

Construction of Underground Linkway using 3-Sided Pipe Roof Method at Keppel Station, Singapore

Terence Ong¹, Joo King Tay², Sai Sai Chen³

¹Arup Singapore Pte Ltd, Singapore, 069547; terence-ls.ong@arup.com

²Arup Singapore Pte Ltd, Singapore, 069547; joo-king.tay@arup.com

³China State Construction Engineering Corporation limited Singapore Branch, 069547; chensaisai@chinaconstruction.com.sg

*Correspondence: terence-ls.ong@arup.com

SUBMITTED 11 July 2025 REVISED 12 August 2025 ACCEPTED 25 August 2025

ABSTRACT Singapore is one of the most densely populated countries in the world. In this land-scarce city-state, public transport plays a crucial role in reducing road congestion, with the Mass Rapid Transit (MRT) system serving as a key component. The Circle Line (CCL) is one of the major lines within the MRT network, designed to connect key points across the island and improve overall transit efficiency. To complete the orbital loop of CCL, three new stations (Keppel, Cantonment and Prince Edward) between Harbour Front and Marina Bay stations are planned. However, the area around the three stations is heavily developed, posing significant challenges for underground construction. This paper presents the construction of a 52 m-long underground linkway at Keppel Station. This underground linkway was proposed to connect one of the entrances to the station's main box. This underground linkway was constructed beneath a live 9-lane carriageway and above two operational MRT tunnels, all of which remained fully functional throughout the construction process. Due to the presence of the carriageway which could not be diverted, a trenchless construction method known as the pipe roof method was adopted instead of the conventional cut-and-cover technique. The method employed a three-sided pipe roof along with ground improvement at the base of the excavation rather than the typical full enclosure pipe box system. This paper outlines the key design considerations and construction challenges encountered. It also discusses the observed performance of the mined excavation and compares it with predictions from a two-dimensional finite element analysis (FEA). The successful application of this design methodology provides a valuable reference for similar underground linkway construction in dense urban environments.

KEYWORDS Underground Linkway; Non-Full Enclosure Pipe Roof Method; Finite Element Analysis

1 INTRODUCTION

In heavily urbanized cities, public transport plays a crucial role in easing traffic congestion. In Singapore, one of the public transport infrastructures is the Mass Rapid Transit (MRT). As of 2024, the MRT lines cover a length of 242.6 km with six operational lines, including the North South, East West, North East, Circle, Downtown and Thomson East Coast Lines, with 143 stations in service. Two new lines (the Jurong Region Line and Cross Island Line), plus various extensions, are under construction and will add 44 more stations which result in increasing total route length to approximately 460 km by 2040. Except for Bishan MRT station which is partly at-grade, the rest of the MRT stations are either elevated or underground.

For underground MRT station construction in Singapore, traditional cut-and-cover methods are often impractical due to the difficulty of implementing extensive temporary traffic diversions. Instead, trenchless techniques that minimize disturbance to existing operations are required (Mckim, 1997). Trenchless construction method such as tunnelling is commonly adopted which in itself consist of various methods, including open face construction, New Austrian Tunneling Method (NATM), drill and blast tunnelling etc. (Chapman et al., 2017). Other trenchless technology includes box jacking for entrances at Thomson-East Coast line Havelock MRT Station (Ng et al., 2019), tunnelling

underneath the 8-lane Jakarta-Bogor highway (Prakoso and Lase, 2014; Prakoso and Sabbah, 2016) and pipe jacking (Qiu et al. 2025). This paper focuses specifically on the pipe-roof method implemented for Keppel Station's underground linkway construction.

The pipe-roofing method employs a micro-tunnel boring machine to install interlocking steel pipes, typically arranged in a rectangular "pipe-box" configuration. These pipes form an enclosed support system, allowing subsequent excavation and in situ casting to proceed safely (Woon et al. 2022). The pipe roof method has been used in a few projects in Singapore (Teo et al., 2016; Ng et al., 2016; Ng et al., 2017). Teo et al. (2016) presented a project in which pipe-roof method was used to construct a 14.5 m wide, 9.1 m high, 96 m long two-lane piped box tunnel located 6 m below the ground surface. The pipe roof was constructed in a rectangular box configuration, and the maximum surface settlement recorded was slightly more than 40 mm. Woon et al. (2022) conducted a 3D finite element analysis on the said project and obtain results which were in good agreement with the measured settlement. Ng et al. (2016) shared a few projects which used the pipe roofing method. The first project discussed was pipe box tunnel being used for two crossings around Bugis MRT station. For the Queen Street crossing, 50 nos. of 900-mm diameter pipes were used to construct a 15.2 m wide and 7.5 m high box tunnel. As for the Beach Road crossing, 66 nos. of 900-mm pipes were used to construct a 22 m wide and 7.5 m high box tunnel. Another project discussed is the Sentosa Gateway Tunnel which crosses the intersection of Kampong Bahru Road and Telok Blangah Road. Pipe box tunnel consisting of 44 nos. of pipes with 813 mm diameter were used to form a 9.4 m wide and 7.6 m high box structure. Lastly, the authors shared the use of pipe box tunnel for an 82 m long pedestrian underpass for the connection of Great World MRT Station to Great World City Mall. In this project, 42 nos. of 813-mm diameter pipes were used to construct a 10.9 m wide and 8.4 m high box tunnel.

Traditionally, the trenchless pipe roof method entails a full enclosure of the piped box tunnel. However, in a bid to innovate while considering cost-effectiveness, the underground linkway at Keppel Station discussed in this paper implements a unique approach: a three-sided pipe roof system with an open grouted base. This unconventional strategy not only addresses the project's specific challenges but also offers valuable insights into the application of this method.

2 KEPPEL STATION UNDERGROUND LINKWAY

2.1 Project Description

Keppel MRT station (Station CC30) is a part of Singapore's Circle Line Stage 6 (CCL6) along with 2 other stations, Cantonment station (Station CC31) and Prince Edward station (Station CC32). The three stations with a total route of 4 km will complete the Circle Line loop by connecting HarbourFront MRT station to Marina Bay MRT station (Figure 1). Keppel MRT station is a two-level underground station located next to the West Coast Highway, adjacent to PSA Corporation Limited's Tanjong Pagar Terminal and Keppel Terminal to the South, and Keppel Distripark to the North (see Figure 2).

The Keppel Station features three entrances, one of which is located approximately 55 m away from the station main box, across the West Coast Highway. The station main box is connected to this entrance via a 52 m underground linkway as shown in Figure 3. Given that the West Coast Highway is a 9-lane carriageway with numerous utilities, traditional cut-and-cover method was deemed too disruptive, hence impractical. As aforementioned, the trenchless pipe roof method was selected for linkway construction.

Following a thorough study of the site's geological conditions and the pipe roofing sequence, a 3-sided pipe roof system was proposed as an alternative to the conventional full enclosure pipe roof method. This design incorporated 1.5 m thick ground improvement at the base of the linkway using Tube-a-Manchette (TAM) grouting, aiming to reduce soil permeability at the base, as illustrated in Figure 4. The details of TAM grouting are discussed by Chua (2022).

A crucial consideration in determining the minimum diameter of the steel pipe is ensuring sufficient space for site personnel to enter and manually remove potential obstructions, such as boulders. As a result, the steel pipe diameter selected was 1.2 m. To enable site personnel to remove obstructions, a retrievable micro Tunnel Boring Machine (mTBM) was employed for pipe roof installation. A total of 19 pipes were used to construct the 3-sided pipe roof system.

