3-D Finite Element Analyses on Interaction between Building Foundations and Tunnels in Soil and Rock

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ABSTRACT: The geotechnical challenges for a high-rise development of up to 82-storey high in limestone formation with two future Mass Rapid Transit (MRT) tunnels cutting across the proposed development area are described in this paper. Due to the close proximity of the proposed tunnels, extensive finite element analyses (including 3-D analyses) were carried out to ensure compliance with the requirements of Railways (Railway Protection Zone) Regulations 1998. Advanced finite element analyses have demonstrated the feasibility of the project in compliance with the strict requirements of Railways Regulations 1998. The analyses result also demonstrated the importance of obtaining reliable rock stiffness parameters to assess impact of the proposed development to the future tunnels in limestone formation.

KEYWORDS: Soil-structure interaction, 3-D finite element, Bored pile foundation, Tunnels in Soil and Rock

1. INTRODUCTION

The project site is located beside Jalan Bukit Bintang, Kuala Lumpur which is a major commercial area of Kuala Lumpur and the proposed development comprises the construction of high-rise residential towers and podium carparks. During planning of the development, it is known that there are stacked tunnels (namely Northbound and Southbound tunnels) at this location for future MRT with the proposed alignment cutting across the site at depths of about 20m to 36m below ground level (mbgl). The current approach for tunnel lining design for MRT has considered surcharge loads above the tunnels of 50kPa which is not adequate to cater for the high-rise development. It is the responsibility of the developer of future development adjacent to the MRT tunnels to ensure the loadings for the proposed development will not impact the adjacent tunnels within acceptable limits. In view of the proximity of the proposed future MRT tunnels to the building foundation, one of the biggest challenges of this project is to control the movement of the MRT tunnels within the stringent criteria stipulated in the Railways (Railway Protection Zone) Regulations 1998 (PU(A) 367 1998).

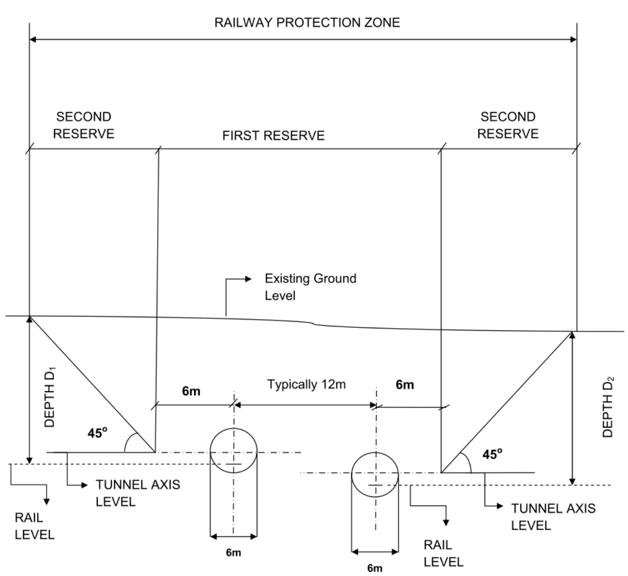

Figure 1 Railway protection zone (Railways Regulations 1998)

Figure 1 presents the designation of railway protection zone. Some of the important criteria are listed as follows:

i. Total movement in the railway structure or tracks not exceeding 15mm in any plane.
ii. Differential movement resulting from the works shall not produce final distortion in the track or its plinth in excess of 3mm in 6m (1:2000) in any plane.
iii. No sheetpiles, piles, foundation, boreholes or wells shall be driven within First Reserve.
iv. For pile foundation constructed in the Second Reserve, the piles are designed so that they are debonded within the zone of influence of the underground structure and develop all of their load either in shear or end bearing from soil located below zone of influence of the structure.

This paper presents evaluation of the impact of pile foundation under full design loadings from the high-rise building on the tunnels particularly in terms of tunnel deformation and structural forces using 3D finite element program (PLAXIS 3D). Both scenario for tunnels in rock or in soil are compared and discussed.

2. GEOLOGY AND GROUND CONDITION

The site is overlying Kuala Lumpur Limestone formation which is normally with karstic features such as undulating rockhead profile and limestone cavities. Subsurface investigation (SI) was carried out in two stages consisting of a total of 44 rotary wash boring boreholes spread across the site. The overburden soil is alluvium fill of about 10m to 15m thick, predominantly consisting of silty SAND, sandy SILT or sandy CLAY with SPT-N values less than 20, except for one of the boreholes where hard stratum (SPT-N > 50) is encountered at 21mbgl and infill cavities are encountered up to 35mbgl. This implies that most of the tunnels within the site are likely to be in rock except potential tunnelling in soil at localised area. The groundwater table is generally about 4mbgl.

