Observational Method for Ground Treatment of Tunnel Cross Passages in Complex Ground Conditions

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ABSTRACT

This paper focuses on the design and review of the ground treatment and rock fissure grouting required to excavate tunnel Cross-Passages in the Liantang / Heung Yuen Wai Boundary Control Point Site Formation and Infrastructure Works – Contract 2 in Hong Kong SAR. The Cross-Passages were expected to go through Tuff in various degrees of weathering (Grade V to Grade III/II). The Site Investigation, SI, showed that SPTs numbers generally ranged from 30 to 50 for the Completely to Highly Decomposed Tuff, CDT / HDT, with localised values as low as 6. Ground Treatment consisting of permeation and rock fissure grouting as well as 120° pipe roof / canopy tubes, was required to ensure not only the stability during excavation but also limit the groundwater inflow. The SI determined in-situ permeabilities ranging from 1x10-5 to 1x10-6 m/s for the CDT and a 21m long probe hole recorded a water inflow in excess of 60 l/minute. A discussion relative to the methods employed for drilling, e.g., pressure balance drilling system, drilling alignment tools used, and grouting techniques, e.g., microfine cement, chemical grout is presented in this paper. The use of drilling survey tools integrated with 3D representation models of the cross-passage and the ground treatment is discussed. A review of the overall performance of the Cross-Passage, e.g., groundwater inflow, stability, is undertaken.

Keywords: Observational Method, Ground Treatment, Cross-Passages

1 Introduction

The Liantang Heung Yuen Wai Boundary Control Point project mainly comprises of site formation of about 23 hectares of land for provision of Boundary Control Point (BCP) buildings and associated facilities. As well as construction of about 11km long dual 2-lane Connecting Road linking up the BCP with Fanling Highway. Integrated in the main project was the Site Formation and Infrastructure Works – Contract 2. This contract comprises the design and construction of a dual two-lane trunk road. This includes an approximately 4.8km long tunnel linking up the proposed Sha Tau Kok Road interchange and Fanling Highway interchange, three ventilation buildings, and an administration building. The project reduced travel times from Fanling Highway to Ping Che area and Heung Yuen Wai, Ta Kwu Ling from 15 and 24 minutes, to 4 and 8 minutes respectively. A combination of Tunnel Boring Machine (TBM), and drill and blast solutions were adopted, to cope with various ground conditions along the 4.8km tunnel route. A dual mode Earth Pressure Balanced TBM was used for the construction of the tunnels in the northern section, where various fault zones were located. Meanwhile, drill and blast techniques were implemented in rock zones in the southern section.

A total of 49 Cross Passages (CPs) were required at maximum 100m intervals to comply with local regulations in Hong Kong. Of all the tunnel CPs four of them were expected to be the most challenging to build due to the ground conditions. These CPs were expected to go through Tuff in various degrees of weathering (Grade V to Grade III/II). CP No. 35, 37 and 40 were expected to be in mixed ground conditions (soil / rock) whereas CP42 was expected to be fully in Completely to Highly Decomposed Tuff, CDT / HDT. The ground investigation determined permeabilities ranging from $1x10^{-5}$ m/s to

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1×10^{-6} m/s within the CDT while the HDT recorded permeabilities in the range of 3×10^{-5} m/s and 2×10^{-7} m/s. The Particle Size Distribution analysis from samples collected at CP42 determined that the CDT / HDT was classified mainly as sandy Silt with fine content ranging from 39% to 55%. Traditionally these ground conditions are not suited for permeation grouting and designers and contractors are led down the path of more expensive ground treatment techniques. These techniques usually range from jet grouting done from the surface or ground freezing if the works are done from within the tunnel. Considering that the CPs were 35m to 50m deep it was decided that the ground treatment would be undertaken from within the South-bound tunnel towards the second TBM drive, North-Bound. This influenced the geometry of the treatment since it would not follow the more common approach of drilling from both tunnels towards the middle of the CP. The programme of works was also stringent since the ground treatment works would have to be completed before the second TBM drive. This meant that the last accessible CP from the first TBM drive would be the first CP the North-Bound tunnel drive would encounter. The internal diameter of the tunnel was 12.6m and an approximately 4.0m wide permanently open lane was required to ensure that the other associated tunneling works could continue without disruption. Therefore, the available working space for the ground treatment works was greatly limited.

An innovative observational approach for these ground and site conditions was envisaged as the ground treatment solution. The ground treatment consisted of permeation grouting in soil and rock fissure grouting as well as a 120 degrees pipe roof / canopy tubes.

2 Site Description and Ground Conditions

2.1 Site Description

The site is located in the upper Northeast of the Hong Kong SAR near the border with Shenzhen close to the town of Sha Tau Kok. This location is the most northern on the project and where the TBM is launched from and received for the 2 main drives. Additional drill and blast excavation occurred from the Southern portal and at Mid-vent portal to excavate a cavern for TBM turnaround. The approximately 4.8km tunnel was at the time of construction the longest land-based road tunnel in Hong Kong.

The ground treatment for the four CPs was carried out from inside the South Bound tunnel and completed ahead of the North Bound tunnel drive. The CPs were 35m to 50m deep and their length varied from approximately 16m to approximately 18m. The works were completed concurrently with the ongoing tunneling works for the South Bound, which required an approximately 4.0m wide permanently open lane. The tunnel had an internal diameter of 12.6m and therefore the working space was greatly limited.

2.2 Ground Conditions

Site investigation campaigns identified that the four CPs would go through Tuff in various degrees of weathering (Grade V to Grade III/II). The Completely to Highly Decomposed Tuff, comprised of medium dense to dense silty / clayey Sands and stiff to hard sandy Silts / Clays, generally recorded Standard Penetration Tests, SPT, ranging from 30-50. However, localized SPT values as low as 6 were recorded indicating that less dense / softer pockets of material existed along some of the CP's alignment. Figure 1 shows the Particle Size Distribution, PSD, of samples collected at CP42. The PSD determined that the CDT / HDT was mainly sandy Silt, slightly gravelly at some of the samples, and with fine content ranging from 39% to 55%.