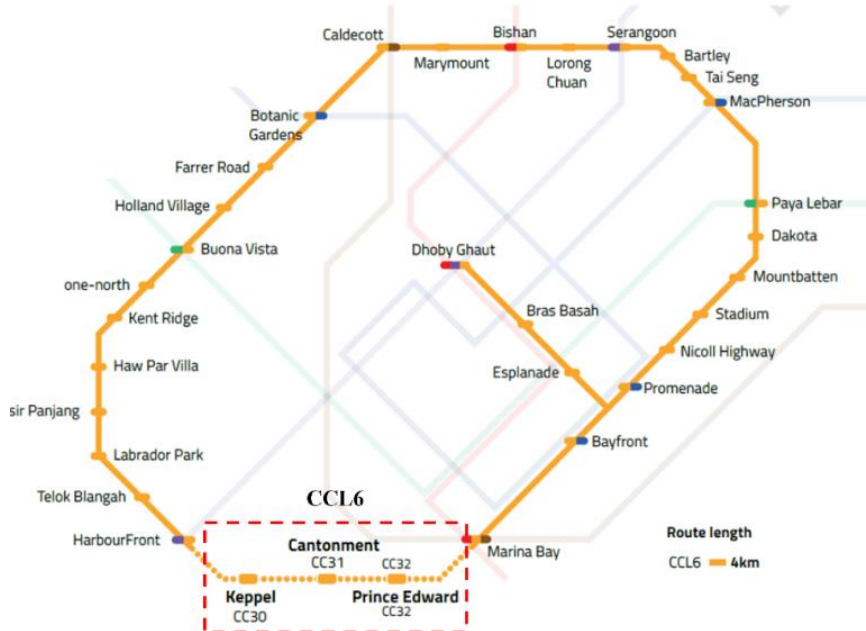


Figure 1. Singapore’s MRT Circle Line Stage 6 Extension (CCL6)



Figure 2. Location of Keppel Station

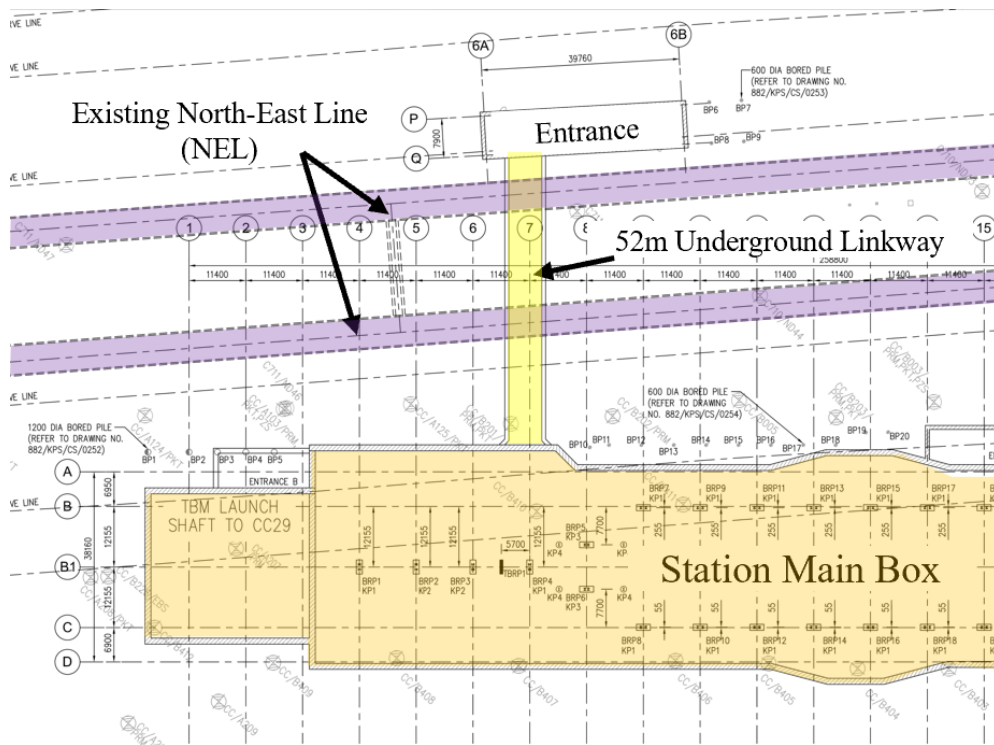


Figure 3. Detail site plan for underground linkway

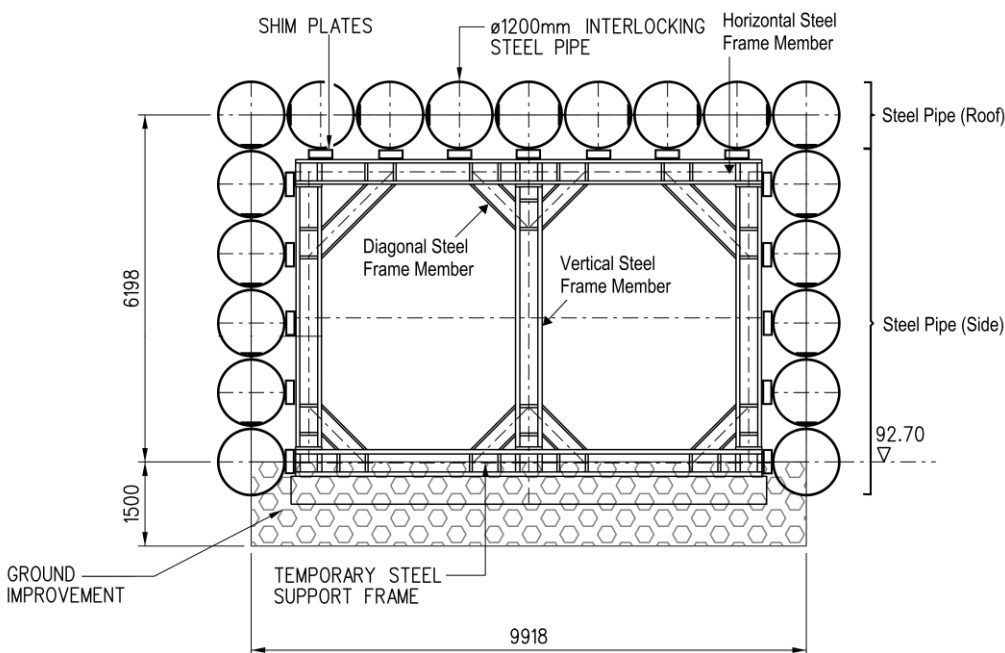


Figure 4. Cross section of proposed 3-sided pipe roof system for underground linkway construction

2.2 Site and Ground Conditions

The proposed 52 m long linkway, connecting the entrance to station main box, is 9.9 m wide and 6.2 m high. It runs beneath the West Coast Highway, perpendicular to the existing North-East Line (NEL) twin tunnels, which are located 10.2 m below the base of the linkway to be constructed.

The ground level at the launch shaft is approximately 104.0 mRL, with the final excavation level (FEL) for the linkway at 92.945 mRL, approximately 11 m below ground level. The ground consists

of a 2 to 3 m thick man-made fill overlying completely weathered Jurong Formation S(V), with SPT-N values ranging from 17 to 100. The soil primarily consists of slightly fine sandy silt and silty sand with rock fragments. Figure 5 shows the geological profile for the Keppel Station's underground linkway to be built. The geotechnical parameters used for geotechnical analysis and design checks are listed in Table 1. A more detailed view of the geological profile is given in Figure 7a.

