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Prediction of ground surface settlement induced by twin tunnelling in urban areas



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ABSTRACT

Tunnelling in urban areas is growing in response to efficient transportation, many urban tunnels are constructed in soft ground at shallow depths. Urban tunnels are usually constructed as twin-parallel tunnels and their adjacent constructions may cause ground settlements that distort and damage the existing structures and utilities above the tunnel. In the past few decades, tunnel boring machines have been used to drill in increasingly difficult geotechnical conditions such as soft ground like soft clay. Metro Line 3 of the Hanoi metro system is designed of twin tunnels horizontally aligned in soft ground. The prediction of ground movements is an important part of the planning stage of any urban tunnelling project. This paper presents the results of numerical simulation by using ABAQUS finite element software to predict the vertical displacement at the surface caused by twin tunnelling of Hanoi pilot light metro line 03. According to numerical simulation results, the maximum vertical displacement at the surface caused by the left line tunnel and twin tunnels bore excavations is values of 12.8 and 21.3 mm, respectively. The maximum vertical displacement can be reached after the shield passes by a distance ranging from 30÷40 m. Twin tunnelling only affects the maximum vertical displacement at approximately 20÷30 m before excavation face tunnel. After the left line tunnel bore excavations, the magnitudes of the vertical displacement directly above the face tunnel ($x = 0$ m) is 7.9 mm coinciding with 61.7% of the maximum vertical displacement. After the twin tunnels bore excavations, The maximum vertical displacement directly above the face tunnel ($x = 0$ m) is 13.1 mm coinciding with 61.5% of the maximum vertical displacement.

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1. Introduction

Bored tunnel constructions cause ground movements. The assessment of the potential effects of these ground movements on infrastructure is an essential aspect of the planning, design and construction of a tunnelling project in an urban environment. The ability to correctly predict the tunneling-induced ground movements is a key aspect to properly estimate potential damages to pre-existing structures and to design protective measures when needed.

Therefore, accurate assessments of these vertical and horizontal movements are required and have been explored by many authors (Peck, 1969; Cording and Hansmire, 1975; Bartlett and Bubbers, 1979; Clough and Schmidt, 1981; O'Reilly and New, 1982; Attewell and Yeates, 1984; Addenbrooke and Potts, 2001). These authors have shown several methods for estimating ground movements based on tunnel diameter, tunnel depth, type of construction method and soil type. Tunnelling in urban areas often involves the excavation of twin tunnels, which are characterized by close proximity to each other. The construction of twin tunnels presents advantages in the reduction of tunnel diameter and limiting soil movement (Hage Chehade and Shahrour, 2008). Moreover, in modern tunnelling, twin tunnel construction is provided for safety reasons. A parallel tube enables emergency escapes via crossways in case of fire and traffic can be controlled uni-directional which reduces the number of accidents.

The layout of these tunnels can take some different geometric arrangements such as side-by-side, stacked or offset. During the construction of each line, two tunnels can be constructed within a relatively short space of time and within reasonably close proximity. These types of projects will herein be referred to as twin tunneling projects.

Irrespective of the construction method or arrangement used, any underground construction will cause a change in the ground stress state. This stress change is due to most excavation methods having unsupported soil at some point during the construction process. Consequently, ground movements will arise which, if apparent at the surface can cause

damage to buildings (Burland et al., 2001). Additionally, sub-surface movements may have a detrimental effect on existing buried services (Mair, 1996). Damage assessments are based on predictions of deformations due to tunnelling and there is consequently a need for accurate prediction methods.

If the assumption is made that the tunnels are parallel then it could be stated that, generally, there are three twin tunnelling arrangements. Two-dimensional idealizations are shown in Figure 1.

