Large Diameter TBM Tunnelling Beneath New Reclamation

Siew-wei Lee, *Director, Golder Associates Hong Kong Limited; email: geoswlee@gmail.com*
Chi-on Leung, *Senior Engineer, Golder Associates Hong Kong Limited; email: chionalex@gmail.com*
Calvin Cheung, *Technical Manager, Dragages Hong Kong Limited; email: calvin.cheung@dragageshk.com*
Tharanga Wijesooriya, *Senior Engineer, Dragages Hong Kong Limited; email: tharanga.wijesooriya@dragageshk.com*
Antoine Schwob, *Engineering Manager, Dragages Hong Kong Limited; email: antoine.schwob@dragageshk.com*

ABSTRACT: *The Tuen Mun-Chek Lap Kok Link (TMCLKL) project successfully completed a 14 m diameter slurry mix-shield tunnel boring machine (TBM) tunnelling in Alluvium, beneath a recently reclaimed land. The in situ clayey Marine Deposits and Alluvium were subject to ongoing consolidation and creep settlements as a result of reclamation loading. This posed a challenge to the design of tunnels which needed to achieve a limiting tunnel squat of 1% of tunnel diameter over an operational life of 120 years. An innovative design was conceived by installing barrettes at the tunnel springlines to reduce the tunnel squat in the long term by sustaining the soil arching load around the tunnel. This paper presents the field performance of tunnelling with respect to TBM operational parameters. Although the barrettes were intended to reduce the tunnel squat, the field measurement showed that they also reduced volume loss ratios caused by tunnelling. The measured volume loss ratios were between 1.27% and 2.07% in the seawall area without barrettes, and smaller than 0.54% in the area with barrettes. In the seawall area, post-tunnelling consolidation settlements completed in 30 to 60 days with a magnitude of 25 mm to 30 mm for each tunnel. In the area where the tunnels were supported by the barrettes, the consolidation settlements were smaller than 5 mm for each tunnel. The tunnel squat was reduced by 40% to 50% at the tunnel section with barrettes compared to that without barrettes. Discussion is also made by comparing the measured volume loss ratios to those reported in the recent TBM tunnelling projects in Hong Kong which ranged from 0.1% to 1.2%. The TMCLKL tunnels were successfully constructed without causing excessive deformation to the overlying reclaimed platform and seawalls.*

KEYWORDS: barrette, confinement pressure, consolidation, reclamation, settlement, TBM, tunnel, volume loss

SITE LOCATION: **Geo-Database**

INTRODUCTION

The Tuen Mun–Chek Lap Kok Link (TMCLKL) consists of a 9 km dual two-lane carriageway between Tuen Mun and Lantau Island, Hong Kong (see Figure 1). The TMCLKL project has successfully completed one of the world's largest TBM tunnels. It provides a strategic link connecting Hong Kong Northwest area, Hong Kong International Airport, Hong Kong-Zhuhai-Macao Bridge (the world's longest sea crossing bridge, 55 km long), and Lantau (the largest island in Hong Kong). The alignment commences in Tuen Mun and heads southeast to a new reclamation named Northern Landfall (NL). It then heads south into a 4.2 km sub-sea tunnel passing beneath a busy navigation channel between Tuen Mun and Lantau Island. The deepest point of the subsea tunnel is 50 m below sea level. The alignment emerges into the Boundary Crossing Facilities reclamation named as Southern Landfall (SL). Finally, it turns eastward on a marine viaduct to connect with Lantau Island. Schwob et al. (2019), Schwob et al. (2020), and Chan et al. (2021a) presented the construction aspects, challenges, and innovations of the TMCLKL tunnelling work.

At Northern Landfall, the 650 m approach tunnels were excavated using a 17.63 m diameter and a 14 m diameter slurry mix-shield TBM for the northbound (N/B) and southbound (S/B) tunnels, respectively (Kwong et al., 2018). The 4.2 km sub-sea tunnels and the 450 m approach tunnels at SL were excavated using two 14 m diameter slurry mix-shield TBMs. The TBM tunnels at SL broke out into the adjacent ramp tunnels constructed using the cut-and-cover method. There were 57 cross

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passages constructed between the two TBM tunnels (Cagnat et al., 2018). Chan & Kwong (2019) and Kwong et al. (2019) discussed the geotechnical risks of the TMCLKL TBM tunnelling and presented some instrumentation monitoring data at the NL, sub-sea, and SL sections.

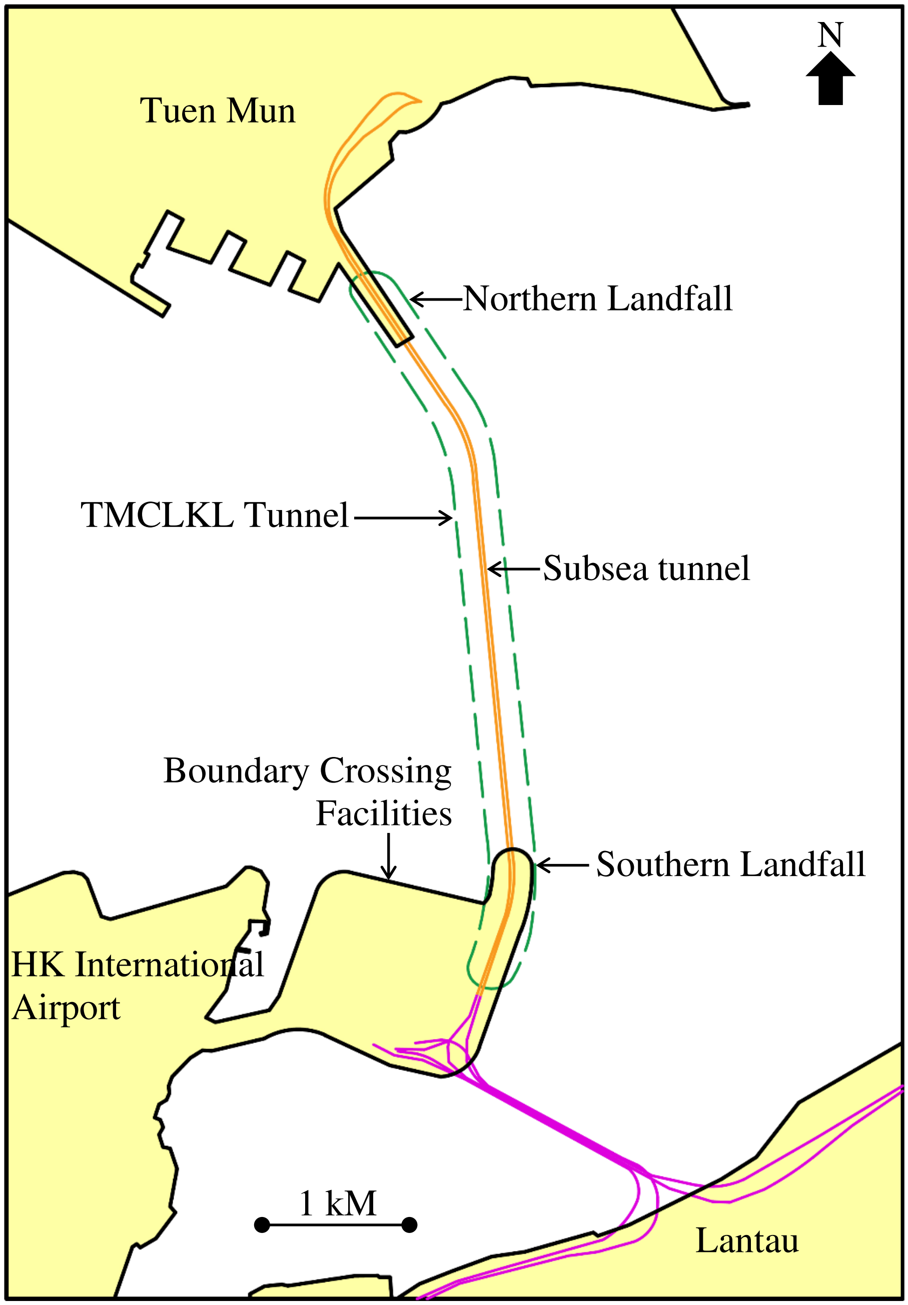


Figure 1. Alignment of Tuen Mun-Chek Lap Kok Link Tunnel.

Schwob et al. (2020) presents the innovative solutions adopted by the contractor, such as the application of a robot for disc cutter maintenance and the pipe-jacking method for the construction of cross passages. A custom-made, articulated robot accessed the excavation chamber under hyperbaric pressure behind the TBM cutterhead, removed worn or damaged disc cutters, and replaced them with new ones. This reduced the risk of workers being exposed to hyperbaric conditions when carrying out the maintenance of disc cutters. A small diameter 3.665 m slurry TBM was used to construct 57 nos. of cross passages between the two main tunnels. The cross passages are approximately 13 m in length each, and they provide a means of egress to pedestrians in case of emergency. The slurry TBM tunnelling method was better in controlling geological risks under high water pressure, in comparison to the original method of ground freezing using brine and hand excavation. These innovation techniques had contributed to the successful completion of the tunnelling works, while improving the construction safety and geotechnical risk management.

This paper presents the design and performance of TBM tunnelling at SL between 2017 and 2019, during which other associated works were also carried out.

GEOLOGICAL SETTINGS

The geotechnical formations which have an influence on the design of tunnelling and segmental tunnel linings at SL are the reclamation Fill, Marine Deposits (MD), Alluvium, completely decomposed granite (CDG), and completely decomposed metasiltsotne (CDMS) formed by the weathering of in situ rocks. Stratigraphy along the tunnelling alignment is presented in Figure 2. The geological basement, which underlies the tunnels, comprises predominantly igneous rocks and sedimentary and metasedimentary rocks.

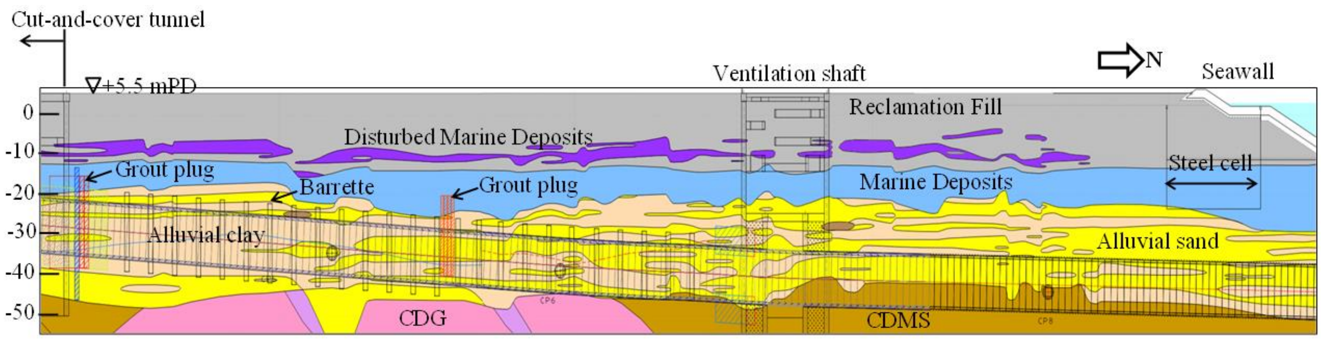


Figure 2. Geological profile at Southern Landfall.

