Effects of shallow over-crossing TBMs on existing tunnels and surface movements

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ABSTRACT: Shallow tunnels have the benefits of low short-term construction costs and long-term operational costs as the station/shafts would be shallow; but tunneling in shallow overburden condition will mean over-crossing the numerous existing tunnels in dense urban areas. These over-crossing can have significant impact on the existing tunnels if the induced heave exceeds the capacity of the existing tunnels. It is often sought to carry out complex 3D numerical model to assess such interaction between the over-crossing tunnels and underlying tunnels. As an alternative, in this paper, a semi-analytical method for a Thomson Line (TSL) tunnel overcrossing an existing North-East Line (NEL) is presented. The proposed method uses Mindlin's solution to estimate equivalent unloading stress on the underlying tunnel. This unloading stress is applied on the tunnel by considering the underlying tunnel as a continuous Euler-Bernoulli beam supported on soil-springs. The results from this simplified proposed formula is compared with PLAXIS 3D numerical model and the actual field readings. The proposed method could be used as a quick (low-cost) estimate for evaluating tunnel response due to over-crossing, especially during tender/preliminary design stages. Also in the paper, the effects of shallow tunnelling and tail void grout pressure on surface settlement is discussed.

1 INTRODUCTION

Construction of a new tunnel in a dense urban environment often encounters the pre-existing structures and facilities such as utilities, foundations, existing tunnels etc. To ensure the safety of the existing tunnels is very important as the new tunnel construction will change the already balanced equilibrium stress state of the ground. The new tunnel can induce additional movements, loads and bending moments. In simple terms, the response of the existing tunnel is a result of interaction between the disturbed soil movements (induced by new tunnel) and the bearing capacity of existing tunnel. However, the interaction could be highly complex based on the configuration of the tunnels and surrounding ground conditions. Understanding of interaction is critical in providing effective protective measures, particularly for aging tunnels. Moreover, because of the rapid development of urban infrastructure, there is a need for quick assessment of the interactions between the existing tunnel and new tunneling activity.

Even if we could make predictions of the "greenfield" ground movements due to tunneling, the presence of a "rigid" element in ground may alter these movements by what is termed as "soil-structure interaction". The estimation of the risk of damage to underground buried structures, typically involves if the structure deforms according to the greenfield ground movements, i.e fully flexible, and ignoring the stiffness of the structural elements. Estimates using this approach can be highly conservative. Hence a variety of approaches have been used to study the impact, namely physical model, field observations, numerical analysis and empirical/analytical methods.

Field observation is straightforward method for understanding the interaction between existing and nearby crossing tunnels. A few such field observations have been used to study the impact due to NATM tunnels in Singapore (Senthilnath 2017) but field observations from a project cannot be directly applied to another situation to understand the behaviour before any construction activity begins at the site unless a database of such behaviour is available. Hence at the design stage, field observations are not applicable.

Finite Element (FE) numerical simulations are most efficient way to investigate the interaction between the existing tunnels and new tunnels. Addenbrooke and Potts (2001) carried out a series of 2D finite element analyses to study the effects of different tunnel relative position (parallel and stacked) and construction sequences on the existing tunnel. Shahrour (2008) conducted a 2D finite element numerical analysis to study the impacts of nearby tunnelling on pre-existing tunnel with different construction procedures. Do et al. (2015) performed a series of 3D numerical analyses to investigate the influences of constructions process on two parallel and stacked tunnels.

Researchers have proposed various analytical or empirical approaches to compute the displacements and bending moment of pipeline due to newly tunneling. Combining the simplified elastic beam on Winkler foundation approach (Zhang & Huang 2014) with unloading stress estimation (Mindlin, 1936), this paper demonstrates a simplified semi-analytical method for evaluation the interaction between existing tunnel and over-crossing tunnel. Estimates from this simplified method are compared with 3D numerical model and actual field results (after the TBM has over-crossed the existing tunnel).

This paper mostly concerns the design phase. The proposed semi-analytical method for interaction between existing tunnel and overlying tunnel is useful for the optimization of the relative position and deciding protective measures (if required).