3. THREE-DIMENSIONAL FINITE ELEMENT MODEL

3.1 Material models and parameters

The parameters adopted for soil strength and stiffness are established from the field investigation and lab testing. Based on past experiences on similar ground conditions and the cross hole seismic tests carried out in the site, a correlation of Young's modulus E_{50} of 3000N (kPa) for SPT-N less than 10 and 2500N (kPa) for SPT-N more than 10 is adopted for the Hardening Soil model in PLAXIS. The unloading and re-loading stiffness is equal to 3 x Young's modulus.

The effective cohesion (c') of limestone based on Hoek-Brown failure criterion was assessed using a software called "RocLab" for application of tunnel. Effective cohesion (c') of 400kPa is adopted, which represents average cohesion value for Grades II to IV limestone rocks (typical range of rock grades for limestone in Malaysia). The Mohr-Coulomb shear strength parameter obtained from simulation of triaxial test results based on Hoek-Brown failure criterion is not an intrinsic property of rock and is dependent on stress range. Therefore, the effective friction angle of 32° was taken according to average suggested value of basic friction angle (intrinsic property) for limestone tested in wet state, after Barton and Choubey (1977). The estimation of rock mass modulus (E_m) is based on intact rock modulus (E_i) and Rock Quality Designation (RQD) of the rock using the empirical method modified after Carter and Kulhawy (1988). The RQD values for bedrock obtained from the site investigation vary significantly from 0% to 100% with an average RQD value of about 60%. For impact assessment purposes, a lower bound value of RQD = 20% and upper bound value of RQD= 50% have been adopted. Based on literature review (Julius et al., 2017) and the Authors' company database from past experiences, the intact rock moduli for limestone in Klang Valley ranges from about 20 GPa to 100 GPa. The adopted lower bound (LB) and upper bound (UB) intact rock modulus in the analysis are 20 GPa and 70 GPa respectively. Mohr-Coulomb model is employed for limestone in the analysis. The derivation of rock mass modulus is summarized in Table 1 and summary of geotechnical parameters used in the analysis is shown in Table 2.

Linear elastic model is assumed for the 275mm thick tunnel lining with 6.35m outer diameter. The bored pile foundation is simulated as embedded beam elements which are connected to the continuum using interface element to represent pile-soil interaction. Bored pile diameter of 600mm is adopted for the podium area (lighter loadings) while bored pile diameter of 1800mm is adopted for the tower area (heavier loadings). The material properties for simulation of bored pile in the FEM model are summarized in Table 3.

Table 1 Rock Mass Modulus

Analysis Cases	**RQD (%)**	**Intact Rock Modulus, E_i (GPa)**	**E_m/ E_i**	**Rock Mass Modulus, E_m (GPa)**	**Adopted E_m for analysis (GPa)**
Lower Bound	20	20	0.05 (Open/ Closed joint)	1.0	1.0
Upper Bound	50	70	0.15 (Closed joint)	10.5	10.0

Table 2 Geotechnical Parameters for Finite Element Analysis

Soil/ Rock Type	**SPT-N**	**Effective cohesion, c' (kPa)**	**Effective friction angle, ϕ '(°)**	**Young's modulus, (kN/m²)**
Silty SAND/ Sandy Silt	0-10	3	28	3000 x SPT-N
	10-50	3	28	2500 x SPT-N
	>50	3	34	2500 x SPT-N
Limestone	-	400	32	1.0E+06 (LB) 1.0E+07 (UB)

Table 3 Embedded Pile Properties

Property	**Unit**	**Value**
Pile Diameter, D	m	0.6 & 1.8
Young's Modulus, E	kN/m²	2.8 x 10^7
Skin Resistance (Soil)	kN/m	SPT-N x πD
Skin Resistance (Rock)	kN/m	500 x πD
Base Resistance, F_{max}	kN	N/A

3.2 Bedrock level

Bored pile socketed into limestone bedrock is adopted as the foundation system for the building. Different elevations of pile toe relative to the tunnel position are expected to result in different impacts to the tunnel. Based on the subsurface investigation results discussed in Section 2, two extreme cases of bedrock levels have been considered for sensitivity studies: (i) shallow bedrock case (10 mbgl), (ii) Deep bedrock case (35.5 mbgl). In the first case, both tunnels are fully embedded in the rock and the piles are terminated above the tunnel axis level. In the second case, the pile toes are below the tunnel axis level, and the upper south bound tunnel is fully in soil while the lower north bound tunnel is partially in rock. No pile debonding was considered in both the cases. Two typical sections representing the two cases of bedrock level are shown in Figures 2 and 3 respectively.

