

DETAILED DESIGN OF THE LAI CHI KOK DRAINAGE TUNNELS

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ABSTRACT

The Lai Chi Kok Transfer Scheme provides flood protection in Hong Kong through a system of tunnels, several drop shafts, a stilling basin and an outfall structure. The system is designed to divert the stormwater flowing from the hinterland catchment and discharge it into the sea before it enters the existing drainage system in the downtown urbanised areas of Lai Chi Kok, Sham Shui Po and Cheung Cha Wan. Atkins was the contractor's designer and this paper describes the detailed design of the tunnels and shafts and the corresponding ground movement impact assessments.

INTRODUCTION

The Lai Chi Kok Transfer Scheme forms part of the overall flood control system for the West Kowloon District implemented by the Drainage Services Department, the Government, of the Hong Kong Special Administrative Region. The scheme features six dropshaft intakes at the watercourses on the Lai Chi Kok hillside to intercept the stormwater runoff, which is directed into the Branch Tunnel. The stormwater runoff is then discharged through the Branch Tunnel into a stilling basin structure at the inlet shaft M1 constructed below Butterfly Valley Road at Wai Man Tsuen. From there, the stormwater runoff travels through the Main Tunnel, which is in the form of an inverted siphon, before being discharged at outlet shaft M2 into Victoria Harbour near Stonecutter Island.

The project adopted a design and build contract approach, with the principal client being responsible for the tunnel alignment and hydraulic design of the systems and contractor responsible for the detailed design of the temporary and permanent tunnel structures based on the engineer's reference design. The contract was awarded to the Leighton-John Holland Joint Venture (JV) in November 2008 and Atkins was employed as the design consultant to provide a full range of integrated design services including civil, structural, geotechnical, environmental, traffic, architectural, electrical and mechanical engineering design. The layout of the project is shown in Figure 1.

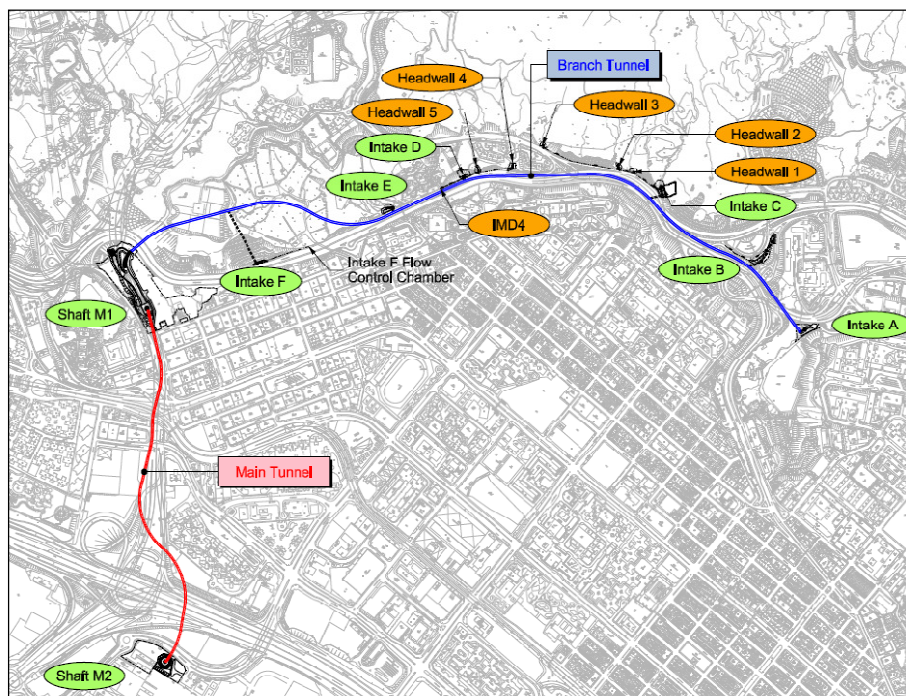


Figure 1 – Layout of the Lai Chi Kok Transfer Scheme

MAIN AND BRANCH TUNNELS

The tunnel part of the Lai Chi Kok Transfer Scheme comprises the Main and Branch Tunnels.

