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Application of High-Pressure Grouting and Pipe Forepoling for Tunnel Excavation in Jatigede Hydropower Project, Indonesia - An Experience

Kaustubh Mani Nepal
PT. Prima Layanan National Engineering (PLNE), Indonesia
**E-mail: kaustubh356@gmail.com*

ABSTRACT

Following the relaxation of COVID-19 restrictions, construction activities for the Jatigede Hydroelectric Power Project resumed. During this period, the construction of the horizontal penstock tunnel encountered two significant challenges. The first challenge involved rehabilitating a tunnel section affected by water infiltration and mudflow. Simultaneously, the second challenge was addressing a well-defined fault zone and excavating the tunnel safely through it. The tunnel crossed various geological materials, including loose collapse material, over-consolidated clay, interlayered disturbed tuffaceous sandstone, and highly permeable volcanic breccia connected to the static groundwater table. A high pressure grouting technique was applied to ensure the construction's safety, stability, and quality. This involved positioning a 90mm grout hole around the tunnel section at a 3 m spacing with different inclinations and lengths. The grouting parameters were adjusted according to the on-site conditions. The effectiveness of the grouting performance was verified by the inspection grouting process using a consistent grout mix (0.8, w/c ratio) with a pressure lower than 2 MPa and an injection around 50lit/min. After acceptance of grouting, the 108 mm diameter canopy pipes were drilled and installed, covering the complete arch of the tunnel. The excavation continued, with the central core left unexcavated until primary support was installed for the newly excavated length. These meticulous measures ensured secure, high-quality tunnel construction even under challenging geological conditions.

Keywords: Fault zone, Water inrush, High-pressure grouting, Grouting parameters, Canopy pipe.

1. INTRODUCTION

This paper presents the experience of the combined use of high-pressure grouting and canopy pipe fore-poling as pre-excavation support in the area close to the lithological boundary of two different rock units at the Horizontal Penstock Tunnel of the Jatigede Hydropower Project (110 MW) in West Java, Indonesia. It is a storage-type hydropower project that generates 5 hours daily to meet peak energy 462.6 GWh demand in the national grid. The design discharge for generating the power is $73m^3$/sec. It is on the Cimanuk River in Sumedang Regency, West Java, Indonesia. It uses the storage water from the Cimanuk River. The salient features of the project are given in Table 1.

The project faced two critical challenges nearly at the end of tunnel excavation. Firstly, there was a well-documented geological issue at the boundary between claystone and volcanic breccia, with intercalated sandstone, characterized by weak and disturbed rock conditions and was also confirmed due to saturation by groundwater. The second was necessary to rehabilitate the collapsed area and secure the tunnel's working environment, especially in a section that had experienced water inrush and mudflow incidents. These two incidents occurred 25 m behind the predicted rock boundary and weak rock location. Both challenges were alarming, and the anxious situation for all parties involved was further exacerbated by two major collapses within 100 m downstream of the tunnel in the past. There was also the looming threat of reoccurrence of water inrush and significant collapses during the remaining part of tunnel construction, which may result in additional time and cost overruns, mainly while working ahead to the breakthrough location. To address the challenges, the combined application of high-pressure grouting and canopy pipes fore poling was installed above the tunnel's crown to the spring line, effectively supporting the weak rock mass and tunnel face by transmitting loads longitudinally through the interaction with surrounding ground conditions. This support was intended to keep the area safe and stable during excavation until the steel-liner installation and backfill concreting were completed. The tunnel excavation was carried out using a mini excavator, leaving a central core unexcavated to support the tunnel face.

Table 1 - Salient features of the project

Dam, 110 m high Storage capacity	Embankment rockfill 877,000,000 m^3
Spillway (Restricted Orifice)	4 Radial Gates
Headrace tunnel, 4.5m diameter Length Grading	Concrete lined 2192.58 m 4 and 1%
Surge Shaft Diameter	Concrete lined, Vertical (Upper – 55 m, Lower – 50 m) Upper Shaft - 16.5 m & Lower Shaft - 11.5m
Penstock Tunnel Length	Steel lined 761m
Two Powerhouse Shafts	28m
Tailrace gate shaft, 17.2m diameter Length	Concrete lined 34.4m
Tailrace Tunnel, diameter 5.8m Length	Concrete lined 1050m
Tailrace Open Channel Length	Concrete lined 255m

2. GEOLOGICAL SETTING ALONG PENSTOCK ALIGNMENT

The tunnel alignment passed through Pliocene volcanic breccia, conglomerate (Folded Breccia, Q_{ob}), tuffaceous sandstone, and Miocene claystone (Cinambo Formation Shale Fragment, Tom_{cu}). The boundary between these two rock formations is believed to have formed due to substantial gravity displacement and faulting, resulting in shearing within the claystone. This geological scenario has been corroborated by investigation methods, such as drill holes and seismic refraction surveys conducted during the feasibility design study of the project.

Rock core information from drilling showed uncertain bedding dips, primarily due to shearing. The limited claystone exposures displayed varying bedding attitudes resulting from faulting or landslides. Drilling complexities, casing installation and removal challenges, and claystone caving and squeezing caused the abandonment of some drill holes. Limited exploratory drill holes reached fresh claystone at the tunnel level, measuring groundwater levels between 29 and 52 m above the tunnel level.

The folded Miocene clay rock, interbedded with fractured sandstone in breccia and claystone, covers the lower horizontal penstock tunnel between PS0+195 and PS0+250, named the transition zone between breccia and claystone. Between PS0+250 and PS0+210, the rock mass comprised fractured claystone characterized by closely spaced, continuous joints and bedding planes with openings exceeding 5 mm filled with clay. These planes were slickensides coated with clay. The rock was fresh to moderately weathered, with very low strength and an RQD of 10-20%. Groundwater was locally flowing, with general conditions damp to dripping. The rock mass was classified as RMR Class V. Likewise, between PS0+210 and PS0+195, the tunnel length was predicted as a boundary between volcanic breccia and claystone, containing interlayered breccia and highly fractured sandstone with claystone. The rock material had medium strength, an RQD of 15-35%, and groundwater conditions were damp to dripping. The rock mass was classified as RMR Class V, and the quality of breccia and sandstone bands within the transition zone was classified as RMR Class IV. The tunnel length was dominated by fractured claystone. The breccia and fault area had locally significant permeability (Figs. 1 and 2). Mostly, the joints oriented 190 – 210/40 – 80 (dip direction/dip), and the bedding planes recorded 020/75 (dip direction/dip) until the fault and transition zone. But, when the tunnel crossed the fault area, the bedding planes has orientation of 170/10 (dip direction/dip).

