Effects of Preloading of Struts on Retaining Structures in Deep Excavations

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ABSTRACT: The performance of an excavation of 19.4 m in depth in soft ground has been reviewed by interpreting the readings of inclinometers in wall of 35 m in length and strain gauges in six levels of struts. Assuming the wall deflections at the first strut level would not move after preloading, the corrected inclinometer readings show that the deflections at the wall toes and at the tips of inclinometers were as much as 43 % and 25 % of the maximum wall deflections respectively. The large toe and tip movements are verified by numerical analyses, which have been conducted to study the effects of preloading of struts as well. The strain gauge readings show that the preloads applied to the struts do not sustain and drop significantly after subsequent preloading of struts. Four cases, namely, struts with full preloads, 50% preload to the first strut level, zero preload and actually observed preloads, have been adopted in the analyses to evaluate the effects of preloads. The results of the numerical analyses using the Mohr-Coulomb model are then compared with the observed wall deflection profiles in the final excavation stages. The Young's moduli for clay and sand layers have been correlated with the soil strengths. It is found that computed peak strut loads are in agreement with the observed peak loads for the upper 3 levels of struts. For the lower 3 levels, the computed strut loads are however as much as 50% larger than those observed.

KEYWORDS: Deep excavation, Struts preloading, Wall deflections, Numerical analyses.

1. INTRODUCTION

Ground settlement which is one of the primary factors affecting the structures adjacent to excavations is closely related to the maximum wall deflections. The maximum wall deflections thus become the most important subject in evaluating the performance of diaphragm walls.

Wall deflections are routinely monitored by using inclinometers. The readings obtained are inevitably affected by the movements at the tips which are assumed to be fixed and wall deflections at other depths are calculated accordingly. Moh and Hwang (2005) and Hwang et al. (2007b) recommended to calibrate inclinometer readings by assuming that the joints between the struts at the first level and the diaphragm walls would not move once these struts are preloaded. This recommendation was based on the finding that the changes in the lengths of these struts were minimal as the load increments and/or decrements in the struts were small.

Since there are most likely underground structures adjacent to excavations in congested cities, and hence, wall deflections are inevitably affected by the presence of these structures. This is particularly true for excavations for underground stations and cut-and-cover tunnels, which are normally constructed underneath major streets with many high-rise buildings alongside, of metro systems. These high-rise buildings normally have basements together with retaining structures left in-place after the completion of construction, hence, deflections of walls in nearby excavations are very likely to be reduced as a result.

Furthermore, there are always entrances, ventilation shafts, etc., structurally annexed to the station walls and, therefore, the rigidity of the walls is much increased and wall deflections are much reduced. Since the structures adjacent to excavations are normally omitted in back analyses, comparison of the results obtained in back analyses with the observed performance of such walls is unrealistic and often leads to erroneous conclusions. It is therefore desirable to have a means to quantify the influence of adjacent structures, and also many other factors which may affect wall deflections, so the performance of walls can be realistically evaluated.

Wall deflections are also, inevitably, affected by the preloads applied on struts. Preloading of struts tends to increase the stiffness of the retaining system and thus reduce wall deflections. But, on the other hand, it might result in increase of the loads in struts as the excavation proceeds.

To illustrate the above-mentioned points, the wall deflections and strut loads obtained in the cut-and-cover construction for the crossover, denoted as Location 1 in Figure 1, next to Songjiang Nanjing Station of Taipei Metro are studied hereinafter. The excavation, refer to Figure 2 for the layout, was carried out to depths varying from 17.7m to 20.2m below the ground level in 7 stages. The pit was retained by diaphragm walls of 1m in thickness installed to a depth of 35m and braced by steel struts at 6 levels as depicted in Figure 3.

Numerical analyses were performed to verify the effects of preloading of struts on wall deflections and loads in struts as excavation proceeded. The results obtained are compared with the observed strut loads to see if the performance of the retaining structures can reasonably be analysed and predicted.

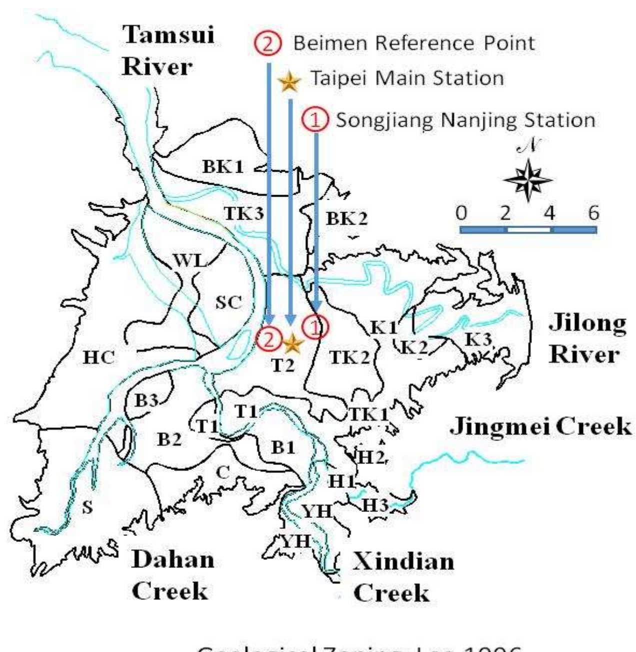

Figure 1 Geological zoning of the Taipei Basin and the locations of sites

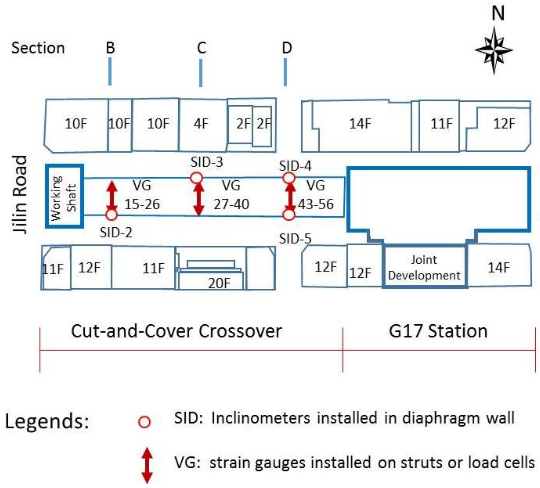

Figure 2 Layout of Songjiang Nanjing Station of the Green Line of Taipei Metro and the cut-and-cover crossover

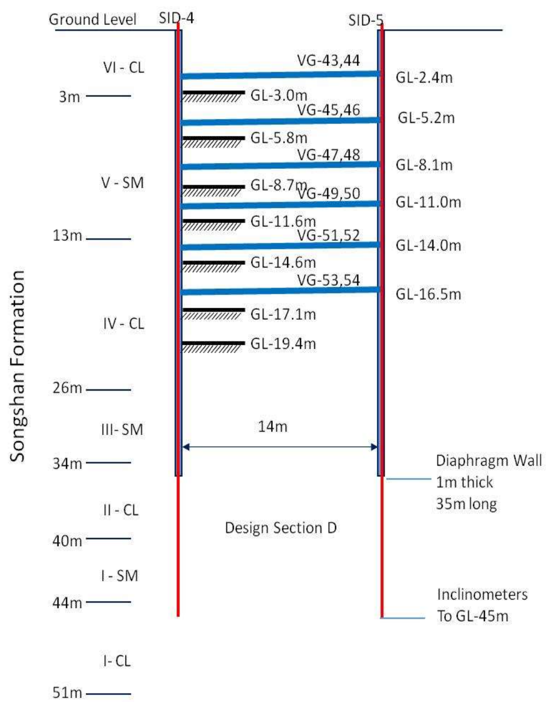

Figure 3 Excavation scheme for the crossover next to Songjiang Nanjing Station of Taipei Metro

2. CORRECTION OF INCLINOMEER READINGS

The case studied herein is of particular interest because the inclinometers were extended below the diaphragm wall toes by 10m. This provides a valuable opportunity to study the soil movements below the toes.

