Stacked Tunnelling Induced Surface Settlements in Soft Soil – A Case Study from Singapore

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ABSTRACT: *The growing demand for public and sustainable transportation in heavily urbanized areas like Singapore requires construction of an increasing number of Metro Lines. Complicated technical challenges are associated with such an activity and settlement prediction models play a key role in assessing the tunnelling-related risk assessments and planning mitigation techniques. Although a significant amount of research has been performed to study the settlements induced by side-by-side twin tunnels, the settlement prediction of stacked tunnelling has been, relatively, limited. In this paper, stacked tunnelling induced settlements are predicted using 2D numerical simulation. The extensive data from the instrumentation measurements collected during the construction of a stacked configuration tunnel in Singapore Downtown Line MRT tunnels have been used to study the validity of the principle of superposition for stacked tunnels. Key observations are presented by comparing empirical, numerical and actual field settlement data. Using the Monte-Carlo simulation technique, the empirical trough parameter is back-calculated to fit the actual settlements observed in Kallang formation. This could be used for future settlement predictions in similar ground conditions.*

KEYWORDS: Stacked tunnelling, soft soil, buildings, TBM, Monte-Carlo simulation

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

Singapore has been expanding the Mass Rapid Transit system in the recent years. The current project in progress is the Downtown Line (DTL) which is the Fifth MRT line in Singapore and upon completion it will be Singapore's longest automated underground line. This will connect the north western and eastern regions of Singapore to the Central Business District and Marina Bay area comprising of 32 stations and is 43 km long. The whole project is being constructed in three stages as shown in Figure 1.

Figure 1. Alignment of Downtown Line in Singapore MRT System.

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Currently, DTL Stage1 is completed and is in operation whereas DTL Stage 2 and DTL Stage 3 are still in the construction phase. Downtown Line Stage 3 is divided further into 3 Packages. This paper discusses the stacked tunnelling induced settlement at the DTL Stage 3 Package A - Contract Bendemeer Station (C933). Contract C933 has 4 EPB drives from the Bendemeer station, 2 drives each on east and west side. The Bendemeer station is being built under the Kallang Bahru Road and will serve commuters accessing to the commercial and industrial buildings in the area. The tunnel alignment on the west drive (towards Jalan Besar station) runs under a number of public roads including the Jalan Besar, the Lavender Street etc. whereas the east side has to undercross the Kallang River. Due to various site constraints along the alignment a significant section of these tunnels have to be stacked. The ground conditions show both tunnels are in Old Alluvium (OA-A) strata and Kallang strata on east side (as shown in Figure 2). Because of the sensitive ground and adjacent critical structures along the alignment, an extensive instrumentation regime has been implemented. Location of instrumentation arrays installed along the alignment is shown in Figure 3.

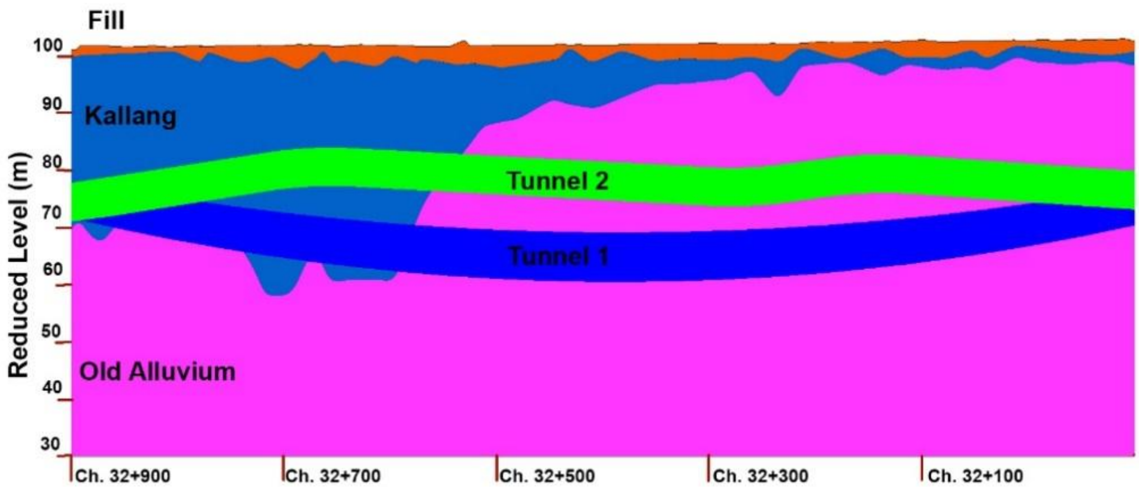


Figure 2. Geological profile along the alignment.

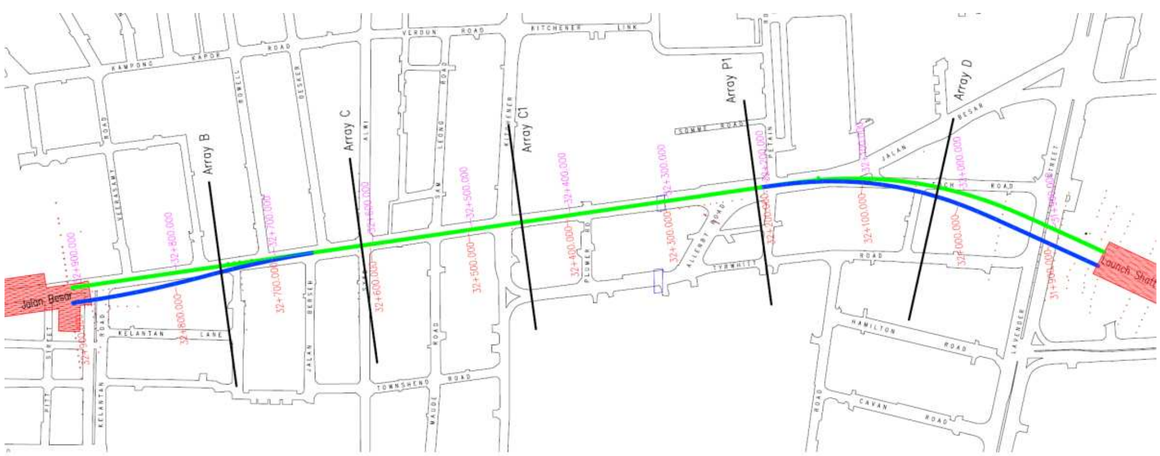


Figure 3. Location of monitoring instrumentation arrays along the alignment.

TBM FACE PRESSURE

Figure 4 shows the target face pressure and actual pressure along the alignment for Tunnel 1 (Bukit Panjang bound tunnel) and Tunnel 2 (Expo bound tunnel). Since the overburden of the two tunnels is different, normalized face pressure is presented in Figure 5 for comparing the TBM face pressure in both drives. Table 1 shows the face pressure at different Chainage locations together with the nearest instrumentation array.