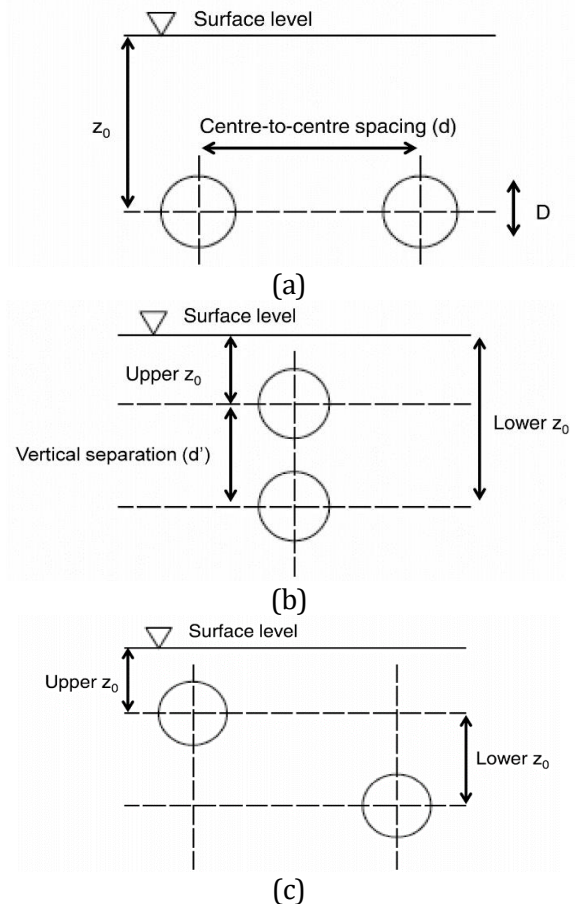


Figure 1. Idealizations of the three twin-tunneling scenarios in the y - z plane: a) Side-by-side; b) Stacked or piggyback; c) Offset, (Mair, 1996).

It can be seen that within these three variations side by side (a) geometry refers to multiple tunnels being constructed at the same horizontal axis depth. Stacked or Piggyback (b) arrangements consist of a second tunnel being constructed directly above or below the first.

Offset (c) could be described as halfway between the Side-by-side and Stacked arrangements. This would be the arrangement when the tunnels' centre line has an offset in both the vertical and horizontal axis.

2. Methods for surface settlement estimation induced by twin tunnel excavation

Some authors showed that the magnitude of the maximum settlement above the second tunnel S_{max} is increased and eccentrically positioned towards the first tunnel. Hunt (2005) proposed a modification factor to the semi-empirical tunneling-induced ground movements caused by a second tunnel. This method was developed after numerical analyses for tunnels excavated in London Clay. This method was based on modifying the ground movements of the second tunnel in an "overlapping zone", this is the soil assumed to have been previously disturbed by the creation of the first tunnel. The ground movements of the second tunnel are modified within this disturbed area. The modified settlement related to the second tunnel excavation is computed according to the following equations.

$$S_{mod} = F \cdot S_v \tag{2}$$

Where: S_{mod} - the modified settlement (m); S_v - the unmodified settlement above the second

tunnel (m), S_v - computed by semi-empirical methods, (m); F - modification factor.

$$F = \left\{ 1 + \left[M \left(1 - \frac{|d + x_a|}{A \cdot K_a \cdot Z^*} \right) \right] \right\} \tag{3}$$

Where: Z^* - the distance from the tunnel axis to the sub-surface level of interest (m); A - the multiple of the trough width parameter (usually taken as 2.5 or 3.0) in a half settlement trough; d - the center-to-center spacing of the tunnels (m); x_a - the lateral distance from the centre-line of the first bored tunnel, (m); K_a - the value of K in the region of the first tunnel bored; M - maximum modification factor.

3. Case studied tunnels

Hanoi Metro Rail System Project line 3: Nhon - Hanoi Station Section is the first pilot project of the Mass Transit Model in the urban area of Hanoi city, Vietnam. The Project Section runs from west to east of the city with a total length of 12.5 km with about 8.5 km of elevated section and 4.0 km of underground section, (SYSTRA, 2012). The plan view of twin tunnels of diameter $D = 6.3$ m, the separation between the two axes of about 15 m and a mean depth $H = 20$ m is shown in Figure 2. To effectively minimize ground movements in these highly-populated areas, Earth pressure balance (EPB) machines were selected. The EPB machine makes use of a rotating cutter-head as a tool of excavation; the excavated material, kept