The tunnels were constructed in Alluvium underlying an area of land recently reclaimed between 2013 and 2016, which was surcharged and installed with prefabricated vertical drains (PVD) to accelerate consolidation. The original seabed level of the reclamation varied between -3 and -11 meters above Principal Datum (mPD). The finished reclamation ground level is +5.5 mPD. The tunnel horizon is 25 m to 55 m below ground level (mbgl). The groundwater level is tidal and typically at 3 mbgl. The tunnel length at SL is 450 m.

The reclamation fill is 12 m to 22 m thick and comprises both dredged sand and processed public fill. In Hong Kong, the public fill refers to inert construction and demolition materials, consisting of rock, concrete, asphalt, rubble, bricks, stones, and earth. Since these materials will not decay or decompose, they are suitable for reuse in reclamation or site formation works. The public fill is predominantly loose to dense, silty sand with occasional gravels and cobbles. Disturbed MD have been identified within the body of the reclamation fill, generally between -6 and -15 mPD, which comprise very soft to firm, sandy, silty clays.

The MD are 3 m to 15 m thick and comprise very soft to soft and locally firm silty clays with occasional shell fragments. The MD are relatively homogenous, although they exhibit local variations in particle size distribution. The pre-reclamation over consolidation ratio (OCR) of MD typically varied from 2.0 at the top of MD to 1.0 at a depth of 15 m. Figure 3 shows the properties of the MD in terms of Casagrande plasticity chart, compression ratio, recompression ratio, coefficient of secondary compression, and coefficient of consolidation. Due to the presence of organic content in the MD, the liquid limits of oven-dried samples were much lower than those of air-dried samples. The moisture contents of the MD ranged from 60% to 110%. The targeted OCR of the MD after the surcharge removal was equal to or greater than 1.2. The ratio of undrained shear strength (c_u) to in situ effective vertical stress (σ'_v) was generally 0.22 based on the pre-reclamation ground investigation (GI) information. At some locations, the MD were still consolidating under the loadings of the reclamation fill and surcharge. This aspect needed to be considered in the design of TBM tunnelling, as the tunnel excavation beneath the MD layer would cause shearing and excess pore water pressures in the MD, leading to further consolidation settlement.

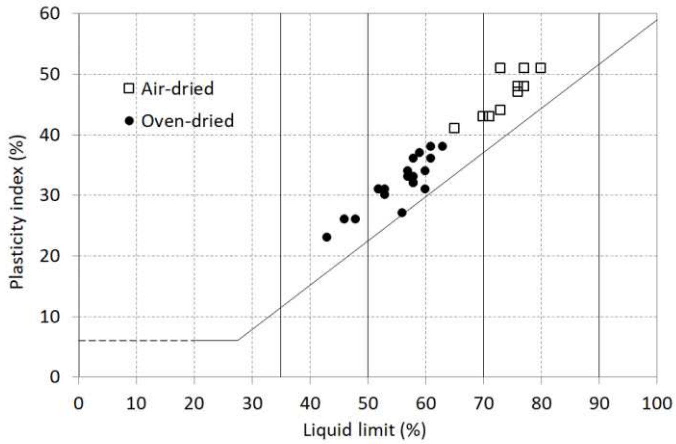


Figure 3(a). MD – Casagrande plasticity chart.

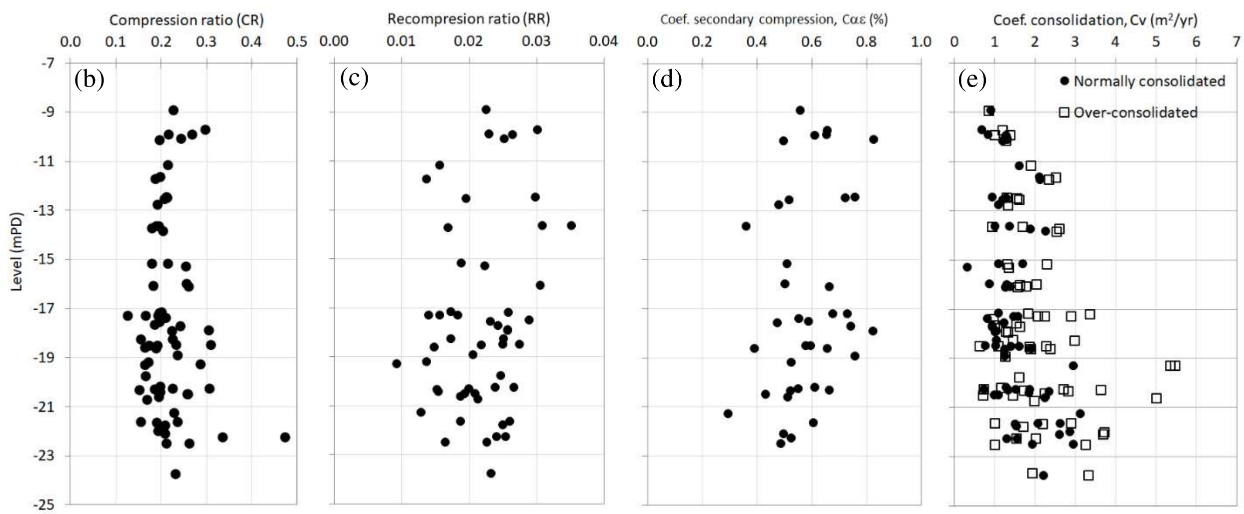


Figure 3(b). MD – Compression ratios measured in oedometer tests where CR = $Cc/(1+e_0)$, Cc = compression index, and e_0 = initial void ratio; Figure 3(c). Recompression ratios where RR = $Cr/(1+e_0)$ and Cr = recompression index; Figure 3(d). Coefficients of secondary compression (w.r.t. strain) where $C\alpha\varepsilon = C\alpha/(1+e_0)$, $C\alpha$ = secondary compression index = $\delta e/\delta \log(t)$, and t = time; Figure 3(e). Coefficients of consolidation interpreted by the log-time method.

The Alluvium is 15 m to 40 m thick and comprises soft to stiff silty and sandy clay, firm to stiff sandy silt, and loose to very dense silty sand. Discrete lenses or beds of fine to coarse gravel have been encountered locally. The TBM tunnelling was carried out in the Alluvium layer. Figure 4 shows the properties of the alluvial clay in terms of the Casagrande plasticity chart, compression ratio, recompression ratio, coefficient of secondary compression, and coefficient of consolidation. The fine-grained Alluvium was classified as intermediate plasticity, being silty clay or clayey silt. The moisture contents ranged from 15% to 50%. The OCR values after the surcharge removal were expected to be between 1.3 and 2.4. The ratio of c_u/σ'_v ranged from 0.37 to 0.47 based on the pre-reclamation GI information.

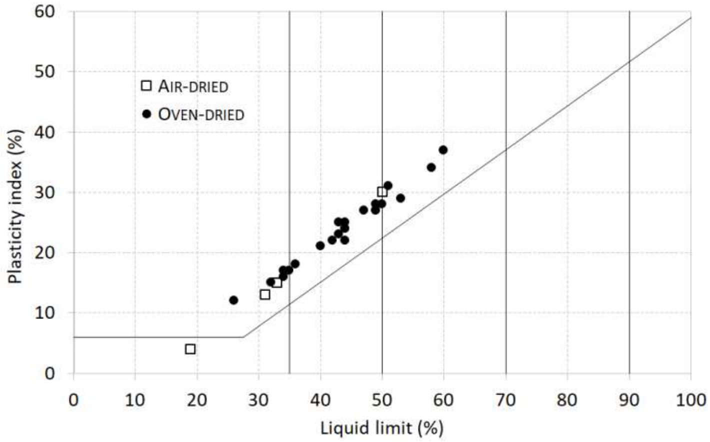


Figure 4(a). Alluvial clay – Casagrande plasticity chart.

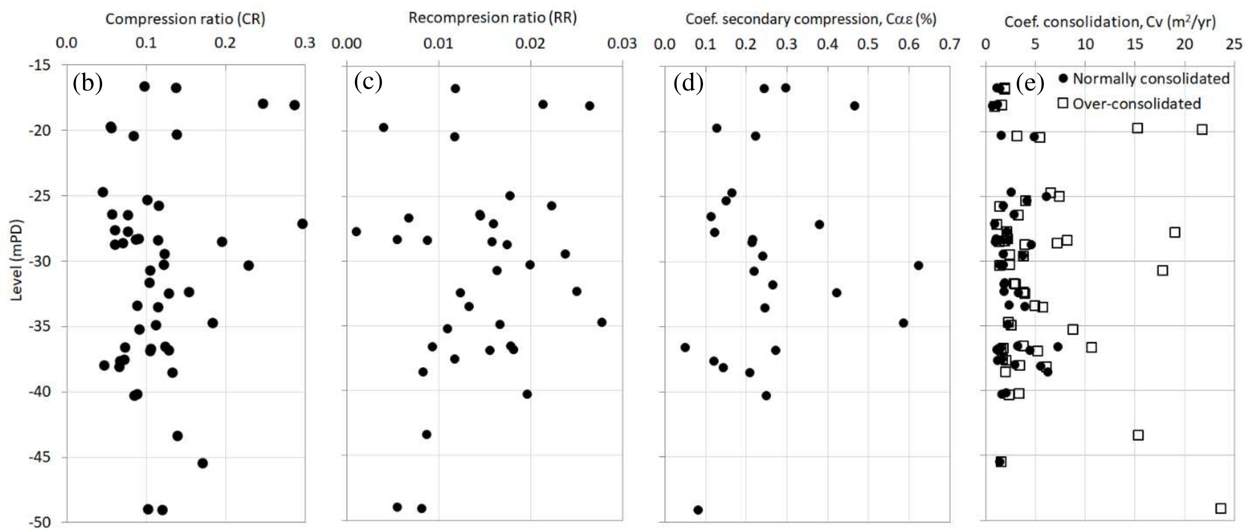


Figure 4(b). Alluvial clay – Compression ratios; Figure 4(c). Recompression ratios; Figure 4(d). Coefficients of secondary compression (w.r.t. strain); Figure 4(e). Coefficients of consolidation interpreted by the log-time method.

Saprolites are completely decomposed (Grade V, e.g. CDG and CDMS) to highly decomposed (Grade IV) granite/metasedimentary rock. They are 8 m to 40 m thick and comprise fine to medium grained granite/metasiltstone which is recovered as stiff to very stiff, sandy, clayey silt and medium dense to very dense silty sand.

Figure 5 shows the typical Standard Penetration Test (SPT) blow count N values recorded in the fill, alluvial sand, CDG, and CDMS. The alluvial sand was loose to very dense, slightly silty to very silty, fine to coarse sand with sub-angular to sub-rounded gravel. The CDG was extremely weak to weak, completely decomposed, fine to medium grained granite, which was recovered as clayey silt and silty sand. The CDMS was extremely weak to weak, completely decomposed metasiltstone, which was recovered as sandy silt or sandy clay.

