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Numerical Study on the Movement of Existing Tunnel Due to Deep Excavation in Shanghai

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ABSTRACT: Adjacent excavation can have significant impact on the stress and deformation of existing tunnels. Several construction techniques have been proposed to reduce the movement of a metro tunnel due to two adjacent excavations in Shanghai. To evaluate the effectiveness of these different methods, the interactive impact of the two adjacent excavations on the crossing tunnel is studied numerically in this paper. The 2D FEM numerical analysis uses the Cam-clay model to simulate the behaviour of soft clay and considers the nonlinear performance of the soil-wall interface and the excavation sequences. The analysis investigates the influence of various factors, including the excavation procedure, installation of resistance piles, and relationship between the tunnel and the retaining wall of excavation. The results show that the crossing tunnel heaves during excavations because the distance between the two adjacent excavations is very small and the diaphragm walls for the original tunnel are used as the retaining structure of the new excavations. This predicted trend is verified by field measurements. The parametric study shows that dividing the whole dividing the whole excavation into several pit excavations and installing resistance piles tied to the tunnel can decrease the vertical displacement of the tunnel effectively.

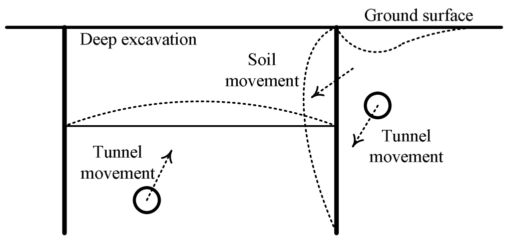
1. INTRODUCTION

Many cities in China have built the metro system in recent years, which is a life line of city transportation. The safety of metro tunnels is extremely important and the conflict between new underground structures and existing tunnels has become a major concern with the development of underground spaces in central urban area. The construction adjacent to the existing tunnels may have significant influence on the stress and deformation of the tunnels. As shown in Figure 1a, the deep excavation will cause movements of the surrounding soil and the adjacent tunnels. The displacement of a running tunnel depends on its location relative to the excavation. The underlying tunnel heaves while the side tunnel settles.

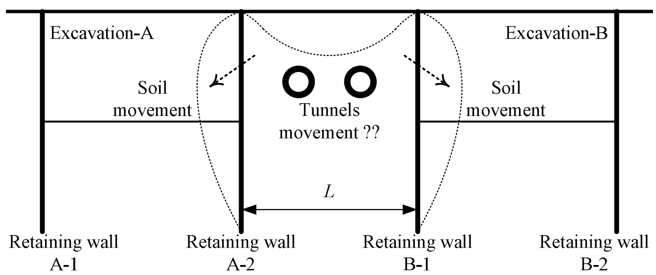
Many deep excavations have been carried out adjacent to running tunnels in different cities. Burford (1988) reported the long-term heave of tunnels due to overlying excavation in London which reaches 50 mm 27 years after the excavation. Chang et al. (2001) reported that a section of tunnel in Taipei was damaged by an adjacent excavation, and gave some advices for excavations near existing tunnels. Sharma et al. (2001) studied a case of large excavation close to two MRT tunnels in Singapore which is partly above the crown of the tunnels. In recent years, many excavation projects in Shanghai and Nanjing have been reported by researchers, such as Ji and Chen (2001), Chen and Zhang (2004), Jian (2006), and Liu et al. (2011). These researchers performed field monitoring to investigate the impact of excavation on adjacent tunnels. Based on field measurements in Shanghai, some efficient methods have been developed and applied in practice to reduce the effect of excavation on existing Metro tunnels (Hu 2003; Jia 2006), such as pumping inside the pit, ground improvement, and divided and zoned excavation. Researchers have also simulated the interaction behaviour between tunnel and excavation by numerical methods. Dolezalova (2001) analysed the effect of a deep open excavation for an office block on the underlying tunnel using 2D FEM. Sharma et al. (2001) presented a modelling of the excavation close to the MRT tunnel using a finite element program.

All aforementioned researchers only studied the influence of single excavation on the adjacent tunnels based on field measurements and numerical studies. Very few researchers have studied the interactive impact of two adjacent excavations on a crossing tunnel, such as that shown in Figure 1b. Hu et al. (2003) presented the design and construction of deep excavations located above and beside a Metro tunnel in Shanghai, and predicted the displacement of the tunnels induced by the deep excavations using 2D FEM.

In this paper, the movement of an existing tunnel crossing two deep excavations in Shanghai is predicted numerically. Considering the interaction between the tunnel and the excavations, the impact of the adjacent deep excavations on the shallow cut-and-cover tunnel is studied by simulating the construction procedure. A parametric study is conducted to investigate the influence of various factors on the tunnel's displacement.



a. Single deep excavation adjacent to running tunnels.



b. Two adjacent excavations on both sides of a running tunnel.

Figure 1. Interaction between excavation and tunnel.

2. BACKGROUND

2.1 Introduction of the Project

The project is located in Lujiazui financial district of Shanghai, where there are many skyscrapers and metro lines. The area of the triangle field is about 37,900 m^2. The project includes one 90 m tall tower and its podium buildings. The total construction area of the building is about 276,900 m^2, while the area of the underground part is about 115, 480 m^2. The basement of the building is a four-floor underground garage so that the excavation of this project is about 23 m in depth.

The subsoil in the field is mainly soft soils comprising Quaternary alluvial and marine deposits. The ground water level is about 0.8 m below the ground surface. The engineering properties of the soil layers in-site are given in Table 1.

Table 1. Properties of soils.

Period	Layer No.	Soil Layer name	H (m)	w (%)	γ (kN/m^3)	e	C' (kPa)	φ' (°)	C_c	CPT-1 p_s (MPa)	CPT-2 q_c (MPa)	CPT-2 fs (kPa)	PMT E_m (MPa)	PMT C_u (kPa)
Q_4^3	1	Filled soil	1.5-6.5											
	2	Silty clay	0.5-2.0	27.4	18.0	0.94	1	30.3	0.168	0.78	/		4.9	45
Q_4^2	3	soft Silty clay	2.5-6.9	34.7	17.2	1.18	5	33.5	0.241	0.84	0.99	13.3	3.1	37.8
	4	Very soft silty clay	8.4-10.0	48.6	17.0	1.43	5	25.3	0.383	0.55	0.51	14.9	6.5	45
Q_4^1	5	Clay	5.0-8.1	34.2	18.2	1.03	2	31.8	0.244	1.01	0.99	26.6	10.5	72
Q_3^2	6	Hard clay	2.4-4.8	22.5	19.9	0.71	5	33.0	0.131	2.34	2.22	72.5	17.7	123
	7-1	Sandy clay	9.4-13.1	29.9	18.9	0.84	0	34.5	0.101	11.44	12.44	91.4	22	/
	7-2	Fine sand	23.5-27.5	27.1	19.1	0.76	0	35.0	0.125	23.75	24.01	139.4	27.8	/
Q_3^1	9-1	sand	18.1-22.8	25.9	19.7	0.68			0.112	23.92	24.05	156.2	/	/
	9-2	coarse sand	2.7-10.0	23.2	19.9	0.61			0.096	24.71	26.26	120.7	/	/

Note: H= average thickness of soil layer; γ= unit weight; w= water content; e= void ratio; c'= effective cohesion of CU test; Φ'= friction angle of CU test; C_c = compressive index; p_s= specific penetration resistance of single bridge CPT; q_c = cone resistance of double bridge CPT; f_s = sleeve friction of double bridge CPT; E_m= elastic module of DMT; C_u= shear strength of DMT.