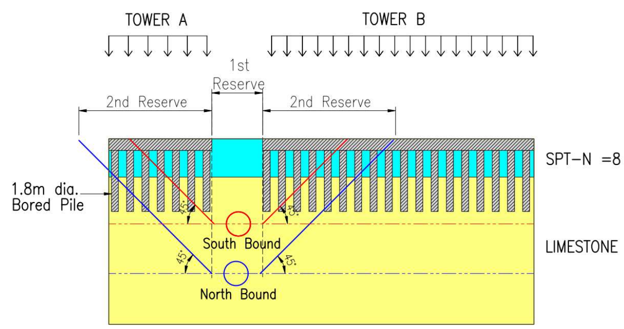

Figure 2 Typical cross section for shallow bedrock case

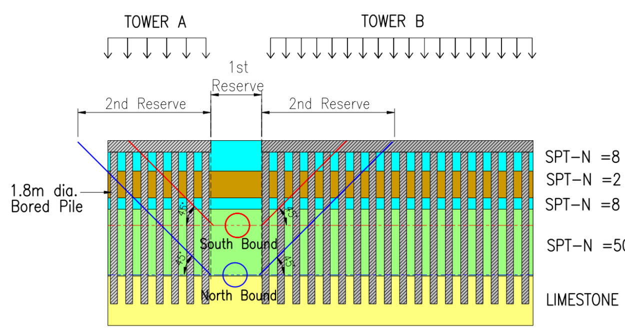

Figure 3 Typical cross section for deep bedrock case

For the shallow bedrock case, both the lower bound and upper bound rock stiffnesses as discussed in Section 3.1 will be considered. As summarized in Table 4, for the deep bedrock case, only the lower bound rock stiffness is modelled as the results are not sensitive to the values of rock stiffness since tunnels are mainly embedded in soil.

Table 4 Analysis Cases

Analysis Case	**Bedrock Level**	**Rock Mass Modulus (kN/m²)**
Case LB-S	Shallow Bedrock (S)	1.0E+06 (LB)
Case UB-S	Shallow Bedrock (S)	1.0E+07 (UB)
Case LB-D	Deep Bedrock (D)	1.0E+06 (LB)

3.3 Plaxis 3D numerical model and loadings assumptions

The finite element models were meshed using 10-node tetrahedral 3D element. To comply with the Railway Regulations, no piles will be located within the First Reserve and as such, the closest horizontal distance between bored piles to edge of tunnel is capped at about 3m. The piles are modelled at uniform spacing of about 2.3 times the pile diameter. During the time when this study was carried out, the final adopted development plot ratio had yet to be finalized and therefore, a higher plot ratio (i.e. higher loading) was adopted in the analysis for feasibility study purposes. For simplicity of model, the column loads from the building were assumed as uniformly distributed load of 17kPa per floor, acting on the pile cap which is simplified as piled raft. As shown in the 3D model in Figure 4, the tunnels transverse the site at varying depths and orientations while the proposed piled foundations are loaded with non-regular loading patterns which could not be properly modelled and assessed using 2D analysis.

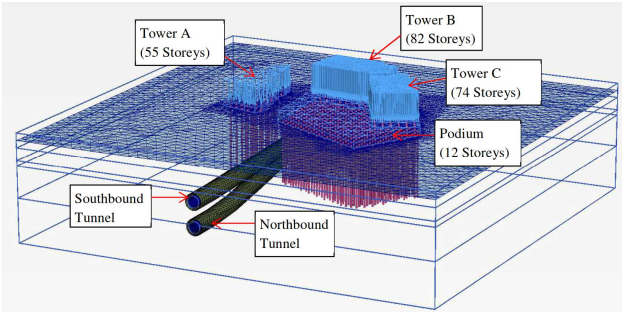

Figure 4 Plaxis 3D model

3.4 Construction sequence

Both construction of tunnels and buildings had yet to commence at the time this impact assessment was carried out. In the analysis, tunnels are in place prior to installation of pile foundation, followed by pile cap excavation and pile cap construction together with application of full building loads. This will be the worst-case scenario in terms of impact to the tunnels after considering uncertainties in the actual work program. In addition, flexibility in the timing of construction can be allowed if the impact assessment shows satisfactory results under this worst-case scenario.

