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INVESTIGATION ON TUNNEL RESPONSES DUE TO ADJACENT LOADED PILE BY 3D FINITE ELEMENT ANALYSIS

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ABSTRACT: Underground structures are popularly utilized in urban development, especially tunnels for both transportation and utilities. The interaction problem between existing tunnel and piles from new constructed structures thus becomes unavoidable in dense population area. The tunnel loses its stability if the additional forces and changed diameters (due to adjacent piles under loading) of tunnel lining are drastically high. This depends on many factors, such as the clearance and pile tip level with respect to tunnel position. For preliminary assessment during the first stage of design, it is necessary to estimate this impact. The concept of tunnel protection zone is commonly adopted. However, to establish the tunnel protection zone for adjacent loaded pile, the understanding on this interaction problem is essential. This study analyzes the effect of adjacent pile under loading on the existing tunnel by 3D finite element method. The case study is the tunnel of MRTA project subjected to an adjacent 1 m bored pile under loading with various lengths and clearances. The additional forces (bending moment and axial force) and the tunnel deformations are investigated. The results show that the additional forces and the tunnel deformations decrease when the clearance increase and become insignificantly increasing when the clearance is larger than 3.5 m. The distribution patterns of additional forces and tunnel deformation are similar.

Keywords: 3D FEA, Tunnel lining deformation, Structure forces, Adjacent pile

1. INTRODUCTION

The underground structures increase in urban development, especially tunnels for both transportation and utilities. The Mass Rapid Transit Authority of Thailand (MRTA) has used the tunnel to solve the traffic congestion problems in Bangkok, Thailand. Generally, the tunnels are aligned below major roads and pass indirectly under buildings. In recent years, numbers of building constructions are increasing. Some new construction projects using a deep pile foundation are close to existing tunnels. The new constructed adjacent piles under loading as shown in Fig.1 may induce the adverse effect on tunnel lining.

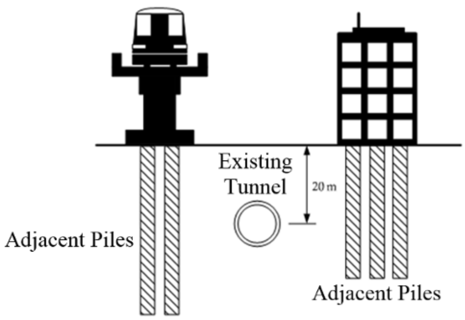

Fig. 1 New construction adjacent to existing tunnel in urban environment

In recent years, 3D finite element method (FEM) was used to analyze the effect on existing tunnel due to loaded bore pile in London [1], the effect of surface construction of multi-storey commercial building on two existing tunnels of Toronto subway in Canada [2] and the effect of a deep open face excavation on existing tunnel in Prague-Czech Republic [3]. In Bangkok, Thailand, the new influence zone considering the pile tip position with respect to tunnel position was presented by [4].

However, those researches focused on the influence of existing tunnel on nearby buildings and foundation. They mostly considered in term of tunnel deformations. Only a few works considered the effect on the structural forces in lining. The 2D and 3D FEM are used to study the structural forces during tunnel excavation or even additional forces in tunnel lining affected by ground stratification, surface buildings and tunnel depth [5]. The numerical and finite element analyses have also been utilized in design of tunnel lining [6-8].

Although the current design practice for tunnel lining considers all possible load scenarios during service of the tunnel, however, the future activities have not been included. It is then necessary to evaluate the possible effect of adjacent pile under loading on tunnel lining in terms of additional structure forces and/or the changes of tunnel diameters. The appropriate mean can then be prepared to tackle this problem.

Therefore, this study examined the tunnel responses due to adjacent pile under loading. The 3D FEM program and Mass Rapid Transit Authority of Thailand (MRTA) data was used in this research. The analysis results in terms of both the change of structural forces (bending moment and axial force) in tunnel lining and the change of tunnel diameters (vertical and horizontal directions) are focused and discussed.

