Design & Construction of Pedestrian Access Tunnels below an Existing Operational Metro Tunnels & Station

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Abstract:

A new underground station is added into Outram Park Station as a part of Thomson Line MRT/metro in Singapore and this is built next to two operating metro lines. Once the underground station is completed, it is required to connect this infrastructure with the platforms of existing metro lines for the passengers to interchange between the metro lines. This paper presents design and construction issues of two such pedestrian linkway access tunnels built for this purpose. Two tunnels with cross-sectional area ranging from 50m^2 to 85m^2 are excavated below an existing operational rail tunnel (with vertical clearance of less than 2.7m) and below an operational metro station. The tunnel excavation below the operational metro tunnel is carried out using pipe roof method and the tunnel excavation below the existing station base slab is carried out by underpinning the structure as the excavation progressed. This paper gives an insight into the design considerations, construction methodology and summarizes the learnings based on instrumentation and monitoring results.

Keywords: Pedestrian Linkway, Pipe jacking, excavation methods, underpinning

1. Introduction

With the construction of new underground public transport systems, there is a need to construct Linkway tunnels to integrate them with the existing transport network and to link them with accessible entrance and exit locations. Often, the preferred construction method for such Linkway tunnels are cut and cover constructions with temporary excavation retaining walls and strutting. However, in a densely populated area, this would require multi-staged traffic diversions and advance utility diversions. In some cases, the Linkway tunnels can also be below existing buildings making it impossible to carry out cut and cover construction method.

The pedestrian Linkway tunnels discussed in this paper are part of infrastructure connecting the new Outram Park Station in Thomson Line (TSL) to existing Outram Park stations in East-West Line (EWL) and North-East Line (NEL) metro routes. Linkway 1 is constructed underneath the existing live tunnels to provide access from NEL to TSL station structure. Linkway 2 is constructed underneath the existing station with linking access directly to the platform of EWL by escalators and staircase. Figure 1 and Figure 2 show schematic locations of Linkway 1 and Linkway 2 respectively.

Linkway 1 tunnel has to accommodate pedestrian Linkway of dimension 13.1m wide and 6.5m deep and this tunnel is constructed below operational EWL tunnels at around 45 degree angle with a minimum vertical clearance of 2.65m. The existing tunnels have an internal clear diameter of 5.3m, built with precast concrete segmental rings of 1.0m width and 225mm thickness. The rings are arranged in 5+1 key configuration with a C40 design concrete grade. The EWL tunnels (EB = East Bound, WB = West Bound) are almost at

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same level as the tunnels are close to the station. The rail levels are ~91.6mRL, which is approximately 1.548m below their respective tunnel axis.

a) Overview of Linkway 1 location (plan)