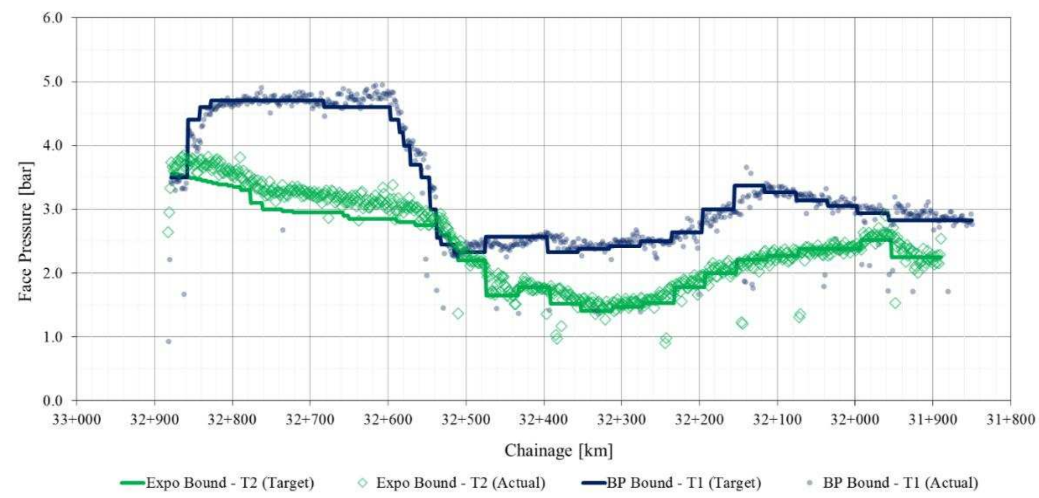


Figure 4. Face Pressure Plot for Tunnel 1 and Tunnel 2.

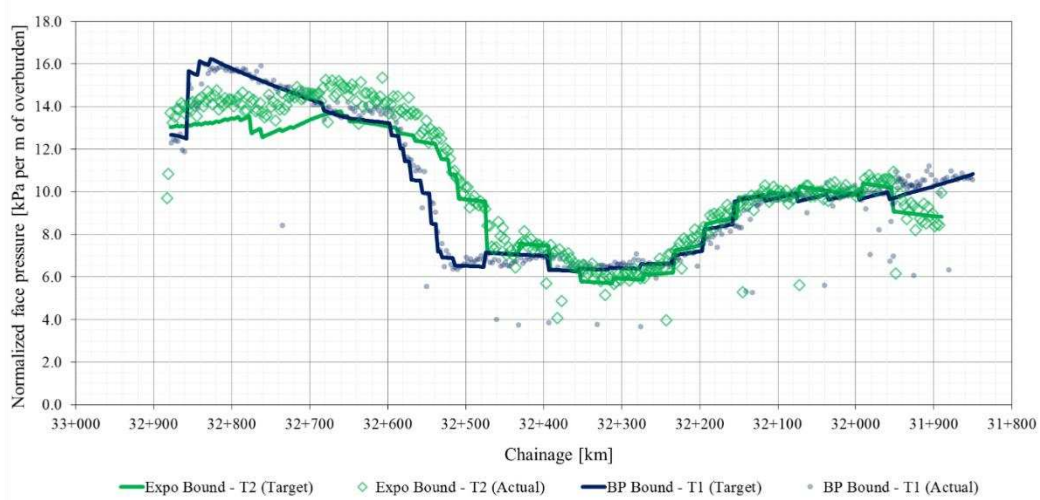


Figure 5. Normalized face pressure.

Table 1. Details of TBM Face Pressure at array locations.

	Ring No.	Instrumentation Array	Actual Face Pressure [bar]
BP Bound (Tunnel 1)	142	D	3.1
	255	P1	2.65
	438	C1	2.5
	488	C2	2.6
	545	C	4.8
	635	B	4.7
Expo Bound (Tunnel 2)	115	D	2.4
	215	P1	1.9
	398	C1	1.7
	448	C2	2.5
	505	C	3.1
	611	B	3.1

EVALUATION OF SETTLEMENTS

Empirical Methods

With recent advancements in numerical modelling capability, finite element modelling has become a very popular tool in prediction of tunnelling induced ground settlement. However, the use of numerical modelling always required understanding of constitutive material model selection to represent soil behaviour. Thus, there has always been an emphasis on simple, but reliable, methods for estimation of tunnelling induced settlements. The empirical formulation commonly utilised in engineering practice is developed by Peck (1969) and Schmidt (1974). Peck (1969) and New and O'Reilly (1992) assumed that the transverse ground settlement trough can be reasonably represented by a Gaussian (or Normal) distribution curve as shown in Figure 6.

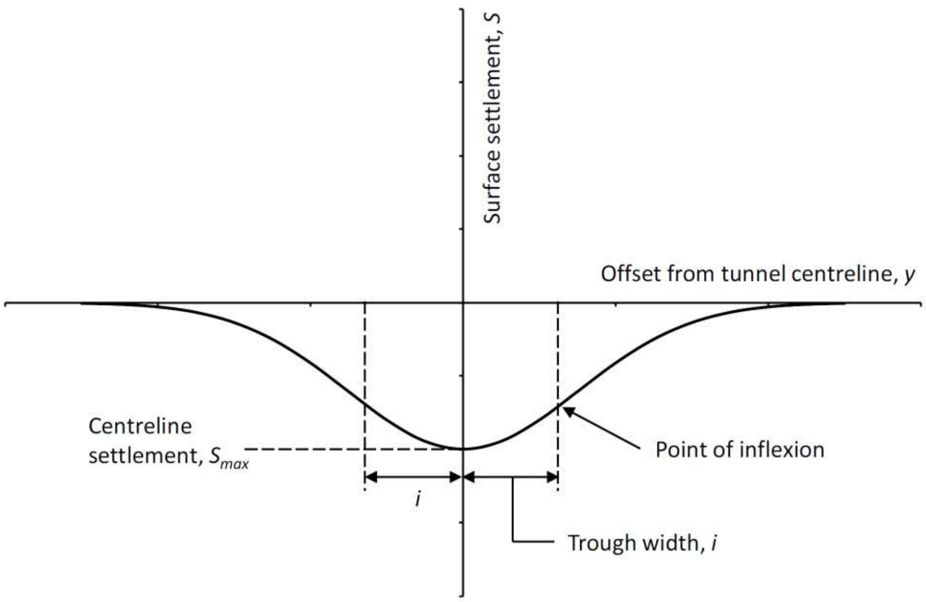


Figure 6. Ground settlement trough induced by a single tunnel.

From the Gaussian distribution curve, the empirical equation for settlement is:

$$S = S_{max} . e^{\frac{-x^2}{2i^2}} \qquad (1)$$

where S is the ground surface settlement at a distance 'x' from the tunnel centre, Smax is the maximum ground settlement at the vertical tunnel axis. Trough width parameter i, is the distance from the tunnel centre line to the point of inflexion of the trough. Mair et al. (1993) suggested that the total half width of the settlement trough is about 2.5i.

As an improvement to Peck's empirical method, Sagaseta et al. (1980) proposed a modified Peck's error curve to consider some additional tunnel and ground parameters like Young's modulus and Poisson's ratio. Romo and Diaz (1981) proposed an empirical formula derived from finite element analysis which considers additional parameters like horizontal stress, face pressure acting at the excavation face and the average value of strain at failure (Romo and Diaz, 1981).