2. METHODOLOGY

2.1 Considered Problem and Variables

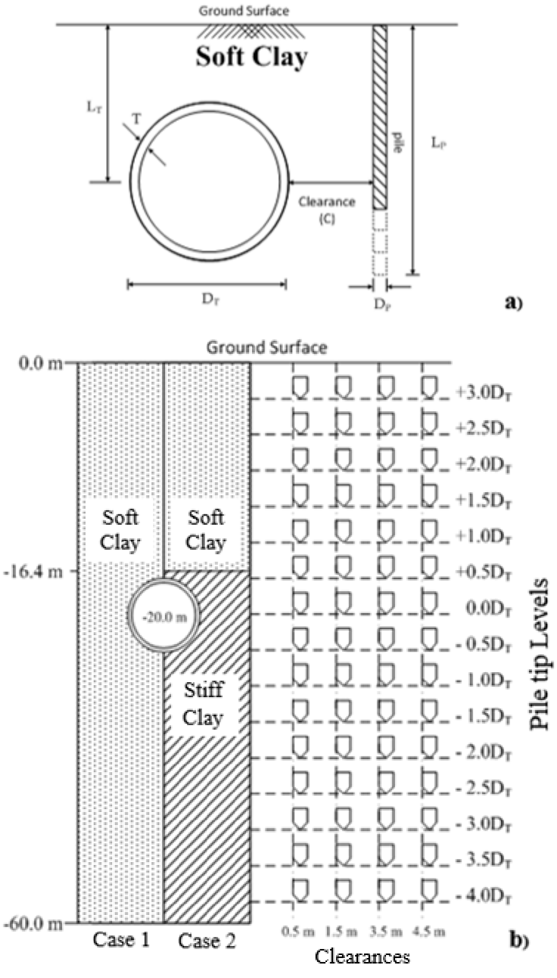

Fig. 2 Geometric parameters considered in this study

The evaluation of the effects of adjacent single pile on existing tunnel is accomplished with the 3D-finite element program (PLAXIS3D). Undrained conditions were used throughout in this study. The analysis did not consider the influence of pile construction. The length and applied working load of the single pile are varied in the analysis. The working loads are derived using the α – method [9].

In this paper, the geometric parameters in the study case are depicted in Fig.2a. The tunnel diameter *(D_T)* of 6.3 m, the lining thickness *(T)* of 0.3 m and the tunnel depth *(L_T)* of 20.0 m below the ground surface, the bored pile diameter *(D_P)* of 1.0 m and the interface of 0.9 which is interaction between soil and pile elements were considered. In addition, two case studies are taken into account. The first case (CASE 1) considered the tunnel located in soft clay. The second case (CASE 2), the tunnel is located in stiff clay. In the analysis, the clearance *(C)* between the bored pile to edge of tunnel and the bored pile length *(L_P)* are varied as shown in Fig 2b.

2.2 Numerical Model

The 3D-finite element mesh and numerical modeling in this paper are shown in Fig.3. The dimension of model is 80 m ($\approx 12.5D_T$) in the transverse direction, 60 m ($\approx 9.5D_T$) in the longitudinal direction and 60 m ($\approx 9.5D_T$) in the vertical direction. The monitoring plane is assigned at the center of longitudinal direction. To directly obtain the structural forces, shell elements are used to model tunnel lining with grouting layer, adapting from [10, 11]. The detail of this technique is presented in the companion paper [12].

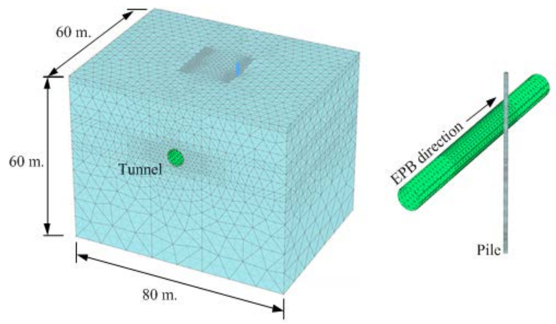

Fig. 3 Model and meshes for tunnel located in soft clay

2.3 Boundary Condition and Initial Conditions

2.3.1 Boundary conditions

Displacement boundary conditions are used in simulation of this study. The left and right side and bottom boundaries are sufficiently extended from the effect of tunnel excavation and pile settlement in the three dimensional analysis. The sides of the mesh including rear side and front side are free to move vertically but restrained against lateral movements. Left and right sides of mesh are controlled against perpendicular movement. The bottom of the mesh is fixed to both horizontal and vertical movement. The surface is allowed for free horizontal and vertical movements.

