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FEM analyses of deep excavations and earth retaining structures in soft clay

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ABSTRACT: In recent years, an increasing amount of excavation work has come close to adjacent structures. To reduce the effects of excavations on adjacent structures, it is becoming increasingly important to predict the displacement of earth retaining walls and the retained soil movement. 2D finite element analyses were performed to simulate the behavior of an earth retaining wall. In this FEM simulation, a Mohr-Coulomb failure model was adopted to consider soil stiffness dependence on shear strain. The influence of the initial Young's modulus and the soil stiffness dependence on confining pressure were examined. FEM calculation results were compared with the monitoring results of excavations in soft clay. Excavation depth was 17.0 m and the retaining wall was soil-cement with H-section steel. Excavation was done using a total of four levels of RC slab struts and the top down method. A good agreement was found with displacements of the retaining wall between the calculation results and observation data using the deformation modulus at small strain E_0 estimated from the results of PS logging. The analysis was effective in predicting the vertical displacement of the retained soil.

1 INTRODUCTION

Recently, excavations that are adjacent to railways and the roads have been increasing in cities. In the adjacent excavations, it is important to control the influence on the adjacent structures as much as possible. It is becoming increasingly important to estimate not only the lateral displacement of earth retaining walls but also the movement of the ground behind the retaining walls.

There are two basic approaches to predict the ground movements caused by excavation. Peck (1969), Aoki et al. (2000) and Moorman (2004) proposed prediction of settlement behind retaining walls based on measurement results. Roscoe (1970) and Aoki et al. (1990) proposed prediction forms of settlement assuming a slip surface behind the retaining wall. The method based on actual measurement results is a simple empirical method, and effective to understand the amount of settlement qualitatively. However, it is difficult to evaluate the influence on the adjacent structure accurately by this method. Moreover, if structures exist behind a retaining wall, it is very difficult to define a slip surface in the soil, so the method of defining a slip surface in soil is not suitable for displacement forecasts of the adjacent structure.

In recent years, FEM analyses are used to estimate the displacement of the adjacent structure. In the FEM analysis, there are two methods for analyzing. One is a step by step analysis, and another is applying displacements as a boundary

condition. In Japan, to predict the behavior of adjacent structures, the FEM analysis applying displacement is generally used (AIJ 2006). With the FEM analysis applying displacement, the prediction accuracy is high because it uses lateral displacement of the wall calculated by beam-spring analysis. However, the influence of the adjacent structure cannot be considered when only the wall displacement is calculated (AIJ 2006).

In this paper, a step by step FEM analysis is performed to predict the wall displacement and ground movement due to excavation in soft ground. In the analysis, the elasto-plastic Mohr-Coulomb failure model is used to examine the influence of the constitutive model. Moreover, to examine the influence of the deformation modulus used for the analysis, the deformation modulus is set from V_s derived from PS logging or N value and cohesion c .

2 OVERVIEW OF EXCAVATIONS AND GROUND CONDITIONS

2.1 Excavations

Figures 1 and 2 shows a plan and section of the excavation and soil profile of the project. The building had multiple basements, and was eight stories above ground. The project excavation covered an area of 135 m × 50 m, at a depth of GL – 17.0 m. There were office buildings and shore protection next to the retaining wall, so the wall displacement caused by excavation had to be strictly controlled.

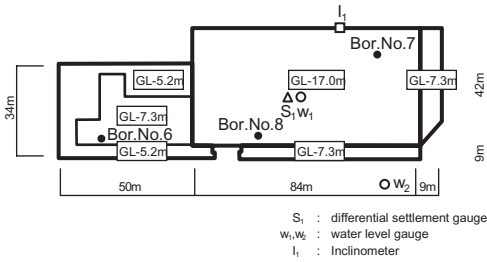


Figure 1. Plan of excavation.

