Excavation of Dry Subsea Rock Tunnels in Hong Kong using Micro-Fine Cement and Colloidal Silica for Groundwater Control

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ABSTRACT: The 20 km of HATS2A sewage conveyance tunnels excavated by drill & blast are placed from 70 to 160 m below sea level and pass underneath the Victoria Harbour shoreline of Hong Kong Island. Extremely strict limits on residual groundwater ingress were imposed to avoid surface settlement and damage, requiring Pre-Excavation Grouting (PEG) to limits between 2.5 and 50 L/min/100 m of tunnel. 89% of the excavated 15 km shows less water ingress than the local limit value. This has been achieved by continuous probe drilling, grouting with micro fine cement only, supplemented with colloidal silica. The implemented overall PEG approach represents a unique combination of elements described in detail in this paper and in references. Of the two main rock types (granite and volcanic tuff), the tuff presented the more demanding case. In tuff, substantially more drilling and grouting materials were needed to reach the targeted tightness of the ground.

1 INTRODUCTION

In tunneling, groundwater ingress control is often necessary to restrict draining of the surrounding ground, thus preventing settlement damage on surface. Installation of a final lining may take too long time to sufficiently limit the volume of water ingress to the tunnel, even if it can be successfully made watertight.

Pre-Excavation Grouting (PEG) on the other hand offers effective pre-treatment of the ground before mining into permeable rock zones and the overall water ingress can be strictly limited to pre-set specifications. However, success requires the applied PEG approach to be an integral part of the excavation process. Furthermore, selected materials, equipment and works procedures must be based on the latest technology proven to work as intended.

This paper describes the groundwater control experiences made during excavation for the Harbour Area Treatment Scheme Stage 2A project (HATS2A), consisting of 20 km of subsea tunneling for sewage conveyance. Additional details are presented by Garshol et al., (2012 and 2013). Excavation is done by drill and blast and the tunnel system is placed from about 70 to 160 m below sea level.

Of the 20 km HATS2A tunnels, about 34% goes through volcanic tuff, while the other 66% traverse primarily granitic rocks (Metcalf & Eddy – Maunsell Joint Venture. 2009). These two main rock types present quite different joint patterns, joint frequencies and water conductivity properties, thus requiring different practical PEG approach.

At the end of September 2013, about 75% of the tunnel length has been excavated.

1.1 HATS2A Project overview

Apart from the tunnel crossing underneath Victoria Harbour, all of 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 highly sensitive to groundwater drawdown that may cause surface settlement. This 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 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. Layout plan of HATS2A Tunnels.

During the project design stage, hydrogeological numerical modeling was used to analyze the maximum allowable residual ingress to the tunnel (after PEG) in order to avoid surface damage by settlement. The selected ingress classes range from 2.5 L/min/100 m of tunnel for the most sensitive area 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.

1.2 Design of PEG works

Once the targeted residual groundwater ingress limits are decided, one major problem remains: How to design the PEG execution to hit the target as closely as possible.

In the case of HATS2A, this very elaborate process has been described by Tattersall et al. (2012). The basis for the referenced paper is an extensive database of core logs, tunneling data, recorded and inferred weakness zones and water loss testing both for volcanic tuff and granite areas.

In combination with this huge Hong Kong database, containing 20,000 core logs as just one part, also the Norwegian principle of Pre Grouting Intensity Classes (PGIC) was employed. This way, a prognosis for the necessary drilling ahead (probe holes, grout holes and control holes) and consumption of grouting materials could be worked out. Water inflow limits for probe holes and control holes that would trigger further grouting under the various targeted residual ingress limits were also specified.

As an example, probe holes or control holes within a tunnel section with targeted residual ingress of 15 L/min/100 m of tunnel would trigger grouting if any single hole yielded > 1.0 L/min or any group of 4 holes made > 3 L/min.

The above residual ingress limits may for many readers not intuitively illustrate the practical result in the tunnel. Ingress of less than 15 L/min/100 m means a practically dry tunnel where an occasional drip and some humid spots in the shotcrete can be observed. See Figure 2.

Figure 2. Ingress < 5 L/min/100 m in granite.

2 PEG EXECUTION

The practical process of PEG execution for this project mainly consists of well known elements and has been described in detail by Garshol et al. (2012). However, the overall combination of elements 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 is four.

When the water inflow from probe holes triggers grouting, micro fine cement (MFC) is always the primary grouting material of choice. No ordinary Portland cement (OPC) is used.
This choice may seem exaggerated to many, but the higher cost per kg pays off well in improved efficiency, less drilling of grout holes and improved execution control and sealing effect.

