Wall-Soil Interaction Effects on Ground Movements Adjacent to Excavations

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ABSTRACT

Accurate prediction of ground movements is essential for assessing the potential risk of damaging structures adjacent to deep excavations. Numerous studies have previously been conducted to estimate the magnitudes and the distributions of ground movements. However, the wall-soil interaction effects have not been fully explored. Particularly, the soft toe condition, the effects of vertical loading on walls and the effects of the excavation widths have seldom been discussed. Presented herein is a parametric study conducted to quantify the influence of wall movements on vertical ground movements. A case history of the excavation in soft ground in the Taipei basin is collected for the studies. The excavation was retained by diaphragm walls of 31.5 m in length. Six cases with excavation widths of 11.2 m and 41.2 m with and without soft toes have been analyzed. The non-linear Hardening-Soil with Small Strain constitutive soil model is adopted. The stiffness parameters for the HSS soil model are validated by comparing the results of analyses with the observed ground movements.

Keywords: Deep Excavation, Surface Settlement, Heave, Hardening Soil, Wall Settlement

1 Introduction

Estimation of ground movements induced by excavation has been a challenging task for geotechnical engineers, particularly if the settlement of the retaining wall is to play a vital role. In the old days, excavations were generally shallow, say, up to 30 m in depth, so the retaining walls were rather stable and the settlements of walls were small. In recent years, as the demand for underground spaces keeps on growing, excavations tend to become deeper and deeper. For example, metro systems are being constructed in many major cities and excavations frequently exceed 30 m in depths for constructing stacked tunnels, river crossings and interchange stations and so on. As such, the settlements of retaining walls could have significant influences on ground movements and should be included in consideration in designs.

Retaining walls are dragged downward by the settling soils behind the walls and dragged upward by the heaving soil inside the pits. The interaction between the upward and the downward movements along both sides of the walls is rather complicated. As an excavation proceeds, the earth in the pit is removed stage by stage and the overburden pressures are reduced accordingly. As a result, the upward resistance on the inner face of the wall is reduced and the wall tends to settle. This is particularly true if the wall is used to support vertical loads such as those from the floor slabs of the top-down construction or ground anchors. In such cases, the settlements of the wall due to the vertical loads could be significant and should not be overlooked. Such complex soil-wall interaction effects can only be analyzed by using non-linear finite element methods.

To investigate the factors which affect the performance of retaining walls (i.e., diaphragm walls for deep excavations), hence, the ground movements, a parametric study has been conducted for this purpose and the results are presented herein.

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2 Case Studied

A hypothetical case resembling a section of the cut-and-cover tunnel in Taipei Metro was adopted as the subject of the study. The excavation was carried out by using the bottom-up method of construction to a depth of 16.5 m in 6 stages and was retained by diaphragm walls of 31.5 m in length and 0.8 m in thickness. A preliminary study on the case was previously reported in Wong and Hwang (2021) and the results of the supplemental analyses are available in an accompanying paper included in this volume of the proceedings (Wong 2022). Readers are encouraged to read these two papers to have a basic understanding of the case.

2.1 Ground Conditions

The site is located in the T2 Geological Zone of the Taipei Basin. Figure 1 depicts the soil strata at the sites and the excavation scheme. The Songshan Formation at the surface comprises six alternating sand (SM) and clay (CL) layers. Sublayers I, III and V are sandy soils with the N values increasing from 5 at 6 m depth to 30 at 44 m depth. Sublayers II, IV and VI are clayey soils. Due to extraction of groundwater in the underlying gravelly Jingmei Formation, significant reduction in water pressures in Songshan Formation and substantial ground settlements as a results. The piezomteric levels in the Jingmei Formation did not recover until mid-1790s. The subsoils in the Songshan Formation in the Taipei Basin are substantially over-consolidated. The hardening soil model with small strain is adopted to simulate the non-linear relationship between stresses and strains of soils. Readers are urged to read Wong (2022) for soil properties and groundwater conditions.

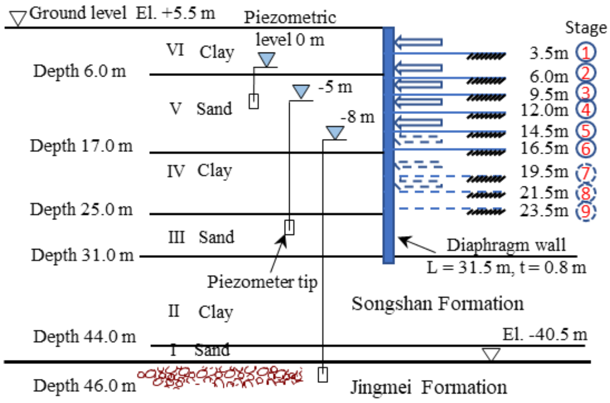


Figure 1: Soil profile of the Cross-over tunnel and excavation scheme

2.2 Parameters Studied

As depicted in Table 1, numerical analyses were carried out for 6 cases to study the influences of the toe condition, loading on the wall, and the width of the pit on wall settlements and ground movements. It is a well-known fact that sludge at the bottoms of bored piles is difficult to be completely removed and is likely to reduce the tip resistance of the piles upon loading. This is the so-called soft-toe condition.

Table 1: Scenarios analyzed

Scenario	Wall length L, m	Excavation width, B, m	Toe stiffness $E_{ref\text{-}50}$, MPa	Toe condition	Vertical load on wall, P_v, kN/m
Case 1	31.5	11.2	13.4	Normal toe	0
Case 2	31.5	11.2	0.134	Soft toe	0
Case 3	31.5	11.2	13.4	Normal toe	63
Case 4	31.5	11.2	0.134	Soft toe	63
Case 5	31.5	11.2	13.4	Normal toe	160
Case 6	31.5	41.2	13.4	Normal toe	0

Diaphragm walls are, in a sense, rectangular bored piles and would be in the same situation. In fact, barrettes, installed by using the diaphragm walling technique, are frequently adopted as foundation piles. Therefore, it is reasonable to expect that the tip resistance at the diaphragm wall toes will be minimal. Cases 1 and 2 simulate the conditions of the walls without and with soft toes. Case 1 is the benchmark case for comparison with the rest of the cases. Cases 3 to 5 are conducted to evaluate the effects of vertical loads on the walls without and with soft toes, and Case 6 is conducted to evaluate the effects of the width of the pit on the ground movement.