Main Tunnel - The Main Tunnel is approximately 1.2 km in length, with an internal diameter of 4.9 m. The ground surface along the alignment was formed using reclamation fill. The 1.2 km Main Tunnel runs beneath a congested urban area in Lai Chi Kok. The Main Tunnel starts from shaft M1, which is situated beneath Butterfly Valley Road and the junction of Cheung Sha Wan and Lai Chi Kok Roads and passes underneath the operating MTR Tsuen Wan Line tunnels and the proposed Express Rail Link (XRL) tunnel. The tunnel also passes in close proximity below and adjacent to the foundations of the Route 8 Lai Chi Kok Viaduct, drainage culverts and the structures of densely populated residential buildings. The southern section of the alignment also passes between various railway lines including the West Rail, Tung Chung and Airport Express lines. The depth of the tunnel crown, which is between 39 m and 42 m below ground level, is governed by the minimum allowable distance from the existing and proposed railway lines. The water flowing through the Main Tunnel travels from drop shaft M1 to rising shaft M2 before being discharged into the sea.

Branch Tunnel - The Branch Tunnel is approximately 2.5 km in length, with an internal diameter of 4.9 m. The alignment of the Branch Tunnel runs along the base of the Lai Chi Kok hillside parallel to Ching Cheung Road and Tai Po Road. The tunnel operates by gravity and carries the stormwater that enters the drop shafts that intercept the existing watercourses at the edge of the catchment areas along Ching Cheung Road. The stormwater is then conveyed to the watercourse at Butterfly Valley before travelling to the stilling basin at shaft M1.

GEOLOGY

The rock along the Main and Branch Tunnels predominantly comprises very strong to extremely strong and extremely abrasive granites of the Kowloon and Shatin granite plutons.

The alignment of the 2.5 km Branch Tunnel is primarily within strong and abrasive granite with an average Unconfined Compressive Strength (UCS) of around 200 MPa. The tunnel intersects with a number of mixed grounds and various thicknesses of transverse shear zones comprising highly decomposed granite of varying thickness. The ground water table generally lies within 1 m of the rock head level and flows towards the tunnel through weaknesses and jointing within the rock mass. High ground water inflow with pressures of up to 7 bar from the overburden would be expected along the geological faults or their associated sheared/shattered zones if the tunnel were excavated by an open mode Tunnel Boring Machine (TBM) or the drill and blast method. If these methods were adopted, probing and grouting would also be required in these zones to control the ground water inflow during construction before the installation of the permanent tunnel lining.

Unlike the Branch Tunnel, the majority of the Main Tunnel alignment is within soft ground, except for approximately the first 200 m. The tunnel first passes through very strong granite near shaft M1, then mixed ground containing corestones and finally full face into completely decomposed granite. The superficial deposits above the Main Tunnel generally consist of reclamation fill overlying marine deposits, which in turn overlie alluvium and completely decomposed granite. The rock along the Main Tunnel alignment is Shatin granite with occasional intrusions of fine to medium grained granite dykes and basalt dykes. The water table generally lies within 1 to 2 m of the existing ground level and the maximum hydrostatic pressure at the tunnel invert is around 4.2 bar. Clearly, the section of tunnel within soft ground would need to be constructed using a closed mode TBM to limit face loss and maintain face stability to protect the infrastructure and buildings above the tunnel alignment.

The Main Tunnel and the Branch Tunnel were excavated through very different geological conditions. The Branch Tunnel passes through very strong granite with UCS of more than 200 MPa with occasional faults and shear zones while the Main Tunnel mainly passes through completely decomposed granite with high hydrostatic pressure in soft ground above 4 bar. For the Branch Tunnel, the cutters needed to be strong enough to cut through abrasive rock while taking into account the necessary practical maintenance requirements. In the Main Tunnel, the key requirement was to excavate the ground while preventing

excessive face loss, which would have induced settlement within the completely decomposed granite and consequently affected the existing building structures.

TUNNEL BORING MACHINE

During the tendering stage, it was assumed that two TBMs would be used to excavate the tunnels with either a closed or open mode TBM for the Branch Tunnel and a close-shield machine for the Main Tunnel. However, the final tender submitted by the JV was based on the use of a single 5.72 m OD mixed-shield TBM, supplied by Herrenknecht, for both the Main and Branch Tunnels. The mixed-shield TBM is designed to be able to excavate within completely decomposed granite and in highly abrasive rock under slurry mode. By providing slurry face support via an air cushion inside the air bubble chamber, the mixed-shield TBM is able to provide better control of the slurry pressure and, hence, the face supporting pressure than the conventional slurry machines. Details of the TBM used for the Lai Chi Kok scheme are shown in Figure 2.

