Tunnels and Underground Cities: Engineering and Innovation meet Archaeology, Architecture and Art – Peila, Viggiani & Celestino (Eds)

Special design considerations for underpinning systems of existing structures due to tunnelling

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ABSTRACT: Protective work such as underpinning and ground improvement are routinely used to reduce tunnelling induced impacts, ensuring safety and serviceability of vulnerable existing building structures. This paper presents two case studies where the construction of the new Klang Valley Mass Rapid Transit- Line 2 in Kuala Lumpur, Malaysia, crosses very closely below the existing structures. The first case involves the underpinning of a 5-storey building located above the proposed tunnel alignment with some of its piles located within the tunnel horizon. Pile removal was designed to be carried out while the building was occupied. In the second case study, the same tunnel crosses beneath an existing reinforced concrete retention pond of a pumping station where partial pile removal was required to allow unobstructed access for the TBM. Despite the improved ground support from jet grouting, additional analyses were required as the underpinning work changes the response of the reinforced concrete structure.

1 INTRODUCTION

Construction of the Sungai Buloh-Serdang-Putrajaya (SSP) Klang Valley Mass Rapid Transit Line 2 (herein referred to as KVMRT2) in Kuala Lumpur, Malaysia began in the second half of 2016 and is in progress at the time of writing this paper. Scheduled to be fully operational by year 2022, it will serve a total of 37 stations over an alignment of 52.2km. Within this alignment, 13.5km of the alignment will be underground, connecting 11 underground stations.

Following the success of the world's first variable density tunnel boring machine (TBM) in KVMRT Line 1 (Bäppler et al., 2001), the new Line 2 utilises the same TBMs for its twin tunnel drives along the underground alignment. The 6.684m external diameter tunnel is made up of seven 275mm thick G50 precast segmental concrete linings.

While the TBMs are capable of addressing a variety of soils and rock of various conditions, tunneling through reinforced concrete piles is not advisable as the ductile rebars are likely to bend instead of being crushed and cored through, potentially jamming the TBM cutterhead.

One of the simplest ways to work around this issue is to design a tunnel alignment that does not clash with foundations of high-rise buildings or structures with pile foundations. However, in an urban environment, due to restrictions of land use or land acquisitions on top of any other technical or financial requirements, there will be situations where it is unavoidable. In these cases where buildings will remain in service throughout, underpinning and pile removal of existing structures are necessary for TBM passage.

This paper presents two case studies for the underpinning and pile removal of existing structures. It aims to highlight some of the less obvious but equally critical considerations for each specific site requirements.

2 CASE STUDY 1- UNDERPINNING DESIGN OF 5-STOREY EDUCATION QUARTER

2.1 *Site background*

The education quarter is a 5-storey reinforced concrete building with a ground floor car park and 4-storey residential quarters for a government primary school, Sekolah Kebangsaan Jalan Raja Muda. The KVMRT2 line consists of twin tunnels (Northbound and Southbound tunnels), that are parallel to each other at this location. The Southbound tunnel cuts under part of the existing education quarter building, indicating potential obstructions from the existing foundations to the tunnel construction works as shown in Figure 1.

Due to unavailability of foundation information of the building, investigation work, including trial pits, boreholes and parallel seismic tests were carried out. The trial pits revealed that the building is supported by 300mm square reinforced concrete (RC) piles. Based on parallel seismic tests conducted on two piles at different columns, the estimated pile lengths were found to be 24.5m and 32m respectively. The depth of tunnel boring machine (TBM) cutter-head is approximately 9m to 16m below ground level (Crown and invert of TBM extrados). As such, all the piles within the TBM extrados need to be removed. Since the building itself it currently occupied and will remain as such throughout tunnel construction, foundation underpinning work is required to protect the building, and ensure safety and long-term serviceability of the building.

The pile removal zone considers a TBM cutter head diameter of 6.684m, including uncertainties of 100mm TBM driving tolerance and pile verticality of 1 in 75 from true vertical position in the direction of the tunnel. All piles within the aforementioned zone will be removed and no underpinning pile can be carried out in this region. Figure 2 shows the photo of the affected columns. A total number of 4 piles (one for each column) have been identified for removal.