2.3 Finite Element Model

The finite element mesh for the excavation cases with a wall length of 31.5 is presented in Figure 2. Readers are urged to read Wong (2022) for the structural properties of the wall and the steel struts. Although the excavation was carried out to a depth of only 16.5m in 6 stages, analyses were conducted to a depth of 23.5m by adding 3 more stages to investigate the stability of the wall if the excavation were continued.

Since the underlying gravelly Jingmei Formation is very stiff and can be assumed as the competent formation with minimal ground movements, conventionally, the bottom of finite element models is placed at the top of the Jingmei formation. However, a Jingmei Formation layer of 15 m in thickness is included in this study to account for some of the contribution from this formation to the ground movements.

The wall is simulated by a plate element that has the zero thickness and by soil clusters of 0.4 m in width on both sides of the wall to achieve the wall thickness of 0.8 m. The soft toe condition for Cases 2 and 4 is simulated by a soil cluster of 0.8 m in width and in thickness at the depths between 31.5 m and 32.3 m. The stiffness for the soft toe is taken as 1/100 of that for the surrounding soil stratum to see if it makes differences.

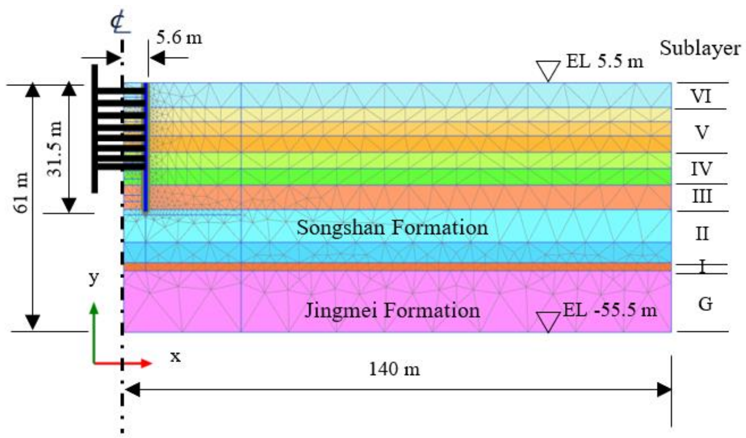


Figure 2: Finite element mesh for Cases 1 to 5

3 Lateral Wall Deflections and Surface Settlements

The computed wall deflections and ground settlements for the various stages of excavations in Case 1 and Case 6 are presented in Figure 3 and Figure 4 respectively. As the wall toe is located at 31.5 m depth, the portions of the profiles above the toe level are wall deflection profiles and those below the toe level are horizontal ground movement profiles. The graphic presentations of the results obtained in Cases 2 to 5 are similar to those for Case 1 and are thus omitted. Instead, the maximum movements are given in Table 2.

3.1 Effects of Depth Of Excavation

As can be noted, there is a general trend that the deeper the excavation, the larger wall deflections and ground settlements would be. The wall deflections and surface settlements for Case 1 and Case 3 are similar to each other at the excavation depths all the way down to 23.5 m. Table 2 summarized the computed maximum wall deflections and maximum surface settlements for the excavation depths of 16.5 m and 23.5 m.

As can be noted, as the depths of excavation increase from 16.5 m to 23.5 m, the maximum wall deflections increase from 33.1 mm to 35.3 mm, or roughly by 6.6 %, and the maximum settlements increase from 22.9 mm to 29.1 mm, or roughly by 27 %. The percentages of increases are of similar magnitudes in the other 3 cases. It appears that the surface settlements are more sensitive to the increase in the depth of excavation than wall deflections. This means, the ratios between the maximum surface settlement and the maximum wall deflection are not constant, but increase as the excavation depths increase. Take Case 1 as an example, this ratio is 22.9/33.1= 69 % for a depth of excavation of 16.5 m and increases to 83 % for a depth of excavation of 23.5 m.

Table 2 shows that differences in wall deflections and surface settlements are insignificant between Case 1 and Cases 2 to 5. For the cases with normal toes and with soft toes, and for the cases with the vertical loads on wall varying from 63 kN/m to 160 kN/m, the wall deflections are virtually the same. The differences in surface settlements are negligible, within 1 mm for the cases with and without vertical loads.