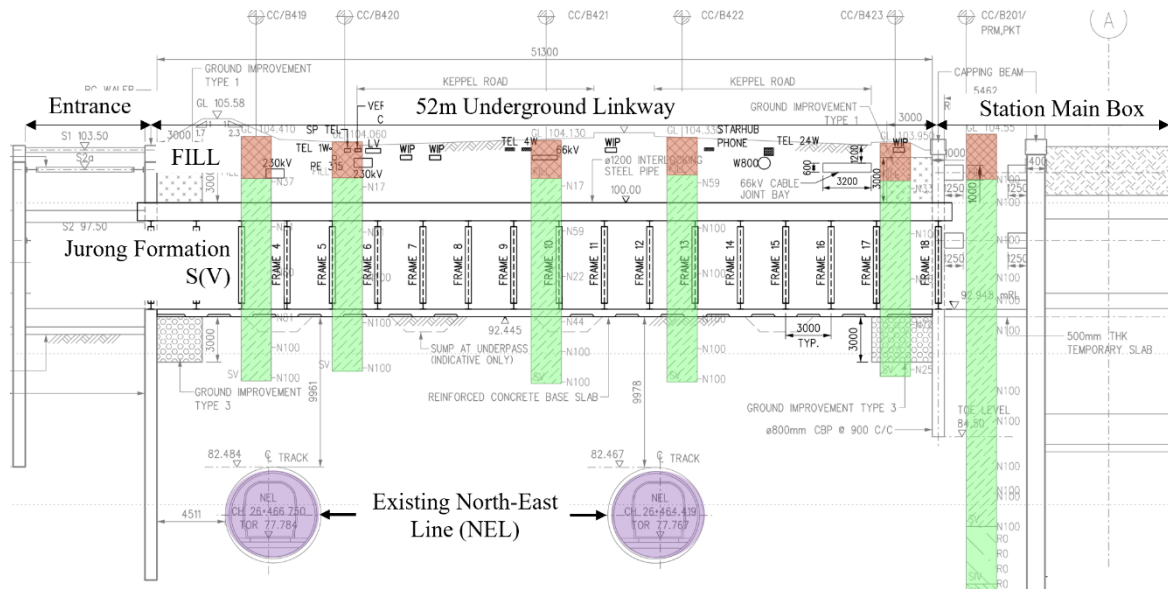


Figure 5. Geological profile

Table 1. Parameters used for the analysis

Soil Type	Unit Weight kN/m ³	Strength and Modulus Parameters			
		c_u kN/m ²	c' kN/m ²	ϕ' degrees	E' MPa
Fill	20	30	0	30	8.7
S(V) (SPT N ≤ 50)	21	5N ≤ 250	5	33	2N ≤ 300
S(V) (SPT N > 50)	21	250	10	34	2N ≤ 300

2.3 Construction Sequence

The construction sequence of the linkway are as follows:

1. Launch micro tunnel boring machine (mTBM) from the entrance to install interlocked steel pipes, forming a three-sided pipe box. The mTBM docks at the station main box at Keppel MRT station to retrieve the boring machine. A total of 19 steel pipes were installed.
2. Ground improvement using horizontal TAM grouting to improve the soil within the pipe box to mitigate water ingress and ensuring excavation face stability. The area of improvement is limited to soil with SPT-N less than 40 within the pipe box and soil below the final excavation level.
3. Erect steel frames supporting both ends of the linkway before excavation to minimize settlement during excavation.
4. Excavate from the entrance towards the station main box with a 1(H):2.5(V) slope and apply lean concrete to protect the slope.
5. Cast a temporary slab and install the steel frame.
6. Steps 4 and 5 were repeated to complete the excavation to the other end. Each excavation advancement was 3 m in two stages (Figure 6), starting from headings 1a, 1b, 3a, 3b, 5a, and 5b and followed by benching 2a, 2b, 4a, 4b, 6a, and 6b.
7. Upon completion of excavation work, steel frames were removed sequentially to cast the reinforced concrete structure of the linkway.

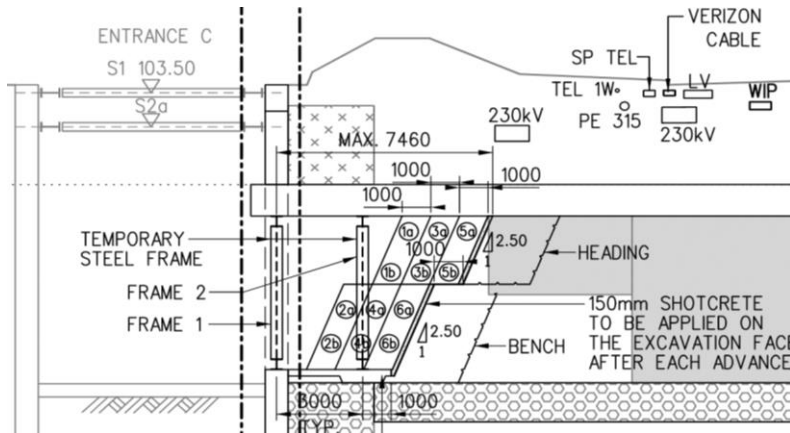


Figure 6. Typical construction sequence

3 DESIGN METHODOLOGY

3.1 Finite-Element Analysis

The two-dimensional finite element method (2D FEM) using PLAXIS was employed to model the underground linkway in the longitudinal section. The steel frames were modeled using node-to-node elements, and the temporary base slab was modeled as an impermeable plate. A surcharge of 25 kPa was applied at the ground surface to account for traffic loading. Given the plane-strain conditions, PLAXIS 2D calculated only the forces on the roof (top side) steel pipes and vertical loads for steel frame design (see Figure 7).