2 SIMPLIFIED ANALYTICAL METHOD

2.1 Mindlin's Solution – Unloading pressure due to new tunnels

Figure 1 & 2 shows the response of existing tunnel due to overcrossing of new tunnel. The removal of pressure due to excavation in the new tunnel causes stress release in the ground. The TBM face pressure maintained during TBM operation is enough to prevent any failure of face but the face pressure is still less than the in-situ stress in the soil hence the stress release in the ground is inevitable. Assuming the stress behaviour due to new tunnel as a semi-infinite space problem, the unloading vertical stress beneath the new tunnel can be evaluated through the Mindlin's solution (Mindlin, 1936).

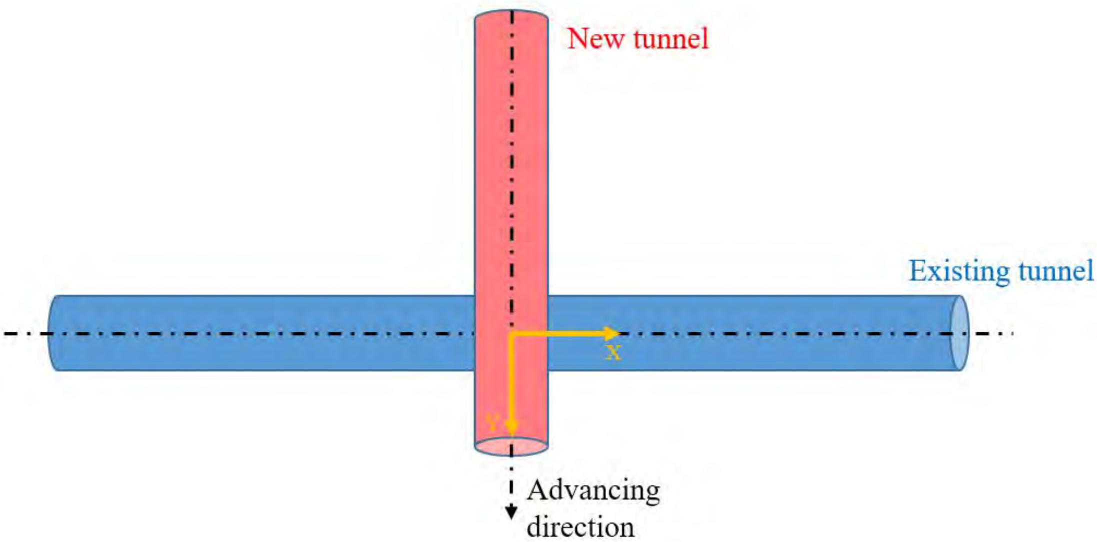

Figure 1. Relative location of existing tunnel and new tunnel (plan)

Mindlin's solution provides the vertical stress σ_z, at an arbitrary point in space (x, y, z) in terms of unloading force, depth of upward point force and Poisson's ratio of soil. The unloading pressure (p) is generated by the difference between the weight of the excavated soils and the tunnel segmental linings per unit length.