Analytical Methods

Sagaseta et al. (1987) proposed a closed form solution by combining fluid flow with elastic solution for half space. This method allows the strain field evaluation in an initially isotropic and homogeneous incompressible soil. Later, Booker and Verruijt (1998) presented an analytical solution using generalisation of the Sagaseta's solution. This method allows for the computation of surface vertical displacements as well as vertical displacements at different depths below ground level and horizontal displacements along a cross-section.

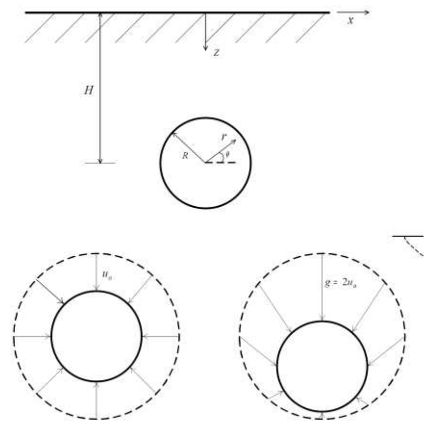


Figure 7. Ground deformation (a) Uniform radial displacement (b) Oval shaped radial displacement.

Loganathan and Poulos (1998) presented a modified form of Booker and Verruijt (1998) by suggesting the use of a modified equivalent ground loss parameter. The surface settlement and lateral displacement can be obtained as

$$Lateral\ displacement\ (u_{x,z=0}) = -\frac{4gR+g^2}{2}e^{\left\{-\frac{1.38x^2}{(H+R)^2}\right\}}\frac{x}{x^2+H^2} \quad (2)$$

$$Surface\ settlement\ (u_{z=0}) = \frac{4gR+g^2}{2}e^{\left\{-\frac{1.38x^2}{(H+R)^2}\right\}}\frac{H}{x^2+H^2} \quad (3)$$

where, g is the gap parameter, H is the depth of tunnel from the ground surface, R is the radius of the tunnel, z is the depth measure from the ground surface and x is the lateral distance from the tunnel centre line. Recent methods like Pinto and Whittle (2014) have proposed simplified analytical closed form solution to account for soil plasticity in the analysis.

Numerical Methods

The tunnelling process is a three dimensional problem which involves stress change and deformation in all three directions. However for the simplified modelling, it is reasonable to assume a plane strain or two dimensional model for a long tunnel section in studying tunnelling induced settlements. In order to simulate tunnel excavation in 2D FE analysis, the effect of the missing third dimension has in some way to be included. Figure 8 shows the 3D arch around the unsupported tunnel heading by displaying rotated principal stress directions. This arch is able to carry the vertical ground loads by transferring them around the unsupported cut stretch.

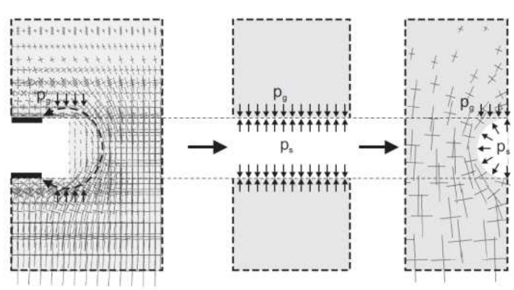
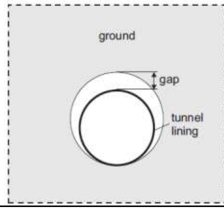
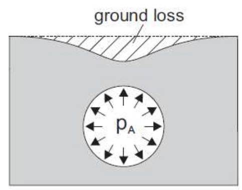
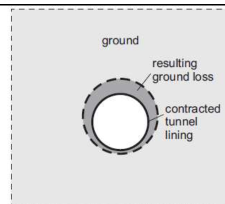
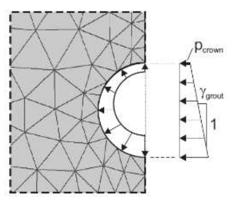


Figure 8. 3D arch support and 2D FE approximation with support pressure.

When using 2D numerical analysis to simulate tunnel excavation, consideration must be given to the ground in front of the shield machine which will move both radially and axially towards the tunnel face. To simulate shield tunnelling 2D FE analysis are frequently used and different methods are used to simulate the ground loss and settlement (Möller 2006). Table 2 lists the different methods possible for simulating the TBM installation in 2D model. Detailed explanation of all the methods are beyond the scope of this paper. In this study, the grout pressure method is found suitable for estimating the settlement troughs based on TBM face pressure and is adopted for our analysis.

Table 2. 2D FEM simulations methods.

Method	Scheme	Reference
Gap Method		Rowe and Kack (1983)
Stress Reduction method		Addenbrooke et al. (1997)
Contraction method		Brinkgreve (1993)
Grout pressure method		Möller (2006)

Mohr-Coulomb's failure criterion is used to model the behaviour of soils involved in the analysis. The soil parameters used for this study are based on the C933 contract Geotechnical Interpretive Baseline Report (GIBR). The soil parameters used in the PLAXIS analysis are tabulated in Table 3.

Table 3. Soil parameters used in this study.

Soil layer	Unit weight [kN/m^3]	Young's modulus, E [MPa]	Cohesion c' [kPa]	Angle of friction phi' [degree]
Fill	19	8	0	30
Kallang – Marine (Lower)	16.5	7.5	0	22
Kallang – Marine (Upper)	16.5	16.5	0	22
Kallang – Fluvial (Lower)	19	9.2	0	24
Kallang – Fluvial (Upper)	19	16.5	0	24
Old Alluvium – OA (A)	20	150	10	32

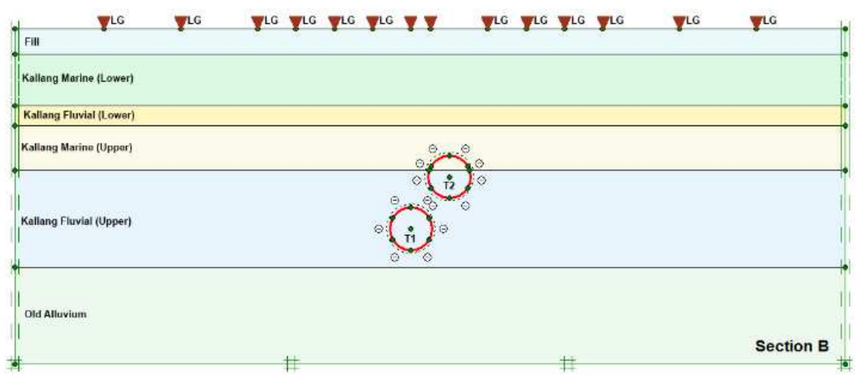
In order to back-assess the actual volume loss during the tunnel drive, the numerical analysis is carried out with the corresponding face pressure. Since the actual groundwater table during the TBM drive (observed in water stand pipes and piezometer) indicated that the water table was very closed to ground level, groundwater is considered at ground level in the

FEM model. After obtaining the initial stresses, all the nodal displacements were then reset to zero prior to the excavation of tunnels. The analysis for this study typically involved the following stages: Stage 1 involved initialization of the stresses. Stage 2 involved the excavation of the first tunnel and subsequent installation of its lining elements. The tunnel lining is simulated as flexible impermeable membrane (with low flexural and axial stiffness). The impermeable membrane ensures prevention of any dissipation of applied face pressure. The pore-water pressure inside the tunnel is modified to simulate the TBM face pressure (based on actual face pressure used for the TBM driving at that particular chainage). Stage 3 involved the excavation of the second tunnel and installation of its lining parameters. The pore pressure is similarly set to simulate the TBM operating face pressure. The settlement trough at the end of stage 2 and stage 3 is extracted from the PLAXIS output. This settlement troughs are integrated to obtain the equivalent volume loss caused by Tunnel 1 and combined volume loss caused by Tunnel 1 + Tunnel 2. The above simulation could have been performed by using tail void grout pressure instead of TBM operating pressure but it was noted that grout pressure data at certain location spiked and were sporadic due to clogging and various other operational issue and hence not considered in this study.