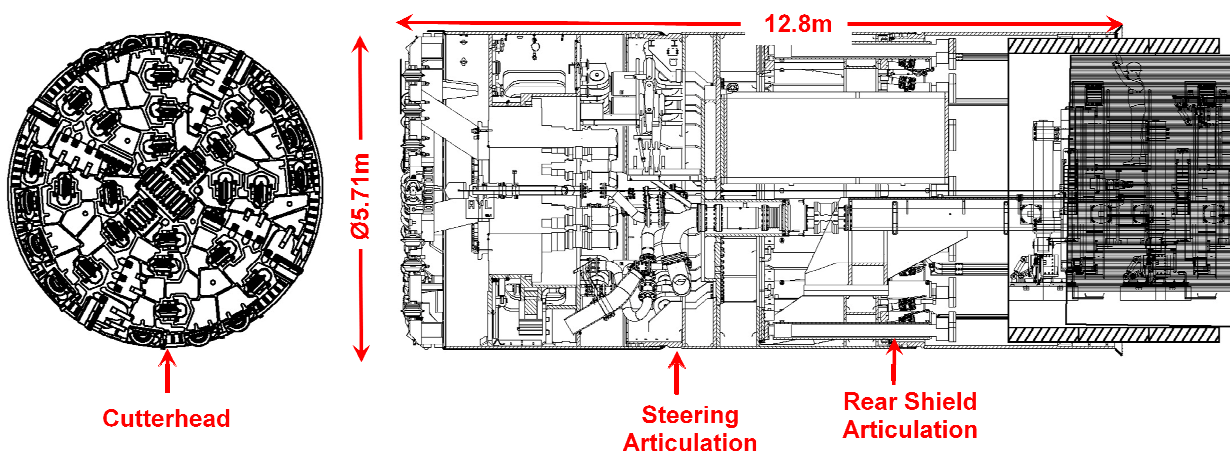


Figure 2: Layout of the TBM used for the Main and Branch Tunnels

The Branch Tunnel was excavated first and the TBM was launched in a pre-excavated chamber constructed using the New Austrian Tunneling Method (NATM) with mixed ground and shallow ground cover below Ching Cheung Road. After being pulled into place at the rock face with sufficient rock cover, the TBM then started its drive towards intakes E to B before finally being retrieved at intake A. At the launching chamber constructed by the NATM, permanent support was provided by segmental lining, with the gap between the shotcrete lining and the segmental lining filled with grout. After completion of the Branch Tunnel, the TBM was transported back to Butterfly Valley and launched again at the 40 m-deep shaft M1 for the second drive to construct the Main Tunnel.

During the course of the TBM tunnelling, the cutterhead and tools needed to be maintained, repaired and inspected at regular intervals, particularly in the sections of mixed ground and ground with corestones along the Main and Branch Tunnels. Interventions were also carried out to repair cutter discs damaged as a result of unforeseen geological conditions. All manned interventions were carried out in a safe working environment and ground treatment or compressed air was adopted as necessary to balance the external earth and ground water pressure. At the mixed ground sections of the Main Tunnel, confinement pressure of around 4.2 bar was necessary, which required special approval from the Labour Department Commissioner as it exceeded the recommended limit of 3.45 bar. This was the first time such a high compressed air level had been used for TBM tunnel intervention in Hong Kong. A detailed submission outlining its use was sent to the Labour Commissioner and approval was eventually given. The JV also drew on overseas experience extensively throughout the process. All manned interventions were completed safely and no cases of compressed air illness were reported.

The maximum allowable working time for people working in high compressed air environments is short. To improve the efficiency of the repair and maintenance of the cutterhead and cutting tools, two man locks were installed inside the TBM for teams to work in turn continuously inside the excavation chamber. The use of the Hambury tables in addition to the French table was approved for working in pressures of up to 4.2 bar to increase the duration of working in compressed air.

SEGMENT DESIGN

The Main Tunnel is permanently supported by precast concrete linings with five standard segments plus one key segment in a ring. The tunnel rings are universal taper rings with a nominal ring width of 1250 mm. The minimum achievable turning radius of the tunnel lining is 185 m, which is less than the actual minimum requirement of 200 m. The smaller allowable tunnel turning radius enabled corrections of the tunnel alignment to be made during the construction of the 200 m radius curves.

The lining is designed to be undrained and gaskets were installed between the segments to fulfil the required water tightness. All of the bolts were first temporarily installed to maintain the compression of the gaskets during the ring erection for water tightness and to maintain the circular shape of the “just” installed tunnel ring. The temporary bolts were progressively removed during the tunnel construction. A combined Ethylene Propylene Diene Monomer (EPDM)/hydrophilic was also used on all four edges of every segment to further enhance the watertightness. Guiding rods were used at all radial joints to provide guidance during the segment installation and shear resistance at the joints.