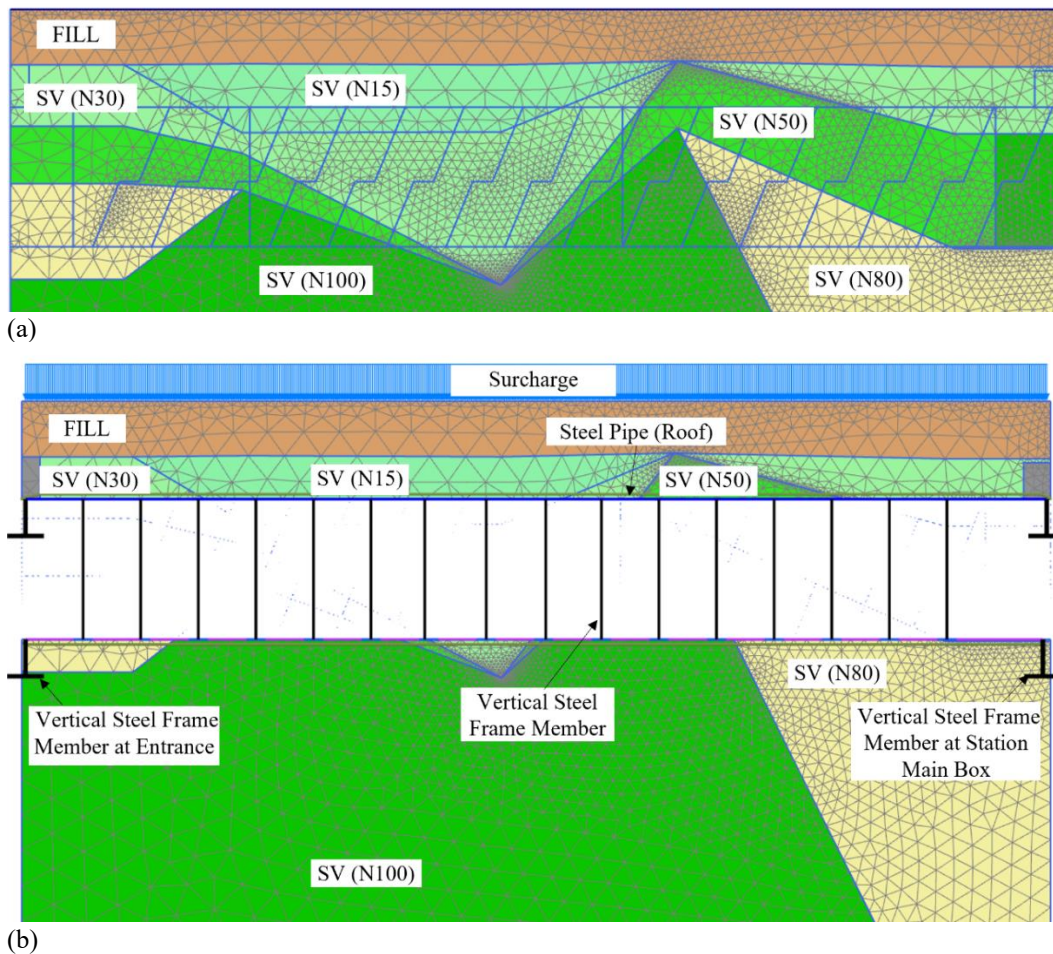


Figure 7. Finite element model: (a) without the structural elements; (b) with the structural elements.

Since the lateral pressure acting on the side of the linkway was found to be lower than the overburden pressure, the top side pipe forces obtained from the 2D PLAXIS model were applied for the design of the side pipes. To determine the horizontal loading for steel frame design, it was assumed that the tributary length for load transfer is identical in both vertical and horizontal directions. This assumption was considered conservative as the tributary length for horizontal loading is expected to be smaller than that for vertical loading, given that the longest unsupported length of the pipes occurs at the top of the slope between the sloping excavation face and steel frame (see Figure 8).

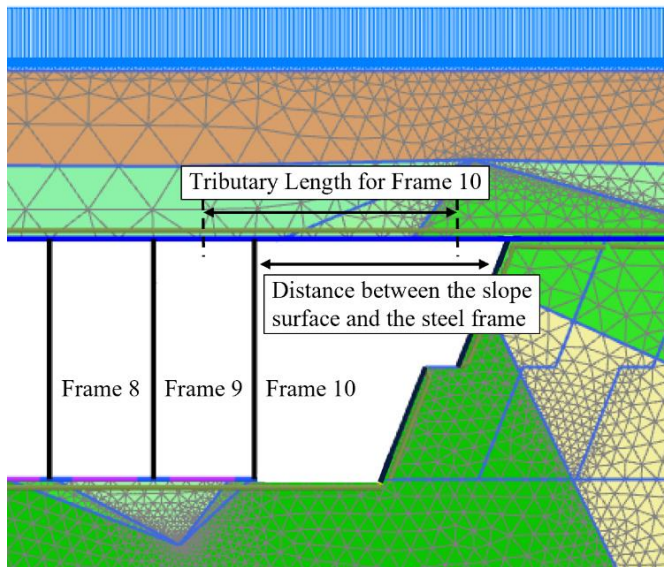


Figure 8. Example of tributary length derivation.

The tributary length (L) was calculated using the following equation.

$$L = \text{Vertical loading on steel frame (node-to-node force in 2D PLAXIS)} / \text{Overburden per meter run along the linkway}$$

Subsequently, the lateral pressures from soil, water, and surcharge (Figure 9) were multiplied by the tributary length to obtain the horizontal loading per frame, as expressed in following equation.

$$\text{Horizontal loading per frame} = \text{Lateral pressure per meter run along the linkway} \times L$$

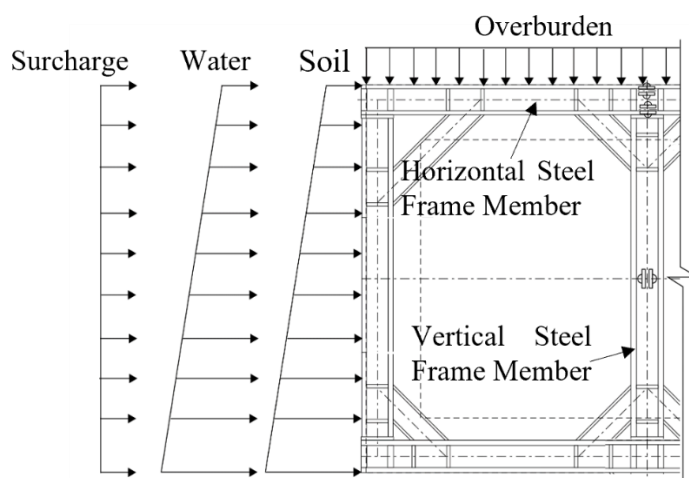


Figure 9. Horizontal and vertical pressures acting on the frame.

3.2 Structural Material Properties

The structural elements considered in the finite element analysis (FEA) i.e. steel pipe (roof) and temporary concrete base slab are idealized as elastic-plate elements with the bending stiffness (EI) and axial stiffness (EA) calculated based on the geometric shape of the element cross section and the material type. The temporary steel frame designed to support steel pipe roof during the excavation are modelled as node-to-node anchor element. The adopted design properties of the structural elements in geotechnical FEA are tabulated in Table 1.

Table 1 Design Properties of Structural Elements in FEA

Structural Element	Spacing (m)	Axial Stiffness, EA (kN/m)	Bending Stiffness, EI (kN/m)	Model Type
1.2 m diameter steel pipe (roof) – 20 mm thick with concrete infilled	Continuous	2.51E+07	3.23 E+06	Plate
Temporary base slab – 500 mm thick	Continuous	16.5E+06	3.43E+05	Plate
Vertical steel frame member	1	1.00E+06	N/A	Node-to-Node

3.3 Instrumentation Monitoring

A real-time inclinometer was installed in the roof pipe to monitor settlement during excavation. Strain gauges were placed at the vertical steel frame member and the horizontal top steel frame member to monitor lateral and vertical forces acting on the steel pipe box. For the horizontal steel frame member, two gauges are proposed at the upper part and the lowest part to eliminate eccentricity errors with respect to the neutral axis of the top beam (refer to Figure 10). Site photo of the strain gages and inclinometer are shown in Figure 11.

Reflector-less settlement markers were installed on the carriageway above the linkway construction area to avoid road closures for settlement readings, enhancing safety for workers (see Figure 12).