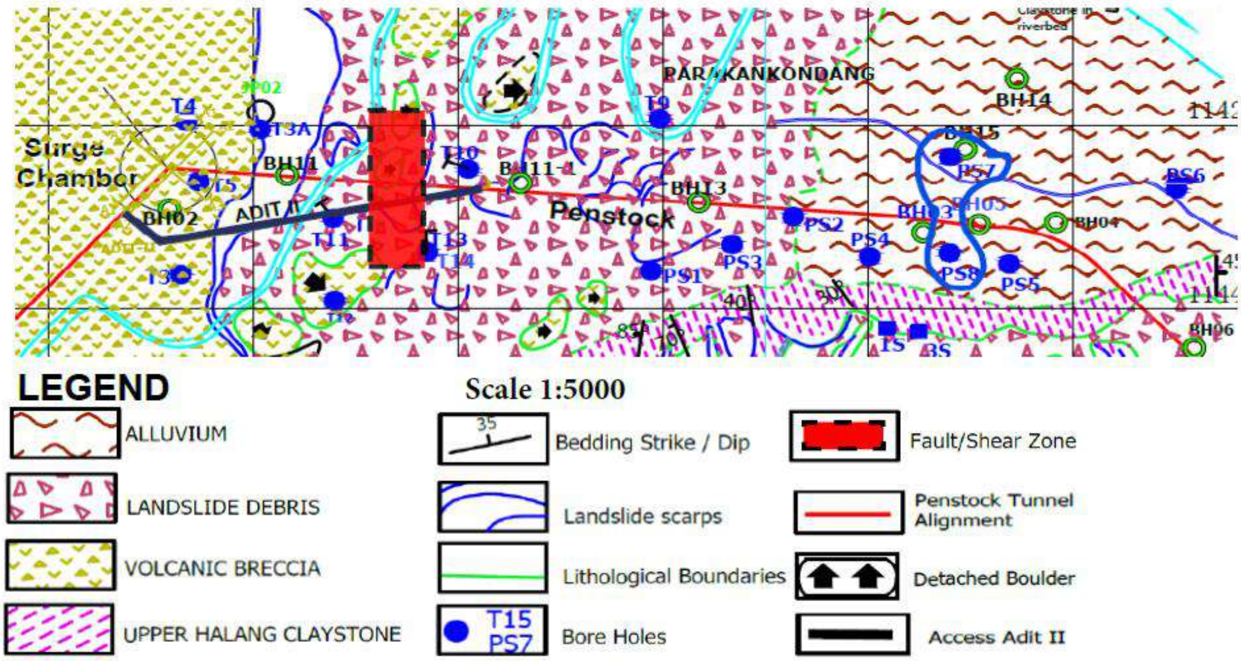

Figure 1 - The layout of the horizontal penstock alignment

Table 2 tabulates the physico-mechanical parameters of rock adopted for the design and analysis of the project structures. The swelling and porosity percentages, 35% swelling and 43% porosity were used for design calculation. RMR rating details are presented in Table 3 of the encountered rock masses in Penstock.

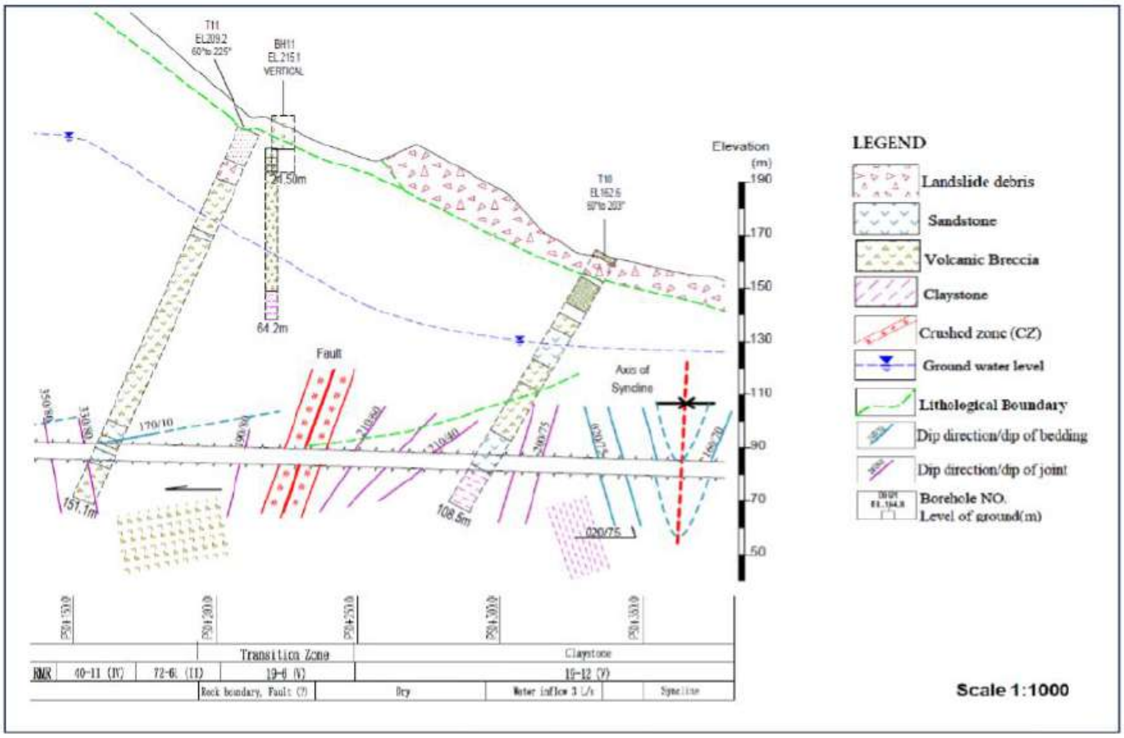

Figure 2 - Longitudinal (as built) section of the tunnel including fault and transition zone (PS0+150 and PS0+350)

Table 2 - Physico-mechanical parameters of the rocks along the tunnel alignment

Rock Mass Class based on RMR	Rock Mass Type	Density, g/cm^3	Shear strength		Deformation Modulus, GPa	Elastic Modulus, GPa	Poisson Ratio	Swelling Percent	Porosity Percent
			Friction coefficient, f	Cohesion, MPa					
V	Fractured zone, HW* Claystone	1.8	0.35	0.10	0.25	0.5	0.4	37	46
IV	Claystone and HW* breccia	1.95	0.40	0.15	0.40	0.8	0.35	32	44
III	Breccia	2.16	0.55	0.3	1.2	1.6	0.3	NA	NA

Notation: RMR - Bieniawski's Rock Mass Rating (Table 3), *HW - Highly weathered

Table 3 - Details of RMR parameters rating of the encountered rocks in penstock

RMR Parameters	Class III	Class IV	Class V
Intact Strength	2	1	0 - 1
RQD	10	3	3
Joint Spacing	8	5	5
Joint Persistence	2	1 - 2	0 - 2
Joint Separation	4	1	0
Joint Roughness	3	0 -1	0
Joint Infillings	2	6	2
Weathering	5 - 6	5 - 6	5 - 6
Groundwater	6 - 10	7 -10	10 - 4
Joint Adjustment	-2	-2	-12
RMR	40 - 44	27 - 33	13 - 11

3. BACKGROUND

The geological conditions were confirmed so far possible through the pilot holes drilled at PS0+221 and PS0+216 ahead of the tunnel face from the downstream side. The fault in the transition zone, marked by disturbed and weak rock, was verified and confirmed between claystone and volcanic breccia. Consequently, in the beginning, canopy pipe fore-poling and grouting through canopy fore-poling pipes were chosen as the pre-excavation support measure to manage the foreseen geological problem.

Pre-excavation support was considered done after completing the canopy pipe installation at PS0+216 to the tunnel spring line. The concrete grouting stage at PS0+216 was removed, and excavation started on October 5, 2021, from PS0+216. The tunnel face was initially dry, but seepage began from outside of some canopy fore-poling pipes' circumference. By the afternoon, seepage increased gradually, leading to loosening and falling rock fragments from the tunnel face. Preventive measures included the 3m long rebars as face bolting and shotcrete application for added thickness immediately after removing the concrete grouting stage. At 21:00 hours, the tunnel face bulged (Fig. 3a) and burst due to pressurized groundwater flow, carrying poorly fractured claystone, boulders, gravels of breccia and sandstone mixed with water and mud (Fig. 3b).

Figure 3a - Tunnel face bulged before the collapse

Figure 3b - Mud flow deposit after the 1st collapse

The next morning, debris cleaning in the tunnel began. The second water inrush happened at 18:40 hrs on October 6, 2021, subsiding by 20:30 hrs. The mudflow volume of the event was estimated to be 1200 m^3. On October 10, 2021, the Adit II tunnel collapsed above the designated position, causing a 5 m deep settlement in the public road above.