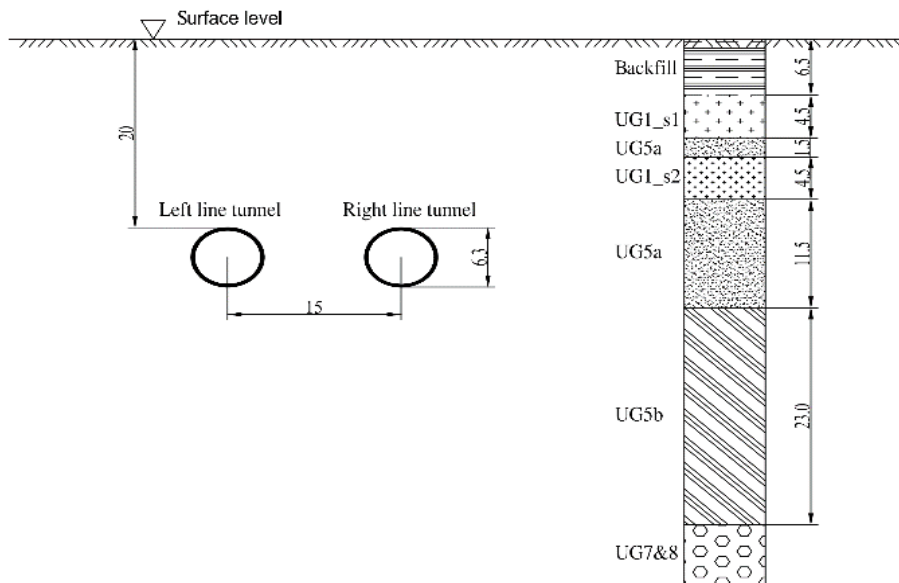


Figure 2. Cross-section view of the twin tunnels and different soil layers.

under pressure in the bulk chamber, ensures face stability and limits surface settlements.

According to the geotechnical engineering exploration report, the soil log between Cat Linh Station and Van Mieu Station, the twin tunnels cross the stratum mainly with Firm to stiff lean clay, Silty and clayey sand medium dense to dense. The order of tunnel construction is first left-line construction and then right-line construction, the physical and mechanical parameters of the rock and soil are determined as shown in Table 1.

Note: GU1_s1 - Firm to stiff lean clay (CL); GU1_s2 - Silt (ML); GU5a - Silty and clayey sand (SC-SM-SW) medium dense to dense; GU5b - Silty and clayey sand (SC-SM-SW) dense to very dense; GU7&8 - Gravel and Coarse sand with gravel (GM-GP-GM-GC) very dense. (SYSTRA, 2012), Geotechnical interpretative report underground section - Design report technical design. Project: Hanoi pilot light metro line 03 Section Nhon Hanoi Railway station). The tunnel lining, set in place inside the shield tail to support the tunnel as the machine advances, consists of concrete cast-in-place rings characterized by a length $l = 1.4$ m and a thickness $d = 30$ cm. The outer and inner diameters of the lining ring are equal to 6.3 m and

5.7 m, respectively. The external diameter of the final lining is always smaller than the excavated one, in order to allow the advancement of the machine and the shield. The tunnel lining is made of concrete with the parameters density $\rho_1 = 2500$ Kg/m³, Young's modulus $E_1 = 35$ GPa and Poisson's ratio $\nu_1 = 0.2$.

3.1. Finite element model

The specific objectives of the present study are to perform three dimensional (3D) nonlinear finite element analysis underground tunnel subjected to predict ground movements induced by twin tunnelling.

The finite element (FE) analyses have been performed using the commercially available FE software Abaqus Version 6.12-3. The Mohr-Coulomb model is adopted to define the behavior of the soil in the numerical analyses, and the calculation parameters of the soil layers are shown in Table 1. Within this study, an advance of twin tunnels was considered. The tunnel tubes have a diameter D of 6.3 m and a spacing of 15 m. A length of 50 m tunnel excavation advance was considered in this study.

Table 1. Physical and mechanical parameters of rock and soil layers, (SYSTRA, 2012).

Geotechnical Unit	Backfill	GU1_s1	GU1_s2	GU5a	GU5b	GU7&8
Thickness h , (m)	6.5	4.5	4.5	11.5	23	26
Density, ρ (Kg/m ³)	1900	1850	1900	2000	2050	2100
Young's modulus, E (MPa)	10	15	25	45	75	100
Poisson's ratio, ν	-	0.3	0.3	0.3	0.3	0.3
Friction angle, ϕ (°)	80	250	25	34	35	40
Dilation angle, ψ (°)	0	0	0	0	0	0
Cohesion c (kPa)	-	10	25	0	0	0
Coefficient of lateral earth pressure at rest, K_0	-	0.58	0.58	0.44	0.43	0.36