2.2 *Geological conditions*

The building is underlain by Kuala Lumpur Limestone formation with overburden soil typically consisting of mainly silty/gravelly SAND or sandy SILT of low SPT-N values (generally less than SPT-N 20). Figure 3 shows a typical geologic section. The subsurface investigation results show that the bedrock is generally encountered at 34m to 45m below ground level which is approximately 18m to 29m below invert level of KVMRT2 tunnel. Based on the nearest borehole information, the groundwater level is about 4.0m below ground level.

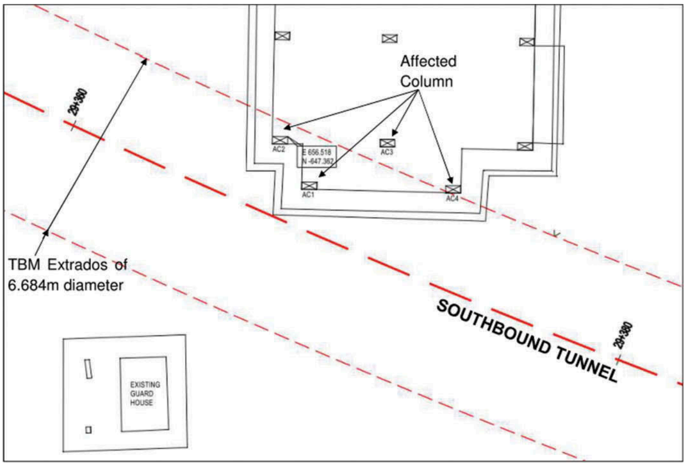

Figure 1. Plan view of affected columns in relation to the southbound tunnel.

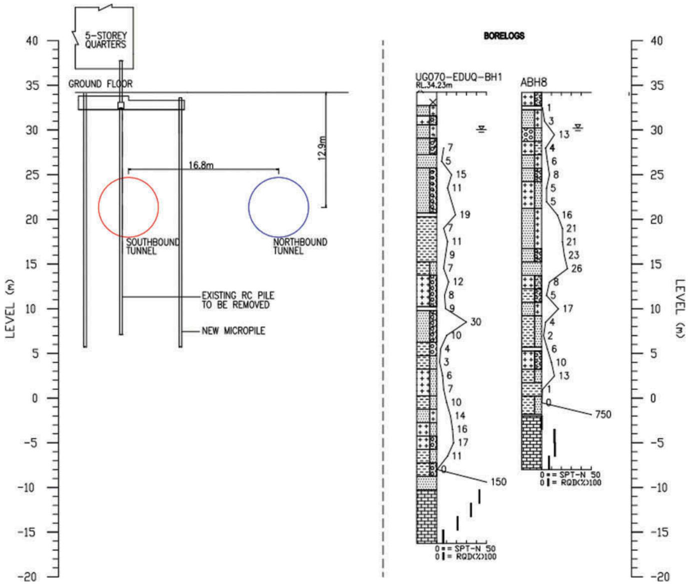

Figure 2. Typical cross section with borelog showing subsoil SPT-N profile.

Figure 3. Photo of affected columns.

2.3 *Loading assumptions*

Due to the absence of loading information for the existing building, the column loadings were estimated based on simple tributary area method. The loads derived by the authors were based on an assumed surcharge load of 15kPa per storey and 10kPa for roof loading. These estimated loads were independently assessed by two other consultants and as a means of assessing the final adopted values, the maximum estimated loading obtained from three parties on each column was conservatively chosen to represent the column loading for analysis.

2.4 *Foundation underpinning scheme and design considerations*

The concept of the proposed foundation underpinning scheme consists of a transfer slab which distributes the column loads to new micropiles beyond the no-pile zone. This meant that the transfer slab had to be designed for a wide effective span of 9.3m, with a thickness of 1.0 to 1.5m. The micropiles are 300mm diameter with design rock socket length of 2.5m. Due to karstic characteristics of the limestone formation, the bedrock level of the site is uneven, causing the installed pile lengths to vary from 28.5m to 50.1m.