The treatment options proposed were as follows:

- Short canopy fore-poling and excavation from the tunnel faces.
- Ground freezing method.
- Long canopy fore-poling from upstream.
- High-pressure grouting with canopy fore-poling.

After analyzing ground conditions, collecting site data and reviewing scientific literature, Option 4 was chosen as the suitable treatment method. This approach utilizes high-pressure grouting combined with pipe canopy fore-poling with bundled rebar and cement slurry filling, effective in very weak ground conditions, known as canopy pipes or umbrella arch fore-poling (Oke et al., 2014). It was selected considering its reliability, duration, cost, tunnel safety and creating a safe working environment.

The high-pressure grouting method was believed to be most effective for treatment based on the following key points (Garshol, 2007):

- High-pressure grouting safeguards underground spaces by curbing groundwater drainage, averting water ingress, and potential settlement issues that can damage infrastructure.
- Systematic probe drilling, a key part of advanced grouting, mitigates the risk of major water influx and unstable ground. While costly and time-consuming, addressing excessive water post-grouting is less effective than advanced grouting, especially in challenging situations.
- Enhancing poor and unstable ground before excavation significantly stabilizes tunnel areas, allowing for safer support installation and minimizing the risk of uncontrolled collapse.
- Advanced grouting reduces ground permeability, preventing hazardous material ingress from the tunnel.
- This grouting method involves precise, high-pressure injection of stable cement-based grout into the soil, confining and controlling fracturing effectively up to 4 MPa (Bruce and Pellegrino, 1995; Henn, 1996).

3.1 Geology of the Lithological Boundary and the Collapse Material

To determine geological conditions, fault/shear zone thickness, and exact breccia/claystone boundary, exploratory pilot holes were drilled in multiple directions from up and downstream tunnel faces. The pilot holes' findings consistently confirmed the faulting/shearing interface between clay-stone and volcanic breccia between PS0+194 and PS0+230. The information collected from the pilot and canopy fore-poling holes was compiled, and the cross-sections (Fig. 4) have been developed, integrating the latest geological data obtained after the collapse event on October 5, 2021. According to a drilling hole, the water table was 47 m above the penstock tunnel crown. On the right side (looking upstream) of the penstock, the estimated claystone cover was about 10 m thick, 1.5 times the excavated diameter. Following the mudflow on October 5 and 6, 2021, which spread a maximum of 700 m from the tunnel face at a 50-80cm thickness, accessing the tunnel face became impossible. The material, mainly composed of volcanic breccia and sandstone, exhibited diameters 20-80 cm, predominantly angular. Approximately 95% of the material was clayey, with shades ranging from grey to black, resulting from a mixture of claystone, sandstone, and breccia, displaying heterogeneous characteristics. The total estimated volume of mudflow for the first and second events was estimated to be 1200 m^3.

3.2 Groundwater Condition and Management

In response to delays induced by the COVID-19 pandemic, the construction of the penstock tunnel was split into two phases: top heading and benching, which were carried out from both directions. The upstream excavation was performed using conventional drilling and blasting in highly permeable volcanic breccia, while a Roadheader was used to excavate the predominantly claystone downstream side.

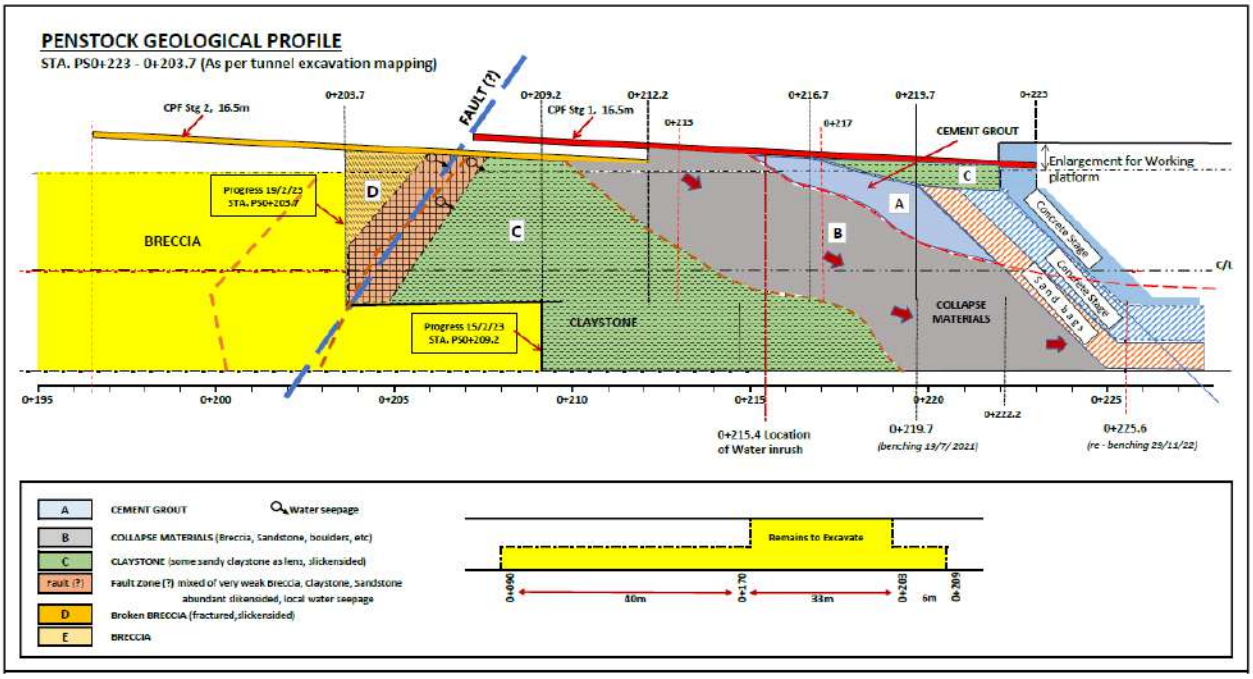

Figure 4 - Reconstructed Penstock geological section after high-pressure grouting and canopy pipes fore-poling at PS0+203.7 – PS0+223

Excavation at the upstream tunnel face was halted 30 m behind the anticipated fault zone, with subsequent excavation planned from the downstream side. Due to the rock permeability (6×10^{-4} cm/s to 6×10^{-5} cm/s) on the upstream side, notable water accumulation was expected during and after benching excavation at the upstream side.

To manage the groundwater inflow in the tunnel the existing drainage system was reinforced with additional water pumps. Three core recovery holes were drilled from upstream to downstream, one of which served as a drainage hole. Two additional holes were drilled on both sides of the upstream tunnel face to locate the boundary between volcanic breccia and claystone, but neither could accurately trace the estimated rock boundary and were subsequently sealed.

Eight holes were drilled downstream to upstream to evaluate the geological condition ahead of the tunnel face and drain accumulated water. Only three drain holes effectively drained water from the upstream side, and the remaining five were sealed due to collapsing and blockages. The existing 70 m vertical water pumping system at the upstream side continued to operate (Fig. 5).

4. PREVAILING PRACTICE OF HIGH-PRESSURE GROUTING

Grouting is a well-established method in underground engineering, with its approach chosen based on local geological conditions (Zhang et al., 2014). The technique used here is soil fracturing or hydrofracturing grouting, although the complexity of the process results in a mix of permeation and compaction grouting within fracture-dominated processes. Various factors like grout properties, pressure, soil/rock type, and ground initial stress state influence the chosen method. Figure 6 illustrates the relationship between grouting pressure and rate in loose soil. This grouting model can be described briefly into five stages for reference only.