2.3.2 Initial conditions

The initial stress state, horizontal effective stress and vertical effective stress are controlled by the coefficient of earth pressure at rest, K_o and unit weight of soil layers. A typical pore water pressure profile from case study in MRTA Blue Line Project in Bangkok, Thailand in term of piezometric drawdown [7] with a water table at depth of 2.0 m below ground surface is adopted as shown in Fig.4.

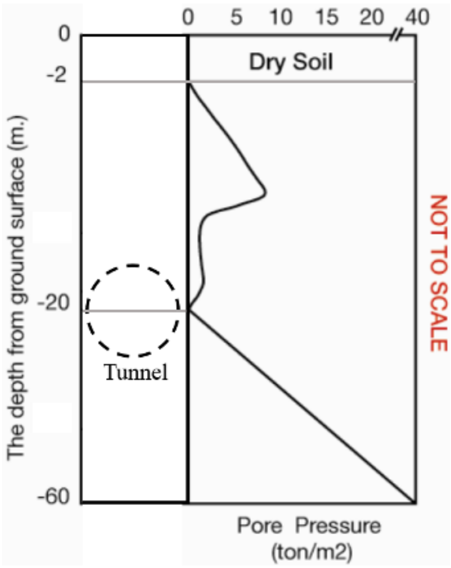

Fig. 4 Pore pressure in MRTA Blue Line Project

2.4 Model Parameters

The linear elastic model is assumed for tunnel lining, pile and grouting layer. The soft and stiff clays are modeled by hardening soil model (HS) [13]. The properties of soils are determined from MRT projects [14]. The material parameters for the simulations of geotechnical work in Bangkok subsoil are calibrated by [15] and has been used in previous research [16].

Table 1 Material properties of the bore pile, tunnel lining and grouting layer [16].

	Young modulus [E] (kN/m^2)	Poisson's ratio [υ]	Unit weight [γ] (kN/m^3)
Bored pile	3.1 x 10^7	0.20	24
Tunnel lining	3.1 x 10^7	0.20	24
Grouting* layer	1 x 10^6	0.30	21

* collected from [10, 11]

Table 2 Soil model parameters [16]

Soil layer	Soft clay	Stiff clay
Material model	HS	HS
E_{oed}^{ref} (kPa)	5,000	60,000
E_{50}^{ref} (kPa)	5,000	60,000
E_{ur}^{ref} (kPa)	15,000	180,000
γ_{sat} (kN/m^3)	16	18
υ' (-)	0.33	0.33
ϕ' (^{o})	22	22
c (kPa)	5	18
m (-)	1	1
p_{ref} (kPa)	100	95

2.5 Measurement of Structural Forces in Lining

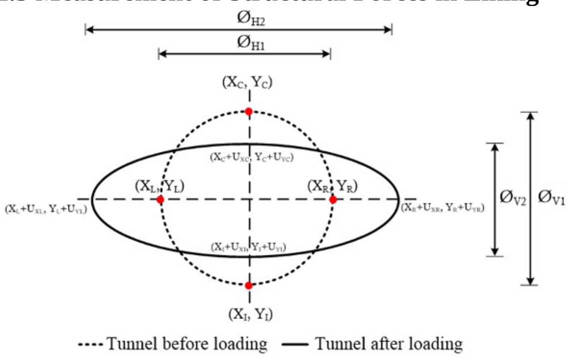

Fig. 5 Positions to observe the structural forces in tunnel lining and tunnel deformations

2.5.1 Structural forces

Based on previous studies on impact of adjacent loaded pile on existing tunnel [5,7], the tunnel deformation is generally considered in term of the changed tunnel diameter on the tunnel axis in vertical (tunnel crown and Invert) and horizontal (tunnel spring line) directions. Therefore, the bending moment and axial force are investigated from the corresponding positions as shown in Fig 5. The calculation of the change of bending moment and axial force are shown in Eq. (1) and (2). When M_1 , N_1 and M_2 , N_2 are structural forces in tunnel lining before and after subjected by pile under loading, respectively.

ΔM = change of bending moment

$$DM = M_2 - M_1 \quad (1)$$

ΔN = change of axial force

$$\Delta N = N_2 - N_1 \quad (2)$$

2.5.2 Tunnel deformations

The tunnel deformations are generally considered in terms of the changed tunnel diameters on the tunnel axis in vertical (tunnel crown to invert) and horizontal (tunnel left to right spring line) directions. Figure 5 depicts the considering method of tunnel deformations. The changes of tunnel diameters were computed following Eq. 3 and 4. When ϕ_1 and ϕ_2 are tunnel diameters before and after being subjected by pile under loading, respectively.

