled when the grout has set without any residual channel caused by bleeding. It is also not necessary to pump grout until refusal to get pressure enough to squeeze out surplus water to avoid such residual channels. This saves grout volume and execution time, while improving the result.

The targeted properties of the cement grout were achieved at w/c-ratio 1.0 or less by also adding a suitable water-reducing admixture. To avoid premature setting of the grout at high ambient temperature, a stabilizer sometimes was added to the mix design. The modern stabilizer admixtures are not changing the properties of the mix, other than delaying the onset of setting.

2.6.2 Colloidal silica (CS)

Colloidal silica is not a chemical grout, since the setting process that produces the gel, does not involve chemical reactions. CS is a suspension of molecular size SiO_2 particles in water (typically smaller than 0.02 µm), which for practical purposes makes it behave like a true liquid. The catalyst that starts the gelling process is a 10% solution of table salt (NaCl) in water. The gel is created by electric bonds between particles and not chemical bonds. The gel is permanently stable and gains strength over years following injection.

It should be noted that CS therefore has nothing to do with Sodium Silicate chemical grout and that all components of CS are very environment friendly. The product allows gelling times from seconds to several hours and is very easy to handle and work with. Because of the microscopic particles and grout viscosity close to water (about 5 cP), the penetration capability is excellent, thus offering an added tool if cement cannot penetrate while further sealing off is required.

2.7 Selection of grouting materials for individual holes

2.7.1 Basic strategy

Two grout materials were available for use: The fast setting, high strength micro-fine pure Portland cement (MFC), or the colloidal silica gel (CS). Selection had to be made based on the local targeted maximum residual ingress, ground conditions and rate of water inflow measured from drilled holes.

The ground rule was to always use MFC when this material could penetrate as required to achieve targeted results.

Figure 6 shows in true scale the relationship between maximum particle size of OPC, MFC and a joint aperture of 0.1 mm, while the CS-dot shown would not be visible at this scale.

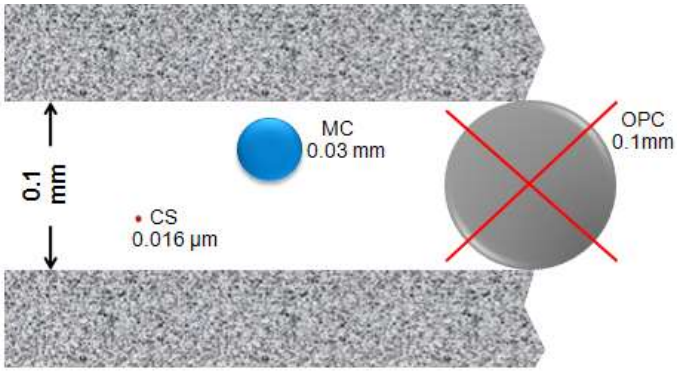

Figure 6. Example of relative particle and joint aperture dimensions.

However, when most grout holes (even when yielding 10's of L/min water ingress) would spike to maximum grouting pressure at very small grout quantity, this would often not provide the required grout spread and sealing effect and CS would be needed as a supplement.

Striking the best balance between the two material choices was influenced by the relative volume cost, cost of drilling and most importantly, the cost of time at the tunnel face.

In very poor ground and through shear zones, borehole spacing frequently became very small. The high strength of cement was then important for improvement of ground stability, but also for avoidance of new holes deviating to merge into a previously drilled and grouted hole. With the weak gel of CS filling previous holes, this would easily take place and the "new" hole would be of no use.

All grouting in tunnel sections with ingress limit of 50 L/min/100 m was done with MFC only. Also, most sections within the 30 L/min/100 m limit were treated with cement only. However, for the stricter limits, CS would become increasingly useful by helping to achieve the target while reducing necessary drilling of boreholes. Use of CS would therefore reduce the time spent for a complete grouting station, especially in the volcanic tuff.

2.7.2 Original selection approach

After drilling all the first stage grout holes and measuring the water inflow, holes yielding more than a decided limit (say 3 L/min) would be grouted by MFC. The remaining holes would then be treated by CS before drilling control holes to check the result of first stage grouting.

The problem found was that holes expected to take MFC could block at almost no grout take, while holes expected to be suitable for CS would build no pumping pressure and the CS grout would just "run away" to be stopped on quantity. Without reaching a reasonable pressure, even CS would not penetrate into the targeted finer joints.

Based on continuous evaluations of performance to find the right balance, the selection limit for MFC was tried as high as 10 L/min within the volcanic rocks and MFC could still be blocked prematurely.