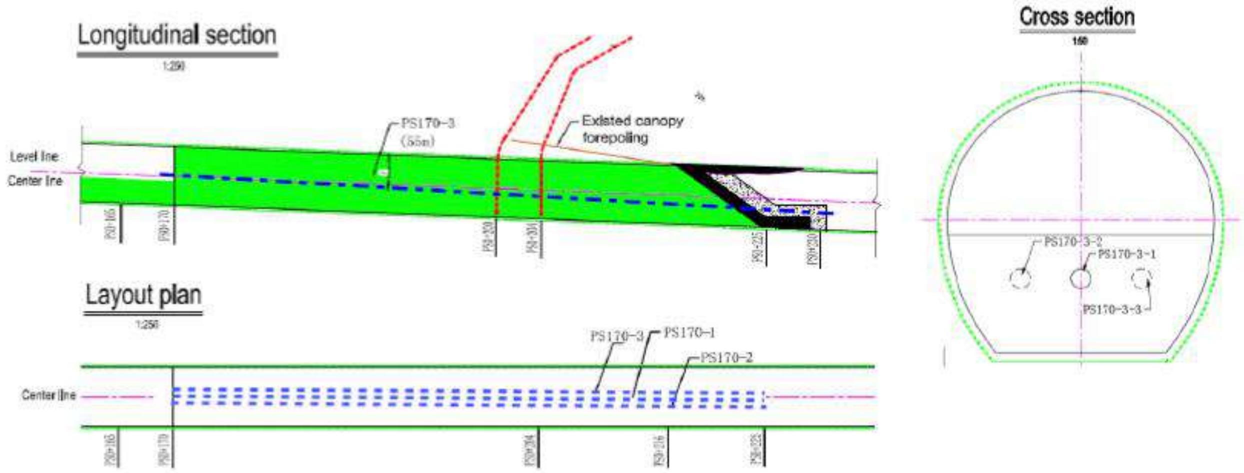

Figure 5 - The sketch shows three operating drainage holes (blue lines) drilled downstream to the upstream side of the tunnel

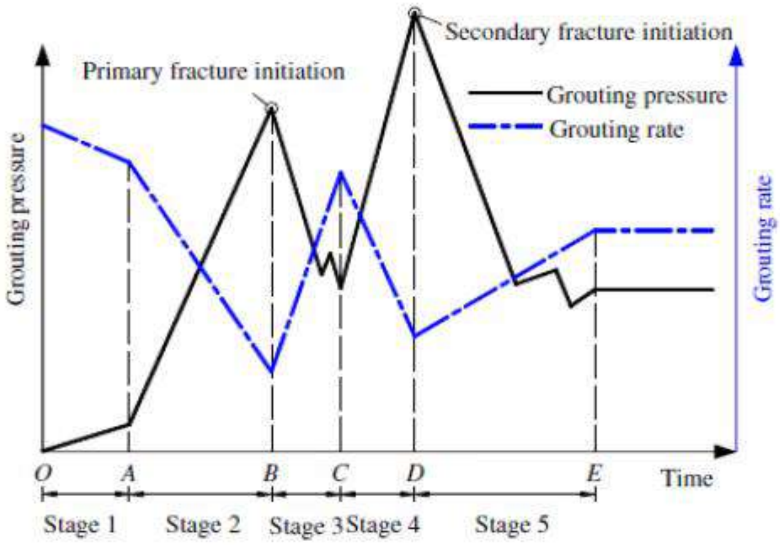

Figure 6 - Grouting pressure and rate during a typical grouting process in loose soil (the vertical axis is scale exaggerated) indicate the gradual increase in grout pressure (Zhang et al., 2014)

4.1 Stage 1: Backfill Grouting and Permeation Grouting

A grouting hole is first filled with pressurized grout, and the grout begins to penetrate the ground from the hole. The injected grout fills the cavities in the loose soil, referred to as backfill grouting. Meanwhile, the injected grout may also permeate the voids between the soil particles, a process called permeation grouting (Nonveiller, 1989; Zhang et al., 2014). The ground permeability is reduced because of backfill and permeation grouting, and the ground is thus strengthened and stiffened. At this stage, the grouting pressure and rate can gradually decrease. In general, the duration of this stage is very short, lasting for only tens of seconds.

4.2 Stage 2: Compaction Grouting

Through continuous grout injection, the soil or fragmented rock is displaced outward, shaping a grout bulb surrounding the pipe. As grouting pressure rises, this bulb expands until the pressure triggers fracturing. The development of the grout bulb shifts and compresses the surrounding loose or fractured material (Wang et al., 2010). In this stage, grouting pressure increases to a peak value, gradually decreasing the grouting injection rate (Zhang et al, 2014).

4.3 Stage 3: Primary Fracture Grouting

During grouting, when the pressure reaches its first peak value, a fracture plane forms in the soil through hydraulic fracturing (Zhang et al., 2014). Assuming the grouting is ongoing in an isotropic elastic medium, the fracture plane is theoretically perpendicular to the secondary principal stress plane along the longitudinal direction of the pipe. The primary fracture is initiated by tensile stress, and the maximum tensile stresses are at the two points in a cross-section of the borehole (points a and b, refer to Fig. 7), where the original tangential stresses are lowest in compression (or highest in tension) after borehole drilling.

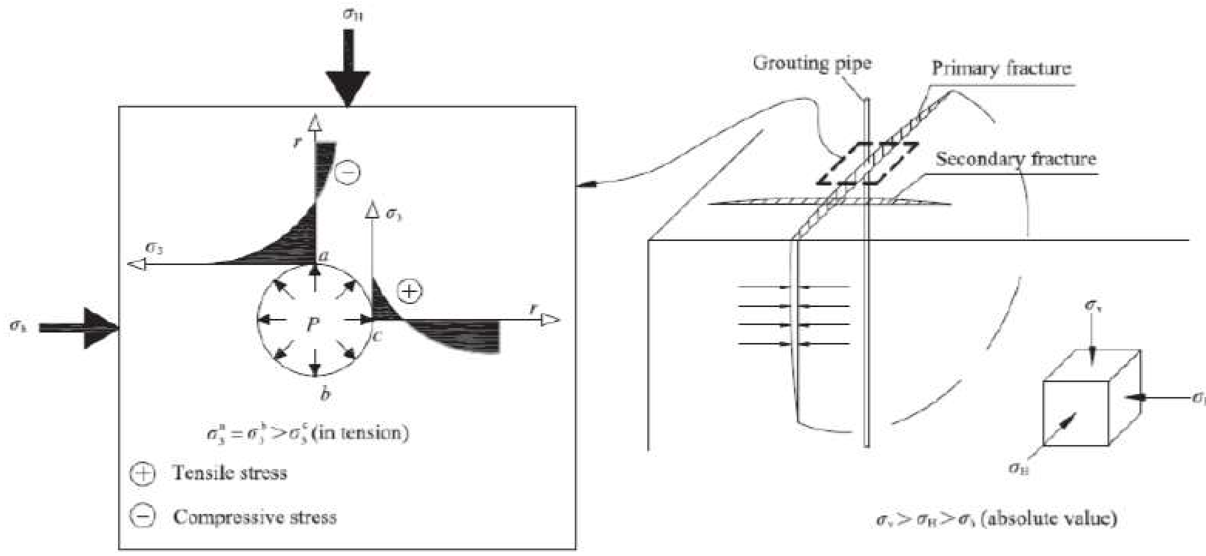

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Fig. 7 - Mechanism of grouting fracture initiation (Zhang et al., 2014)

The opening of the fracture changes the boundary conditions, resulting in a sudden drop in injection pressure. Although the grouting pressure is reduced after fracture opening, the fracture develops very fast due to the stress concentration at the tip of the fracture. The ground fractures continuously until the pressure around the fracture tip can no longer create further fractures. Meanwhile, with the development of the fracture, more grout is required to fill the initiated fracture, leading to an incremental increase in the grouting rate (Zhang et al., 2014).