$\Delta\phi_V$ = change of vertical directions

$$\Delta\phi_V = \phi_{V2} - \phi_{V1} \quad (3)$$

$\Delta\phi_H$ = change of horizontal directions

$$\Delta\phi_H = \phi_{H2} - \phi_{H1} \quad (4)$$

3. ANALYSIS RESULTS

3.1 Structural Forces in Tunnel Lining

The structural forces (bending moment and axial force) of tunnel ling induced by adjacent pile under loading are observed and presented in this section.

3.1.1 Bending Moment

Figure 6 depicts the bending moment at crown, invert and spring line in non-dimensional form when the tunnel is located in soft clay and the clearances between the pile and the tunnel are 0.5 and 4.5 m. The values shown in the figure are normalized by the initial bending moment (M_i) values (after completion of tunnel construction) at their positions. When the pile tip is above the tunnel axis, the bending moment gradually increases with pile length and the maximum value occurs when the pile tip is at the depth of 0.7-075L_T (0.75-0.5D_T above the spring line). The maximum values happen at the invert with the value of 35% of the initial moment. After that, the moment gradually decreases with increasing pile length. The minimum value occurs at a depth in the range of 1.0-1.25L_T. When the pile tip extends to the depth of 1.25L_T to 1.75L_T, the bending moment becomes increasing again and gradually decreases until constant or remains unchanged with extending pile tip position underneath this level.

Note that the values of change of bending moment in case of clearance of 4.5 m are much smaller than those in case of clearance of 0.5 m.

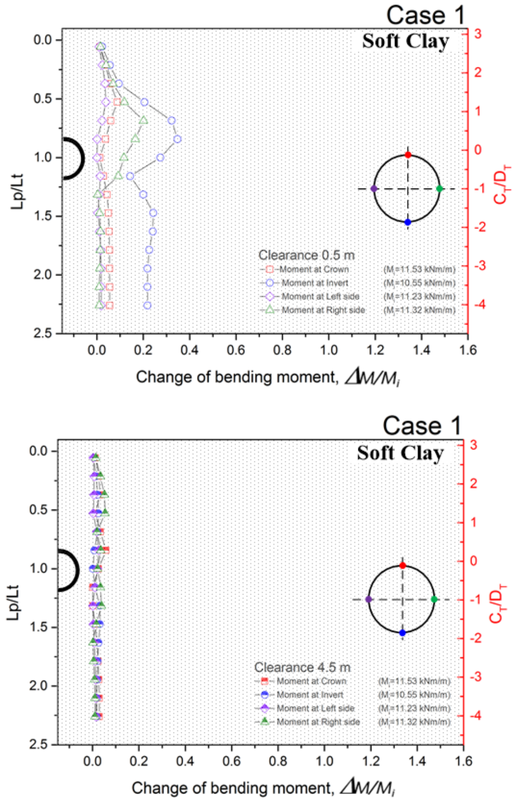

Fig. 6 The change of bending moment in tunnel lining due to adjacent loaded pile in soft clay

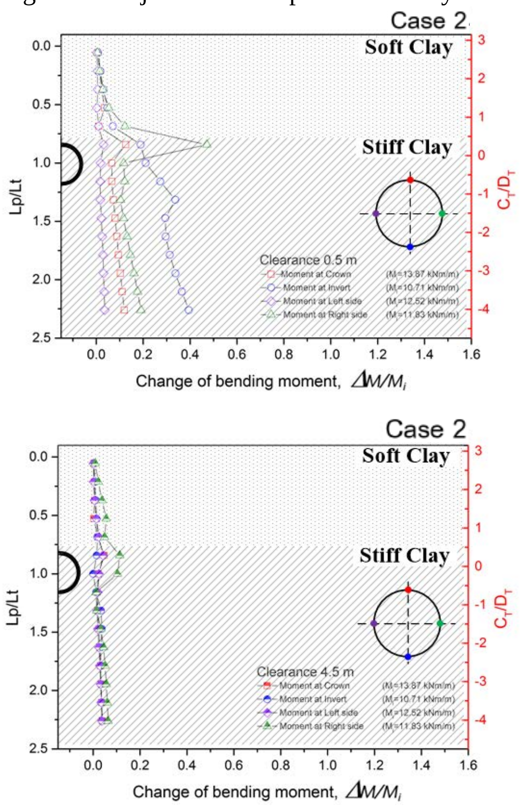

Fig. 7 The change of bending moment in tunnel lining due to adjacent loaded pile in stiff clay