The original readings of the 4 inclinometers installed in the diaphragm walls on the two sides of the crossover are shown in Figure 4. A maximum reading of 26.7mm was recorded by SID-3 and a maximum reading of 8.9mm was recorded at the toe by SID-4. The drastic differences in readings between the diaphragm wall toes and the soil immediately beneath are also of interest. They demonstrate the capability of inclinometers to monitor movements at interfaces between two materials with drastically different stiffness.

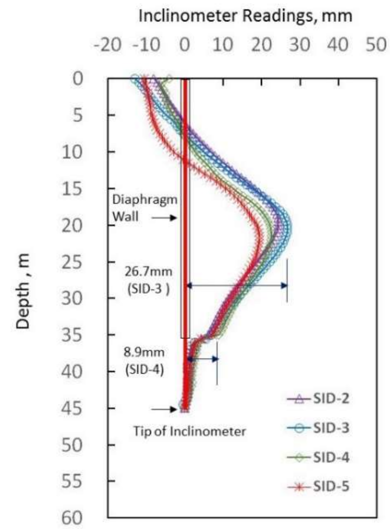

Figure 4 Original inclinometer readings obtained at the end of excavation before correction

2.1 Adjustment to include missing data

Back analyses for the performance of excavations are performed based on comparing the theoretical soil and structural response with the observations made during constructions. It is therefore vital to ensure that the instrument readings collected during construction truly represent what has occurred for the results of back analyses to be meaningful. It is quite common for the importance of instrumentation and monitoring to be overlooked, or even purposely ignored. As such, the instruments might not be properly installed and/or readings might not be properly taken. In such cases, the readings obtained might not be representative of the real response of the ground and/or the real response of the structures.

The initial readings for inclinometers SID-3, SID-4 and SID-5, for example, were taken rather too late. They were taken after the first stage of excavation had already been completed and the struts at the first level preloaded, therefore, the data for the first stage were missing from the records. Fortunately, the readings of SID-2 were taken properly since the beginning of excavation. As can be noted from Figure 5(a), wall deflections of, as much as, 6mm were recorded right after the preloading of the struts at the first level and deflections of this magnitude are certainly too large, in comparison with the maximum reading of 26.7mm, to be ignored. For practical purposes, it is reasonable to adjust the readings of these inclinometers by adding the deflections for Stage 1 excavation obtained by SID-2 to the readings obtained by other inclinometers.

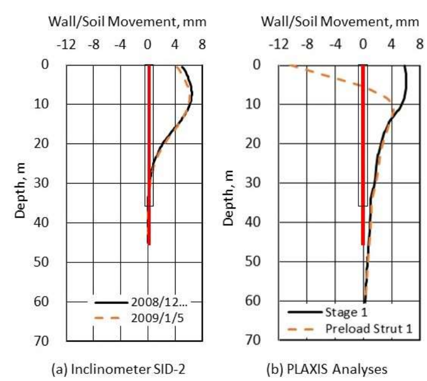

Figure 5 Influences of preloading the strut at the first level on wall deflections

2.2 Correction of readings to account for movements at tips of inclinometers

Furthermore, there is always the possibility that readings are interpreted incorrectly resulting in misleading wall deflections. Wall deflections are normally calculated from inclinometer readings with the assumption that the inclinometers were sufficiently long, or were embedded in a hardpan, so the tips of inclinometers would not move as the excavation proceeded. As such, the tips are normally assumed to be fixed and the wall deflections at other depths are calculated from the inclinometer readings accordingly.

Ideally, wall deflections so obtained can be verified by studying the changes in the lengths of the struts computed based on the loads in struts. In fact, this was exactly the purpose to install inclinometers at the two ends of the struts, e.g., between Inclinometers SID-4 and SID-5, in the case studied herein. However, the fact that the tips of inclinometers did move and wall deflections computed based on inclinometer readings were indeed affected by the movements at the tips totally defeated this purpose.

It has been suggested to calibrate inclinometer readings by assuming the joints between the struts at the first level and the diaphragm walls to be unmoved once these struts are preloaded (Moh and Hwang 2005; Hwang et al. 2012). Figure 6 shows the strut loads recorded by two strain gauges, namely, VG-43 and VG-44, installed on the first level strut between Inclinometers SID-4 and SID-5. The strut was preloaded to 71 tons, which is the average of the readings of the two gauges, at the beginning. The load in the strut increased to an average of 87 tons in the second stage of excavation and dropped to a minimum of -3 tons in the subsequent stages. For a strut with an sectional area of 173.9 cm^2, a length of 14m and a Young's Modulus, i.e. the E-value, of 200,000 N/mm^2 for steel, the increment of 16 tones, i.e., from 71 tons to 87 tons, corresponds to a shortening of 0.6mm of the strut, or an inward movement of 0.3mm at each end; and the decrement of 74 tons, i.e., from 71 tons to -3 tons, corresponds to a lengthening of 3mm of the strut, or an outward movement of 1.5mm at each end. Movements of such magnitudes are negligible for practical purposes and the joints between the struts and the diaphragm walls can indeed be assumed fixed for calibrating the inclinometer readings at other depths.

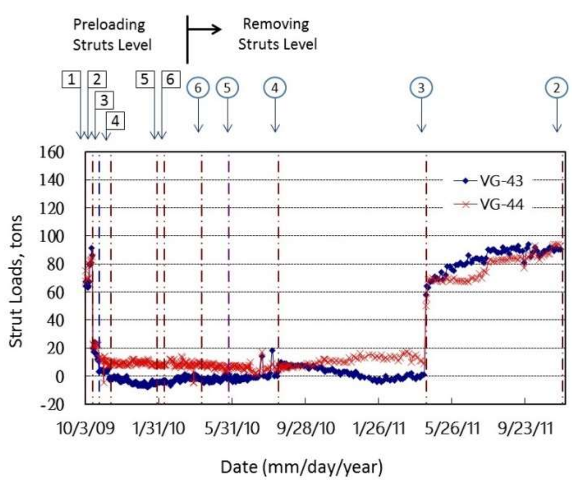

Figure 6 Loads in the 1st level strut at the location of Inclinometers SID-4 and SID-5

2.3 Wall deflections and movements at diaphragm wall toes and tips of inclinometers after corrections

The wall deflections computed based on the inclinometer readings which have been duly corrected as discussed above, are shown in Figure 7. The suffix "A" following the names of the inclinometers denotes that the readings have been adjusted to include the missing data for the movements occurring in the first stage of excavation as discussed in Section 2.1; and the suffix "C" denotes that the readings have been corrected to account for tip movements as discussed in Section 2.2. The readings for SID-2 were corrected, but not adjusted because the movements for the first stage excavation were already included.