4. RESULTS OF ANALYSIS

4.1 Tunnel displacement

In all the three analysis cases, the maximum tunnel displacement occurred at the upper Southbound tunnel close to Tower B with highest imposed load intensity as shown in Figure 5.

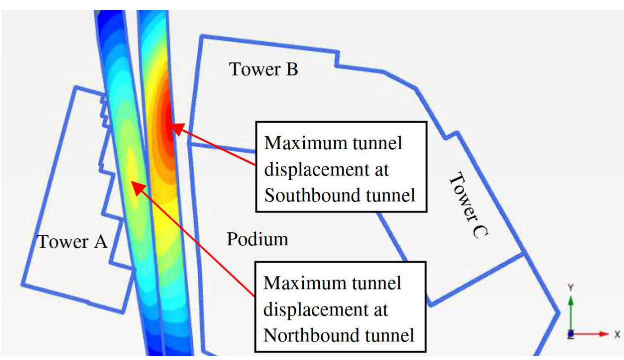

Figure 5 Maximum tunnel displacement location

The direction of the tunnel movement is generally downwards and away from the piles under higher loadings. Tunnel crown is subjected to the highest displacement. In the shallow bedrock case, the estimated maximum tunnel displacement is about 14.1mm for rock stiffness of 1E+06 kPa and is only 1.5mm for rock stiffness of 1E+07 kPa, which shows that the tunnel displacement is significantly affected by rock stiffness. In the deep bedrock case (Case LB-D) with tunnel in soil, the displacements of the tunnel generally show similar trend with slightly lower magnitude compared to those of Case LB-S. In all the three analysis cases, the total movement of tunnel at any plane does not exceed the allowable 15mm set by the Railway Regulations even when the extreme cases with conservative assumptions are considered. The tunnel deformations at the section where maximum displacement occurred are shown in Figure 6.

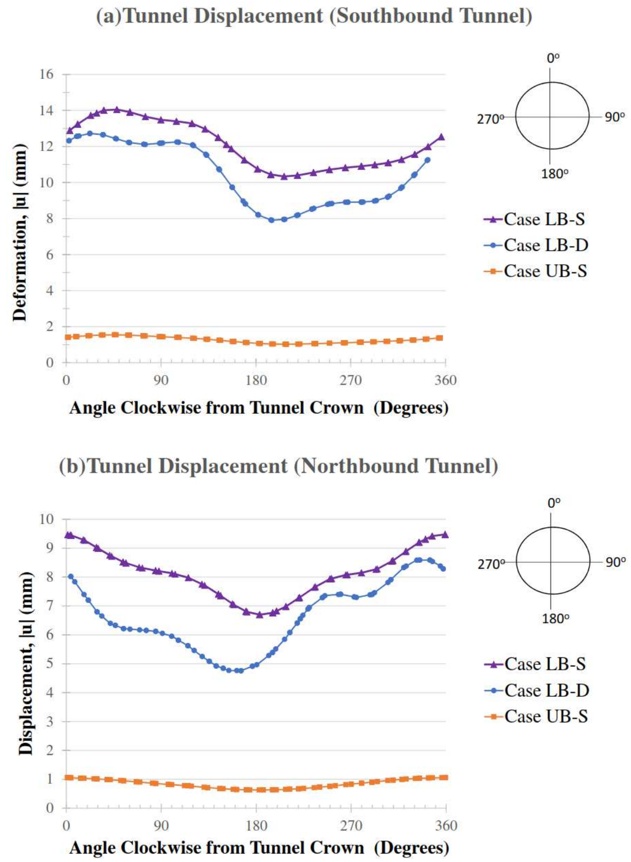

Figure 6 Tunnel deformation at section with maximum displacement (a) Upper Southbound Tunnel. (b) Lower Northbound Tunnel.