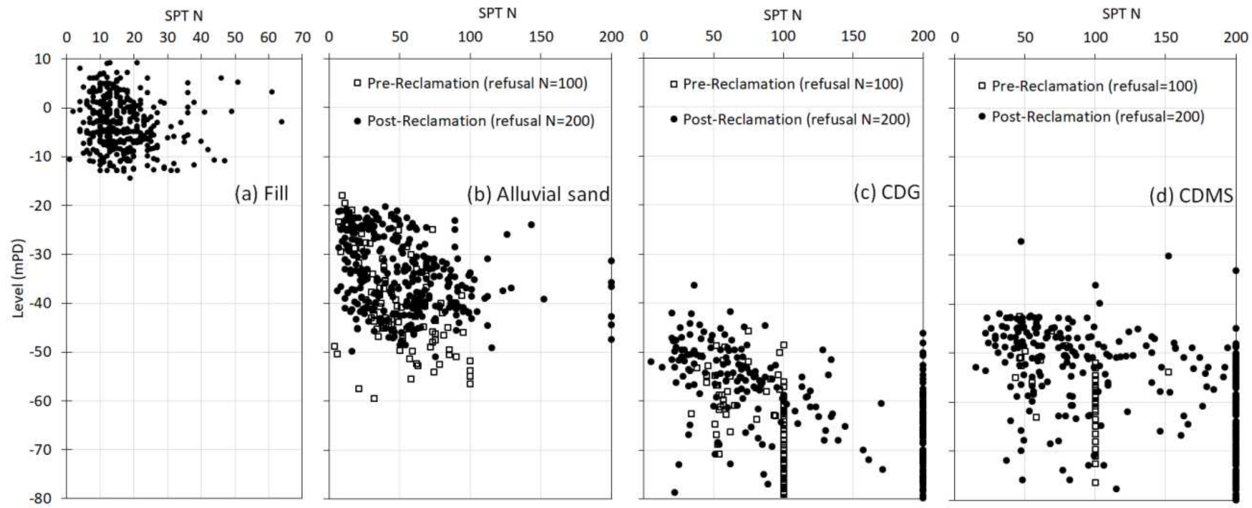


Figure 5. SPT blow counts N recorded in (a) fill; (b) Alluvial sand; (c) CDG; and (d) CDMS.

Figure 6 shows the results of effective cohesion (c') and friction angle (ϕ') from triaxial consolidation undrained tests on the MD, alluvial clay, CDG, and CDMS samples. The MD and alluvial clay had a ϕ' of 28°, whilst the CDG and CDMS had a ϕ' of 32° with c' ranging from 3 kPa to 5 kPa.

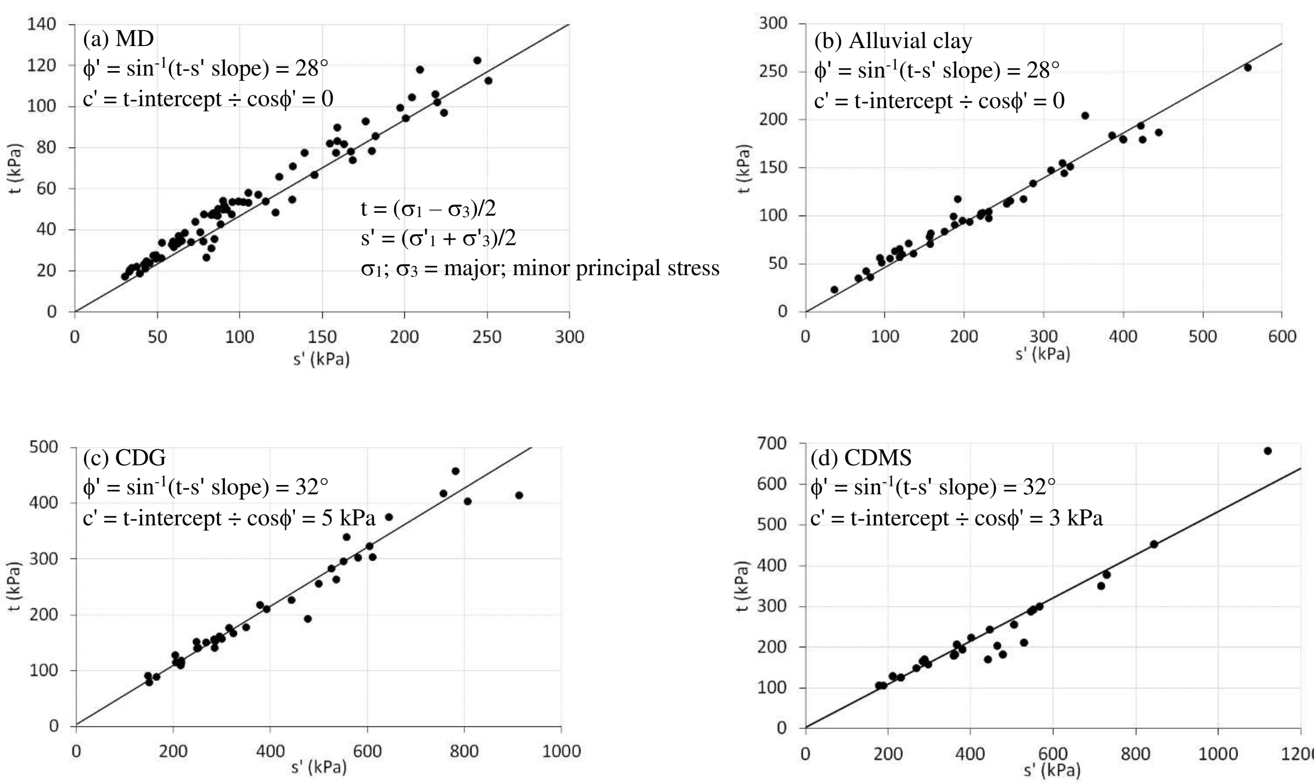


Figure 6. Parameters c' and ϕ' measured in triaxial consolidated undrained compression tests (with measurement of pore water pressures) on (a) MD; (b) Alluvial clay; (c) CDG; and (d) CDMS.

The tunnels were to be constructed in the Alluvium layer. The design parameters of alluvial clay that predominantly governed the behavior of tunnels were the clay thickness, OCR, compression ratio, recompression ratio, coefficient of consolidation, and undrained shear strength. Discrete gravel units were indentified in the Alluvium. This might create local instances of highly pemeable ground, potentially causing a tunnel face stability problem due to leakage of slurry providing confinement pressure. The tunneller was made aware of such locations beforehand.

RECLAMATION

Figure 7 shows the SL reclamation in July 2018. The Contract required that the post-handover settlement should not exceed 500 mm for an operational life of 120 years. The SL reclamation included a seawall perimeter which comprised 31 m diameter steel cells extending through the MD and approximately 5 m into the underlying Alluvium. The steel cells were formed by interlocking sheet piles. A sloping rockfill seawall was constructed in front of the seaward side of the steel cells. Stone columns were installed within these steel cells and beneath the sloping rockfill seawall. The steel cells (see Figures 2 and 8), together with the stone columns, created an earth-retaining gravity structure that could temporarily support the reclamation loading whilst the soft clay within the seawall steel cells and beneath the reclaimed land was gaining strength.

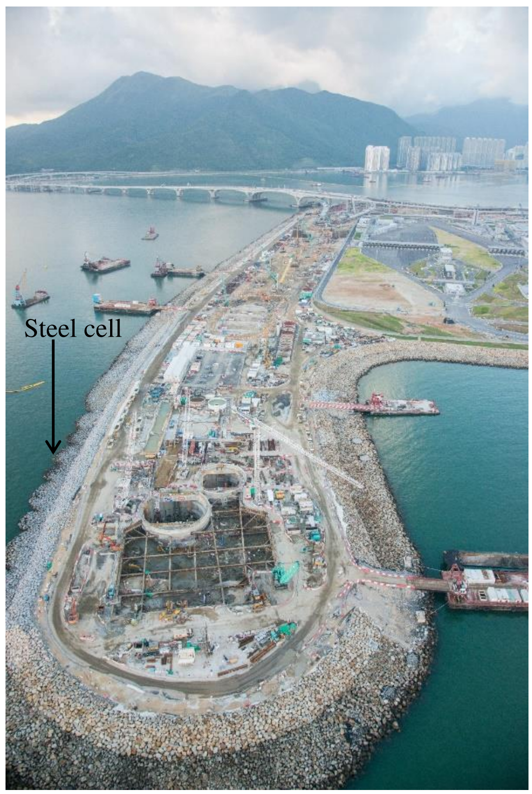


Figure 7. Southern Landfall reclamation in July 2018.

In the main reclamation area, PVDs at 1 m to 1.2 m triangular spacing were installed to accelerate the dissipation of excess porewater pressures and settlements within the MD and alluvial clay. Surcharging of the reclamation was undertaken by raising the reclamation fill level up to a level of +8.5 mPD or +11.5 mPD, depending on the locations. At certain locations, ground improvement in the form of deep cement mixing (DCM) columns were installed behind the seawalls extending from the fill through the MD to mitigate the risk of seawall lateral movement.

At the time the TBM arrived at the SL reclamation, the ground settlement was ongoing due to consolidation associated with the reclamation loading (see Figure 8). The settlement rate ranged between 1.5 mm/day and 2.2 mm/day near the seawall just before the arrival of the TBM in early 2017. These rates were higher than the original design estimation of substantial completion of the consolidation settlement at the arrival of TBM. For the section between the ventilation shafts and cut-and-cover tunnels, the settlement rate was less than 0.1 mm/day before the TBM arrival in April 2018, because much of the consolidation had taken place throughout 2017.