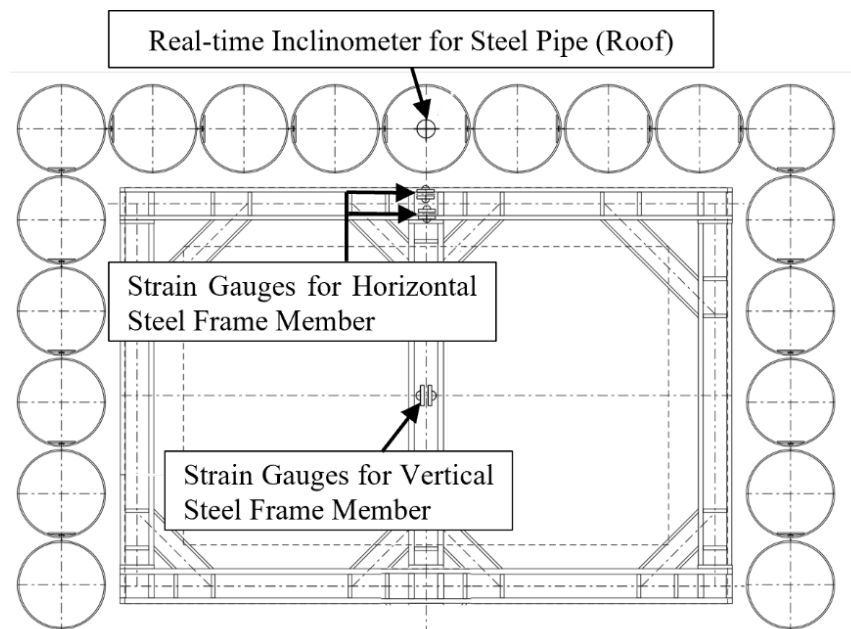


Figure 10. Instrumentation monitoring for steel pipe and steel frame members.

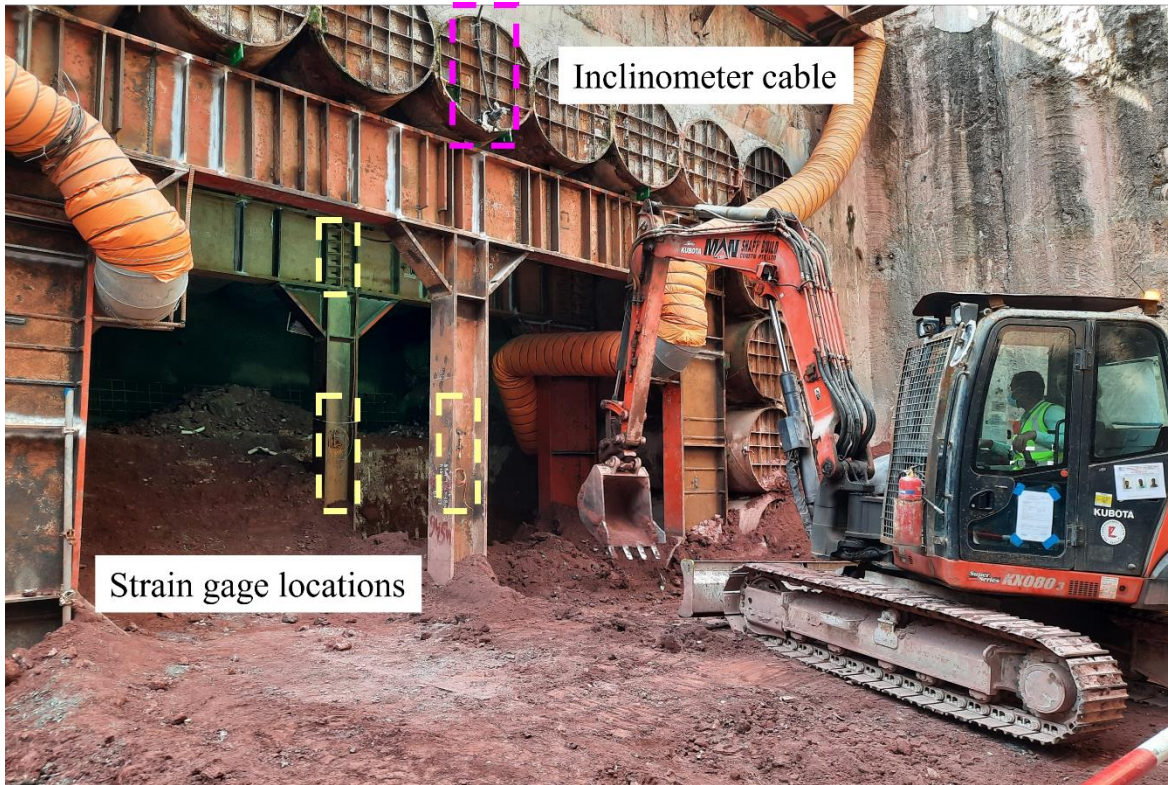


Figure 11. Site photo of instrumentation monitoring for steel pipe (roof) and steel frame members.