When single borehole inflow was 50-100 litres per minute or more, MFC would be used in any case. The probability of reaching maximum quantity at pretty low pressure would then be high. Therefore, after starting with w/c-ratio 1.0 and pumping 500 to 1000 L with very little pressure increase, lowering of the water cement ratio would be done in steps. The target would be to get as close as possible to maximum pressure of 60 to 70 bar before reaching the quantity limit (say 4000 L in such cases).

Also, when pumping CS, stop on maximum pressure or maximum quantity would be implemented. Preferably, the stop should be reached by pressure and for best possible result the pumping time should be > 50% of the gel time of the grout being used.

Quite frequently, holes yielding very little water and therefore selected for grouting by CS, would reach stop criterion on quantity at very low pressure. When this happened, the penetration efficiency of the CS would not be utilized due to lack of pressure and the hole should have been grouted by MFC.

2.7.3 Modified selection approach

The primary problems of material selection consisted of how to avoid 1) waste of CS if pumped into a borehole with channels of too high conductivity and 2) waste of the borehole if MFC spiked to maximum pressure just after filling the borehole volume, when it turned out that predicting the outcome was not possible.

The above-mentioned problems are of course caused by the normal (and unknown) conductivity contrast along a 20-30 m long borehole. The water ingress can be measured, but whether this water comes from very few and large channels or a high number of small cracks and joints will be unknown. The water yield from a drilled hole therefore turns out to be a very unreliable parameter for the selection of grouting material for use in any given borehole.

Especially in the tuff, the above described problem was frequently encountered.

The successful solution consisted of starting all grout holes by CS almost regardless of measured water yield. If pressure went up to the specified maximum of 40 to 50 bar within 200 L of CS grout, the hole would be stopped on pressure. If steady pressure increase was observed and a bit more than 200 L of grout was needed to get to 50 bar, the hole would also stop on the pressure limit, or occasionally go to quantity stop. In the above cases, maximum benefit of the borehole would be achieved.

If about 200 L of CS grout could be pumped with no real increase in pumping pressure, an immediate switch from CS to MFC would be implemented (just disconnecting the CS-hose and connecting the MFC-hose for continued pumping with just a few seconds of stop). Typically, the cement grout would stop on pressure with a decent quantity of grout being placed but would sometimes even reach stop on quantity of cement.

This modified approach did not cause any over-consumption of expensive CS and clearly reduced the number of grouting stages needed to avoid water inflow from control holes triggering further grouting. It therefore also saved construction time and overall cost.

2.8 Grouting equipment

Both Contractors used equipment sets containing 3 parallel grout lines. 3 grout pumps could run simultaneously and could be individually supplied with grout as per requirements of the individual boreholes. Of course, the monitoring and logging of grouting parameters like pressure and flow rate was also executed in parallel. In daily operation, one or two pumps would mostly be used with one pump in backup. See Figure 7, showing the main grouting platforms.

One important difference between the two equipment packages was the dosing facility of the AMV-unit in CN24. It had a built-in dosing pump that was electronically tied in to the operating system so that dosage of an additive or accelerator could be fed to the grout at the packer at a pre-set percentage. When the dosage was set, the overall output could be changed without disturbing the pre-set dosage of e.g. accelerator. This single feature made it much easier and more efficient to handle accelerator dosage if it was necessary to temporarily use accelerator to provide blockage of cement grout backflow (backflow meaning loss of grout back to the tunnel through leakage in the face or elsewhere).

Contract 24 Contract 23

Figure 7. Main grouting equipments, each with 3 individual grouting lines.

At CN24 the dosage system was also used to pump CS in a true 2-component fashion. Rather than making batches of mixed component A and B for pumping through a single pump, they could use the cement grout-pump for the CS suspension (component A) and run the 10% salt water solution (component B) through the dosing pump to the packer. This way it was easy to modify setting time and other parameters during the process, without wasting material or being stuck with wrong setting times.

2.9 Ground water control results

2.9.1 Residual ingress limits

The five ingress limits mentioned above were allocated based on the local level of risk for surface settlement damage. The distribution within tunnel sections in granite and tuff can be seen in Figure 8 (the 650 m long section of 2.5 L/min/100 m is included with the 5 L/min/100 m section).