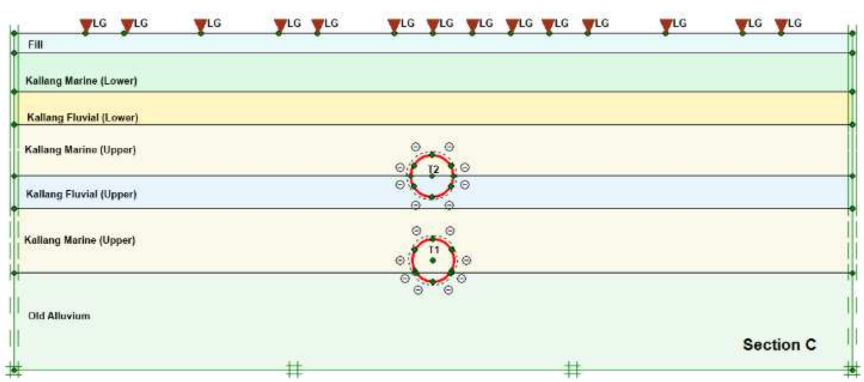
Actual Settlement (Field Measurements)

A total of six monitoring arrays were installed at ground surface along the stretch under discussion. Each array consists of 11 to 16 settlements markers used to monitor the transverse ground settlement during and after the excavation of the stacked tunnels. The location of these settlement markers are shown in Figure 9. Maximum field measurements recorded 1-2 days after the TBM has crossed the monitoring array has been considered as the instantaneous settlement caused due the TBM crossing (i.e, the TBM just crossed the monitoring arrays when the readings were recorded for this analysis).

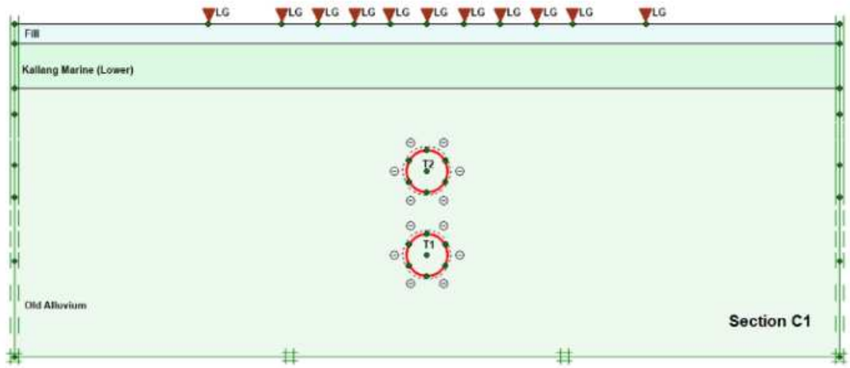
The development of transverse ground surface settlement troughs of each array during the tunnelling is shown in Figure 10. The maximum ground surface settlement observed from this study ranged from 5 mm to 15 mm. Comparison of these estimates are performed at the six array location along the stretch under study. The corresponding ground information and tunnel alignment data are represented in Figure 9.