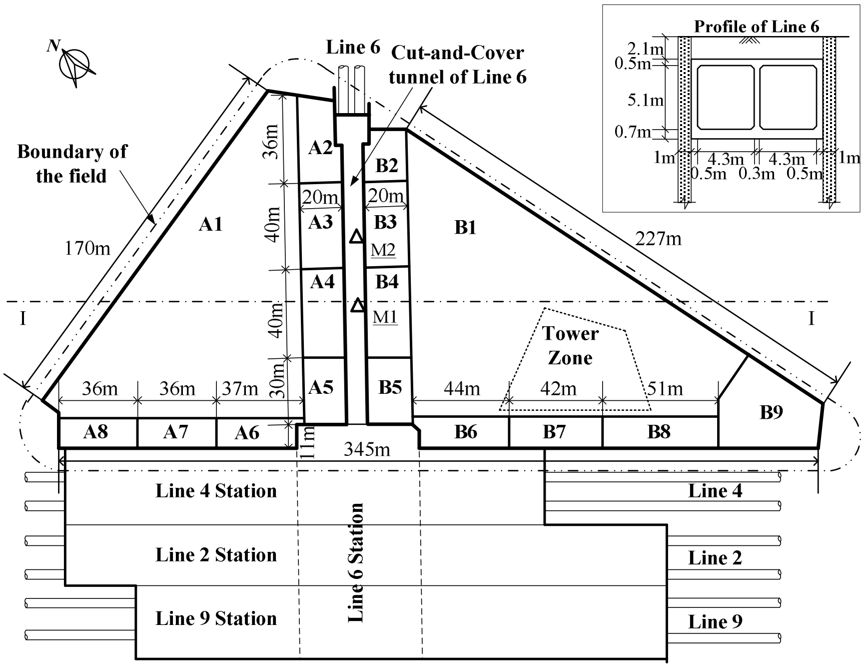


Figure 2. The scheme of excavation and adjacent Metro tunnels.

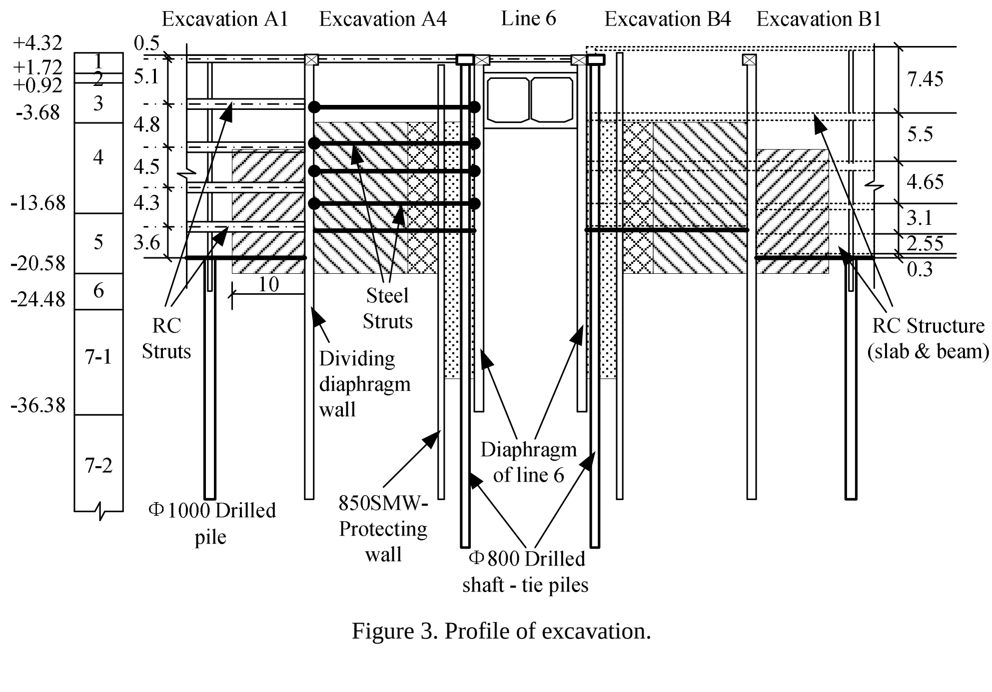


Figure 3. Profile of excavation.

As shown in Figure 2, the field is adjacent to an important interchange station, the Century Avenue Station. So there are four metro lines close to the excavation. Moreover, Metro Line 6 crosses the whole excavation area and divides it in two parts. It needs to be noted that the excavation project was planned before the construction of Metro Line 6 but was not started until three years after the metro line started running. So the crossing part of Line 6 was constructed using the cut-and-cover method in order to reduce the impact of the planned adjacent excavation. The section of the cut-and-cover tunnel is also sketched in Figure 2. To use the space efficiently, both the diaphragm walls of the cut-and-cover tunnel and the metro station will be used as the retaining structure of the new excavation. Therefore, both the tunnel and the station will be impacted by the excavation directly. The depth of the Line 6 tunnel is only 8 to 10 m which is much smaller than the depth of the new excavation. Other 6 shield tunnels are also very close to the new excavation, with the smallest clearance only about 12 m. Therefore, it is very important to control the movement of adjacent metro tunnels within the allowable limit of 20 mm.

2.2 Procedure for Excavation

To reduce the movement of the adjacent metro tunnels within the allowable value, some construction techniques are proposed based on the experience in Shanghai, especially for the crossing part of Line 6.

(1) The excavation is divided into two large pits and 15 small pits between the metro structures (tunnels and station) and the large pits, as shown in Figure 2. The areas of the two large pits are respectively 9,246 m^2 and 8,679 m^2, and the depth is the same at 23 m. The width of the small pits neighbouring to Line 6 is about 20 m, while the width of the small pits near the Metro station is about 15 m. The depth of small pits A5-A8 and B5-B8 is 14.75 m, and the depth of all other small pits is 19.75 m. So the width to depth ratio is about 1 for all small pits.

(2) The soft soil surrounding the metro tunnels is improved using the deep mixing method, as shown in Figure 3. The bottom of improved soil is 5 m deeper than the bottom of excavation. The soil mixing wall (SMW) method is adopted to construct the protecting walls at both sides of the cut-and-cover tunnel, and the depth of ground improvement between the protecting walls and the tunnel is increased from 25 m to 36 m.

(3) Diaphragm walls are adopted as the retaining structure of all excavation pits. The thickness of the outer diaphragm is 1.2 m, and the dividing wall between the large and small pits is 1.0 m thick. The depth of these diaphragm walls is 50 m. The dividing wall between small pits is 0.8 m thick and 36 m deep. The existing walls of the metro station and Line 6, which are 1.0 m thick and 40 m deep, are also used as the retaining structure of the new excavation.

(4) Drilled shafts of diameter 800 mm, 62 – 66 m long, and socketed into soil layer No.9 are installed as resistance piles and tied with the diaphragm wall of Line 6 to control its vertical displacement.

