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Groundwater Management Strategies for Ensuring Tunnel Safety at the Hospital Kuala Lumpur Station

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ABSTRACT: *This paper presents a detailed case history of groundwater management implemented to ensure the safety of temporary twin TBM bored tunnels within the excavation and construction of an underground metro station in an urban environment. The metro station, constructed using the top-down construction method with tunnels bored prior to the deep excavation, is located within the Kenny Hill Formation in Kuala Lumpur, Malaysia, interfacing with the weathered quartz intrusions. The weathered material of the quartz intrusions consists of significant amounts of medium and fine sand, which are considered to be erodible. The presence of groundwater and highly permeable materials posed risks of tunnel flotation and challenges to preserving the stability and safety of the deep excavation. This study focuses on the methods and techniques employed to temporarily manage groundwater levels within the excavation to minimize the subsequent impact on the tunnel integrity. Through comprehensive monitoring and data analysis, the effectiveness of the groundwater management strategy is evaluated. The findings demonstrate that well-planned and executed groundwater management can significantly enhance tunnel safety, minimizing the risks associated with tunnel flotation and instability. This case history provides valuable insights into effective groundwater management practices in similar geological and urban settings.*

KEYWORDS: groundwater management, hydrogeology, tunnel safety, tunnel flotation, deep excavation, weathered quartz

SITE LOCATION: Geographic Database

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INTRODUCTION

The Klang Valley Mass Rapid Transit (KVMRT) project consists of three metro lines following a "wheel and spoke" concept, with two northwest-southeast radial lines and one circle line looping around Kuala Lumpur, Malaysia. The two radial lines – the Kajang Line and the Putrajaya Line – achieved full line opening on 17 July 2017 and 16 March 2023, respectively. Upon execution and completion, the Circle Line will further enhance connectivity in Kuala Lumpur by looping around the city's business districts.

The development of this large-scale infrastructure project has not only opened up tremendous opportunities and new frontiers for geotechnical engineering in Malaysia but also provided a wealth of information on unique geotechnical challenges and accomplishments. It has also led to technological breakthroughs and design innovations in underground space engineering. Khoo & Ooi (2023) provide a summary review of the geotechnical challenges faced, and innovations created during the urban underground construction of this project.

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Underground construction in the Klang Valley is intensified by the inherent geotechnical challenges posed by complex ground conditions, which range from hard granite and heterogeneous Kenny Hill Formation to extreme karstic limestone with fully developed weathered profiles and soft recent deposits, including alluvium and mine tailings that are under-consolidated in places due to past mining activities. Each underground station presents its own unique challenges, making it crucial to document each case history and lesson learned for future reference. Some key references can be found in Khoo & Ooi (2023) and Khoo & Abdul Rahman (2024a, 2024b).

THE CASE STUDY

The Project Site

The case study presented in this paper focuses on a 28.9 m deep excavation and tunnelling works at the Hospital Kuala Lumpur Station, which is part of the KVMRT Putrajaya Line. This station, measuring 139 m in length, features a main trapezoidal-shaped underground structure with three entrances and above-ground ventilation structures. Twin bored tunnels are designed to connect to both the east and west ends of the station box.

Due to the full development of the hospital area, the metro station had to be located opposite the hospital, connected by a 110 m long underground adit crossing Jalan Tun Razak, one of the city's main roads. The main road accomodates eight lanes of vehicle traffic, four of which ramp up along a fly over. To the east of the station is Istana Budaya, a national theater and heritage building. Several nursing hostels (four-story apartments) and bungalows are situated to the north of the station. The existing Malaysia Medical Academic building to the west was demolished to make way for the metro development and future transit-oriented development. The site is relatively flat, with elevations ranging from Reduced Level (RL) 39.0 m to RL 41.8 m. Figure 1 illustrates the site location and surrounding structures.

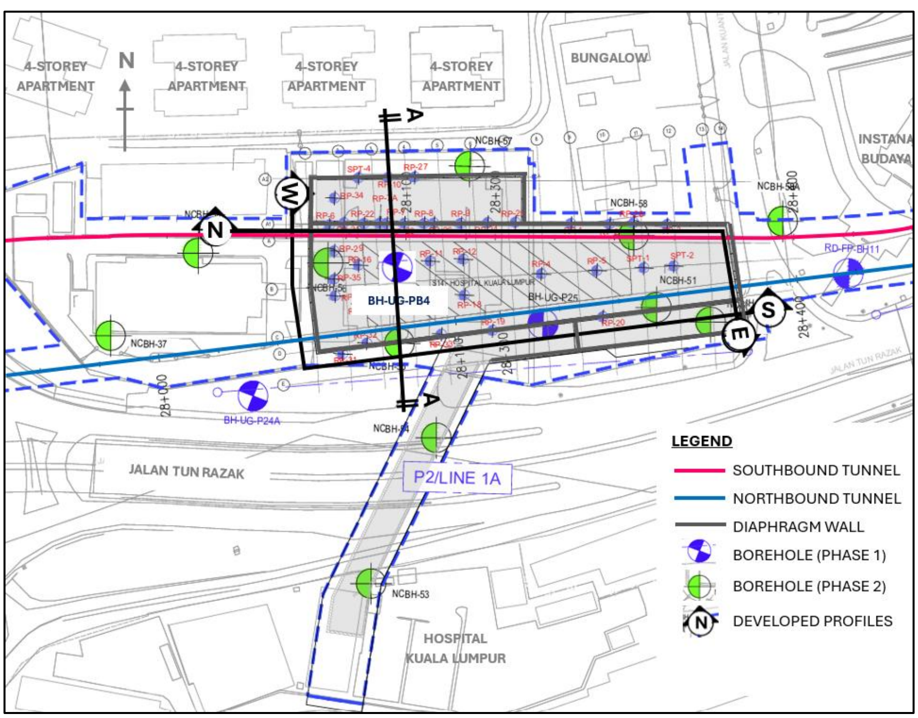

Figure 1. *Site location plan.*

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DOI: 10.4417/IJGCH-08-02-03

Geological Conditions

The geology of the site area is complex. According to the published geology map (Figure 2), the station footprint is expected to be underlain by the Kuala Lumpur Limestone, in close proximity to the interfaces of the Kenny Hill Formation and the Quartz Dyke. The Quartz Dyke, a manifestation of the Kuala Lumpur Fault Zone, trends approximately ENE-WSW (with a general strike of around 100° to 105°) along the northern boundary of the Hospital Kuala Lumpur Station. However, multiple stages of site-specific exploratory boreholes (see Figure 1) have confirmed that the site is actually underlain by the Kenny Hill Formation with weathered quartz intrusions, rather than the Kuala Lumpur Limestone Formation. Tan & Komoo (1990) and Tan (2017) provide a comprehensive outline of the geology in Kuala Lumpur, offering comparative insights into the Kenny Hill Formation, Kuala Lumpur Limestone, and igneous intrusions (including quartz veins).

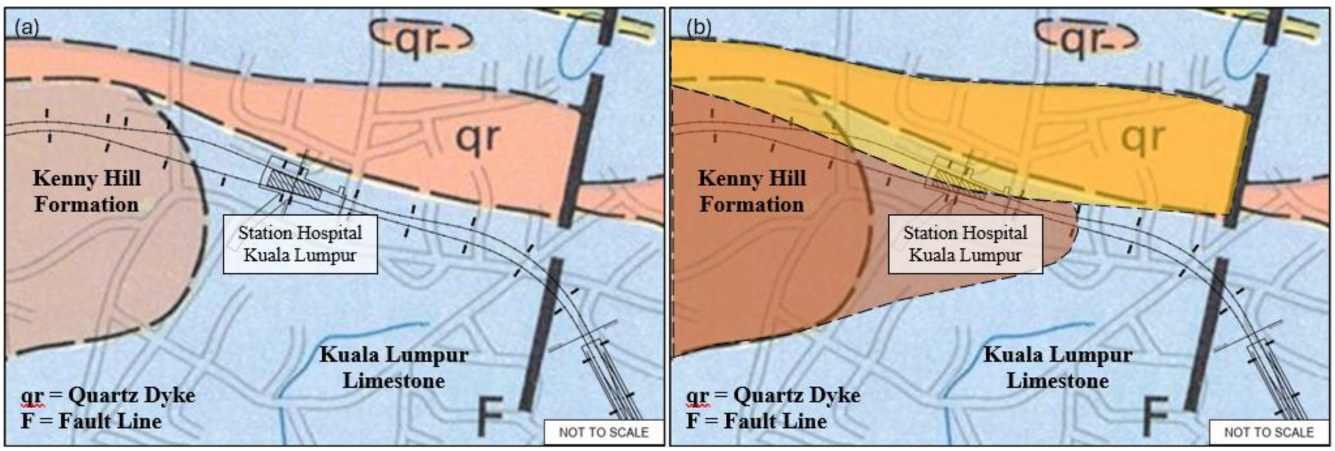

Figure 2. *Bedrock geology (a) based on the published map, and (b) based on the authors' geological interpretation.*

The Quartz Dyke fills planes of weakness or voids in the rocks formed during faulting. It can be several meters wide and extends up to a few hundred meters. The characteristics of the Quartz Dyke cutting through other host rocks are illustrated in Figure 3. Where quartz is much more resistant to weathering than the host rock, it generally forms vertical wall-like structures that rise above the typically lower and more irregular host rock surface (Yeap, 1985). In Kuala Lumpur, most of the faults are sub-vertical, so the quartz dykes filling these faults are also sub-vertical. A prominent example is the Klang Gates Quartz Ridge, located about 5-7 km northeast of Hospital Kuala Lumpur Station. This large vertical dyke/ridge fills the Kuala Lumpur fault to the north of the city. Geotechnically, there are limited published references on these materials.

