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Soil displacements due to TBM tunnelling in Ho Chi Minh city – Vietnam

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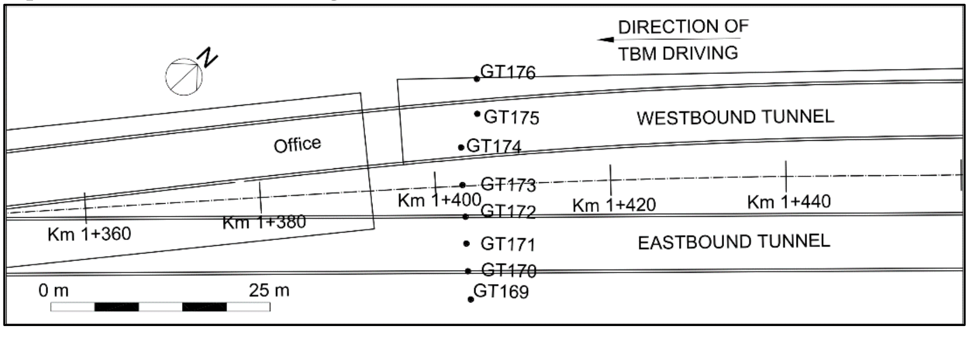
ABSTRACT: The limited space available above ground in Ho Chi Minh city, the largest city of Vietnam, is no longer adequate for new roads to cope with the growing volume of traffic. This congested situation has been addressed, in part, by new tunnels constructed in the heart of the city for a mass transit system with new metro lines. The tunnelling works started in May 2017 and were completed in June 2018. The 781m long tunnel is the first tunnel that has been constructed by TBM in an urban area in Vietnam. The event offers a unique opportunity to gain data and experience for future tunnelling projects in Vietnam. The paper presents technical data including soil conditions, tunnel geometries, adopted construction techniques, monitoring instrumentation and soil displacements from the East bound tunnel excavation.

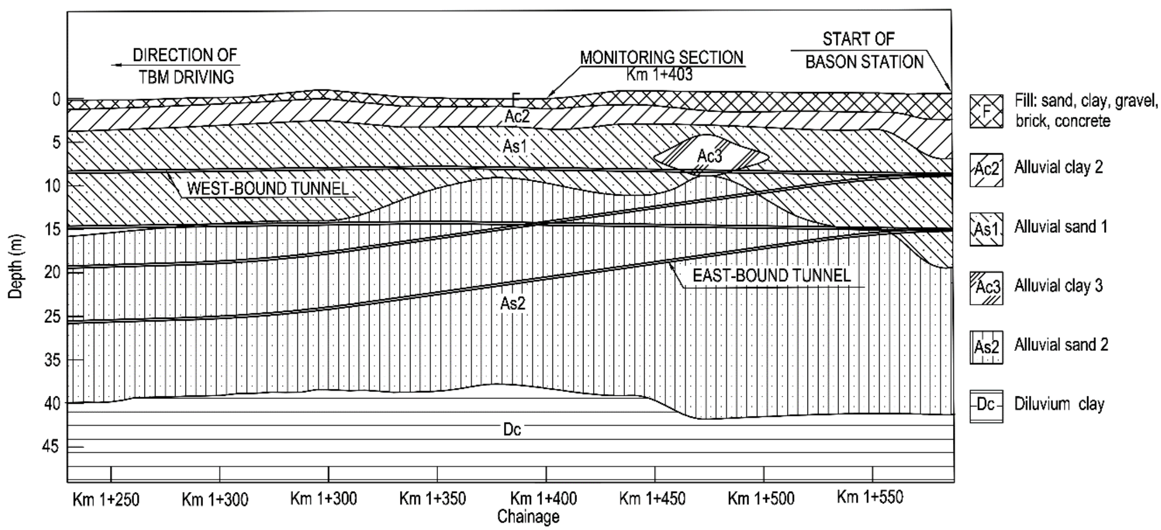
1 INTRODUCTION

The tunnel section belongs to the line number 1 in the metro system of Ho Chi Minh city Vietnam as an effort by the government to improve the capacity of public transport, alleviate congestion and reduce traffic jams. The total length of the line is 19.7km which includes 781m of twin tunnels.

Figures 1a and 1b present the plan and longitudinal profile of the tunnel along with the monitoring points and geotechnical data. The East-Bound (EB) tunnel (Figure 1) was constructed first and related field data at the section km 1+403 will be presented and discussed in this paper. At this section, the tunnel depth is $z_0 = 17.6m$. The external and internal diameters of the tunnel are 6.65m and 6.05m respectively.



a) The plan of the route of the tunnels and locations of monitoring points at chainage km 1+403.



b) Longitudinal profile of the tunnel route and geotechnical strata.