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. However, for target ingress limit of 15 L/min/100 m or less, it becomes more efficient to use colloidal silica (CS) as a supplement. In complicated ground conditions, CS may even be beneficial at higher targeted ingress limit.

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

Furthermore, modern grouting equipment is 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 is required.

After all stages of grouting, the result must be checked by drilling control holes to measure the water inflow to these holes. No excavation is allowed until PEG result is verified to be acceptable by measured inflow to control holes.

2.1 Observational design update

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

If residual ingress is too high, it is possible that post grouting and/or water re-charge must be employed to mitigate the problem.

The other consideration is about not yet excavated tunnel length. Modifications to grouting trigger limits and other aspects of the execution may be made to achieve results closer to the target.

Complicated shear zones within sections of very strict ingress limit have turned out to represent the biggest challenge. The risk of encountering ingress above the limit seems higher under such circumstances.

3 ROCK CHARACTERISTICS

The three main geological units found along the tunnel alignment include the Kowloon Granite, and the volcanic tuff of Mount Davis Formation and Ap Lei Chau Formation. In general, granitic bedrock has fewer effective joint sets and is relatively more susceptible to deep weathering and hydrothermal alteration than the volcanic rocks. Micro fracturing can be extensive near faults and within 100 m of rockhead due to stress-relief effects (GEO, 2007).

Metcalf & Eddy – Maunsell Joint Venture (2009) and Tattersall et al. (2012) discussed that the effective transmissivity of discontinuities at any location can vary by about four orders of magnitude, making it impossible to predict reliably the rate of tunnel inflow and equivalent mass permeability in any small segment of tunnel. Therefore, it is expected that the full range of transmissivity indicated by the Lugeon distribution plots from all the collected relevant site data will be encountered. Allowance for this vast range must be made when assessing potential groundwater inflows and groundwater control strategies.

3.1 Granite

Kowloon granite is the main rock type encountered in the HATS2A project. It consists of generally medium grained granite. This granitic bedrock contains about 10% by volume of pegmatite veins, dyke intrusions and metamorphosed tuff xenoliths as well as quartz and calcite veins. The secondary dykes and veins tend to be more common near faults. At tunnel depth and outside the influence of faults and zones of hydrothermal alteration most of the granitic bedrock shows generally decomposition (weathering) of Grades I and II, which means from no visible to slightly decomposed rocks as defined by GEO (2000) with relatively uniform material characteristics over long distances.

Based on the available ground investigation data, the 50-percentile unconfined compressive strength value of the granitic rock is 180 MPa.

3.2 Groundwater control in granite

Some tunnel sections have passed through excellent quality, almost massive granite where no grouting is required even for the strictest ingress limit. At the other end of the scale, broken rock has produced several probe holes in
the same screen each one yielding over 400 L/min (maximum 1000 L/min) and many of the following grout holes giving more than 100 L/min as illustrated in Figure 3.

Figure 3. Probe hole hitting 15 bar water channel.

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

Another variant is the shear zones with crushed and highly weathered rock. Clay gouge and other fines are typically present, causing problems for grout penetration and sufficient grout spread.

In a normal 20 m long borehole in virgin ground, the water conductivity contrast is mostly very high. This means that along the length of the hole, there will typically be one or very few channels that generate most of the water. This has the effect that most of the grout will take the path of least resistance and frequently the grouting will stop on quantity limit at low grouting pressure. Even though such cases often lead to more than one grouting stage, it is quite easy to bring large original ingress flows in fresh granite down to practically nothing.

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

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

3.3 Volcanic tuff

The two geological formations of volcanic rocks include the Mount Davis Formation consisting of predominantly volcanic coarse ash crystal tuff with subordinate fine ash tuff and meta-sedimentary rocks, and the Ap Lei Chau Formation consisting of predominantly volcanic fine ash vitric tuff with eutaxitic layers, coarse ash tuff, thin lava and epiclastic layers.

The volcanic rocks contain about 15% by volume of dyke rock intrusions, pegmatite veins and veins of quartz and calcite. The secondary dykes and veins tend to be more common near faults and contacts with the main intrusions of granitic rock. At tunnel depth most of the volcanic rock outside the influence of faults and zones of hydrothermal alteration shows decomposition (weathering) Grades I and II.

The volcanic rocks are highly variable with regard to jointing intensity and orientation of joints and bedding. This makes it more difficult to maintain optimal excavation rates than when tunnelling in granitic bedrock.

Based on the available ground investigation data, the 50-percentile unconfined compressive strength value of the volcanic rocks is 240 MPa.