(5) To reduce the horizontal movement of Line 6, a symmetrical excavation plan is adopted, which means the pits on both sides of the tunnel should be excavated symmetrically at same time.

(6) The reinforcement concrete struts with a cross-section of 1200 × 900 mm are installed at five depths of the large pits (see Figures 3 and 4) so that the excavation is divided into 6 layers with a thickness respectively of 1.2, 5, 4.8, 4.5, 4.3, and 3m. The diagonal braces in the large pits have a cross-section of 600 × 600 mm. The zoned procedure is adopted in the excavation of each layer, and the central zones would be excavated prior to the edge zones, as shown in Figure 4.

(7) After the excavation and structure construction of the two large pits is completed, the small pits will be excavated in two groups to reduce both construction duration and the displacement of the Metro structures. The small pits at even numbers are excavated prior to those at odd numbers. One level of RC struts with a cross-section of 600 × 900 mm and four levels of 609 mm diameter steel pipe struts are used in each pit.

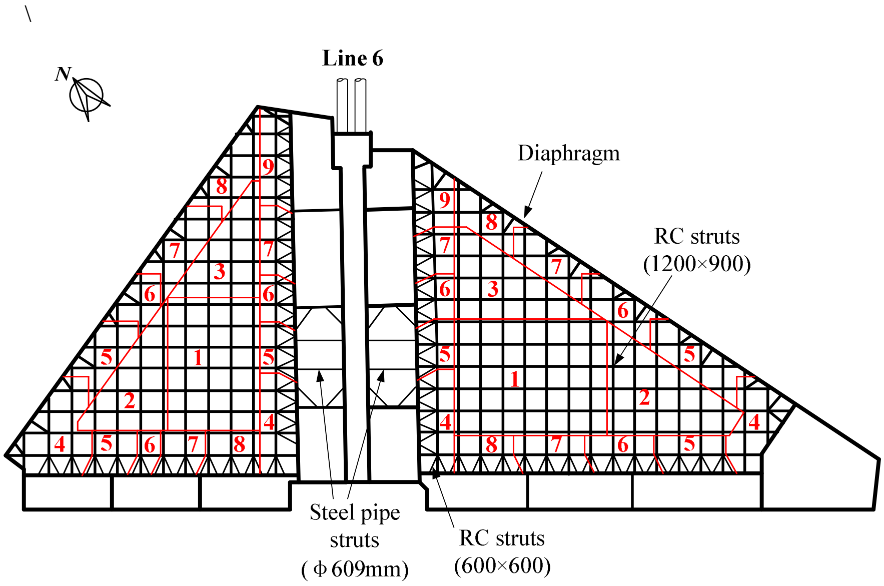


Figure 4. Reinforced concrete (RC) struts and excavation procedure of large pits.

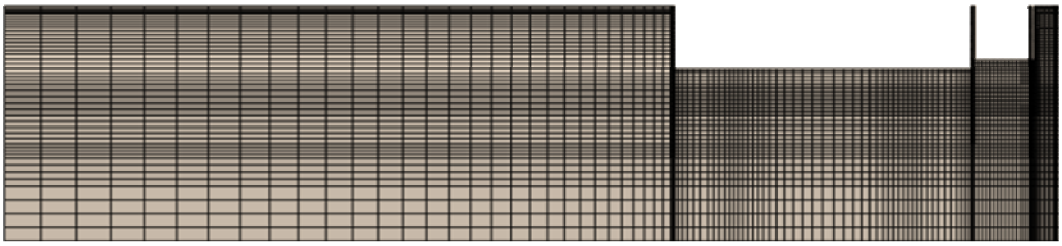


Figure 5. Meshes of the FEM model.

3. NUMERICAL ANALYSIS OF EXCAVATION PLAN

3.1 FE Model and Parameters

The influence of excavation on the metro tunnels is investigated numerically in order to optimize the construction plan before the construction starts. Because Metro Line 6 is closest to the excavation area and will be impacted the most, it is selected for the numerical analysis. To simplify the problem, the two-dimensional FEM model is adopted. The section for numerical analysis (Section I-I) is shown in Figure 2, which is almost symmetric.

Considering the symmetric section and the symmetric excavation plan as aforementioned, only a half of the section is analyzed and the FEM meshing is shown in Figure 5. The size of the half section is 250 m in width and 90 m in depth, which is much larger than that of the excavation. Symmetric boundary conditions are imposed on the symmetric surface of the model. The lateral displacement is restricted on the side surface and the vertical displacement is restricted on the bottom surface.

The columns supporting the RC struts are simplified as beams, and the beams below the floor are considered in the estimation of the stiffness of the floor slabs. The piles in the excavation area are simplified as beams coupled with soil to simulate their influence on soil movement. The resistance pile of Line 6, the joint between the resistance pile and the diaphragm wall of Line 6, and the surrounding soil improvement are all simulated in the model (see Figure 6). The resistance piles are simplified as walls and the stiffness of the equivalent wall is reduced by considering both the piles and the soils between them.

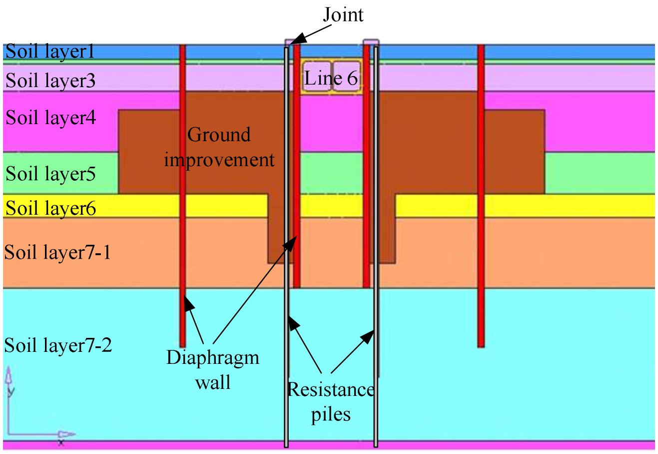


Figure 6. Relationship between Line 6 tunnel and excavation.

The Cam-clay model is adopted to simulate the nonlinear behaviour of soils. The parameters of soils are obtained empirically from the soil investigation report, such as those shown in Table 2. The mixed soil is assumed to be an elastic material with a unit weight of 18 kN/m^3 and an elastic modulus of 200MPa. All the structure materials are assumed to be elastic. The elastic modulus of the reinforced concrete (RC) struts and the steel pipe struts are 30 GPa and 210 GPa respectively, and their unit weights are 25 kN/m^3 and 78 kN/m^3 respectively. Considering the joints between different panels of the diaphragm wall, the elastic modulus of the diaphragm wall is assumed to be 18 GPa. A Poisson's ratio of 0.2 is used for all elastic structure materials.