b) Plan of Linkway 1 location

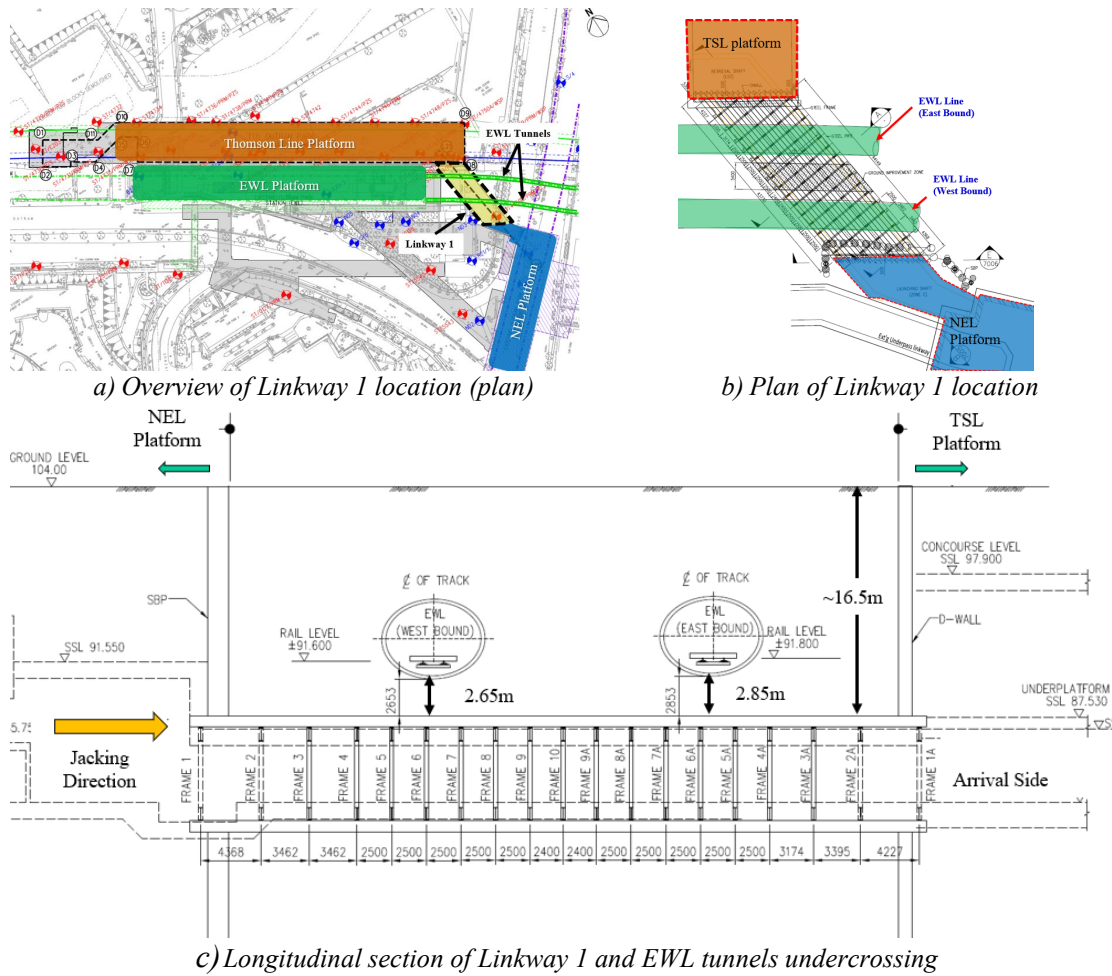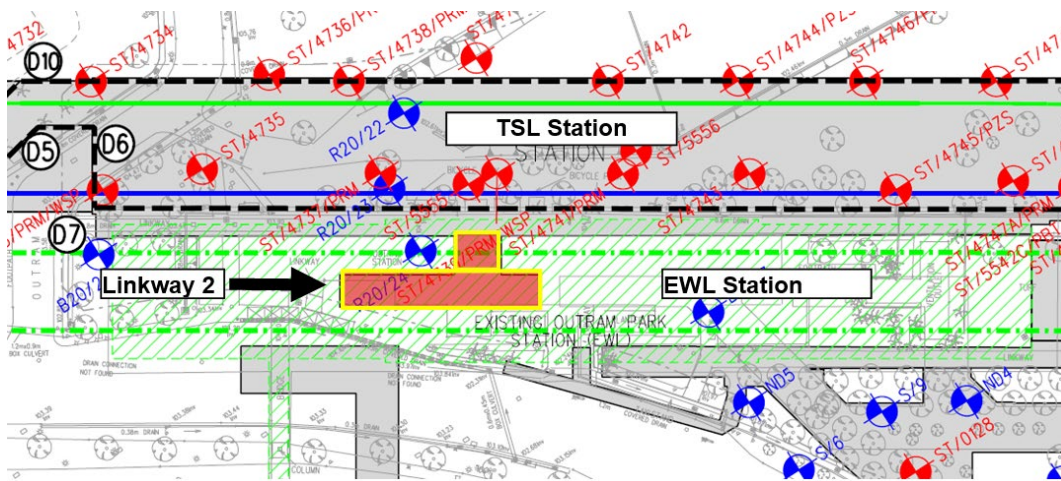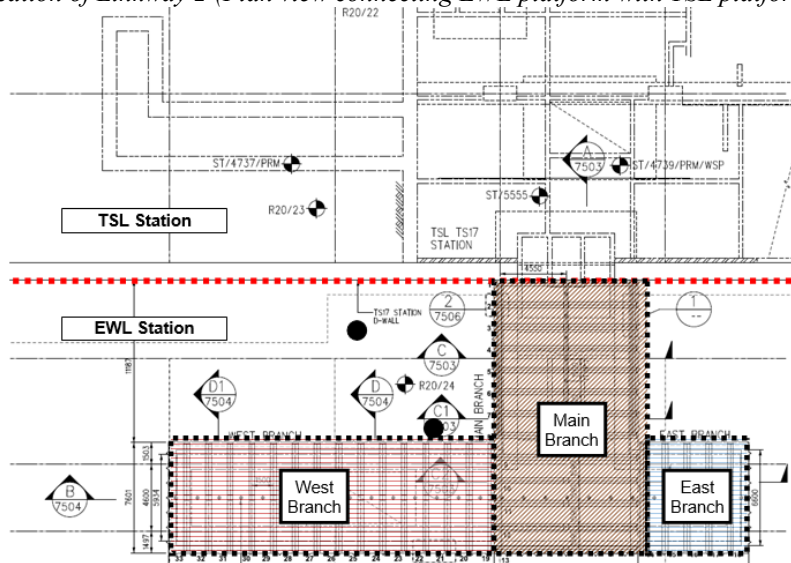*c) Longitudinal section of Linkway 1 and EWL tunnels undercrossing*
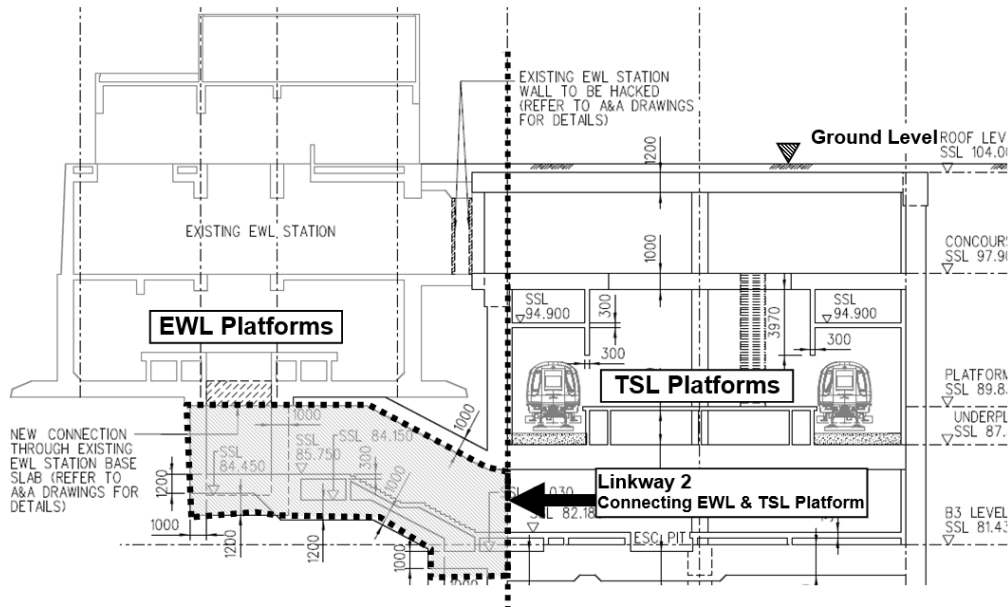
Fig 1. Schematic location of Linkway 1