4.4 Stages 4 And 5: Secondary Compaction Grouting and Secondary Fracture Grouting

As the grout flow becomes confined, the pressure increases and can widen the primary fracture. If the grouting pressure reaches a certain level, a secondary fracture may be initiated and filled, and subsequent grouting can initiate more fractures with varying paths. The initiation location and propagation path of the secondary grouting fractures vary significantly.

Finally, it is important to note that pressure-grouting applications do not always encompass all permeability. Additionally, not every grouting procedure consistently manifests the primary and even the secondary fractures as part of the process (Zhang et al., 2014).

5. SITE PREPARATION AND PRELIMINARY WORKS

Before grouting began, a 1 m thick concrete stage (pad) was constructed at the face to balance the grouting pressure. To anchor the concrete stage at the tunnel face, 25 mm anchor bolts were installed with a spacing of 1m and a length of 2 m. These anchor bolts were connected to the reinforcement of the concrete stage. The concrete stage served as the platform for the drilling rig, and the tunnel was enlarged to 4.5 – 5 m to provide sufficient space for the drilling and grouting activities.

To prevent grout and pressure backflow behind the original tunnel support during grouting, the primary support to 15 m length behind the tunnel face was restrengthened. This was achieved by adding extra H125 steel ribs, which, combined with the pre-existing steel ribs installed before the collapse on October 5, 2021, formed a complete ring support, along with the invert. The newly added steel ribs were covered with shotcrete, and the invert was concreted 30 cm.

6. GROUTING

6.1 Grout Holes Layouts and Cross-Sections

According to the site's geological condition, the grout hole length was designed to cover 27 m (excluding the grouting stage). The grout hole spacing was less than 3m, and the radial grouting coverage was 5 m outside the tunnel excavation line. A grout hole diameter was 90 mm. The layout and cross-section grout holes are shown in Figs. 8a to 8d. The whole grouting exercise, including deformation treatment due to grout injection behind the working face, took 5 months.

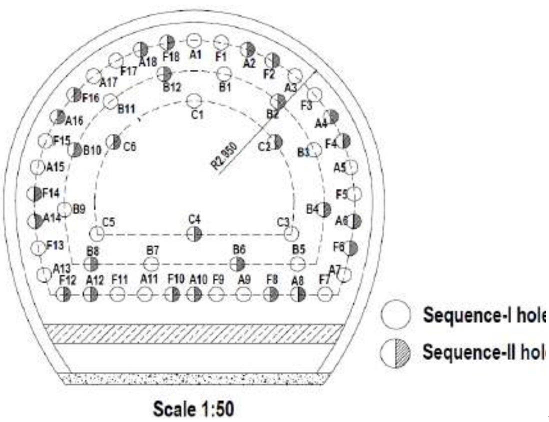

Figure 8a - Schematic layout of the grout holes at PS0+223

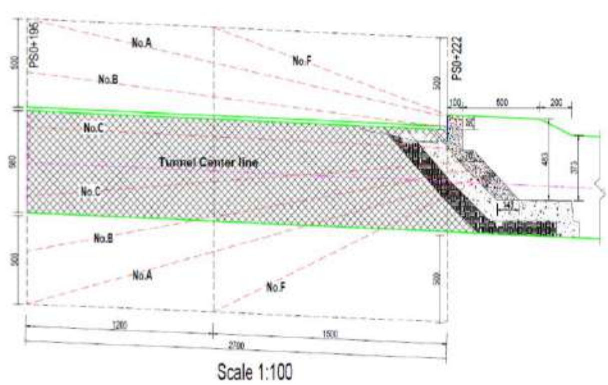

Figure 8b - Longitudinal section of the tunnel shows the grout hole layout

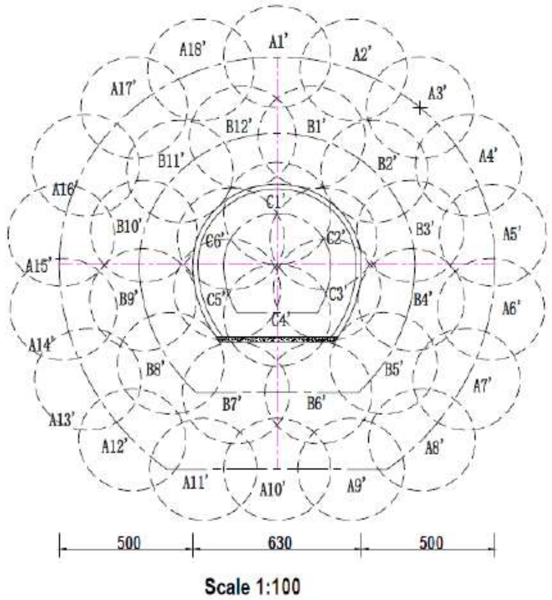

Figure 8c - Schematic layout grouting holes and influence area

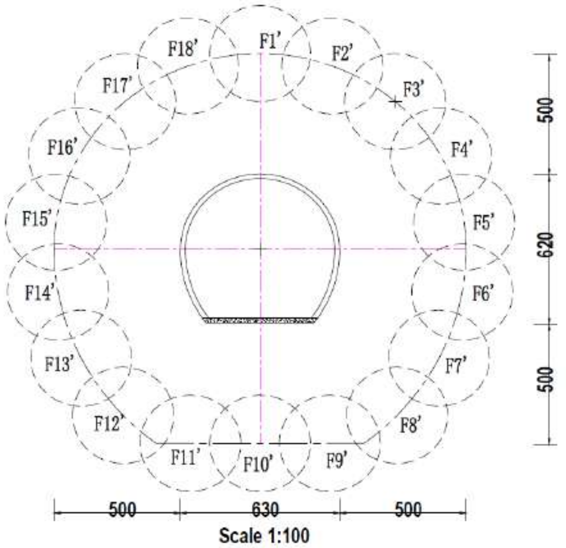

Figure 8d - Layout of supplementary grouting holes and influence area

6.2 Grouting Parameters

A total of two grouting sections were set, one with 54 grouting holes and another with 36 holes (Fig 8c., A, B and C Series) 27 m long as the main holes for the grouting ahead of the tunnel face, including 15 m long and 18 holes as the supplementary holes (Fig 8d., F Series). The designed grout penetration radius around the hole was 2 m. The grouting stage at a time depended on the hole stability after drilling length. In general, the grout stage was 3-5 m. The details of the grouting information are given in Table 4.

Table 4 - Grouting information

Nos.	Name	Information	Remarks
1.	Grout coverage	27 m longitudinal length	
		5 m out of tunnel excavation boundary	
2.	Grout penetration radius	2 m	
3.	Grout Injection rate	10 – 90 l/min	The injecting rate was adjusted during grouting.
4.	Final grout hole spacing	Less than 3 m	
5.	Grouting pressure	2– 4 MPa	The pressure was adjusted during grouting at the site.
6.	Grout Types	Portland cement Type I plus rw-HPC	
		Cement and water glass dual slurry	
7.	Grouting Method	Forward, Backward	
		Horizontal sleeve pipe	For stable hole
8.	Total Number of grout holes	54	
Grouting sequence: first perimeter holes section, then the supplementary holes, then inside holes up to downward alternatively.			

6.3 Admixtures Used

The rw-HPC cement admixture enhanced the properties of cement slurry in grouting, reducing the initial setting time to less than 90 min. This improved grout strength, water dispersion resistance, and fluidity while minimizing bleeding. The setting time was adjusted to 30-60 min.