4.2 Tunnel angular distortion

The tunnel differential movement was checked in transverse plane as well as longitudinal plane. In any plane, the allowable distortion in the track or its plinth is limited to 1:2000 (5 x 10^{-4}) according to the Railway Regulations. In all the three analysis cases, the governing angular distortion occurred at the longitudinal plane of the upper Southbound tunnel and the analyses results indicated that the induced tunnel angular distortions are within the acceptable range. Similar to the tunnel displacement, Case LB-S shows highest distortion, followed by Case LB-D, and Case UB-S shows negligible distortion. The maximum angular distortion is summarized in Table 5.

Table 5 Summary of FEM Results

Description	Case LB-S	Case UB-S	Case LB-D
Maximum tunnel displacement (mm)	14.05mm	1.54mm	12.73mm
Maximum tunnel angular distortion	3.32 x 10^{-4}	5.63 x 10^{-5}	3.00 x 10^{-4}
Maximum tunnel bending moment (kNm/m)	45.92	11.94	72.04
Maximum axial force at maximum bending moment (kN/m)	-1964.31	-866.00	-1324.81

4.3 Tunnel structural forces

The induced tunnel lining forces in terms of axial and bending moment forces (N-M) due to the proposed development are also checked. The checking is commonly carried out by plotting the calculated lining forces in the tunnel lining N-M capacity diagram as shown in Figure 7. The tunnel lining forces after the piles are loaded to full loadings are within the tunnel lining structural capacity, thus are acceptable.

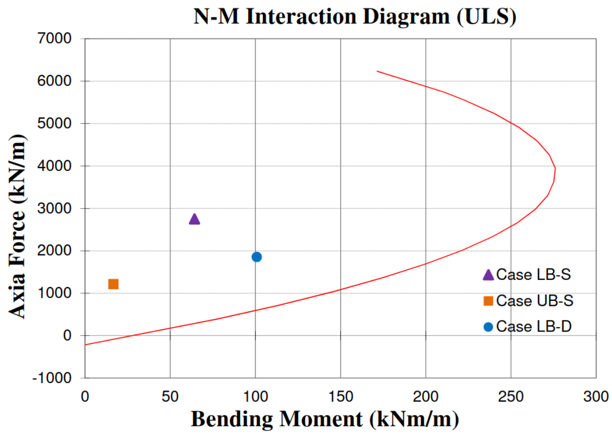

Figure 7 Tunnel lining N-M interaction diagram

5. CONCLUSION

This paper evaluates the impact of high-rise building foundation on adjacent tunnels using three-dimensional numerical models (Plaxis 3D). The summary of the FEM results for all the three analysis cases discussed in Section 4 is presented in Table 5. The main results of the analyses are as follows:

i. 3D FEM analyses showed that the tunnels' response, in term of tunnel deformation, is within the acceptable limits set by Railway Regulations, despite no debonding of piles in the Second Reserve.

ii. For tunnelling in rock, the tunnel response is very sensitive to the adopted rock mass modulus. The range of rock mass modulus can vary widely subjected to in-situ rock conditions. Therefore, selection of rock mass modulus for design/impact assessment is crucial especially for critical structures such as tunnels. Therefore, it is suggested to perform in-situ tests such as borehole dilatometer or borehole jack to measure the in-situ rock mass modulus and correlations should be established based on back-analysis of actual deformations of completed tunnels in similar geotechnical conditions.

6. REFERENCES

Barton, N. and Choubey, V. (1977). "The shear strength of rock joints in theory and practice", Rock Mechanics, vol. 10, pp. 1-54.

Carter, J.P. and Kulhawy, F.H. (1988). "Analysis and Design of Drilled Shaft Foundation Socketed into Rock." Final Report, Project 1493-4, EPRI EL-5918, Cornell University, Ithica, NY.

Hoek, E. and Brown, E.T. (1997) "Practical Estimates of Rock Mass Strength" International Journal Rock Mechanics Mining Science, Vol. 34, pp. 1165-1185.

Julius, K. K., Thomas, F., Musa S.D.M. and Raymond, J. D., (2017) "Elastic Parameters from Compressional and Shear Wave Tomographic Survey: A Case Study from Kuala Lumpur, Malaysia". Journal of Environment and Engineering Geophysics, Volume 22, Issue 4, p427-434.

Ministry of Transportation Malaysia (1998). Railway Act 1991. Railways (Railway Protection Zone) Regulations (PU(A) 367 1998). Malaysia Government Gazette.