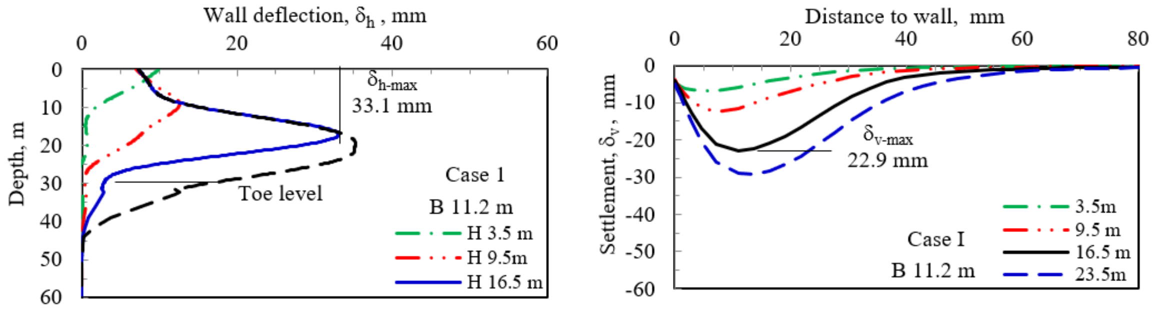


Figure 3: Computed wall deflections and surface settlements for Case 1

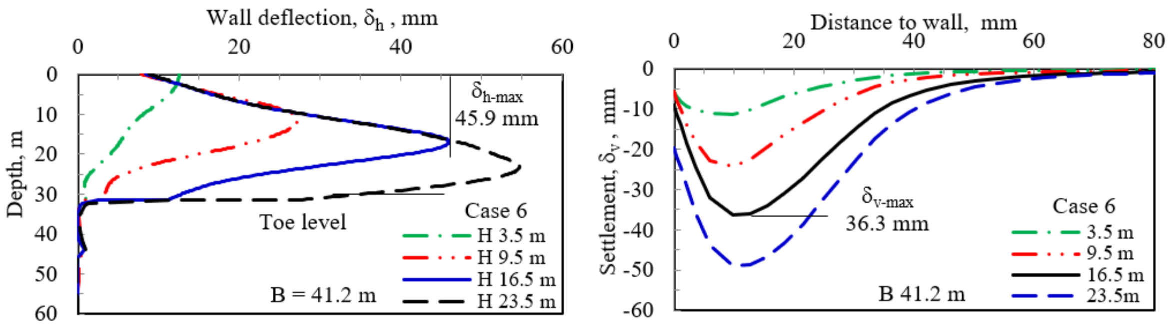


Figure 4: Computed wall deflections and surface settlements for Case 6

Table 2: Computed wall and ground movements for the excavation depths 16.5 m and 23.5 m

Studied Case	Vertical load on wall, P_v, kN/m	Excava-tion width B, m	Toe condition	Excava-tion depth, H m	Maximum deflection		Maximum settlement	
					Wall deflection $\delta_{h\text{-max}}$, mm	Incre-ment %	Surface settlement $\delta_{v\text{-max}}$, mm	Incre-ment %
Case 1	0	11.2	Normal	16.5	33.1		22.9	
				23.5	35.3	6.6	29.1	27
Case 2	0	11.2	Soft	16.5	33.2		23.2	
				23.5	35.3	6.5	29.6	28
Case 3	63	11.2	Normal	16.5	33.1		23.2	
				23.5	35.2	6.6	29.5	27
Case 4	63	11.2	Soft	16.5	33.1		23.4	
				23.5	35.3	6.6	30.0	28
Case 5	160	11.2	Normal	16.5	33.1		23.4	
				23.5	35.3	6.4	29.9	27
Case 6	0	41.2	Normal	16.5	45.9		36.3	
				23.5	54.6	18.9	48.9	35

3.2 Effects of Excavation Width

Larger excavation widths would likely cause larger wall deflections and surface settlements. The comparison between Figures 3 and 4 shows that for the wider excavation in Case 6, the computed maximum wall deflection for the excavation depth of 16.5 m is 45.9 mm, which is 39 % larger than the maximum wall deflection of 33.1 mm for the narrower excavation in Case 1. Similarly, the maximum surface settlement for Case 6 is 36.3 mm, which is 59 % larger than the maximum wall deflection of 22.9 mm obtained in Case 1.

Table 3 summarizes the maximum wall deflections near the excavation levels, the toe deflections at 31.0 m and at 31.5 m depths and the horizontal ground movements occurring at 32.3 m depth for Case 1 and Case 6. It can be noted that the wall deflections for Case 6 above the toe level for various excavation depths are larger than those for Case 1. The ratio in maximum deflections between Case 6 and Case 1 is 1.39 for the excavation depth of 16.5 m. The toe deflections occurring at 31.5 m depth are identical between Case 1 and Case 6.

The trend for the wider excavation to cause larger wall deflection reverses for the horizontal ground movements beneath the wall toe. Table 3 summarizes that the lateral ground movements occurring at 0.8 m below the toe level are 0.6 mm for the excavation depth of 16.5 m for Case 6, which is less than the value of 2.8 mm for Case 1. For the excavation depth of 23.5 m, the lateral ground movement at that depth for Case 6 is 1.2 mm, which is less than 12.5 mm for that of Case 1. The ratios in ground deflection at that depth between Case 6 and Case 1 vary from 0.1 to 0.2 for the excavation depths larger than 16.5 m.

The normal trend of the wider excavation width resulting in larger wall deflection could be partially attributable to the stiffness of the propping system, which uses the elastic steel struts as the primary lateral supports. The reverse trend in ground movements below the wall toe level could be attributed to the nonlinearity of soil. With the wider excavation width, the ground would be in smaller strain levels and the soil stiffnesses would be larger. As a result, less ground movements would occur in Case 6 than those in Case 1.

Table 3: Horizontal wall and ground movements along the wall for Case 1 and Case 6

Scenario	Excavation depth H, m	Wall deflection, mm			Ground deflection, mm	Ratio of Case 6/Case 1	
		Maximum	0.5 m above toe level	At toe level	0.8 m below toe level (at 32.3 m depth)	Maximum deflection	0.8 m below toe
Case 1	16.5	33.1	2.8	2.6	2.8	-	-
	23.5	35.3	13.4	11.9	12.5	-	-
Case 6	16.5	45.9	11.7	2.6	0.6	1.39	0.21
	23.5	54.6	29.8	11.9	1.2	1.55	0.10