Figure 7 depicts the analysis results when the tunnel is located in stiff clay. With increasing depth of the pile tip above the tunnel axis, the bending moment gradually increases and the maximum value occurs at a depth of $0.8L_T$, where the pile tip penetrates into the stiff clay layer. After that the moment decreases until the pile tip is located at depth in the range of 1.0-1.25 L_T before increasing with pile length again. However, the ratios of increase of moment ($\Delta M/M_i$) in this case is much smaller than those of the previous case (tunnel in soft clay). Note that the maximum value appears at the crown for this case.

CASE 1, the maximum increments of bending moment at crown and invert increase 8-10 and 5-35 percent, respectively. While at the left and right sides of spring line, 3-5 and 10-25 percent of maximum additional bending moments are observed.

CASE 2, the maximum increments of bending moment at crown and invert increase 10-20 and 10-55 percent, respectively. While at the left and right sides of spring line, 5-10 and 8-50 percent of maximum additional bending moments are observed.

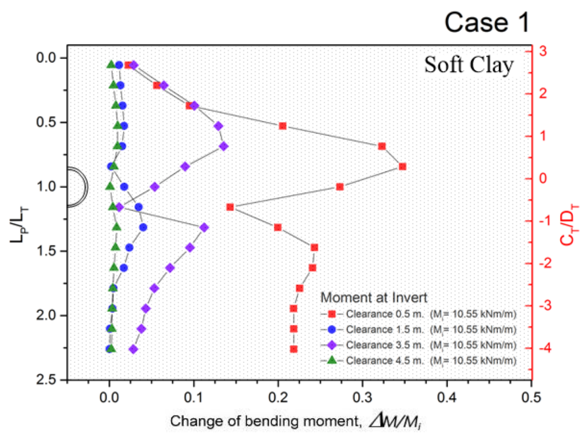

Fig. 8 The change of bending moment in tunnel lining at invert due to adjacent loaded pile in soft clay

To investigate the effect of clearance on induced bending moment, the change of bending moment in tunnel lining due to adjacent pile with various clearance is shown in Fig.8. Only the case of tunnel located in soft clay and at the invert is shown. The change of bending moment decreases when clearance increases. From the figure, it is seen that only small change in tunnel lining moment occurs if the clearance is larger than 3.5 m. This agrees well with the previous study [7]. The clearance of 3.5 m seems to be the safe distance for this condition.

3.1.2 Axial Force

Figure 9 depicts the axial force at crown, invert and spring line in non-dimensional form when the tunnel is located in soft clay and the clearances between the pile and the tunnel are 0.5 and 4.5 m. The values shown in the figure are normalized by the initial axial force (N_i) values (before having a loaded pile) at their positions. When the pile tip is above the tunnel axis, the axial force dramatically increases with pile length and the maximum value occurs when the pile tip is at the depth of $0.8L_T$ ($0.5D_T$ above the tunnel spring line). Unlike the bending moment, the maximum value occurs at the right side of spring line. After that, the axial force gradually decreases with increasing pile length. The minimum value occurs at a depth in the range of 1.0-$1.25L_T$. When the pile tip extends to the depth below the tunnel axis, the axial force becomes constant or remains unchanged in all clearance. Note that the values of change of axial force in case of clearance of 4.5 m are much smaller than those in case of clearance of 0.5 m. In CASE 2 (tunnel located in stiff clay), the ratios of increase of axial force ($\Delta N/N_i$) is much smaller than those of the CASE 1.