The surface-to-surface contact model in Abaqus is applied to model the soil-wall interface so that the slippage and separation could be considered if there is enough relative displacement. Experiences show that the friction coefficient between soil and concrete is about $\tan[(1/2 \sim 2/3)\varphi']$, where φ' is the effective friction angle of soil. Using the soil properties shown in Table 1, the friction coefficient of the interface is determined accordingly. The shaft resistance of the resistance piles is determined based on the value of p_s from CPT by using the methods recommended in the Shanghai Foundation Design codes (DGJ08-37-2002 and DGJ08-11-2010, Eq.1) and considering the difference between driven piles and drilled shafts. The friction coefficient, μ, and the critical shear stress of interface, f_s, used in the numerical analyses are shown in Table 2. The critical shear stress is reduced for the equivalent wall of resistance piles to consider the spaces between the piles, which is given as f_{sp} in Table 2.

$$f_0 = \begin{cases} p_s/20\,(\text{kPa}) & \text{for clay, and } p_s \le 1000\text{kPa} \\ p_s/40 + 25\,(\text{kPa}) & \text{for clay, and } p_s > 1000\text{kPa} \\ p_s/50\,(\text{kPa}) & \text{for sand} \end{cases} \quad (1)$$

Table 3 shows the schedule of the construction activities in simulated in the numerical analyses based on the construction procedure of this project.

3.2 Numerical Results

Figure 7 shows the distribution of the horizontal displacements of the retaining structure and the soil. The largest horizontal displacement of the outer diaphragm wall is 25.8 mm at the end of the large pit excavation, and it increases to 30.7 mm when all excavations are finished. The horizontal displacement of the dividing wall between the large pit and the small pits is 5.7 mm at the end of the large pit excavation, and it is reduced to 3.2 mm when the small pit excavations are finished.

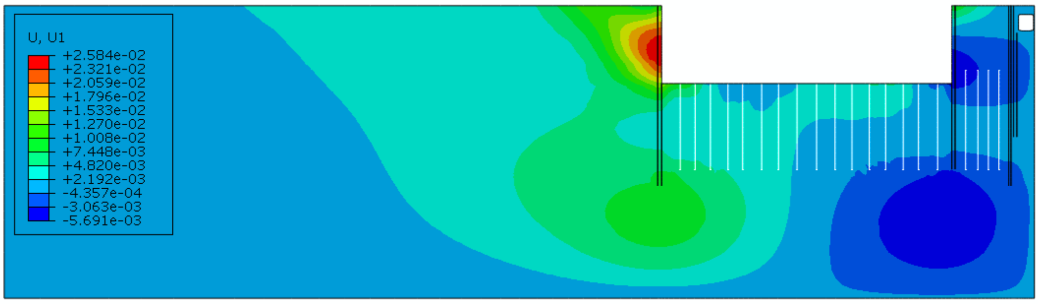
Table 2. Parameters adopted in FEM analysis.

Layer No.	H (m)	Soil			Interface								
					friction coefficient			critical shear stress					
		λ	κ	M	φ' (°)	Range of μ	μ	p_s (MPa)	f_0 (kPa)	f_1 (kPa)	Limits (kPa)	f_s (kPa)	f_{sp} (kPa)
1	2.6	0.073	0.0061	0.24			0.3					20	7
2	0.8	0.073	0.0061	0.24	30.3	0.27~0.37	0.3	0.78	39	29	15~30	20	7
3	4.6	0.105	0.0087	1.38	33.5	0.30~0.41	0.25	0.84	42	32	15~25	20	7
4	10	0.166	0.0139	0.69	25.3	0.22~0.30	0.25	0.55	28	21	15~30	20	7
5	6.9	0.106	0.0088	1.29	31.8	0.28~0.39	0.35	1.01	50	38	40~55	40	13
6	3.9	0.057	0.0047	1.20	33	0.30~0.40	0.3	2.34	84	63	50~80	65	22
7-1	11.9	0.044	0.0037	1.20	34.5	0.31~0.42	0.4	11.44	229	172	55~75	75	25
7-2	25.3	0.054	0.0045	1.20	35	0.32~0.43	0.4	23.75	475	356	55~80	75	25
9-1	20.0	0.049	0.0041	1.10			0.4	23.92	478	359	70~100	100	33
9-2	4.0	0.042	0.0035	1.10				24.71	494	371	70~100		

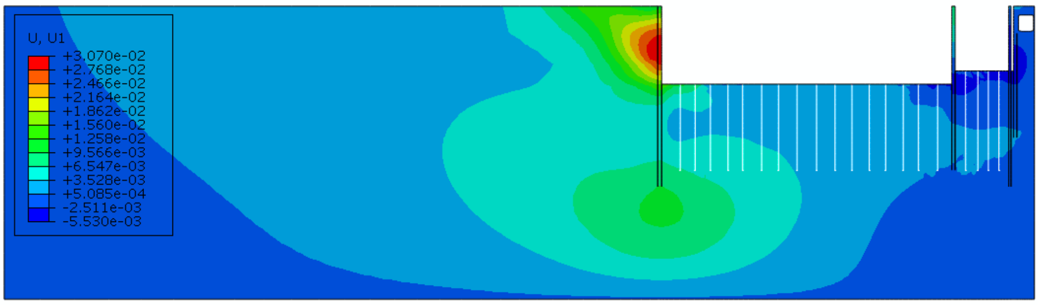
Note: The range of μ is calculated with $\mu = \tan[(1/2 \sim 2/3)\varphi']$; f_0 is calculated by equations (1) of driven piles; f_1=0.75 f_0 considering the difference between driven piles and drilled shafts; the limits for the drilled shaft's side friction are recommended by DGJ08-11-2010.

Table 3. Excavation sequence.

Step No.		Construction activities
Initial	1	Geotress condition, and Construct diaphragm wall and pile foundation
Large pit	2	Construct the 1st level of strut, excavate to depth of 6.2 m in center zone
	3	Excavate to depth of 6.2 m at surrounding zone
	4	Construct the 2nd level of strut, excavate to depth of 11 m in center zone
	5	Excavate to depth of 11 m at surrounding zone
	6	Construct the 3rd level of strut, excavate to depth of 15.5 m in center zone
	7	Excavate to depth of 15.5 m at surrounding zone
	8	Construct the 4th level of strut, excavate to depth of 19.8 m in center zone
	9	Excavate to depth of 19.8 m at surrounding zone
	10	Construct the 5th level of strut, excavate to depth of 22.8 m in center zone
	11	Excavate to depth of 22.8 m at surrounding zone
	12	Construction the bottom slab of basement
	13	Construct the substructure of floor B4, B3, and B2; remove struts respectively
Small pit	14	Construct the 1st level of strut, excavate to depth of 5.65 m
	15	Construct the 2nd level of strut, excavate to depth of 9.85 m
	16	Construct the 3rd level of strut, excavate to depth of 13.85 m
	17	Construct the 4th level of strut, excavate to depth of 16.59 m
	18	Construct the 5th level of strut, excavate to depth of 19.75 m
	19	Construction the bottom slab of basement
	20	Construct the substructure of floor B3, and B2; remove struts respectively
Final	21	Construct the substructure of floor B1; remove up part of dividing wall



a. At the end of large excavation.



b. At the end of small excavation.

Figure 7. Horizontal displacement of retaining structure and soil (unit: m).

The horizontal displacement of the existing diaphragm wall of Line 6 is shown in Figure 8. The upper part of the diaphragm wall only has very small displacement because of the high stiffness of the tunnel structure and the symmetric excavation. However, the lower part of the wall below the tunnel moves inward to the pit, with a maximum displacement of 1.6 mm after large pit excavation and 5.5 mm after small pit excavations. Most of the horizontal displacement is caused by the small pit excavations. Therefore dividing the whole excavation into several pit excavations reduces the movement of tunnel's retaining structure.