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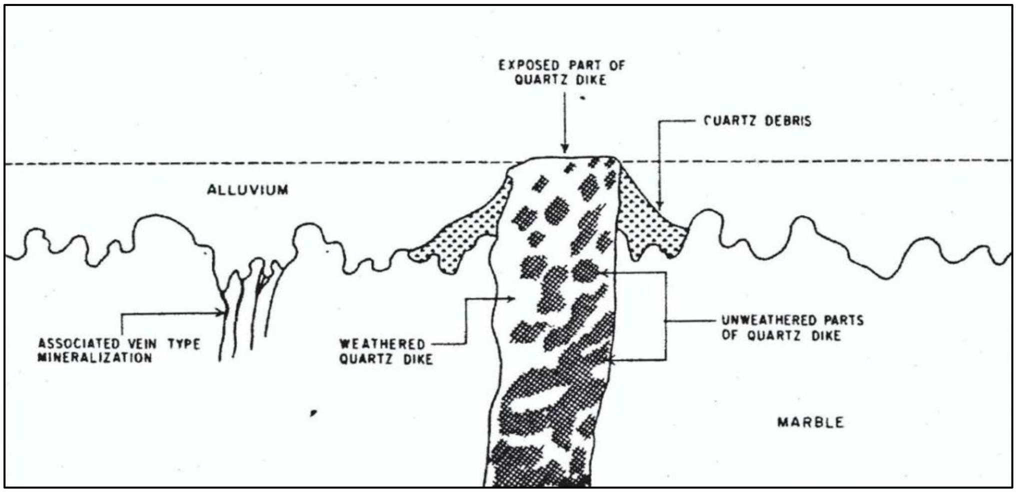

Figure 3. *Schematic section through a quartz dyke in marble (Yeap, 1985). The sketch closely reflects the actual site conditions.*

The Kenny Hill Formation, deposited after the Kuala Lumpur Limestone, consists of a monotonous sequence of metamorphic rocks, primarily quartzite and phyllite. These rocks can be intruded by quartz veins. The phyllite and quartzite beds range from a few meters to several meters thick, with phyllite typically being finely laminated and quartzite mostly fine to medium-grained with occasional phyllite lenses.

The damp tropical climate, with daily temperatures ranging from 23° to 32° Celsius and an average annual rainfall of 2,600 mm, has caused the Kenny Hill Formation to undergo intense chemical weathering. The overburden soils, which result from this weathering, depend heavily on the mineral composition of the parent rock. Raj (1983) reported that the distribution of different weathering stages within the residual soil profile is extremely complex. Typically, clay soils are derived from phyllite, and clayey sand from quartzite. The weathered soils of the Kenny Hill Formation are normally characterized by the interbedding of sandy silty clay and clayey silty sand, with gravelly materials also common. Toh et al. (1989) and Wong & Singh (1996) have discussed some engineering properties of the Kenny Hill Formation in Kuala Lumpur.

Geotechnical Characterization

A total of 11 exploratory boreholes and 35 rock probing tests were conducted within the site to establish the subsurface stratification. The inferred subsurface profiles along the northeastern and southwestern boundaries of the station box are depicted in Figures 4(a) and 4(b), respecively. Geologically, the northern side is underlain by weathered quartz, aligning with the published geology map as discussed above, whereas the southern side is predominantly composed of the Kenny Hill Formation. This formation consists of the residual soil of quartzite and phyllite, occasionally with quartz intrusions. Above these strata is a layer of young alluvium (SPT < 10) below the ground surface, ranging from 7.5 m to 15 m thick, interbedded with soft to medium stiff cohesive and loose cohesionless soils.

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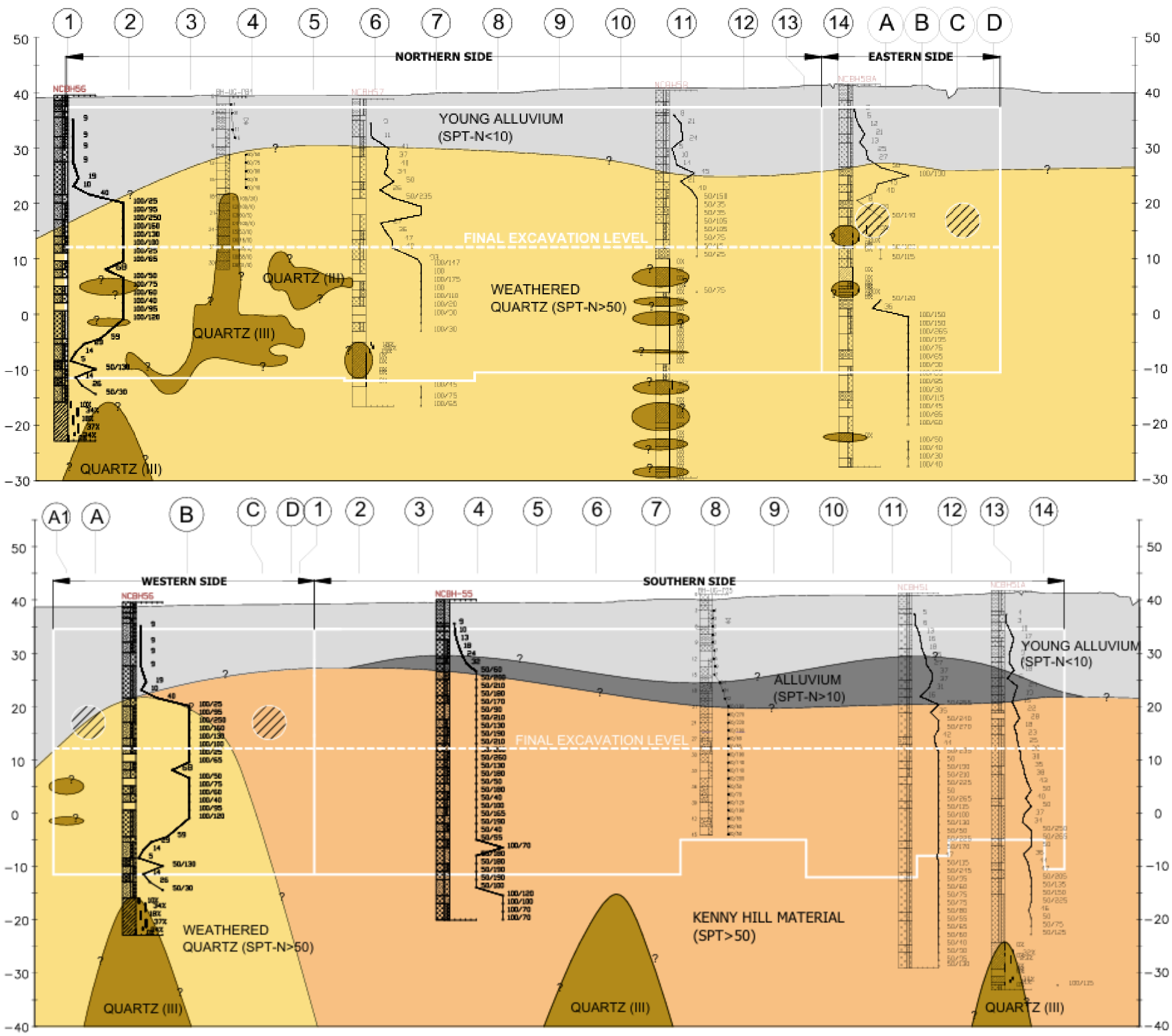

Figure 4. *Inferred longitudinal subsurface profiles along northern side (top) and southern side (bottom).*

The weathered quartz underlying the alluvium along the northern side appears as whitish, medium to coarse-grained, and angular to sub-angular sand with an SPT value greater than 50. Figure 5 shows a weathered quartz sample obtained in SPT split spoon. As illustrated in Figure 3, the unweathered portions of quartz are often found between layers of weathered quartz, which exhibit an undulating to sub-vertical profile with poor core recovery and a Rock Quality Designation (RQD) of 0%. Notably, the quartz encountered at borehole BH-UG-PB4 (see Figure 1) extended vertically into the station formation level by approximately 9 m. This quartz, typically found as sporadic porous crystalline matrix, has a Cerchar Abrasiveness Index (CAI) ranging between 2 and 5.