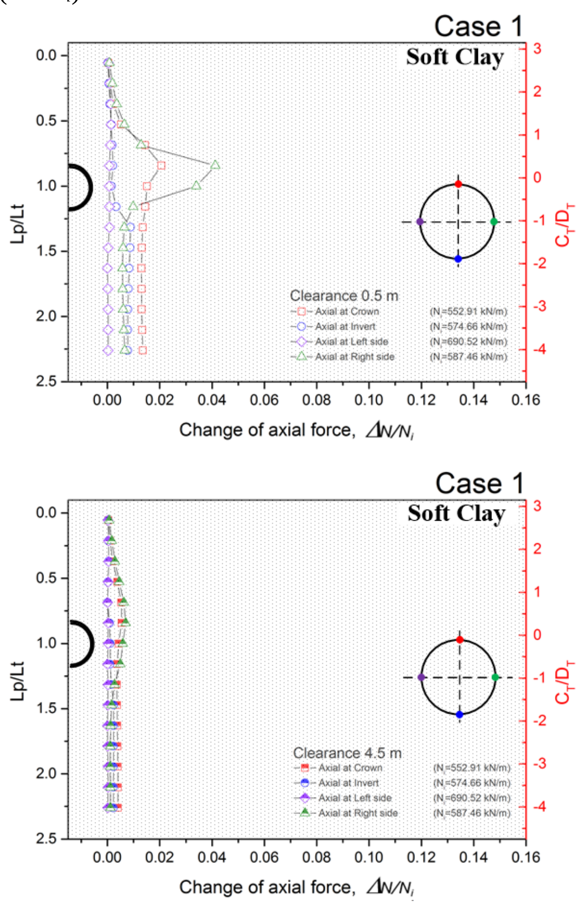

Fig. 9 The change of axial force in tunnel lining due to adjacent loaded pile in soft clay

CASE 1, the maximum increments of axial force at crown and invert increase 1-2 and 0.5-1 percent, respectively. While at the left and right sides of spring line, 0. 5-1 and 1-4 percent of maximum additional axial forces are observed.

CASE 2, the maximum increments of axial force at crown and invert increase 1-7 and 0.5-1 percent, respectively. While at the left and right sides of spring line, 5-10 and 0. 5-10 percent of maximum additional axial forces are observed.

3.2 Tunnel Deformations

Generally, the tunnel deformations are considered in term of the changed tunnel diameters on both tunnel axes.

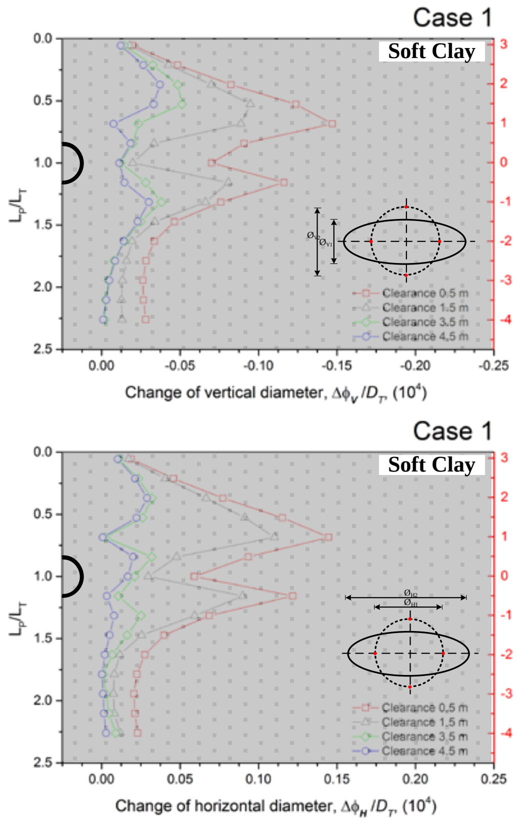

Fig. 10 Changes of tunnel diameter in soft clay due to adjacent loaded pile

Figure 10 shows the changes of tunnel diameters in vertical ($\Delta\phi_V$) and horizontal ($\Delta\phi_H$) directions for the tunnel located in soft clay (CASE 1) with all clearances. The changes of tunnel diameter are normalized by the tunnel diameter (D_T) and plotted against the normalized depth of pile tip, L_P/L_T. The normalized depth of tunnel position to tunnel diameter, C_T/D_T, is also provided in the y-axis on the right side. The negative and positive signs of magnitude in the x-axis denote a contraction and extension of the tunnel diameter, respectively. The magnitudes of $\Delta\phi_V$ and $\Delta\phi_H$ decrease when the clearance increases. At the pile tip is located above the tunnel axis, $\Delta\phi_V$ gradually increases with pile length until the maximum value occurs at of $0.70L_T$. After that, $\Delta\phi_V$ gradually decreases with increasing the pile length. The minimum value occurs at a depth of $1.0L_T$ (at the tunnel spring line). When the pile tip is extended below to the depth of $1.00L_T$ to $1.25L_T$, $\Delta\phi_V$ increases again. Then, $\Delta\phi_V$ gradually decreases until the pile tip is located about $1.75L_T$. The change rate of $\Delta\phi_V$ becomes insignificant or remains unchanged when the pile tip is extended below to the depth of $1.75L_T$. For $\Delta\phi_H$, a similar tendency to $\Delta\phi_V$ can be observed and the magnitude is as close but $\Delta\phi_H$ becomes slightly increasing when the pile tip is extended below to the depth of $1.75L_T$.