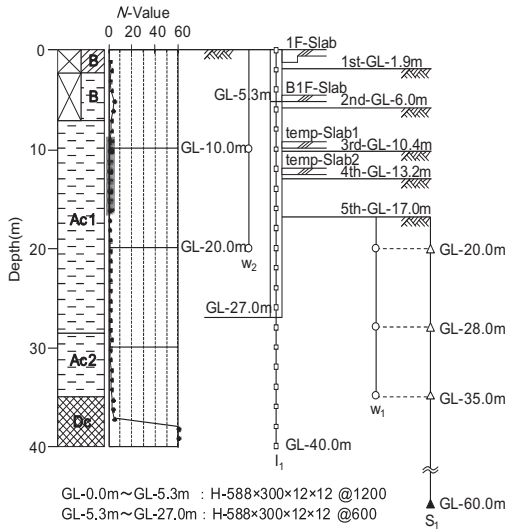


Figure 2. Section of excavation.

Excavation was done using a total of four levels of supporting struts, including two basement floors and two temporary RC slabs. The top down method was used to build these supports.

The retaining wall comprised soil cement up to 28 m, reinforced with H-section steel (H588 × 300) of 27 m length. The pitch of the H-section steel down to GL − 5 m was 1200 mm, because the lateral pressure that acted on the wall was lower close to ground level, and from GL − 5 m to GL − 27 m was 600 mm. Ground water was pumped from dewatering wells.

Table 1 shows the excavation monitoring equipment and instrument locations are shown Figures 1 and 2. The lateral displacement of the retaining wall, the water pressure that acts on the wall, the vertical displacement of the ground and water pressure in the ground were measured. Because the retaining wall was not embedded in stiff ground, the retaining wall toe was assumed to move into the excavation (Tanaka, 1994). Therefore, the bottom

Table 1. Monitoring instruments.

Items	Equipment	Measurement depth
Lateral displacement of the wall	Inclinometer	GL − 0 m–GL − 40 m (@2.0 m)
Water pressure on the wall	Water-level gauge	GL − 10 m, −20 m
Vertical displacement of the ground	Differential settlement gauge	GL − 20 m, −28 m, −35 m
Water pressure in the ground	Water-level gauge	GL − 20 m, −28 m, −35 m

of the inclinometer was embedded in the mudstone by about 5 m. Because this project involved deep excavation into soft clay there was a risk of base heave. Therefore, vertical heave was measured in the center of the excavation. It was assumed at GL − 60 m in the mudstone there was zero vertical displacement. The lateral displacement of, and the water pressure on the retaining wall were measured from the completion of retaining wall construction to the completion of the excavation. The vertical displacement and water pressure in the ground were measured from the second excavation to the end of the excavation work.

2.2 Ground condition

The project location was at the mouth of the Katabira River and the east side of the East Japan Railway Company's Yokohama Station. The site was reclaimed prior to the 1930's. The Soil profile at the site is shown in Figure 3. To the south of the site, alluvial sand exists between the soft clay and mudstone. Soft clay layers extend from the ground surface to a depth of about 35 m. The thickness of the soft clay layer is almost constant across the site.

Soil properties are shown in Table 2 and Figure 4. Natural water content w_n of the soft clay ranged 70% to 100%, and was close to the liquid limit. The values of on drained shear strength were ranged 40 kpa to 70 kpa. Shear wave velocities were in the range to 100 m/s to 120 m/s in the soft clay layer and 460 m/s to 610 m/s in the mudstone. The groundwater level was 1.6 m below ground level and was GL − 4.1 m in the mudstone layer.