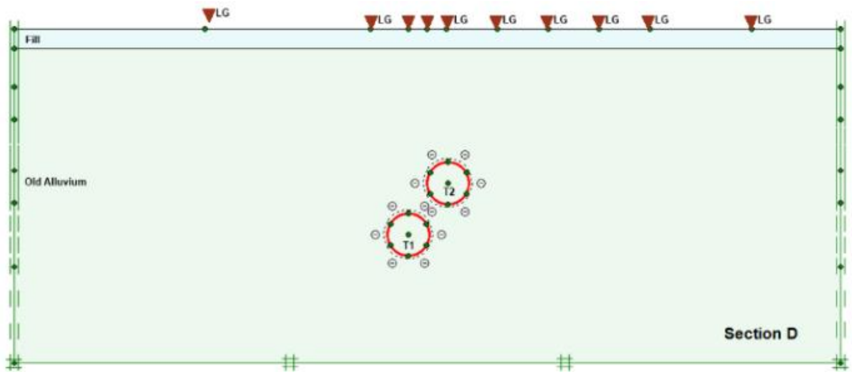
(a) Section B



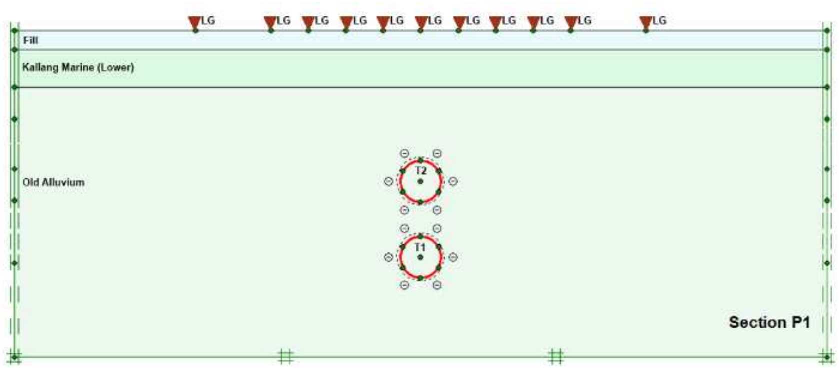
(b) Section C



(c) Section C1



(d) Section D

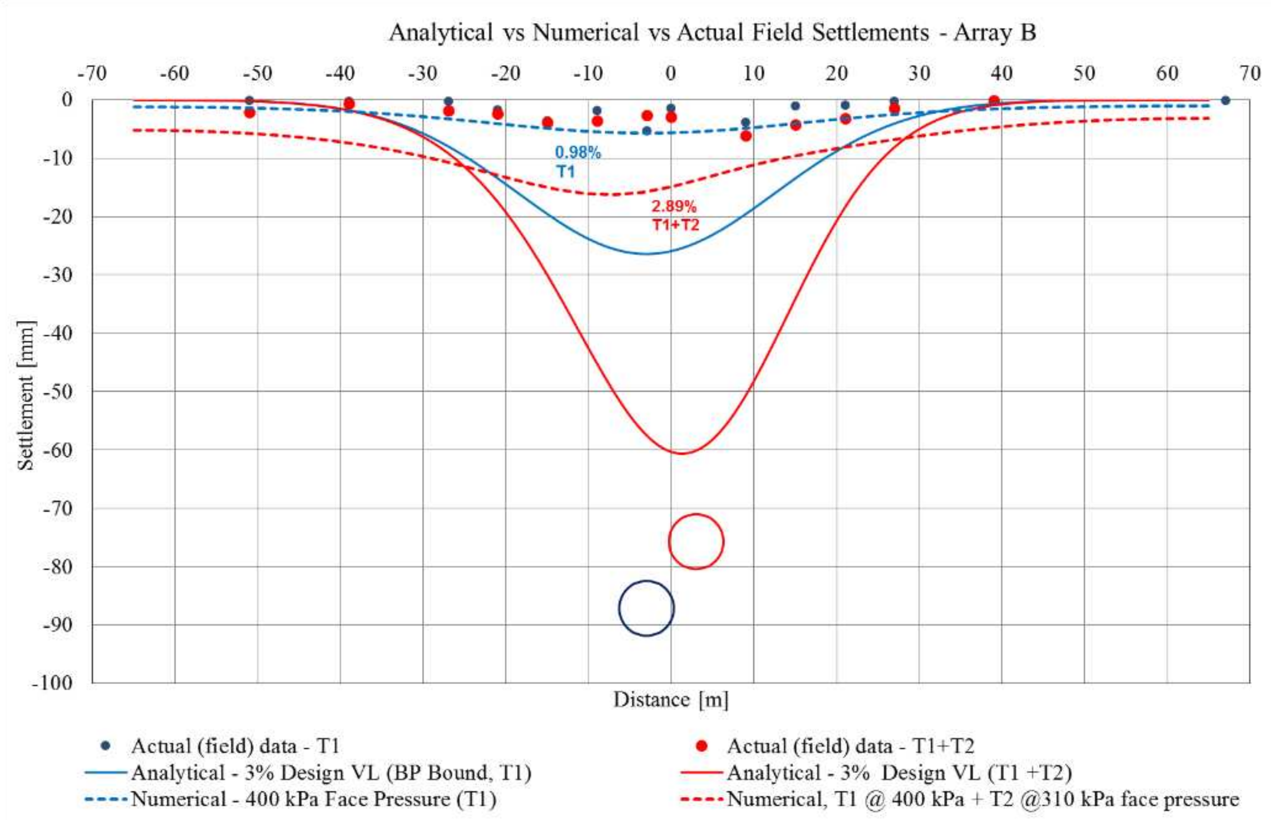


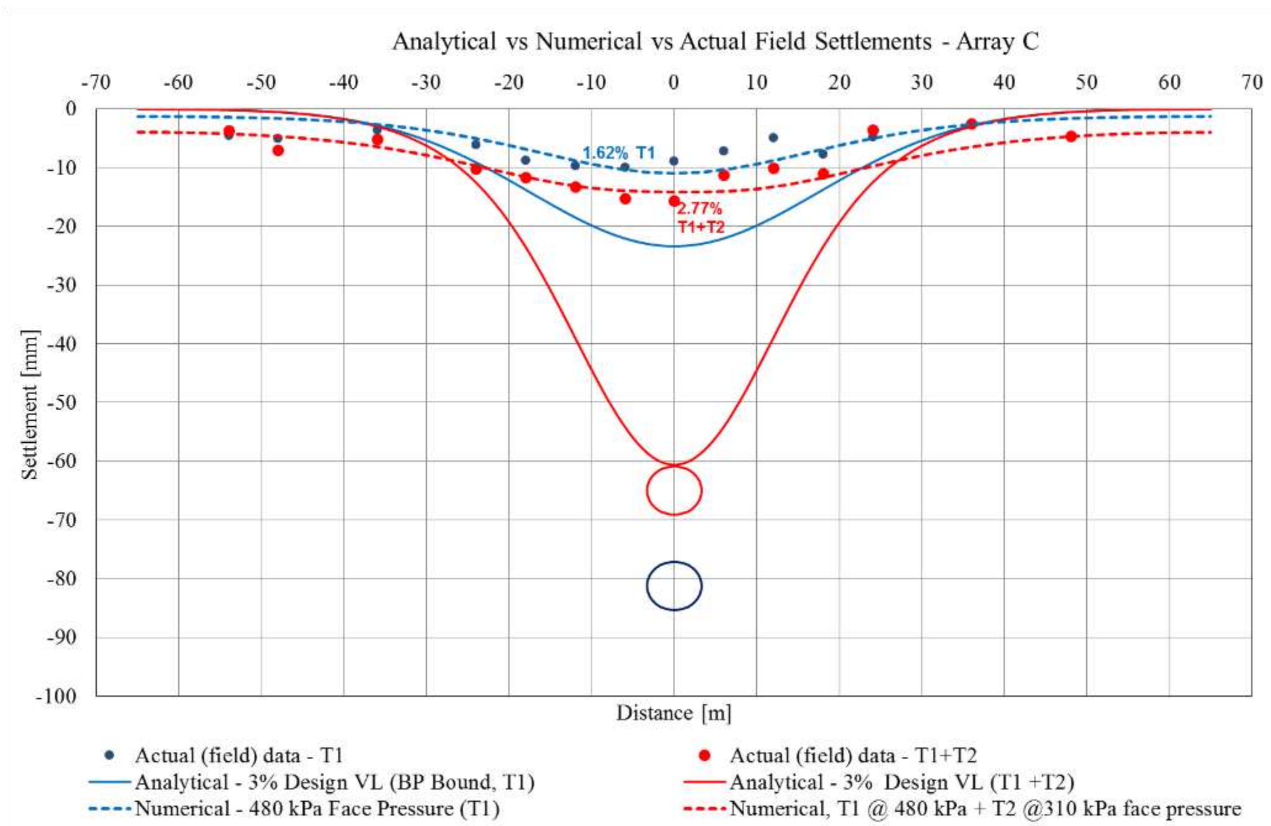
(e) Section D

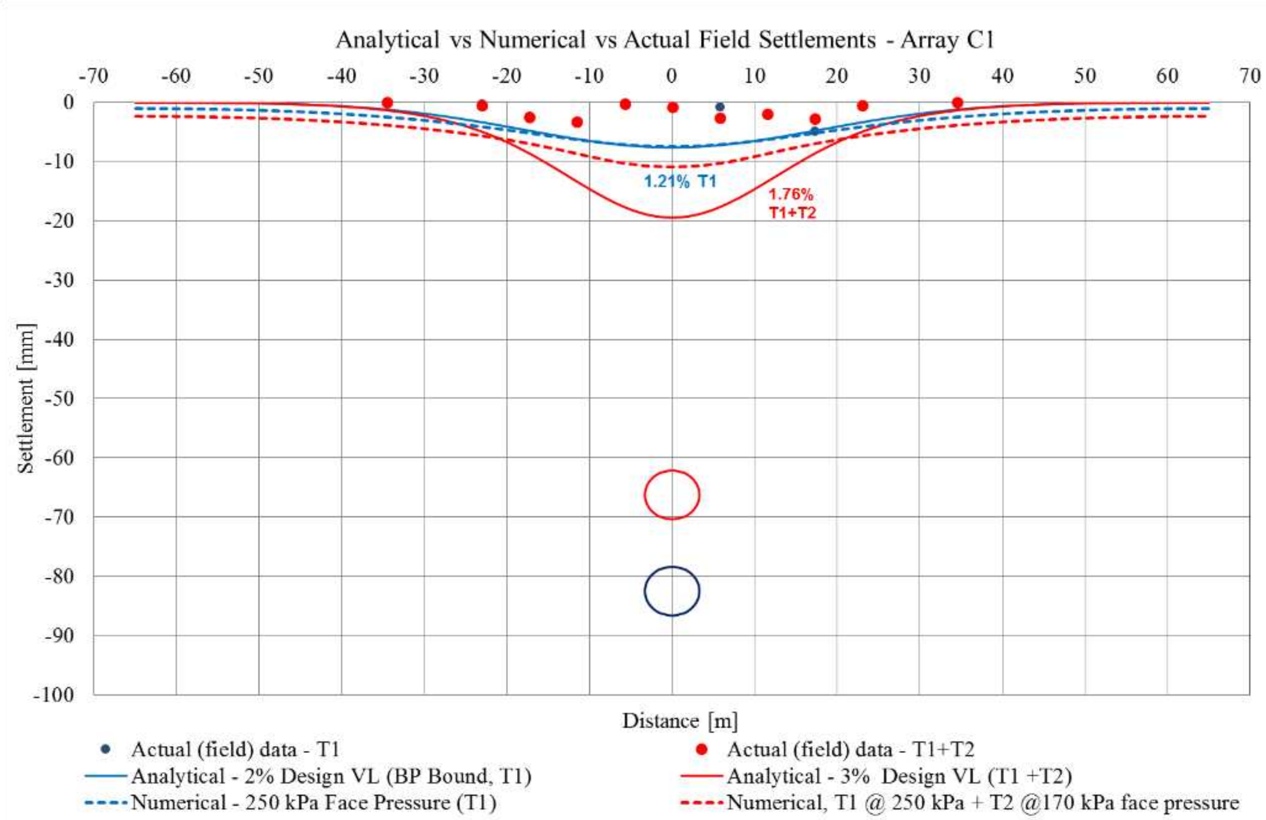
Figure 9. Ground information and array arrangement.

Table 4. Comparison of actual and design Volume Loss.

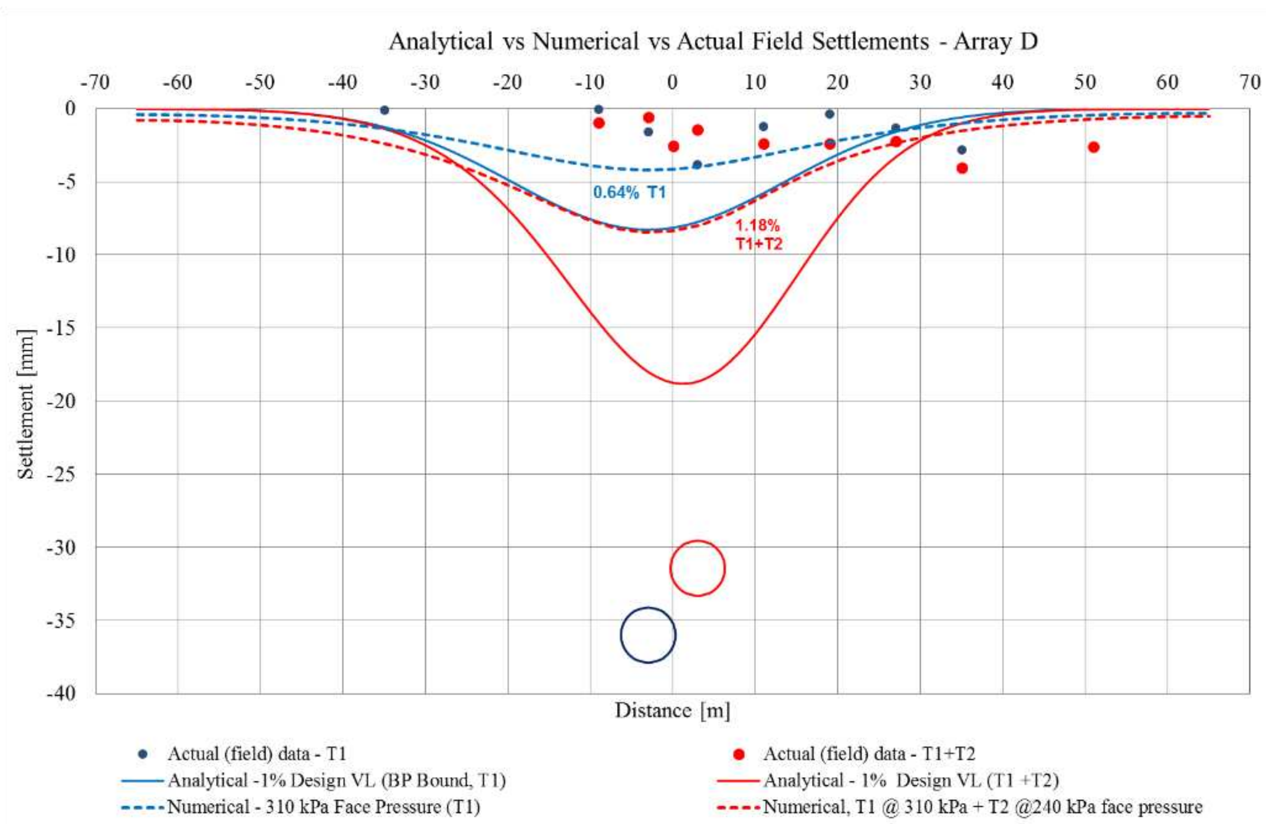
Array	Design Volume Loss [%]	Expected Volume loss for the applied face pressure [%]	Actual Volume Loss (back calculated based on actual trough) [%]
		Numerical	Observed
B	3	1.5	0.5
C	3	1.4	1.24
C1	1	0.88	0.12
D	1	0.6	0.1
P1	1	0.75	0.17


Analytical vs Numerical vs Actual Field Settlements - Array B
0.98%
T1
2.89%
T1+T2
Settlement [mm]
Distance [m]
Actual (field) data - T1
Actual (field) data - T1+T2
Analytical - 3% Design VL (BP Bound, T1)
Analytical - 3% Design VL (T1 +T2)
Numerical - 400 kPa Face Pressure (T1)
Numerical, T1 @ 400 kPa + T2 @310 kPa face pressure


Analytical vs Numerical vs Actual Field Settlements - Array C
1.62% T1
2.77%
T1+T2
Settlement [mm]
Distance [m]
Actual (field) data - T1
Actual (field) data - T1+T2
Analytical - 3% Design VL (BP Bound, T1)
Analytical - 3% Design VL (T1 +T2)
Numerical - 480 kPa Face Pressure (T1)
Numerical, T1 @ 480 kPa + T2 @310 kPa face pressure



Analytical vs Numerical vs Actual Field Settlements - Array C1
Settlement [mm]
Distance [m]
1.21% T1
1.76% T1+T2
Actual (field) data - T1
Actual (field) data - T1+T2
Analytical - 2% Design VL (BP Bound, T1)
Analytical - 3% Design VL (T1 +T2)
Numerical - 250 kPa Face Pressure (T1)
Numerical, T1 @ 250 kPa + T2 @170 kPa face pressure