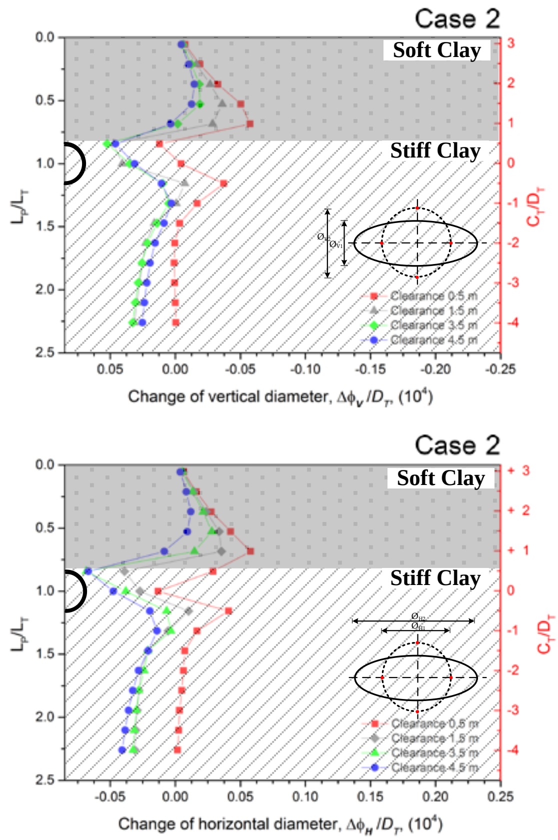

Fig. 11 Changes of tunnel diameter in stiff clay due to adjacent loaded pile

The results of tunnel deformations for tunnel located in stiff clay are shown in Fig. 11. The tunnel deformations in both vertical ($\Delta\phi_V$) and horizontal ($\Delta\phi_H$) directions gradually increase when the pile tip is located above the tunnel axis. The maximum value occurs at pile tip of $0.70L_T$. When the pile tip is extended to stiff clay, $\Delta\phi_V$ and $\Delta\phi_H$ decrease. The minimum value occurs at a depth of $0.80L_T$ (at the tunnel crown). After that, $\Delta\phi_V$ and $\Delta\phi_H$ have similar tendency to those in case of tunnel located in soft clay, when the pile tip is extended below to the depth of 1.00L_T. Unlike the case of tunnel located in soft clay, the magnitude of $\Delta\phi_V$ and $\Delta\phi_H$ in this case are smaller than those in case of tunnel embedded in soft clay.

In all cases, the distribution patterns and magnitudes of changes in tunnel diameter in both vertical ($\Delta\phi_V$) and horizontal ($\Delta\phi_H$) directions at the clearance of 3.5 m are similar to those of the clearance of 4.5 m. Nevertheless, the changes of tunnel diameter in both directions are emphatically influenced at the clearance of 0.5 m and 1.5 m. The tunnel deformation dramatically decreases and increases when the pile tip is situated in the range of 0.7L_T to 1.25L_T.

By comparing with the result between structural forces and tunnel deformations, the positions of pile tip to induce maximum deformations and additional forces in tunnel lining are close. Their patterns along the depth are also similar.