4 Wall Settlements

4.1 Soft toe Effects

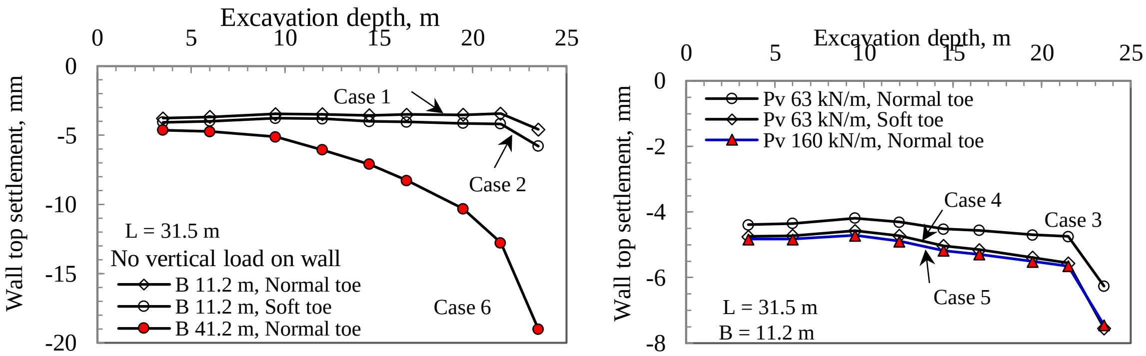
The settlements of the walls in Case 1 to Case 6 are summarized in Table 4 and depicted in Figure 5. For excavation depths down to 23.5 m, the settlements at wall tops would be 4.6 mm and 5.8 mm for the walls without (Case 1) and with soft toes (Case 2) respectively. The soft toe condition would increase the settlements of the walls by only 1.2 mm.

4.2 Effects of Vertical Loading on Wall Settlements

To study how vertical loads on diaphragm walls would affect the settlements of the wall and the ground, analyses were conducted in Cases 3 to 5. Ideally, loads should be applied stage by stage to account for the vertical components of strut loads and loads from superstructures, if any. However, such a process would raise numerous questions regarding how the magnitudes of loads are determined and how the loads should be applied. Since this is a qualitative study, the unnecessary complication will cause confusion and defeat the purpose of the study. For simplicity, in the finite element models for Case 3 (with normal toe) and Case 4 (with soft toe), a vertical load, P_v, of 63 kN/m is applied along the entire length of the wall to simulate the loads from 5 levels of struts and waling and 10 kPa from the traffic deck for the bottom-up construction. In Case 5, this vertical load is increased to 160 kN/m to account for the weight of 5 levels of concrete slabs of 0.25 m in thickness, supported on king posts of 8 m span aligned along the transverse direction in top-down constructions and loads of 10 kPa on the uppermost slab. The toe stiffnesses and the loads applied for Case 3 to Case 5 are summarized in Table 1.

Table 4: Wall settlements for the excavation depths of 16.5 m and 23.5 m

Scenario	Excavation width, B, m	Toe condition	Vertical load on wall, P_v, kN/m	Excavation depth, H, m	Wall settlement, mm		Wall shortening, mm
					Wall top	Wall toe	
Case 1	11.2	Normal	0	16.5	-3.5	-2.8	0.7
				23.5	-4.6	-3.6	1.0
Case 2	11.2	Soft	0	16.5	-4.0	-3.4	0.7
				23.5	-5.8	-4.8	1.0
Case 3	11.2	Normal	63	16.5	-4.6	-3.8	0.8
				23.5	-6.3	-5.2	1.1
Case 4	11.2	Soft	63	16.5	-5.2	-4.4	0.8
				23.5	-7.5	-6.5	1.0
Case 5	11.2	Normal	160	16.5	-5.3	-4.5	0.6
				23.5	-7.4	-6.3	1.1
Case 6	41.2	Normal	0	16.5	-8.3	-7.5	0.8
				23.5	-19.0	-18.1	0.9



(a) No vertical load (b) With vertical load

Figure 5: Computed wall settlements for cases with and without vertical loads

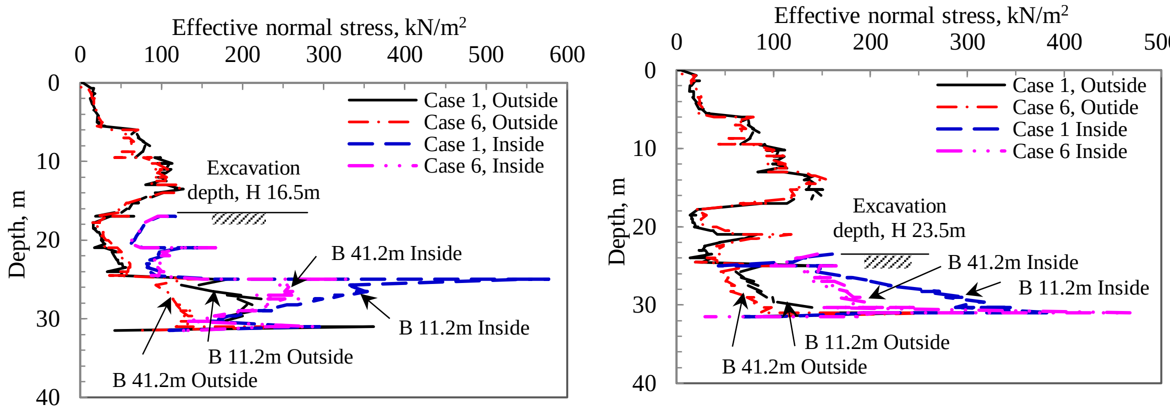
As can be noted from Figure 5 and Table 4, the computed settlements of the wall top for a depth of excavation of 16.5 m are 3.5 mm in Case 1, 4.6 mm in Case 3, 5.3 mm in Case 5 and 8.3 mm in Case 6. It can also be noted in Figure 5(b) that the performance of Case 4 with soft toe would be similar to that for Case 5 with normal toe. The difference in the wall settlements between Case 4 and Case 5 is within 0.2 mm.

4.3 4.3 Effects of Excavation Width

The settlements of the wall top obtained in Case 6 with an excavation width of 41.2 m are presented in Figure 5(a) and compared with those obtained in Cases 1 to 4 in Table 4. Compared with those cases having a width of excavation of 11.2 m, larger wall settlements would occur with a larger excavation width. At the excavation depth of 23.5 m, the settlement of the wall top would be as large as 19.0 mm, which is 4 times that of the case with the excavation width of 11.2 m.