CPs Nos. 35, 37, and 40 were in mixed ground conditions, soil / rock, with estimated percentages of soil vs rock varying from 15% to 30% at CP37 and CP40, respectively. During the South Bound TBM drive the geological inspections confirmed the absence of grade V Tuff, CDT. Therefore, the ground treatment for these CPs would consist of rock fissure grouting only. The grade IV Tuff, Highly Decomposed Tuff, is not considered rock for structural design purposes; however, for grouting works it's expected that the grout injection will displace the water through fissures rather than through the soil mass. Therefore, from an injection / grouting point of view the HDT was expected to behave as a weak rock. CP42 was expected to be located completely within the CDT / HDT zone, see Figure 2, and the groundwater table was 25m above the crown of the CP.

Falling head tests undertaken at the crown of the CPs, within the CDT, determined permeabilities ranging from $1x10^{-5}$ m/s to $1x10^{-6}$ m/s while complementary ground investigation recorded permeabilities in the range of $3x10^{-5}$ m/s and $2x10^{-7}$ m/s for the HDT. A 21m long probe hole was performed at CP42 prior to the ground treatment and a water inflow larger than 60 l/min was recorded.

These ground conditions usually lead designers and contractors to either ground freezing or ground mixing techniques from the surface, e.g., jet grouting. This is due to the difficulty of employing permeation grouting techniques in a satisfactory manner.

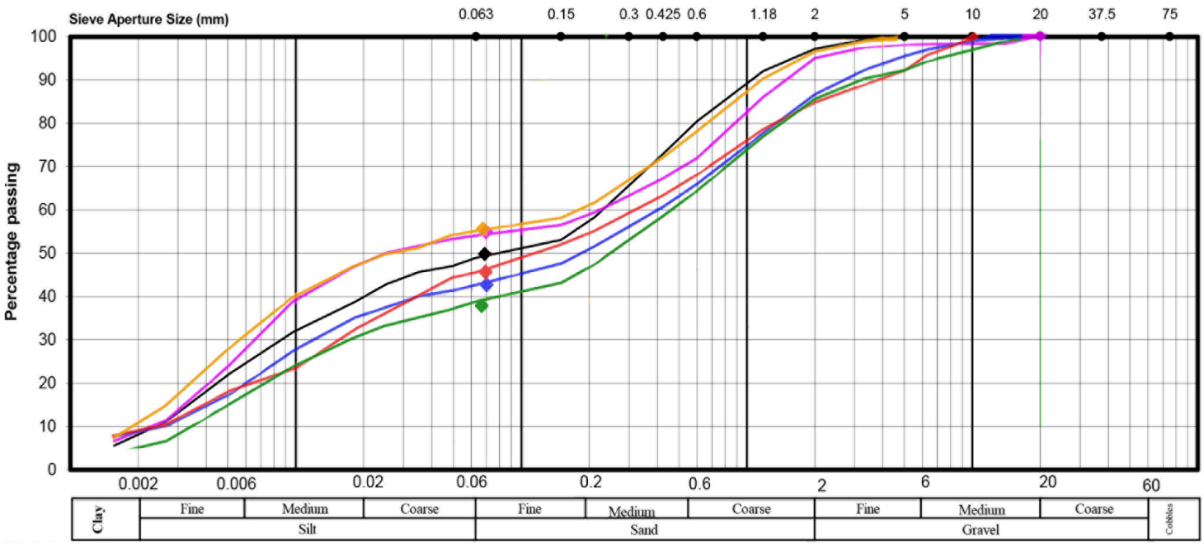

Figure 1: Particle Size Distribution from Samples at CP42

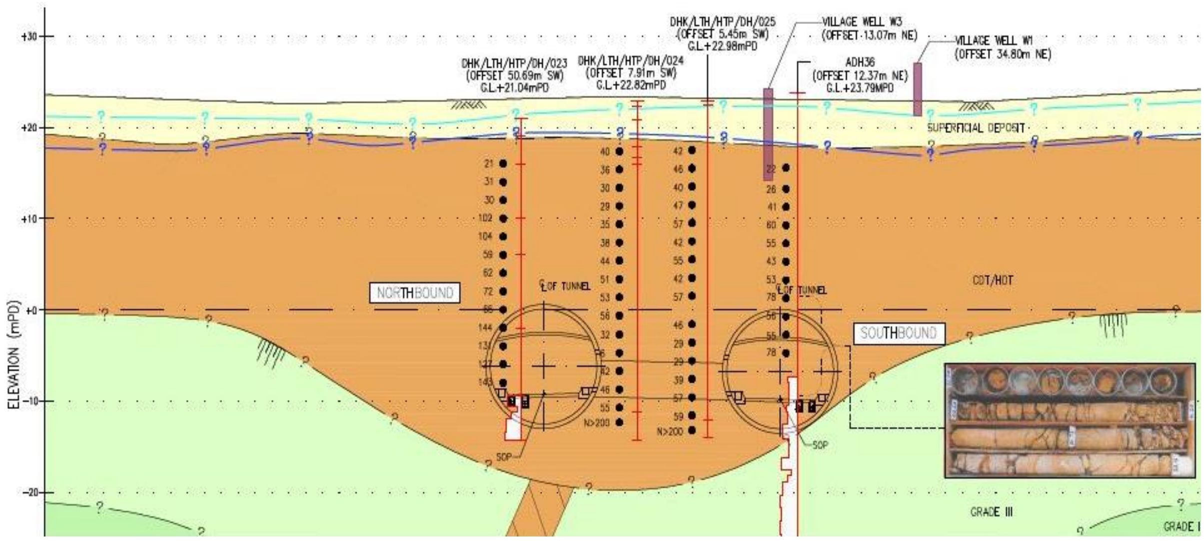

Figure 2: CP42 Geological Section

3 Ground Treatment Design

Considering the size of the tunnel, internal diameter of 12.6m, and the depth of the Cross Passages, from 35m deep to 50m, it was decided that the most effective option was to carry out the ground