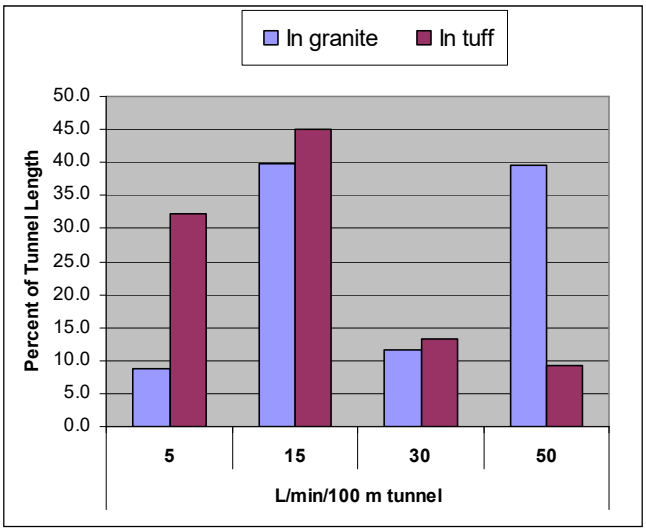

Figure 8. Ingress limit distribution in the two rock types.

It should be noted that the percentage of tunnel length with ingress limits of 15 and 30 L/min/100 m were roughly the same for the two rock types. Through volcanic tuff, 32% of the tunnel length had to satisfy 5 L/min/100 m or less, while in granite only 9% of tunnel length had this requirement.

The limit of 50 L/min/100 m applied to as much as 40% of tunnel length through granite, primarily due to the about 4000 m tunnel length under Victoria Harbour. In tuff, this relaxed limit applied to only 9% of the tunnel length.

As could be expected, the stricter ingress limits were more demanding and would typically require more drilling and more grout material (in otherwise equal rock conditions). When observing grouting results from HATS2A tunnelling, the above distribution of ingress limits should be kept in mind. In addition, the two rock types did in average present different levels of complexity regarding fulfilment of a given ingress limit.

2.9.2 Ingress limits and actual ingress

The specified residual ingress limits were allocated to varying lengths of tunnel. If the limits are weighted against the tunnel lengths on which they were applied and tunnel sections in granite and volcanic tuff are viewed separately, the *average* requirement for the two rock types were:

- For tunnelling in granite: 30.2 L/min/100 m
- For tunnelling in tuff: 16.9 L/min/100 m

What stands out is the large difference between the two average limits which was primarily caused by the 4 km of tunnel length in granite under Victoria Harbour, where the highest limit applied. This did of course lift the weighted average significantly for the excavation in granite.

The actually measured ingress average results also weighted against tunnel length and split on granite and tuff comes out as follows:

- Tunnel length in granite: 4.3 L/min/100 m (86% less than the limit)
- Tunnel length in tuff: 7.5 L/min/100 m (56% less than the limit)

In average, the measured ingress was significantly lower than the required limits, but in terms of avoiding surface settlement damage, satisfactory average might not necessarily be acceptable if shorter sections of tunnel had ingress above the local limit. In total, 6% of the tunnel length was in this category (failing the local ingress limit) and the corresponding ingress values were:

- Failed length in granite: 53.0 L/min/100 m
- Failed length in tuff: 21.6 L/min/100 m

The monitoring of piezometers and settlement markers around the tunnel sections with ingress above the limit did not indicate any problems and mitigation did not become necessary.

Overall, 96% of the 20 km of tunnel did satisfy the ingress limits by a wide margin, as shown by the average ingress in granite of only 2.7 L/min (required 30.2) and 6.0 L/min (required 16.9) all per 100 m of tunnel.

2.9.3 Resource consumption

Granite is a rock type that can be found on all continents and the typical characteristics are often quite similar. In terms of how granites behave when considering ground water and grouting, the same applies. Typically, there will be 3 close to orthogonal main joint sets and some random joints and of course with some variation in jointing intensity. It can be said that the granites of HATS2A are well within a normal range of variation for the rock type. This also applies to the amounts of drilling, injected cement (MFC) and the use of colloidal silica (CS). The overall numbers for ground water control in granite are given in Table 1.

Because the granite is such an abundant and normal rock type, the lower part of the table just illustrates how the numbers in volcanic tuff compares with the granite used as a yard-stick, to more easily see how different the volcanic tuff behaved.

Table 1. Consumed resources for GW-control in granite and tuff.