3 MEASURED RESULTS

3.1 Lateral displacement of retaining wall

Figure 5 shows the lateral displacement of the retaining wall. Lateral displacement gradually increased

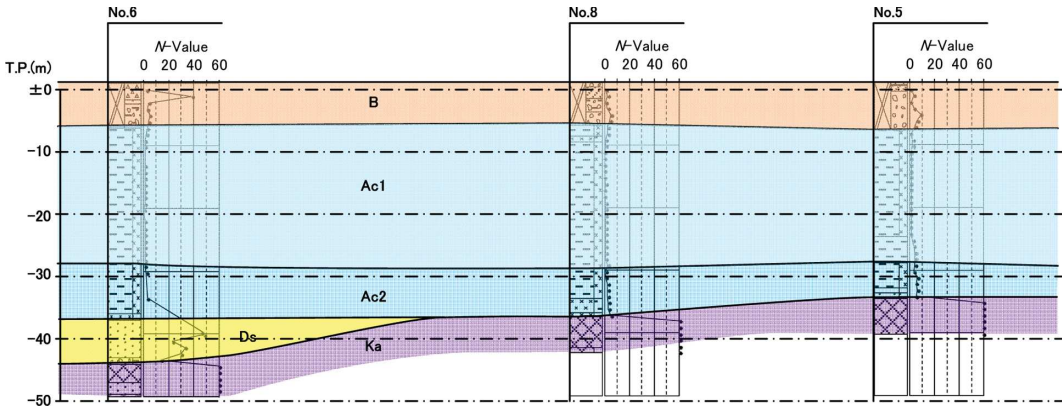


Figure 3. Soil profile.

Table 2. Soil properties.

Soil layer	w_n (%)	γ (g/m ³)	c (kN/m ²)	V_s (m/sec)
Fill (B)	–	1.70	–	160
Clay (Ac1)	70	1.57	45	100
Clay (Ac2)	90	1.45	65	120
Sand (Ds)	–	1.82	–	460
Clay (Ka)	–	1.90	–	610

during excavation, reaching its maximum near the base of the excavation. The lateral displacement of the earth retaining wall was up to 90 mm at the final excavation stage. The lateral displacement of the toe of the retaining wall (GL – 26 m) was less than 50 mm.

3.2 Water pressure

Figure 6(a) shows the water pressure that acted on the retaining wall and Figure 6(b) shows the water pressures below the base of the excavation. The water pressure that acts on the back of the retaining wall hardly changed over the excavation period. On the other hand, water pressure below the base of the excavation decreased as the excavation progressed. During excavation, the ground water was pumped from deep wells, so water pressures inside the excavation changed. Water pressures inside the excavation were not able to be measured because of equipment problems after the fourth excavation.

3.3 Vertical heave of excavation

Figure 7 shows the vertical displacement of the center of the excavation over time. Vertical displacement gradually increased during excavation,

and was up to 45 mm at the final excavation stage. Large displacement was measured near the excavation base at GL-20 m and GL-28 m in the final excavation stage. Vertical displacement increased rapidly in the initiation phase of the final excavation. However, it was judged that it was not heaving because the increase in displacement was gradual afterwards. The depth distribution of the vertical displacement at each excavation stage is shown in Figure 8. The vertical displacement in the excavation bottom was about 45 mm. However, the vertical displacement in the very stiff mudstone was about 2 mm.

4 OUTLINE OF FEM ANALYSIS

4.1 Mesh and analysis condition

The FEM analysis was carried out in the two dimensional plane strain condition and total stress analysis was adopted. The FEM mesh is shown in Figure 9. The width of the mesh was assumed to be 101 m, and the distance from the retaining wall to the mesh boundary was assumed to be 80 m, which was near 5 times in depth of excavation (17 m). The depth of the mesh was assumed to be GL – 60 m, equal to the depth of the base of the differential settlement gauge. The analysis steps are shown in Table 3. An initial stress was analyzed to create self weight and 11 steps were analyzed to the final excavation depth following the retaining wall, excavation struts installation procedure.

