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A case of a braced excavation in Bangkok clay

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ABSTRACT: This paper describes a case of an engineering project where a shoring system called Island Method for a deep and large scale excavation was carried out in Bangkok soft clay ground. The displacement of the retaining wall at the first excavation stage was larger than the computed value by using a design method based on the assumption of a beam on elastic foundation. This induced the authors to analyze the problem by employing a soil-water coupling finite element program based on Critical State Soil Mechanics concepts in order to estimate the deformation and slope stability at the final excavation stage. Since the displacement of the retaining wall must be kept small to secure the capacity for the underground car park, we made some modification of the shoring system and the excavation method. The displacement of the supporting wall were kept smaller than expected.

1 INTRODUCTION

The construction presented in this paper was a braced excavation for a building with a 39 level tower and three underground basement floors in Bangkok, Thailand. Figure 1 shows the plan of project. This construction aimed to build a underground car park in this tall building and had the following characteristics: (1) The area is a large scale excavation work with dimensions of 100m × 180m, 12m and 10m deep. (2) The geometry of the excavation is a complex shape. (3) Some parts of the excavation area faced on existing buildings and a heavy traffic road.

It is generally concluded for excavation works in Bangkok that a supporting system with diaphragm wall and fully braced temporary support is more suitable (Balasubramaniam et al. 1992). In this case, however, since the plan dimensions of the excavation are too large to expect the supporting effect of bracing, a shoring system using both the secant piles for retaining walls and the Island construction method with raking struts for bracing system are adopted for most of excavation work. Figure 2 shows the construction sequence and a general cross section. Figure 3 shows the plan view of raking struts, horizontal struts, the instrumentation and its arrangement plan.

For adapting the Island Method to soft clay ground, excessive deformation of walls and slope stability problems in front of the walls caused by excavation are potential problems (Architectural Institute of Japan 1988). It is therefore essential for a successful deep excavation that there is careful field observation and quick feedback of its results to the practical work.

2 GROUND CONDITION

Figure 4 shows the soil profile of the ground. Near the ground surface, a 1.5m thick clay layer called weathered crust is deposited on a very soft clay layer of about 10m.

The unconfined compression strength q_u of the soft clay is very small, being only 20 to 40 kPa, and the sensitivity ratio is 3 through 7, while heaving

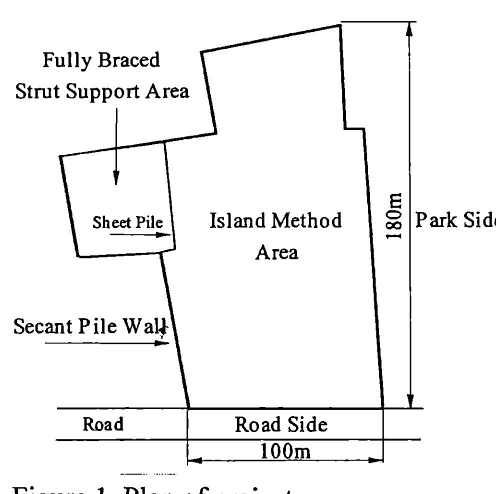

Figure 1. Plan of project.