The closed form analysis model of Muir Wood (1975) and Curtis (1976) and numerical analysis were used for the structural design of the precast segmental lining to accommodate the ground and groundwater loads. In the closed form analysis, the lining was assumed to be deformed in an elliptical shape and the ground was expected to behave as an elastic continuum material. The hoop thrusts and bending moments induced by the soil-structure interaction were evaluated accordingly. For the numerical analysis, we adopted the finite element programme PLAXIS for the soft ground and mix ground sections and SAP2000 for the rock sections. We used these programmes to work out the worst combined bending moment and axial forces at critical sections, which were identified from the continuum analysis and from experience at sections considered to be critical for the lining design. In addition to the ground loading, the effects of the jacking for the TBM propulsion, grout and secondary grout pressure, and poor ring construction, handling and stacking were considered in the design of the segments.

The Branch Tunnel operates as a gravity drain and the external ground loading is always larger than the internal water pressure. Therefore, the design simply took into account the most critical external earth, hydraulic and surcharge pressures and their combination. However, due to the fact that the Main Tunnel is an inverted siphon, the design was slightly more complicated. The permanent tunnel lining is subject to high internal water pressures that are determined by the water head at shaft M1 and shaft M2. The design water head was taken as the top level of the over-flow weir at the top of shaft M1 and the top level of shaft M2. Any additional water above these levels is discharged into the existing box culvert connected to the surface structure of shaft M1 or to the outfall structure of shaft M2. The hydraulic head along the alignment was designed to linearly increase from shaft M1 to shaft M2.

In general, the internal water pressure is counter-balanced by the hoop forces induced by the external water pressure and soil loads. However, because the soil load acting on the tunnel lining varies with differing ground stiffness and the time since the lining was installed in the ground, we needed to conduct a sensitivity analysis to determine the minimum hoop stress against zero soil loading and various levels of ground stiffness. When the internal hydraulic pressure of the tunnel is higher than the sum of the external hydraulic pressure induced from the natural ground water and the soil pressure, the tunnel lining is subject to the net internal pressure pushing it outward. Because there are no physical connections holding the segments together, the segment radial joints may not be in compression and may tend to open up depending on the degree of the ground stiffness restraint.

A Finite Element Method (FEM) model was used to assess the risk of physical movement of the segment joints and ground due to the internal hydraulic pressure along the alignment. In the model, the ground medium directly adjacent to the tunnel lining was compressed outward by the internal water pressure applied to the ground through the segmental linings. The external diameter of the tunnel increased slightly when the ground was compressed. The interaction between the tunnel lining and the ground was modelled based on the variation in the soil spring predicted by Young's modulus. This model tends to be conservative as it ignores the compressive forces that act within the tunnel lining as a result of the ground load. Hence, the model is very much a theoretical worst case and only applies in the unlikely case that the external soil load is absent or diminished (i.e. as a result of annular voids, etc.). The model also ignores the variation in in-situ stresses around the tunnel (i.e. the bedding spring stiffness and in-situ stress normally dictate that any joint opening will occur in the tunnel soffit rather than the invert) and the fact that any sag ovalisation of the tunnel lining ring (i.e. sag of the soffit) would need to be reversed by overcoming the bending moments prior to any

joint passing from compression into tension. Nevertheless, the model is useful in providing a theoretical worst case such that avoidance of failure can be assured.