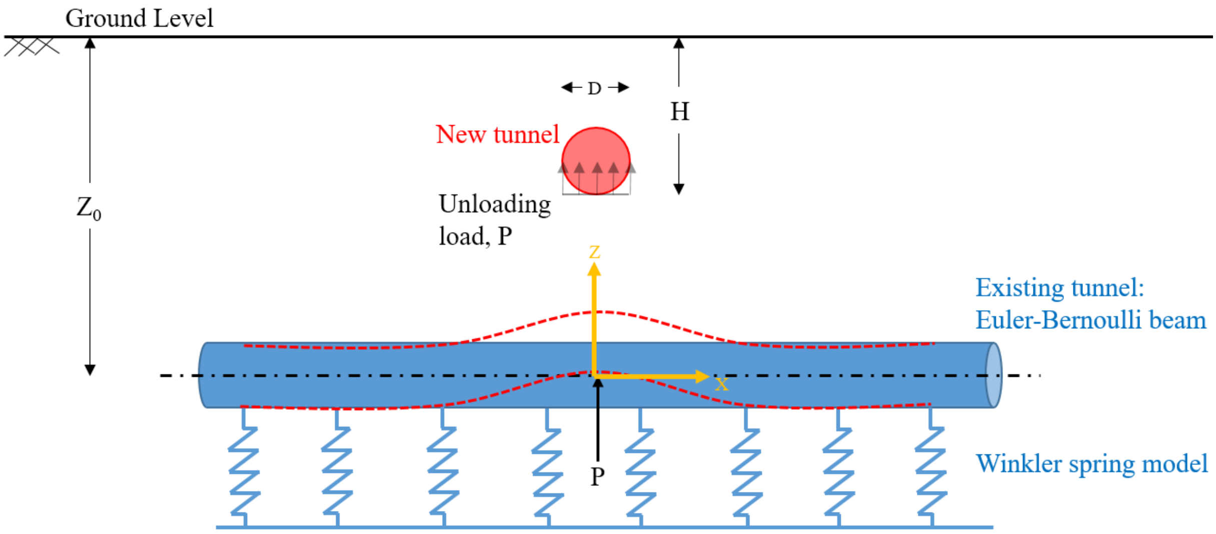

Figure 2. Behaviour of existing tunnel due to overcrossing

$$p = \frac{\gamma_s \pi R^2 - \gamma_t(\pi R_o^2 - \pi R_i^2)}{2R_o} \tag{1}$$

where, γ_s and γ_t are the unit weight of the excavated soil and the segmental lining. R, R_0 and R_i are the excavated radius, outer radius and inner radius of segmental lining. By using above unloading pressure in the Mindlin's equation, unloading pressure on the existing tunnel, as a function of x: q(x) could be obtained.

2.2 *Existing tunnel as continuous Euler-Bernoulli beam*

In this solution, the existing tunnel is simply considered as a continuous Euler-Bernoulli beam resting on Winkler foundation. The length of the underlying tunnel is assumed to be sufficiently long so that each end of the beam is not affected by the new tunnel. By first principles, the governing equation for beam is given by:

$$EI\frac{d^4W(x)}{dx^4} + kD_eW(x) = q(x)D_e \tag{2}$$

where, EI is the equivalent stiffness of the existing tunnel, W(x) is the vertical displacement of the existing tunnel, k is the subgrade modulus coefficient, D is the outer diameter of the existing tunnel and q(x) is the additional vertical unloading pressure on the beam caused by over crossing (as estimated in Section 2.1). Combining the equation (1) and (2) and solving the differential equations, following general solution of vertical displacement for a concentrated load P acting at the centre of the infinite beam can be obtained as:

$$W(x) = \frac{P\beta}{2K}e^{-\beta x}(\cos\beta x + \sin\beta x) \tag{3}$$

where, P is the unloading force (defined as a point load or as a function of x), β is constant based on EI of beam and soil spring constant and x is the location at which displacement is calculated. Based on beam theory, the corresponding bending moment, M(x) and shear force, Q(x) can be calculated as follows.

$$M(x) = -EI\frac{d^2W(x)}{dx^2} \tag{4}$$

$$Q(x) = -EI\frac{d^3W(x)}{dx^3} \quad (5)$$

By simplifying the formula to estimate maximum heave due to over-crossing tunnel, following simplified relations is obtained. P is calculated by integrating the q(x) distribution (as defined in section 2.1). For a quick estimation, equivalent P is calculated by considering that p is acting over 2D distance on existing tunnel.

$$Heave_{max} = \frac{P}{2K}\sqrt[4]{\frac{K}{4EI}} \quad (6)$$

It could be noted that this formula doesn't consider the relative distance of existing tunnel and over-crossing tunnel. Hence formula is applicable only for tunnels which are in very close proximity (~0.5D). In other cases, full integral form of q(x) need to be used to estimate reduced unloading pressure due to the actual distance between existing tunnel and new tunnel.

In modelling the tunnel as continuous Euler Bernouli beam, the subgrade modulus coefficient k is important parameter which govern the interaction between soil and the tunnel. Empirical formula proposed by Vesic (1961) is often used for this purpose however for a tunnel buried at a certain depth below the surface, the soil-structure interaction exhibits a high sensitivity to embedment depth and it has been documented that Vesic's elastic subgrade modulus may lead to misleading results (Attewell et al 1986). For our estimation, a more rational subgrade modulus estimation proposed by Yu et al (2013) is used.

$$K = kB = \frac{3.08}{\eta}\frac{E_s}{1-\mu^2}\sqrt[8]{\frac{E_sB^4}{EI}}$$

with

$$\eta = \begin{cases} 2.18 & \text{when } \frac{z_0}{B} \leqslant 0.5 \\ 1 + \frac{1}{1.7z_0/B} & \text{when } \frac{z_0}{B} > 0.5 \end{cases} \quad (7)$$

where B is equal to the diameter of the tunnel.