Figure 1. The route of the tunnels and geotechnical strata.

2 GROUND CONDITIONS

The geotechnical strata along the tunnel route comprises layers of Fill, Alluvial Clay, Alluvial Sand, and Diluvium clay. The detailed descriptions of the ground are presented in Figure 2. Most of the length of the twin tunnel lie in the sand layers (Figure 1b, Figure 2).

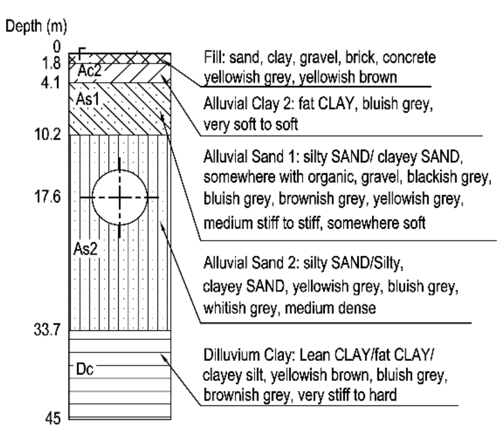


Figure 2. Detailed description of soil layers at section km 1+403.

3 CONSTRUCTION METHOD

An Earth Pressure Balanced (EPB) tunnel boring machine (TBM) was chosen to construct the tunnels as it uses techniques most suited to sandy ground conditions. The EPB TBM had an excavation diameter of 6.82m. The diameter of the shield was 6.79m and the overall length of the TBM, from excavation plate to tail, was 8.5m. The average excavation speed achieved was approximately 10m per day.

The precast concrete tunnel lining rings were formed of 5 segments plus a key-piece which all had a thickness of 300mm and a nominal length of 1.2m. These segments were erected inside the EPB TBM shield and when the TBM tail had advanced beyond the completed tunnel lining ring, grout was injected to fill the cavity between the tunnel extrados and the excavated soil so as to minimise soil settlement. The tail void grouting procedure together with the support pressure at the excavation face are essential to control soil settlement.

The support pressure, used to balance soil and water pressures, is provided by the thrust from the excavation face and the chamber pressure which is controlled by the volume of the soils and conditioning agents being fed into the chamber and the rate of spoils being discharged by the screw conveyor. The support pressure can be increased by reducing the rate of spoil discharging from the chamber while maintaining the advancing rate of the TBM and vice versa (Mair, 2008).

4 BACKGROUND TO GROUND SETTLMENT DUE TO TUNNELLING

Previous researchers (Martos, 1958; Peck, 1969; O'Reilly and New, 1982; Nyren, 1998; Dimmock, 2003; Wan et al, 2017a; Le and Taylor, 2018) demonstrated that the profile of tunnelling induced surface settlement has the shape of an inverse Gaussian curve (Figure 3) and can be described by Equation 1. The parameters in Equations 1 are depicted in *Figure 3.*

$$S = S_{max} exp\left(\frac{-y^2}{2i^2}\right) \quad (1)$$

where S is surface settlement, y is the distance from the tunnel centre line to the settlement point in the transverse direction, S_{max} is the maximum settlement (usually corresponding to $y = 0$), and i is the distance from the centreline to the point of inflexion in transverse direction.

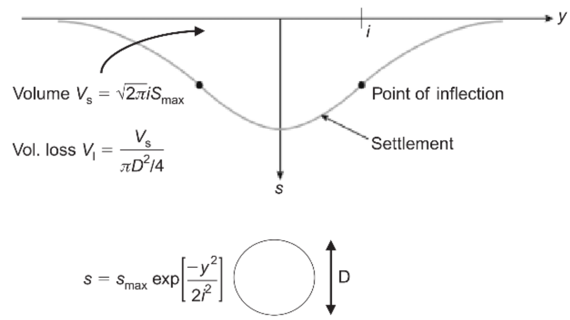


Figure 3: Usage of Gaussian curve to represent settlement trough (Mair, 2008).

By integrating Equation 1 with respect to y, the relationship between the area of the settlement trough, V_S (Figure 3), and S_{max} can be shown to be;

$$S_{max} = V_S / \sqrt{2\pi} i; \quad (2)$$

A commonly used value to indicate the ratio of the magnitude of the settlement trough, V_S, with the excavation area, V_{exc}, is the volume loss ratio, V_L;

$$V_L = V_S / (\pi \frac{D^2}{4}); \quad (3)$$

where D is the excavation diameter. In this paper, $D = 6.82m$.