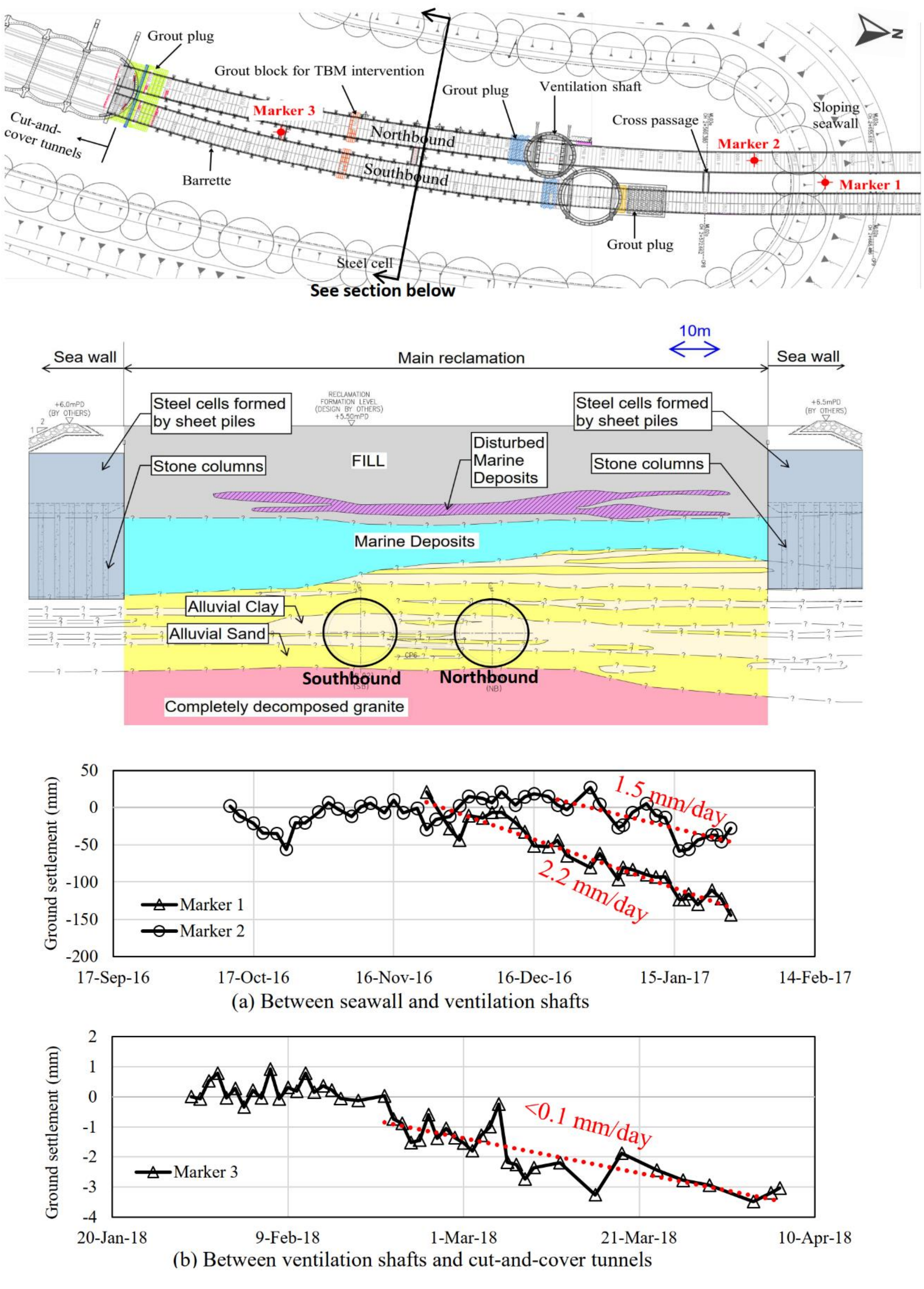


Figure 8. Measured ground surface settlements before TBM arrival.

The ongoing settlement issue imposed a challenge to the tunnels in satisfying the serviceability requirement in the long-term, i.e., a limiting tunnel squat of 1% of tunnel diameter (130 mm) over an operational life of 120 years. Therefore, an innovative barrette system was designed and constructed to address this issue.

SEGMENTAL TUNNEL LININGS AND BARRETTE SYSTEM

Two slurry mix-shield TBMs, viz. S881 and S882, were deployed to construct the tunnels at SL (see Figure 9). Mix-shield TBMs have been designed for heterogeneous ground conditions encountering clay/sand/gravel and high water pressure up to 15 bars (https://www.herrenknecht.com/en/products/productdetail/mixshield/). Precise control of slurry support pressure at the tunnel face was managed by using an automatically controlled air cushion. The TBM allowed for the erection of segmental tunnel linings immediately behind the TBM shield as the permanent ground support. Chan et al. (2021b) presented other special TBM features implemented at the TMCLKL project, such as automated monitoring and replacement of 19-inch disc cutters at cutterhead, hyperbaric intervention by compressed air (up to 4.2 bars) and saturation diving (up to 6 bars), stone crushers, etc.

Figure 9. 14 m diameter slurry mix-shield TBMs.

The segmental tunnel linings have an internal diameter of 12.4 m and a lining thickness of 550 mm (see Figure 10). The excavated tunnel diameter was approximately 14 m, including a 250 mm overcut annulus. The soil cover over tunnel diameter ratios ranged from 2 to 3. The width of a lining ring is 2.2 m, comprising 6 standard segments, 2 counter keys, and 1 large key. The lining segments are made of Grade 55 precast reinforced concrete. The connectors at the circumferential joints were temporary spear bolts or bicones/shear cones at special locations (e.g., opening for cross passage). The contact between the radial joints was flat, and the contact width was 370 mm. Ethylene propylene diene monomer (EPDM) gaskets were provided at the radial and circumferential joins near the extrados of the lining segments for watertightness.

As per the Contract requirement, the limiting tunnel squat (i.e., change in the tunnel diameter) is 1%, which is 130 mm over the operational life of 120 years. The average of the internal and external diameters of the tunnel is 12.95 m. The 130 mm squat limit needs to cater for the installation tolerance of linings, soil loadings, and tunnel internal structure loadings.

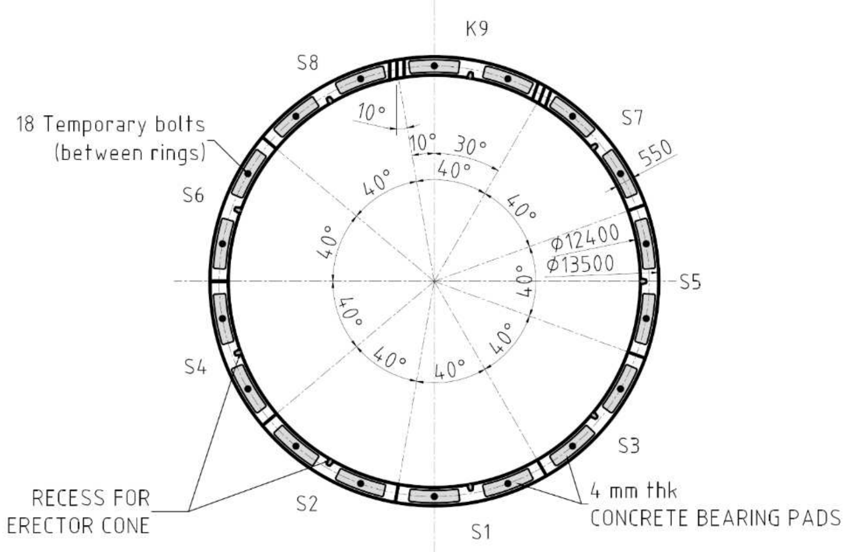


Figure 10. Segmental tunnel linings (dimension in mm).

The TBM would tunnel through the Alluvium layer, comprising interlayers of clay and sand, as seen in Figure 2. The TBM excavation would generate excess pore water pressures in the alluvial clay layer and the MD layer above. This would lead to consolidation settlement or compression in the alluvial clay and the MD layers, as well as secondary compression (creep) in the long term. These would cause the tunnel lining to deform. A barrette system was therefore designed as a mitigative measure to reduce the tunnel squat in the long term. As part of the barrette design process, a series of 3D finite element analyses were carried out to assess the effectiveness of the barrettes in attracting soil stresses around the excavated tunnels and to optimise the geometry and spacing of the barrettes. Figure 11 shows the predicted soil arching around the excavated tunnels and concentration of stresses in the barrettes at a vertical cross-section in the 3D analysis.

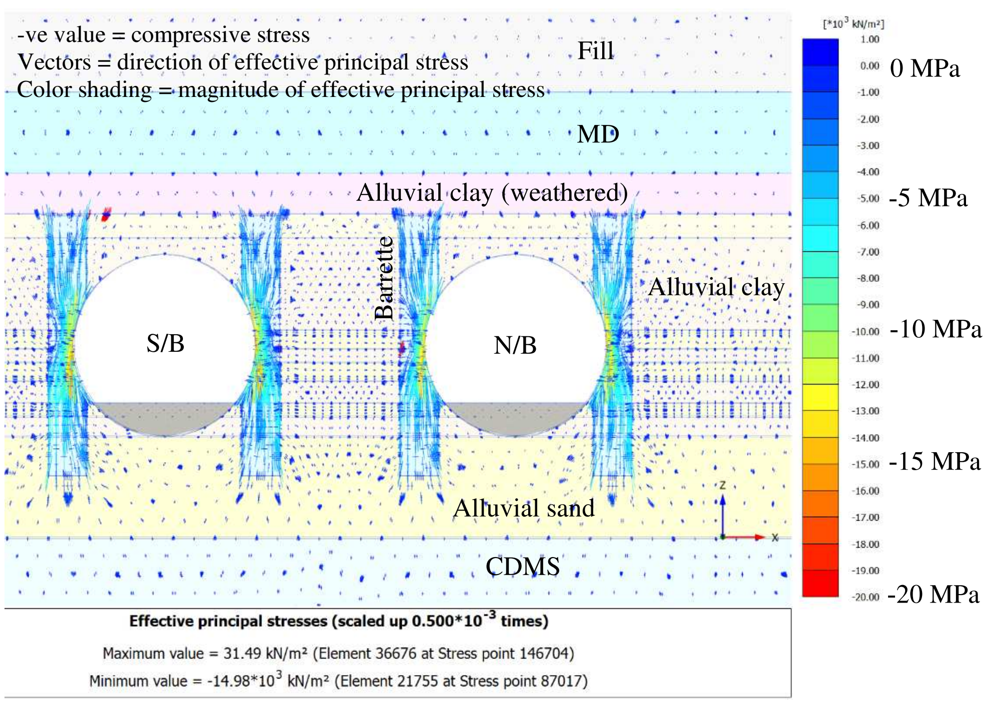
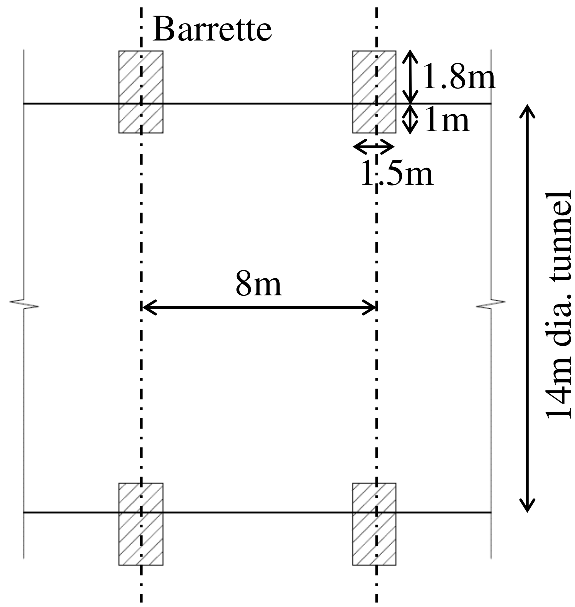
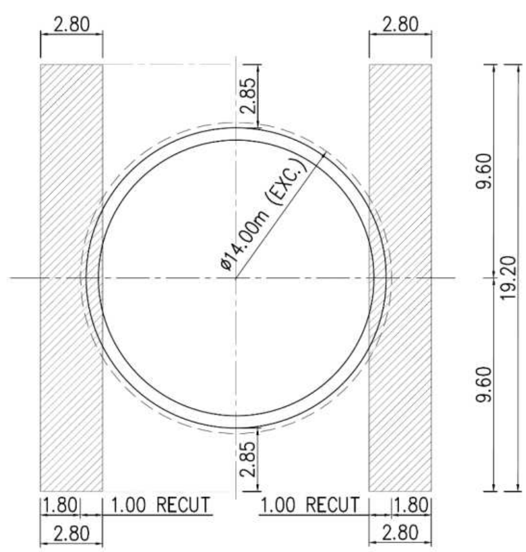


Figure 11. Soil arching (indicated by rotation of direction of prinipal stresses) and concentration of stresses in barrettes predicted by 3D analysis.