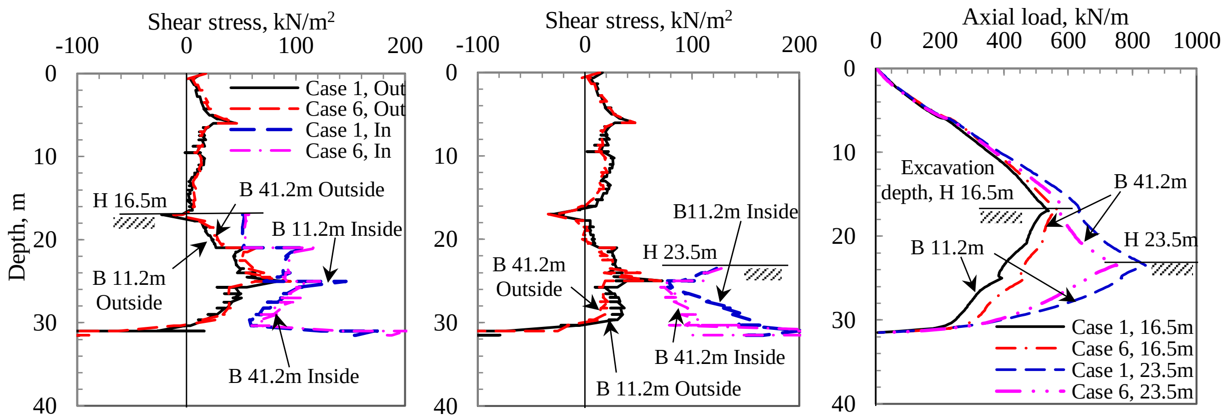
The larger settlements at the wall top for larger excavation width for Case 6 could be caused by less earth pressures acting along the wall. Figure 6 presents the effective normal stress acting on the active and the passive sides of the wall for Case 1 and Case 6 for the excavation depths of 16.5 m and 23.5 m. At the depths between 25 m and the toe of 31.5 m, where the wall is embedded in the sandy sublayer III, the effective normal stresses developed in Case 6 are less than those developed in Case 1.

Figure 7 presents the shear stresses mobilized on both sides of the wall and the axial loads along the wall. Figure 7(a) shows that there is not much difference in shear stresses between Case 1 and Case 6 for the excavation depth of 16.5 m. Figure 7(b) however shows that the shear stresses mobilized on the passive side of the wall in Case 1 would be larger than those in Case 6 for the excavation depth of 23.5 m.



(a) Excavation depth 16.5 m (b) Excavation depth 23.5 m

Figure 6: Effective normal stresses on the active and the passive sides of wall for Case 1 and Case 6



(a) Excavation depth 16.5 m (b) Excavation depth 23.5 m (c) Axial load along wall

Figure 7: Shear stresses on both sides of the wall and axial load along the wall for Case 1 and Case 6

Table 5: Mobilization of wall toe resistances for Case 1 and Case 6

Scenario	Excavation dimension, m		Axial force on wall kN/m		Ratio of axial force between Case 6 & Case 1		Wall toe settlement mm
	Width, B	Depth, H	Maximum	At toe	Maximum	At toe	
Case 1	11.2	16.5	539.0	6.4	-	-	-2.8
		23.5	841.2	10.4	-	-	-3.6
Case 6	41.2	16.5	570.5	11.1	1.06	1.73	-7.5
		23.5	751.4	27.4	0.89	2.63	-18.1

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The downward wall movements, as those summarized in Table 4, would mobilize the base resistance at the wall toe. Figure 7(c) presents the variation of the axial loads on the wall with depth. The axial load profiles show that the maximum axial loads occur at the excavation depths. This load-transfer distribution indicates that the shearing stresses acting on the active side induce the negative skin friction (NSF) along the wall. That NSF is counter-reacted by the positive skin friction on the passive side of the wall below the excavation level.

Table 5 summarized the maximum axial loads acting along the wall and those transferred to the toe for Case 1 and Case 6. Compared with Case 1, the axial load at the wall toe for Case 6 increases by 82 %, from 6.4 kN/m to 11.1 kN/m for the excavation depth of 16.5 m. The toe settlement of 7.5 mm is required to mobilize the toe resistance of 11.1 kN/m. Similarly, for the excavation depth of 23.5 m, the toe settlement of 18.1 mm would be required to mobilize the toe resistance of 27.4 kN/m. For excavations with large widths and large depths, it would be prudent to monitor wall settlements.

4.4 Case History on Monitoring of Wall Settlements

Monitoring on wall settlements was conducted for metro station BL16 of the Nangang Line located at the eastern rim of the Taipei Basin (Chen et al.1999). The site is located in the K1 Geological Zone of the Taipei Basin and the pit was retained by diaphragm walls of 1 m in thickness and 31.5 m in length. Due to the poor ground conditions, concrete cross-wall panels of 1 m in thickness, 3 m in depth, and spacing at 5 m were installed beneath the final excavation level to reduce wall deflections and ground movements. It was thus able to limit the maximum wall deflection to 25 mm as the final excavation depth of 15.2 m was reached.

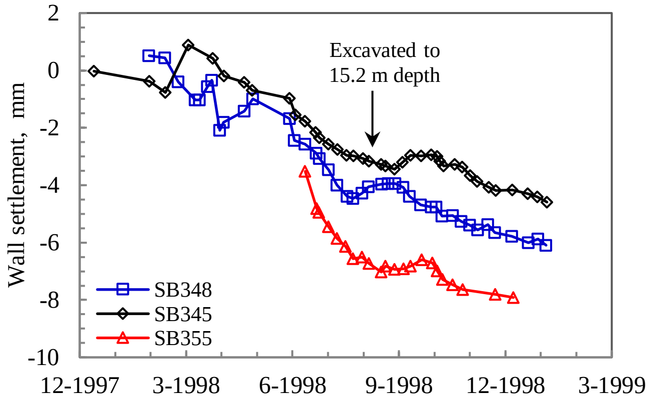


Figure 8: Wall settlement observed at Station BL16 (after Chen et al. 1999)

Figure 8 shows that the settlements recorded by 3 settlement markers installed on the wall top ranged from 3 mm to 7 mm in the final excavation stage and increased to 5 mm to 8 mm in the backfilling stage. As summarized in Table 4, the computed settlement at the wall top for Case 1 at the excavation depth of 16.5 m is 3.5 mm without soft toe. For Case 2 with soft toe, the wall top settlement at the excavation depth of 16.5 m is 4.0 mm. The computed wall top movements are in agreement with those observed in the case history of Station BL16. The prolonged wall settlements would likely be caused by the consolidation of soft clay in the Songshan Formation due to the lowering of the groundwater table.