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES

DOI: 10.4417/IJGCH-08-02-03

Figure 5. *Typical weathered quartz obtained in SPT split spoon (Boon et al., 2023).*

Along the southern side, a thin alluvium bed of an SPT > 10 and thickness varying from 3 m to 9 m was encountered before the Kenny Hill Formation. This layer is predominantly grey and composed of stiff to hard clay/silt with SPT values ranging from 18 to 50. The residual soil of the Kenny Hill Formation, comprising stiff to hard sandy silt and dense silty sand, underlies the alluvium. The SPT values are generally greater than 50.

Figure 6 presents a typical transverse subsurface profiles through the station box, showing that the station box is underlain by two distinct geological units. Interestingly, the soil permeabilities also reveal two clusters, with the weathered quartz being more permeable, as shown in Figure 7. For the weathered quartz, Lugeon values vary between tight to 30 Lu based on the in-situ permeability tests carried out.

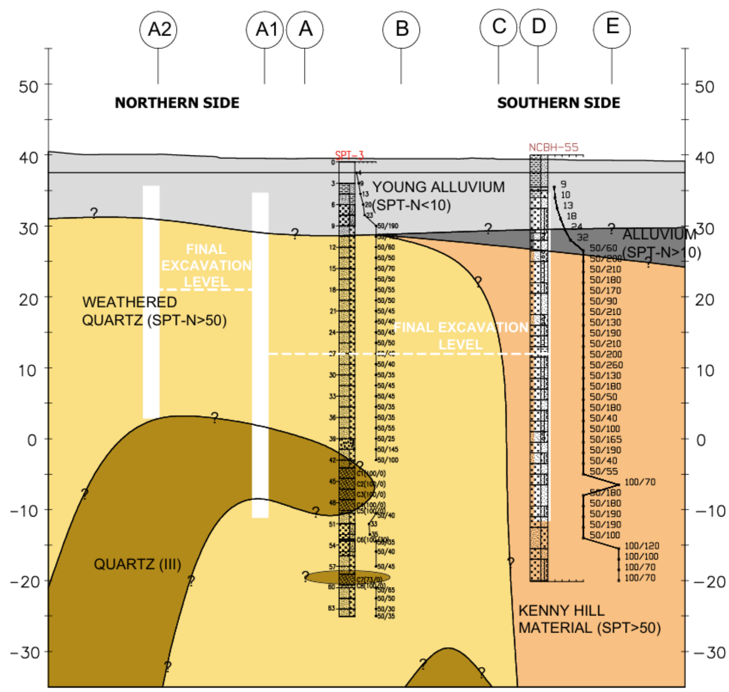

Figure 6. *Inferred transverse subsurface profiles at Section A-A.*

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES
DOI: 10.4417/IJGCH-08-02-03

Figure 8 presents the range of index properties of the Kenny Hill's materials and weathered quartz collected from the site. Overall, the results indicate that the Kenny Hill materials exhibit higher plasticity and moisture content compared to the weathered quartz. The liquid limit and plasticity index of the Kenny Hill samples generally fall within the range of low to medium plastic clays, whereas the weathered quartz samples show very low plasticity, consistent with their coarse and silty composition. The fines content for the weathered quartz is comparatively lower and more variable, reflecting its heterogeneity and higher permeability. These contrasts highlight the distinct geotechnical behaviour between the two geological units, particularly in terms of compressibility and groundwater response.

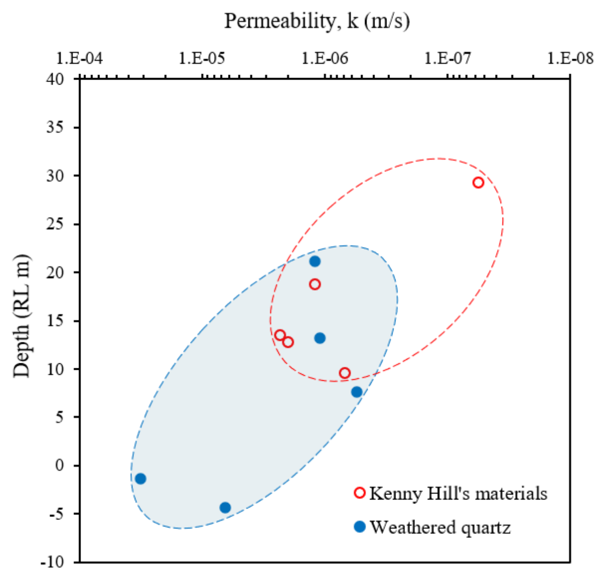

Figure 7. *Soil permeability clusters.*

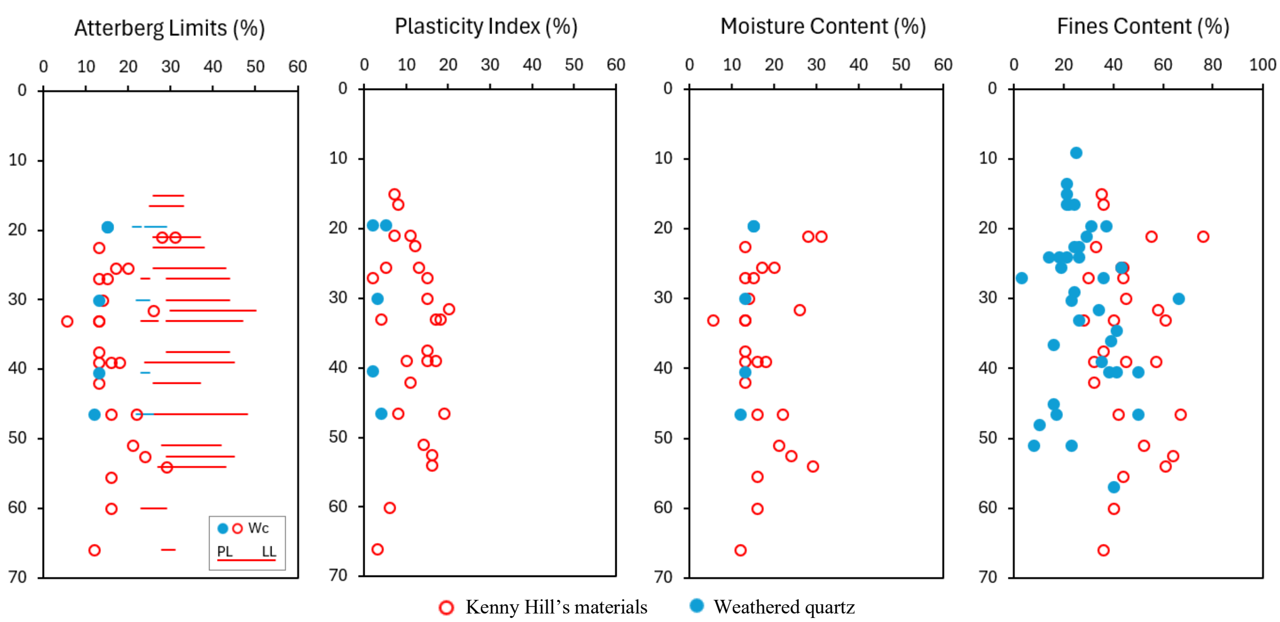

Figure 8. *Soil index properties – Kenny Hill's materials vs. weathered quartz.*

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The natural groundwater table follows the ground topography and was generally measured at 3.4 m to 5.7 m below ground level, resulting in a band of water fluctuation between RL 35.5 m to RL 37.4 m.

Design and Construction Scheme

The three-story underground station, with a trapezoidal-shaped box varying from 56.4 m wide at the west to 23.7 m wide at the east, was constructed using a top-down method. The main box is surrounded by 1.2 m thick diaphgram walls, and the site entrances have 1 m thick diaphgram walls due to shallower excavation requirements. Lateral earth restraints were provided by the permanent slabs supported by plunge-in columns where required. The Tunnel Boring Machines (TBM) bored through the station after the installation of the diaphgram walls and before casting the roof slab. A typical cross-section describing the construction sequence is illustrated in Figure 9.

The toe levels of the diaphgram walls were determined based on excavation stability checks and the provision of load-bearing requirements for future top-side development. Refer to Figures 4(a) and 4(b) for the indicative developed diaphragm wall elevations.