Of the 4 inclinometers, SID-3CA gave the largest wall deflections of 36.3mm, while SID-5CA gave the largest movements of 16.7 mm at the toe of diaphragm wall. It has become quite common nowadays to install inclinometers in diaphragm walls and stop at the toe levels of the diaphragm walls to reduce construction costs. The toe movements were 43% of the maximum wall deflections for the case studied and analyses would certainly lead to misleading conclusions if inclinometer readings were not corrected.

It can also be noted from Figure 7 that the tips of the inclinometers still moved by, as much as, 10.2mm, or about 25% of the maximum wall deflections, even with a 10m extension below the toes of the diaphragm walls. The importance of correcting inclinometer readings is readily evident.

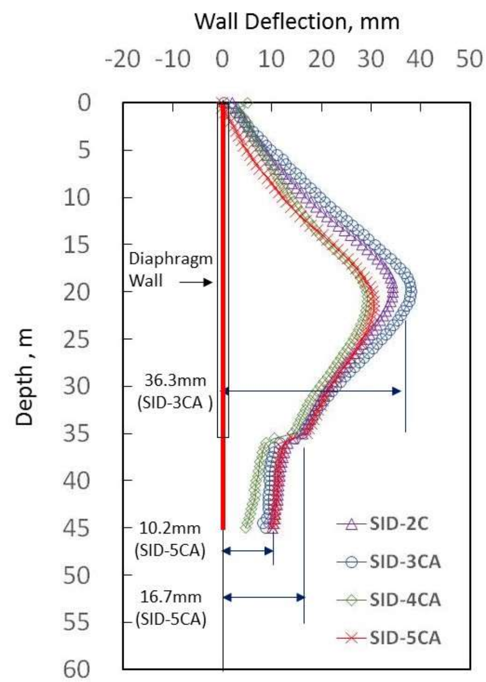

Figure 7 Final wall deflections computed based on the inclinometer readings with corrections

3. FINITE ELEMENT MODEL AND MATERIAL PROPERTIES

Numerical analyses were previously conducted by using the two-dimensional finite element computer program PLAXIS (PLAXIS BV 2011) with the soil strengths recommended by the Detailed Design Consultant, i.e., the designer, and the results are presented in Hwang and Moh (2017). It is one of the purposes of this paper to show the results of analyses which adopt, instead of those proposed by the designer, the strengths of clays proposed in an advanced research specifically conducted for revealing the geotechnical conditions for the design and construction of the metro system (Chin et al. 1994;2006).

3.1 Finite Element Model

Figure 8 shows the finite element model adopted. The deepest borehole at this site reached a maximum depth of 51m and a remark is noted in the borehole log indicating that the soils below this depth are Type SM (silty sand) soils. Based on the local geology, it was estimated that the Jingmei Formation, which is considered as a hardpan for the finite element analyses, lies at a depth of, roughly, 64m.

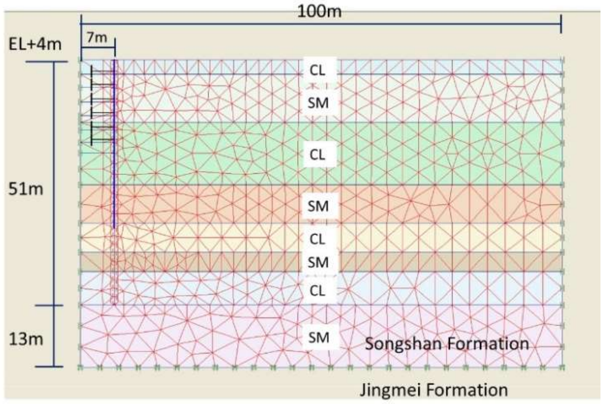

Figure 8 Finite element model for the Crossover next to Songjiang Nanjing Station

3.2 Properties of Structural Members

The diaphragm walls were simulated by plate elements and an E value of 25,000 MPa was adopted for concrete with an unconfined compressive strength, i.e. f'c value, of 280 kg/cm^2 (28 MPa). The EI (I = moment of inertia) and EA (A = sectional area) values of the diaphragm walls were reduced by 30%, giving a value of 1,464 MN*m for the former and 17,570 MN/m for the later, following the normal practice to account for the influence of tremieing and degradation of concrete during excavation. Struts were represented by anchor-to-anchor rods. The structural properties of the struts are shown in Table 1.

Table 1 Stiffness of struts adopted in numerical analyses

Level	Members	Sectional Area A (cm^2)	Stiffness, AE/S (MN/m)	Design Preload kN/m
1	1H350x350x12x19	1 x 173.9	891	178
2, 3	1H400x400x13x21	1 x 218.7	1121	339, 494
4	1H428x407x20x35	1 x 360.7	1849	428
5, 6	1H414x405x18x28	1 x 295.4	1514	458, 538
Note: Spacing between struts S = 4m				

3.3 Groundwater table and water pressures

Figure 9 shows the water pressures acting on the outer face of the diaphragm walls suggested by the Detailed Design Consultant. Inside the pit, the water table is assumed to be maintained at a depth of 1m below the bottom of excavation. Since the diaphragm wall penetrated 1 m into the clay layer, it has been assumed in the analyses that there are piezometric head differences at the toe level of the diaphragm wall between the outside and the inside of the station box in the various stages of excavation.

3.4 Soil Properties

The site is located right on the border dividing the T2 and TK2 Zones (Lee 1996) in the central Taipei Basin, refer to Figure 1. The lowering of piezometric level in the underlying Jingmei Formation, as depicted in Figure 10, has led to significant reductions of water pressures in the Songshan Formation and, as a result, ground has settled by more than 2m at Beimen Class 1 Reference Point which, as depicted in Figure 1, is located at a distance of about 2 km west southwest to this site. Due to the reduction of porewater pressures as a result of lowering the piezometric pressures in the Jingmei Formation, all the subsoils in the Songshan Formation in the central city area were substantially over-consolidated. This is particularly true for Layer II because the underlying Layer I is very permeable and the piezometric level in Sublayer I was practically the same as the piezometric level in the Jingmei Formation.