In this analysis, the coefficient of earth pressure at rest K_0 was assumed to be 1.0 in the initial stress analysis, based on measured results showing K_0 ranged 0.9 to 1.0 (Kanaya et al., 1973, Kuroyanagi et al., 1977) in soft clay. Moreover, the building self weight at each excavation stage was considered as

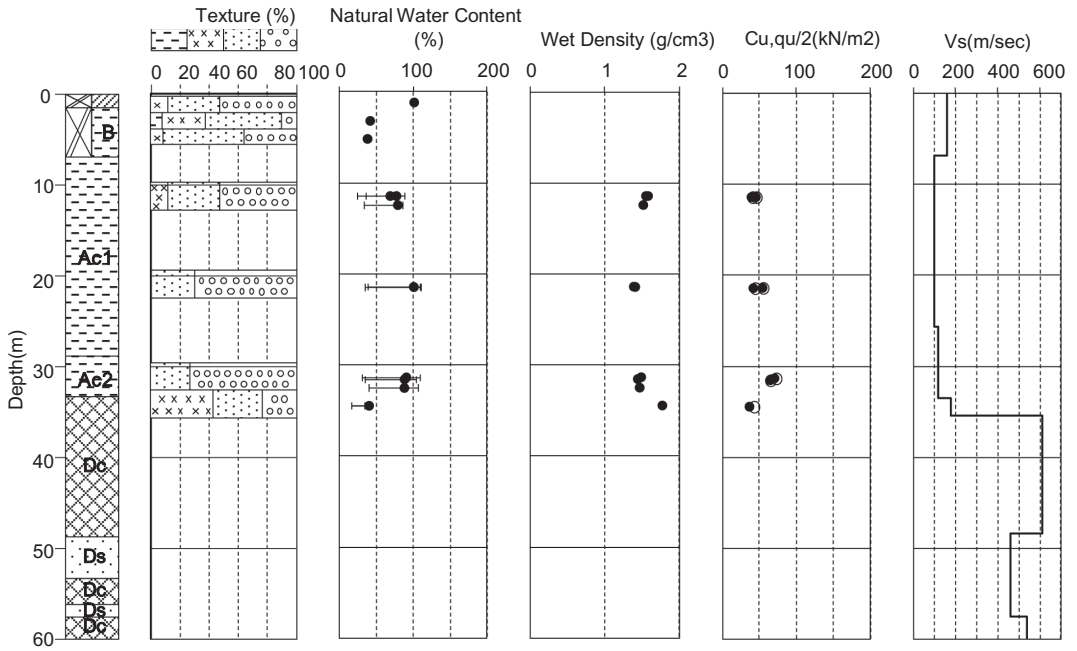


Figure 4. Soil properties.

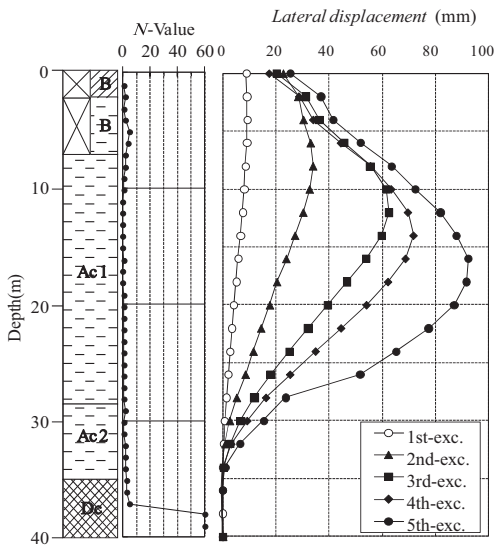


Figure 5. Lateral displacement of earth retaining wall.

a vertical load, because the top down method was adopted in construction (see Table 4). The building self weight was applied at the depth of 2/3 of the length of the bored piles (AIJ, 2000) as a uniform load.

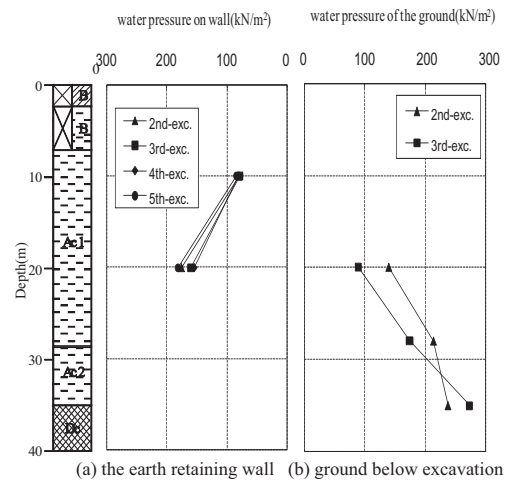


Figure 6. Water pressure.