Analytical vs Numerical vs Actual Field Settlements - Array D
Settlement [mm]
Distance [m]
0.64% T1
1.18% T1+T2
Actual (field) data - T1
Actual (field) data - T1+T2
Analytical -1% Design VL (BP Bound, T1)
Analytical - 1% Design VL (T1 +T2)
Numerical - 310 kPa Face Pressure (T1)
Numerical, T1 @ 310 kPa + T2 @240 kPa face pressure

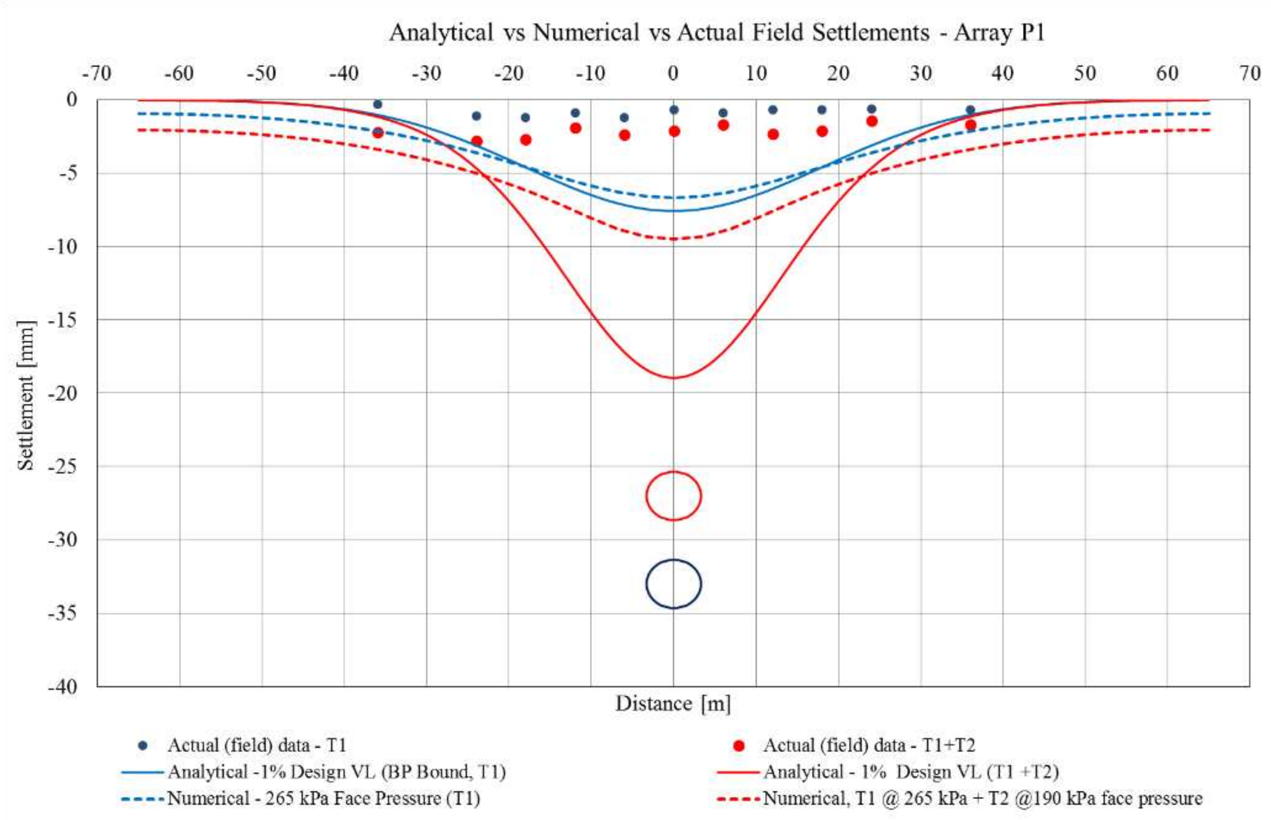
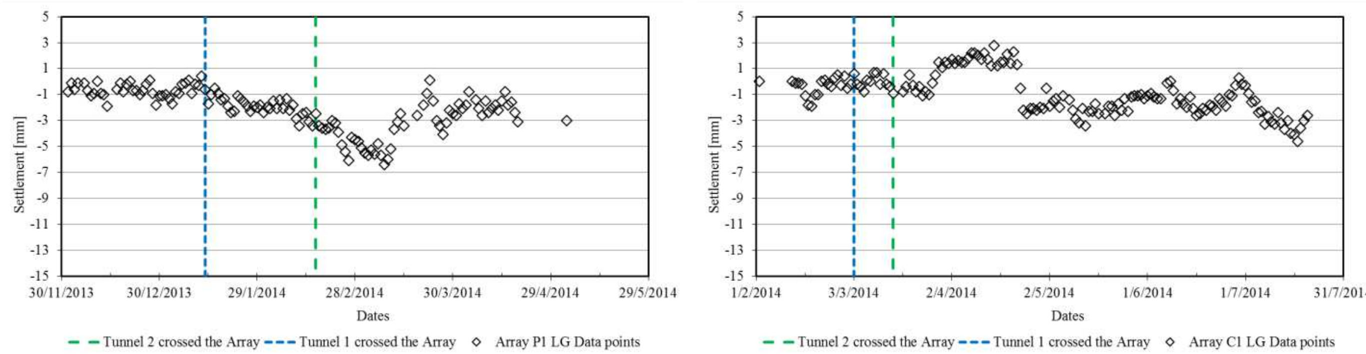


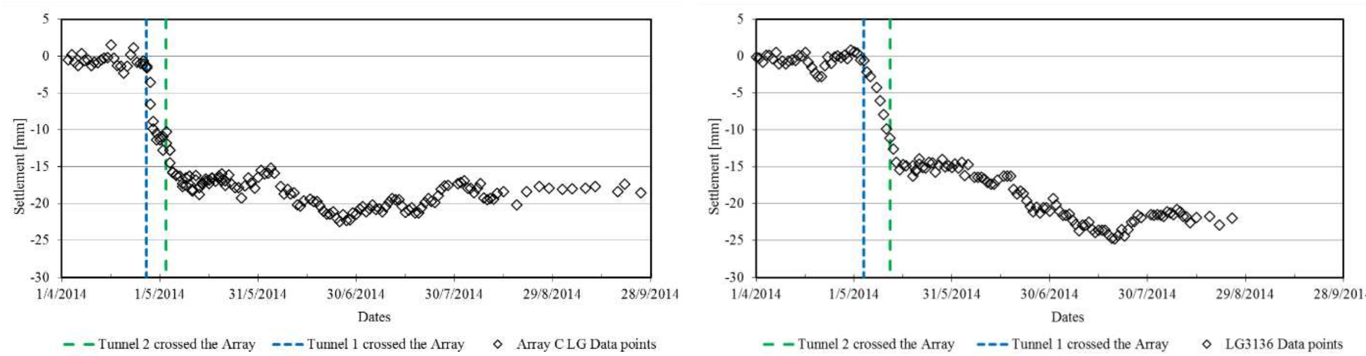
Figure 10. Comparison of Empirical results, numerical results and field measurements.