Above equation considers the depth at which the existing tunnel is situated. This formulation is used only to arrive at the subgrade modulus values (based on depth and diameter of tunnel). The distance between the overcut of the tunnel and existing tunnel is considered in the Mindlin's equation and if the distance is more than ~0.5D, the unloading pressure should be applied on a winkler spring model in a simple structural software (such as STAAD.Pro) with q(x) being derived from the Mindlin's equation to estimate the induced heave in the existing tunnel.

Young's modulus of the soil surrounding the existing tunnel is taken based on Geotechnical Interpretative Baseline Report (GIBR). Estimating correct EI is a key factor as it reflects the capacity of tunnel to resist the soil movement induced by adjacent construction. In reality, shield tunnel is a composite structure composed of segmental pieces bolted together. Due to these joints, the bending stiffness of the tunnel is much less than an equivalent cylindrical tube. The joints greatly reduce the equivalent EI based on number of bolts, bolt properties and length of the bolt. Figure 3 represents the expected behaviour of tunnel in bending due to joints between the segment rings.

Longitudinal continuous model presented by Shiba et al 1998 is considered in this analysis which takes in to various factors such as bolt stiffness, number of bolts, lining stiffness and tunnel geometry. Longitudinal equivalent bending stiffness of tunnel:

$$EI = \frac{\cos^3 \varphi}{\cos \varphi + (\varphi + \pi/2) \sin \varphi} E_c I_c$$

$$\varphi + \cot \varphi = \pi \left(0.5 + \frac{n k_b l}{E_c A_c} \right)$$

$$k_b = E_b A_b / l_b \qquad (8)$$

where k_b is the elastic stiffness of longitudinal joints, E_b is the Young's modulus of bolt, A_b is the section area of bolt, l_b is the length of bolt, n is the number of longitudinal bolts, l is the width of tunnel segment, E_c is the Young's modulus of the tunnel segment, φ is the angle of neutral axis, I_c is the longitudinal inertia moment of the section of a segment and A_c is the sectional area of tunnel segments. Equation 8 can be solved using MS Excel, built in add-in "Solver" function to determine φ.

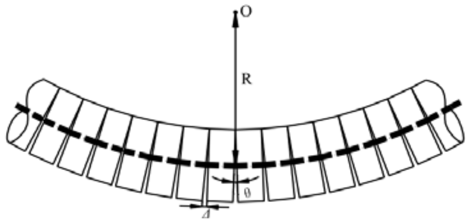

Figure 3. Bending deformation mode of tunnel

3 3D NUMERICAL MODEL FROM CASE STUDY

In order to examine the validity of the proposed simplified analysis, a comparison is made to the results from numerical model and field instruments. In LTA Thomson Line contract T222, between Outram Park station (OTP) and Maxwell Station (MAX), the TSL twin bored tunnels over cross the existing North-East Line (NEL) twin tunnels. The vertical minimum clearance between the extrados of NEL tunnels and T222 tunnels is 2.4m. In addition to TSL and NEL tunnel, East West Line (EWL) tunnels are also in the proximity but are running parallel (at a distance of >1D) to the TSL tunnel. Relative locations of tunnels are showed in Figure 4. EWL tunnels are indicated in model just for reference and hence results of EWL tunnels are not discussed in this paper.

The inner diameter of NEL tunnel is 5.8m with a lining thickness of 250mm. The nominal width of segment is 1.2m. Lining parameters of new tunnel and existing tunnel under our study are summarized in Table 1. The geometry of T222 tunnel which crosses over the NEL tunnel at a right angle, is clearly a three-dimensional problem. Three-dimensional finite element program Plaxis 3D is used to simulate the T222 bored tunnel excavation and its effect on NEL tunnels. Figure 5 presents the numerical model indicating all the 6 bored tunnels.