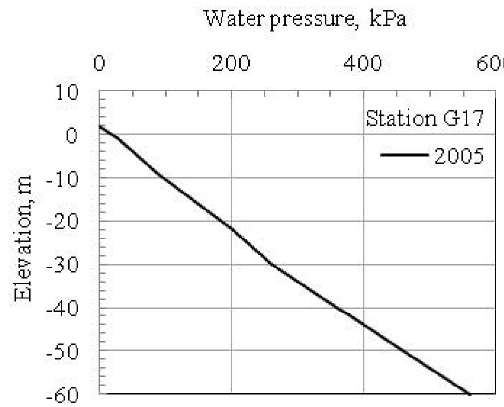

Figure 9 Groundwater pressures on the outer face of diaphragm wall

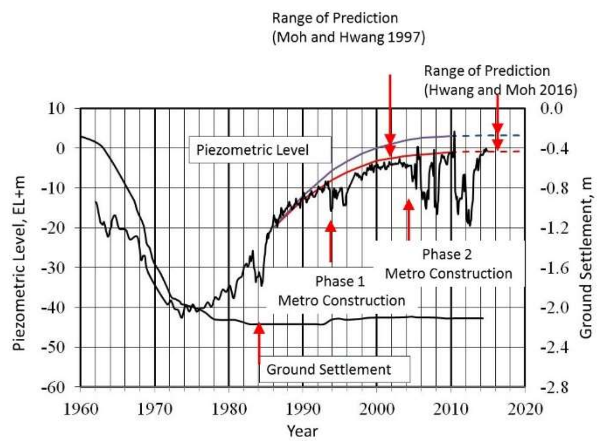

Figure 10 Piezometric level in the Jingmei Formation and ground settlement at Beimen Class 1 Reference Point near Beimen Station

The properties of the sublayers in the Songshan Formation have been well discussed in literatures (Moh and Ou 1979; MAA1987). An advanced study was conducted by Geotechnical Engineering Specialty Consultant engaged by the Department of Rapid Transit Systems of Taipei City Government in the very early stage of the metro construction as a designated task to study the characteristics of Taipei clays (Chin et al., 1994, 2006; Chin and Liu 1997; and Hu et al., 1996). It was conducted in collaboration with a research team from Massachusetts Institute of Technology (MIT). Hwang et al. (2013) summarized the results of the study and suggested Figure 11 be adopted for the clays in the T2, TK2 and K1 Zones for practical applications. An undrained shearing strength, i.e., Su, of 55 kPa can be obtained from Figure 11 for clays above a depth of 15m. The strengths of soils below this depth can be expressed as follow:

$$Su = 70 + 5.14\,(D-15) \qquad (1)$$

where D is the depth and Su is the shearing strength in kPa. This figure is meant for the ground conditions in 1990 when the Designated Task was conducted and, as mentioned in Hwang et al. (2013), is expected to be valid subsequent to 1990.

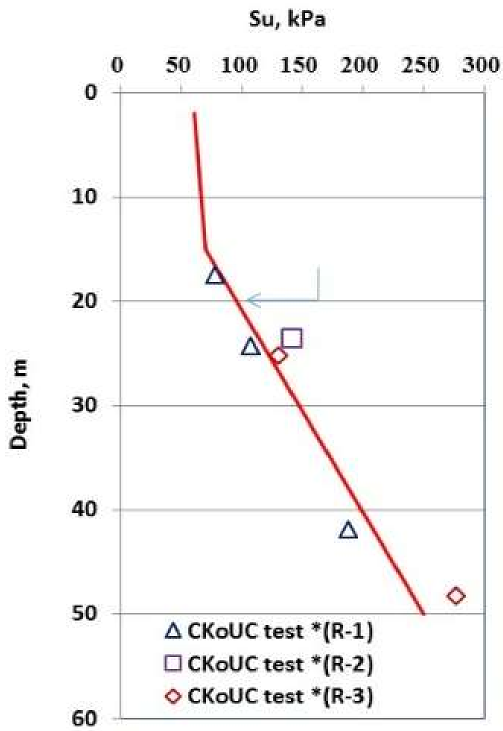

Figure 11 Estimated undrained shear strengths of clays in T2, TK2 and K1 Zones (Hwang et al. 2013)

The soil properties adopted in the finite element analyses are given in Table 2. The effective shear strength parameters, i.e., the c' and Φ' values, for sandy strata follow those adopted by the Detailed Design Consultant for Songjiang Nanjing Station (MAA 2005). For the clayey layers, the Su values for the clay layers of the Songshan Formation are assessed by Eq. 1, and c' = Su and Φ' = 0° are assumed in the analyses.

Soils were modelled by 15-node elements. The linear elastic-perfectly plastic Mohr-Coulomb Model was adopted to simulate the stress-strain behavior of soils. Although more advanced models, rather than the Mohr-Coulomb Model, are often suggested to be used to simulate the nonlinear behavior of soils in academic studies, the degree of sophistication of the Mohr-Coulomb Model is nevertheless compatible with the great uncertainty associated with the soil strengths and the diversity in the observed performance of the soil-structural interaction systems. For this reason, the Mohr-Coulomb Model remains to be the most popular model adopted in practice.

The Young's moduli, E', were correlated to soil strengths by using the following empirical relationships:

$$E' = 300\ Su \quad \text{(for clayey soils)} \tag{2}$$

$$E' = 2\ N \quad \text{(in MPa for sandy soils)} \tag{3}$$

in which Su = undrained shearing strength, and N = blow counts in standard penetration tests. The E'/Su ratios adopted in Equations 2 and 3 are in fact determined by back-analyses, which is in reality a curve-fitting process by trial and error, conducted under this study. That E'/Su ratio of 300 for clay is significantly less than the empirical ratio of 500 adopted in earlier studies such as those reported by Hwang et al. (2012), Hsiung & Hwang (2009b) and Hwang et al. (2016). The larger E'/Su ratio of 500 is mainly associated with the lower Su values adopted in those earlier studies. It should therefore be noted that, Equation 2 should only be adopted together with the shearing strengths obtained from Figure 11. Because the shearing strengths of clays adopted by the designers, as often is the case in design calculations in most projects, are much lower due to inadequate sampling and testing, larger correlation coefficients should be adopted accordingly. The empirical E'/N ratio of 2 (in MPa) for sand given in Eq. 3 is within the range of 2 to 3 adopted in those earlier studies.

It is noted that the calibrated E' values for soils are obtained by matching the calculated wall deflection profiles with those observed in the final excavation stages. As the soil moduli are nonlinear in nature, the use of a constant E' value in the Mohr-coulomb model tends to over-estimate the deflections in the early stages of excavation. As shown in Figures 14 to 16, the calculated wall deflections in the early stages are larger than those observed by, as much as, 30 %. Such moderate percentage of over-estimation in wall deflections in early stages would however be insignificant in excavation projects as the maximum deflection in the final stage would be of the interest.

4. CASES ANALYSED

To investigate the effects of preloads of struts on the performance of retaining structures, four cases were analysed as depicted in Table 3. They are:

Case 1: Full design preloads were applied at all levels
Case 2: The preload at the first level is reduced by 50%
Case 3: No preload at any level
Case 4: The preloads recorded by monitoring in Section D were applied
In addition, analyses were performed for
Case 5: Walls of 0.8m in thickness with the preloads recorded by monitoring
The results obtained in individual cases are discussed as follows.

Table 2 Soil properties and soil parameters adopted in the PLAXIS analyses (MAA 2005; Hwang et al. 2013)

Depth (m)	Soil Type	γ_t (kN/m^3)	N (blows)	S_u (kPa)	c' (kPa)	Φ' (deg)	E' (MN/m^2)	Poisson's Ratio, υ'
0-3	CL	18.8	4	55			16.5	0.35
3-13	SM	19.2	8	-	0	32	16.0	0.30
13-26	CL	18.6	6	95			28.5	0.35
26-34	SM	19.4	13	-	0	32	26.0	0.30
34-40	CL	18.9	13	180			54.0	0.35
40-44	SM	19.7	21	-	0	32	42.0	0.30
44-51	CL	19.9	20	240			72.0	0.35
51-64	SM	19.9	25		0	32	50	0.35

Table 3 Preloads applied in PLAXIS analyses

Strut Level	Preloads (tons)			
	Case 1	Case 2	Case 3	Case 4 (BM)
1	71.2	35.6	0	68
2	135.6	135.6	0	136
3	197.6	197.6	0	188
4	171.2	171.2	0	156
5	183.2	183.2	0	170
6	215.2	215.2	0	200

The wall movements obtained from the finite element analyses for the four cases of walls of 1m in thickness are compared with the inclinometer readings obtained by Inclinometer SID-3 which recorded the largest wall deflection in Table 4. It should be noted that the readings obtained by inclinometers have been corrected, as discussed in Section 2, by assuming that the connection to the wall at the first strut level were fixed once the struts were preloaded, and the movements of the tips of inclinometers were computed accordingly. This is why the same movement of 5.26mm is shown for SID-3CA for the strut at the first level after preloading and at the final excavation, giving a net increment of 0mm.