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treatment from within the tunnel, South-bound tunnel towards the North-Bound. This influenced the geometry of the treatment since it would not follow the more common approach of drilling from both tunnels towards the middle of the CP. However, it had the programme advantage of allowing the CP excavation to start right after the second TBM drive. CP's length varied from approximately 16m to approximately 18m while drilling lengths and correspondent ground treatment varied from 20m to 23m long. Two types of grouting were envisaged, permeation grouting for the soil and rock fissure grouting. Considering the insitu permeability results as well as the PSD curves the ground treatment was designed as a combination of Microfine Cement and Chemical Grout, grouted at pressure using the Tube a Manchette, TAM, grouting method with double packers. The Microfine Cement was envisaged to fill the voids in the soil mass while the Chemical Grout was meant to displace air / water from the inter particle pores. The Chemical Grout mix was designed to achieve a viscosity similar to water so that its penetration radius could be increased. This was achieved by reducing the amount of sodium silicate favoring the ability of the treatment to reduce permeability by sacrificing strength. The mix was tested with soil collected from SPT samplers from boreholes undertaken in the vicinity of CP42 and adjusted to meet optimum results. Chemical Grout mix was only required at CP42 due to its geological / geotechnical conditions. The ground improvement for the other CPs was undertaken with MFC only.

There were two discrete parts to the treatment, the "plug" (Stage 1 & 2), see Figure 3 and Figure 4 and the "tube" (Stage 3a & 3b), see Figure 5 and Figure 6. The future North-Bound Tunnel, second TBM drive, is shown to indicate the relative position of the ground treatment. The purpose of the "tube" is to encapsulate the CP while the "plug" is meant to seal the distal end. The "plug" was designed to act as a bulkhead providing a seal from water ingress and stabilizing the ground mass against collapse whilst securing the formwork for the concrete bulkhead for TBM passage. Meanwhile the southbound permanent opening remains open for excavation.

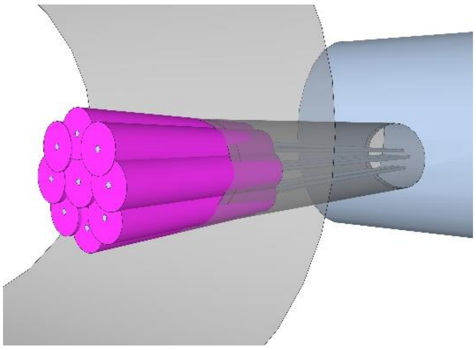

Figure 3: Stage 1 (inner plug)

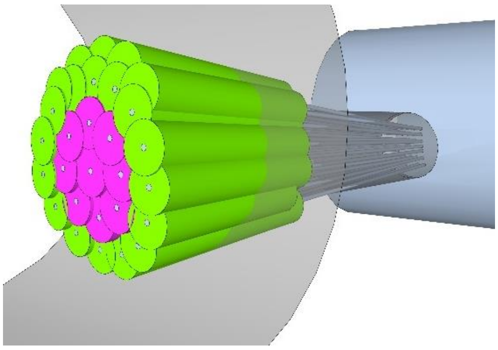

Figure 4: Stage 1 (outer plug)

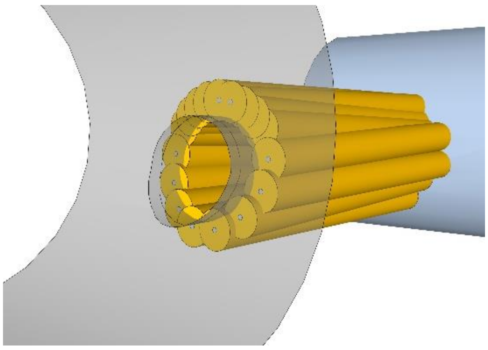

Figure 5: Stage 3a (inner tube and canopy tube / roof)

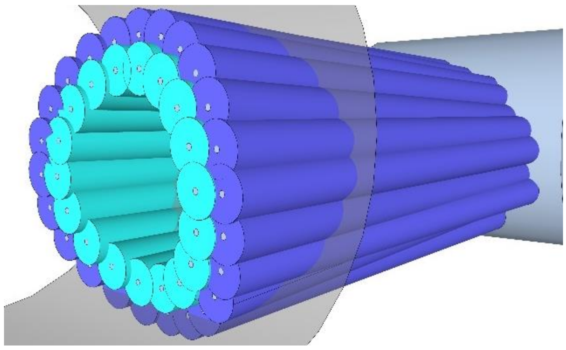

Figure 6: Stage 3b (Ring 2 and 3) (outer tube)

The combined treatment meant to create a 3m thick effective grout envelope around the whole excavation with a 120° pipe roof / canopy tubes crown. The later meant to ensure the stability of the ground during the excavation stage while the grouting works main objective was to limit groundwater inflow to the excavation. The canopy tubes were designed to terminate at least 1.0m away from the future position of the Northbound tunnel lining. Stages 1 and 2 were designed to depart from the permanent CP opening and radiate outwards towards the North-bound tunnel. Stages 3a and 3b were designed approximately parallel to the CP alignment subject to the segment drilling limitations. As discussed in Point 2.2 above it was assumed that grouting in grade IV or better rock (HDT) would behave like Rock Fissure Grouting. A triangular pattern was designed spaced at 2.4m center to center and assumed that the grouting could penetrate a radius of 1.6m. The pattern assumed that each drill hole would achieve a 800mm grout overlap with the adjacent holes, see Figure 7. For grade V rock (CDT) the triangular pattern spacing was reduced to 1.2m with a grout penetration radius of 0.8m per hole, see Figure 8. The assumed grout overlap between adjacent drillholes would be of 400mm. The ground treatment design is highly dependent of the assumed grout penetration radius as this drives the ground treatment pattern. At an initial stage the design assumed a certain degree of conservatism into the grouting pattern that was to be validated by site trials.