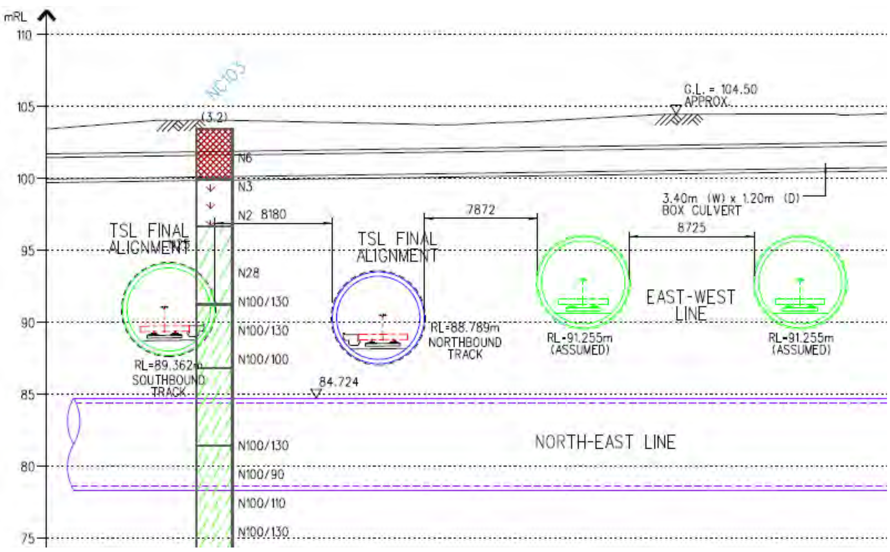

Figure 4. Relative location of tunnels existing tunnels (EWL & NEL) and new tunnels (TSL/T222)

Based on Shiba's method (Shiba et al., 1988) the calculated longitudinal equivalent bending stiffness EI of the NEL tunnel is 63,271 MNm2. The estimated Young's modulus of Jurong formation SV is 200 MPa. The Poisson’s ratio of the Jurong Formation is assumed to be 0.25 in the analysis.

Table 1. Tunnel lining parameter table

	External dia [mm]	Lining thickness [mm]	Lining width [mm]	Youngs Modulus of lining, Ec [MPa]
NEL Tunnel (Exiting tunnel)	6300	250	1200	28,000
TSL Tunnel (New tunnel)	6350	275	1400	32,000

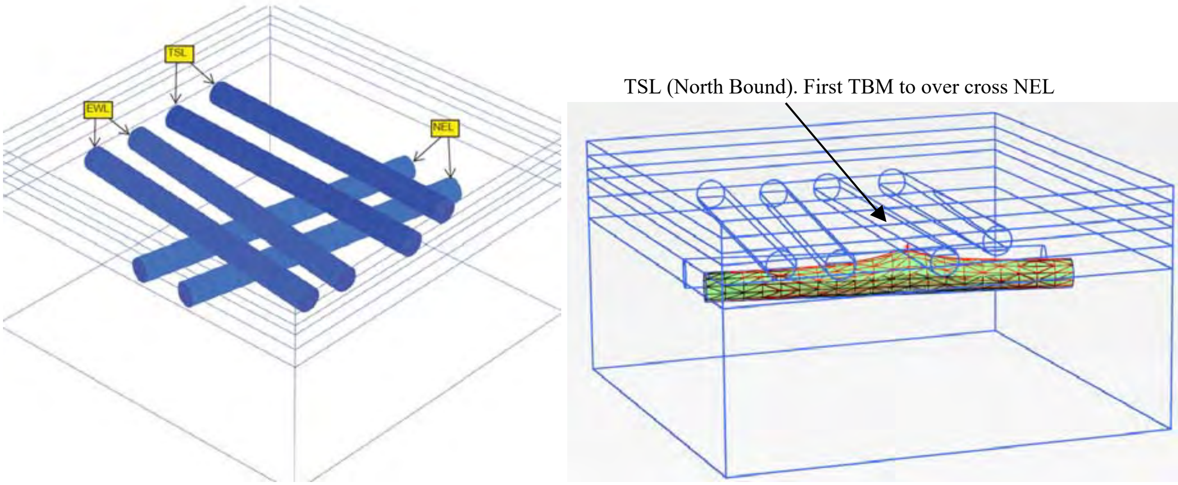

Figure 5. 3D numerical model indicating heave in NEL tunnel due to TSL tunnel excavation

Figure 6 presents the output which estimates maximum heave of 3.4mm due to single tunnel over-crossing NEL tunnel.