Table 4 Summary of Wall Movements

Item	**Movements, mm**				
	SID-3CA	PLAXIS Analyses			
		Case 1	Case 2	Case 3	Case 4
Maximum	36.3	35.9	36.0	41.6	38.6
1st strut level after preloading	5.26	-3.52	2.15	7.59	-3.02
1st strut level at final excavation	5.26	-5.03	0.86	7.66	-4.54
1st strut level net increment	0	-1.51	-1.29	0.07	-1.52
Toe of Diaphragm wall	14.6	16.7	16.7	16.5	16.6
Tip of Inclinometer	8.1	8.5	8.6	8.4	7.9

The net increments of wall movements subsequent to preloading at the first strut level obtained by the finite element analyses, varying from -1.52mm (outward) to 0.07mm (inward), are compatible with what was discussed in Section 2.2. The excellent agreement between the computed movements at the tips with that obtained at the tip of SID-3 fully justifies the corrections made to the inclinometer readings.

4.1 Case 1: Analyses with full design preloads at all levels

The profiles obtained for Case 1, i.e., the case with full design preloads applied to struts at all levels, are compared with the readings, duly corrected, obtained by Inclinometer SID-3 in Figure 12. As can be noted, the maximum wall deflections obtained at various stages of excavation are, in general, in a fair agreement with the inclinometer readings.

The analyses, however, fail to show the drastic difference between the movement at the toe of the wall and the soil immediately below the toe due to the limitation of the finite element scheme. A special type of element is required for these differences to be shown. This, nevertheless, is beyond the scope of this paper.

4.2 Case 2: Analyses with the preload at the first level reduced by 50%

Large outward movements, refer to Figure 5(b), were obtained from the finite element analysed as the struts at the 1st level were preloaded. This was presumably due to the use of Mohr-Coulomb Model which under-estimates the soil moduli in the early stages of excavation. On the other hand, it might be a result of heavy preload applied to the struts at the 1st level.

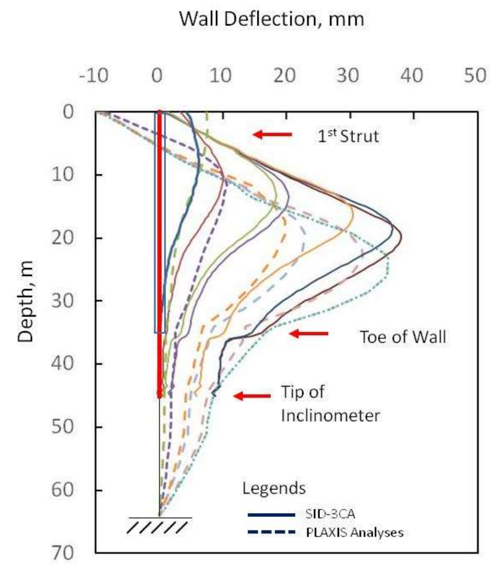

Figure 12 Comparison of computed wall deflections with inclinometer readings – Case 1 with full design preloads

The strut at the first level at the location of SID-2 was preloaded to 72.5 tons (the average of 2 readings) on 1 January 2009 and, as shown in Figure 5(a) the inclinometer readings taken on 5 January indicated that the wall at this level had hardly moved as compared to those taken on 27 December 2008.

The preloads are line loads applied to all the struts at the same level simultaneously in two-dimensional numerical analyses; while in reality, struts were preloaded individually, one by one. Each time a strut was preloaded, the load was essentially a point load resisted by the entire wall and the wall movement would be smaller than what would have been if all the struts at the same level were preloaded simultaneously. Wall movements due to subsequent preloading of neighbouring struts could even be reduced because the loads in the struts which had already been preloaded were not sustaining. Figure 13 shows the fact that the strut load dropped gradually to nearly a half, from the average 72.5 tons to the residual34.0 tons in nine days subsequent to preloading due to applying preloads to the neighbouring struts.

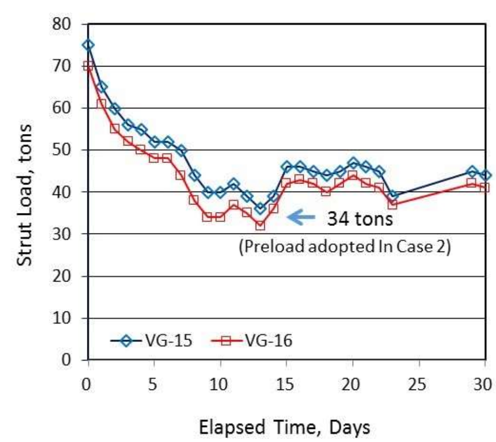

Figure 13 Strut loads at the first level subsequent to preloading, Section B

This residual load can be considered as an effective load in the struts at the first level and should be the load to be adopted in the finite element analyses. The residual load in Stage 1 excavation rose again from the average 34.0 tons to 44.0 tons on day 14whenStage 2 excavation commenced.

Analyses were performed for the case, i.e., Case 2, in which the preload in the strut at the first level reduced to a half. As depicted in Figure 14, the outward movements at the top of the wall were much reduced. On the other hand, the maximum wall deflection is nearly unaffected as can be noted by comparing Figure 14 with Figure 12.

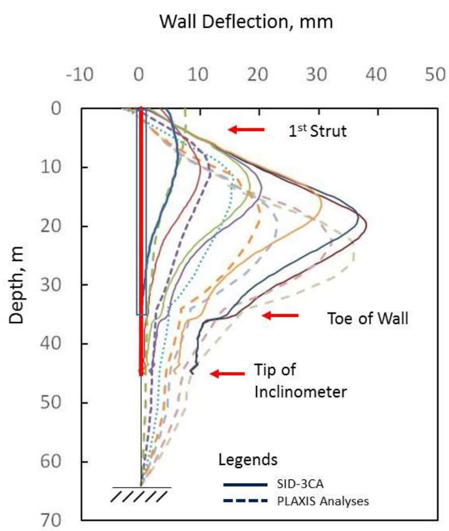

Figure 14 Comparison of results of numerical analyses with inclinometer readings - Case 2 with the preload in Level 1 struts reduced by 50%

4.3 Case 3: Analyses with no preloads at all levels

For academic interest, analyses have also been carried out for the case with preloads in all stages of excavation omitted and the results are shown in Figure 15. As can be noted that the deflection profiles computed for the upper portion of the wall agree better with those obtained based on the inclinometer readings in comparison with those obtained in the previous cases, however, the maximum wall deflection increases from 36mm in Case 2 to 41.6mm at the end of the final excavation.