The segment coring / drilling was set out with the objective of creating the least damage to the permanent segmental concrete lining. To achieve this “no drill zones” were specified by the main contractor’s designer in view of maintaining both the watertightness of the tunnel and the structural integrity of the lining. In addition, it was specified that the drill holes maximum diameter was limited to 125mm and that they could not be spaced less than 300mm in any direction to ensure that no consecutive steel reinforcement bars were damaged. These requirements increased the drillholes spacing, which then caused extra drillholes to be added to the initial scheme so that the ground treatment envelope could be ensured.

The criteria for grouting were based on an observational approach, which depended on volume and pressure, and was verified using the methodology described in Point 5.1.

Table 1 summarizes the grouting criteria where the volume represents a percentage of the treated ground envelope. In soil the assumed grout intake is the sum for both Microfine Cement and Chemical Grout.

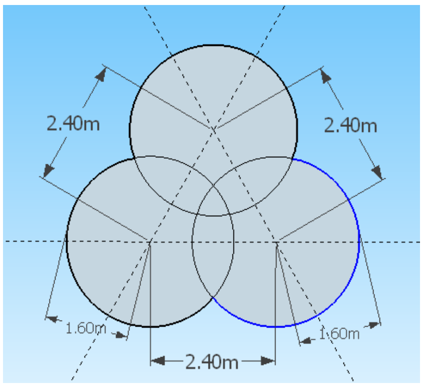

Figure 7 – Ground Treatment Pattern for Grade IV or better Tuff

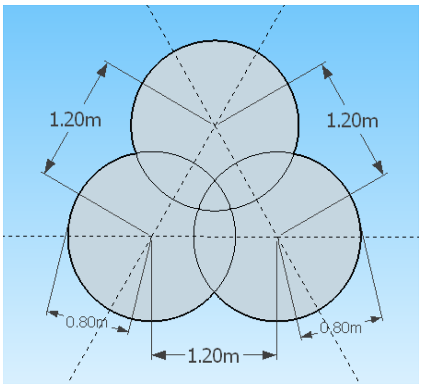

Figure 8 – Ground Treatment Pattern for Grade V Tuff

Table 1: Grouting Criteria per Treatment

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Grouting Method	Grout Type	Pressure (bar)		Treated Volume (%)
		< 5.0m behind tunnel lining	≥ 5.0m behind tunnel lining	
Rock Fissure Grouting	Microfine Cement Mix	15	25	10.6
Permeation Grouting (TAM)	Microfine Cement Mix	10	15	17.4
	Chemical Grout Mix	10	20	18.3

The grouting sequence was planned from the bottom of the CPs to the top and from the inside out with the inner rings being grouted before the outer rings. This was meant to push the water away from the future alignment of the CP / future excavation zone. The last stage of grouting was done through the canopy pipes at the top of the excavated CP crown. Canopy steel pipes with 70mm outer diameter and 20mm thick were installed in all CPs (35, 37, 40 and 42). The PVC TAM pipes were smaller in diameter with only 60mm but maintained the same inner diameter of the steel pipes since they were only 10mm thick.

Prior to the execution works a site trial consisting of 3 nos. of 4.0m long holes were drilled and grouted to confirm the suitability of the ground treatment and the Chemical Grout mix. A verification hole was carried out in the middle of the treatment to validate the suitability of the grout pattern to limit the groundwater inflow. From the trials it became apparent that the high temperatures, both ambient temperature inside the tunnel and the temperature of water being supplied, were negatively impacting the gel time of the mix. Therefore, new tests were done using an alternative supply of water, that was chilled before use and with a temperature of approximately 7°C, and using a refrigerated container to store the other mix constituents. The results from the trial were satisfactory at increasing the gel time and both measures, cooled water and refrigerated container for the Silicate and hardener, were implemented.

4 Site Implementation

All drilling and grouting were completed from the South-bound tunnel towards the Northbound tunnel, while keeping circulation in the South-bound tunnel for its other tunneling associated works. This required advance planning for the site layout and testing of different configurations using 3D models, see Figure 9. The works were carried out from July 2016 to July 2017. The treatment envelope required the drilling to be both executed from the backfilled tunnel invert level and using an elevated platform, see Plate 1 and Plate 2.