4.2 Numerical model and soil parameters

In the excavation analysis, unloading caused by excavation and loading caused by the building self weight and the preload to the retaining wall are caused at the same time. In this analysis, the Mohr-Coulomb failure model (Hashiguchi, 2009, Shigeno, 2010) was adopted to predict the lateral deformation of the retaining wall and the vertical

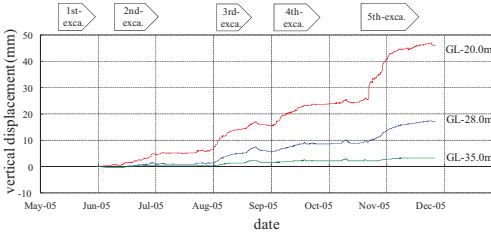


Figure 7. Relation between pate vertical displacement and time.

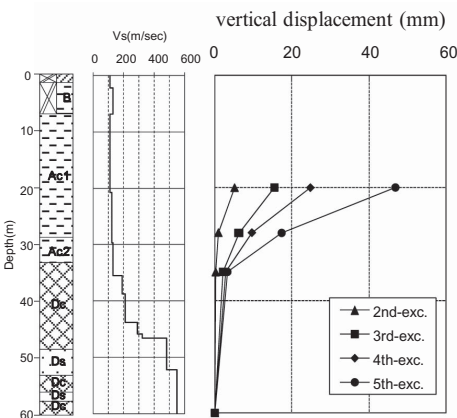


Figure 8. Depth distribution of vertical displacement.

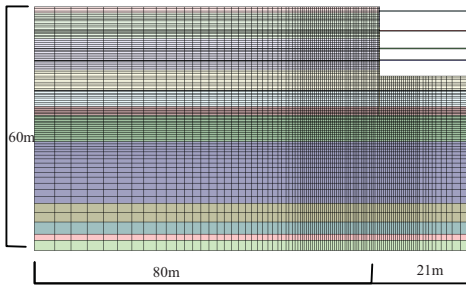


Figure 9. FEM analysis mesh.

displacement of the excavation base. The stress and strain relations in the Mohr-Coulomb model are shown in Equation 1 and 2 below:

$$\dot{\sigma} = E \dot{\varepsilon} - \Lambda E \frac{\partial f}{\partial \sigma} \quad \Lambda = \frac{\frac{\partial f}{\partial \sigma} E \dot{\varepsilon}}{UC \left\| \frac{\partial f}{\partial \sigma} \right\| + \frac{\partial f}{\partial \sigma} E \frac{\partial f}{\partial \sigma}} \quad (1)$$

Table 3. Analysis steps.

Step	Construction condition	Remarks
Step 1	Initial stress	$K_o = 1.0$
Step 2	Installation of the wall	
Step 3	1st excavation	GL – 1.9 m
Step 4	Installation of 1st layer struts	GL – 1.0 m
Step 5	2nd excavation	GL – 6.0 m
Step 6	Installation of 2nd layer struts	GL – 5.9 m
Step 7	3rd excavation	GL – 10.4 m
Step 8	Installation of 3rd layer struts	GL – 10.3 m
Step 9	4th excavation	GL – 13.2 m
Step 10	Installation of 4th layer struts	GL – 13.1 m
Step 11	5th excavation	GL – 17.0 m

Table 4. Building self weights.

Step	Excavation condition	Constructed floor	Weights (kN/m ²)
Step 3	1st excavation	–	0.0
Step 5	2nd excavation	1st floor	10.0
Step 7	3rd excavation	1st, B1 floors 2nd floor	27.0
Step 9	4th excavation	1st, B1, tem. 1 floors 2nd–6th floors	62.0
Step 11	5th excavation	1st, B1, temp.1, 2 floors 2nd–7th floors	74.0

$$U = m \cot \left(\frac{\pi \langle R - R_e \rangle}{2 \quad 1 - R_e} \right) \quad (2)$$

where C = cohesion; R = ratio to yielding surface of Tresca ($0 < R < 1$); R_e = ratio in elasticity limit; and m = Model parameter.