Figure 9 shows the distributions of the vertical displacements of the retaining structure and the soil. The maximum settlement of the ground surface is 11.0 mm during the large excavation. The maximum heave of the excavation surface is 95.3 mm after the large excavation is completed, and there is almost no influence from the small pit excavations on the soil in the large pit. The heave of the soil in the small pits is 44.7 mm after the pit excavations are finished.

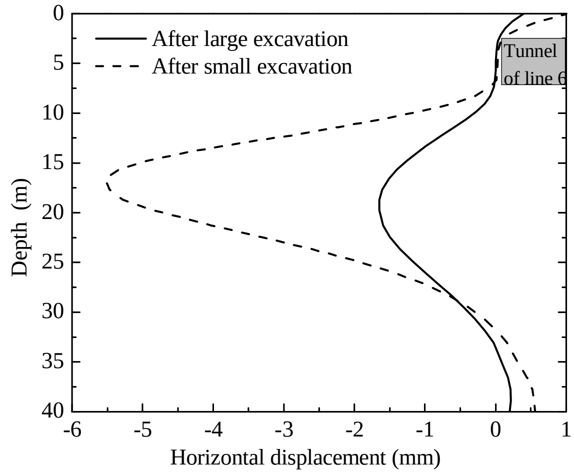
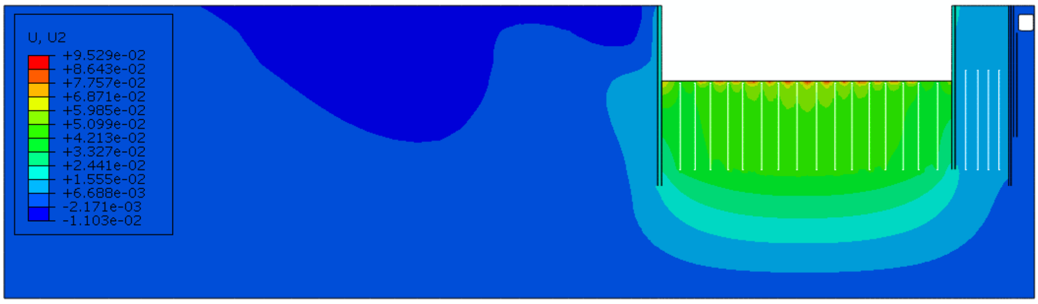
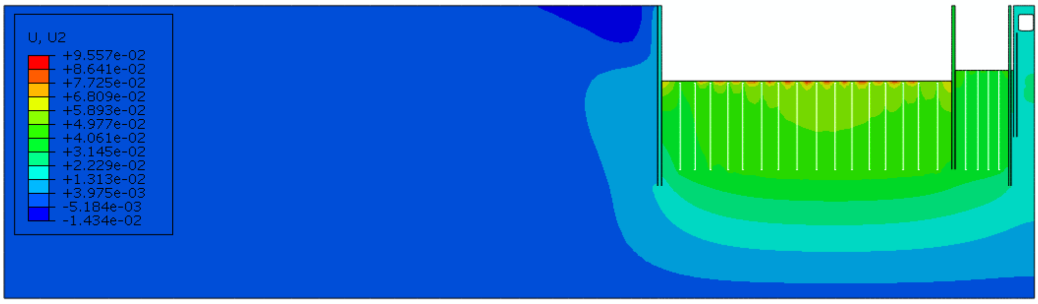


Figure 8. Horizontal displacement of diaphragm wall of Line 6.



a. At the end of large excavation.



b. At the end of small excavation.

Figure 9. Vertical displacement of retaining structure and soil (unit: m).

The final vertical displacement of the Line 6 tunnel is 21.4 mm, which is much larger than the horizontal displacement (almost zero under symmetric excavation). Therefore, it is more important to study the vertical displacement of the tunnel. To study the influence of excavation on the cut-and-cover tunnel of Line 6, the vertical displacements on four typical points as shown in Figure 10 are recorded. Point A is about 11.3 m, or half of the excavation depth, from the outer diaphragm wall; point B is at the top of the outer diaphragm wall; point C is at the top of the dividing wall; and point D is at the surface above the tunnel of Line 6. Figure 11 shows the variation of the vertical displacements at the four points during excavation. The vertical displacement at Point A is settlement as expected for normal excavation. The settlement caused by the large pit excavation is about 9.6 mm, and it is increased to 13.0 mm after the small pit excavations. Both the outer diaphragm wall and the dividing wall heave during excavation, with the heaves respectively of 16.6 mm and 19.2 mm after the large pit excavation. The heave of the dividing wall is increased to 38.7 mm after the small pit excavations, but there is essentially no increase for the heave of the outer diaphragm wall. Since the cut-and-cover tunnel of Line 6 is connected to the retaining wall of the small pits, it heaves during the whole excavation. The heaves of the tunnel caused by the large pit excavation and the small pit excavations are respectively 5.2 mm and 16.2 mm y, leading to a total heave of 21.4 mm. So the heave of the tunnel is caused mostly by the small pit excavations, and the heave of the tunnel during the small pit excavations is almost same as that of the dividing wall.

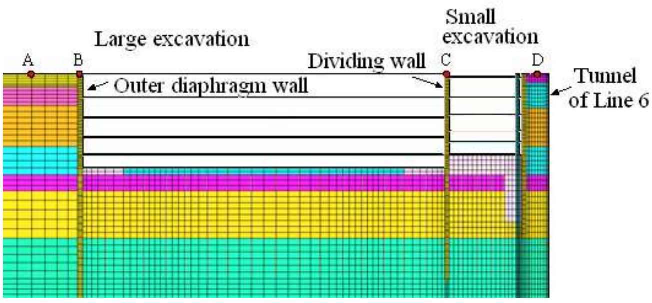


Figure 10. Location of four typical points investigated.

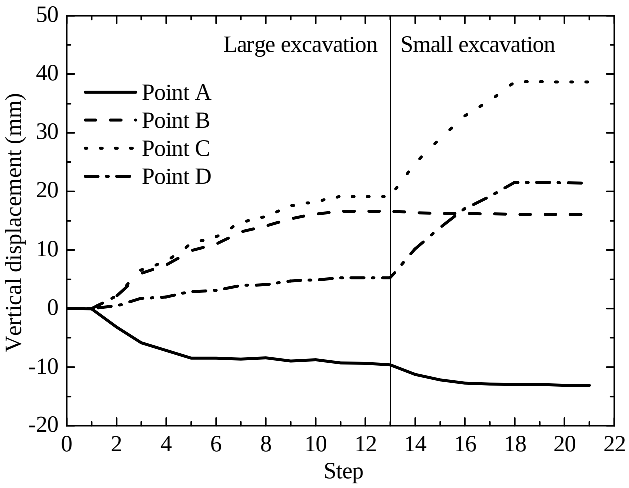


Figure 11. Vertical displacements of four typical points for the Original Plan.

4. PARAMETRIC STUDY

4.1 Influence of Excavation Procedure

The excavation project is divided into two large pits and several small pits in order to reduce the impact on the existing Metro tunnel. Considering the possible horizontal displacement of Line 6 and the excavation duration, the excavation procedure as described earlier is proposed based on the past experience. The pits on both sides of Line 6 are constructed symmetrically and the large pits are excavated before the small pits. This procedure is called the "Large First Plan (Original Plan)".