Linkway 2 excavation is carried out below the existing EWL station to create an underground link between TSL platforms and EWL platforms. Due to high volume of passengers anticipated at this interchange, the passenger Linkway will be of 9.3m wide and 6.2m deep. The existing station is seated on a raft foundation which is mostly highly weathered rock of siltstone and mudstone (known locally as Jurong Formation with weathering grade ranging from SIV to SVI). Linkway 2 excavation will begin from TSL platform and progress towards the centre of EWL platform. Along the centre of EWL station slab, two side drift (east branch and left branch) tunnels are excavated to accommodate the staircase and escalator parallel to the platform. The central portion of the base slab with opening in the slab, is a thickened strip that extends the full length of the EWL station. This strip is 1.5m thick and includes two hidden reinforced concrete beams located within its depth. These beams in the raft were designed in 1990's to act as ground bearing strip footings to the internal station columns sitting exactly above this EWL central station slab.

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a) Location of Linkway 2 (Plan view connecting EWL platform with TSL platform)

b) Plan of Linkway 2: Main Branch and side branches (plan view)

c) *Longitudinal section of Linkway 2 (along main branch)*

Fig 2. Schematic location of Linkway 2

2. Geology and Ground Condition

Both the proposed Linkway tunnels will be excavated in Jurong formation. Nearest boreholes indicate that the tunnel excavation will encounter weathering grade SV and SIV. Few boreholes suggest slightly sandy / gravelly silt at the crown. During the soil investigation it was noted that weathered layers of sandstone in combination with water pressure could lead to instability of excavation. Within the first 3-4m from ground, there is a thin layer of fill material underlain by fluvial deposit and occasional pockets of organic clay. Below this is a completely weathered zone of Jurong formation. The properties improve generally with depth. The Jurong formation is of late Triassic to early Jurassic age and comprises interbedded sandstones, siltstones and mudstones that in location, dip steeply. The rock mass is dominated by the fractures and depending upon the condition of the fractures varied flow of water ingress are experienced. Under wet condition, the rock mass is found to deteriorate rapidly. Rock strength was variable ranging generally from 0.5 MPa to 5 MPa. For design purposes, following parameters were used.

Table 1: Design Parameters

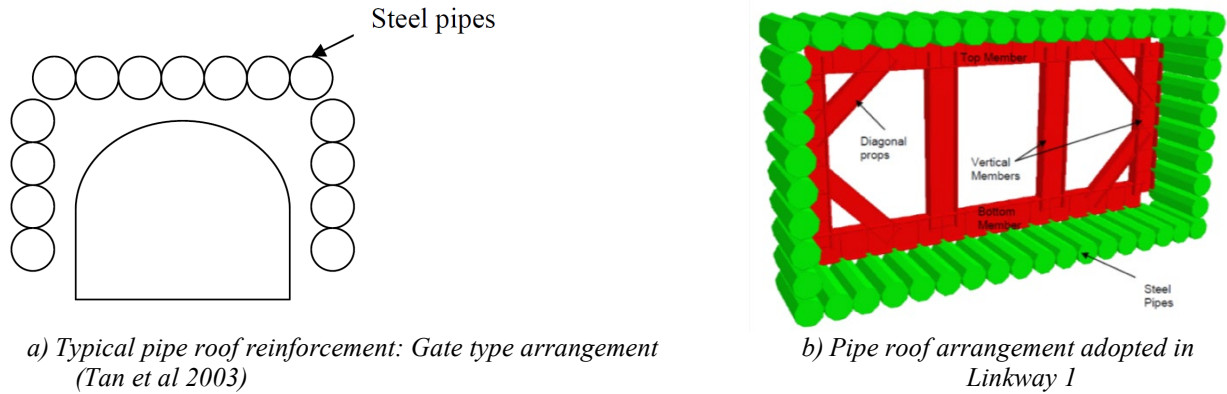|	Jurong Formation			
	SVI	SV	SIV	SIII
Unit weight [kN/m^3]	20	21	24	25
Effective angle of friction [degree]	31	33	33	36
Effective cohesion [kPa]	0	10	15	30
Undrained shear strength cu [kPa]	100	250	NA	NA
Young's Modulus E' [MPa]	33	208	250	400

Field permeability test indicated permeability in the range of 1×10^{-7} m/sec for the weathered Jurong Formation.