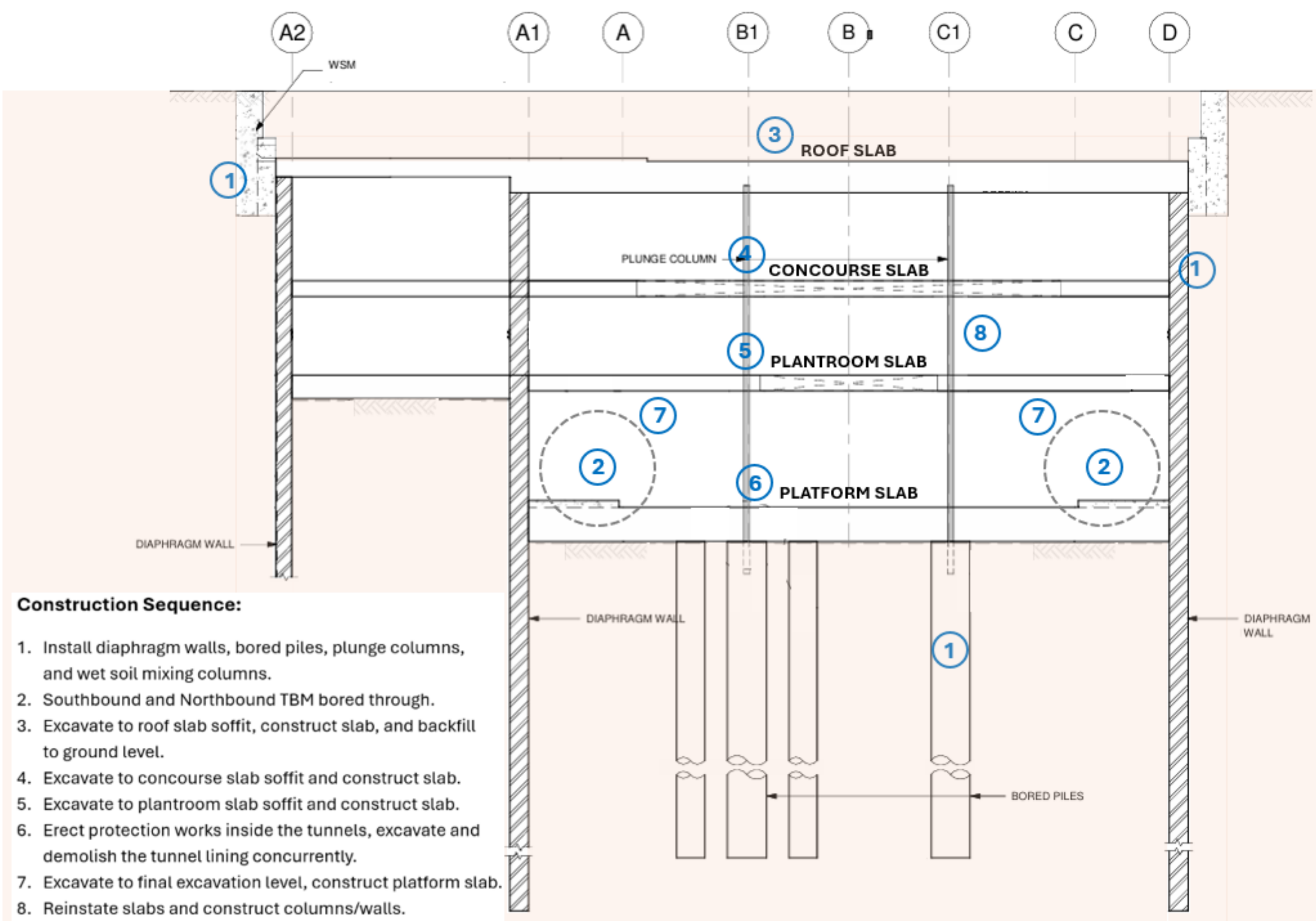

Figure 9. *Typical cross-section of the station and construction sequence.*

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES

DOI: 10.4417/IJGCH-08-02-03

CHALLENGES AND POTENTIAL RISKS

Excavation Stability

The excavation stability for a station with TBM bore-through construction requires additional considerations compared to a conventional deep excavation design. In this case, the tunnels are situated on the passive side of the diaphgram wall, where the soil cover above the tunnel decreases as excavation progresses (see Figure 10). This scenario increases the risk of tunnel deformations and movement. The tunnel must not fail prior to the casting of the Plant Room Slab (Stage 5 in Figure 9). The failure of the tunnel ring is akin to an unplanned excavation from the current excavation level; i.e., taking the last restraint as the Concourse Slab (Stage 4 in Figure 9) down to the depth of the tunnel invert (which is close to the Final Excavation Level). This would lead to increased stresses in the retaining system (Figure 10). Understanding the interaction between the tunnel ring and the wall is crucial in ensuring that appropriate failure mechanisms are not overlooked in the design of earth retaining and stabilizing structures. Boon et al. (2016, 2023) discussed this design concern with relevant case studies. The additional forces can arise from: (i) the reduction of soil cover above the tunnel crown, compounded by the lateral forces from the diaphragm wall, and (ii) water uplift pressures leading to tunnel flotation which can arise from buoyancy and exacerbation by seepage, and which would further increase the water pressures on the retained side of the excavation.

The tunnel lining may fail when the ovalisation forces exceed its ultimate structural capcity, and may float when the dead weight of the tunnel lining and overburden above it is unable to resist the water uplift pressures. As a countermeasure, the tunnels within the bore-through station section can be either (i) temporarily backfilled to reinstate the passive resistance against the retaining wall and to overcome tunnel flotation, or (ii) the tunnel segment linings can be designed with greater capacity to prevent excessive deformation, provided that the tunnel stability and ultimate limit state condition are safeguarded, to allow stress transfer from the wall to the soil. The second solution, however, may not resolve the problem of flotation. In this case history, the latter solution was adopted, with the tunnel lining designed to accommodate the reduction of overburden during progressive excavation and the increased lateral deformation of the adjacent diaphgram wall. This approach was coupled with a site observational strategy to monitor and manage the situation in real time.

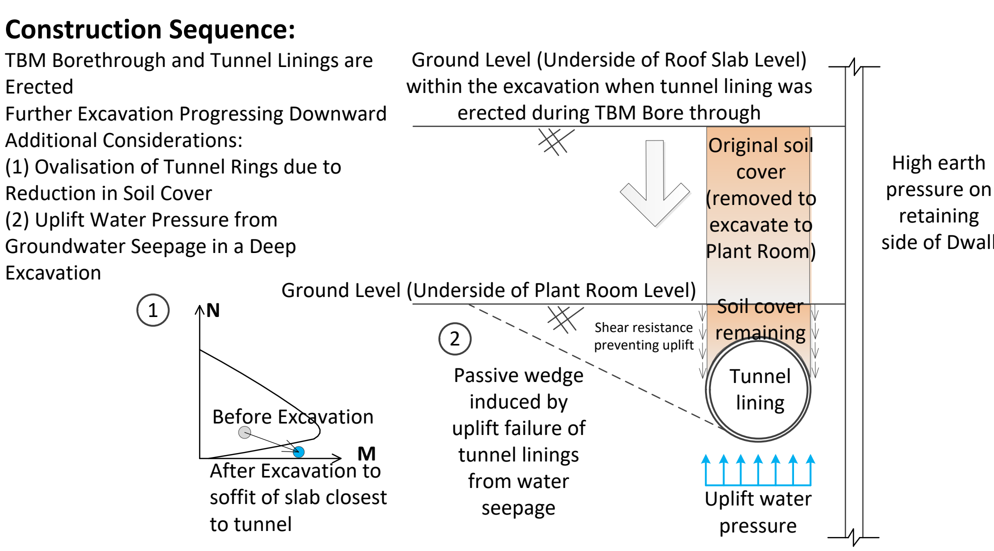

Figure 10. Illustration of potential loads on tunnel lining with a bore-through construction sequence (modified after Boon et al., 2016).

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES
DOI: 10.4417/IJGCH-08-02-03

Tunnel Flotation

The buoyancy forces have to be resisted by dead weight of the tunnel lining, the soil cover and the shear resistance of the soil above it (Figure 10). In deep excavations, the groundwater forces may exceed the buoyancy forces, due to the presence of groundwater seepage, if the retaining wall is not socketed into an impermeable strata, i.e. an aquitard. Depending on the permeability of the soil, the efficiency of the some solutions to overcome flotation may be affected. For example, for less permeable ground, where the water inflow is negligible or manageable, the water pressure may be relieved by coring-through the segments given that the segments within the excavation are temporary. However, for permeable ground conditions with risk of eroding material, this may inhibit the tunnelling operations and safety. As excavation progressed at this site, water seepage observed on the northern side was greater than typically experienced in the Kenny Hill Formation due to the more permeable weathered quartz discussed in the previous section. Verification was carried out at a few tunnel rings by turning on the valve of grouting ports while excavation was still at a higher level. For some tunnel rings located in the envisaged quartz intrusion area, when the valve was released, water carrying fines flowed into the lining (Figure 11). In contrast, tunnel rings on the southern side, within the Kenny Hill Formation, remained relatively dry.