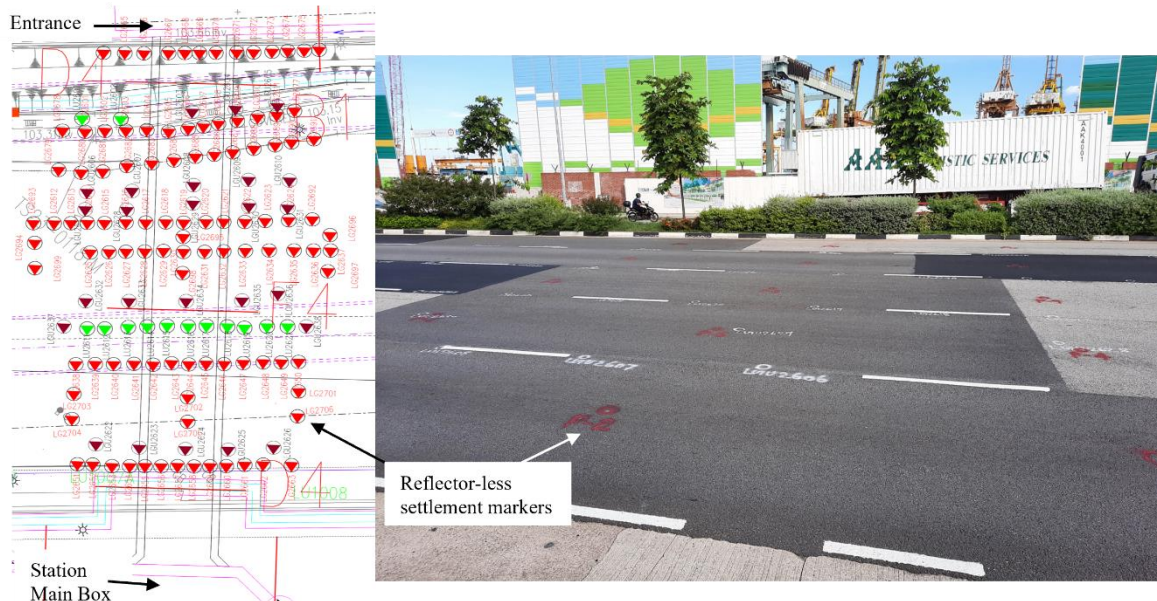


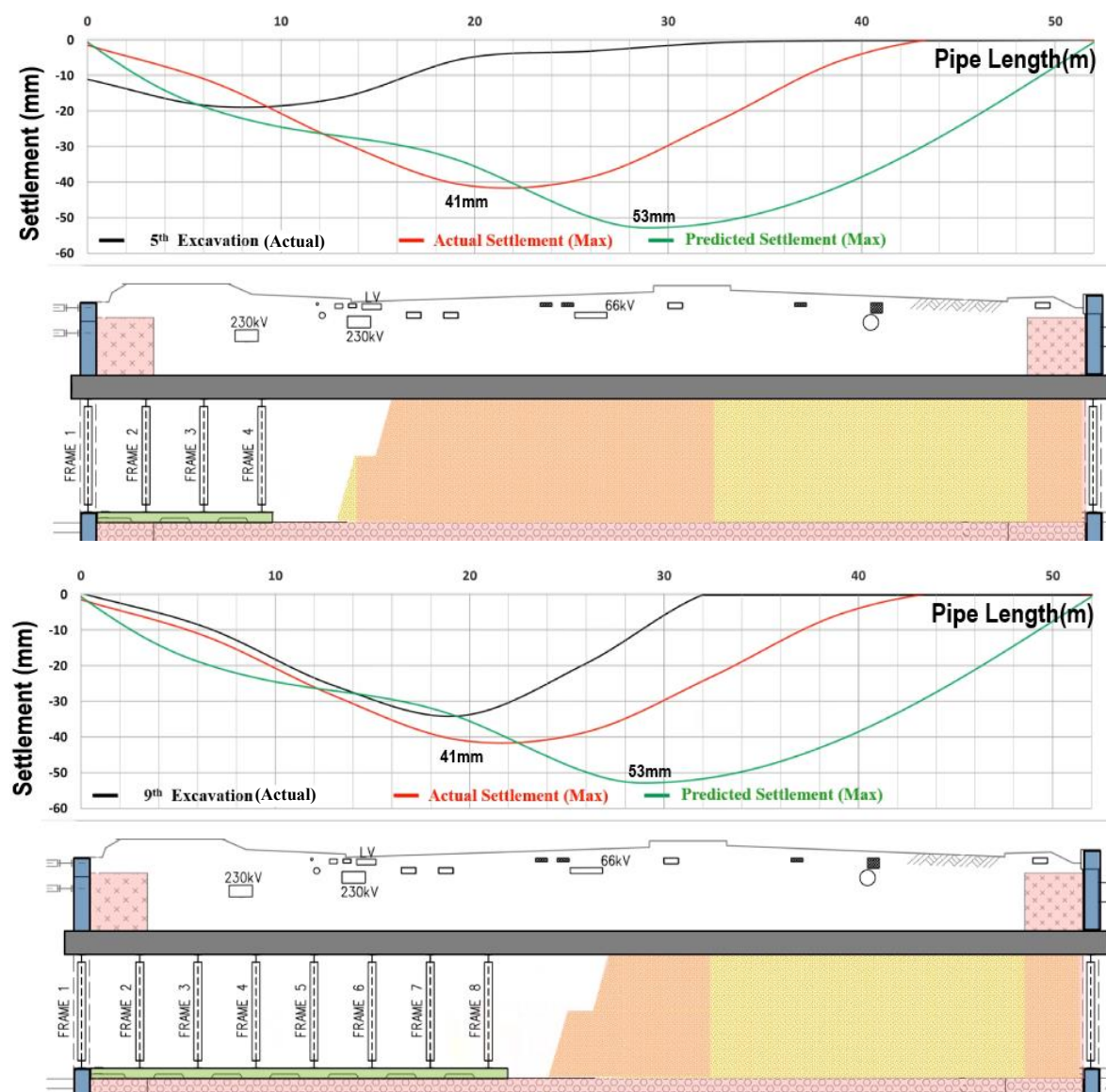
Figure 12. Reflector-less ground settlement markers instrumentation monitoring.

4 OBSERVATION FROM CONSTRUCTION PHASE

4.1 Pipe Roof Settlement Trends

Figure 13 presents the measured pipe roof settlement profile recorded during the 5th, 9th, 13th and 17th stage of excavation, as well as the predicted maximum settlement. From the figure, it can be seen that as the excavation progressed, the maximum settlement observed shifted towards the excavation face, until the 13th stage of excavation, in which the maximum settlement was observed. Contrary to the finite element analysis prediction, the maximum settlement occurred during the 13th stage of

excavation rather than the 17th stage. Instead, some ‘heave’ of about 5 mm can be seen during the 17th excavation stage from 0 to 25 m mark. This might be because of how inclinometer data is interpreted. Inclinometer measures deflection (in terms of angle) instead of displacement. This means that the settlement/displacement has to be calculated based on the deflection. While calculating the deflection, it was assumed that pipe at the end of excavation (54 m mark) did not experience any settlement, i.e., 0 mm settlement. Therefore, if settlement occurred at the pipe located at the end of excavation, by re-zeroing the reading, it would translate to a ‘heave’ on the rest of the reading. Due to the near identical shape of the settlement profile from 0 to 25 m mark, it could be assumed that the pipe at 54 m mark settled by about 4 mm. Nevertheless, the maximum measured settlement (41 mm) was less than the predicted settlement (53mm). The maximum settlement location was also different, with the measured settlement occurring at the 20.5 m mark, vs. the predicted settlement occurring at the 29.5 m mark. These two differences could be due to some discrepancy between the modelled geological profile vs. the real geological profile, as well as some discrepancy between the stiffness value used vs. real value. The weaker soil (SV N15) could be thicker towards the beginning of the excavation, and its stiffness could be higher, resulting in lower settlement measured.