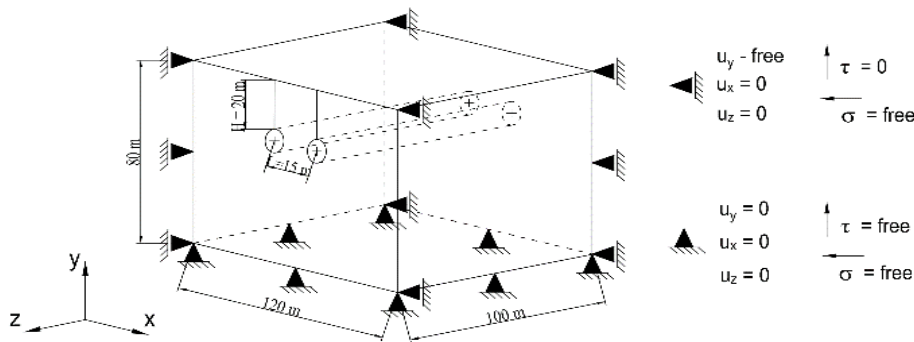


Figure 3. Boundary condition of bottom, surface and vertical boundaries.

The 3D model was set up with dimensions of 120 m in length and 100 m in width in x and z-direction, respectively and a depth of 80m in y-direction as displayed in Figure 3. The top model boundary ($y = 80$ m) was set to be free, whereas the vertical movement at the bottom boundary ($y = 0$ m) was fixed. Considering symmetry about the y-z plane with zero transverse displacements (x-axis) at $x = 0$ m and $x = 120$ m, and zero longitudinal movements (z-axis) at $z = 0$ m and $z = 100$ m.

The excavation and construction of the tunnel were simulated in a step-by-step procedure, incorporating the “element death” approach, which is widely employed in finite element analysis of excavation problems. Modeling stages are as follows:

- (a) Defining the circumferential area of soil around the twin tunnel.
- (b) Selecting an appropriate constitutive model and determining the required parameters.
- (c) Applying the boundary conditions and initial stresses.
- (d) Applying the initial geostatic stresses and satisfying the general equilibrium.
- (e) In the step, the first tunnel (left tunnel) is excavated by deactivating the corresponding excavation volume elements.
- (f) The tunnel lining of the left tunnel is added. Reactivate the elements for the tunnel lining in the step. The excavation of the left tunnel is completed.
- (g) Repeat step (e) - (f) for the right tunnel.

3.2. FE results and field measurements

Figure 4 (a) presents the magnitudes of the vertical displacement of the soil when the single tunnel (left line) excavation is completed. Figure 4 (b) shows vertical displacements of the soil when the twin tunnel (left line and right line) excavation is completed.

Figure 5 (a) presents the final magnitudes of the vertical displacement induced by excavation during the left line tunnel and twin tunnels. The maximum vertical displacement point appears over center of the tunnel after the excavation left line. The maximum vertical displacement at the surface is 12.8 mm, and the surface settlement curve obeys the deformation law of Peck’s settlement trough. After the right line (twin tunnels) is excavated, the maximum vertical displacement point (21.3 mm) stabilizes at approximately 7.5 m away from the centerline of the left line tunnel.

Figure 6 presents the ground movements along the longitudinal direction of tunnel left and twin tunnels.

As shown in Figure 6, after the left line tunnel bore excavations, the vertical displacement directly above the tunnel face ($x = 0$ m) is 7.9 mm coinciding with 61.7% of the maximum vertical displacement. After the twin tunnels excavations, the maximum vertical displacement directly above the face tunnel ($x = 0$ m) is 13.1 mm coinciding with 61.5% of the maximum vertical displacement. The maximum vertical displacement can be reached after the shield passes by a distance ranging from 30÷40 m.