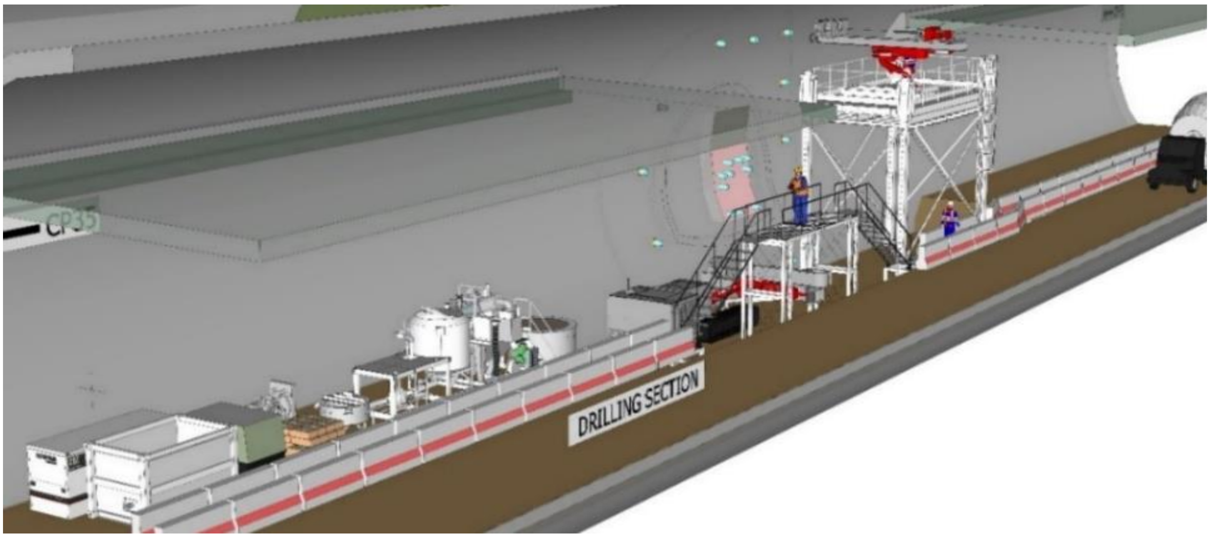

Figure 9: Planning Stage of Site Layout

Plate 1: CP42 Drilling Works from Tunnel Invert Level

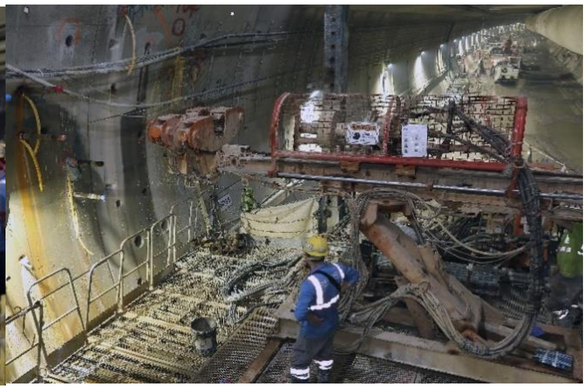

Plate 2: CP42 Drilling Works from Elevated Platform

Variations in ground conditions throughout the Cross Passage would impact the grouting design assumptions and the grouting technique (Permeation Grouting or Rock Fissure Grouting) as detailed in Point 3. Therefore, the drilling operations were continuously monitored to access the drilling spoil and drilling parameters, e.g., drilling speed, drill bits abrasion, and verify if the assumed geology was correct. For CP42 bentonite was used for drilling stability, generally up to 4m before the end of the envisaged drilling hole when it was replaced by a sleeve grout mixture. For the other CPs the drilling medium was water since the drilling did not encounter Grade V Tuff. Once the sleeve grout was applied and drilling finished the rods would be extracted and the Tube a Manchette installed. The sleeve grout mix contained bentonite to ensure its reduced strength when compared to the MFC and therefore, not impart the grouting activities. Grouting using a double packer in either PVC, Plate 3, or Steel TAM pipes with sleeves at 0.33mm centers would then be carried out, with the latter only used for the canopy.

Investigation holes were drilled under Blow Out Preventors, BOP, Pressure Balance Drilling for CPs 35, 37 and CP40 to confirm that there was no adverse impact to existing ground conditions. The existence of grade V Tuff had been ruled out through geological inspection during the Southbound TBM drive and therefore, the rock fissure grouting technique was validated for these three CPs ground treatments.

For CP42 where drilling was expected to be within Grade V Tuff drilling was done in a similar fashion to what was employed at other CPs, using the BOP. At some discrete locations, mainly during the drilling in the upper half of the CP face, drillholes were found to be collapsing before the TAM pipe could be installed. As a remedial measure the drillholes were re-drilled to clear the obstruction and backfilled with a BC grout mix with pressure locked in to seal loose blocks on the borehole wall. The drillholes were then re-drilled, if the pipe could not be inserted then the process was repeated but in telescopic sequence with 5m stages which was largely successful. The use of this technique also served to limit deviations.

Plate 3: TAM Grouting Pipes

Plate 4: Grouting Dialog

4.1 Pressure Balance Drilling

To mitigate the risk of groundwater drawdown, groundwater inflow and washout / erosion of fine soil particles during drilling a pressurised drilling system, Figure 10, was deemed required. Using a pressurised system with a pressure vessel containing a compressed outflow pipe, Blow Out Preventor, allowed for the borehole pressure to be maintained, see Plate 5. Once the drilling fluid pressure mirrors the insitu pressure the inflow of water into the bore is prevented. If this pressure is held slightly above the formation pressure, then it allows a filter cake to form on the borehole walls, which improves the stability of the drillhole. This system facilitated the installation of the Tube-a-Manchette, TAM by maintaining the grout pressure during its installation.

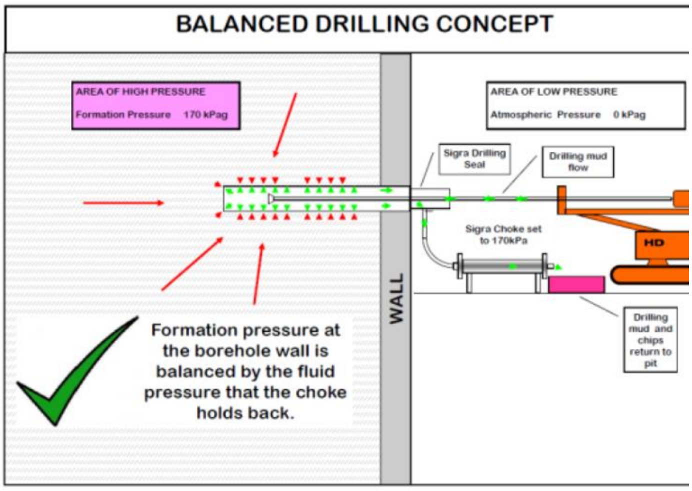

Figure 10: Pressurised Drilling System Schematics, courtesy of Sigra Lt

Plate 5: Blow out Preventor

Blow out preventors are usually only used in specific drilling applications, and therefore, most drilling staff needs to be thoroughly briefed on how to use the equipment to ensure proper utilization and minimize wear and tear. It was specified that the system would be used for all drilling holes of all CPs, which proved to be counterproductive. When used in Grade III/II rock the Blow Out Preventor was not really required since the risk of drillhole collapse was minimum and water inflow was manageable. Therefore, using the equipment in these geological conditions led to improper use and an underappreciation of the equipment’s capability. Since maintaining the pressure was not required for stability, the choke was being continually balanced as it benefited production. Afterwards when the drilling progressed to the Grade V Tuff the habit of continually balance the choke pressure was carried on. This behavior watered down the drilling fluid, bentonite, causing drillhole collapses, which would have been avoided with proper use. Stricter quality controls were required, as well as re-training staff regarding the proper use of the Blow Out Preventor.