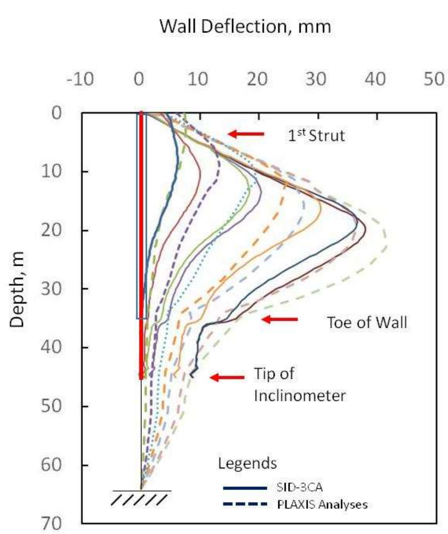

Figure 15 Comparison of results of numerical analyses with inclinometer readings - Case 3 without preloads at all levels

4.4 Case 4: Analyses with preloads recorded by monitoring

For back analyses, such as the case of interest, the real strut loads are available and can be used in the analyses directly. As depicted in Figure 13, the loads in struts dropped subsequent to preloading in a few days. The lowest strut loads developed during the period between the preloading and the commencement of excavation for the next stage are considered as the residual loads, which shall be the loads to be adopted in the analyses. The computed wall deflections with the measured residual strut loads at all the levels in Section D are compared with the inclinometer readings in Figure 16. The case shown is considered as the "benchmark case" for the parametric studies presented hereinafter.

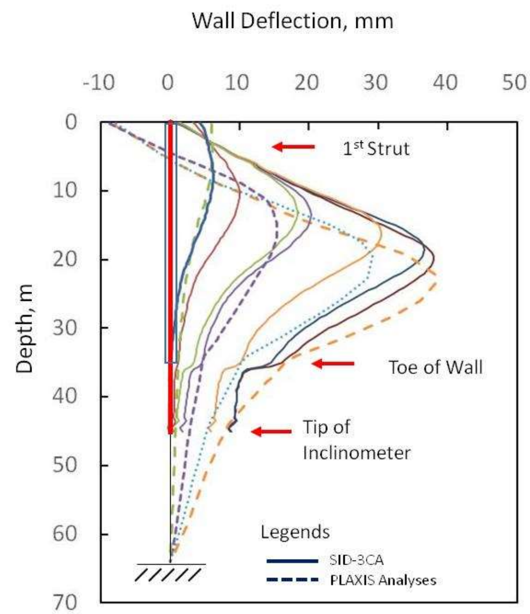

Figure 16 Comparison of results of numerical analyses with inclinometer readings - Case 4 with preloads obtained from monitoring in Section D

4.5 Case 5: Analyses for 0.8m walls with preloads

It is generally believed that the preloading of struts increases the rigidity of the retaining system and, hence, reduces wall deflections. Moh and Hwang (2017) shows that preloading of struts has similar effects on the performance of retaining system as thickening the walls. To illustrate this point, analyses were performed for walls of 0.8m in thickness and the results are to be discussed in Section 5.3.

5. WALL DEFLECTION PATHS AND REFERENCE ENVELOPE

The large differences between the wall deflection profiles for different inclinometers shown in Figure 7 fully demonstrate the ambiguity faced in back analyses. Different results will be obtained if a different set of inclinometer readings is selected to compare with the results of numerical analyses.

To establish a consistent methodology for back analyses, Moh and Hwang (2005) suggested to plot wall deflections versus depth of excavation, in a log-log scale, and designate such plots as "wall deflection paths". It was further suggested to take the upper envelope of the wall deflection paths obtained at a specific site, or for a specific set of parameters, as "reference envelope" to compare with the results of conventional two dimensional numerical analyses which are usually conducted for excavations in green field without structures or utilities in the vicinity. This suggestion was based on the belief that wall deflections are likely to be reduced by many factors and the upper envelope of wall deflection paths will be closer to what will be obtained in numerical analyses carried out for excavations in green field.

5.1 Reference envelope of the site

The wall deflection paths obtained by the 4 inclinometers are shown in Figure 17. As discussed in Section 4, wall deflections, particularly at shallow depths, are likely to be affected by how the preloads are applied at the site. Excavation and preloading of struts at excavation sites are never carried out in the ways specified in designs. They are carried out in a rather unpredictable sequence as there are site constraints and, besides, project progress always prevail. Furthermore, coordination among subcontractors is often difficult, and as such, over-excavation occurs rather frequently and delay in strutting is quite common. For these reasons, the data for shallow excavation are erratic and should be ignored; and only the data for excavations exceeding 10m in depth should be considered in establishing reference envelopes as suggested in previous studies (Moh and Hwang 2005; Hwang et al. 2006, 2007a, 2007b). After all, deep excavations are the ones of primary concern.

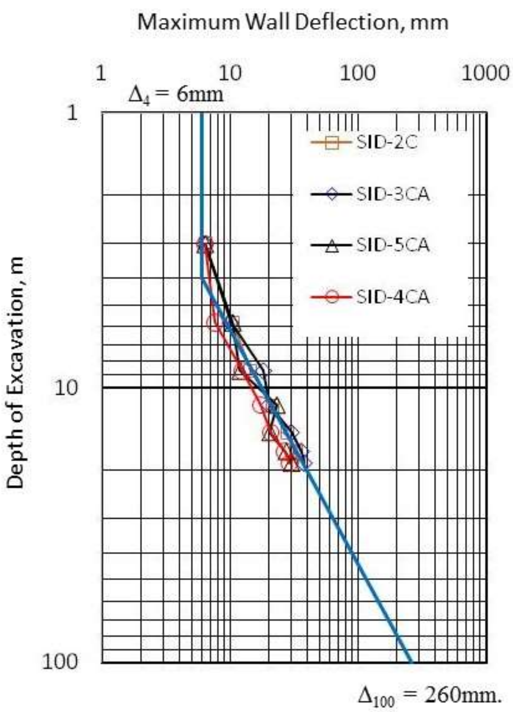

Figure 17 Effects of preload on wall deflection paths and reference envelopes for the crossover

For convenience, reference envelopes are defined by the wall deflections for a depth of excavation of 4m, i.e., Δ_4 and the wall deflection projected to a depth of excavation of 100m, i.e., Δ_{100}. The extension of the envelope to a depth of 100m is merely for the convenience in expressing the relationship between wall deflection and depth of excavation and does not imply the validity of the said relationship below the final depth of excavation.

Accordingly, the reference envelope for the case of interest can be expressed as Δ_4 = 6mm and Δ_{100}= 260mm as depicted in Figure 17. The reference envelope obtained at a specific site can be used as the baseline for parametric studies. The influences of various parameters on wall deflections can be quantified by comparing the Δ_4 and Δ_{100} values obtained with those of the reference envelope.

5.2 Wall deflection paths obtained by finite element analyses

The wall deflection path obtained by PLAXIS analyses for the benchmark case, i.e., Case 4, in which the preloads applied in the analyses equal to the residual preloads observed, is given in Figure 18 and can be defined by Δ_4 = 6mm and Δ_{100} = 260mm. This wall deflection path is the same as the reference envelope given in Figure 17. This validates the use of the combination of the soil properties given in Table 2 together with Equations 1 to 3.