As shown in Figure 12, the unreinforced barrettes were constructed at both sides of the tunnel springline prior to tunnel excavation. The barrettes were constructed by using the diaphragm wall trenching method. The excavated trench, sized 1.5 m x 2.8 m, was tremie concreted without reinforcement. The excavated trench from the ground surface level to the top of barrette was backfilled with sand. A lower concrete grade C20 was adopted to ensure that the TBM could excavate through the barrettes without difficulty. The barrettes extended from 2.85 m above the tunnel crown to 2.85 m below the invert, with a total height of 19.2 m. On plan, they were 2.8 m (breadth) x 1.5 m (width), and spaced at 8 m center-to-center. They functioned by attracting soil arching load caused by the tunnel excavation, thereby reducing the increase of stresses (excess pore pressures) in the clay layers. The concentrated stress in the barrette was transmitted to the denser/stiffer stratum below the tunnel invert, e.g., alluvial sand.



(a) Plan view.



(b) Cross-sectional view.

Figure 12. Barrettes were installed at tunnel springline to reduce tunnel squat (dimensions in meters).

TUNNELLING PERFORMANCE

Applied Versus Design Confinement Pressures

In the slurry mix-shield TBMs, precise control of slurry support pressure at the tunnel face was managed by adjusting the pressure in the compressed air plenum chamber. The design slurry pressures were calculated based on GEO Report No. 249 (GEO, 2009), fulfilling both stability (ultimate limit state) and deformation (serviceability limit state) requirements. At locations involving more complicated ground conditions, a 3D finite element analysis was also carried out as a sanity check to the results of GEO Report No. 249. Figure 13 compares the applied and design slurry pressures at the tunnel axis level along the N/B tunnel. The slurry pressures along the S/B tunnel are of similar trend and hence not presented.

When the TBM approached the seawall, the applied slurry pressure was gradually increased due to an increase in soil overburden pressure. In the section between the seawall and the ventilation shafts, the applied slurry pressures were kept higher than the design pressures by up to 50 kPa. This was a precautionary measure to control the ground settlement and seawall stability in this area of the newly reclaimed land.

In the section between the ventilation shafts and the cut-and-cover tunnels, where the TBM tunnel was supported by the barrettes, the applied slurry pressures generally varied within ± 30 kPa from the design pressures. Please note that a variation of ±20 kPa was considered in the design slurry pressure. Although the field variation was slightly larger by ±10 kPa, it was considered acceptable because the magnitude of the applied slurry pressures was relatively large, between 450 kPa and 500 kPa.

At the locations where compressed air interventions were carried out to service the TBM cutter head, a temporary drop in the applied slurry pressures was observed, which recovered quickly when the TBM tunnelling recommenced. Grout blocks were installed at these planned intervention locations. They were designed to maintain the tunnel face stability when the TBM converted from the slurry mode to compressed air mode with reduced pressure. This was the reason why a drop in the slurry pressure was observed. When the compressed air intervention was complete, the slurry pressure was raised to the targeted level to resume the TBM excavation.

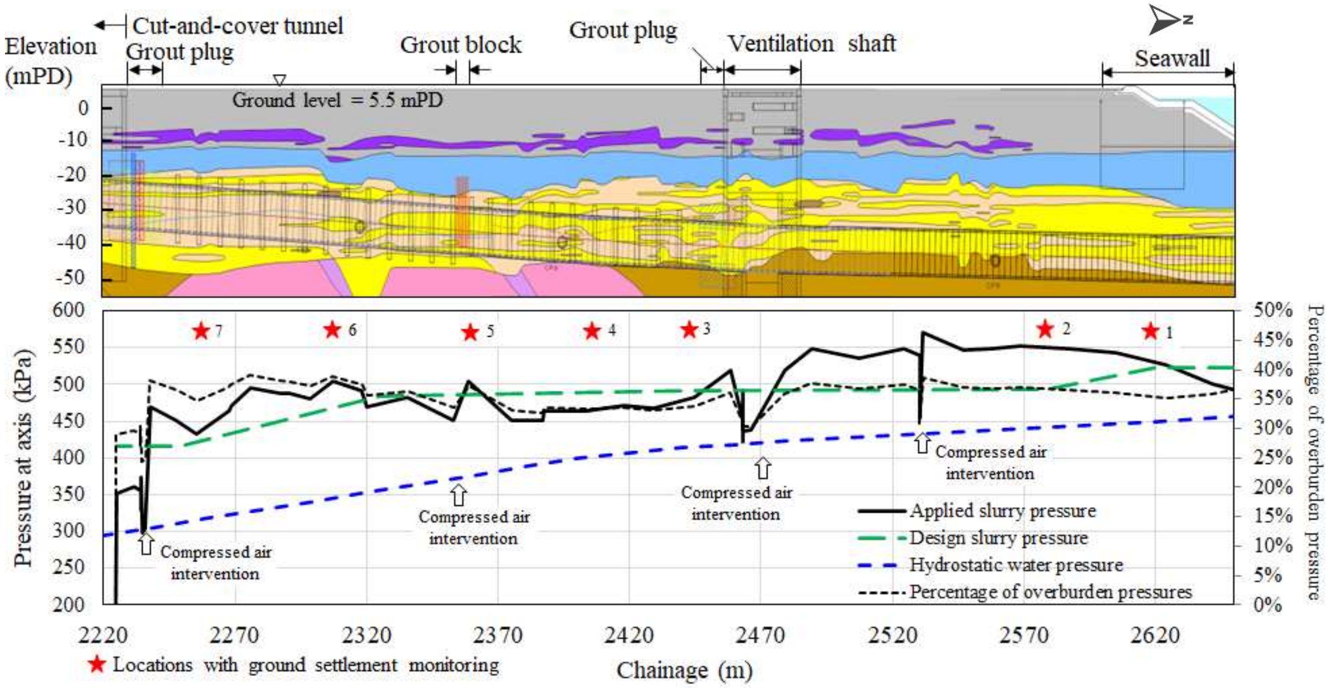


Figure 13. Applied versus design TBM face slurry pressure at tunnel axis of N/B tunnel.

Compressed Air Interventions

During interventions, the TBM switched to the compressed air mode to balance the water and earth pressures. To maintain the tunnel face stability, at the planned intervention locations a grout block of 19 m high × 19 m wide × 5.7 m thick was installed using cutter-soil mixing before the TBM reached the locations (see Figure 14). In the cutter-soil mixing method, soils were excavated by two cutter wheels rotating about horizontal axes, while injecting cement grout into the soils. This created in situ mixing of the cement grout with the soils. This technique was suitable for the strength and stiffness of the alluvial clay/sand encountered, due to its high speed of construction and relatively lower strength required of the mixed material. The grout block improved the mechanical properties of the ground with a minimum unconfined compressive strength of 1.5 MPa and Young's modulus of 300 MPa at 28 days by in situ mixing of the soil with cement, thereby reducing the risk of tunnel face stability problem. The permeability of the grout block was 1×10^{-7} m/s or less.

The slurry level within the excavation chamber was drawn down to 1.2 m below the tunnel axis to carry out partial face intervention. According to the GEO Report No. 249, the design compressed air pressure shall be set to the higher of (i) the water pressure at a level 1 m above the base of the exposed face, and (ii) the average slurry pressure over the exposed face required to maintain stability. For a sea level varying from +0.3 mPD to +2.1 mPD, the corresponding hydrostatic groundwater pressure at 1 m above the slurry drawdown level would be 3.70 to 3.88 bars in the barrette-supported tunnel area. The applied compressed air pressure was approximately 3.94 bars, which was considered adequate.

Figure 14 presents the trend of ground settlements measured before, during and after a 7-day intervention carried out at the N/B tunnel. When the TBM went past settlement markers A, B, and C, an increase in settlement by 5 mm to 9 mm was observed. The TBM reached the grout block on February 19th, 2018. Since then, some ongoing consolidation settlements were observed. For example, marker B located above the tunnel centreline recorded a settlement increase from 9 mm to 12 mm from February 19th, 2018 to March 28th, 2018. The compressed air intervention was carried out from March 28th, 2018 to April 4th, 2018, during which there was no sudden increase in settlement. Thereafter, the consolidation settlement continued and stabilised at the end of April 2018, e.g., 16 mm at marker B. These results suggested that with the grout block in place, the compressed air intervention was successfully carried out without causing any additional settlement related to the temporary TBM stoppage.

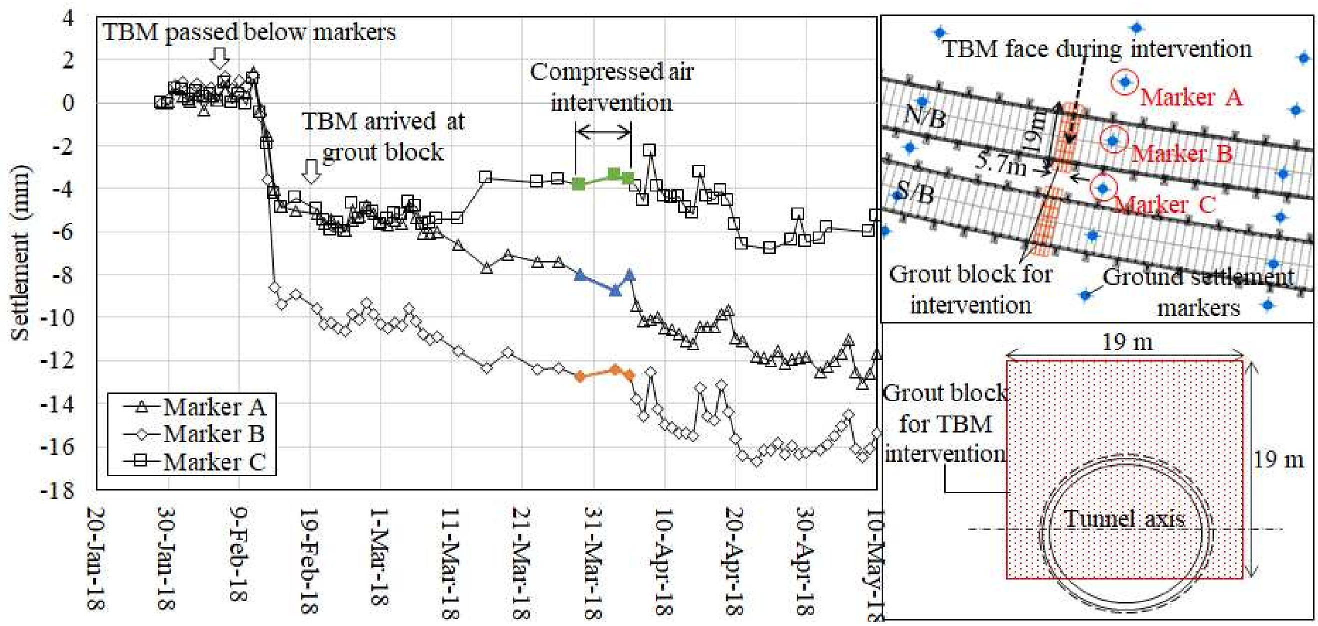


Figure 14. Ground surface settlements during a compressed air intervention.