4.2 Alignment Tool

Correctly setting up the accurate dip and azimuth for each drillhole prior to drilling is increasingly challenging underground. Normally this process is conducted by a surveyor with a total station, a process that can take up to 90 minutes, and that can create extensive standing time whilst the drilling team waits for the survey team to be mobilized. A proprietary system, which was developed for underground mining exploration in Australia, known as the Azimuth Aligner™, Plate 6 and Plate 7, was used in this project.

Plate 6: Aligner unit, clamped to leading drill rod

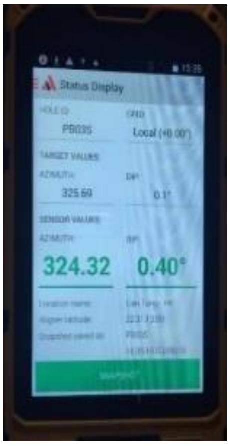

Plate 7: Azimuth Aligner Readout Unit

The unit is clamped to the lead rod and provides a real time readout of the current dip and azimuth whilst providing the closure distance to the planned / design dip and azimuth. This works by measuring the divergence of a laser beam by background magnetism and unaffected by ferrous objects. On site this was used for the alignment of the core barrel for concrete coring and for the alignment of the drilling rig. The tool provided accuracy to 0.2°, which equated to 60mm at 15m length.

The use of this alignment tool was an innovation that proved to be a very reliable and effective with the advantage of allowing for periodic checks in case the drilling mast is pulled out of line for any reason.

5 Cross Passage Treatment Performance Review

5.1 Grouting Cartography

The Grouting Cartography is a Visual Management Tool, which was implemented to track the grouting progress, ensure a proper grouting strategy, and guarantee the performance of the grouting treatment, see Figure 11. The Grouting Cartography consists of spatial mapping of the treatment zone displaying the intake of grout and grouting pressure achieved at each grouting stage. The grouting cartographies supported the technical decisions and were reviewed on a bi-monthly basis. Firstly, they were used to define the re-grouting strategy, e.g., where the grout intake and / or pressure were not satisfactory. For example, the zones in which the manchettes did not achieved a minimum of 2.5 bars were re-grouted. Secondly, the cartographies were key to decide when to transition from the Microfine Cement Stage to the Chemical grout. When it could be observed that the MFC already filled in the voids and increases of pressure did not result in further intake it meant that the treatment could be switched to the next stage with the Chemical Grout. Finally, the grouting cartography was also used to monitor the grouting performance in zones where drillholes had recorded non-compliant deviations. The grout intake of adjacent holes would be reviewed and re-assessed to ensure that the volume of soil to be treated could be achieved despite the deviations.

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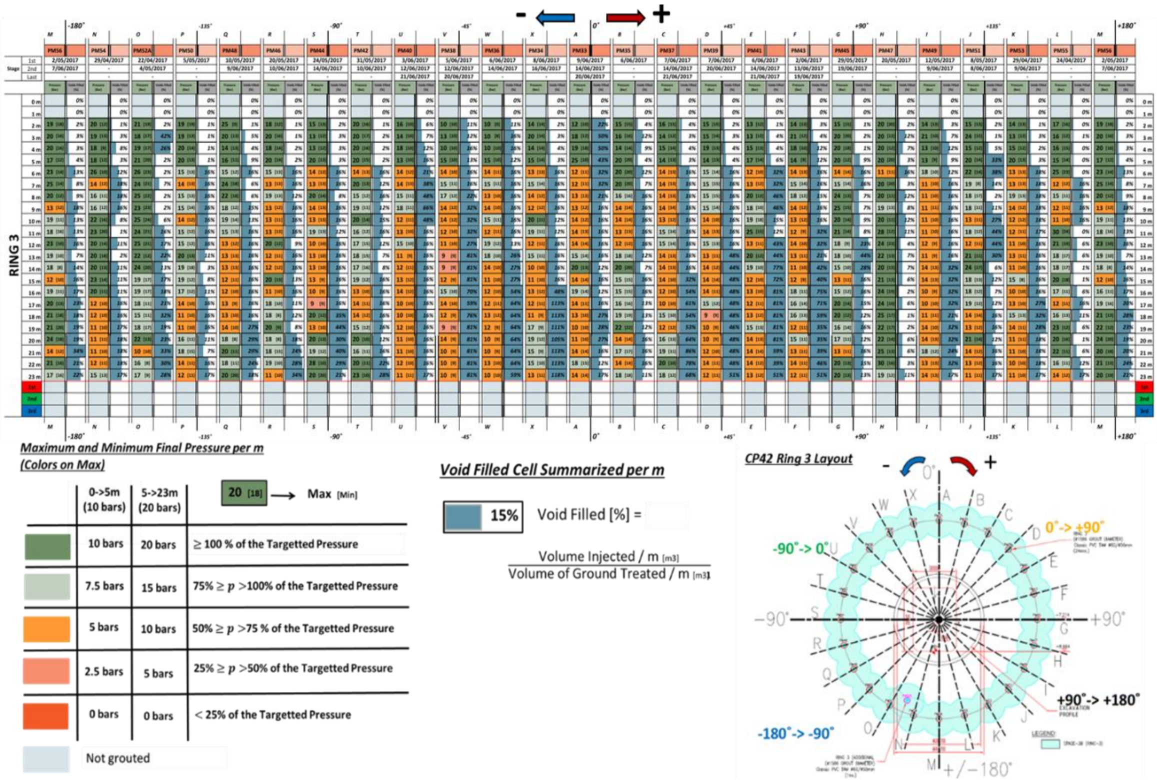