Moreover, the change in the soil stiffness as a function of confining pressure is given by Equation 3 below:

$$E = E_o \left(\frac{\sigma_m}{\sigma_{mo}} \right)^{0.5} \quad (3)$$

where E_o = Initial deformation modulus; σ_{mo} = Initial confining pressure; and σ_m = Confining pressure.

Soil stiffness and soil strength parameters used for the analysis are shown in Tables 5 and 6 respectively. The small strain deformation modulus E_o was derived from V_s from PS logging and used for the Mohr-Coulomb model. The deformation modulus E_{s0} was obtained from N -values

Table 5. Soil stiffness parameters.

Soil layer	Bottom depth (m)	<i>N</i> -Value	E_o (MN/m ²)	E_{50} (MN/m ²)
Fill	7.8	4	124.0	4.2
A-Clay1	24.5	1	45.7	8.8
A-Clay2	33.2	5	61.7	15.3
D-Clay1	48.4	99	2020.0	378.0
D-Sand1	53.0	141	1090.0	395.0
D-Clay2	56.0	102	1150.0	378.0
D-Sand1	57.5	144	1090.0	403.0
D-Clay3	60.0	97	1580.0	378.0

Table 6. Soil strength parameters.

Soil layer	Bottom depth (m)	γ (kN/m ³)	<i>c</i> (kN/m ²)	ν	<i>m</i>
Fill	7.8	17.0	20.0	0.4	2000
A-Clay1	24.5	14.0	42.0	0.4	50
A-Clay2	33.2	15.0	73.0	0.4	50
D-Clay1	48.4	19.0	1800.0	0.4	65
D-Sand1	53.0	19.5	300.0	0.3	90
D-Clay2	56.0	19.0	1800.0	0.4	65
D-Sand1	57.5	19.5	300.0	0.3	90
D-Clay3	60.0	19.0	1800.0	0.4	65

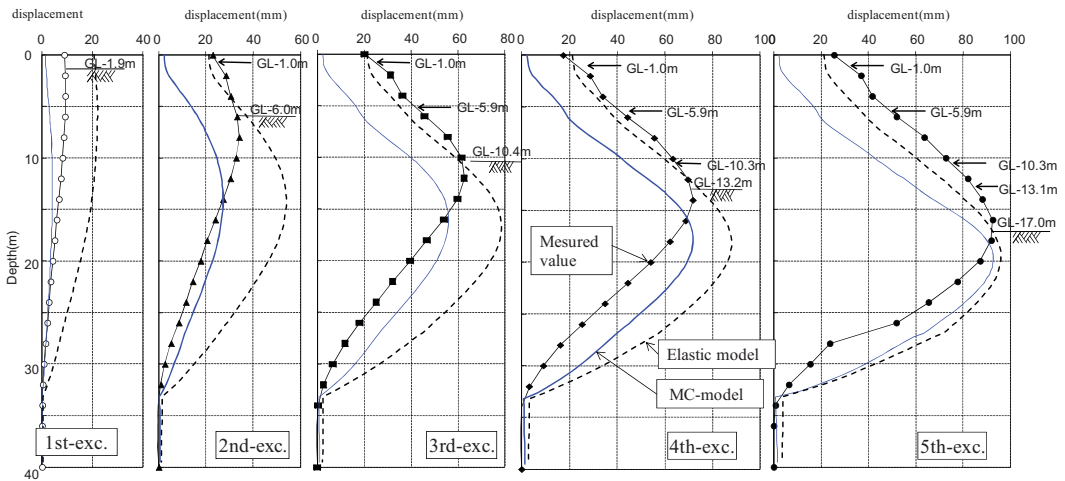


Figure 10. Comparison between analytical and measured value (lateral displacement).

or cohesion *c* and used for elasticity analysis. The retaining wall and struts were modeled with as beams.