If the small pits are constructed before the large pits, which is called the "Small First Plan (2nd Plan)", the influence of excavation on the tunnel of Line 6 would be different. For this construction plan, the variation of the vertical displacements during excavation at the same four points, A, B, C and D, is obtained as shown in Figure 12. The small pit excavations have essentially no influence of the displacement at Points A and B; but the large pit excavation causes settlement at Point A and heave at Point B. Points C and D have about the same heave during the small pit excavations but Point C has much large heave than Point D during the large pit excavation.

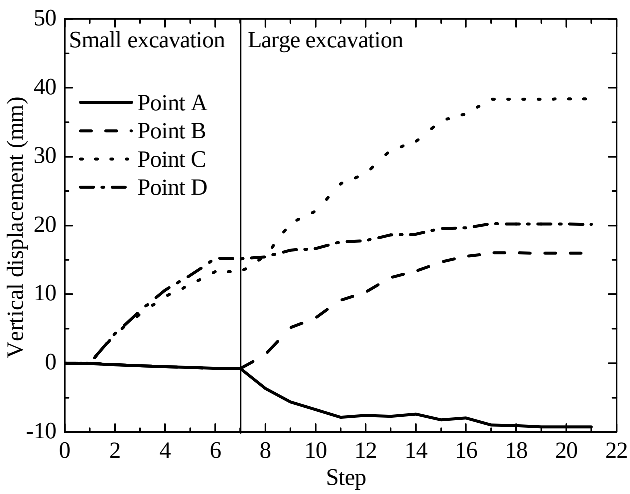


Figure 12. Vertical displacements of four typical points for the 2nd plan.

Comparing Figures 11 and 12, it can be seen that the final heaves at Points B and C are respectively 16.0 mm and 38.4 mm for the "Small First Plan", which are about the same for the "Large First Plan". So the excavation procedure has essentially no effect on the movement of the retaining structures of the large pit. The vertical displacements at Point D caused by the small pit excavations and the large pit excavation are respectively 15.2 mm and 5.0 mm for the "Small First Plan", both of which are smaller than those for the "Large First Plan". So the "Small First Plan" can reduce the heave of the running tunnel by about 6%.

To investigate the influence of excavation procedure in more detail, another two asymmetrical excavation plan is also simulated. The four simulated pits, Left Large Pit (LL), Left Small Pit (LS), Right Large Pit (RL), and Right Small Pit (RS), as shown in Figure 13. The parameters and boundary conditions in the FEM analysis are similar to those used in the original model. The excavation procedure of the two asymmetrical excavation plans, which are named as the 3rd and 4th plan, is described in Table 4.

The vertical displacements at seven typical points as shown in Figure 13 are recorded and shown in Figure 14. It shows that the settlement of ground surface and the heave of the outer wall are caused mainly by the nearest large pit excavation. The vertical displacement of the dividing wall is impacted by both adjacent pit excavations. Results of these six points, Points A to C and E to G as shown in Figure 13, are similar to the original plan, while the movement of Line 6 (Point D) changes. Table 5 compares the vertical displacements of Line 6 caused by the symmetrical and asymmetrical excavations. The heave of the Line 6 tunnel from the asymmetrical excavation is about 6-10% smaller.

In the original plan, the pits on both sides of the tunnel are constructed in a symmetrical way. So there is almost no horizontal movement for Line 6 tunnel. The asymmetrical excavation, however, can cause horizontal displacement of Line 6, which is 6.4 mm for the 3rd Plan, and 4.9 mm for the 4th Plan. The result of the 3rd Plan is larger than that of the 4th Plan because its unloading is much more asymmetrical. The vertical and horizontal movements together may cause much more hazards for the running tunnel.

Table 4. Excavation procedure for different plans.

Excavation Plan		Excavation procedure
Symmetrical excavation	Original Plan	Large first: LL+RL→LS+RS
	2nd Plan	Small first: LS+RS→LL+RL
Asymmetrical excavation	3rd Plan	LL→LS→RL→RS
	4th Plan	LL→RL→LS→RS

Table 5. Comparison of tunnel's heave for different excavation plans.

Excavation Plan		Heave during divided excavations (mm)				Total (mm)
Symmetrical excavation	Original Plan	LL+RL		LS+RS		21.41
		5.22		16.19		
	2nd Plan	LS+RS		LL+RL		20.16
		15.22		4.94		
Asymmetrical excavation	3rd Plan	LL	LS	RL	RS	19.41
		2.26	7.40	2.58	7.17	
	4th Plan	LL	RL	LS	RS	20.15
		2.26	2.96	7.66	7.27	

4.2 Influence of Resistance Piles

The resistance piles are designed to reduce the vertical displacement of the running tunnel. If the resistance piles are removed in the original plan, the vertical displacements at the typical points would develop as shown in Figure 15. The location of the typical points is the same as defined in Figure 10. The vertical displacements at Points A, B, and C are similar to those in Figure 11. The final heave at Point D, however, is 32.3 mm, which is much larger than that when the resistance piles are included. The heave at Point D during the small pit excavations increases from 16.2 mm to 25.2 mm, more than 50%, when the resistance piles are removed. Therefore, installation of the resistance piles is an efficient and effective way to control the movement of the running tunnel.