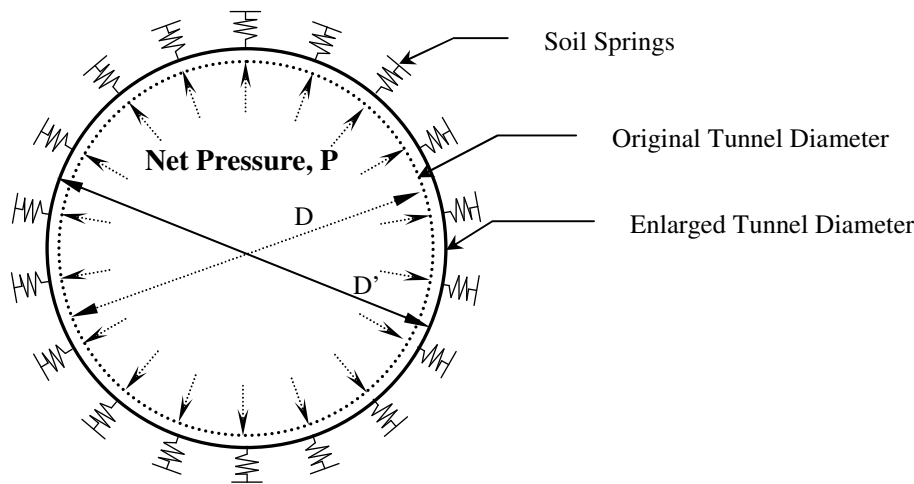


Figure 3 – Model of the Main Tunnel under extreme Internal Hydraulic Pressure

Several models with different ground stiffness were used to investigate the sensitivity of joint opening versus the stiffness of the ground. Possible effects including loss of water tightness from the gasket, water egress from the gaps of the joint under high water pressure and the segment falling due to loss of physical constraint were also studied. These models demonstrated that the segments and lining rings are stable under the extreme internal water pressure along the tunnel alignment. In addition, guiding rods were added to all of the radial joints which provide a physical interlock to further limit the freedom of movement of the segments under extreme conditions.

SHAFTS M1

Shaft M1 is located at the centre of the existing open channel and the centre of the large stilling basin, which connects to a decked channel that collects water from the Kowloon Group Reservoirs (KGR) and the Branch Tunnel through a stepped channel. During construction, a staged flow diversion and flooding provisions were provided in the form of pre-cast concrete panels to allow the construction of the decked channel. The panels also maintained the flow of the open channel and separated the existing open channel from the work area of shaft M1.

Shaft M1 has an internal diameter of 10 m and is situated 40 m below the existing ground surface. The shaft is supported by an undrained reinforced concrete lining, which is designed to be effectively maintenance free for the whole of its 120-year design life. The superficial deposits in the vicinity of the shaft are predominantly reclamation fill overlying alluvium. The rock underlying shaft M1 is Shatin granite (Jkt) and the contact zone between the Kowloon granite and the Shatin granite is located below the surface structure of the stilling basin, at approximately 10 m north of shaft M1. The upper soil section of the shaft excavation, which is about 6 m deep, is supported by a pipe pile cofferdam with permanent reinforced concrete shaft lining as the supporting ring beam. The pipe piles were socketted into rock by 1 m. Grout curtaining and 2 m toe grouting into the rock were carried out behind the pipe pile wall as a groundwater cut-off to prevent water ingress and excessive ground settlement. The rock section, including the backshunt and TBM launching cavern were excavated using the drill and blast method. Temporary support in the form of rock dowels and shotcrete were installed as the excavation proceeded. The rock excavation was carried out in stages as required and temporary supports were installed prior to starting the next phase of excavation. The backshunt at the base of the shaft was temporarily constructed to facilitate the tunnel excavation logistics and was backfilled using crushed rock with lean mix concrete after tunnel completion.

Shaft M2 and the Outfall

The flow from the Main Tunnel discharges into the outfall structure via rising shaft M2, which is located at the seafront near Stonecutter Island. The outfall consists of a 24 m-wide octagonal cascade at the connection with shaft M2. The octagonal cascade consists of a series of 0.5 m-high and 0.5 m-wide steps from the top to the cut off level of shaft M2.

The superficial deposits in the vicinity of shaft M2 predominantly consist of reclamation fill overlying alluvium, which in turn overlays completely decomposed granite, with portions of marine deposits. The rock underlying shaft M2 is Shatin granite, which is generally moderately or slightly weathered.

Shaft M2 is 10 m in internal diameter and is about 38.5 m deep wholly within soft-ground. A 1.5m thick circular diaphragm wall was selected as both the temporary and permanent structure of the shaft. The circular diaphragm wall was constructed from the existing ground level and then demolished simultaneously with the excavation work for the outfall construction until the level for connecting with the outfall base slab was reached

Shaft M2 was used as the TBM retrieval shaft and a 500 mm thick cast in-situ un-reinforced secondary concrete lining was installed within the permanent diaphragm wall. Soft eye was provided at the diaphragm wall with glass fibre polymer bars for the Main Tunnel break-through. The last segmental lining was installed beyond the diaphragm wall with an interface collar constructed to connect the shaft lining and the segments. A hydrophilic strip was installed at the interface between the segmental lining and the interface collar to ensure water-tightness. Reinforcement was provided between 2.5 m above the crown of the shaft opening and 0.45 m below the invert of the opening to provide additional capacity to the diaphragm wall around the tunnel opening. Grouting was used to seal the annular space between the segmental lining and the diaphragm wall at the junction.

The internal lining meets the requirements for hydraulic performance and durability and was designed to resist the full hydrostatic pressure. Hydraulic cut-off during the temporary stage was crucial for safe excavation, especially given the depth of the shaft, as the formation level was still within the soft section of ground located adjacent to the sea. Toe grouting below the diaphragm wall with a 2 m grout curtain was used as a temporary groundwater cut-off and a pumping test was undertaken to ensure hydraulic stability when excavation was carried out within the diaphragm wall.