3. Design Concept

3.1 Linkway 1

Tunnel displacement causes deformation of the ground between the tunnel and the subsurface infrastructure and hence results in subsidence at the underground infrastructure. In this case, the clearance between the Linkway tunnel and the existing EWL tunnels is less than 3m hence there is a high risk of EWL tunnel damage if there is any face collapse during Linkway 1 excavation. To address this concern, based on experience in Singapore (Badurdeen and Senthilnath 2017), it was proposed to install pipe-jacked steel pipes to form a reinforcement layer between the EWL tunnel and the Linkway 1 excavation tunnel. This formed a high stiffness boundary and provided an advance face support to enhance the stability of the tunnel prior to tunnel excavation. The steel pipes of 0.813m diameter was proposed to be jacked over the projected tunnel crown to form a layer of reinforced layer which prevents the flow of soil into the tunnel at the periphery and at the face when the Linkway 1 excavation is in progress. Subsequently, steel rib/ steel frame supports are installed inside the Linkway tunnel at 2.5m spacing as the tunnel excavation progressed. Typical roof reinforcement with pipes is shown in figure 3(a). Lim et al. (2000) observed that around half of the settlement during the tunnel excavation in Jurong formation could

be attributed to elastic compression during water drawdown. Hence in our design scheme, we proposed a water tight system with interlocking pipes (provided with T-clutch joints) – shown in Figure 2 (b) and (c) and ground water recharge wells were planned along the excavation length to recharge the ground water drawdown due to excavation as a contingency measure.

a) Typical pipe roof reinforcement: Gate type arrangement (Tan et al 2003)

b) Pipe roof arrangement adopted in Linkway 1

c) Details of adopted pipe roof arrangement and steel frame supports

Fig 3. Proposed water-tight piperoof system

3.1.1 Installation of Pipe-Roof and tunnel excavation

Once all the interlocking pipes (52 nos) are jacked, first frame is installed within the shaft – (acting as a portal frame) and the excavation of tunnel was started within the box structure created by the interlocking pipes. The subsequent support frames are installed as the excavation progressed along the Linkway tunnel. The pipes at the roof level (pipe no. 1 to 18) act as a continuous beam supported at the frame locations. In intermediate stage of post-excavation (and just before the installation of frame), the roof level beams experience the maximum bending moment and deflection. The last installed frame experiences the maximum loading as the excavation proceeds. With the next frame installation, there was slight unloading of the previous frame. This was anticipated in the design and hence a staged excavation model was used to simulate the complete excavation cycle. Figure 4 presents typical bending moment distribution in the top pipe-roof in a given stage.

Stage 8 (Just before installation of Frame 7)   Pipe No. 1 (middle pipe of linkway)

Max: 838.515 kNm   Max: 837.994 kNm

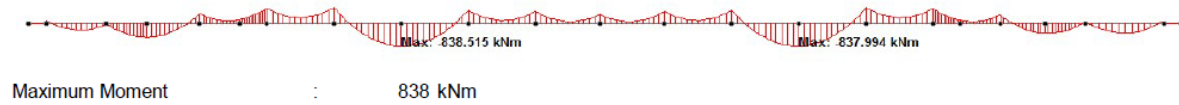Maximum Moment   :   838 kNm

Fig 4. Typical bending moment distribution in the roof pipe

The usual approach of Rabchiwich / NATM is to utilize the strength of the soil. But to utilize the strength, we need to mobilize some movement in the ground. However, in our case, the allowable limits are very strict hence the member sizes are relatively heavy and also happen to take more loads as stiffer elements tend to attract forces.

3.1.2 Impact on existing tunnels during tunnel excavation

To assess the ground deformation due to Linkway 1 excavation, a 3D model numerical analysis was carried out. Since the analysis was on a 3D model, no ground relaxation was applied prior to any excavation step or activation of ground support elements. The model includes simulation of each excavation step and subsequent installation of steel frames. To arrive at reasonably conservative estimate of construction impact on existing tunnel, the ground support of the excavation face in Linkway 1 (such as glass fibre face dowels, sprayed concrete, etc) was not considered in the analysis model. The analysis was carried out with an assumption that the Linkway 1 excavation will be carried out from both sides of the tunnel however during the final stages, as the unexcavated soil between the either face of excavation is reduced, the construction sequence is simulated (and was proposed in design drawings) for excavation from one side. Figure 5 presents the numerical model and visualization of frame location / unexcavated soil during the final rounds of excavation.
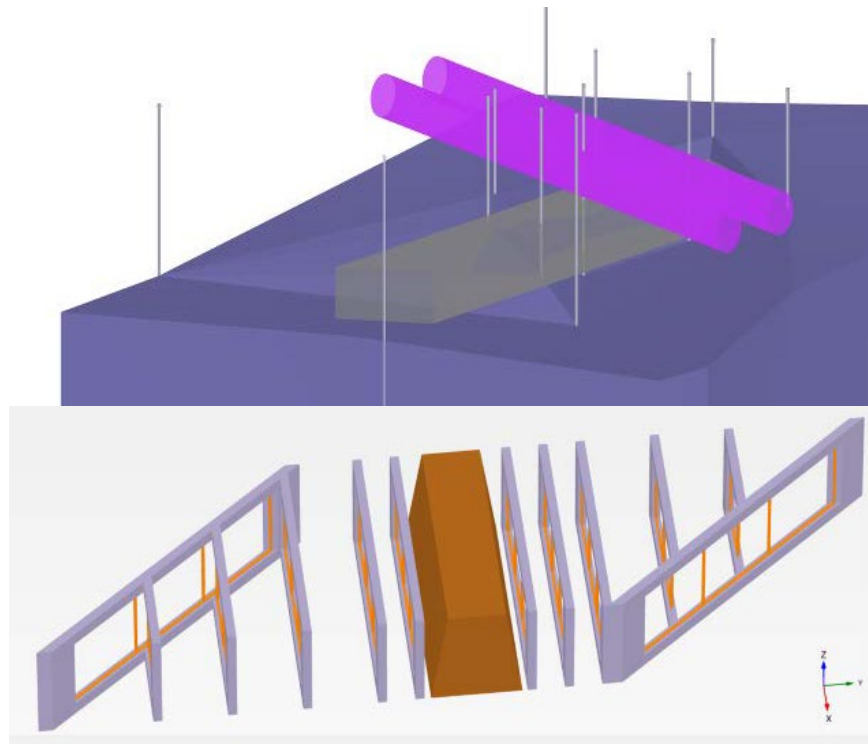
Fig 5. Numerical model and excavation sequence visualization