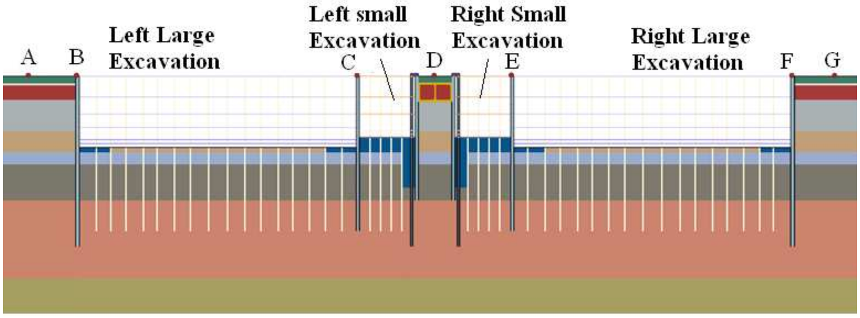
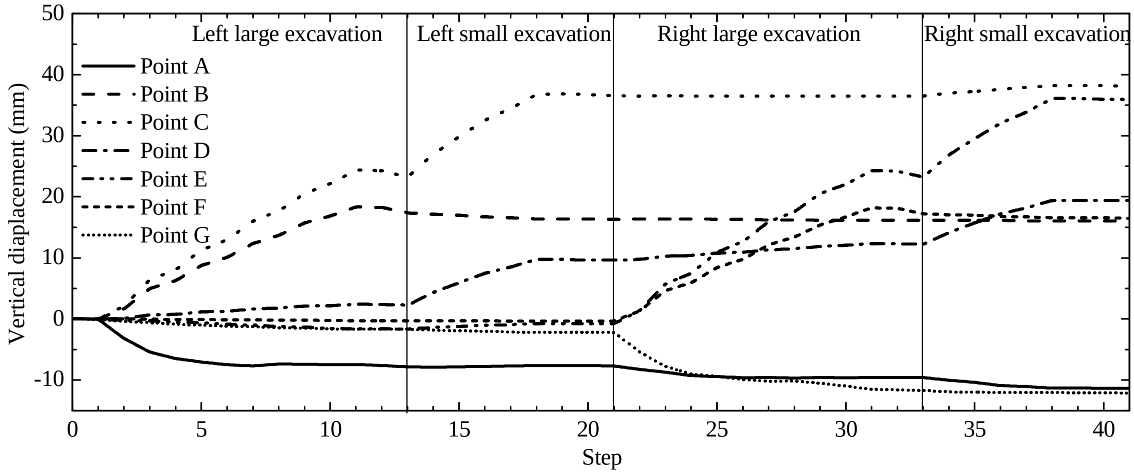
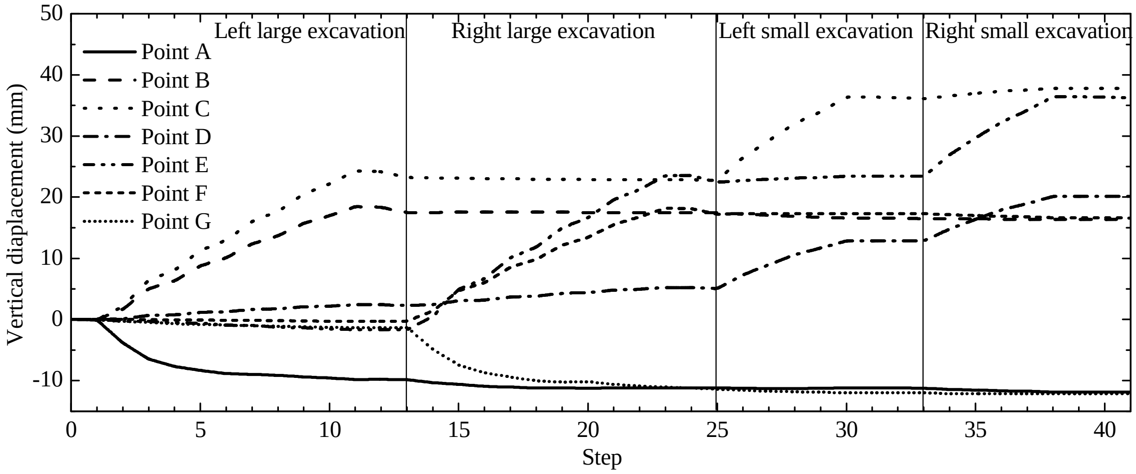


Figure 13. Location of seven typical points in asymmetrical excavation.



a. Results of 3rd plan



b. Results of 4th plan

Figure 14. Vertical displacements of seven typical points for the asymmetrical excavation.

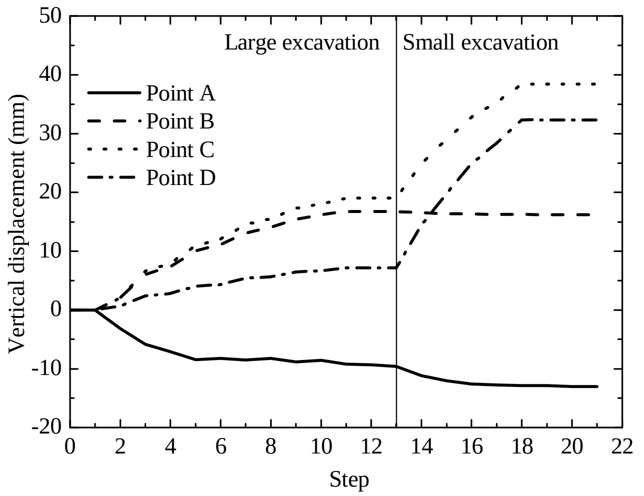


Figure 15. Vertical displacements of four typical points without resistance piles.

4.3 Influence of the RetainingWall of Cut-and-Cover Tunnel

The crossing part of Line 6 was constructed by the cut-and-cover method by considering the influence of the future excavation. The existing diaphragm wall of Line 6 is used as the retaining structure of the new excavation. Therefore, the deep excavation impacts Line 6 directly, and the heave of the tunnel is same as that of the diaphragm wall. If the crossing part of Line 6 was constructed as shield tunnels, this project would become an excavation adjacent to running shield tunnels, which has been reported in the literature (see, e.g., Hu 2003). But for this project, there are close excavations on both sides of the tunnel, and the distance between the excavations is less than 10 m.

Changing the cut-and-cover tunnel into a shield tunnel, separating the tunnel and the retaining structure of the excavation, and simulating the same excavation procedure as the original plan, the vertical displacements at four typical points can be obtained as shown in Figure 16, where Point D is the crown of the shield tunnel. Comparing Figures 16 and 11, it can be seen that the variation of displacements at Points A, B, and C is similar for the two types of tunnels. The vertical displacement at Point D for the shield tunnel, however, is very different from that for the cut-and-cover tunnel. The shield tunnel settles at the beginning of the large pit excavation, and then heaves when the excavation is deep enough. At the end of the large pit excavation, the heave of the shield tunnel is about 2.5 mm. The heave of the shield tunnel during the small excavations is about 9.1 mm. So the final heave of the shield tunnel is 11.6 mm which is about half of the heave for the cut-and-cover tunnel. Based on past experience (Chang et al. 2001), excavation will cause settlement of an adjacent shield tunnel as shows in Figure 1a. When there are excavations on both sides of a shield tunnel (Figure 1b), the tunnel will also settle if the distance between the excavations is large enough, say about twice the excavation depth (Hu et al. 2003). In our case, the shield tunnel does settle at the beginning of the large pit excavation. Since the clearance between the tunnel and the pits is very small (about half of the excavation depth) and both sides of the tunnel are unloaded, the shield tunnel would heave at the end of excavation. As shown in Figure 17, the vertical displacement of the soil adjacent to the diaphragm, u_{s1}, is smaller than the heave of the diaphragm, u_d, because of the slippage on the interface. The displacement of the soil decreases with the increase of distance from the diaphragm. So the vertical movement of the shield tunnel, u_t, is smaller than u_{s1}. The vertical movement of the cut-and-cover tunnel, u_t, however, would be the same as the displacement of the diaphragm because they tightly connected together. So the influence of the excavation on the adjacent shield tunnel is much smaller because the soil between the tunnel and the diaphragm wall can deform.

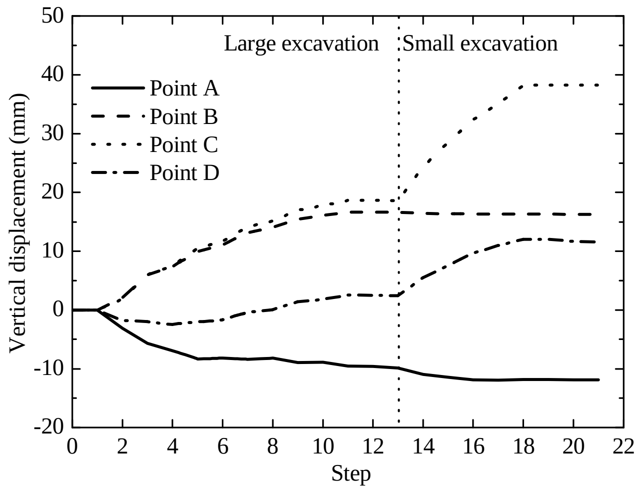


Figure 16. Vertical displacements of four typical points for a shield tunnel.