Wan et al., (2017b) suggested 5 main components of volume loss in TBM tunnelling (depicted in Figure 4);

1) Face movement;
2) Over-excavation;
3) Shield tapering;
4) Tail void closure
5) Lining deformation

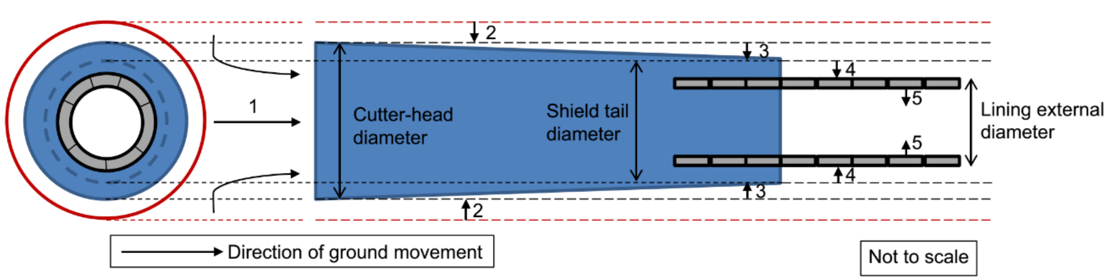


Figure 4. Components of volume loss in TBM tunnelling (Wan et al., 2017b).

Attewell and Woodman (1982) found that that the ground settlement directly above the tunnel excavation face is approximately $0.5S_{max}$ for tunnels in stiff clay without face support. For TBM tunnelling in which face support is provided, previous researchers (Mair and Taylor, 1997; Wongsaroj et al., 2006; Sugiyama et al., 1999) observed that the vertical displacement directly above the tunnel excavation face is considerably less than $0.5S_{max}$ (Figure 5).

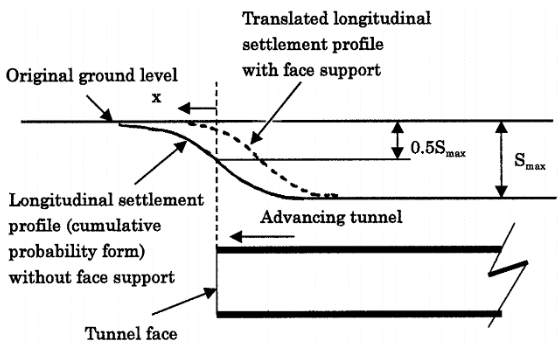


Figure 5. Longitudinal surface settlement troughs with and without face support (after Mair and Taylor, 1997)

Leca and New (2007) suggested the contribution of the mentioned components, in TBM tunnelling, to the total ground settlements as below;

- Face intake (component 1): 10 to 20%; this component is dependent on the support pressure at the excavation face.
- Along the shield body (components 2 and 3): 40 to 50%;
- Behind the tail skin (components 4 and 5): 30 to 40%.

From Equation 1, it can be seen that the surface settlement curve depends on two main parameters i and V_L which are discussed in the following sections.

Settlement trough width i

The width of the settlement trough is dictated by the value i and the settlement trough width can extend up to $3i$. Therefore, a good prediction of i is of paramount importance to determine the width of the settlement trough and hence the area affected by ground settlement due to tunnelling.

O'Reilly and New (1982) proposed the relationship between i and z_0 as below;

For tunnels in clay;

$$i = 0.43z_0 + 1.1 \tag{4}$$

For tunnels in sand;

$$i = 0.28z_0 - 0.1 \tag{5}$$

For practical purposes, i is often determined by;

$$i = Kz_0 \tag{6}$$

where K is a dimensionless trough width parameter and depends on the soil properties; z_0 is the tunnel depth.

Following suggestions by O'Reilly and New (1982), it has become common practice to assume $K = 0.5$ for clay and $K = 0.25$ for granular soils.

In urban areas, it is of great interest to estimate the maximum soil settlement S_{max} as it indicates the level of potential damage to surrounding buildings. The maximum settlement S_{max} depends on volume loss V_L and i as described in Equations 2 and 3.

Volume loss V_L

Volume loss V_L depends on many factors including soil conditions, tunnelling technique, tunnel geometry and quality of workmanship hence it is difficult to estimate V_L. A common approach to predict V_L is to use field data from case studies of similar projects and engineering judgement.

5 MONITORING SCHEME

A part of the monitoring scheme was to measure surface settlement along with the position of the TBM. In this paper, the relative position of the TBM to the monitoring section is assigned a positive value when the TBM was advancing towards the monitoring section and a negative value when the TBM was moving away from the monitoring section. The position of the TBM was recorded every 5 seconds so that it could be linked to the surface settlement data.