3.4 Groundwater control in tuff

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

The typical joint spacing is very close and most joints are of short extension and with a fair share of random orientation. The resulting joint apertures become very small. The water ingress to a borehole may 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 will yield water inflow of 100 L/min and more to single boreholes.
Typically, there is a significant tendency for holes grouted with MFC to rapidly go to maximum pressure within a relatively small grout quantity. Especially in areas of strict ingress limit (5 L/min/100 m), this has 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 is that the consumption of MFC is much lower than expected and specified.

Like in granite, the main problem has not been 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 becomes a problem, CS has turned out to be an invaluable supplement.

Also in tuff disturbed by faulting, the use of double overlap grout fans was implemented locally.

4 SELECTION OF GROUT MATERIAL

4.1 Basic strategy

Along all of the tunnel system, two grout materials were available for use: A 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 and the 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.

However, when most grout holes would spike to maximum grouting pressure at very small quantity, this would often not provide the required 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 becomes very small. The high strength of cement is then important for improvement of ground stability, but also to avoid that new holes deviate to merge into a previously drilled and grouted hole. With the weak gel of CS filling previous holes, this would often happen 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 actually achieve the target while reducing drilling of boreholes. Use of CS would therefore reduce the time spent for a complete grouting station.

4.2 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.

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 could build no pumping pressure and the grout would just “run away” to be stopped on quantity.

Based on continuous evaluations of performance to find the right balance, the selection limit could be as high as 10 L/min within the volcanic rocks, where MFC would be blocked prematurely far too frequently at lower inflow numbers.

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 a case).

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 happens, the
penetration efficiency of the CS is not utilized and the hole should have been grouted by MFC.

4.3 Modified selection approach

The primary problem 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.

The above mentioned problems are of course linked to the normal conductivity contrast along a 20 m long borehole. We can measure the water inflow, but we do not know whether this water comes from very few and large channels or a high number of small cracks and joints. The water yield from a drilled hole therefore turns out to be a very unreliable parameter for the selection of grouting material to use in a given borehole.

Especially in the tuff, the above described problem was frequently encountered. The successful solution consisted of starting all grout holes by CS regardless of water yield. If pressure went up to the specified maximum of 40 to 50 bar within 200 L of grout, the hole would be stopped on pressure. If steady pressure increase would require a bit more than 200 L of grout, the hole would also stop on the pressure limit, or occasionally go to quantity stop. In the above cases, maximum benefit of the hole would be achieved.

If about 200 L of CS grout could be pumped with no significant increase in pumping pressure, an immediate switch from CS to MFC would be done (disconnecting the CS-hose and connecting the MFC-hose for continued pumping without any 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.

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.

5 RESULTS IN TUFF AND GRANITE

5.1 Residual ingress limit fulfillment

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 4 (the 2.5 L/min/100 m section is included with the 5 L/min/100 m section).

It should be noted that the percentage of tunnel length with ingress limits of 15 and 30 L/min/100 m are roughly the same for the two rock types.

Through volcanic tuff, 32% of the tunnel length has to satisfy 5 L/min/100 m or less, while in granite only 9% of tunnel length has this requirement.

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

As can be expected, the stricter ingress limits are more demanding and will typically require more drilling and more grout material (in otherwise equal rock conditions). When observing grouting results from the first 75% of HATSA2 tunneling, the above distribution of ingress limits should be kept in mind. In addition, the two rock types do in average present different levels of complexity regarding fulfillment of a given ingress limit.

For all tunneling executed so far (tuff and granite), 89% of the tunnel length have actual residual ingress less than the limit. In tuff, this
success rate drops to 82% while granite separately shows 92% for measured sections. This is a reflection that relatively more tunnel length must satisfy 5 L/min/100 m in the tuff and the fact that tuff in itself is more demanding to seal off. Still, all the 650 m of tunnel length with the strictest ingress limit of 2.5 L/min/100 m through volcanic tuff, do successfully satisfy this limit (with some help of good rock conditions).

For an overview of measured residual ingress results compared with required maximum ingress limit, see Table 1. The ingress numbers are averages weighted by tunnel length within the different targeted ingress limits and measurement data available per 18 October 2013. The final numbers will probably be slightly different.

Table 1. Residual ingress overview

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Granite Avg L/min/100 m</th>
<th>Tuff Avg L/min/100 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ingress limit less than</td>
<td>30.2</td>
<td>16.2</td>
</tr>
<tr>
<td>Actual overall ingress</td>
<td>5.2</td>
<td>7.9</td>
</tr>
<tr>
<td>Sections within limit</td>
<td>1.7</td>
<td>4.6</td>
</tr>
<tr>
<td>Sections failed</td>
<td>44.0</td>
<td>22.6</td>
</tr>
</tbody>
</table>

Measurement of residual ingress is mostly made in 100 m tunnel segments and for sections leaking more than the limit, two actions may be initiated. 1) Evaluation of groundwater piezometers and settlement instruments to decide about possible need for mitigation measures and 2) evaluation of the need for adjustment of the PEG execution for tunnel parts not yet excavated (to avoid again missing the target). Only a minor tunnel section in granite has been post grouted.