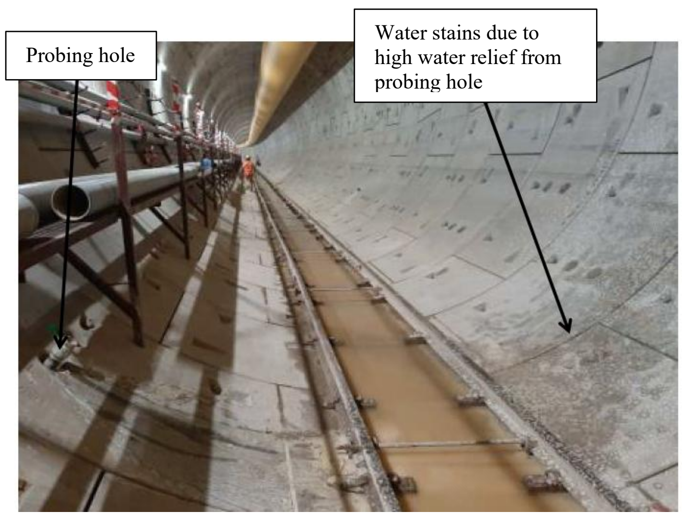

Figure 11. *Observation of high water seepage through temporary tunnel probing.*

GROUNDWATER MANAGEMENT STRATEGIES

Active Relief Wells at the Northern Side

Further construction-stage observations and assessments revealed that the presence of water and highly permeable materials posed substantial risks that the water pressures may be high and the relieving of water pressure directly from the segments is not feasible, given that this may also lead to the undesirable washing-in of fines within the weathered quartz unit. To mitigate the risk of tunnel uplift as excavation progresses from the underside of Concourse Slab to the underside of Plant Room Slab (Figure 9), pre-planned contingency active relief wells in the northern section were activated to operate in tandem with the excavation. Figure 12 illustrates the arrangement of the relief well system, showing both the cross-section of the excavation and the construction details of the well. As shown, each relief well consists of a slotted PVC casing installed within a gravel filter, with a submersible pump positioned near the well toe to exact groundwater from the highly permeable quartz layer.

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES

DOI: 10.4417/IJGCH-08-02-03

The pumping was comissioned before excavating below the Concourse Level, and the groundwater level for the extraction was regulated to the Final Excavation Level (FEL). This resulted in controlled groundwater lowering within the deep excavation, without exceeding the originally planned drawdown level at the FEL.

Monitoring of the tunnel lining with optical prisms, comprising convergence and heave measurements, was conducted along the tunnel lining, with monitoring point spacing determined based on initial observations of water inflow. Excavation progressed in stages from east to west. As an additional mitigation measure, the progress for the Plant Room Slab casting was managed so that the concrete slab casting activity was prioritized in order to limit the longitudinal unrestrained span of the tunnel lining. The monitoring results and their interpretation will be discussed in the subsequent section of this paper. Note that the monitoring of the tunnel segments is typically carried out at locations deemed critical, for example at cross-pages, adits, or close-proximity tunnelling. The monitoring of this location within the station was a special requirement arising from the observations discussed here.

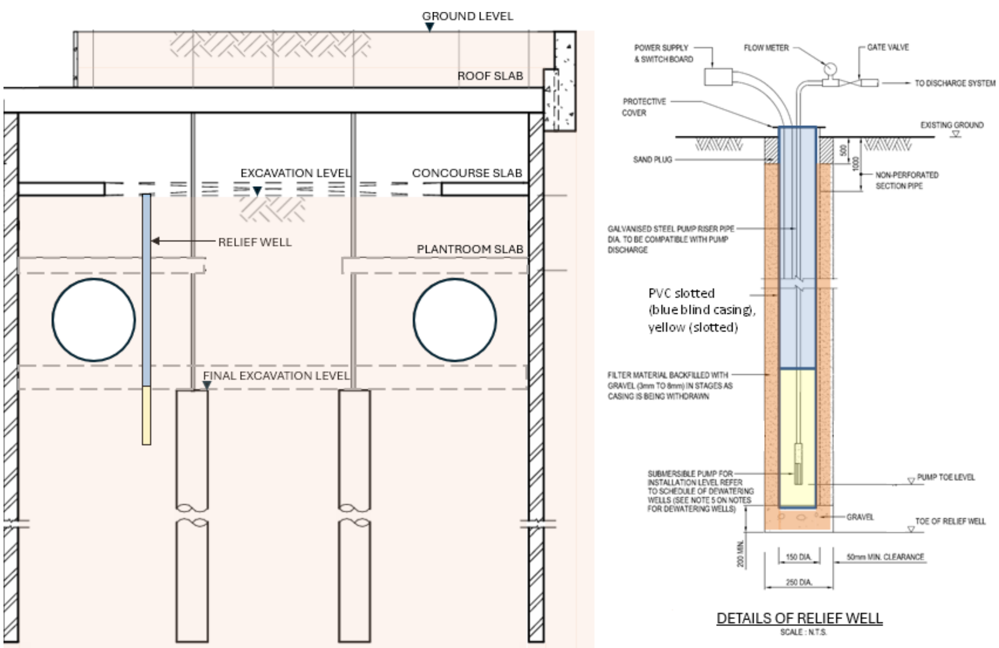

Figure 12. *Active relief well introduced in the northern section of excavation.*

Risk and Impact Assessment of Dewatering

While dewatering serves as a mitigation measure to ensure tunnel safety, the risks and impacts of groundwater lowering on adjacent structures are mitigated with instrumentation monitoring coupled with a monitoring action plan. The building structures in the vicinity were monitored with ground settlements being the key performance indicator. Most of the structures were planned to be demolished to make way for the metro station and future redevelopment, and were predominantly on piled foundations which are less susceptible to groundwater drawdown effects.

Nevertheless, based on the monitoring of groundwater levels, recharge wells were comissioned in several phases around sensitive receivers, such as adjacent to sensitive buildings and critical utilities. The purpose of these recharge wells was to minimize the

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impact of groundwater level drawdown, where possible, with the understanding that settlement monitoring would provide a more direct measure of the ground performance. Four to five numbers of recharge wells were installed down to a depth of approximately 12-18 m. Figure 13 shows a site photo of the recharge well installed beside a piezometer standpipe.

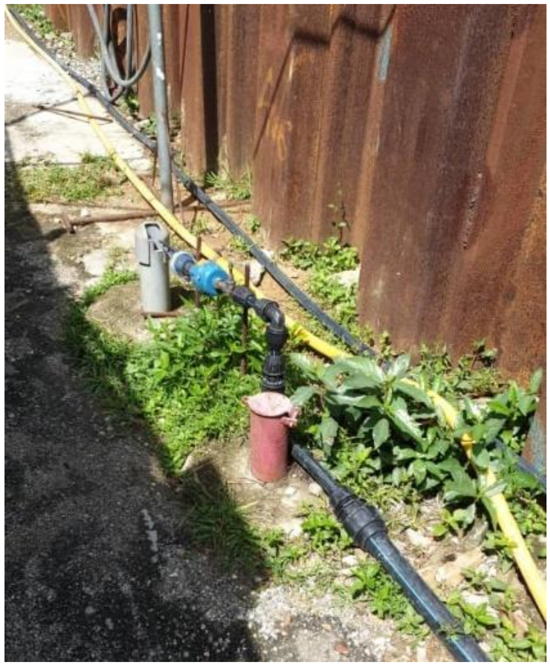

Figure 13. *Site photo showing recharge well and piezometer standpipe.*

INSTRUMENTATION MONITORING

Hydrogeological Regime and Groundwater Monitoring

Instrumentation monitoring is a crucial component in the overall strategy for managing the risk of groundwater lowering, particularly concerning groundwater response to ground settlement and the manifestation of settlements. Consequently, an extensive and carefully designed instrumentation regime was integral to ensuring that any inevitable differences between design prediction and construction could be addressed swiftly.

A comprehensive suite of monitoring instruments was employed, including water standpipes, vibrating wire piezometers, ground settlement markers, inclinometers, building settlement markers, tilt meters, and optical prisms. Each instrument was strategically placed at specific locations and/or depths to maximize the potential benefits and provide valuable feedback. The instrumentation for groundwater monitoring is illustrated in Figure 14.