Figure 11: Chemical Grouting Cartography CP42 Ring 3

5.2 Drillhole Deviation and 3D Model

All the drillholes were surveyed after TAM installation using a Reflex Gyro™ system. The Gyro™ has an integrated GPS based compass, unaffected by magnetic interference, which measured azimuth and dip at 1.0m intervals. After Analysing the data from 169 surveys it was observed that the trend of deviation showed a downward fall to the right-hand side most likely due to gravity and the rotational bias of the tricone drill bit (clockwise direction). This can be explained by the weight of the non-return valve and the leading rod, which causes the drill string to sit on the invert of the borehole, and the anulus gap between the drill bit and the leading rod. Based on this data the theoretical position of the drillholes was adjusted to terminate 200mm above their previous position so that some of the drilling deviations impact could be minimized. While the trend was expected the magnitude of the deviation was unknown and therefore could only be corrected once there was enough data to support it.

At tender stage the deviation criteria were 2% for drillholes and 1% for canopy tubes. While these are generally achievable when drilling sub horizontal drillholes in homogeneous soil the varying strength and deformability characteristics of the residual soil found at CP42 exacerbated the drilling deviations. Special consideration for the heterogeneous nature of the soil should have been exercised and the deviation criteria re-accessed. The data showed that 22%, 42%, 51% and 58% of the drillholes at CP42 were out of tolerance respectively at 5m, 10m, 15m and 20m length. To ensure that the drilling deviations recorded did not impact the effectiveness of the ground treatment a 3D model with the respective as-built coordinated was created. In Figure 12 the design drillholes are shown in grey while the as built drillholes are shown in red.

The theoretical grout penetration, as detailed in Point 2 above, was then centered along the as built drilling alignment in order to determine possible gaps to the 3m ground treatment envelope, Figure 13 and Figure 14. Gaps in the ground treatment were either corrected by adding additional drillholes with associated grouting or increasing the volume stop criteria, discussed in Point 5.1.

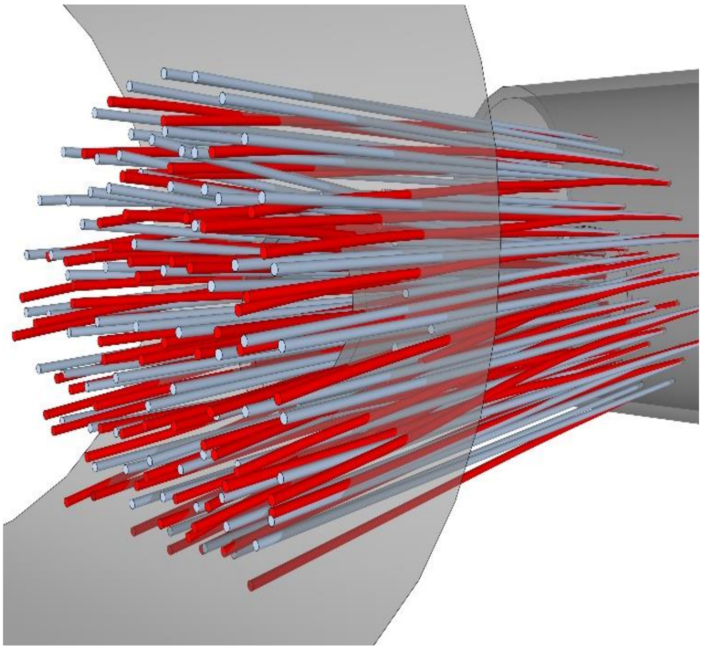

Figure 12: Drillholes Alignment: Design, grey, vs As Built, red

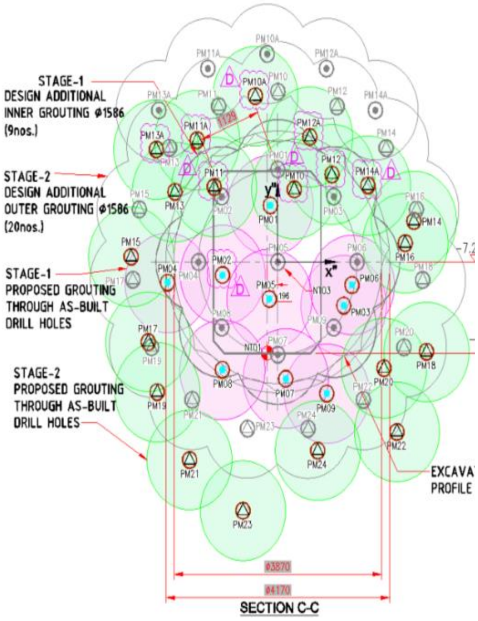

Figure 13: Stage 1 and 2 Design vs As-build Treatment Envelopes

Using CP42 as an example a total number of 11 additional drillholes, 7 in Stage 2 and 4 in Stage 3, were added to compensate for the recorded deviations and maintain an effective ground treatment envelope. There was a considerable improvement in the magnitude of deviations when comparing the early stages with the later ones.

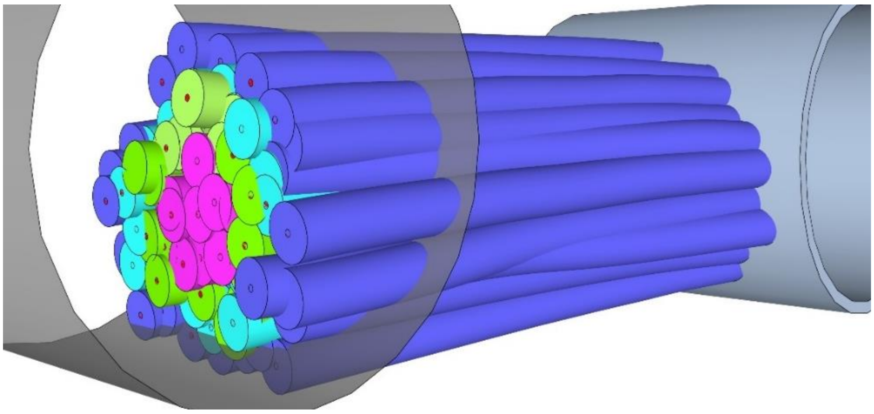

Figure 14: Theoretical Ground Treatment Envelope, Adjusted with drilling As Built Information

5.3 Verification Drill Holes and Cross Passage Excavation

Probing was conducted in CP42 prior to the start of production drilling. Drillhole PB01 was drilled on the 19th of September 2016 and recorded 60.33 l/min across its 21m length.