Figure 10 shows settlement predictions based on the empirical method, numerical method and the actual field settlement data. The empirical estimate of settlement pattern is shown in solid line (blue and red, blue line indicates the settlement due to T1 alone, whereas red line indicates the combined settlement due to T1 and T2). Dotted line represents the settlement trough from numerical simulation. Actual field settlement data are shown as points – scatter plot. Table 4 presents the comparison of design volume loss, expected volume loss and actual volume loss. Expected volume loss is calculated by integrating the settlement trough obtained from the numerical analysis. The design volume loss presented in second column of Table 4 is the volume loss considered for the emperical settlement prediction.

Among the five array sections studied, three sections (Array C, C1 and P1) are the sections in which the tunnels are stacked nearly one above the other. Maximum surface settlements at these sections over a period of six months are studied and presented in Figure 11. The settlement vs time plot for the tunnels passing through old alluvium (OA-A) soil (array P1 and C1) do not indicate any particular pattern. However, the settlement vs time plot in Kallang formation (array C) clearly indicates that most of the ground movement occurs immediately after mining of first tunnel. Longitudinal ground settlement profile in Kallang formation is represented using scattered plot in Figure 12. Positive X axis indicates distance behind the TBM cutter head and Negative X axis indicated distance ahead of TBM cutter head (i.e, the TBM is considered to move from right to left direction). Scattered data on left side of the vertical line (monitored section) indicates settlement observed ahead of the TBM. From the (polynomial) trend line, it can be clearly noted that the ground settlement starts significantly ahead of the TBM excavation face and the settlement continues even after the TBM has passed the monitored section.



(a) In Old Alluvium (OA-A)



(b) In Kallang Formation

Figure 11. Maximum surface settlement over time recorded by various arrays.

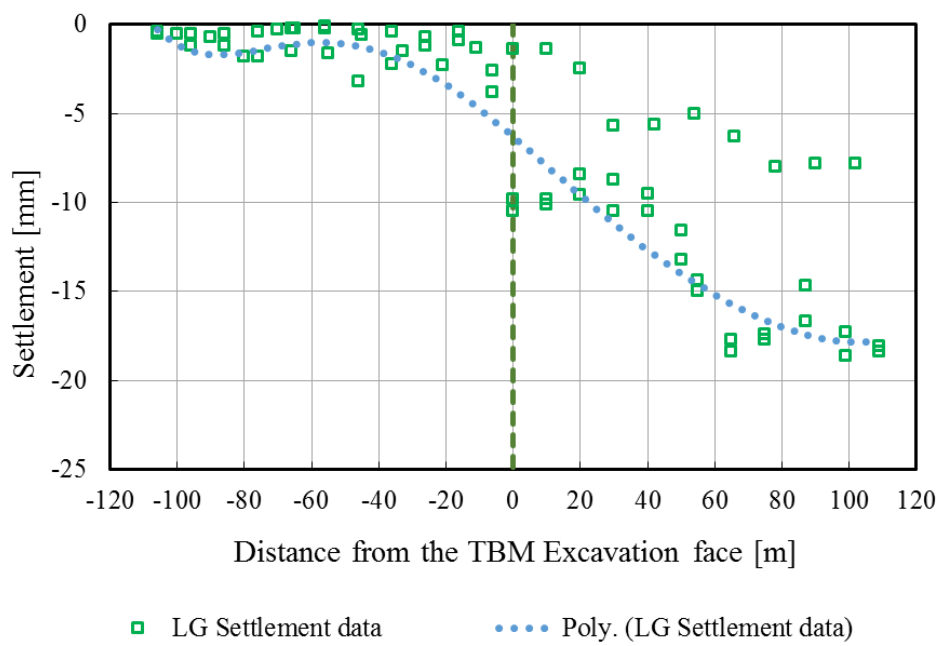


Figure 12. Surface settlement ahead of TBM excavation face.

RESULTS AND DISCUSSION

Stacked Tunnel Settlements

The results presented in Figure 10 indicate the surface settlement trough using numerical analysis and Peck's formulation. In general, the trend of the settlement trough is similar in all the scenarios. Most of the ground movement seems to occur during the excavation of the first tunnel and the total volume loss is observed to be less than the summation of the volume loss of the two tunnels separately. Peck's empirical estimation (for the given trough parameter), which are based on the Gaussian form, seem to overestimate the settlements. Numerical analysis for the exact TBM face pressure predicted settlement values which are similar in magnitude to the actual field measurements.

The width of the settlement trough is more appropriately predicted by the numerical analysis because the exact face pressure is used to predict the settlement pattern. It is to be noted that Rowe and Cack (1983) reported that the numerical analysis overestimates the width of the settlement trough if the settlement trough is calculated based on the contraction parameter in PLAXIS software. It is demonstrated in this study that the use of actual face pressure in estimating the deformations results in realistic settlement trough.

Applicability of Peck's Empirical Formula

This study examines the validity of the principle of superposition and appropriateness of the trough parameter (k) prescribed in LTA CDC guidelines for the encountered soil in this project. Although Peck's equation is more superior in estimating the transverse ground surface settlements induced by tunnelling than the cumbersome FE analysis (Chen et al. 2012), the choice of trough parameters plays a key role in estimating the settlement pattern which is closer to reality. In the present analysis, the trough parameter is set as per LTA CDC guidelines and it is observed that the assumed trough parameter (k) does not represent a realistic settlement trough. Available field data is used to curve fit and back-calculate the appropriate parameter for the Kallang formation and presented in Table 5. In order to achieve a good Gaussian curve fit, Monte Carlo simulation with a large number of fictitious k values are performed to determine The root mean square error (RMSE) for each of the fictitious k values. The median value of k with the least error is chosen as the appropriate trough parameter for the selected soil type.

Table 5. Recommended trough parameter k.

Soil layer	Recommended trough parameter	LTA CDC Guidelines
Kallang Formation	0.7	0.5

Excavation Parameters Influencing Surface Settlements

The role of TBM operating pressure and grouting pressure in the volume loss has been reported in recent projects (Fargnoli et al. 2013, Russo et al. 2013, Shirlaw et al. 2002 and Dimmock et al. 2002). In this study, an attempt was made to explore the relationship between volume loss and face pressure (based on TBM operational parameters in Kallang formation). Since the face pressure and grouting pressure have similar effect in volume loss control, the data points were combined to plot this trend. The data is mostly dispersed and no particular conclusion could be drawn based on our limited data; hence is not included in this paper. Usually higher face pressure / backfill grout pressure (for a given ground condition) will lead to lower volume loss.

CONCLUSION

This paper focused on the settlements of stacked tunnels of C933 Contract in Downtown Line 3 in Singapore MRT System, constructed using EPB machine. A series of ground settlement analyses have been carried out for a stacked tunnel using an empirical equation (that assumed principle of superposition) as well as numerical simulation and were compared with the actual field settlement data. Key observations are presented based on this comparison. It is demonstrated that the use of TBM face pressure in the numerical simulation of settlement troughs (i.e using "grout pressure" method instead of empirical method) results in realistic settlement prediction. Further, based on the field settlement data and using Monte-Carlo simulation technique, the empirical trough parameter (k) is back-calculated to fit the actual settlements observed in the Kallang formation. This could be used for future settlement predictions in similar ground conditions. Further studies using 3D numerical simulation to consider settlements due to overcut and consolidation of tail void grout could be considered to refine these observations.

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