4. CONCLUSION

This paper analyzes the effect of existing tunnel due to adjacent single pile under loading by 3D finite element method. The pile tip is positioned with various lengths and clearances. The changes of structural forces in tunnel lining are observed. The main results of the analysis are as follows:

1. The structural forces in tunnel lining located in stiff clay (CASE 2) are larger than those of tunnel located in soft clay (CASE 1). Nevertheless, the tunnel deformations in case 2 are smaller than those in case 1. This is due to that the working load of pile in stiff clay (CASE 2) is larger than that in soft clay (CASE 1).
2. The tunnel deformations ($\Delta\phi_V$ and $\Delta\phi_H$) and additional forces decrease when the clearance increases and become insignificantly increasing when the clearance is larger than 3.5 m. The maximum structural forces in tunnel lining occur when the pile tip is located at the depth of about 0.8L_T in all clearances and all cases.
3. The pattern of change in structural forces of tunnel lining corresponds to that in tunnel deformations.
4. It is possible that the increase in structural forces in the lining due to adjacent pile is significant and cannot be neglected. The possible increasing structural forces due to future construction should be considered in tunnel lining design. Otherwise, the lining must be checked if it can resist the additional forces due to the planned construction.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

[1] Schroeder FC, Potts DM and addenbrooke TI, “The influence of pile group loading on existing tunnels”, Vol. 54, Geotechnique, 2004, pp. 351-362.

[2] Abdel M-Meguid, Rowe RK and Lo KY, “3D effects of surface construction over existing subway tunnels”, Vol. 2, Geomechanics, 2002, pp. 447-469.

[3] Dolezalova M, “Tunnel complex unloaded by a deep excavation”, Vol. 28, Computer Geotechnics, 2001, pp.469-493.

[4] Lueprasert P, Jongpradist P, Charoenpak K, Chaipanna P and Suwansawat S, “Three dimensional finite element analysis for preliminary establishment of tunnel influence zone subject to pile loading”, Vol. 9, Maejo Int. J. of science technology, 2015, pp. 209-223.

[5] Katebi H, Rezaei AH, Hajialilue M-Bonab and Tarifard A, “Assessment the influence of ground stratification, tunnel and surface buildings specifications on shield tunnel lining loads (by FEM)” Vol. 49, Tunnelling and Underground Space Technology, 2015, pp. 67-78.

[6] Bakker KJ, “Structural design of linings for bored tunnel in soft ground”, Vol. 48, Delft University of technology, 2003.

[7] Suwansawat S, “Earth Pressure Balance (EPB) Shield Tunneling in Bangkok: Ground Response and Prediction of Surface Settlements Using Artificial Neural Networks”, Ph. D. Thesis of Massachusetts Institute of Technology, 2002.

[8] Bobbier PG, Moller SC and Vermeer PA, “Bending moments and normal forces in tunnel linings”, Conf. on 5th European Conference of Numerical Methods in Geotechnical Engineering, 2002.

[9] Skempton AW, “Cast in-situ bored piles in London Clay”, Geotechnique, 2, 1959, pp. 153-173.

[10] Kasper T and Mescgke G, “A 3D finite element simulation model for TBM tunnelling in soft ground”, Vol. 28, Int. J. for Numerical and Analytical Methods in Geomechanics, 2004, pp. 1441-1460.

[11] Kasper T and Mescgke G, " A numerical study of the effect of soil and grout material properties and cover depth in shield tunnelling" , Vol. 33, Computers and Geotechnics, 2006, pp. 234-247.
[12] Lueprasert P, Jongpradist P and Suwansawat S, " 3D-FEM of EPB shield tunnel excavation using shell element and grouting layer" , Int. Journal. of GEOMATE (submitted).
[13] Schanz T, Vermeer PA and Bonnier PG, " The hardening soil model: Formulation and verification" , in Proc. of PLAXIS symposium on Beyond 2000 in Computational Geotechnics, Rotterdam, Netherlands, 1999, pp. 281-296.
[14] Prust RE, Davies J and Hu S," Pressure meter Investigation for Mass Rapid Transit in Bangkok, Thailand" Transport. Res. Res. J. of the Transportation Research Board, 2005, pp. 207-217.
[15] Rukdeechuai T, Jongpradist P, Wonglert A and Kaewsri T, " Influence of soilmodels on numerical simulation of geotechnical works in Bangkok subsoil" , Vol. 20, EIT Research and Development Journal, 2009, pp. 17-28.
[16] Jongpradist P, Kaewsri T, Sawatparnich A, Suwansawat S, Youwat S, kongkitkul W and Sunitsakul J, " Development of tunneling influence zones for adjacent pile foundations by numerical analyses" , Vol. 34, Tunnelling and Underground Space Technology, 2013, pp. 96-109.