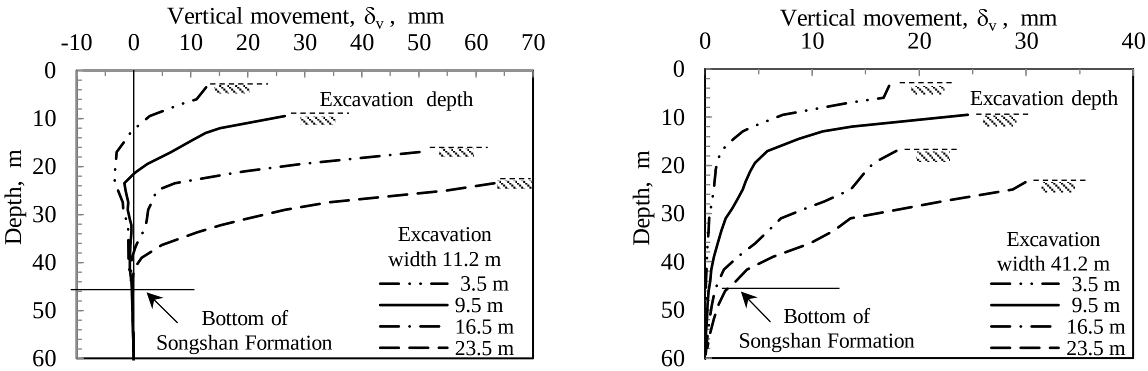
The width of excavation for Station BL16 is approximately 25 m. Interpolating the results obtained in Case 3 and Case 6, refer to Table 4, the settlement of the wall top for an interpolated excavation width of 26.5 m with traffic deck would be around (4.6 + 8.3)/2 = 6.5 mm. As shown in Figure 8, this interpolated wall top settlement is consistent with the values ranging from 3.2 mm to 7.0 mm recorded by the 3 settlement markers at Station BL16. The consistency between the observed and computed wall settlements validates the results of the parametric studies.

5 Heave in Excavation Trench

5.1 Computed Ground Heaves for Narrow Excavation

Figure 9 presents the heaves induced along the central axis of the trench in Case 1 and Case 6. The computed results show that the maximum heave in each stage occurs at the corresponding excavation level. The heave zone propagates downward as excavation depths increase and diminishes at the base of the Songshan Formation, where the underlying gravelly Jingmei Formation has much larger stiffnesses than those for the sandy and clayey Songshan sublayers.

As shown in Figure 9(a) and summarized in Table 6, the maximum ground heave for the excavation width of 11.2 m (i.e., Case 1) would increase from 12.7 mm to 63.2 mm as the depths of excavation increase from 3.5 m to 23.5 m. It is interesting to note that, instead of heaving, the soils below a depth of 10 m actually settle in Stage 1 excavation to a depth of 3.5 m. The largest settlement in the trench is 3.3 mm, which is slightly less than the settlements of the wall of 3.8 mm. Essentially the soils sunk together with the walls due to the plugging effects because of the narrowness of the trench.



(a) Case 1 (b)Case 6

Figure 9: Profiles for heaves induced along the central axis of excavation for Case 1 and Case 6

Table 6: Computed heaves and settlements along the central axis and the wall

Studied Case	Excavation width, B, m	Excavation depth, H, m	Ground movement along axis, mm		Wall settlement, mm	
			Maximum heave	Maximum settlement	Top	toe
Case 1	11.2	3.5	12.7	-3.3	-3.8	-3.6
		9.5	26.5	-1.6	-3.5	-3.1
		16.5	50.1	-0.5	-3.5	-2.8
		23.5	63.2	-0.1	-4.6	-3.6
Case 6	41.2	3.5	17.2	0	-4.6	-4.3
		9.5	24.6	0	-5.1	-4.6
		16.5	17.9	0	-8.3	-7.5
		23.5	29.9	0	-19.0	-18.1

5.2 Computed Ground Heaves for Wide Excavation

It is envisaged that the plugging effects would diminish as the width of the trench increases. As depicted in Figure 9(b), the settlement does not occur along the axis of excavation in Case 6 with an excavation width of 41.2 m. Figure 9(b) shows that the maximum ground heave for the excavation width of 41.2 m (i.e., Case 6) would increase from 17.2 mm to 29.9 mm as the depths of excavation increase from 3.5 m to 23.5 m. For the excavation depths exceeding 16.5 m, the heaves along the axis for the wider Case 6 are 36 % (H = 16.5 m) to 47 % (H = 23.5 m) of those for the narrower excavation in Case 1.

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5.3 Case History on Monitoring of Ground Heaves

It is desirable to compare the results of analyses on ground heave inside the excavation trenches with field observations. However, the case histories on the ground heave inside the pits are rather limited. The cases on heaves subsequent to the end of excavation are even less. Nash et al. (1996) presented the observed heave profiles inside an excavation area in Gault Clay in the Cambridge area in London, UK. The over-consolidation Gault clay has the thickness of 40 m which overlies Lower Greensand, which is an aquifer at depth. Prior to excavation the initial pore pressures were about 7 mOD (metre above Ordnance Datum), which was 3 m below the ground level of 10 mOD. The excavation area was 65 m x 45 m in plan. The excavation was carried out to -0.5 mOD, giving a total depth of excavation of 10.5 m. The pit was retained by diaphragm walls of 17 m in length.