To minimize the impact of the loading of the piles to the KVMRT2 tunnel, the micropiles are de-bonded up to 2m above tunnel invert level using bituminous membrane sheet (applied with grease) attached to the permanent casing. In addition, the permanent casing also serves as protection to the micropiles from the pile removal (i.e. coring works)as well as tunneling works.

Overlapped grouted columns were adopted as a temporary earth retaining system to facilitate the excavation and construction of transfer slab. It deserves to be highlighted that pile removal wouldonly be carried out after the load transfer structure is put in place. The method of pile removal will be further discussed in the subsequent section.

It is important to consider the reaction of working piles and transfer slab at different stages of construction to ensure adequacy of the underpinning system. This includes but is not limited to the following checks:

i. Impact of coring of existing pile (pile removal method) on newly constructed micropiles and adjacent existing RC piles.
ii. Expected pile head movement and transfer slab deflections post load application; permanent condition.
iii. Building impact assessment due to tunneling (Impact of tunneling on newly installed micropiles)

The new micropiles are expected to settle after mobilization of building loads to the pile and again during tunneling due to volume loss. As such, pre-loading on the newly installed micropiles is necessary to control the movements and differential settlements of the building.

These micropiles will be preloaded to the combined self-weight of the transfer slab, column load and weight of backfill above the transfer slab. This is achieved through the use of reaction frames where they are anchored to the transfer slab, providing the required reaction to transfer loads to the micropiles via hydraulic jack (see Figure 4). Upon reaching the targeted preload, the gap between the micropile and transfer slab will be filled with non-shrink grout to complete the load transfer process.

2.5 *Methods of pile removal*

Two feasible options to remove existing piles were considered. Option 1 was to remove the pile manually by creating an access via hand-dug caisson shaft with horizontal mined adit, while Option 2 was to core the piles in inclined direction from ground surface. In order to avoid prolonging the planned tunneling schedule, the tunnel intervention method to remove the pile from TBM cutterhead during tunneling works was not taken into consideration.

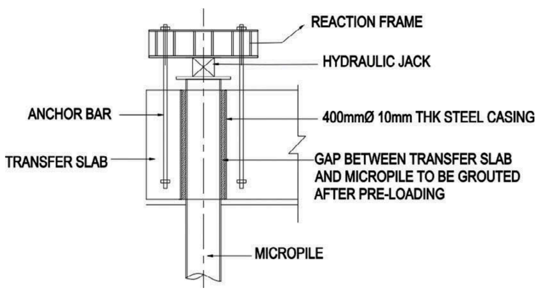

Figure 4. Pre-loading jack configuration.

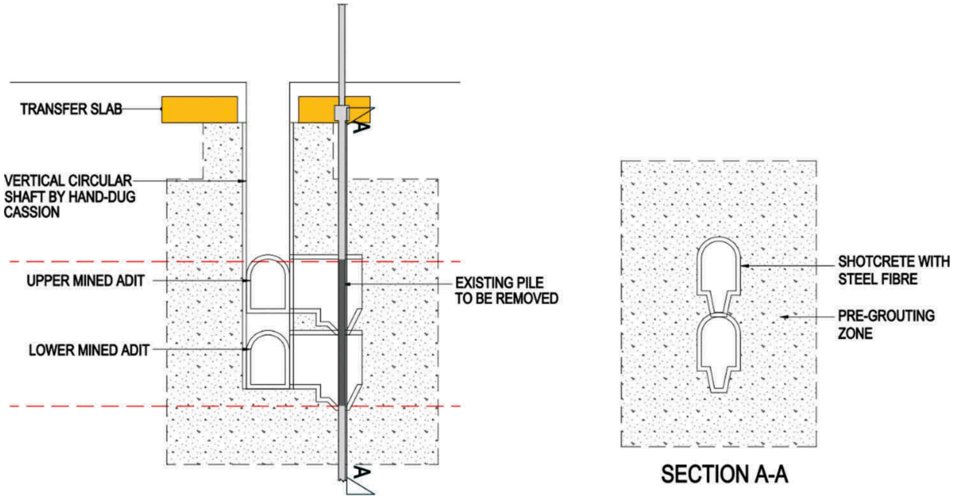

Figure 5. Caisson vertical shaft and horizontal mined adit for pile removal.