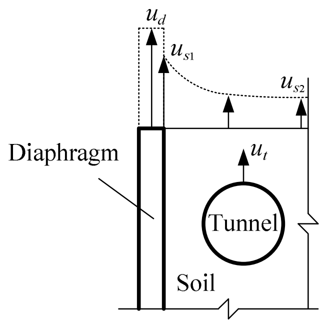


Figure 17. Sketch for the vertical displacement of diaphragm and adjacent shield tunnel.

5. DISCUSSIONS AND FIELD MEASUREMENTS

The numerical studies show that the tunnel of Line 6 heaves during excavations because the clearance between the two adjacent excavations is very small and the tunnel's diaphragm wall is used as the retaining structure of the new excavations. Asymmetrical excavation can reduce the tunnel' heave because it decreases the unloading magnitude during the excavation, but it causes larger horizontal movement of the tunnel which may impact the running tunnel. Therefore, symmetrical excavation should be adopted in the project.

Figure 18 compares the heave of the tunnel at four conditions. The Small First Plan can reduce the tunnel's heave by 6%. Considering the total construction duration, however, the Large First Plan (Original Plan) was adopted. The small pits increase the distance between the large pit and the running tunnel so that the influence of the large pit excavation is reduced. Most of the tunnel's heave is caused by the small pit excavations, which counts about 75% of the total heave. The resistance pile tied to the tunnel can control the vertical displacement of the tunnel effectively. The heave of the tunnel would increase from 21.4 mm to 32.3 mm if the resistance piles are not used. As stated earlier, the deep excavation impacts the tunnel directly because the existing diaphragm walls of the cut-and-cover tunnel are used as the retaining structure of the new excavations. That's why the heave of a shield tunnel would be only half of the cut-and-cover tunnel.

It needs to be noted that the influence of some other factors is not considered in this numerical analysis, such as dewatering, disturbance of soil, the creep deformation of soil, and the later loads on the substructures in large pits. For example, dewatering in deep soil layers and surcharge on the running tunnel may be adopted during excavation if the heave of tunnel is too large.

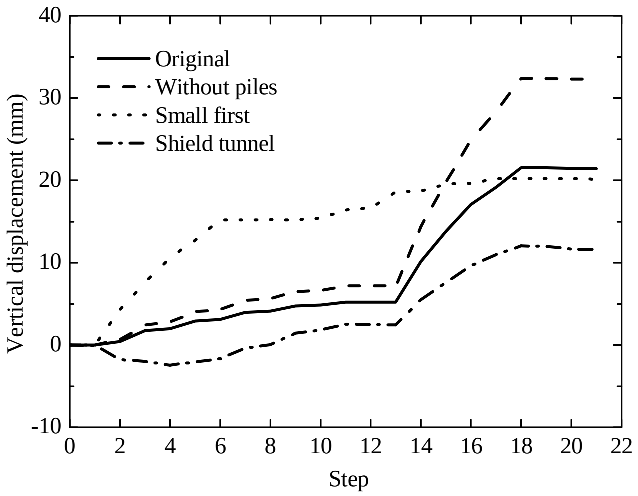


Figure 18. Comparison of tunnel's heave during excavation for 4 different conditions.

The excavation of this project was started in December, 2010. As of June 15, 2011, it has finished the 10th step as shown in Table 3. Two monitoring systems have been installed to measure the vertical displacements of the tunnel during the excavation. The field measurement results obtained so far are shown in Figure 19 and compared with the numerical results of the original plan. The locations of the measurement points are shown in Figure 2. The field measurements show that the crossing cut-and-cover tunnel heaves during the excavation. Since the heave of the tunnel increases rapidly even during the early stage of excavation, dewatering inside the pits is applied. After dewatering, the heave of the tunnel is stable and then decreases significantly.

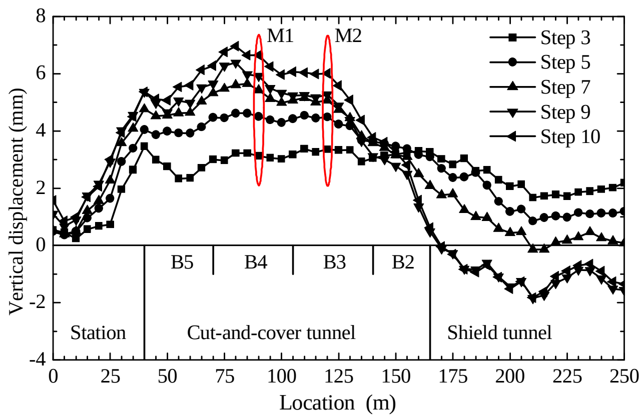
In general, the numerical predictions are in good agreement with the field measurements, especially considering the predicted trend of heave as opposed to the conventional trend of settlement as shown in Figure 1. The predicted heave values are smaller than the measurements mainly because the 2D numerical model used may have enlarged the restraint effect of the resistance piles although the stiffness of pile has been reduced considering their spaces. Moreover, treating the piles in the 2D model as a continuous wall divides the excavation into two parts and only the part between the diaphragm wall and the resistance piles can affect the diaphragm wall directly.

6. CONCLUSIONS

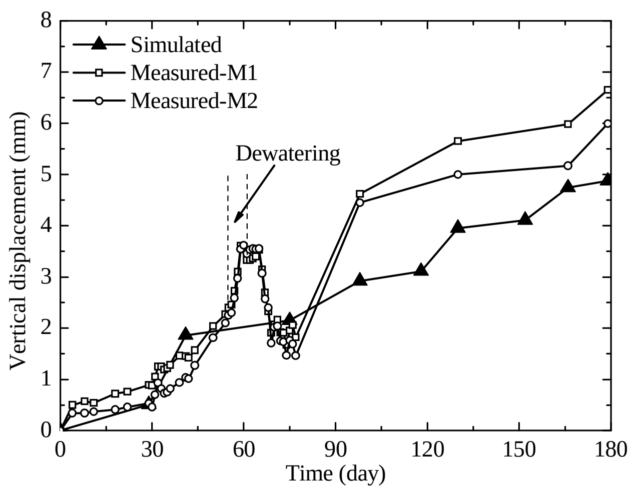
2D FEM modelling is used to study the effect of adjacent excavations on crossing tunnels and the numerical results are compared with the field measurements. Based on the study, the following conclusions can be drawn.

(1) The crossing tunnel is impacted by excavations on both sides. The cut-and-cover tunnel heaves during excavations because the distance between the two adjacent excavations is very small and the tunnel's diaphragm wall is used as the retaining structure of the new excavations. The predicted trend is verified by field measurements.

(2) The small pits close to the tunnel can reduce the influence of the large pit excavations because the small pits increase the distance between large pit excavation and the running tunnel. Most of the tunnel's heave is caused by the small pit excavations, counting about 75% of the total heave.



a. Distribution of the vertical displacement of Line 6 at different excavation steps



b. Variation of the vertical displacement of Line 6 during excavation

Figure 19. Field measurements of tunnel's heave during excavation.

(3) Asymmetrical excavation can reduce the tunnel' heave because it decreases the unloading during excavation, but it causes much larger horizontal displacement than the symmetrical excavation. The resistance piles tied to the tunnel can decrease the vertical displacement of the tunnel effectively.

7. ACKNOWLEDGEMENT

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