Extensometers were available at a distance of 5m from the centreline of the excavation area at depths ranging from 4 mOD to -25.1 mOD. The development of heaves in different stages of excavation is shown in Figure 10. As the excavation reached its final level of -0.5 mOD, a heave of 30 mm was recorded. Monitoring on pneumatic piezometers indicated that the piezometric levels were originally between +6 mOD and +7 mOD and dropped to around -7 mOD as the excavation reached the final level. The piezometric levels rose to -0.5 mOD in 4.8 years after casting the base slab. The extensometer monitoring showed that the ground heave continued as a result of swell of soils linearly against time (in a log scale), from 30 mm to 110 mm, during this 4.8-year period.

As can be noted from Figure 9(b), as the excavation reached a depth of 9.5 m in Stage 3 in Case 6, the heave computed was 24.6 mm, which is of a similar magnitude of 30 mm as that reported by Nash et al. (1996) for a depth of excavation of 10.5 m. Since the ground conditions at the two sites were quite different, it is unrealistic to expect the heaves to be of the same in magnitudes. Nevertheless, the similar tendency presented in Figure 9(b) and in Figure 10 is very encouraging and gives confidence to the validity of the numerical analyses.

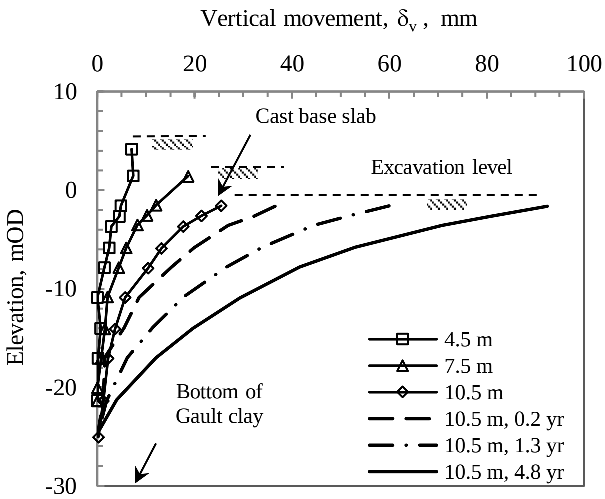


Figure 10: Heaves during excavation and subsequent heaves (Nash et al. 1996)

6 Ground Movement in Horizontal Sections

6.1 Ground Movements in Horizontal Section for Narrow Excavation

The computed vertical ground movements in the horizontal sections at depths of 17 m and 24 m for Case 1 are shown in Figure 11 and summarized in Table 7. Comparing with the results for Case 2 summarized in Table 4, the largest differences in settlements between normal toe and soft toe for wall lengths of 31.5 is around 0.5 mm for the excavation depth of 16.5 m. The vertical movement profiles for Case 2 with soft toe are similar with those for Case 1 and are thus not shown for sake of clarify.

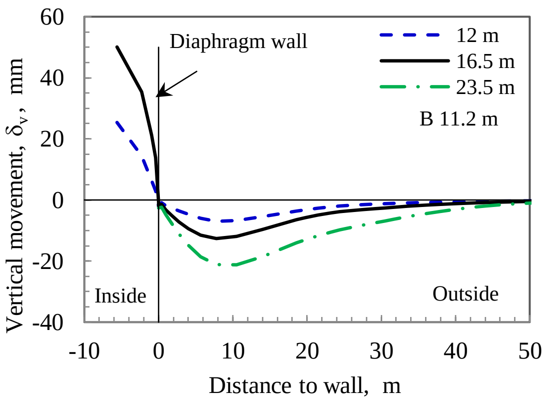
Table 7: Maximum vertical ground movements for Case 1 and Case 6

Studied case	Excavation area	Excavation depth, H, m	Maximum vertical movements, $\delta_{v\text{-}max}$ mm		Ratio of Case 1/Case 6	
			At 17 m depth	At 24 m depth	17 m depth	24 m depth
Case 1 B 11.2 m	Inside	16.5	50.2	5.7	-	-
		23.5	-	60.6	-	-
	Outside	16.5	-12.6	-3.5	-	-
		23.5	-21.2	-9.6	-	-
Case 6 B 41.2 m	Inside	16.5	30.5	14.0	0.61	2.46
		23.5	-	43.2	-	0.71
	Outside	16.5	-21.0	-8.4	1.67	2.40
		23.5	-38.7	-25.8	1.83	2.69

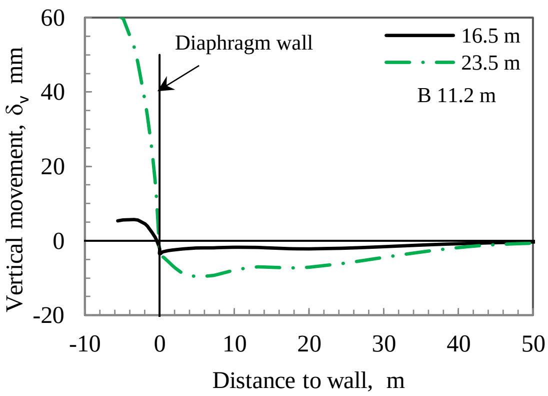
6.2 Ground Movements in Horizontal Section for Wide Excavation

Case 6 has an excavation width of 41.2 m, which is larger than the wall length of 31.5 m. The computed vertical ground movements for Case 6 in the horizontal sections at depths of 17 m and 24 m are shown in Figure 12. In comparison with the vertical ground movement profiles for the narrow excavation in Case 1 shown in Figure 11, the wider excavation Case 6 would cause smaller heaves and larger settlements for the horizontal sections above the wall toe levels. Table 7 shows that at the excavation depth of 16.5 m, the computed heave occurring inside the excavation trench at the horizontal section of 17 m depth for Case 6 is 30.5 mm, which is 61 % of 50.2 mm for that in Case 1. At the excavation depths of 23.5 m, the computed heave inside the pit at the section of 24 m depth for Case 6 is 43.2 mm, which is 71 % of 60.6 mm for the heave for Case 1.