Ground surface settlements were measured using precise levelling techniques. As the tunnel route is mostly beneath the streets with buildings along each side, most of the measurement sections only have 5 data points covering a distance of approximately 12m in the transverse direction of the tunnel centreline.

The section at the chainage point Km1+403 was chosen for analysis in this paper because it had 8 data points located in an area with minimum disturbance from the public and traffic (Figure 1a). Surface settlement reading were taken once a day when the TBM was within 100m to 10m from the measurement section and twice a day when the TBM was within 10m and -40m of the section.

6 RESULTS

Transverse settlement trough

Figure 6 presents surface settlement in the transverse direction at the chainage Km1+403 when the TBM advanced from +31.578m to -40.413m.

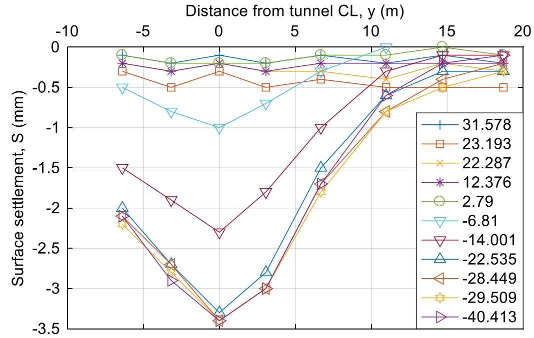


Figure 6. Surface settlement at the measured section with the corresponding positions of the TBM.

It can be seen that when the tunnel was advancing to the monitoring section (from 31.578m to 2.79m) surface settlements were very small and less than 0.5mm. After the TBM had passed the monitoring section (from -6.81m), the surface settlement increased considerably to a maximum value of approximately 3.5mm and appeared to be stable when the TBM had advanced to -22.535m ($1.3z_0$) from the monitoring section. In addition the settlement curves show a clear Gaussian distribution (Figure 6).

Jones and Clayton (2013) suggested a nonlinear regression procedure for Gaussian curve-fitting to settlement data to obtain V_L and K values. The procedure involves calculation of the sum of absolute errors (SAE, Equation 7) where the error was defined as the difference between the measured data value and the Gaussian curve value (Equation 1) at the same distance y from the tunnel centreline. The pair of K and V_L that give the smallest SAE are the best fit values for the Gaussian curve.

$$SAE=\sum_{m=1}^{n}\left|\frac{V_S}{\sqrt{2\pi}i}\exp\left(\frac{-y_m^2}{2i^2}\right)-S_m\right| \qquad (7)$$

where n=8 is the total number of measurement points, S_m is the measured settlement value.

It is worth noting that in front of the TBM face, the surface settlement was very small (less than 0.5mm) which is close to the measurement error of the precise levelling technique (approximately

0.3mm). Therefore, it is not practical to determine a Gaussian curve for those data. As a consequence, only surface settlement data when the TBM was from 2.79m to -40.413m are analysed. Figure 7 demonstrates the fitting between the theoretical Gaussian curve with the measured data when TBM was at -40.413m.

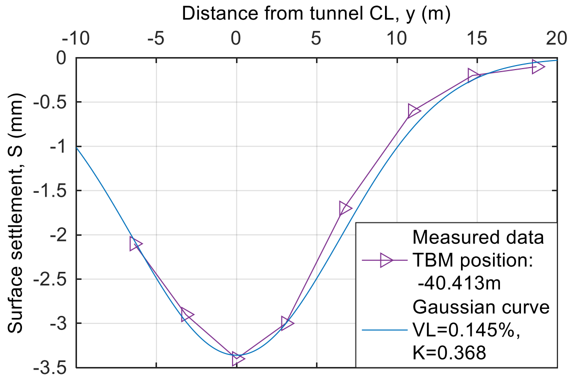


Figure 7. Comparison between the theoretical Gaussian settlement curve with the corresponding measured points.

Table 1 presents the K and V_L values of the settlement curves at the monitoring section with different TBM positions. When the surface settlement became stable, i.e. when TBM went pass the monitoring section by more than 22m, the K values was found to be within the range of $K = 0.341\ to\ 0.368$. The value of V_L was below 0.15% which resulted in small surface settlements.