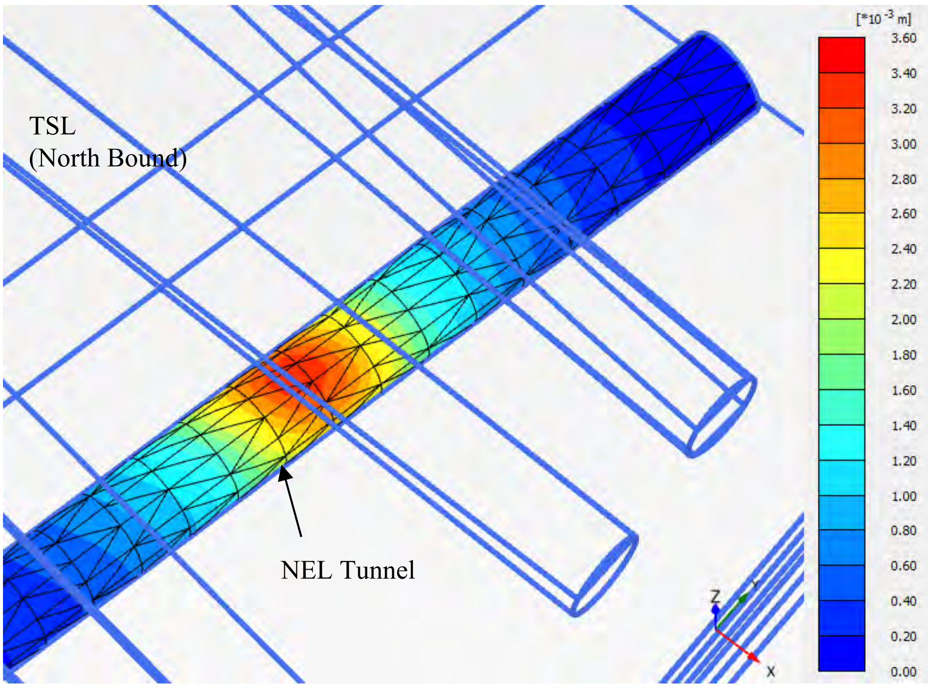

Figure 6. Maximum heave in NEL tunnel due to first TBM overcrossing

4 FIELD MEASUREMENTS AND COMPARISON

An array of 5 prism in cross section, is installed at every 3m along the NEL tunnel to observe the behaviour of NEL tunnel as the TSL tunnelling activity progresses above NEL tunnel. Figure 7 presents the comparison of estimated heave in NEL tunnel based on proposed simplified formula and 3D numerical model. In addition, actual observed heave in NEL tunnel, from instrumentation data is indicated as a scatter plot. It could be noted that the maximum heave predicted from FEM model and the simplified analytical model are within 10% range. However, the zone of influence is very narrow for the analytical model. FEM model suggests that NEL tunnel experienced heave for about 60m along the tunnel, however analytical model indicates a range of 40m. This is due to our assumptions in equation (6) but this is likely to be acceptable prediction for preliminary studies. When compared to actual heave from instrument readings, both the methods are found to be conservative. This conservatism stems from several aspects of finite element modelling of the actual situation. The two most important aspects of conservatism are identified to be due to the method of simulation TBM tunnel progress and due to conservative estimates of geotechnical parameters.