For the plan view design of the diaphragm wall, the hoop spring, which simulates soil under compression around the shaft, was determined and used in the Finite Element Method (FEM) analysis. The PLAXIS model for vertical structural analysis was used for the sections based on an equivalent strut stiffness simulation of the elastic shortening of the circular shafts and local shortening of the joints in the diaphragm wall panels. In addition to the normal earth, water and surcharge loads, seismic loading was incorporated into the design to reflect the Hong Kong conditions.

The 3-dimensional (3D) finite element method was employed to study the stress distribution around the shaft opening for the Main Tunnel. The shaft lining was modelled as a structure comprising several "shell elements" placed in a 3D space. The shell elements were supported by a series of springs which represent the ground stiffness where the lining action applies passive pressure to the soil surrounding the shaft. A combination of loading cases was determined and applied directly to the shaft wall. The analysis output from the models included the axial force, bending moment, shear force and deflection in both the radial (horizontal) and vertical directions, in which the bending moment and axial forces were extracted and input into the column interaction diagram to determine the required reinforcement content.

Figure 4 shows the results of the 3D analysis of the reinforced collar structure and diaphragm wall at the TBM break-through opening.

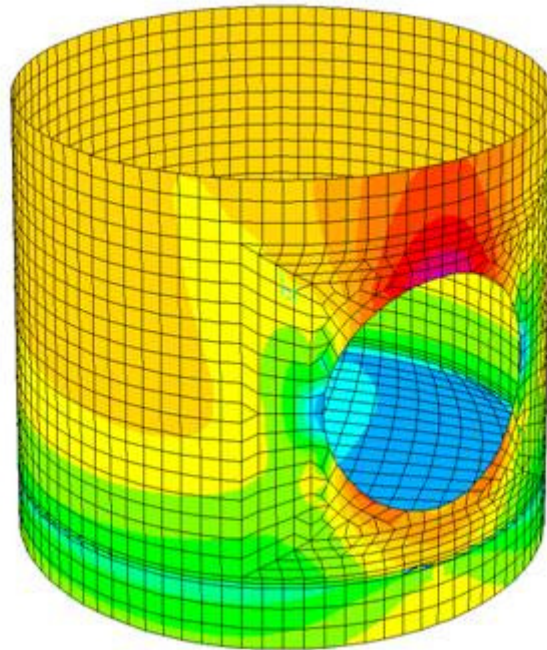


Figure 4 - Shell analysis of the reinforced collar

Grillage analysis was adopted for the structural analysis of the shaft base, which was modelled as a plate element and the supporting soil was modelled as elastic springs. The base slab is subjected to its self-weight, the self-weight of the secondary lining and the internal and external hydrostatic pressure.

GROUND MOVEMENT AND EXISTING BUILDING IMPACT ASSESSMENT

Studies were carried out to determine the impact of the ground movement induced by the tunnel and shaft excavation on structures located in close proximity to the tunnel alignment and to identify any remedial measures that needed to be undertaken prior to the tunnelling to ensure that the ground movement would not have an adverse impact on the existing infrastructure. A general analysis was undertaken to forecast the extent of the zone and magnitude of the ground movement induced by the tunnel excavation. The ground movement was predicted mainly based on the empirical approach, considering the ground movement at the surface takes the form of an inverted Gaussian curve. The influence zone was defined as 50 m from the centre of the alignment, at boundaries where the estimated green field settlement is almost zero. For the intake structures and shafts, the zone of influence for the braced open cut excavations was taken as two times the depth of excavation from the excavation boundary.

The combined settlement induced by the tunnels/shafts/intakes was examined to assess the effect on the adjacent structures and utilities. The settlement contours in Figure 5 show the magnitude and extent of the predicted ground settlement due to the construction (i.e. short-term) along and adjacent to the tunnel alignment.

A staged assessment approach for damage risk prediction was adopted for this project. The approach consists of a preliminary stage assessment, a secondary stage assessment and detailed design analysis. The surface settlement contours in the green-field condition were adopted for the preliminary stage assessment. Based on the maximum settlement and maximum slope of the ground at the ground surface, the possible degree of damage to the adjacent buildings was assessed according to Rankin (1988). The second stage assessment drew on the research by Burland et al. (1977). The buildings were assumed to act like equivalent beams and the degree of damage was assessed according to the tensile strain induced by the ground movement.