Additionally, water glass was mixed in a water-to-cement ratio of 0.8:1 and an equal volume of water glass. This dual grout mixture provided stability, allowed precise control of the setting time from a few to tens of minutes, and effectively prevented excessive groundwater ingress and grout communication.

6.4 Grout Mix Proportions

The mix proportions adopted for the grouting are given in Table 5. At the site, grout mix consistency and bleeding percentage were monitored to maintain the quality.

6.5 Grouting Pressure Adopted

The high grouting method involves locally confined and controlled fracturing of a soil/rock unit by injecting a stable grout, cement-based grout at high pressure, for example, a maximum of up to 4 MPa (Bruce and Pellegrino, 1995; Henn, 1996). The grouting pressure was designed as 3 – 5 MPa at the beginning. However, during the grouting process, the grouting conditions were adjusted based on the on-site grouting conditions and the deformation near the tunnel grouting area. After adjustment, the maximum applied grouting pressure in the first stage (0 - 4m) did not exceed 2MPa, gradually increasing during grouting between 4 - 27m to a maximum of 3.5MPa (±0.5MPa). Since the rock conditions on the left-side tunnel face (looking up-stream) were better than that on the right side (looking upstream) of the grouting face, the maximum grouting pressure at the left-side tunnel face (looking upstream) hole was 3MPa, and that at the right-side tunnel face hole was limited to a maximum of 2.5MPa.

Table 5 - Grout mix ratios adopted during the grouting process

No.	Grout type	Water Cement Ratio (W/C)	Admixture Ratio by cement weight	Initial Setting time in minutes	Final Setting time in minutes	Bleeding Percent	Remark
1.	Cement and water + HPC	0.8:1	10% - 15%	67	297	1,61	Normal
2.	Cement: Water and Water Glass	1:1, and 0.8:1	10% -15%	18	105	<5	Grout Communication and water seepage
3.	Cement: Water	1:1, 0.8:1	-	343	490	<19 - 33	Normal

6.6 Grouting Completion Criteria

Based on the grout hole stability conditions, a pressure of 3 MPa was maintained during grouting, and the injection rate was flexibly adjusted by comparing it with nearby grout-completed holes. As the grouting pressure increased and the injection rate decreased rapidly for 5 min, the grouting was considered completed. When the completion conditions are not met, the grouting pressure and mix type are adjusted accordingly, demonstrating the adaptability of the process.

The pressure was limited to 2 MPa during the verification grouting, and the injection rate was kept lower to 20 l/min. This process continued for 5-10 min before being considered complete. All other grouting procedures remained consistent with the main process. In case of a pause during the process for any reason, depending on the time lapsed, the holes were re-drilled and grouted using the standard procedure mentioned above.

6.7 Adopted Grouting Method

The grouting methods included forward and backward stage grouting and horizontal sleeve valve grouting. This comprehensive approach, covering all possible scenarios, ensured the project's successful implementation of the grouting process. Initially, all holes underwent forward grouting for the first 0-12 m, after which the suitable grouting method was selected based on the conditions that were encountered. Quality was maintained by continuously monitoring grout mix consistency and bleeding percentage.

The grouting process began by drilling a hole and installing a seamless pipe with a control valve. A hole was drilled to the design depth and sealed to prevent slurry leakage. A larger drill bit was used at low speed for poor and loose rock, followed by installing an orifice pipe. Forward grouting and drilling were conducted in stages, with the stage length adjusted based on hole stability. Backward grouting was employed when the grout hole could not remain intact after drilling. The drill pipe was pulled out and left as a grouting pipe, with the surrounding gap sealed using a water glass slurry. Once stabilized, cement milk was introduced, and the process moved backwards in stages. Sleeve valve grouting (Fig. 9) involved a tube (Fig.10) fitted with a one-way rubber jacket to control grout flow. The rubber sleeve opened under pressure, allowing the grout to exit and preventing backflow. The sleeve automatically closed when external pressure exceeded internal pressure, ensuring efficient grout distribution and minimizing leakage.

7. ANALYSIS OF GROUTING RECORDS

The cement intake in different grouting series and the specific cement intake per unit length in kg are shown in Fig 11. The summaries of the total drill length, total cement intake, and Cement Intake per m are presented in Table 6.

Figure 9 - Schematic stage grout injection (Nos. 1, 2 and 3) through a sleeve pipe (Figure not in scale)

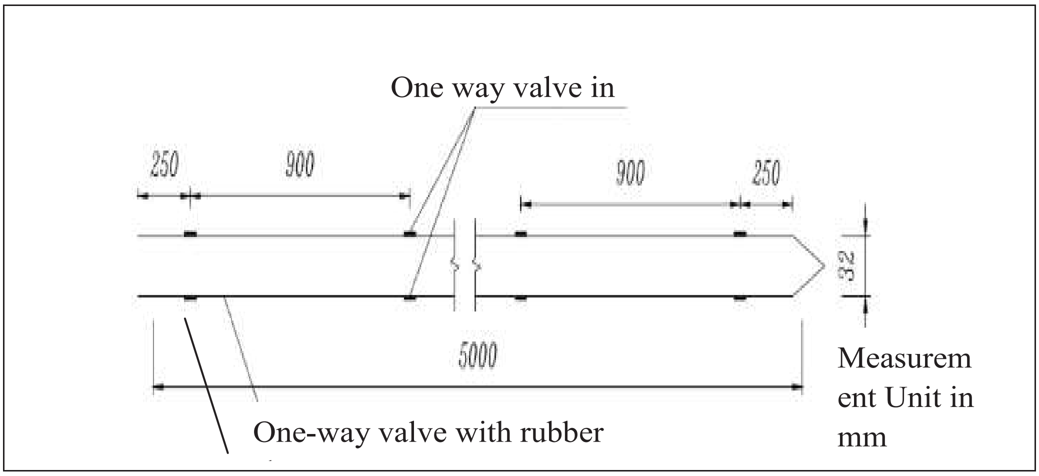

Figure 10 - Details of the sleeve grout pipe

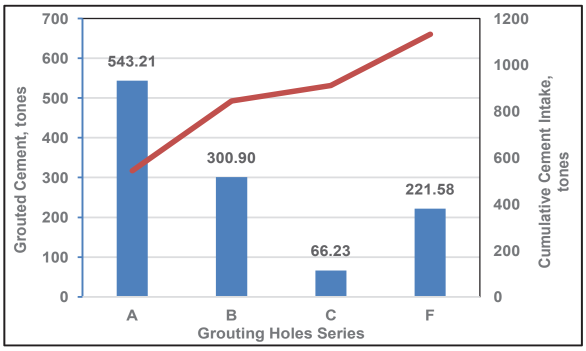

Figure 11a. - Total cement Intake (t)

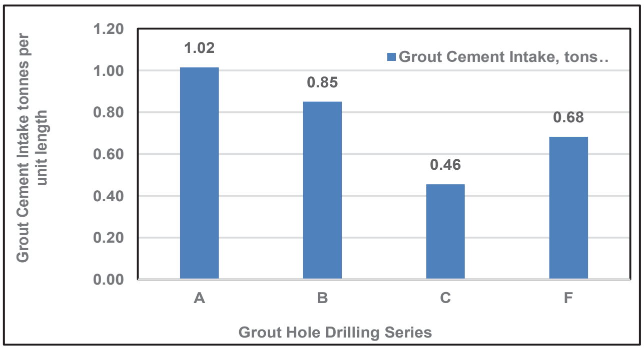

Figure 11b. - Cement intake (t) per unit length

8. VERIFICATION OF GROUTING PERFORMANCE

The following steps were taken to assess whether the grouting was carried out satisfactorily and regarded as completed. The performance grouting was also carried out in two stages. The first grouting stage was verified when 11 m drilling and grouting out of 29 holes were completed. The second stage of verification grouting was carried out after the grouting of all holes was completed to the designed length of 27 m. It was started only after evaluating the first stage of performance grouting. The outcome of the first stage helped adjust the grout pressure and injection rate for the second grouting stage. A total of six holes were drilled for inspection purposes: three core recovery check holes (223-3, 223-4, and 223-6) and three non-core recovery check holes (223-1, 223-2, and 223-5). The check holes for the first stage (No. 1 – 3), grouted length 0 – 11 m, and the second stage (No. 4 – 6), grouted length 11- 27 m. These check holes (Fig. 12) were strategically positioned to ensure that they penetrated the 5 m of grouted thickness around the tunnel excavation boundary line and the 30 m length of grouted thickness ahead of the current tunnel face. During the check holes drilling, the following monitoring was carried out: the drilling rate, checking for incidents during drilling, and observing the color of drill water return and water seepage from verification holes.