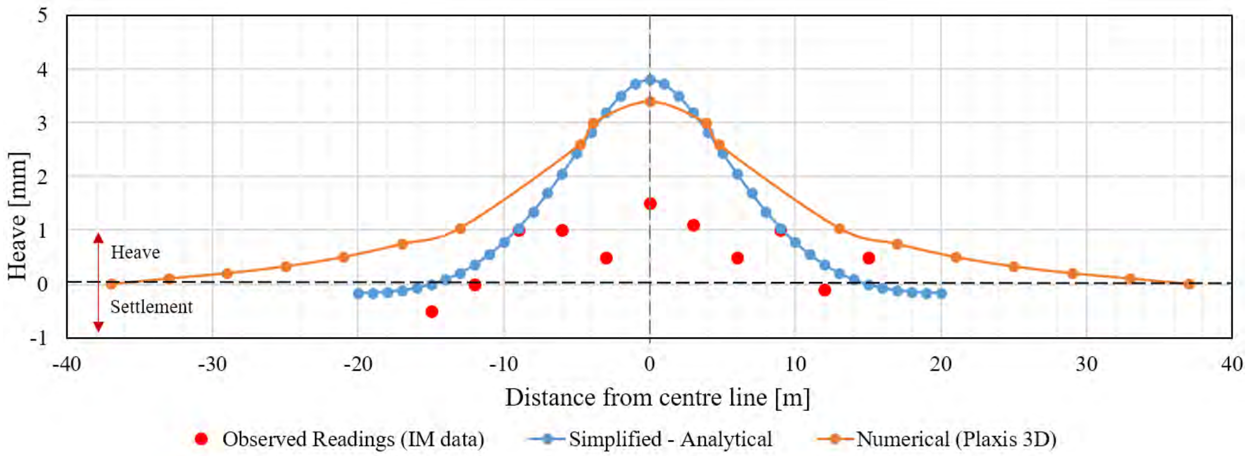

Figure 7. Comparison of numerical estimation, analytical estimation and actual heave readings from site

The same analytical formula could be used for multiple TBM tunnels over-crossing as well using the principle of superposition.

5 EFFECT OF SHALLOW TBM ON SURFACE HEAVE

In addition to the above observation regarding the performance of existing tunnels, the surface settlements / heave was closely monitored during the shallow TBM drive. The tunnel drive presented in the above case study is at overburden of less than 9m (i.e, less than 1.5 times diameter of TBM) and is soft ground (Estuarine and fill layer with SPT $N_{300} \sim 25$). Exactly above the overcrossing, sharp increase in heaving readings were observed. Figure 8 shows the location of LG marker with reference to TBM and existing tunnels (NEL tunnels). The heave location is the region of lowest overburden for this drive. After the TBM crossed the shallow overburden region, the heaving completely stabilized and started settling slightly. Figure 9 shows the timeline of instrument measurements for the instrument which has registered maximum readings. From the instrument readings it can be noted that because of overcrossing, higher face pressure was maintained to limit the movement of existing tunnels but this has caused slight heave at the surface level as the TBM is close to the surface. While the heave stabilized as the TBM passed this location, the heave slightly increased again (possibility due to tail void grouting pressure). The performance was quickly observed and the operating parameters for the next TBM drive was optimized. Further, the ground instruments were closely monitored to observe the progress of trend. Face pressure was reduced to the lower bound of approved face pressure and tail void grouting pressure was controlled to limit such future instances.