In addition to assessing the ground displacement due to Linkway 1 excavation, it was important to understand the behaviour of the tunnels and change in diameter of tunnel (if any) due to Linkway 1 excavation. Following structural limit states were studied in this project to ensure that the existing tunnel linings are within structural limits.

- Ovalisation: To check distortion involving increase in vertical diameter and decrease in horizontal diameter. This causes increase in bending moments in the concrete segments and causes rotation at the segment joints.

- Squat: To check distortion in EWL tunnel segments involving increase in horizontal diameter increase and decrease in vertical diameter. The effects are like ovalisation. However, in this case, the tunnels had already squatted when taking up the ground loads. Therefore, the remaining tolerable movement was the difference between the existing squat and the tolerable squat.

- Stepping & Opening of Circular Joint: In the longitudinal profile, the tunnel tends to have two types of deformation modes a) Bending mode of deformation and b) shear mode of deformation. Based on the type of behaviour, it is possible to have either opening of joints or stepping between the segments.

Figure 6 illustrates the different structural limit states described above for damage / impact assessment of existing tunnels.
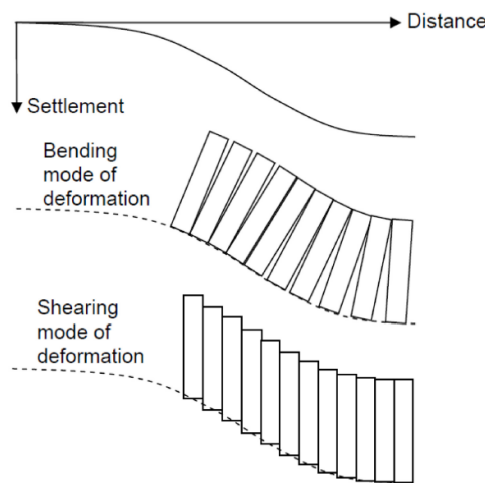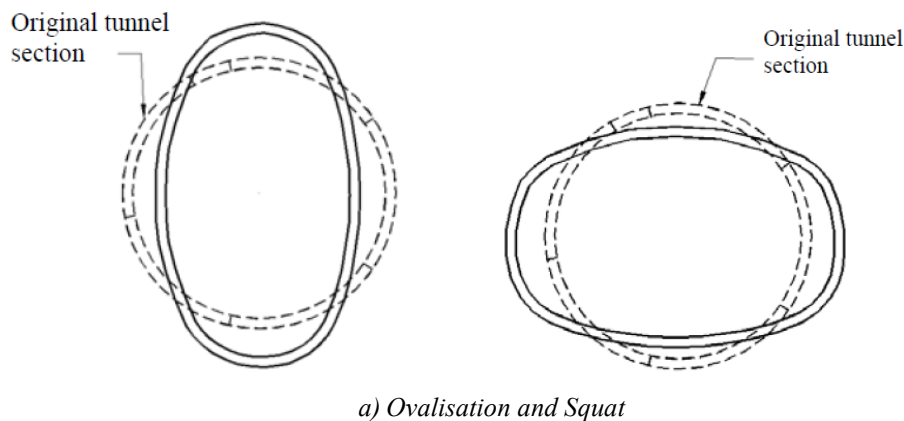