Resource consumption in **Granite:**	
Resource Item	**Per m Tunnel**
Drilling for probing and PEG (m)	21.6
Injected MFC (kg)	273.5
Injected CS (kg)	52.1
Resource consumption in **Tuff:**	
Resource Item	**% difference**
Drilling for probing and PEG (m)	+38
Injected MFC (kg)	-3
Injected CS (kg)	+370

What is significant is that despite 38% more drilling in volcanic tuff, the cement quantity placed was still 3% less than the recorded amount in granite. Furthermore, due to the average ingress limit being only 56% of the one in granite it became necessary to use 4.7 times more CS to satisfy the ingress limits.

2.10 Ground stability improvement

The combination of high groundwater pressure, highly water conductive ground and extremely poor and soft ground in shear zones can cause serious stability problems and significant construction time delay. Stage 1 of the sewage conveyance system finished over 10 years ago mainly on the Kowloon side, is one example of the above described difficulties. Located in basically the same rock formations, the Stage 1 suffered several face collapse situations that cost several months of delay time.

If an extremely poor shear zone like described above is excavated into without or with insufficient groundwater cut-off, hydraulic collapse and/or flushing and erosion can lead to a complete and rapid face area collapse and in a worst case, even piping that may connect to the sea bottom. To stop, control and stabilize such areas after the fact is both difficult and risky.

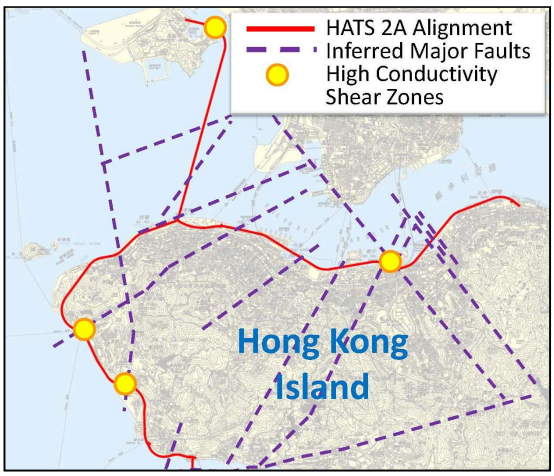

Figure 9. High Conductivity Shear Zones.

HATS2A has passed at least four zones (see Figure 9) that without proper PEG could have easily developed into similar situations as experienced in Stage 1. None of them caused loss of stability control at any stage. Of course, it cannot be positively proven that executed PEG can be fully credited with this favourable outcome, since nobody knows the ground behaviour in these locations without PEG.

However, the geological mapping in the tunnels document extremely poor ground and the water inflow measured through probe- and grout holes do support that if not sealed off, serious collapse would be quite possible, if not likely.

In this context, one should fully recognize the importance of high early strength of the MFC and the penetration capability of the CS. Without CS, there is a much higher risk of untreated “windows” in the ground surrounding the tunnel due to clay and other fines and when advancing the tunnel face, a local blow-out could happen. An extreme example of CS usage illustrating this point is described in Tunnel Business Magazine from the Arrowhead East tunnel in California [9].

3 CONCLUSIONS

Ground water control for the HATS2A project was required to prevent surface settlement and damage to structures in the densely built-up areas above the tunnel.

The water ingress limits were extremely strict, illustrated by the fact that 27% of the tunnel length allowed < 5 L/min/100 m of tunnel, which is unprecedented at 150 mbsl. The local combination of very sensitive structures founded on marine sediments and reclaimed land represented a high risk of damage.

These ingress limits could only be satisfied by using D&B excavation, since this allows efficient drilling of probe- and grout holes in required numbers, positions and angles. In total, 480 km of boreholes were drilled ahead of the tunnel face and with the required flexibility of drilling locations and angels actually used, it would be impossible to copy this in case of TBM excavation.

Furthermore, the choice of using exclusively MFC supplemented by CS was crucial. Combined with non-bleed mix designs and modern grouting procedures and equipment, even 10 cm wide open joints were successfully sealed off as well as intensely fractured zones with clay gouge in both granite and volcanic tuff.

Very important was the fact that drilling and grouting quantities were re-measurable, meaning that they were paid for by unit prices on actual quantities. This allowed continuous communication with the Contractors on the best approach for successful grouting, rather than by enforcing anything that would cost the Contractor extra.

20 km of tunnel were excavated and 94% showed measured ingress well below the ingress limits, in fact 86% less in the granite and 56% less in volcanic tuff.

Considering that the ground water control contributed to preventing face collapse incidents, and provided practically dry working conditions in the tunnels, it is considered fair to grade the ground water control works at HATS2A as highly successful.

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