Outside the excavation trench, the settlements for the wider excavation Case 6 are larger than those with the narrower excavation in Case 1. For example, Table 7 shows that at the excavation depth of 16.5 m, the maximum settlement at the section of 17 m depth for Case 6 is 21.0 mm, which is 167 % of the settlement of 12.6 mm for Case 1. At the excavation depth of 23.5 m, the maximum settlement outside at the section of 24 m depth is 25.8 mm, which is 269 % of the settlement of 9.6 mm for the narrow Case 1. The finding that wider excavation would cause larger ground settlements outside and less heave inside the excavation trench in the final excavation is consistent with the common experience.

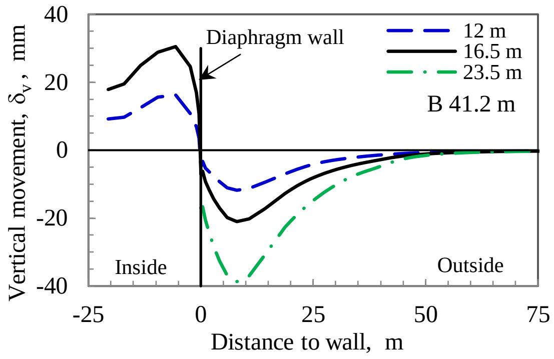


(a) Section at 17 m depth

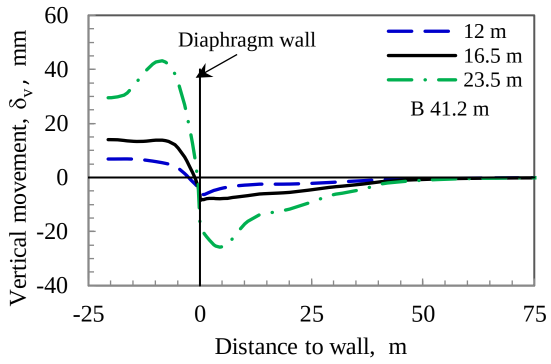


(b) Section at 24 m depth

Figure 11: Vertical movements on the inner and the outer side of narrow excavation - Case 1



(a) Section at 17 m depth



(b) Section at 24 m depth

Figure 12: Vertical movements on the inner and the outer side of wide excavation - Case 6

6.3 Case History on Monitoring of Ground Movements in Horizontal Section

Panchal et al. (2017) conducted a centrifuge modelling test on an underwater excavation case to study the relationship between the ground movements and basal heave below the final excavation level. The soil model comprised Speswhite kaolin clay mixed to a water content of 120 %. The excavation depth and the half-width for the prototype model were 12.0 m and 24 m respectively. The wall length was 20.8 m. The wall toe was 20 m above the base of the prototype model. The wall was deliberately very stiff, with a prototype stiffness equivalent to a 2.1 m thick concrete diaphragm wall. The LVDT and digital imaging were used to measure the ground displacements. Pore pressure transducers were embedded in the soil model on both sides of the wall.

The excavation was simulated by reducing air pressure of 202 kPa in a latex bag over a period of 3 minutes to simulate the excavation in 2 months. Measurements on displacements and pore pressures continued for a further 30 minutes, equivalent to 18 months at the prototype scale, after excavation to the observed long-term ground response. The test simulates the immediate and the long-term ground movements arising from unloading and soil softening respectively. As excavation proceeded, the 2 pore pressure transducers next to the wall on both sides recorded reduction in pore pressures. In the post-excavation stage, the negative pore pressures dissipated within 20 minutes.

Figure 13 shows the vertical ground movements on both sides of the wall in the horizontal section at the final formation level of 12 m in depth. Based on readings of the pore pressure transducers, the long-term changes in the vertical movements are a result of the dissipation of pore pressures. The trend of variations in ground heaves and settlements presented in Figures 11 and 12 is similar to that observed in the centrifuge tests presented in Figure 13.

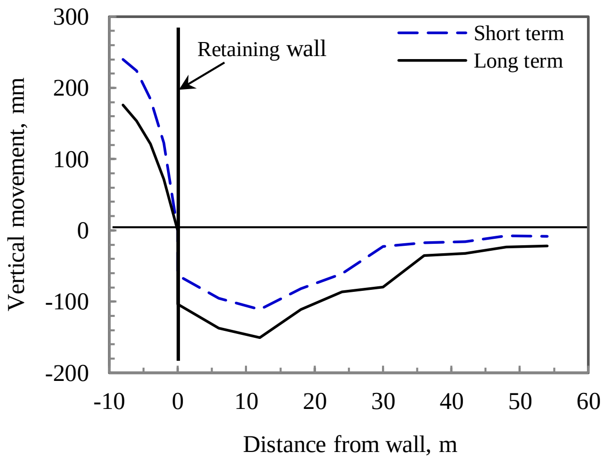


Figure 13: Vertical movement profiles obtained from centrifuge test (after Panchal et al. 2017)

7 Conclusions

Parametric studies have been conducted on excavations supported with diaphragm walls in soft ground with various excavation widths, wall toe stiffnesses and vertical loads on wall. The Hardening-Soil with small-strains stiffness is adopted for the constitutive soil model in the numerical analysis. The computed results have been verified with case histories on ground heaves and on wall settlements. The following conclusions could be drawn from the parametric studies:

(1) Narrower excavations would cause larger heave inside and smaller settlements outside the excavation trench than those occurring for wider excavations.

(2) Wider excavation width could cause lower in earth pressures acting on the passive side of the wall. The shear stresses along the wall on the passive side would then be less than those of the narrow excavations.

(3) The soft toe effects and the vertical loads on wall would be insignificant to wall deflections, ground settlements and to wall settlements.

Amongst the potential attributing factors evaluated in this study, the excavation width is the most dominant factor affecting the magnitude of wall deflections, surface settlements and wall settlements. Due to the non-linear behavior of soils, the wall-soil interaction and its effects on the wall and ground movements are complicate problems. The influences of wall lengths and excavation widths to wall and ground movements shall be the topics for the future studies.

8 Publisher's Note

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