Measured Ground Surface Settlements and Volume Loss Ratios

Ground surface settlements measured at seven cross-sections (Sections 1 to 7 in Figure 13) were used to determine the volume loss (VL) ratios induced by the TBM tunnelling. The VL ratio is defined as the area of ground surface settlement trough divided by the tunnel cross-sectional area. Table 1 summarizes the VL ratios determined from the measured settlement troughs at the seven sections. These VL ratios are given in four different stages: (1) after the 1st tunnel excavation; (2) consolidation and just before the 2nd tunnel excavation; (3) after the 2nd tunnel excavation; and (4) final, which was after completion of the consolidation. In stages (3) and (4), the VL ratios are calculated from the combined cross-sectional area of the two excavated tunnels. Three sections are chosen to present the settlement troughs graphically, as shown in Figures 15, 16, and 17 for Sections 1, 2, and 5, respectively. Section 1 is located at the seawall, comprising the steel cells and sloping rock armour, where there was an increase in soil overburden when the TBM entered the SL reclamation from the subsea section. Section 2 is located within the reclamation to the north of the ventilation shafts, without any barrettes. Section 5 is located within the reclamation to the south of the ventilation shafts, where barrettes were installed prior to the arrival of the TBMs.

Table 1. Measured volume loss ratios at seven monitoring sections of ground surface settlements.

Section	After 1st tunnel excavation (area of one tunnel = 153.9 m^2)		Consolidation and just before 2nd tunnel excavation		After 2nd tunnel excavation (area of two tunnels = 307.9 m^2)		Final, after completion of consolidation	
	Volume loss (m^2)	Volume loss ratio* (%)	Volume loss (m^2)	Volume loss ratio* (%)	Volume loss (m^2)	Volume loss ratio* (%)	Volume loss (m^2)	Volume loss ratio* (%)
1	1.28	0.83	3.01	1.96	5.08	1.65	6.37	2.07
2	0.49	0.32	0.98	0.64	2.09	0.68	3.91	1.27
3	0.92	0.06	0.18	0.12	0.73	0.24	0.92	0.30
4	0.71	0.46	0.83	0.54	1.66	0.54	1.72	0.56
5	0.41	0.27	0.56	0.37	0.92	0.30	1.04	0.34
6	0.06	0.04	0.12	0.08	0.21	0.07	0.22	0.07
7	0.06	0.04	0.23	0.15	0.22	0.07	0.22	0.07

* Volume loss ratio = cumulative area of ground surface settlement divided by the combined cross-sectional area of excavated tunnels.

At Section 1 (Figure 15), the 1st TBM for the S/B tunnel drove underneath the monitoring section in January 2017. Up to 30 mm ground settlement was measured when the TBM went past the section by 42 m, i.e., three times the excavation diameter. The corresponding VL ratio induced was 0.83%. Due to consolidation, the settlement continued to increase to 55 mm by April 2017, when the 2nd TBM for the N/B tunnel arrived. After the 2nd TBM drive, the total VL ratio for the two tunnels combined was 1.65% with a maximum settlement of 60 mm, and further increased to 2.07% with a maximum settlement of 75 mm after consolidation. The design limits were 2% and 100 mm, respectively. Please note that at Section 1, the toe of the seawall steel cells was only 13 m above the TBM (i.e., less than one tunnel diameter). Although the design VL ratio of 2% was slightly exceeded, there was no sign of distress at the seawalls.

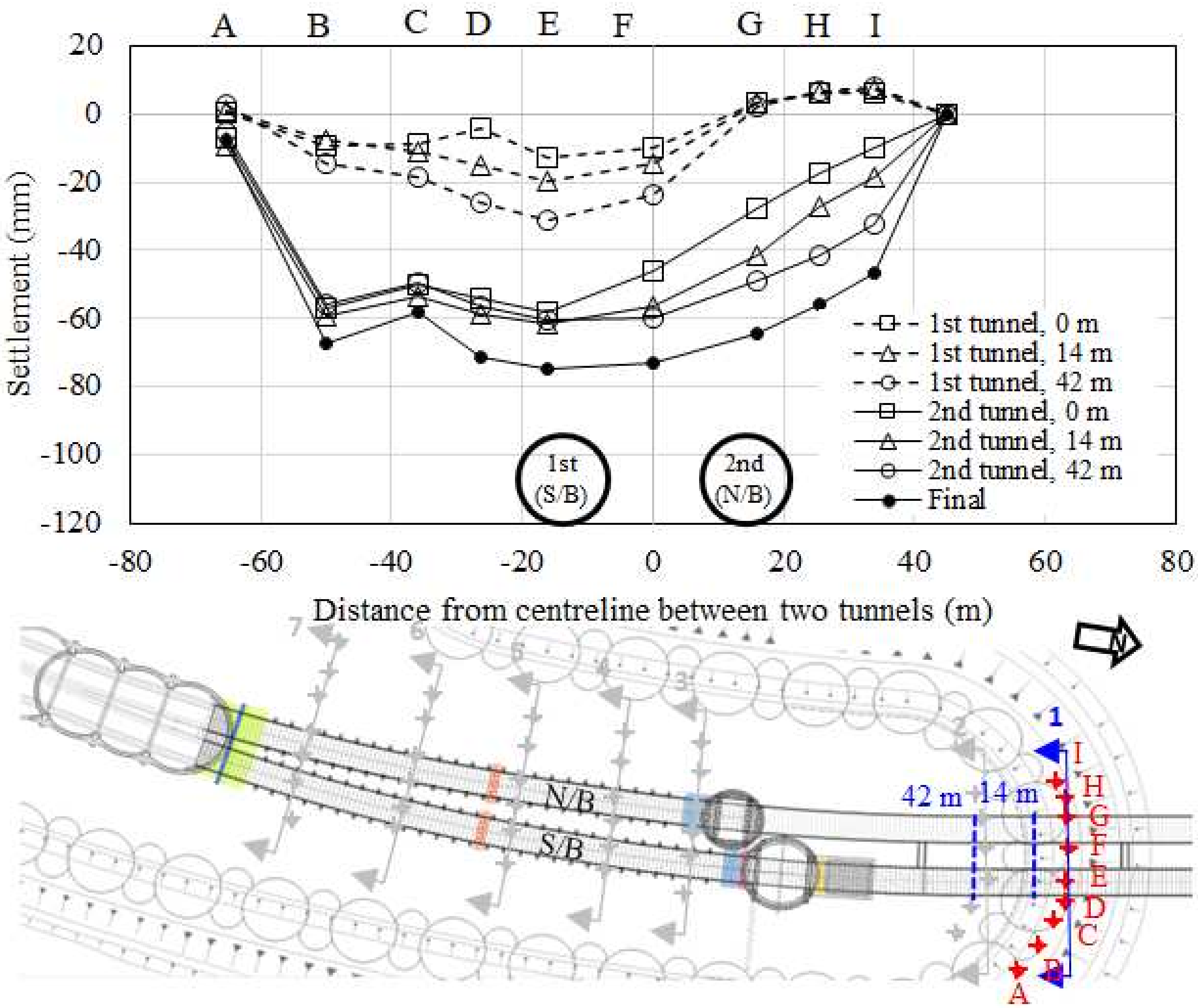


Figure 15: Ground surface settlements induced by tunnel excavation under seawall at Section 1.

At Section 2 (Figure 16), the applied slurry pressures were 40 to 50 kPa larger than the design values (Figure 13). The larger applied pressures had been checked so as not to create a ground heave problem. The 1st TBM drive caused a VL ratio of 0.32% and a maximum ground settlement of 21 mm. Due to consolidation, they increased to 0.64% and 36 mm in the subsequent three months just before the 2nd TBM arrived. The VL ratio and maximum settlement were 0.68% and 43 mm, respectively, after the 2nd TBM drive, and increased to 1.27% and 64 mm, respectively, after consolidation.

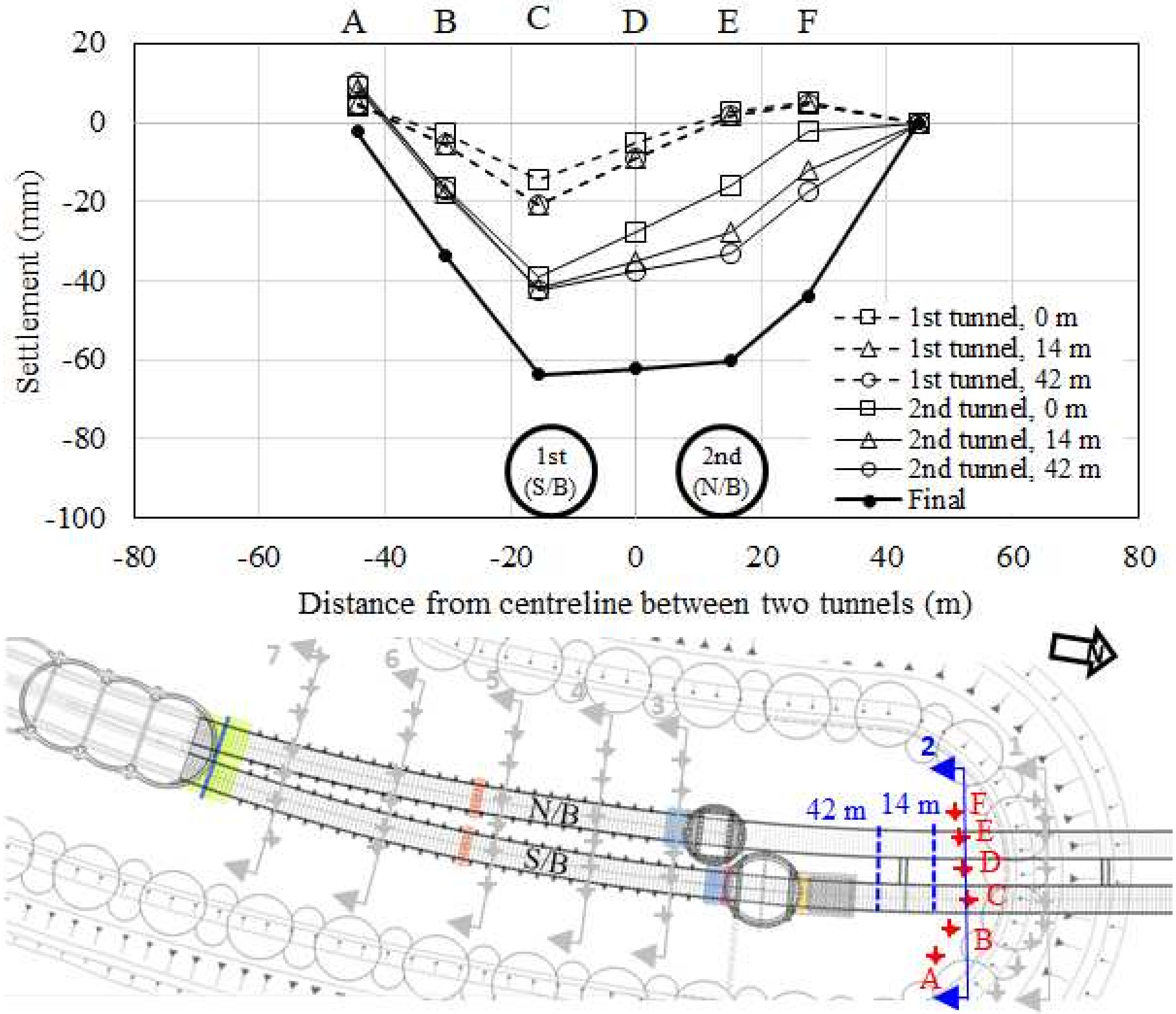


Figure 16: Ground surface settlements induced by tunnel excavation under reclamation without barrettes at Section 2.