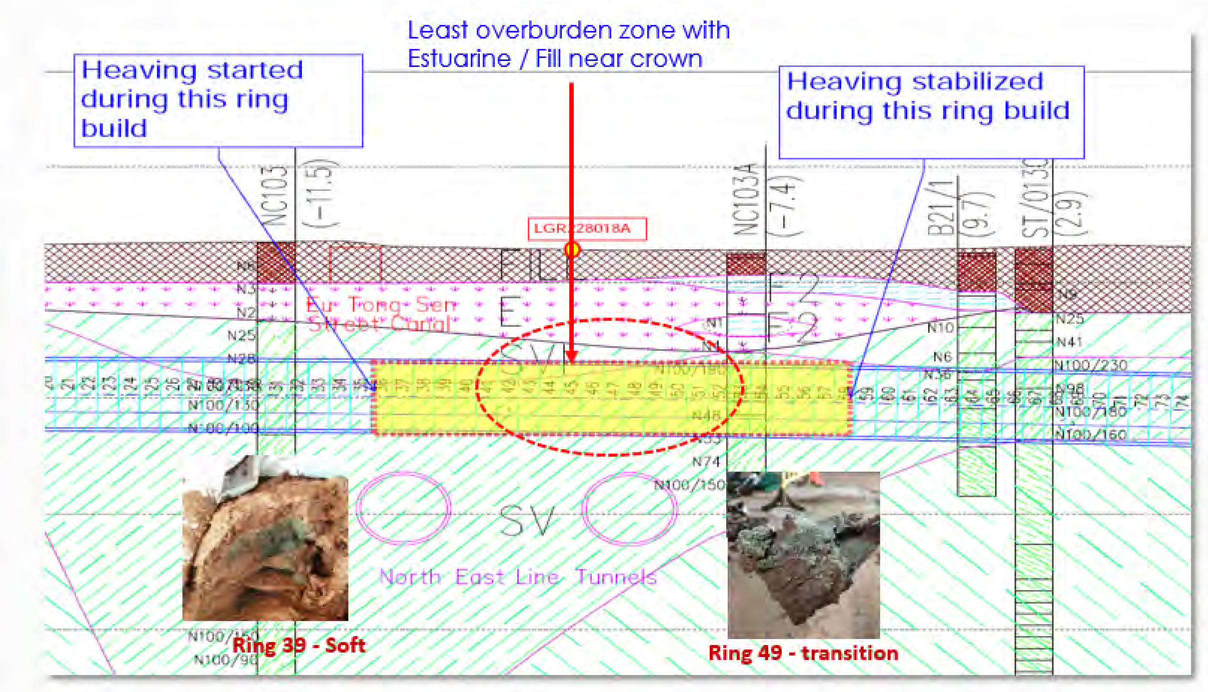

Figure 8. Relative location of existing tunnels, overcrossing TBM tunnel and surface instrument locations

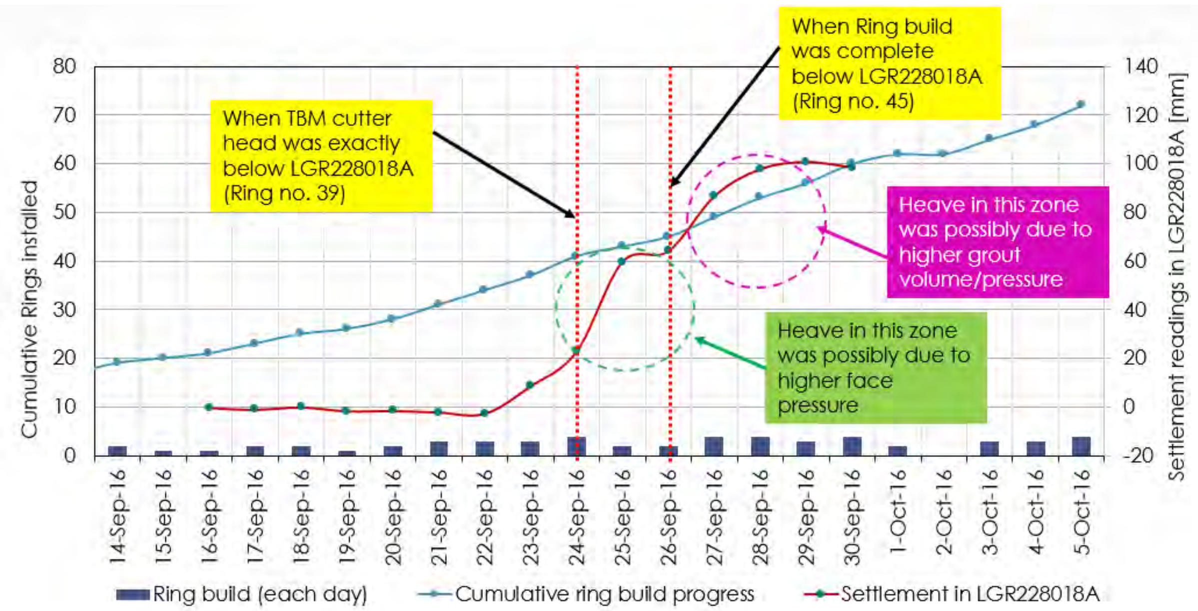

Figure 9. Timeline of heave in the instrument LGR228018A

6 CONCLUSIONS

The proposed simplified formula, which   Mindlin' solution and Euler-Bernouli theory gives a quick understanding of the interactions between the over-crossing tunneling and the existing tunnel in the primarily design stage. The formula is easy to incorporate in design routine and this quick assessment could be used to study the impact of clearance distance etc. In addition, using the principle of superposition, influence of multiple new tunnels could be quickly assessed. The applicability of formula is examined against a case study. The estimated results using the simplified formula are in reasonable agreement with the FEM results and conservative compared to actual field results. However, the method cannot be used to estimate the long term / time dependent movements of existing tunnel in cohesive soil and appropriate constitutive model of soil and FEM analysis should be applied for such problems. It could also be concluded that close monitoring and interpretation of surface and utility readings could be used to verify our assumptions and refine the TBM parameters for the subsequent drives.

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