Upon completion of the ground treatment at CP42 a total of 4 no. post grouting probes were drilled, orientated to terminate within the grouted envelope. CTR-001 to CTR-004 were drilled on the 23rd & 24th of June 2017 with a length of 15m each. The recorded inflow is summarised in Table 2.

Table 2: Recorded Inflow in both Pre-Treatment and Post-Treatment Probes

	Probing	Post-Treatment Confirmation			
DH ID	PB-01	CTR-001	CTR-002	CTR-003	CTR-004
Flow (l/min)	60.33 (43.09)*	0.15	0.19	0.22	0.21

* Pro-rata flow at 15m length

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From Table 2 the expected groundwater inflow to the excavation was greatly reduced due to the ground treatment. The post grouting probes were used to determine the acceptability of the ground treatment by both the main contractor and its designer.

The excavation progressed in a dry and stable manner, Plate 8 and Plate 10, and without additional grouting being required. Plate 9 shows an excavated grouted soil block where it is visible that the Microfine cement penetrated through the sleeve grout. Lattice girders and shotcrete were used as the primary lining of the Cross Passages and was placed during the excavation as planned and prior to the permanent structural lining, Plate 11. The ground treatment allowed for an efficient excavation that was able to meet the planned programme and without adverse impact, e.g. water drawdown outside the CP.

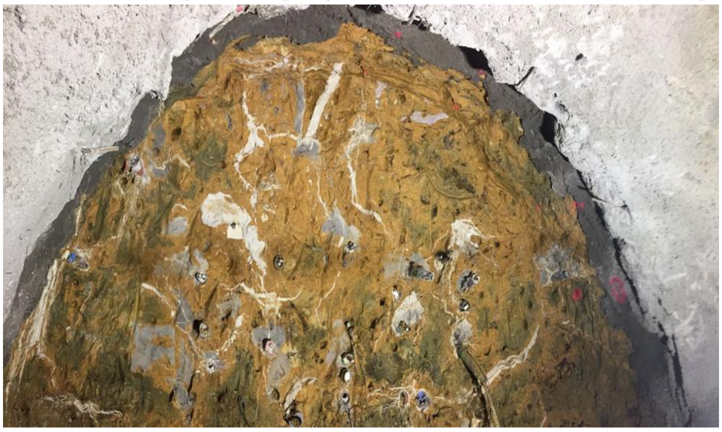

Plate 8: CP Excavation within the “Tube” and prior to the “Plug”

Plate 9: Treated Excavated Soil Block

Plate 10: Cross Passage Excavation (View from Permanent Opening

Plate 11: Cross Passage with Permanent Structural Lining

6 Conclusions

The observational approach used when designing the 3.0m thick grouting envelope to the CP’s allowed the design to be flexible and efficient. When changes where encountered, e.g. grout intake, drilling deviations, the design was able to adapt and remain efficient. The ground conditions required the permeation grouting to be divided into two stages, with a first stage with Microfine Cement and the second with Chemical Grout. This ground treatment solution was more favorable in terms of programme and cost when comparing to typical solutions adopted in similar ground conditions, e.g. ground freezing and jet grouting from the surface. The 3D model and the Grouting Cartography were paramount to the review and implementation of the ground treatment.

The as-built survey of the drillholes using the Reflex Gyro™ system revealed the magnitude of the deviation, which exhibited a downward fall to the right-hand side trend. The ground treatment design was then adjusted to compensate for this deviation. To ensure that the recorded drilling deviations did not impact the effectiveness of the ground treatment a 3D model with the respective as-built coordinates was created. Where gaps on the ground treatment due to the deviation were considerable, additional drillholes with associated grouting were specified. For CP42 a total of 11 additional drillholes were specified to compensate for the recorded deviations. When these gaps were marginal the volume stop criteria of adjacent holes was adjusted to ensure the grout treatment was sufficient.

The grouting progress was tracked using the Grouting Cartography, which consists of the special mapping of the treatment zones displaying intake of grout and pressure. The Grouting Cartography was reviewed bi-monthly and was used to define where to re-grout, when to transition from Microfine Cement to Chemical grouting and if minor deviations could be compensated by increasing the intake at adjacent drillholes.

The Pressure Balance Drilling tool, Blow Out Preventor, helped minimize drilling hole collapses, groundwater drawdown / inflow and washout and facilitated the installation of the TAM, pipes. However, the use of the BOP in less adverse soil conditions let to an improper use of the tool that required enhanced quality controls and staff training. It would have been preferable to specify the use of the tool only in soil or mixed soil conditions where it could bring the most value. The Azimuth Aligner proved to be a very reliable and effective tool specially to set out the drilling positions from inside the tunnel. When compared to the traditional process of setting out with a surveying team the alignment tool greatly minimized standing times and allowed for period checks to ensure the dip and azimuth were maintained.

A total of 4 no. post grouting probes were drilled upon completion of the ground treatment at CP42, which were orientated to terminate within the grouted envelope. The maximum recorded inflow was 0.22 l/min, testament to the effectiveness of the ground treatment when compared with the initial inflow of 43.09 l/min. The excavation of CP42 progressed in dry and stable manner until completion, which allowed the main contractor to meet the planned programme of works, with no adverse impact.

7 Publisher’s Note

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