a) Ovalisation and Squat

b) Stepping and opening of circular joint

Fig 6. Structural Limit states considered for existing tunnels

3.1.3 Jacking force

Minimum jacking force is calculated based on minimum required force to overcome the skin friction along the pipe contact surface area and maximum jacking force is calculated based on pressure transmission along the steel pipe and joint configuration between the pipes using ATV nomograms (Röhner et al 2010). Figure 7 illustrates the components considered for estimation of jacking force.
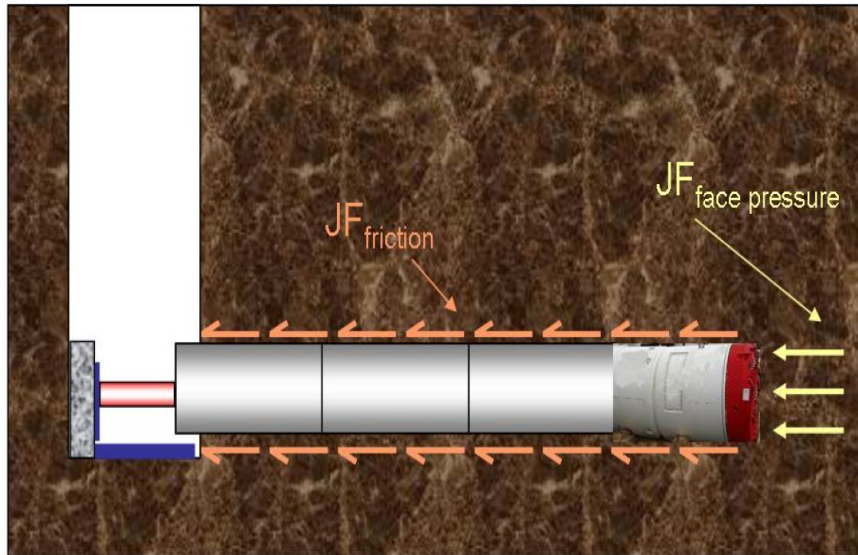

Fig 7. Jacking force estimation

3.1.4 Factors influencing face stability during pipe jacking

Face stability calculations are required to determine the likelihood of the excavated face moving or collapsing into the void created during excavation. Face stability depends upon the type and variability of the ground being excavated, the ambient stress and ground water conditions, the rate of advancement and the construction methods adopted. The methods for calculating stability of tunnel faces are generally well-established and are adjusted for pipejacking / microtunnelling in terms of rate of advance, size of face and face support conditions. Face pressure of 1.2 bar to 2.2 bar is recommended based on the soil condition at face and the depth of the pipe.

3.1.5 Machinery & Equipment

Because of the size of the support frames, the clearance between the vertical members had to be carefully checked for excavation, turning and reversing of the excavators along the Linkways. During the excavation, the diagonal member in the support frames were sequenced to allow the movement of vehicles. Figure 8 shows the tight site constraints faced during the excavation.

Fig 8. Site constraint for movement of excavators

To ensure proper contact between the frames and the pipe-roof, grout bags above the frames were included as design element. In addition, jacking up of the frame using hydraulic jacks and temporary additional vertical members were carried out. The frames are jacked up and shim plates are installed in the gap created by jacking and the bolts are tightened and sealed. The temporary additional vertical members are reused for next frame jacking.

3.1.6 Monitoring and instrumentation of Linkway 1

To clearly understand the actual ground response during Linkway 1 excavation, a series of real time prism targets were installed within the operational tunnels and were monitored by automated robotic total stations installed within the tunnel. Each monitoring array consists of five prism targets (as indicated in Figure 9) to understand the movement of tunnel as well as to understand deformation in the tunnel shape (if any).
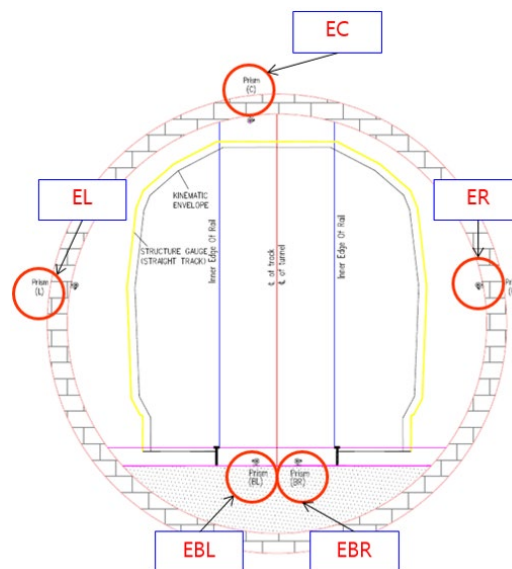

Fig 9. Prism monitoring locations

An automated site specific data visualization tool as proposed by Senthilnath G T (2017) was developed for this project to represent the change in shape and longitudinal profile