INTERNATIONAL JOURNAL OF
GEOENGINEERING
CASE HISTORIES

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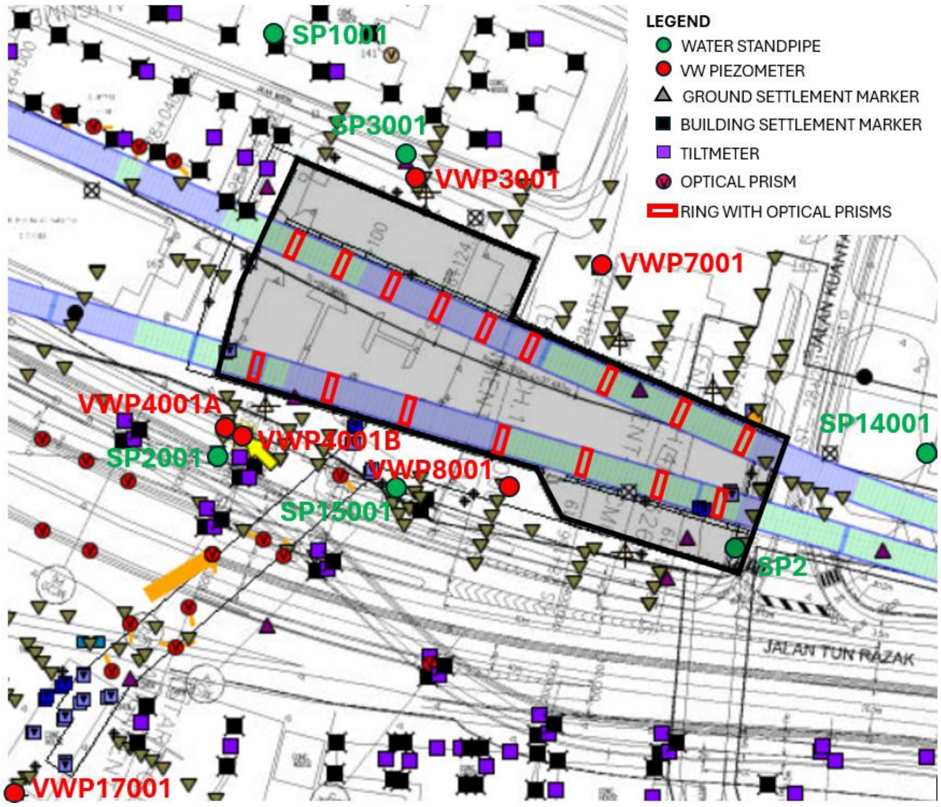

Figure 14. *Instrumentation for groundwater monitoring.*

Note that the active relief wells (with pumps) were installed midway during excavation at the Concourse Level within the excavation footprint, so that the water pressures below the tunnel invert elevation could be relieved. The objective of the relief wells was not to drawdown the water level within the excavation footprint entirely, but merely to avert the risk of uplift close to the tunnel. The water level was measured directly at the relief wells, instead of having additional standpipes or vibrating wire piezometers within the excavation footprint. Based on site observations, the pumping was only intermittent, as the pumps were set to activate only when the water would be higher than the pre-set level. The pump volume for each well was 1-2 m^3/hour, being placed next to the tunnel ring (see Figure 12).

The recorded groundwater levels throughout the entire construction period, including the groundwater control activities, are presented in Figures 15(a) and 15(b) for the northern and southern sections, respectively. The groundwater level profile clearly shows significant drawdown, particularly in the northern section, where a lowering of about 10 m was observed. However, the groundwater rapidly recovered once the base slab was cast, following the completion of excavation to the final level, during which the removal of the tunnel lining was carried out concurrently. In contrast, the groundwater response in the southern section was more gradual and relatively insignificant, except at VWP4001B. This contrast highlights the influence of local soil conditions on hydrogeological behaviour, where the northern section comprising predominantly weathered quartz with relatively higher permeability exhibited a more rapid drawdown, whereas the sourthern section, underlain by more clayey Kenny Hill materials, showed a slower and less pronounced groundwater response.

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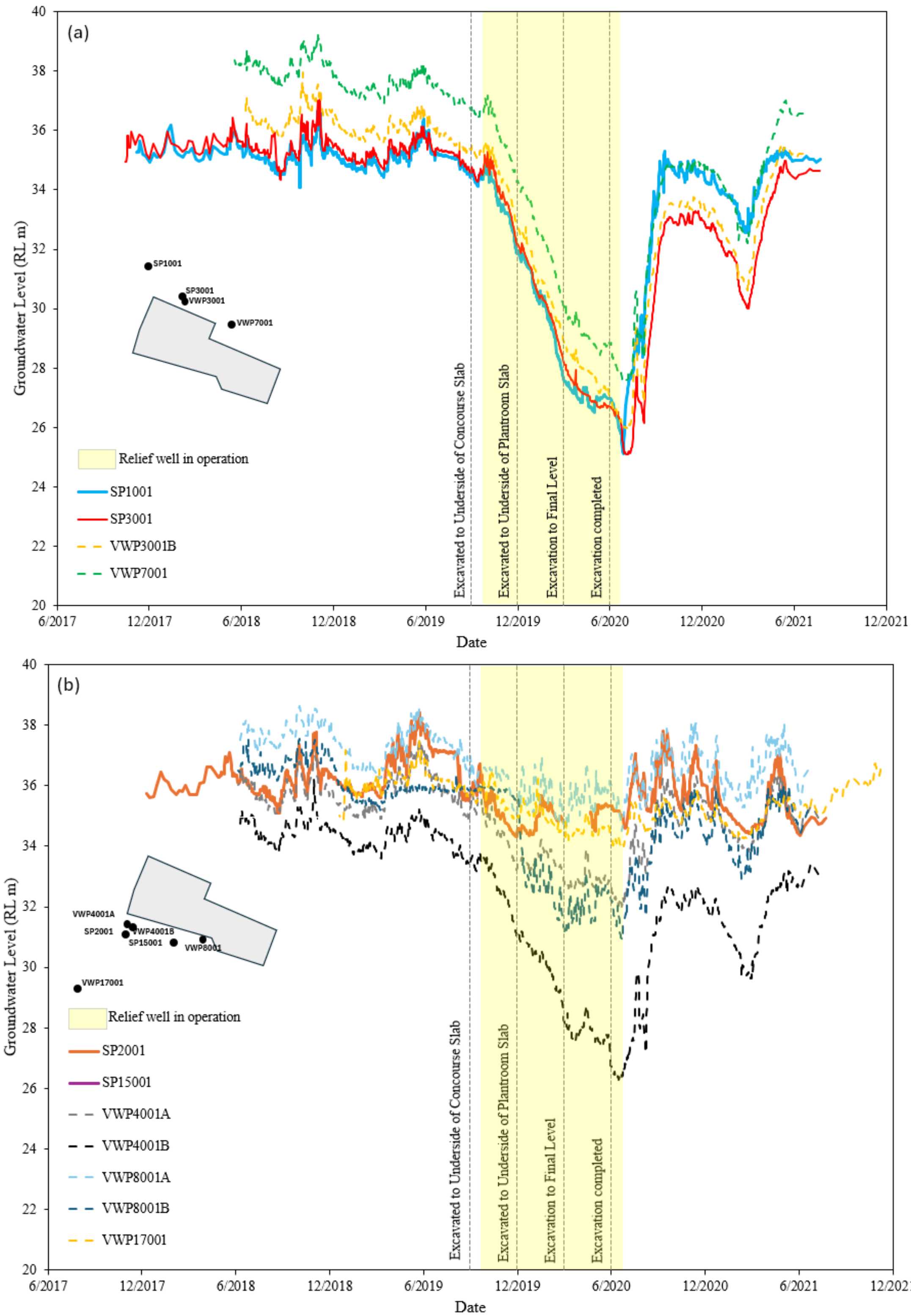

Figure 15. *Monitored groundwater levels throughout the construction (a) northern section (predominantly weathered quartz), and (b) southern section (predominantly Kenny Hill materials).*

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Ground and Building Settlements

The recorded ground settlements throughout the same period are presented in Figures 16(a) and 16(b) for the northern and southern sections, respectively. An array of ground settlement markers was installed perpendicular to the excavation, with series GSM1000, GSM3000, and GSM5000 located in the northern section, and series GSM2000 and GSM4000 in the southern section.