2.5.1 *Option 1: Manual cutting of pile via caisson vertical shaft and horizontal mined adit*

Given that the length of the pile removal is approximately the diameter of TBM cutter head, a single adit would be excessively large and not cost-effective. Therefore, two levels of horizontal horse-shoe shaped mined adit with localized deepened excavation; below the pile location, has been proposed as shown in Figure 5.

In view of high groundwater table above tunnel horizon, pre-construction ground treatment (i.e. jet grout block) is required to ensure stability and dry condition inside the shaft and adits during excavation. Once grouted block has gained strength, the vertical shaft will be excavated first, followed by mining of horizontal adit towards the pile before pile cutting and removal. The horizontal mining would start from the lowest adit which is subsequently backfilled upon completion of the interim pile removal before repeating the process for the upper adit.

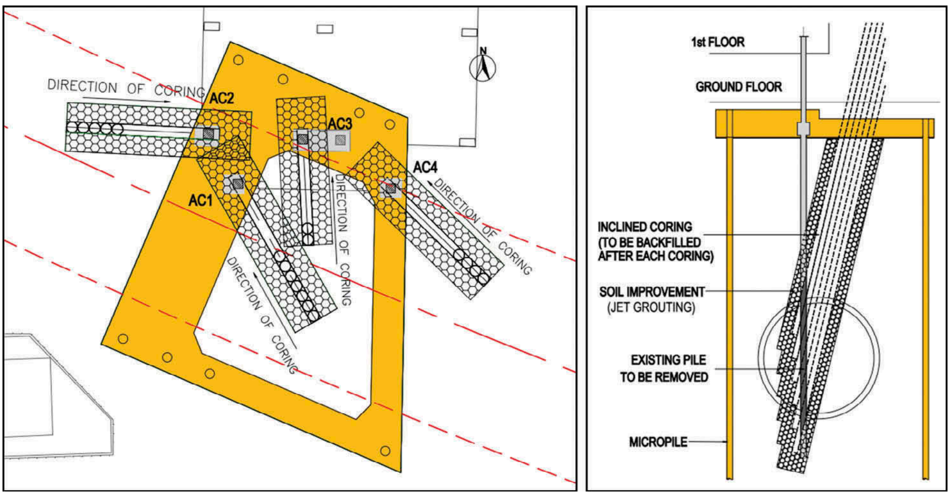

Figure 6. Pile removal from ground surface using coring rig (Left: Layout plan; Right: Typical section).

Based on KVMRT Line 1 experience, this option had been successfully implemented by local contractors as described in Khoo et al. (2015). The working space required for this option is small and therefore feasible for a site with headroom constraint. However, the drawback of Option 1 is the slow excavation progress. Considering the target was to remove all obstacle piles prior to tunnel arrival within tight a schedule, this option was not adopted.

2.5.2 *Option 2: Inclined coring of piles*

As the affected piles are located at the edge of the building, there is sufficient open space along-side the building to make use of a drilling rig for pile removal. The core diameter is 350mm (slightly larger than the existing RC square pile size of 300mm) and angle of coring ranges from 13° to 16° from vertical (see Figure 6).

In order to ensure stability of drilled hole during coring work, ground improvement by means of jet grouting block was carried out for an extent of 600mm surrounding the edge of the inclined drilled hole. Once the grouted block has gained strength, coring is to be carried out at the planned direction and distance. The void left by the coring will be backfilled with cement? grout.

The next coring can only begin once the minimum grout strength of 1MPa is achieved to ensure stability of adjacent drilled hole. Minor adjustments of coring direction on site may be required subject to coring results. The acceptance of the pile removal will be subject to verification of the extracted material.