The second stage assessment was carried out on structures that were classified as being at the risk of slight or above damage in the first stage assessment, structures founded on friction piles, structures of particular sensitivity including those in poor condition or with shallow or unknown foundations, structures that contained sensitive equipment and all major forms of infrastructure.

The detailed design analysis was carried out on the structures that showed risk of slight damage or worse in the secondary assessment and the major forms of infrastructure that were sensitive to movement. The detailed analysis considered the features of the structures, TBM tunnels, cut and cover excavation schemes, the foundation system of the building, the building structural rigidity and continuity, the dimensions of the building and the soil-structure interaction. In this project, a detailed design analysis was carried out on the major important structures and structures that were considered most likely to be influenced by the tunnel construction, including the Lai Chi Kok interchange and the decked nullah.

DETAILED DESIGN ASSESSMENT

Staged approaches to predicting the ground movement and assessing the risk of structural damage were adopted for the preliminary and detailed construction risk assessment (CRA). The risk of damage to the existing Lai Chi Kok interchange and the decked nullah was classified as “very slight” and “negligible” in the first stage and second stage assessments, respectively. Nevertheless, given that they are major structures, a detailed assessment was carried out to ensure that the ground movement generated during the tunnel construction would have no adverse effects on the structures.

The Lai Chi Kok interchange and decked nullah are supported by friction piles and are located very close to the tunnel alignment. The Lai Chi Kok interchange is a viaduct structure that crosses the Main Tunnel alignment, which passes beneath the piles of the bridge. The decked nullah runs parallel with the Main Tunnel but unlike the viaduct, the tunnel was excavated in mixed ground.

Critical sections of the structures were selected for PLAXIS analysis based on the clearance between the Main Tunnel alignment and the existing foundations.

The PLAXIS model was based on a two-dimensional plan strain analysis and the piles were modelled as plate elements. This approach gives more conservative results, as the piles are assumed to be continuous and hence more adversely affected by the tunnel induced settlement. The basic steps of the analysis of the foundations included creating a computer model of the piles under the influence of tunnelling assuming a volume loss of 1%, extracting the additional bending moment and axial force from the analysis output and finally assessing the structural capacity and foundation capacity of the piles. It was found that the bending moment and axial compression forces generated on the pre-cast concrete piles of both the Lai Chi Kok interchange and the decked nullah were negligible and that there was sufficient capacity when compared with the bending stress generated by the allowable concrete stress. It was concluded that the bending and axial capacity of the pre-cast concrete piles would not be adversely affected by the Main Tunnel construction. The cross section of the tunnel and the Lai Chi Kok interchange and the corresponding PLAXIS model are shown in Figures 7 and 8.