of the tunnel as the Linkway1 tunnel excavation proceeds. The data visualization tool indicated uniform settlement of all the points (in a given cross section) which indicates there was no ovalization or squatting of the tunnel. Hence longitudinal profile settlement (i.e stepping and opening of circumferential joints) was critical during the Linkway 1 excavation. Figure 10 (a) and (b) presents typical prism readings in cross-section and longitudinal section of the tunnel alignment.
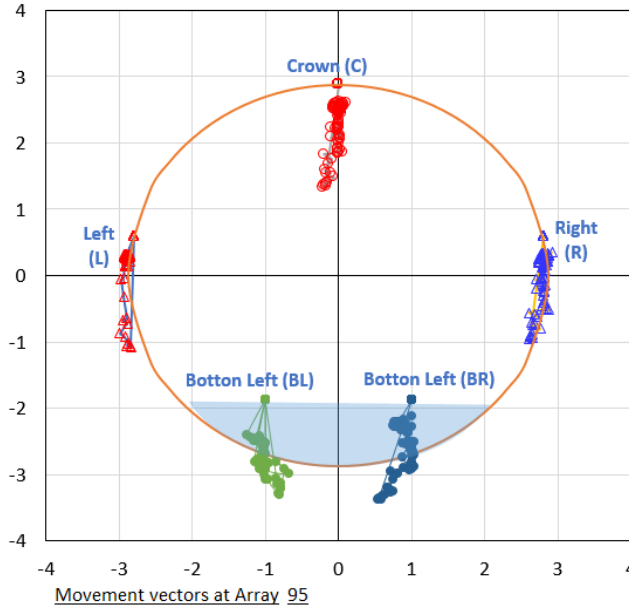
(a) Tunnel displacement in cross section
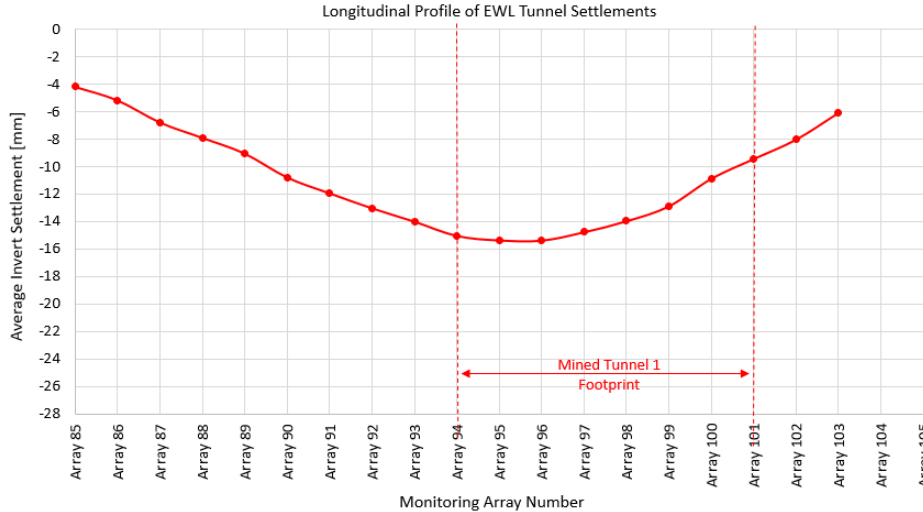
(b) Tunnel displacement in longitudinal section

Fig 10. Tunnel movement visualization

3.2 Linkway 2

As shown in Figure 11, the Linkway 2 excavation starts from the TSL platform box (by creating an opening in the D-wall) and the excavation proceeds below the existing EWL station base slab. Excavation was planned with steel support frames installed to underpin the base slab as the excavation progressed. However, as the excavation height is more

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than 10m, the excavation face is divided into top heading and bench excavation. The load transfer of underpinning members during bench excavation is ensured using a truss arrangement as shown in Figure 12 (the truss arrangement is not shown in the 3d render for clarity) The steel frames along with shotcrete sprayed on side walls and invert resist the lateral earth pressure and uplift pressure from the invert in Linkway 2 in the temporary stage. To check the composite behaviour of steel sets with shotcrete lining, the forces from the analysis are cross checked against the capacity curve derived based on Carranza-Torres and Diederichs (2009). For simplicity, contribution of wire mesh in the bending capacity of the shotcrete lining is not considered. The forces obtained from composite – unit width model is split between Steel frame and shotcrete based on their EI and EA ratio for the design capacity checks.
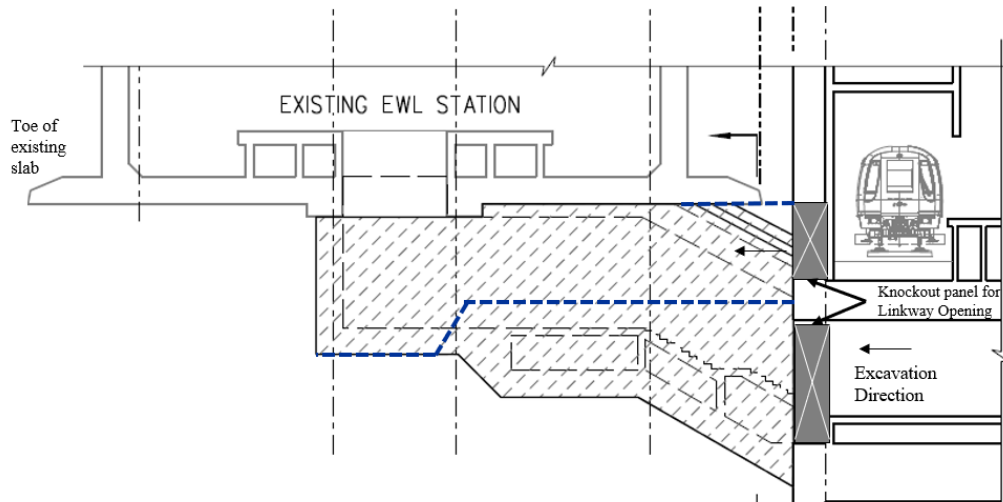