It should be noted at that unlike Option 1, the pile removal from Option 2 may not be as complete or thorough, even within the tunnel horizon. However, this is considered manageable for the tunnel operation as long as the majority of the pile material (especially the pile reinforcement), can be removed; preventing jamming of TBM's system. This option has been adopted as it offers a much shorter pile removal duration as compared to Option 1.

3 CASE STUDY 2- DATO' KERAMAT PUMPING STATION

3.1 *Site background*

Constructed in 2001, the Dato' Keramat pumping station is a manmade reinforced concrete retention pond located at Kampung Dato' Keramat, next to Klang River. As part of a wider flood mitigation scheme initiated by the Department of Drainage and Irrigation Malaysia (JPS), the 1400 m^2 wide two-tiered retention pond was designed to retain an operation volume of approximately 3700 m^3 of water, diverted from the adjacent Klang river.

The retention pond itself is supported by 200 mm square RC piles in a grid pattern with 2 m centre-to-centre spacing. An elevated pump house control room is situated on the south-west corner of the retention pond. No as-built information was made available. Therefore, it was expected for the actual foundation installed on-site to differ slightly from the construction drawings. Based on the construction drawings, pile lengths of 18 m are expected.

Both north and southbound tunnels of KVMRT2 will cross the retention pond along its width in a stacked alignment as shown in Figures 7 and 8. The axis depth for the shallower southbound tunnel is approximately 16.5m below ground level(mbgl) where it has a gentle gradient which was higher on the north side of the retention pond while the deeper northbound tunnel is relatively level with an axis depth of approximately 28 mbgl. Since the piles are within the southbound tunnel's horizon, pile removal is required for affected areas (i.e. along the tunnel alignment with a width of 3m to each side of the tunnel extrados). The adopted pile removal method consists of loosening the soil surrounding the pile with a steel casing (internal diameter of 330mm) before lifting it out from its position.

3.2 *Subsoil condition*

The pump house is situated on limestone formation where the underlying ground is of alluvial soil. This consists of silty/gravelly SAND and CLAY. Prediction of rock head levels in

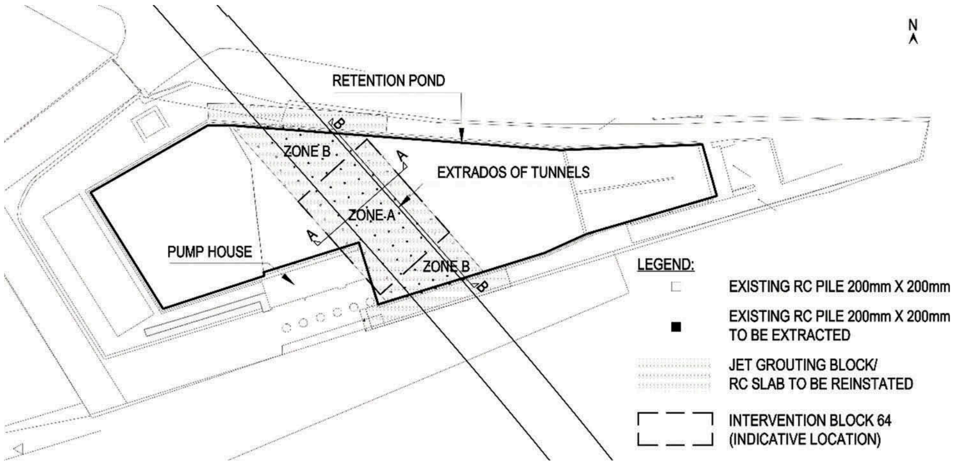

Figure 7. Layout of Dato' Keramat Pumping Station.

limestone formation is generally very difficult due to its karstic features, but the role it plays in this case study is less important given that the rock head level recorded by nearby boreholes are much deeper than the areas of concern. Groundwater level was relatively high with an adopted design level of 2 mbgl. The interpreted cross section A-A of the analysis is shown in Figure 8 below.