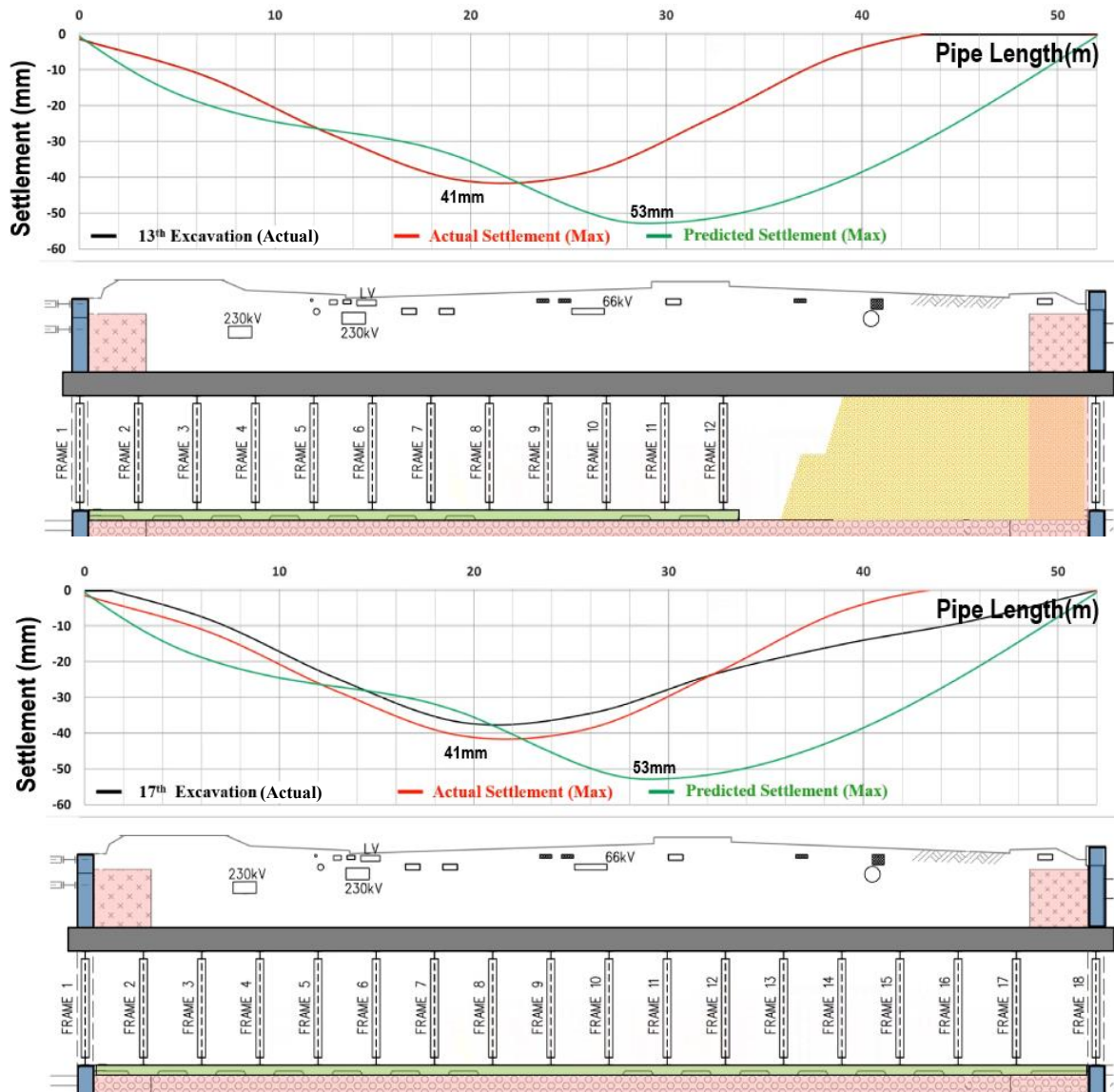


Figure 13. Settlement profile for the top steel pipes during 5th excavation, 9th excavation, 13th excavation, and 17th excavation.

4.2 Steel Frame Forces

Figure 14 compares measured and predicted axial forces for the vertical and horizontal steel-frame members. The predicted horizontal forces were calculated using the tributary-length method described earlier. The measured axial forces of vertical and horizontal members for the steel frames in the middle of linkway (Frame 6 to 11) were higher than those near the ends of the linkway. This aligns with the prediction analyzed from the finite element model. It also can be seen that, except for frame 3 and 7, all the predicted values were higher than the measured values. The underestimation for frame 3 was due to improper placement of the strain gage, while frame 7 could be due to the thicker softer soil (SV N15) around frame 7. Nevertheless, this meant that the 2D finite element analysis can safely predict the pipe roof construction, without the need to conduct a 3D finite element analysis. However, it should be noted that regardless of the method to determine horizontal loading, even with a 3D finite element analysis, it is essential to establish contingency measures to address potential exceedances of predicted steel frame forces. One such contingency measure is to reduce the spacing or add additional steel frames.

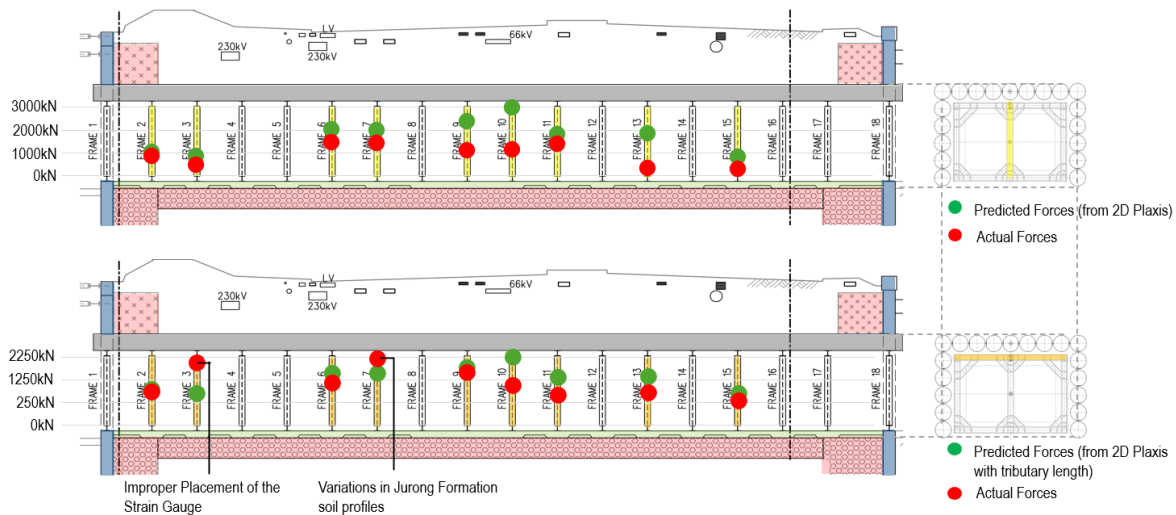


Figure 14. Measured versus predicted forces for vertical and horizontal steel members.

5 CONSTRUCTION DOCUMENTATION

After the diaphragm walls for both the launching shaft (entrance) and the retrieving shaft (main station box) were installed, grouting work with the minimum requirement of strength for the micro-tunnel boring machine launching was then installed at both shafts. Each shaft was then excavated down to the level at which the steel pipe (roof) would be launched. At that depth, temporary formwork and a tunnel-eye seal were installed at the launching shaft, and the micro-tunnel boring machine (mTBM) was then drove the interlocking steel pipes toward the station box, see Figure 15. The mTBM was then retrieved from the station-box shaft. With the steel pipe (roof) installation complete, excavation in both shafts continued to the final base elevation of the underground linkway. As the shafts excavation progressed downward, steel pipes (side) were installed in tandem, and each steel pipe both the roof and side segment was immediately filled with grout to achieve the required stiffness.

After all roof and side steel pipes were in place, horizontal grouting was performed from both the entrance and station-box shafts, see Figure 17. This horizontal grouting served to (1) increase soil strength in mid-linkway zones where SPT $N < 50$ was anticipated, and (2) reduce permeability at the linkway base which preventing water ingress during subsequent mining excavation.

Following horizontal grouting work, sections of the diaphragm wall were broken out to initiate uni-directional mining from the entrance shaft toward the main station box. The mining excavation work proceeded on a controlled slope, with lean concrete applied continuously to stabilize the cut face (Section 2.3, Point 4; Figure 16 top left). Once the slope was established, a temporary base slab and steel support frame were installed, see Figure 16. Following the step-by-step excavation and support sequence outlined in Section 2.3, Point 6, mining advanced until the linkway reached the station box, see Figure 18. Finally, the temporary steel frames were removed in sequence, and the permanent reinforced-concrete linkway structure was cast.



Figure 15. Photograph of micro tunnel boring machine being launched from the launching shaft (entrance).



Figure 16. Photograph of horizontal grouting work.