Fig 11. Excavation of main branch of Linkway 2
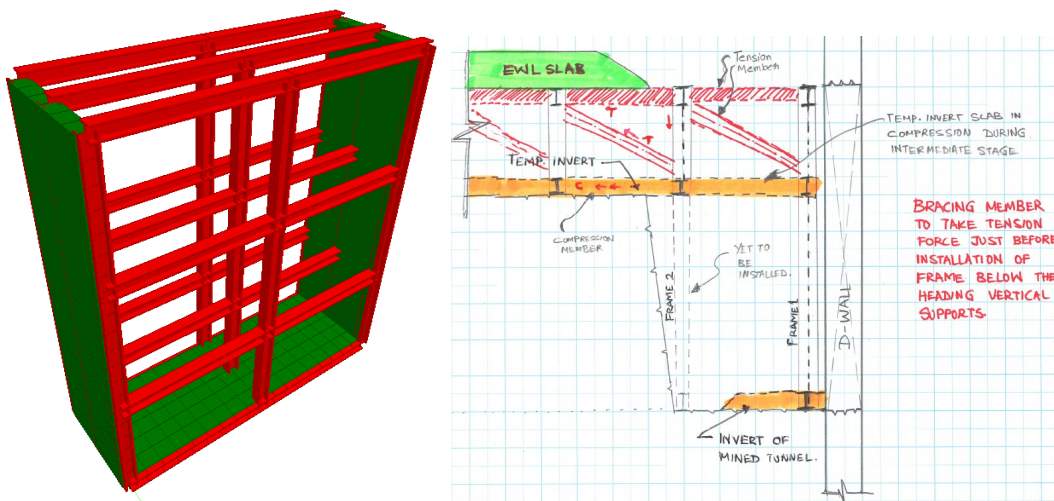

Fig 12. 3D render of underpinning system and load transfer system in truss member

3.2.1 Impact on station slab

The Linkway 2 excavation is carried out below EWL structure which was built in 1980s. The structure was designed using the former British code of practice CP110 which was later superseded by BS 8110. Despite this, current design checks in the EWL slab due to

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Linkway excavation was carried out as per the latest BS 8110. To understand the impact of Linkway construction on the slab, it was necessary to understand the in-situ stresses in the structural members of the EWL station. For this purpose, the client had used Autodesk Robot to create 3D Finite Element (FE) models of the existing EWL station. This model is used to understand the existing condition at various reference sections along the Linkway tunnel.

The additional stresses due to Linkway 2 excavation is estimated based on free span and column loadings on the slab. In addition, to limit the centre stress in the slab, an intermediate vertical support is proposed for the main branch of excavation in temporary case. A 2D PLAXIS analysis is carried out to estimate the maximum ground displacement and slab movement due to mining activity. Without any face support, for 1.5m round length and 7m wide excavation, ahead-of-face ground relaxation is estimated to be in the range of 50 to 55% (based on Panet and Sakurai formulation). However, because of face bolting, ground relaxation of 30% is considered in the FE analysis. Maximum EWL slab displacement is estimated to be less than 5mm.

3.2.2 Face Support and Grouting

Design of tunnel face reinforcement is made to limit the ahead of face ground relaxation. The tunnel face reinforcement ensures that the unsupported span of EWL slab is maintained to a minimum length of less than 2.5m (1.5m round length + 1.0m allowance for slope/ground relaxation zone). Stability of face is based on limit equilibrium of a failure mechanism that consists of a wedge and a prism. Mohr-Coulomb failure condition is assumed for the ground. Inclination of the slip plane depend on the shear strength and on the stratigraphy of the ground and has been determined iteratively. The slip plane which results in maximum support requirements is considered for the face support requirements. During the ongoing excavation (without anchorage plates), the supporting effect of the face bolts relies solely on the strength of the bond between grout and soil. If anchorage plates are present (during an excavation standstill) and the bond length inside the wedge is insufficient, the bolt force will be transmitted to the ground through the plates. Minimum number of face support bolts are calculated based on limit equilibrium check and design based on nomograms by Anagnostou et al (2007).

3.2.3 Monitoring and instrumentation of Linkway 2

To monitor the ground performance during Linkway 2 excavation, a series of settlement markers were installed on the EWL base slab and in addition, strain gauges were installed on the alternate support frames to measure the axial loads coming on the steel sets. Except for the T-Junction (between the main branch and the side branches) axial loads in all the support frames were well within the prediction. This could be attributed to conservative approach used for loading consideration on the EWL slab to avoid utilizing the EWL slabs to their structural limits.

4. Discussions and Conclusion

Experience gained during the construction of the Linkway confirmed that if dry conditions are maintained, the Jurong formation (Sandstone, siltstone and mudstone with varying degrees of weathering) is a good material for tunnel excavation with advance roof support. From the observation of the ground performance during tunnelling, it became very clear that Jurong formation with high SPT blow counts is fairly incompressible and as anticipated, it experienced limited plastic settlements during the load redistribution in the

ground. The actual ground movements due to pipe roof installation slightly exceeded the design prediction however the measured settlement during the Linkway excavation agree well with the prediction. The instrument readings suggested that influence of Linkway 1 excavation had a temporal variance as the excavation progressed. This suggests that a detailed analysis of response at each stage needs to be studied for a project of similar nature. The final ground response is not necessarily the most critical movement during the Linkway 1 excavation.

Currently temporary works for both the Linkway tunnels are completed and permanent works RC casting is in progress. For the successful completion of construction activities within the railway protection zone, very detailed sequence planning, periodical interface meeting with other designers and full compliance with requirements of the Code of Practice for Railway Protection works and/or railway operator was of utmost importance. This has ensured minimum disruption to the operation of the railway system.

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