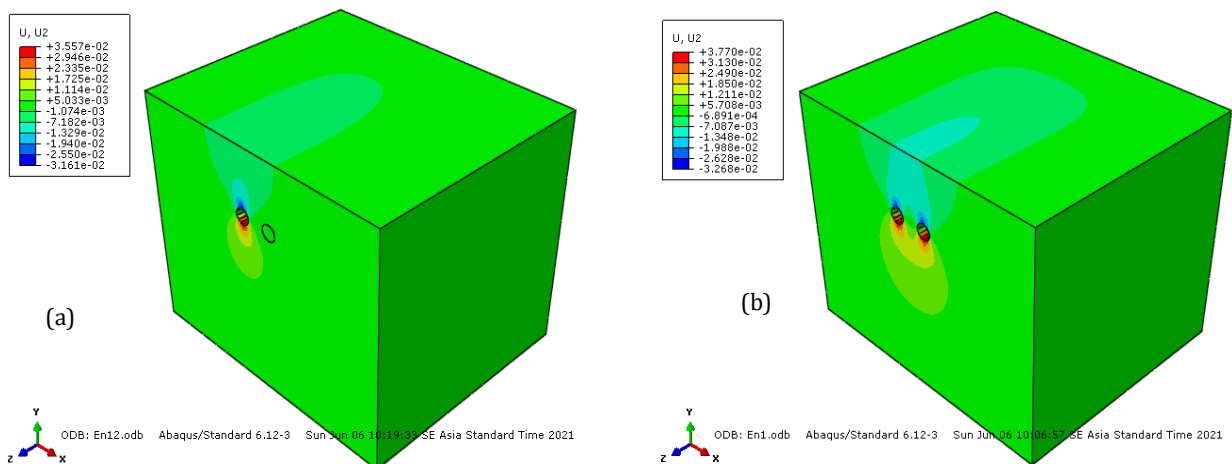


Figure 4. Vertical displacements after the right tunnel excavation.
(a) left tunnel excavation; (b) twin tunnel excavation.

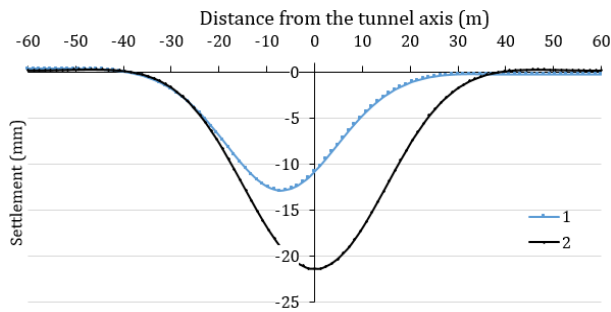


Figure 5. Surface settlement lateral trough: 1 - left tunnel; 2 - twin tunnel.

Twin tunnelling only affects the maximum vertical displacement at approximately 20÷30 m before excavation face tunnel.

Figures 5, 6 present the final magnitudes of the vertical displacement induced by excavation during the left line tunnel and twin tunnel. The vertical displacement curve has been compared with the prediction methods outlined earlier. These are shown to have good agreement with Hunt (2005).

4. Conclusions

In this study, we have performed 3D computational modeling to investigate the vertical displacement due to twin tunnelling. The model has been conducted by the use of existing geological reports of Hanoi pilot light metro line 03. The following can be drawn based on the study:

- The maximum vertical displacement at the surface caused by the left line tunnel and twin tunnels bore excavations is values of 12.8 and 21.3 mm, respectively.

- After the left line tunnel bore excavations, the magnitudes of the vertical displacement directly above the face tunnel ($x = 0$ m) is 7.9 mm coinciding with 61.7% of The maximum vertical displacement. After the twin tunnels bore excavations, The maximum vertical displacement directly above the face tunnel ($x = 0$ m) is 13.1 mm coinciding with 61.5% of the maximum vertical displacement.

- The maximum vertical displacement can be reached after the shield passes by a distance ranging from 30÷40 m.

- Twin tunnelling only affects the vertical displacement at approximately 20÷30 m before excavation face tunnel.

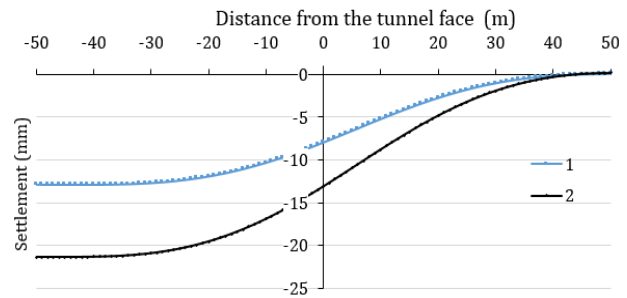


Figure 6. Longitudinal settlements: 1 - left tunnel; 2 - twin tunnel.

- Tunnelling in soft grounds causes ground movements, which may have an impact on adjacent structures.

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Author Contributions

Thai Ngoc Do contributed to the numerical model test and drafted the manuscript; Protosenya Anatoliy Grigorevich reviewed and edited the manuscript; Chinh Cong Thi Vo collected and analyzed the data.

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