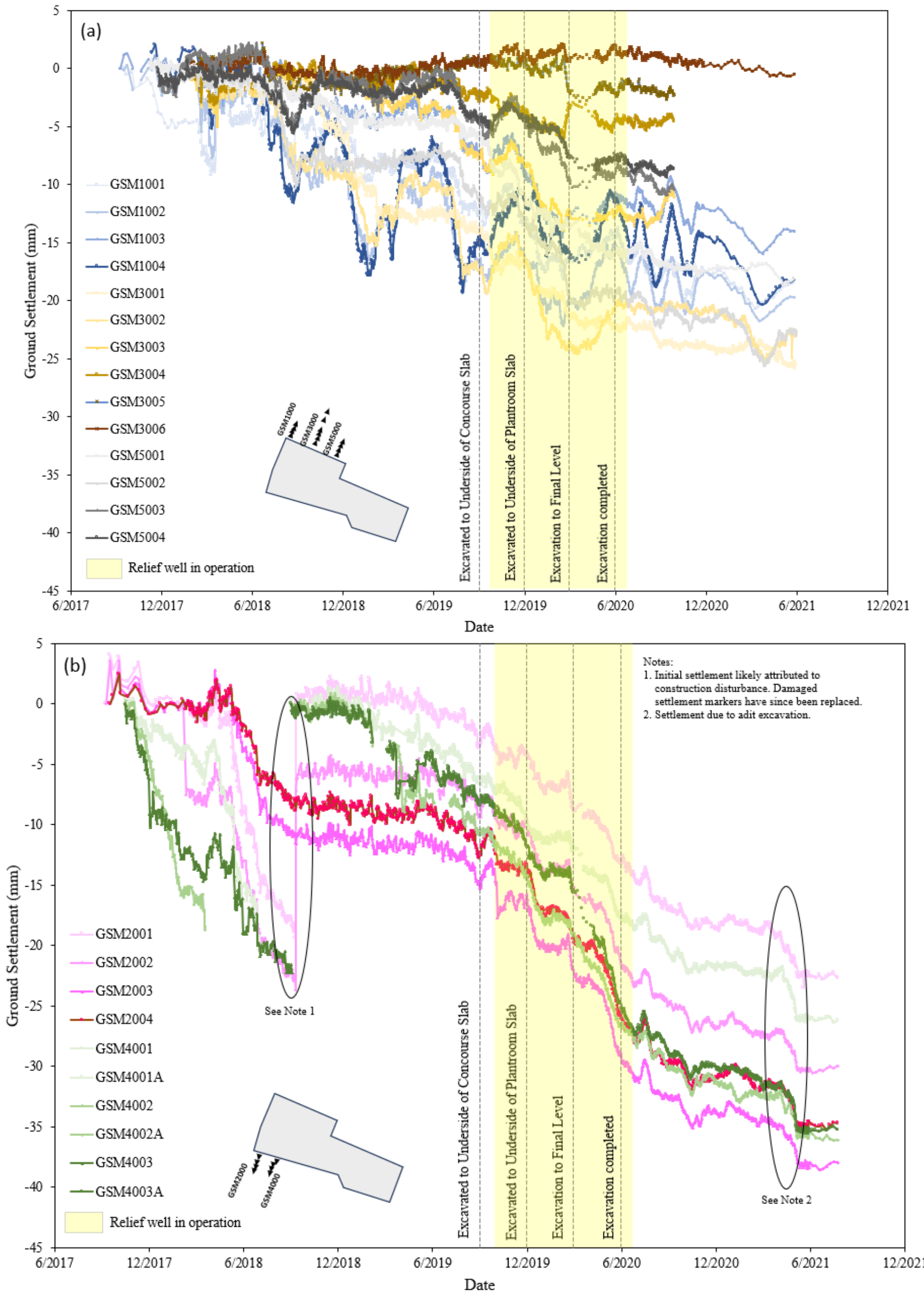

Figure 16. *Recorded ground settlements throughout the construction (a) northern section, and (b) southern section. Some of the settlement markers were within the site compound and affected by mobilization of plant and machinery during the earlier phases of the project.*

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES
DOI: 10.4417/IJGCH-08-02-03

The ground settlement profile appears to be more directly related to ground movements caused by bulk excavation within the station box, where significant stress relief and wall deflections occurrred due to the removal of overburden. In contrast, the correlation between settlement and groundwater drawdown is less distinct, suggesting that the effects of groundwater lowering on ground settlement are secondary and site-dependent. Ground settlements were first registered in tandem with ground movements attributed to the installation of the diaphragm wall, coupled with excavation to the Roof Slab Level. Building settlement monitoring for the apartment blocks located on the northern side showed negligible response to both the groundwater lowering and the deep excavation, as evidenced in Figure 17. This can be attributed to their pile foundations, which extend well below the excavation level, thereby minimizing the influence of stress changes and groundwater drawdown on the foundation level.

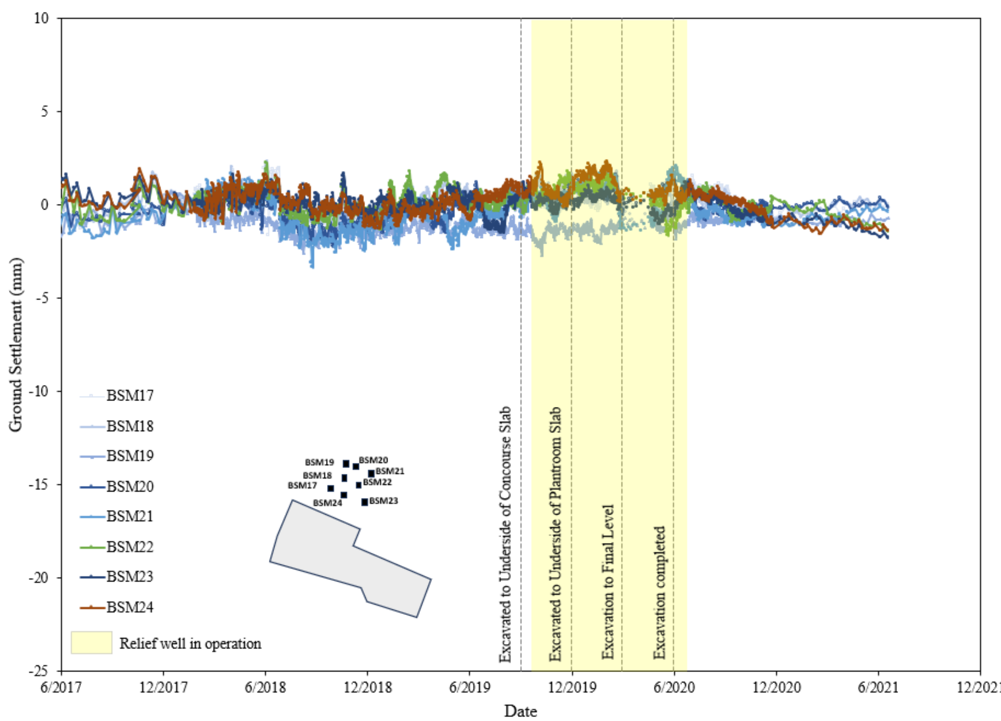

Figure 17. *Recorded building settlements.*

CONSTRUCTION PERFORMANCE

Tunnel Lining Performance

The tunnel linings were monitored using optical prisms to measure displacements (heave) and convergence. The monitoring spans during the critical phase of excavating between the Concourse Level to the Plant Room Level (Figure 9), where the soil cover above the installed tunnel linings was reduced during excavation. During this period, the relief wells were activated The interpreted results comprising the maximum values for heave and convergence of the optical prisms are presented in Figures 18(a) and 18(b), respectively, with the optical prism configuration shown in Figure 18(b). As noted earlier, excavation from the Concourse Level to the underside of the Plant Room Slab, leaving only a 1.4 m soil cover above the tunnel linings, commenced from the eastern end closer to the Kenny Hill Formation and progressed westward.

INTERNATIONAL JOURNAL OF GEOENGINEERING CASE HISTORIES

DOI: 10.4417/IJGCH-08-02-03

In tandem with the excavation, the plantroom slab was cast expediently to limit the “unrestraint” longitudinal span of the tunnel lining. The progressive movement with excavation occurring longitudinally from East to West is presented in Figure 18. It was observed that overall heave increased significantly as excavation advanced from 73.4 m to 107 m which is incidentally at the midspan of the excavation box between the two restraints provided by the diaphragm walls. Note that precast segmental rings, by virtue of their installation positions, are embedded in the diaphragm wall which would provide fixity to the the tunnel rings. The incremental heave began to taper off as excavation progresses closer to the support boundary condition provided by the diaphragm wall, i.e. the station end wall, as discussed in Boon et al. (2023). Similar observations were noted regarding tunnel convergence; however, it remained well within the trigger value. The magnitude of convergence observed was consistent with the expected deformation pattern arising from stress redistribution during staged excavation and partial unloading above the tunnel crown.

Interestingly, in comparison to similar conditions in the Kenny Hill Formation presented by Boon et al. (2016), the heave measured at the tunnel ring was of a similar magnitude to the convergence. This similarity observed previously may be attributed to the lack of water seepage at that site, where the ground was predominantly clayey and dry based on field observations. In contrast, the heave monitored in this Hospital Kuala Lumpur Station project site was three to four times greater, underscoring the heightened risk of tunnel flotation due to greater water inflow based on site observations, and inferred greater seepage forces. The resulting unbalanced vertical loads would induce greater uplift movement, as illustrated in Figure 10.