As can be noted from Figure 18 and as confirmed in previous studies (Hwang et al. 2012; Hwang et el. 2016), wall deflection paths for deep excavations are normally linear in the range of depths of excavation of 10m to 20m. Accordingly, the relationship between Δ_4 , Δ_{100} and depth of excavation can be expressed as follows:

$$\frac{\log(\Delta_D)-\log(\Delta_4)}{\log(D)-\log(4)} = \frac{\log(\Delta_{100})-\mathrm{Log}(\Delta_4)}{\log(100)-\mathrm{lo}\ \ (4)} \tag{4}$$

in which D = depth of excavation, Δ_D= maximum wall deflection for a depth of excavation of D.

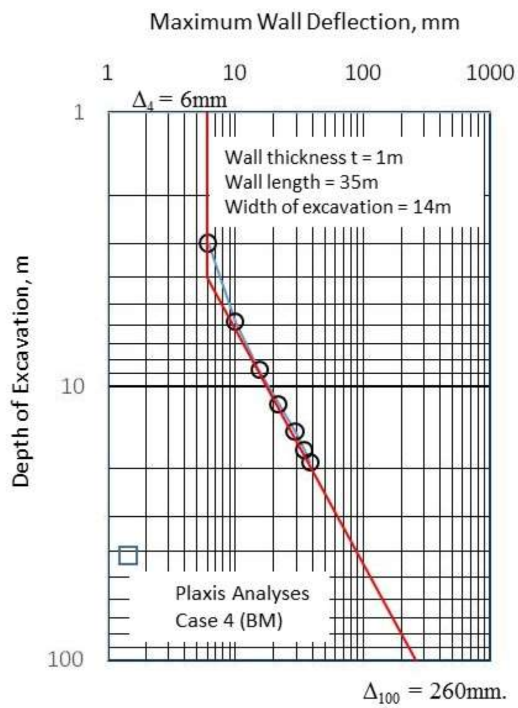

Figure 18 Wall deflection path obtained by PLAXIS analyses for Case 4: the benchmark case

It has been further confirmed in previous studies that wall deflection paths at a specific site tend to give the same Δ_4 values, regardless of preloads. This is logic as the Δ_4 values, in a sense, correspond to the maximum wall deflections at the end of the first stage of excavation and before the preloading of the struts at the first level; and are therefore unaffected by the preloading of struts. It has also been confirmed by the previous finite element analyses that the Δ_4 values are unaffected by the lengths and the thicknesses of walls as well (Hwang et al. 2012). Further discussion however is beyond the scope of this paper.

For Δ_4 = 6mm and D = 19.4m, the Δ_{100} values are related to the maximum wall deflections, i.e., Δ_D, as follows:

$$\log(\Delta_{100}) = 2.0386\log(\Delta_D) - 0.8083 \tag{5}$$

This equation can be used to compute the maximum wall deflections in different stages of excavation at this site based on the Δ_{100} value obtained for other cases.

5.3 Effects of preloading of struts on wall deflections

The Δ_4 and Δ_{100} values of the wall deflection paths obtained for all the 5 cases are compared in Table 5. Because the differences in wall deflections are too small to be visualized in graphs, the Δ_{100} values given in the table were computed based on the maximum wall deflections at the final excavation using Equation 5. As can be noted from this table, the omission of preloads in Case 3 does increase the maximum wall deflection by 3mm as compared with Case 4. It can also be noted, the performance of 0.8m walls with preloads, i.e., Case 5, is comparable with that of Case 3 in which preloads are omitted. In other words, the rigidity of the retaining system with 0.8m thick walls with preloads is the same as that of the retaining system with 1m thick walls without preloads.

It should be emphasized, however, in addition to the benefit shown, preloading of struts also has the benefit of closing up the gaps between the walls and the walings and between the walings and the struts; hence further reduce wall deflections.

Table 5 Effects of preloads on wall deflections

Item	Movements, mm					
	Observed	PLAXIS Analyses				
		Wall Thickness t = 1m				t = 0.8m
	SID-3CA	Case 1	Case 2	Case 3	Case 4	Case 5
Δ_D D = 19.4m	36.3	35.9	36.0	41.6	38.6	41.3
Δ_4	6	6	6	6	6	6
Δ_{100}	260	230	230	310	260	310

6. STRUT LOADS AS OBSERVED AND AS COMPUTED

The peak loads observed in the three cross-sections shown in Figure 2 and those obtained from the finite element analyses are summarized in Table 6. As can be noted, the loads in the 3 cross-sections differ considerably despite the fact that the subsoils are fairly uniform. The schemes of excavation was supposed to be the same in these cross-sections except that the excavation in Section B was slightly shallower and the level of the last strut was adjusted accordingly. The differences between the highs and the lows varied by, upto, 50.6% (Level 6). As mentioned in Section 5.1, excavation and preloading of struts are never carried out in the ways specified in designs. The differences in strut loads were presumably caused by the variations from the specified excavation procedures, particular, over-excavation.

6.1 Changes in strut loads as excavation proceeded

Figures 19 to 24 compare the loads in struts computed in the finite element analyses for the benchmark case, i.e., Case 4, with those observed in Design Section D during excavation. The agreement between the two sets of data is amazingly good for Level 1 and Level 2 struts and is reasonably good for Level 3 strut. For Level 4, 5 and 6 struts, the loads in the struts subsequent to preloading did not increase as much as expected but dropped as expected subsequently.

The peak loads in these struts were over-estimated by 30% to 50% which are well in the range of uncertainties for geotechnical engineering and are comparable with the differences among the observed strut loads in the 3 cross-sections. It should be noted, however, the design loads are not necessarily governed by the loads induced in the excavation stage. As depicted in Figure 6, the strut loads in the subsequent construction may exceed what is experienced in the excavation stage as side walls and floor slabs are cast and the struts at lower levels were removed.

Table 6 Comparison of the observed peak strut loads with the computed peak strut loads

Strut Level	Observed peak strut load, tons				Computed peak strut load, tons				
	Section B (SID-2) D = 18.9m	Section C (SID-3) D = 19.4m	Section D (SID-4/5) D = 19.4m	Average	Case 1 t = 1m D = 19.4m	Case 2 t = 1m D = 19.4m	Case 3 t = 1m D = 19.4m	Case 4 t = 1m 19.4m	Case 5 t = 0.8m D = 19.4m
1	94	112	94	100	112	86	61	108	102
2	166	240	187	198	219	226	113	223	218
3	321	343	343	336	303	307	148	303	296
4	230	230	222	227	255	257	191	292	283
5	238	158	217	204	322	324	191	306	308
6	190	216	241	228 (1)	350	350	175	345	353
Note (1): The average of the preloads in Section C and D only because the excavation for Section B was shallower									