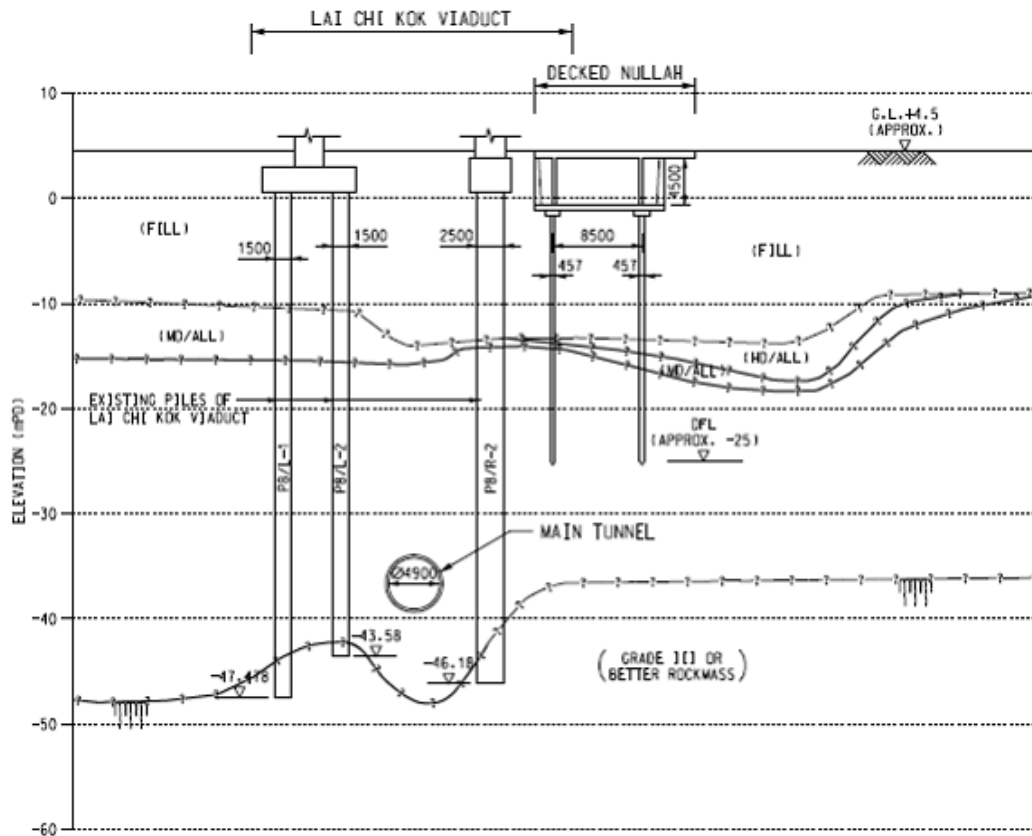


Figure 7 – Cross section of the Lai Chi Kok Interchange and the Main Tunnel

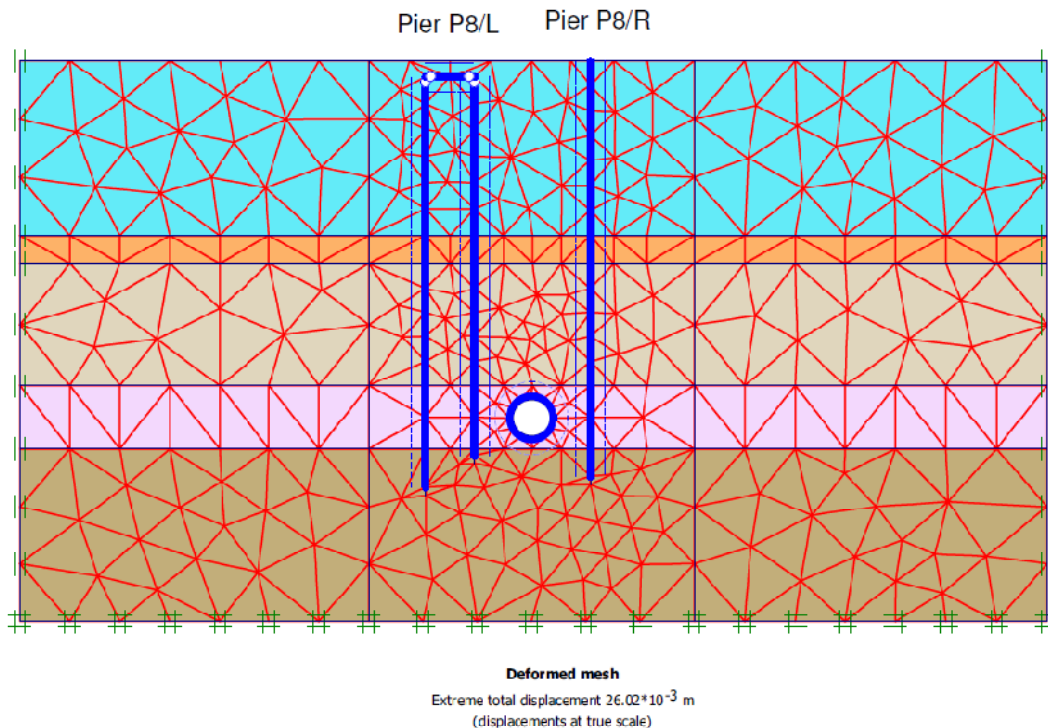


Figure 8 – FEM model of the Lai Chi Kok Interchange and the Main Tunnel

For the aboveground interchange structure, because the pile head is connected structurally and monolithically to the pile cap, the movement of the supporting piers will be equivalent to that of the pile cap. The settlement and horizontal movement of the pile heads were extracted from the PLAXIS analysis results and were used to estimate the effects of the tunnel construction on the piers, bridge deck and bearings of the existing Lai Chi Kok interchange and decked nullah structures. The results demonstrated that the tunnel would have a minimal impact and no adverse effects on the aboveground structures were anticipated.

CONCLUSION

This project involved numerous challenges including compressed air intervention at a hydrostatic pressure of 4.2 bar, the use of a mixed-shield TBM for both rock and mix ground tunnelling and urban tunnelling in close proximity to the foundations of buildings, viaducts, operating railway lines and culverts. The construction was completed successfully without any adverse effects on the existing structures and railway tunnels.

ACKNOWLEDGEMENTS

The authors would like to express their sincere thanks to the DSD, AECOM and Leighton-John Holland JV for their permission to publish this paper.

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