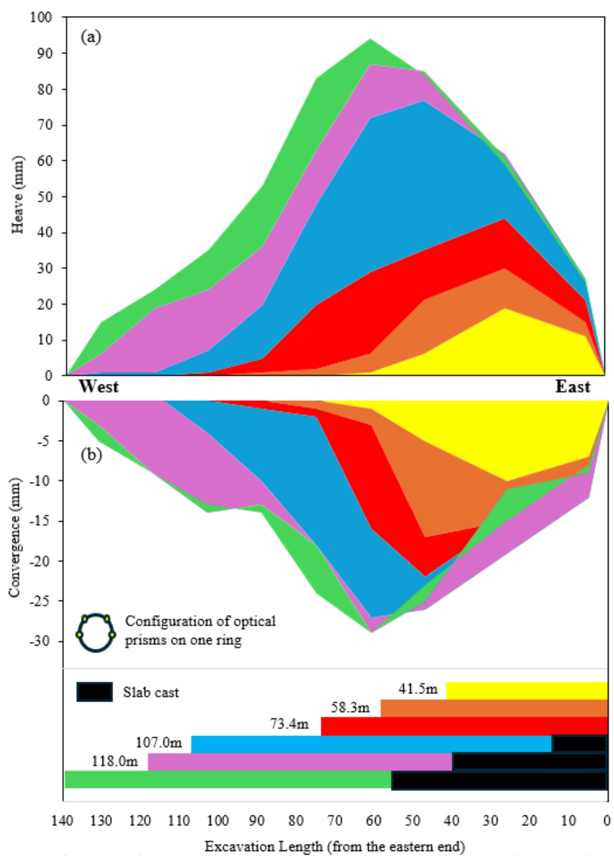

Figure 18. *Monitoring of tunnel rings (a) heave, and (b) convergence.*

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Diaphragm Wall Performance

The measured diaphragm wall deflection using inclinometers from the concourse to final excavation level is presented in Figure 19. The increase in diaphragm wall deflection during the excavation from the Concourse Level to the Plant Room Level is of almost similar magnitude compared to the convergence measured in the tunnel rings (Figure 18). This observation indicates a consistent relationship between the wall deflection and the tunnel convergence, suggesting that both phenomena are influenced by similar factors during excavation. A monitoring plan was in place to ensure that the diaphragm walls are monitored against the design predictions, i.e. Alert, Action and Alarm. The Alarm value was set as the predicted value for the retaining walls of deep excavations (being 67 mm in this case). Based on Figure 19, the measured deflection at the final excavation level was 55 mm, being less than the Alarm value of 67 mm. Typically, for deep excavation projects, where monitoring triggers are breached, in-place contingency measures would generally be to revise the predictions and propose mitigation action plans if there is insufficient reserve capacities in the retaining systems, such as introducing intermediate strutting. In this case, such measures were not required, as the deformations were within acceptable limits.

It is likely that the diaphragm wall moved during the TBM bore through and had been demonstrated in Boon et al. (2016) for another site. However, for the Hospital Kuala Lumpur Station, the excavation to the underside of the Roof Slab is 5-6 m below ground (Figure 9). While baseline monitoring was taken prior to excavation to the roof slab, the inclinometer sleeves through the diaphragm walls could only be resumed once backfill was completed above the roof slab to provide staging to access the inclinometers. Due to the lack of staging to access the inclinometers during TBM boring, only the total cumulative movement is available, comprising the TBM bore-through and excavation to the concourse level, after the roof slab level is backfilled (orange line in the inclinometer profile in Figure 19).

As the activation of relief wells occurred concurrently with the excavation commencement from the Concourse Level to the underside of the Plant Room level, the wall movement arising from the activation of relief wells alone were masked by the concurrent activities but thought to be beneficial.

As the project site was constructed using a top-down methodology, casting the intermediate slabs restraining the diaphragm wall more quickly with longitudinal excavation would enable the excavation to benefit from three-dimensional effects, as shown in Figure 18. This mitigation was consciously pursued at the Plant Room Slab level immediately overlying the tunnels. An expedient casting sequence would minimize the exposure of the tunnels to uplift and ovalization forces. This was further validated when comparing the tunnel heave with the slab casting dates, as shown in Figure 19. The tunnel upward heave stabilized after the Plant Room Slab immediately above it was cast. The diaphragm wall deflection was less affected by the longitudinal effects of excavation progress, likely because the diaphragm wall panels were well restrained in the adopted top-down construction method. Due to the lateral extents of the restraints provided by the reinforced concrete slabs, this will minimize the longitudinal flexure of the diaphragm wall panels significantly, by comparison to a steel waler and strut system. On the other hand, the tunnel heave was still found to be very sensitive to longitudinal excavation, as shown in Figure 18.

Overall, both the excavation and tunnel lining stability were maintained throughout the construction period, thanks to the proactive groundwater management measures. While a formal framework for an observational method was not pre-established, the case history discussed in this paper provides an example of how an effective response plan for groundwater

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management could be implemented during the delivery of a complex project to ensure timely and safe delivery. Note that the relief wells adjacent to the affected tunnel were a contingency measure which was not mandatory initially, but had to be activated based on the actual site observations.

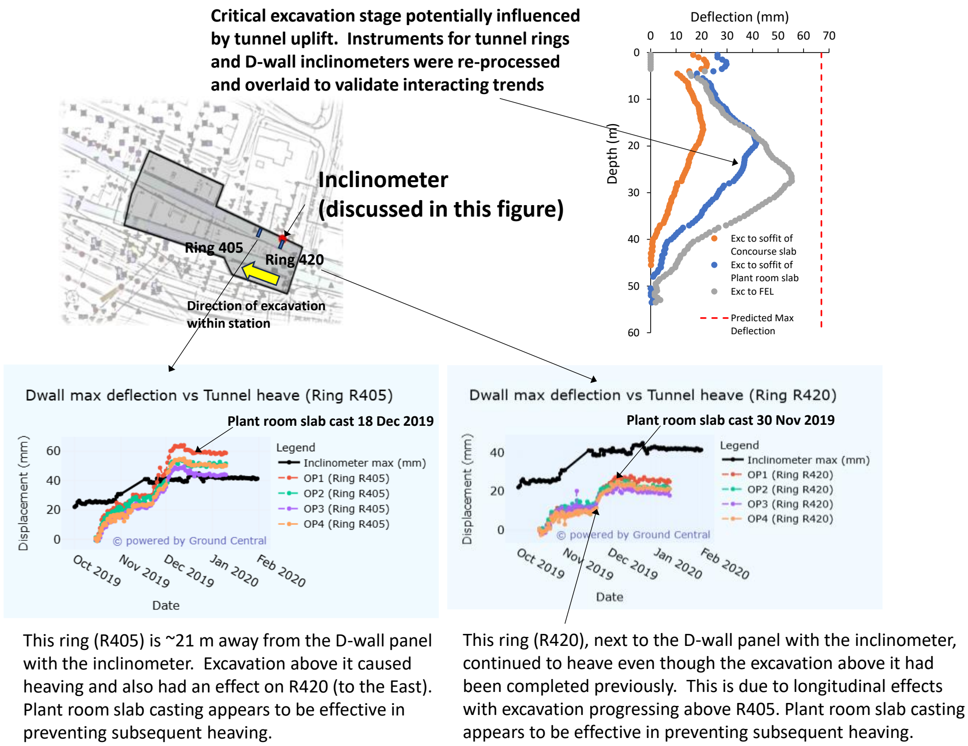

Figure 19. *Monitored diapghram wall deflections and overlaying against tunnel upward heave at two tunnel rings to validate (i) potential mechanisms in Figure 10 and (ii) the effectiveness of progressive slab casting above the tunnels. Longitudinal effects are evident in the tunnel rings (see Figure 18) and can be influenced by excavation activities before and after the ring position.*

CONCLUSION

This case history illustrates the critical role of effective groundwater management strategies in ensuring tunnel safety during the construction of an underground metro station. The project involved the deep excavation in a complex geological setting, with the challenges posed by weathered quartz intrusions and highly permeable materials. These ground conditions presented substantial risks, particularly uplift of the tunnel due to high seepage forces and potential instability of the retaining walls. The study demonstrates that a careful groundwater control strategy, coupled with real-time monitoring, is vital for mitigating these risks. The problem discussed in this case history is relevant specifically to a station box construction which has to accommodate TBM bore-throughs prior to deep excavation works within the station footprint. This construction sequence may occur in large underground metro projects, for stations which are located in between the launching and retrieval stations, depending on the site access dates and program of various interfacing activities. The soil cover reduces as excavation progresses, and can be critical when the counterweight and self-weight may be insufficient to overcome the upward water seepage forces.

The groundwater management system combined active relief wells and recharge controls. Relief wells were installed to lower the piezometric level adjacent to the tunnel, thereby reducing risks of tunnel uplift. Concurrently, recharge wells were placed

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adjacent to sensitive buildings to minimize groundwater drawdown and ground settlements. Greater drawdown was observed in the northern section, where the ground was more permeable, while the southern section, underlain by less permeable materials, exhibited minimal changes in groundwater levels. The system achieved the intended relief targets, confirming its effectiveness in controlling tunnel flotation and wall stability.

Instrumentation monitoring, including groundwater levels, ground settlements, and tunnel lining performance, confirmed that the excavation proceeded with minimal impact on the surrounding structures. The observed tunnel heave and diaphragm wall deflections were maintained within acceptable limits, largely due to the timely casting of the Plant Room Slab and strategic excavation sequencing. Overall performance remained satisfactory throughout, ensuring both tunnel and structural stability.

In conclusion, this case history underscores the importance of intergrating comprehensive groundwater control measures, continuous monitoring, and adaptive construction strategies to mitigate the challenges posed by complex geological conditions. The lessons learned from this project provide valuable insights for future tunnelling and deep excavation projects in similar geological settings.

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