3.3 *Design considerations and proposed solution*

Ground improvement via jet grouting was proposed to support the slab post pile removal. In the initial stages, it was only intended for the jet grout block (JGB) to extend 3m from the southbound tunnel extrados. Subsequently, it was decided to utilize the scheme to serve as an intervention block for both tunnels; facilitating the maintenance of the TBM. Thus, the JGB at the central portion (Zone A) under the retention pond was extended further to encompass the deeper northbound tunnels (see Figure 8). Zone B as indicated in Figure 7 represents the area outside of the intervention block where JGB extends 3m below the invert of the southbound tunnel. Piles outside of these

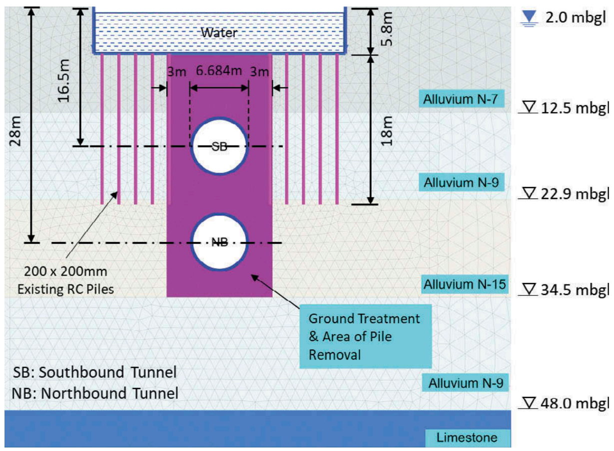

Figure 8. Section A-A of Dato' Keramat Pumping Station.

zones remain in place. Note that the construction drawing records that each pile has a working load of 300kN.

Tunnelling within the improved ground is beneficial in terms of the stress reductions on the tunnel linings at a given volume. This is because the JGB is designed to be self-supporting, allowing forces to arch over the circular tunnels. JGB is expected to achieve an unconfined strength of 1 MPa in 28 days with stiffness of 150MPa as well as a permeability within the region of 1 x 10^{-7} m/s. Along with the increased stiffness and strength of the JGB, magnitudes of soil movements toward the tunnels would similarly be less, reducing surface settlements.

While the above summarizes the pros of having ground improvement, the improved stiffness introduces an additional problem. The retention pond will resume operation upon reinstatement of the base slab, after pile removal. The base slab is therefore expected to take full water loading. Even though the settlements directly above JGB will be very small due to the its high stiffness, the sides which are seated on the existing loose/soft ground will deform much more under the same area load, creating differential settlement. This is exacerbated with the high stiffness of the piles immediately adjacent to JGB. Even though the existing slab within zones A and B will be demolished, the adequacy of its initially design steel reinforcements would need to be reassessed to cater for this hogging moment. A two-dimensional finite element analysis (FEA) was carried out using a commercial geotechnical FEA software, Plaxis 2D in order to evaluate the potential slab and pile settlement that may occur. A similar analysis was carried out to ascertain the slab settlement at Zone B. Structural forces for the base slab were not adopted directly from the FEA given the unique shape of the retention point and the skewed angle that the tunnel passes through it.

Soil spring stiffness was then computed by dividing the allowable bearing capacity of JGB with the settlements obtained from Plaxis. Similarly, the spring stiffness of the pile is taken as the working load divided by pile settlement. This was then modelled in a dedicated structural FEA software, SAFE 12. This allowed for a more realistic approach whereby local stiffening effects (e.g. corner of slab and wall of retention pond) can be accounted for.

A full model of the retention pond was not necessary as the area of concern lies is relatively small. The authors note that the response of the slab towards the extreme sides of the retention pond were less representative but the model was sufficiently wide to capture the slab response over Zones A and B.

In line with expectations, the edges of JGB experienced large hogging moments (refer to Figure 9). Across zone A, the results showed negligible sagging moment while in zones B, due to a lower stiffness from the shorter length of JGB, the result indicated a sagging moment at midspan and hogging moments towards the sides; similar to zone A.

The maximum hogging moment of 117.7 kNm/m was recorded in near the south border of zones A and B. This has exceeded the allowable moment capacity of the existing slab based on the existing reinforcement design at 95kNm/m. At these locations, additional reinforcement was required during reinstatement.