5.2 Quantities consumed

Granite is a pretty abundant rock type around the world and using this as a benchmark, we can note the following quantity numbers for HATS2A as found in Tables 2 and 3.

Table 2. Consumption in granite

<table>
<thead>
<tr>
<th>Resource Item</th>
<th>Per m tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling for PEG (m)</td>
<td>18.2</td>
</tr>
<tr>
<td>Injected MFC (kg)</td>
<td>234.4</td>
</tr>
<tr>
<td>Injected CS (kg)</td>
<td>88.7</td>
</tr>
</tbody>
</table>

The weighted average of tunnel sizes in granite represents a blasting line circumference length of 13.3 m.

The circumference length for tunnels in the volcanic tuff is in average 15.2 m, or 14% longer. If we harmonize the numbers of actual consumption in tuff to the same size of tunnel as in granite, the comparable figures are presented in Table 3.

Table 3. Consumption in tuff compared with granite

<table>
<thead>
<tr>
<th>Resource Item</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling for PEG (m)</td>
<td>+61</td>
</tr>
<tr>
<td>Injected MFC (kg)</td>
<td>+17</td>
</tr>
<tr>
<td>Injected CS (kg)</td>
<td>+165</td>
</tr>
</tbody>
</table>

The increased consumption in tuff as shown in Table 3, even though corrected for tunnel size, includes the extra resources required due to 3 times higher percentage of tunneling within the two strictest ingress limits. On equal terms, the differences would be somewhat smaller.

What still stands out as significantly different is the consumption of CS. Because of the predominance of highly fractured ground, where a large number of narrow cracks and joints cause limitation to penetration by MFC, the use of CS became very high. 52.1% by weight of total grouting materials in tuff was CS.

5.3 Accelerated grout

If the grout is flowing back into the tunnel together with pressurized running water, highly accelerated grout may help stop the grout loss. This technique is also useful for control of grout spread and to build pressure when encountering very large channels.

The accelerator must be added to the cement grout at the packer through a separate hose and a Y-piece with non-return valve. When pumping accelerated CS, 2-component pumping should be considered rather than working with batches. Furthermore, 2-component fast foaming Polyurethane (PU) can be used to block concentrated water leakage at the face.

Prevention is still far better than having to use the above solutions. The target must always be to keep a tight bulkhead in front of the tunnel face where packers can be placed, allowing all the pumped grout to spread into the rock forward of the face and not be wasted on the tunnel invert.
5.4 Ground stability improvement

It is a well-known fact that the combination of high groundwater pressure, highly water conductive ground and extremely poor and soft ground can cause serious stability problems and significant construction time delay.

Stage 1 of this 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 face collapse situations that cost several months in delay time.

Figure 5. High Conductivity Shear Zones.

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. To stop, control and stabilize such areas after the fact is both difficult and risky.

HATS2A has so far passed at least four zones (Figure 5) that without proper PEG could have easily caused 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 credited with this favourable outcome, since nobody knows the ground behaviour in these locations without PEG. However, the geological mapping documents extremely poor ground and the water inflow measured through probe- and grout holes do support that if not sealed off, a serious collapse would be quite possible.

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 blow-out could have happened. An extreme example of CS usage illustrating this point is described by Garshol in Tunnel Business Magazine (2007) from the Arrowhead East tunnel in California.

6 CONCLUSION

Based on the experience from Stage 1 sewage conveyance system tunneling, the groundwater control approach for HATS2A has been completely re-engineered. Partly because of the high priority and strict demands on low residual ingress to the tunnels, drill and blast excavation was decided, allowing the necessary freedom to drill ahead as required.

In spite of some early doubt as to whether the strictest ingress limits could be achieved, the PEG works have successfully satisfied the requirements. About 90% of the excavated tunnel length show less water ingress than the targeted maximum.

The successful groundwater control can be attributed to a unique combination of some important elements:

- Continuous probing ahead.
- All cement grouting done with MFC.
- Dual stop criteria and very high stop pressure.
- Grouting by CS where MFC cannot penetrate (amounting to 40% by weight of total grouting materials consumed).
- All grouting stations verified by control holes before further excavation.

PEG in volcanic tuff required more drilling and substantially more grouting materials to achieve the targeted results.

The fact that about 15 km of tunneling have been executed without any face collapse or flushing/erosion situation indicates that the strict groundwater control has significantly improved the overall stability and open stable time in zones of extremely poor ground.
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