Table 6 - Summary of total drilled length, cement intake and cement intake /m hole series

Hole Series	Drilled Length, (m)	Cement Injected, (t)	Consumption Specific Cement (t) / m
A	535	543.21	1.233
B	353.7	300.90	0.930
C	145.5	66.23	0.540
F	324.6	221.58	0.711
Total	1358	1131.92	0.955

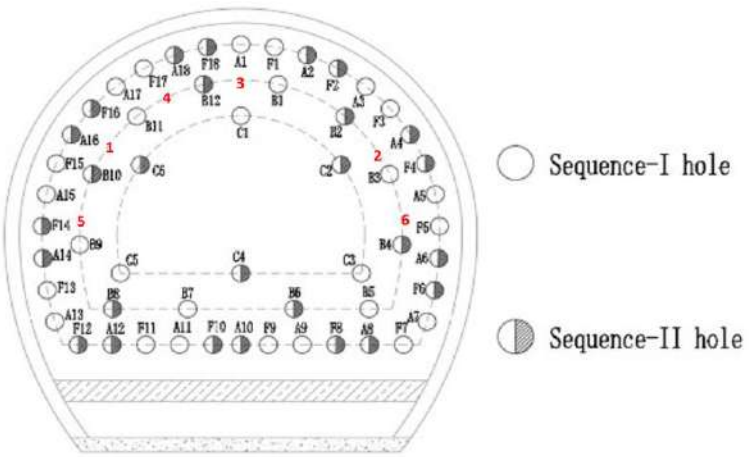

Figure 12 - Layout of the check holes for the first stage (No. 1 – 3), grouted length 0 – 11m, and the second stage (No. 4 – 6), grouted length 11- 27m

No water pressure testing was conducted to verify the grouting performance. However, the check grouting process involved injecting a standard grout mix design with a water-to-cement ratio of 0.8:1, which was previously used for the main grouting at the location. The grout pressure was set at 2-3 MPa during the first and 2 MPa during the second stages.

The check holes' lengths were 0-11 m and 11-30 m in the first and second stages of check grouting, respectively. The core drilling results were not as satisfactory as expected due to core loss, drill-recovered material washout by drill water, and mechanical breakdown of the weak claystone during drilling. Many cement fragments and nodules of cemented clay of various sizes were recovered. These served as indicators of the cement grout diffusion in the area, as shown in the recovery of these nodules and cemented clay fragments. However, no water was found to be flowing out from the drill holes; all core and non-core drill holes remained dry. The presence of cement (grout)-mixed clay nodules and the rock fragments' surfaces coated with cement paste helped conclude and estimate that the grouting works effectively.

After the evaluation of the real-time p-q-t curves, and made the following conclusion:

- The grouting mechanism conforms to each hole wall stability and no sand, water, or mud inrush.
- The real-time grouting p – q - t curve trend followed as expected.
- In Fig.13a, below, the grout injection decreases from the beginning, and the pressure increases at first and stabilizes after 30 min. It was considered acceptable when grout pressure did not rise till 10 min. The pressure curve could not rise to the design's final pressure, showing an upward trend, and the injection curve shows a downward trend from the beginning.
- In Fig. 13b below, the grout injection curve decreases, and the pressure curve cannot increase and remain stable till the grout injection becomes zero. The injection curve shows a downward trend from the beginning and becomes zero injection till 5 min.

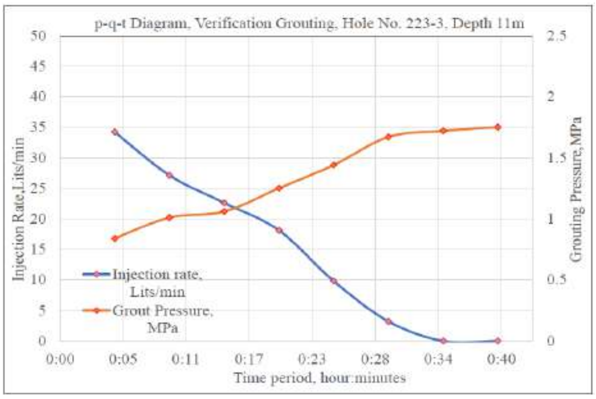

Figure 13a - Check hole No. 223-3; drilled depth 11 m, p-q-t diagram

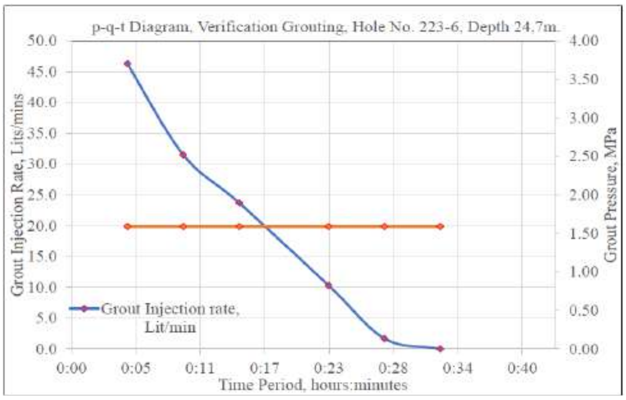

Figure 13b Check hole No. 223-4; drilled depth 25 m, p-q-t diagram

Finally, it accepted that the grouting results qualified and met the requirements. After two rounds of excavation, it was confirmed that the overall grouting was working well. However, as prevention, another supplementary grouting was kept stand-by, if needed during excavation, as it was difficult to confirm the exact. The exact situation could only be confirmed after evaluating the excavation face and surrounding of the tunnel walls. The white light grey straps in the photos below are the grout diffusion lines. The photos presented in Figs 14a to 14d are from the different locations of the tunnel faces.

9. ADVERSE CONDITIONS ENCOUNTERED DURING GROUTING

The following displacement happened during the advanced grouting process behind the tunnel grouting face. The greatest distortion occurred at the right side of the tunnel arch up to 19 m (Chainages PS0+223 - PS0+242) behind the grouting face (Table 7). The issue was the buckling of previously installed steel sets. The deformation mostly occurred while adopting around 1.8 MPa to 3.7 MPa pressure to the grout injection at the right tunnel arch in the first grouting stage. The tunnel deformation during the grouting process was mostly caused by the cracking and falling of shotcrete chunks and the buckling of steel ribs. The maximum buckling of steel ribs observed was 50 cm at PS0+223 at the crown of the tunnel. When deformation was noticed, the grouting activities were immediately halted, and treatment was started. Deformations observed in the tunnel arch after the grouting are shown in Figs. 15a and 15b.

Figure 14a – Collapsed material after grouting with damaged previously installed canopy pipes.