The adopted bearing capacity of JGB was taken as 300 kPa. This was based on previous local experience on similar ground conditions. Nonetheless, it is important to highlight that the actual bearing capacity could vary on site, owing soil variability as well as uncertainties in workmanship, this is in-line with the load cases considered for piled raft foundation (Tan et al., 2004).

To cater for this, a sensitivity study was carried out by varying the soil stiffness. This was essentially achieved by increasing or decreasing bearing capacity by a prescribed amount varying from 100 kPa to 400 kPa. The respective maximum bending moment is summarized in the table below.

Due to the high spring stiffness of the piles at the sides of JGB, a high bearing capacity or stiffness of JGB would result in a smaller contrast in stiffness. This would eventually yield smaller differential displacements and thus hogging moments. It can be observed from Table 1 that the results follow this trend.

Fluctuations for maximum sagging moments on the other hand were relatively small at less than 7% across the range of bearing capacities. This is due to the fact that the maximum

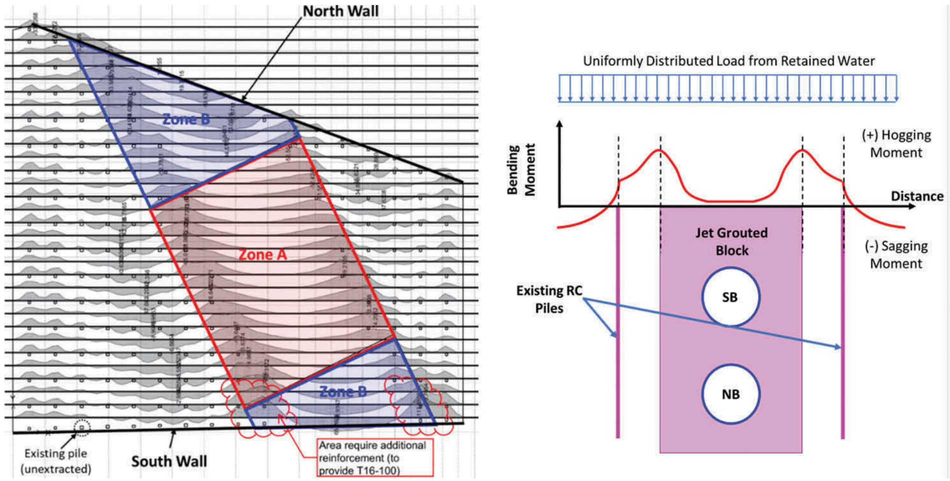

Figure 9. Results of bending moment analyses for the RC slab (left) with sketch of bending moment diagram illustrating the effects of the stiffen ground response with jet grouted block (right).

Table 1. Summary of maximum bending moments from sensitivity analyses.

JGB Bearing Capacity (kPa)	Max. Hogging Moment (kNm/m)	Max. Sagging Moment (kNm/m)
100	147.9	-146.9
150	107.9	-141.6
200	98.4	-141.7
300	86.5	-146.4
400	82.4	-151.9

sagging moment generally occurred at midspan of JGB which was less affected by the varying relative stiffness of the piles located at a distance.

Given the affected locations were relatively small, the final slab reinforcement design was based on the worst-case scenario with a bearing capacity of 100kPa. The authors believe that it was justified to take a conservative approach to the design to account for the uncertainties at a relatively small construction cost.

4 CONCLUSIONS

This paper has presented two case studies where the existing structures were required to be underpinned for pile removal to allow passage for the TBM. In the first case study, various considerations were given to the underpinning and pile removal techniques as the 5-storey residential building remained occupied throughout the construction. The second case study discussed the double-edged effect of ground improvement as an underpinning solution where the improved ground stiffness imposes additional hogging moments on the RC slabs of the pumping station.

It is hoped that this paper will help in highlighting the importance of careful deliberation, particularly for less obvious secondary effects during the designing process of protective works due to tunnelling. It is important to note that the aforementioned protective works are still ongoing at the time of writing this paper, therefore, the possibility of minor field revisions remain.

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