At Section 5 (Figure 17) to the south of the ventilation shafts, the 1st TBM drive was carried out for the N/B tunnel instead. The tunnels were supported by the barrettes. The 1st TBM drive caused a VL ratio of 0.27% and maximum 15 mm ground settlement. Due to consolidation, they increased to 0.37% and 16 mm, respectively, in the following three months just before the 2nd TBM arrived. After the 2nd TBM drive and at completion of the consolidation, the final VL ratio and settlement were 0.34% and 17 mm, respectively. Please note that at Section 5, the applied slurry pressures were as much as 30 kPa lower than the design value (Figure 13), yet the induced VL ratio and maximum settlement were smaller than those at Section 2. This signifies the effectiveness of the barrettes in reducing the ground settlement caused by the tunnelling.

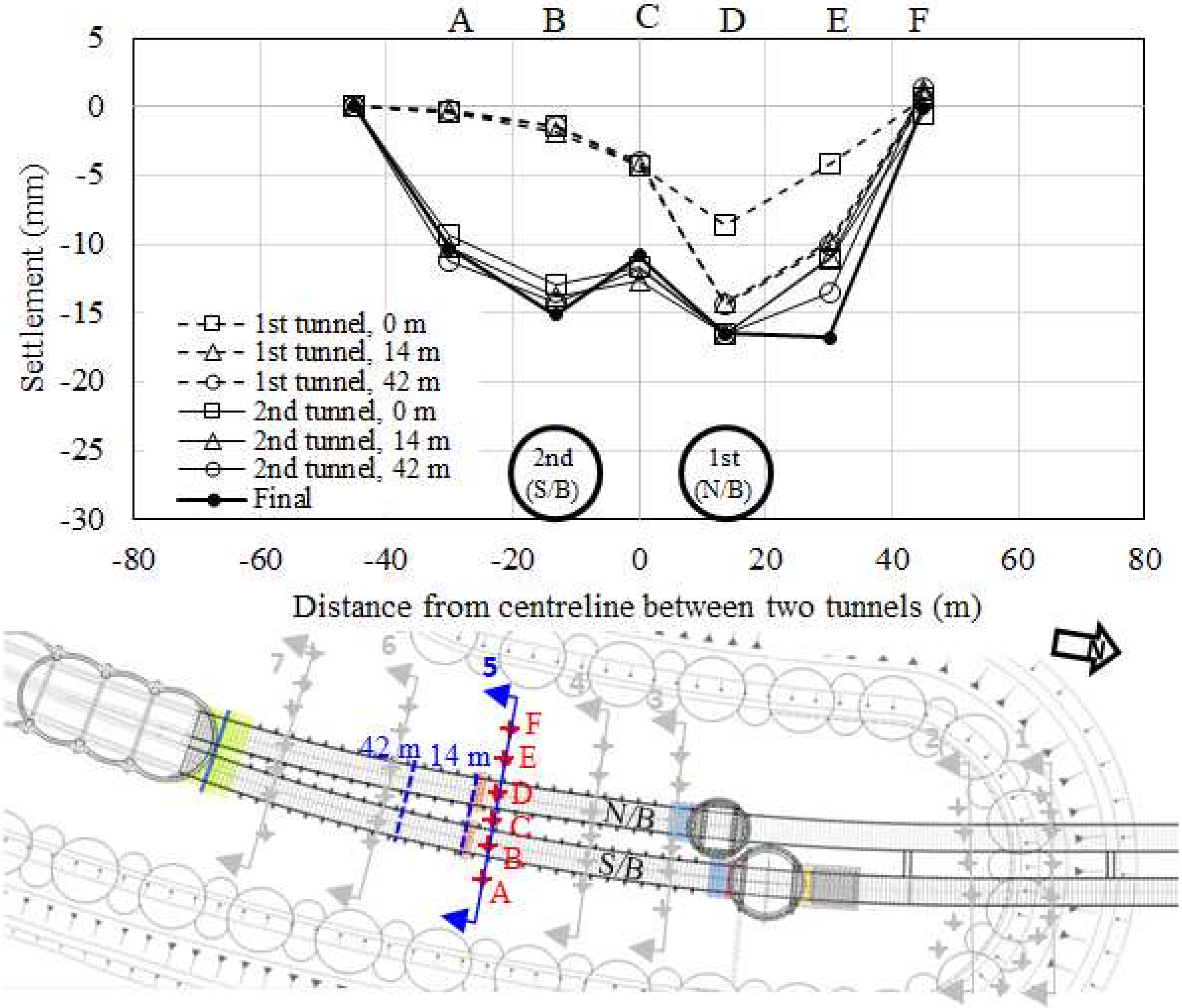


Figure 17: Ground surface settlements induced by tunnel excavation under reclamation with barrettes at Section 5.

The measured VL ratios were generally smaller than the predicted VL ratios at the design stage using finite element analysis, which modelled the confinement pressure supproting the tunnel excavation. The gain in soil strength and stiffness over time had contributed to smaller ground settlements induced by tunnelling. The only exception occurred at Section 1, where the measured VL ratio of 1.65% after the 2nd tunnel excavation was larger than the predicted 0.9%. This was because the ground was undergoing a higher rate of consolidation when the TBM entered the reclamation zone at Section 1. The sloping seawall there and the assemblage of seawall rubble mound resulted in a rapid change of soil overburden and stiffness/compactness, which might have contributed to the measured ground settlement being larger than the prediction.

Tunnelling-Induced Consolidation Settlements

The TBM tunnelling in alluvial clay would cause excess pore water pressures, the dissipation of which caused consolidation settlements over time. Figure 18 and Figure 19 present the consolidation settlements after the 1st and 2nd TBM drives, respectively. The presented settlement values refer to their increases after the TBMs went past by more than 42 m, i.e., three times the excavation diameter, from the marker location. The consolidation settlements at/near the seawall markers A1, A2, B1, and B2 were distinctively larger than the consolidation settlements at the tunnel section supported by the barrettes. At the seawall markers, the consolidation settlements reached 25 mm to 30 mm in approximately 60 days. The tunnelling-induced consolidation settlements accounted for 0.75% to 0.97% of the final VL ratios of 1.27% to 2.07% measured at/near the seawalls.

Despite the area to the south of the ventilation shafts having a thicker alluvial clay layer (see Figure 2), the observed consolidation settlements at markers A3 to A7 and B3 to B7 were smaller than 5 mm. The consolidation settlements only contributed to 0.03% - 0.10% of the final VL ratios of 0.07% - 0.56%. This signifies that due to the stiffness of the barrettes, they had attracted a significant portion of soil arching load caused by the tunnel excavation, and the soil load was transmitted down to the underlying dense/stiff Alluvium. This resulted in less excess pore water pressures generated in the alluvial clay layer and the MD above, hence the consolidation settlements were smaller.

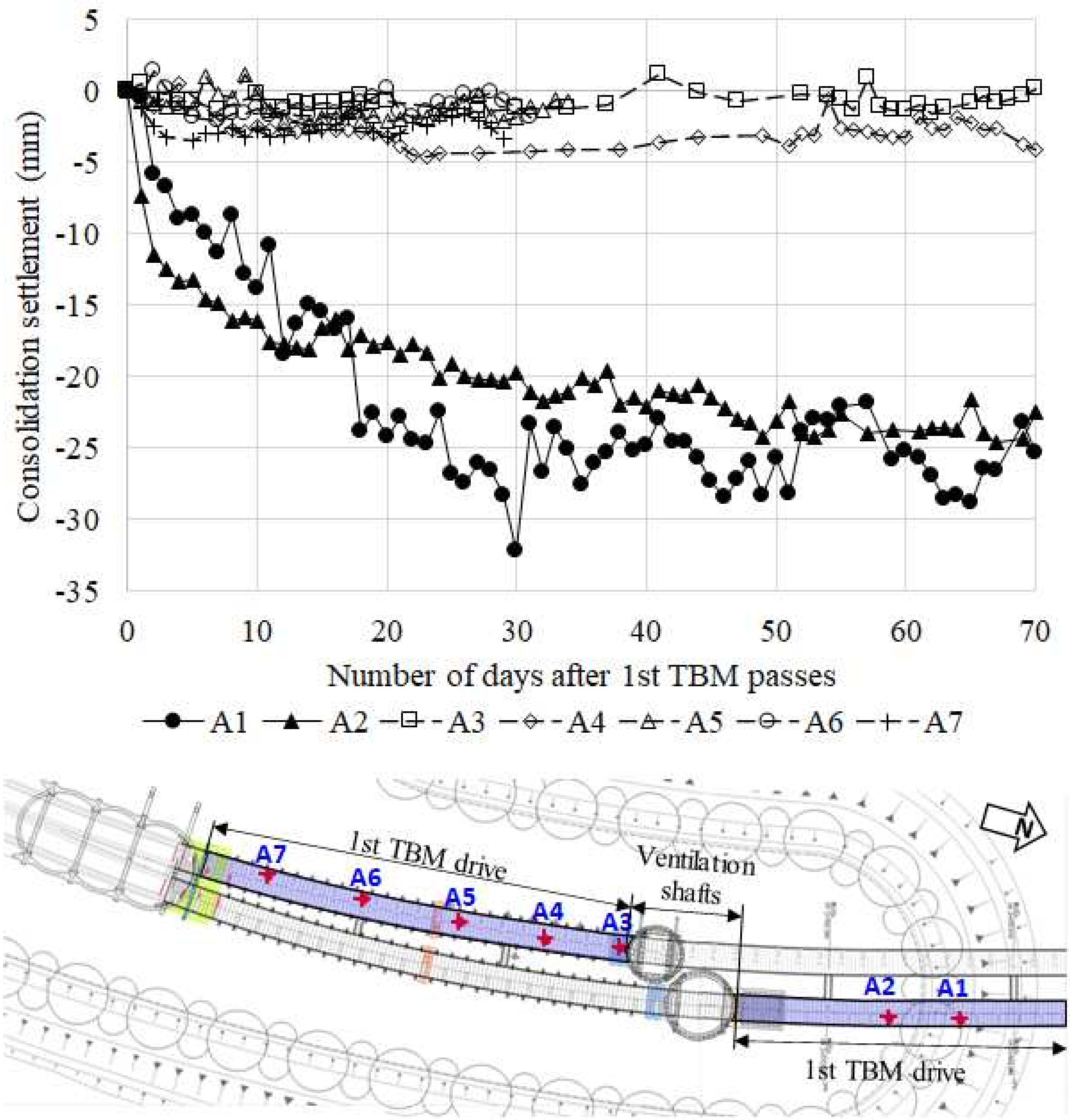


Figure 18: Tunnelling-induced consolidation settlements after 1st TBM drive.