5 ANALYSIS RESULTS

5.1 Lateral displacement of retaining wall

Figure 10 shows a comparison between the analytical values obtained with the FEM analysis and actual measured values. The analytical values showed good agreement with the measured values at each stage of excavation though some distribution shapes were different. Displacement below the wall also corresponds well.

The result of the elastic analysis using E_{50} in Table 5 is shown in Figure10. The analyzed value is much higher than the measured value in the early excavation stages. However, the analytical value corresponds to the actual measurement value comparatively well in the fifth excavation

stage. It was thought that the stiffness has not decreased much because the excavation unloading is small in the early stages of the excavation. This leads us to conclude that to forecast the retaining wall displacement in each excavation stage, it is necessary to evaluate the soil stiffness corresponding to the amount of excavation unloading.

5.2 Vertical displacement of excavation base

The lateral distribution of the vertical displacement of the excavation base is shown in Figure 11. The depth distribution of the vertical displacement in the centre of the site is shown in Figure 12. The analytical values showed reasonable agreement with the measured values at each stage of excavation.

The result of the elasticity analysis using E_{50} in Table.5 is shown in Figure11. The analytical value is much larger than the actual measurement

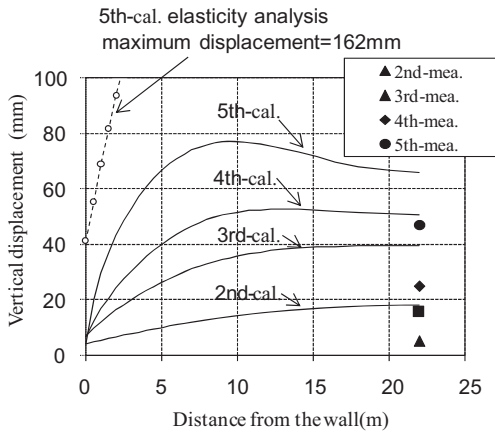


Figure 11. Comparison between analytical and measured value (vertical displacement away from wall).

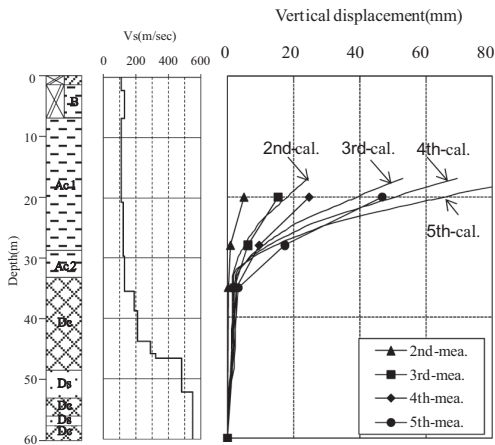


Figure 12. Comparison between analytical and measured value (vertical displacement with depth).

value, because a reducing stiffness E was used in the analysis. It is important to adopt an appropriate soil stiffness considering the stress strain conditions to predict the vertical displacement appropriately.

6 CONCLUSIONS

The lateral displacement of the retaining wall and the vertical displacement of soil in soft clay were evaluated by FEM analysis. In the FEM analysis, the deformation modulus at small strain E_0 and

Mohr-Coulomb model was adopted. The following conclusions were made:

1. The analytical values of maximum lateral displacement of the retaining wall showed good agreement with the measured values at each stage of excavation.
2. The analytical values of vertical displacement of soil showed reasonable agreement with the measured values at each stage of excavation.
3. An elastic analysis was executed for comparison. The analytical value was much higher than the actual measurement value at the end of excavation.
4. The vertical displacement of the ground was over predicted by the elastic analysis.
5. It is necessary to evaluate the soil deformation modulus appropriately to forecast lateral and vertical displacements. It is concluded that the Mohr-Coulomb model and deformation modulus at small strain E_0 are suitable for excavation analysis.

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