Table 1. Values of K and V_L with different TBM positions

Position of TBM (m)	Date	V_L (%)	i (m)	K
2.79	02/08/2017	0.008	4.996	0.284
-6.81	03/08/2017	0.030	4.308	0.245
-14	04/08/2017	0.084	5.467	0.311
-22.54	05/08/2017	0.131	6.004	0.341
-28.45	06/08/2017	0.149	6.481	0.368
-29.51	07/08/2017	0.149	6.482	0.368
-40.413	08/08/2017	0.145	6.480	0.368

Longitudinal surface settlement

In order to assess the contributions of different components to surface settlement, Figure 8 plots the maximum surface settlement S_{max} (above the tunnel centreline) and the volume loss V_L along with the corresponding TBM positions.

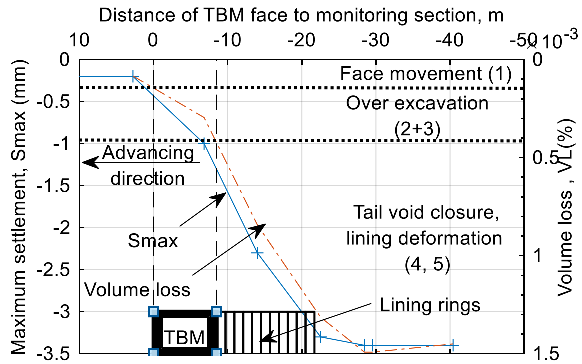


Figure 8. Maximum surface settlement and volume loss during TBM driving at chainage km 1+403.

From Figure 8, it can be seen that component 1 caused a volume loss of approximately 0.015% which is minor compared to the contributions of other components. This means good support pressure in front of the TBM face was provided during the excavation hence soil settlement was minimised.

Interestingly, components 2 and 3 (along the shield body) only induced a volume loss of approximately 0.025% despite the ratio between the void (depicted in Figure 9) and the volume of excavation, V_{exc}, is 4.923% as calculated in Equations 8 and 9.

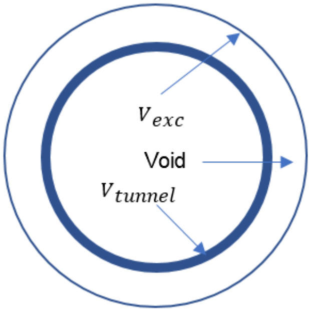


Figure 9. Ratio of void and the volume of excavation, V_{exc}.

$$Void = \frac{\pi}{4}(6.82^2 - 6.65^2) = 1.798m^2 \quad (8)$$

$$\frac{Void}{V_{exc}} = \frac{1.798}{\frac{\pi}{4}6.82^2} = 4.923\% \quad (9)$$

There are two possible explanations for the fact that the resulted volume loss was much smaller than the excavated void.

The first reason was the "stand-up time" of the soil. At the tunnel depth, the soil was medium dense to dense, well-graded sand which has considerable degree of interlocking between particles. Therefore, when the sand was excavated and shear stress increased, dilation of sand occured as a result of the reduction of the interlocking degree between particles. The dilation of sand caused a local reduction of pore pressure which lead to a corresponding increase in effective stress. The increase in effective stress of the soil is largely responsible to the "stand-up time" of soil that delays radial movement towards the excavated cavity before the final tunnel was erected.

The second reason was the grout was quickly injected into the void to support the excavated cavity and limits the tail void closure which resulted in a volume loss of 0.1% (Figure 8) which is much smaller than the Void/V_{exc} ratio of 4.923%. It is worth noting that the volume loss of 0.1% was contributed to by tail void closure and lining deformation and they are the main contributors to the surface settlement in this case. A similar observation was also reported by Wongsaroj et al. (2006) where an EPB TBM was used for tunnel excavation.

7 CONCLUSION

Field measurement on surface settlement from the excavation of the East bound tunnel, the first tunnel for metro line in Vietnam, using EPB TBM, was presented and analysed. From there, the following conclusions can be made;

1. The obtained transverse settlement trough showed a clear Gaussian curve and the corresponding values of K and V_L was determined using nonlinear regression method. The K value of soil at the considered section was found to be in the range of $K = 0.341\ to\ 0.368$ and the volume loss V_L was generally less than $V_L < 0.15\%$. These determined values will be a useful reference for future tunnelling projects in Ho Chi Minh city.
2. The EBP TBM was proved to be suitable with the encountered soil conditions and tunnel depth. The maximum surface settlement was less than 4mm which is excellent and a success of the project. This means good control of soil settlement was achieved by means of sufficient face support pressure and swift grout injection behind the tunnel linings.
3. The volume loss caused by face movement was minor which is similar to other projects where TBM was used. The main component of volume loss came from tail void closure and lining deformation.
4. Investigation on the "stand-up time" of soil and the effect of grouting behind the tunnel lining will give better insights into the development of soil settlement caused by TBM tunnelling.

8 ACKNOWLEDGEMENT

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