Figure 14b – Thick white lines, indicating grout penetration, observed during the excavation of collapsed material after grouting

Figure 14c - In-situ claystone, after grouting with damaged previously installed canopy pipes


Figure 14d - Grout penetration (thick grey) lines seen at the tunnel face during the excavation

Table 7 - Shows the chronology of major deformations behind the tunnel face during the grouting process

Sr. No	Station	Grouting pressure, MPa	Deformation, cm
1	PS0+230 – PS0+235	1.8 – 1.9	~10
2	PS0+235 - 0+260	2.9	~10 – 15
3	PS0+239 - PS0+246	3.6 - 3.7	>30
4	PS0+223 - PS0+230 and PS0+239 - PS0+242	3.44	> 50
5	PS0+236.5	3.45	~20

The right wall (looking upstream) had a relatively thin claystone layer compared to the left wall at the grouting length. The pore pressure in the transition zone pushed the claystone inward. Since the claystone was impervious, the grouting pressure was transferred backwards through the transition zone.

The treatment for the first to third deformations involved replacing steel support and reinforcement with added steel support and shotcrete. The collapse at PS0+223 hindered the grouting activities for 104 days.

Figure 15a - The peeling of shotcrete and buckling of steel ribs. The deformation, 10 – 15cm (within the blue circle), occurred at the right-side tunnel arch during grouting with pressure at 2.9 MPa

Figure 15b - Maximum deformation (within the red rectangle) occurred during grouting at the right-side crown when the injection pressure was 3.4 MPa

10. TUNNEL EXCAVATION

The canopy pipe fore-poling was designed to be 28 m long. The installation was executed in two stages to ensure precise drilling and correct angular placement. Each stage involved installing a 16.5 m long pipe, with a 5 m overlap between consecutive pipes (Fig. 16). The drilling angle of the canopy pipe hole was meticulously maintained at an upward inclination between 5^0 and 10^0 degrees. The holes were drilled to cover the complete tunnel arch and canopy pipes were inserted into the holes.

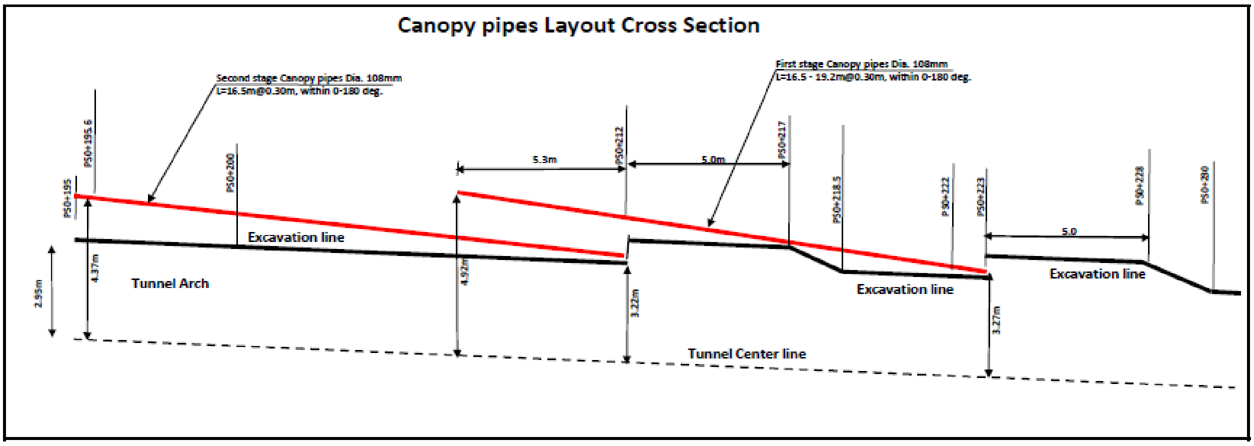

Figure 16 - Stages of canopy pipe installations adopted. The red lines indicate the first and second stages of canopy pipe installations

A 108 mm steel pipe was installed as a canopy pipe umbrella. The canopy pipe was reinforced by inserting a bundle of 3 nos. of coupled 25 mm diameter rebars slightly shorter than the canopy pipe length after the pipes were filled with cement grout. The details of the reinforcement of the pipe canopy are shown in Fig.17.

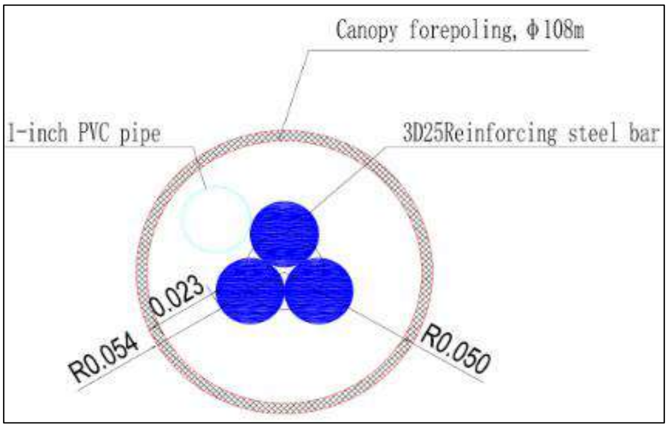

Fig 17 - Details of a canopy pipe reinforcement

After completion of the first stage of canopy fore-poling pipes installations, excavation of collapse material (after grouting) has been carried out from PS0+223 to PS0+212. In the second stage (from PS0+212 to PS0+195), the 16.5 m long canopy pipes were installed following the same procedure as during the first stage. Thereafter the tunnel excavation resumed. A mini excavator was used for the top heading and benching excavation, keeping a 0.5 m centre core at the tunnel face during the advancement supported with wire mesh (8 mm wire having an opening of 150mm × 150mm).

The benching excavation closely followed the top heading excavation, keeping 6-7 m from the advancing tunnel face. The tunnel section was supported by I200a steel beams (flange width 100 mm, flange thickness 7.8 mm, web width or depth 200 mm and web thickness 5.2 mm) spaced 0.5 m apart. Wire mesh (150 × 150 mm), shotcrete (C30 grade, 150 mm thick) and steel ribs (I200a) were applied to the tunnel walls and the crown. A steel rib was installed first at the crown, and the steel rib sides of the top heading section were extended temporarily to the invert of the top heading. It was then covered with 30-50 cm of C25 grade concrete, creating a closed ring support in the excavated section to the advancing tunnel face.

For tunnel deformation monitoring, a convergence section with three monitoring points (one at the crown and one on each of the left and right tunnel walls) was set up every 1-2 m along the length of the top heading tunnel after excavation. Deformation monitoring continued in the full tunnel section until the steel liner installation was completed at the station. Notably, no significant tunnel deformation was seen during the top heading excavation and after the benching excavation.

11. CONCLUSIONS

The paper presents detailed work experience on the implementation of reinforced canopy pipes fore-poling in high-pressure grouting areas within the penstock of the Jatigede Hydroelectric Power project. This was done to manage water inrushes and mudflows, and to provide pre-excavation support. The study also describes the utilization of an admixture, rw-HPC, which enhances cement setting time and strength properties and water glass injection to control material permeability and grout communication. Grouting parameters were tailored to accommodate geological uncertainties and interactions with ground grouting materials. This adaptive strategy successfully rehabilitated tunnels following collapses triggered by water and mud inrushes, making it applicable for weak, water-saturated tunnel sections. Safety measures encompassed high-pressure grouting in loose ground, reinforced canopy pipes for tunnel crown support, and a comprehensive support system during excavation, which included central rock core support and steel reinforcement. While water pressure testing for grouting performance verification was not conducted, inspection grouting was utilized. The tunnel excavation proceeded steadily with meticulous top-heading and benching excavation. The work experience underscores a thorough approach to weak rock tunnel construction, ensuring safety and stability throughout the excavation.

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