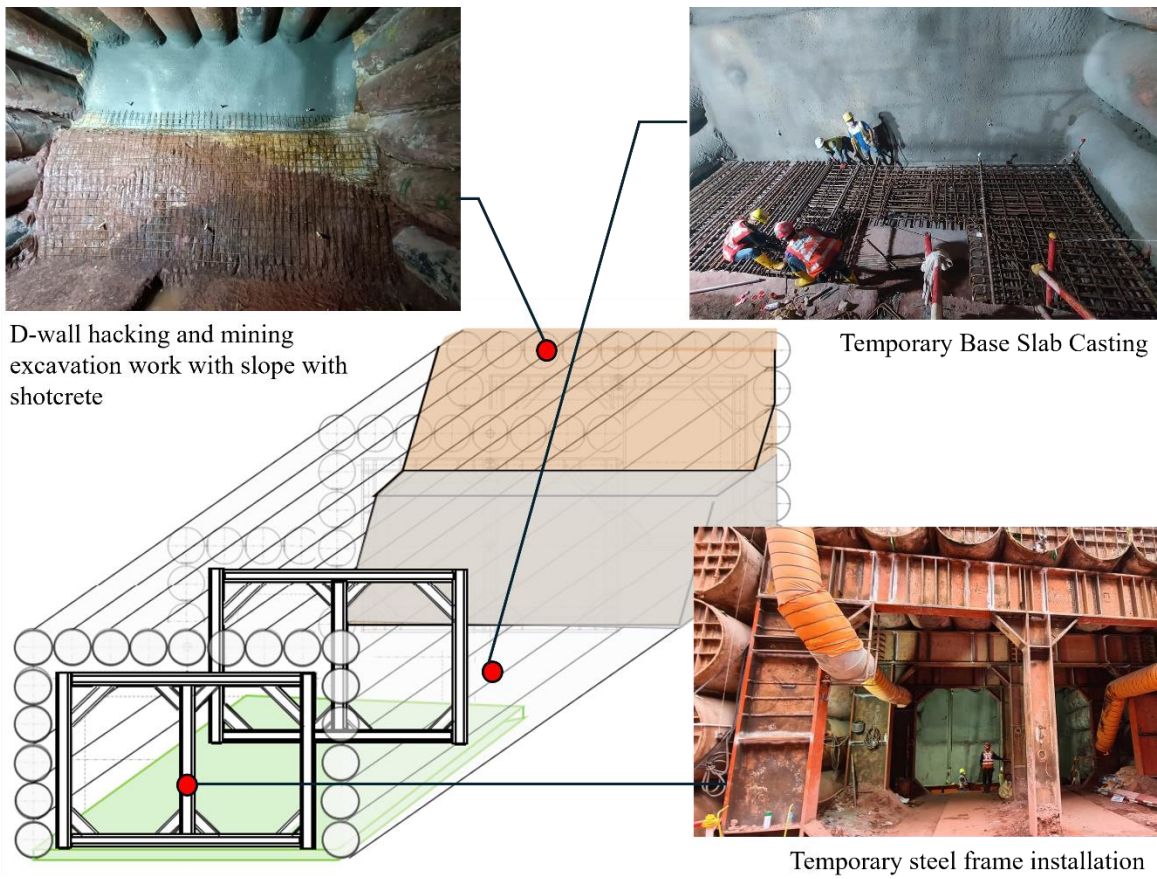


Figure 17. Photographs and schematic diagram of the mining excavation work.



Figure 18. Photograph after the mining excavation work was completed.

6 CONCLUSIONS

This paper presented a tunnel construction using 3-sided pipe roof system with a grouted base, which was implemented for the Keppel MRT station underground linkway construction. Despite challenging conditions of heavy traffic on the 9-lane carriageway and numerous utilities above the excavation, the construction was successful as no hindrance of the traffic nor any damage on the road was recorded.

During mining excavation work, the pipe roof deformed in line with design predictions. The maximum settlement was about 10 mm less than predicted, although it occurred at a slightly different location than anticipated. These discrepancies likely arise from assumptions in the soil-strength profile and natural variations in subsurface conditions. All the recorded forces in the vertical steel frames were lower than the predicted values. Whereas, for the horizontal steel frames, only two of the recorded forces were higher than the predicted values. For one frame, the higher recorded force was due to misalignment in strain gage installation, while for the other frame could be due to the aforementioned variation in geological profile.

Nevertheless, from the results, it can be reasonably concluded that a 2D finite element analysis by assuming that the tributary lengths in both vertical and horizontal directions are identical can obtain a safe design, without the need for a full 3D finite element analysis. However, regardless of the approach to determine horizontal loading, even with 3D modelling, it is crucial to establish contingency measures to address potential exceedances of predicted forces given that actual ground conditions may differ from the design assumption.

The successful application of the design methodology as discussed above makes it a valuable reference for the design and construction of linkways in densely populated urban environments.

DISCLAIMER

The authors declare no conflict of interest.

AVAILABILITY OF DATA AND MATERIALS

All data are available from the author.

REFERENCES

- Chapman, D. Metje, N., & Stärk, A., 2017. *Introduction to Tunnel Construction (2nd edition)*. Crc Press.
- Chua, T. S., 2002. Application of Grouting Techniques in Singapore. *Proceedings of IES Conference on Case Studies in Geotechnical Engineering, Singapore*.
- Mckim, R. A., 1997. Selection Method for Trenchless Technology. *Journal of Infrastructure System*, 3(3), pp. 119-125. [https://doi.org/10.1061/\(ASCE\)1076-0342\(1997\)3:3\(119\)](https://doi.org/10.1061/(ASCE)1076-0342(1997)3:3(119))
- Ng, D. C. C., Prasad, P., Tew, B. W., Neo, C. W., Pong, K. F., & Supargo, R., 2011. Geotechnical and engineering challenges for Downtown Line Stage 2 C917 and C918 projects. In *Proceedings for Underground Singapore 2011*, pp. 196-207.
- Ng, T. G., Yogarajah, I., Woon, K. X., Teo, S. C., & Adalarasu, J., 2016. Trenchless construction of underground infrastructures in Singapore. In *Proceedings of the 19th Southeast Asian Geotechnical Conference and 2nd AGSSEA Conference, Malaysia*, pp. 149-155.
- Ng, C. C., Ong, C. W., Ng, Y., Lee, L. G., Yong, K. W., Ooi, T. A., Soh, J. M. & Neo, C. W., 2019. Valuable Lessons Learnt from Case Histories of Singapore Underground Transit Construction and Future Challenges. *Proceeding of the 1st Malaysian Geotechnical Society and Geotechnical Society of Singapore Conference*.
- Prakoso, W. A., & Lase, Y., 2014. Evaluation of Tunnel Jacking Induced Ground Surface Heave. *Proceeding of 8th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*, pp. 123-128.

Prakoso, W. A., & Sabbah, A. B., 2016. Effect of Nose Blade Angle on Face Stability of Jacked Box Tunnelling. *Jurnal Teknologi (Sciences & Engineering)*, 78(8-6), pp. 9-14. <https://doi.org/10.11113/jt.v78.9633>

Qiu, D., Wang, L., Zu, Y. & Qing, Y., 2025. The Development of the Pipe Jacking Guidance Technology. *Applied Sciences*, 15(4), p. 2137. <https://doi.org/10.3390/app15042137>

Teo, S. C., Ng, T. G., Yogarajah, I., Woon, K. X., & Dang, H. P., 2016. Two-lane vehicle underpass using pipe roof method at Sentosa Gateway Tunnel. *Proceeding of the 8th Asian Young Geotechnical Engineers Conference, Astana*, pp. 105-110

Woon, E., Tse, I., Teo, S. C., & Ng., T. G., 2022. Three-dimensional finite element analysis of a pipe box tunnel in sedimentary rock. *Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering, Sydney*, pp. 273-277.