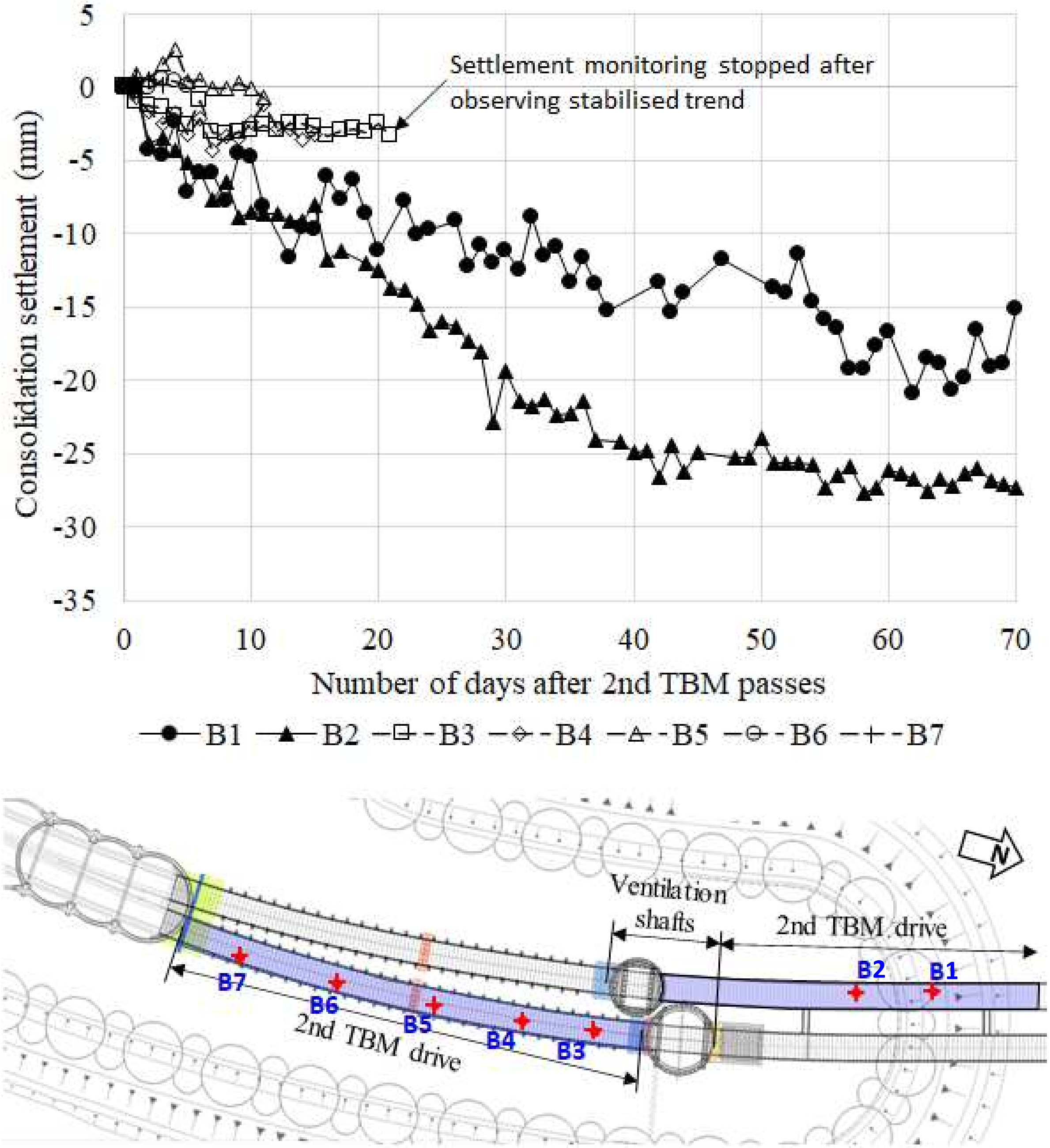


Figure 19: Tunnelling-induced consolidation settlements after 2nd TBM drive.

Deformation of Tunnel Linings

Chan & Kwong (2019) presents the monitoring data of tunnel squat in the short term. The tunnel squat was measured by convergence monitoring using total stations, which measured the positions of optical targets installed onto the internal face of the tunnel linings. For the section without barrettes between the seawall and ventilation shafts, the measured maximum squat was up to 97 mm at the N/B tunnel and 61 mm at the S/B tunnel. For the section with barrettes between the ventilation shafts and cut-and-cover tunnels, the measured squat was up to 59 mm and 32 mm at the N/B and S/B tunnel, respectively. The S/B tunnel showed less deformation than the N/B tunnel because the S/B tunnel was constructed later, allowing more time for the ground to consolidate and gain strength. The reduction in tunnel squat by approximately 40% to 50% for the section with barrettes compared to that without barrettes was primarily attributed to the effectiveness of barrettes in restraining tunnel squat as a result of the enhanced soil arching and secondarily to the gain in soil strength from consolidation.

DISCUSSION

To assess the performance of TBM tunnelling at the Southern Landfall (SL) of TMCLKL, the measured VL ratios at SL have been compared to those from recent TBM tunnelling projects in Hong Kong. Table 2 summarizes the VL ratios reported by Kwok et al. (2017), Park et al. (2018), and Kwong et al. (2019) in the well-documented case histories. They are generally smaller than 1.2%, with an average of approximately 0.5%. At SL, the measured final VL ratios are 2.07% at Section 1 (seawall), 1.27% at Section 2 (without barrettes), and 0.34% at Section 5 (with barrettes).

At Section 1, the TBM tunnelling was located beneath the toe of seawall steel cells with a vertical separation of 13 m. The applied confinement pressure was similar to the design confinement pressure, referring to Figure 13. The settlement at depth induced by tunnelling was greater than that at the ground surface. The steel cells might have transmitted the settlement at depth to the ground surface level. This resulted in a larger VL ratio of 2.07% compared to a maximum of 1.2% reported in the Hong Kong case histories.

At Section 2 (without barrettes), the measured VL ratio of 1.27% falls close to the reported maximum of 1.2%, despite the applied confinement pressure being 50 kPa higher than the design confinement pressure (see Figure 13). This could be due to the conditions of the recently surcharged ground underlying the new reclamation, where some remaining consolidation and/or creep settlements were ongoing in the clay layers.

At Sections 3 to 7 (with barrettes), the measured VL ratios range from 0.07% to 0.56% with an average of 0.27%, referring to Table 1. These values are generally smaller than the average of 0.5% reported in the Hong Kong literature. The applied confinement pressures were +/- 40 kPa within the design confinement pressures (see Figure 13). This was likely caused by the barrettes attracting soil arching load above the tunnel crown arising from the tunnel excavation, and they transmitted the soil load into the deeper dense/stiff Alluvium.

Table 2. Volume loss ratios measured in recent TBM tunnelling projects in Hong Kong.

Reference	TBM type	Excavated diameter (m)	Excavated soils	Tunnel cover ÷ tunnel diameter	Groundwater level (mbgl[#])	Volume loss ratio (%)
Kwok et al. (2017)	Slurry mix-shield, variable density	7.5 Stacked twin tunnels	Fill, MD, Alluvium, CDG, HDG, MDG^	0.5 to 3	1 to 3	0.1 to 1.0
Park et al. (2018)	Slurry pressure balanced	7.5 Stacked twin tunnels	CDG, HDG, MDG	1 to 2	1 to 3	0.24 to 0.63 (average 0.43)
Kwong et al. (2019)	Slurry mix-shield	14 and 17.6 Side by side twin tunnels	Fill, MD, Alluvium, CDG, HD, MDG	0.5 to 2	1 to 3	0.1 to 1.2 (average 0.49)

^ HDG = highly decomposed granite, MDG = moderately decomposed granite
[#] mbgl = meter below ground level

CONCLUSIONS

The TMCLKL project successfully carried out 14 m diameter slurry mix-shield TBM tunnelling beneath a recently reclaimed land at Southern Landfall in challenging ground conditions. An innovative design was conceived by installing barrettes sized 1.5 m × 2.8 m × 19 m at the tunnel springlines at 8 m c/c spacings to attract soil arching load caused by the tunnel excavation and transmit the soil load to the deeper dense/stiff Alluvium, and to limit the tunnel squat caused by clay compression in the long term. Compressed air interventions were successfully carried out in the pre-installed grout plugs without causing any significant additional settlement.

Recent TBM tunnelling projects in Hong Kong reported VL ratios of smaller than 1.2%, with an average of approximately 0.5%. The TBM tunnelling in Alluvium at the Southern Landfall of TMCLKL inevitably caused some ground surface settlements. Interpretation of the settlement results revealed that the tunnelling beneath:

- the seawall steel cells (Section 1) caused the largest VL ratio of 2.07%, as the steel cells might have transmitted the settlement at depth caused by tunnelling to the ground surface level;
- the reclamation without barrettes (Section 2) caused a VL ratio of 1.27%; and
- the reclamation with barrettes (Sections 3 to 7) caused the smallest VL ratios of 0.07% to 0.56% (average 0.27%). This demonstrated the effectiveness of the barrettes in reducing the ground settlement caused by the TBM tunnelling.

Post-tunnelling consolidation settlements were larger at/near the seawall area, and they typically completed in 30 to 60 days with a magnitude of 25 mm to 30 mm for each tunnel. Approximately half of the final volume loss ratios at/near the seawall area were contributed by the consolidation settlements. In the area where the ground around the tunnel section was installed with unreinforced concrete barrettes, the consolidation settlements were smaller than 5 mm for each tunnel bore and they only contributed up to one-fifth of the final volume loss ratios. The tunnel squat was reduced by 40% to 50% at the tunnel section with barrettes compared to the section without barrettes. The gain in soil strength due to consolidation also contributed partly to the reduction of tunnel deformation.

At the completion of the tunnelling-induced settlements, there was no distress caused to the recent reclamation platform and seawalls. The TMCLKL tunnels provide a valuable case history for future large diameter tunnels to be constructed in heterogeneous ground conditions underlying new reclamations and soils developing consolidation and creep settlements.

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