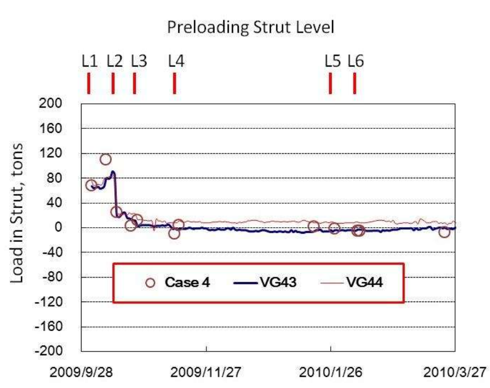

Figure 19 Loads in Level 1 struts, Section D

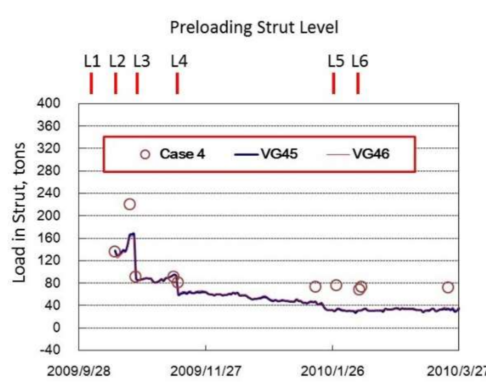

Figure 20 Loads in Level 2 struts, Section D

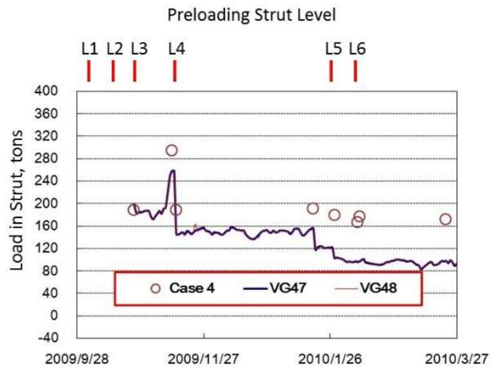

Figure 21 Loads in Level 3 struts, Section D

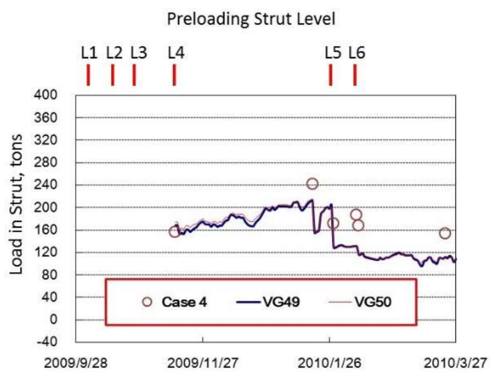

Figure 22 Loads in Level 4 struts, Section D

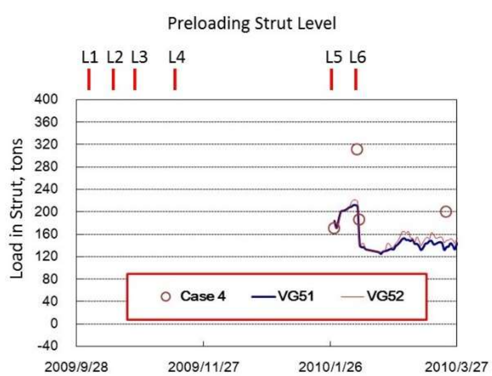

Figure 23 Loads in Level 5 struts, Section D

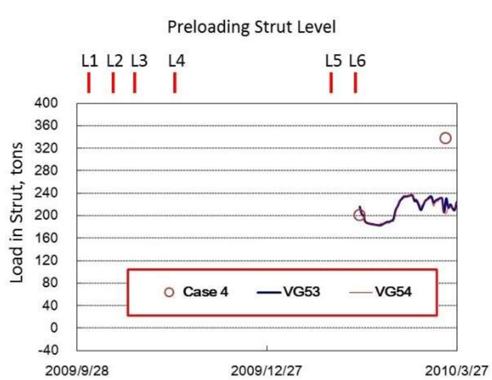

Figure 24 Loads in Level 6 struts, Section D

6.2 Computed strut loads as compared with the average loads

The averages of the observed peak loads are supposed to be compared with the computed peak loads in the benchmark case, i.e., Case 4. For the upper 3 levels, the differences are within 15%. For the lower 3 levels, the computed loads exceed the averages of the observed loads by, upto, 50%.

6.3 Effects of preloads on strut loads

Preloading of struts does increase strut loads by 53% (Level 4) upto 104% (Level 3) as can be noted by comparing the results obtained for Case 4 with those obtained for Case 3 in which preloads were omitted.

6.4 Effects of wall thickness on strut loads

The differences in strut loads are affected by wall thickness only slightly as can be noted by comparing the results obtained for Case 4, with those obtained for Case 5 in which the wall thickness was reduced to 800mm.

7. CONCLUSIONS

Numerical analyses using the linear elastic-perfectly plastic Mohr-Coulomb stress-strain model and the software PLAXIS have been conducted on a case history of an excavation. The excavation of 19.4 m in depth in soft ground was supported by diaphragm wall of 35 m in length and 1 m in thickness. The computed results are compared with the observed wall deflections and measured strut loads. Based on results of studies the following conclusions could be drawn:

(1) For inclinometers with their toes not embedded in competent strata, the readings should be corrected to account for potential toe movements of the inclinometers. The corrections can be conducted by assuming the wall deflections at the first strut level would not move after preloading.
(2) The strain gauge readings shows that the preloads applied to the struts do not sustain and drop significantly after subsequent preloading. In the numerical analyses, the preload should be properly reduced to be consistent with the real situations.
(3) Back-analyses on the excavation case history adopting the actual observed preloads of struts show that the computed peak loads agree with those observed for the upper 3 levels of struts. The lower 3 levels of struts are however over-estimated by as much as 50 %. Application of strut preloading could achieve the effect of larger wall rigidity without preloading. For example, the maximum wall deflection of 0.8m thick walls with preloads is the same as that of 1m thick walls without preloads.
(4) The wall deflection paths and the reference envelopes are useful tool for assessing the wall performance and for selecting the representative wall deflection profiles for back-analysing excavation case histories.
(5) Based on the benchmark case calibrated with the observed wall deflection profiles in the final excavation stages, the back-analysed Young's moduli for clay layers could be represented by the empirical relationship of 300 times undrained shearing strength that obtained by compression testing on undisturbed Class B soil specimens.
(6) The back-analysed Young's moduli for sand layers is 2N (in MPa), where N is the blow-counts for the Standard Penetration Tests.

As the soil moduli are nonlinear in nature, the use of constant E' values in the Mohr-coulomb model tends to over-estimate the wall deflections in the early stages of excavation. It should be noted that the empirical Young's moduli for clay and sand layers are coherent to the method of testing and to the numerical model adopted in the back-analysing procedures. Application of these stiffness parameters in design of excavation works should be verified with field monitoring.

If better estimation in wall deflections in the early stages of excavation would be required, various E' values may be adopted for various excavation stages instead of a constant stiffness value. The empirical E'/Su and E'/N ratios may be assessed by matching the deflection profiles in the relevant stages of excavation. It should be noted that the wall deflections occurring in early stages and in shallow depths of excavation would likely be affected by various conditions such as struts preloading, presence of nearby basements, diaphragm walls, pile foundations, vehicular underpasses, tunnels and stormwater culverts, surcharge loads due to shallow foundations and by nearby ground treatment activities.

The case history reported in this paper shows that with proper interpretation the inclinometers and strain gauges are reliable and valuable instruments for monitoring the performance of deep excavation projects.

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