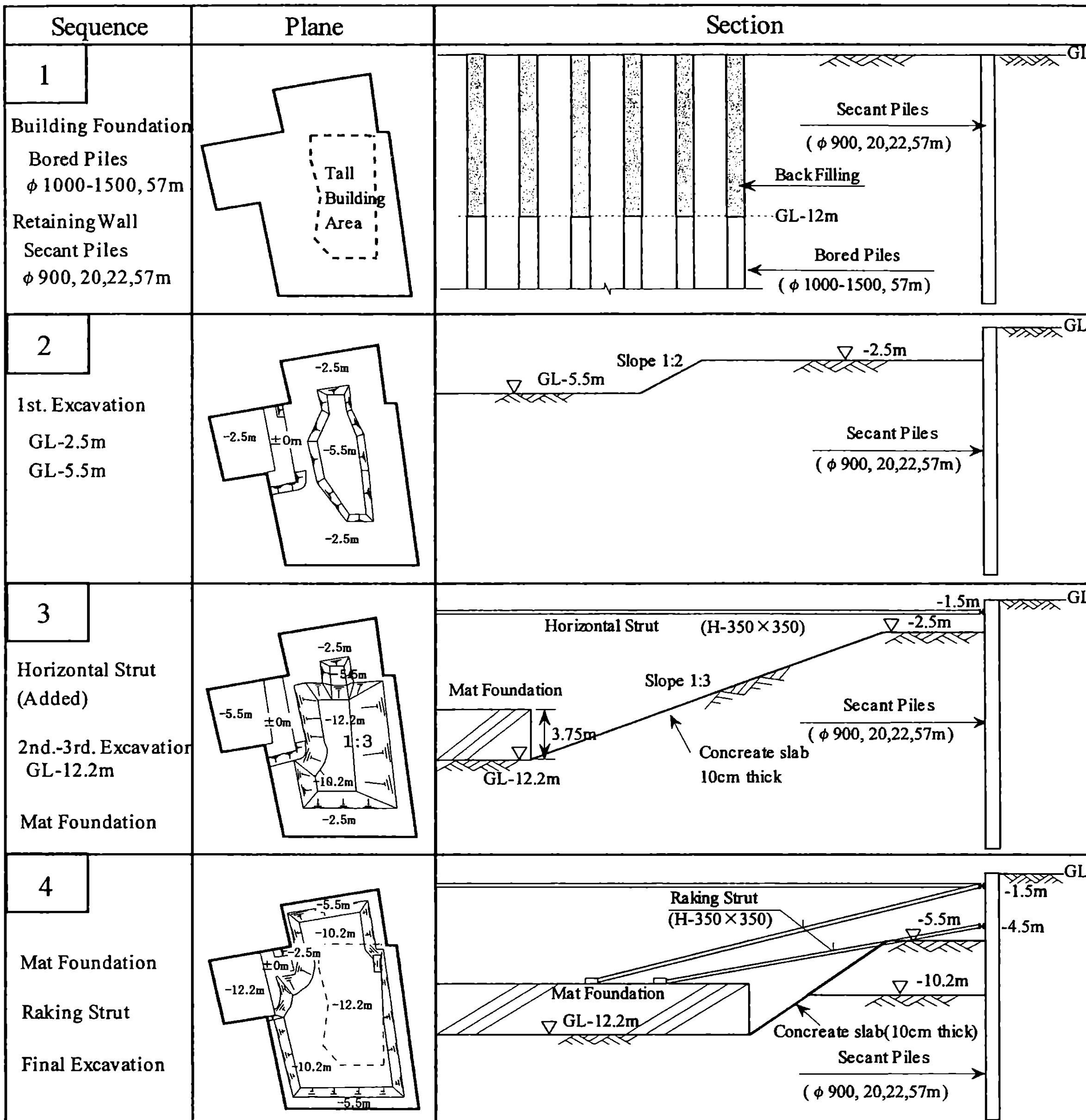

Figure 2. Construction sequence.

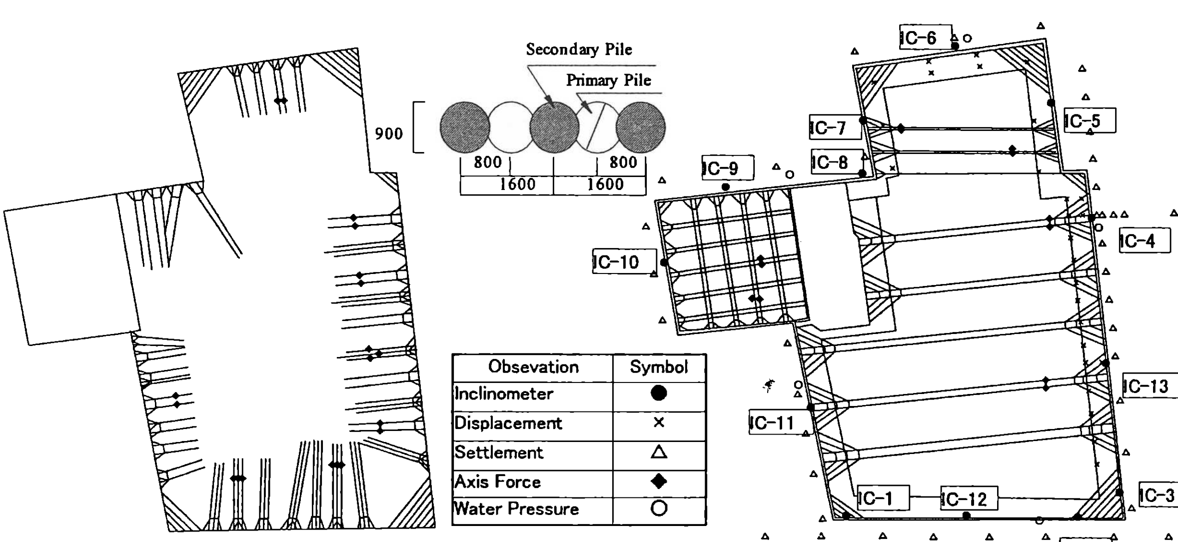

Figure 3. Layout plan of raking struts, horizontal struts and instrumentation.

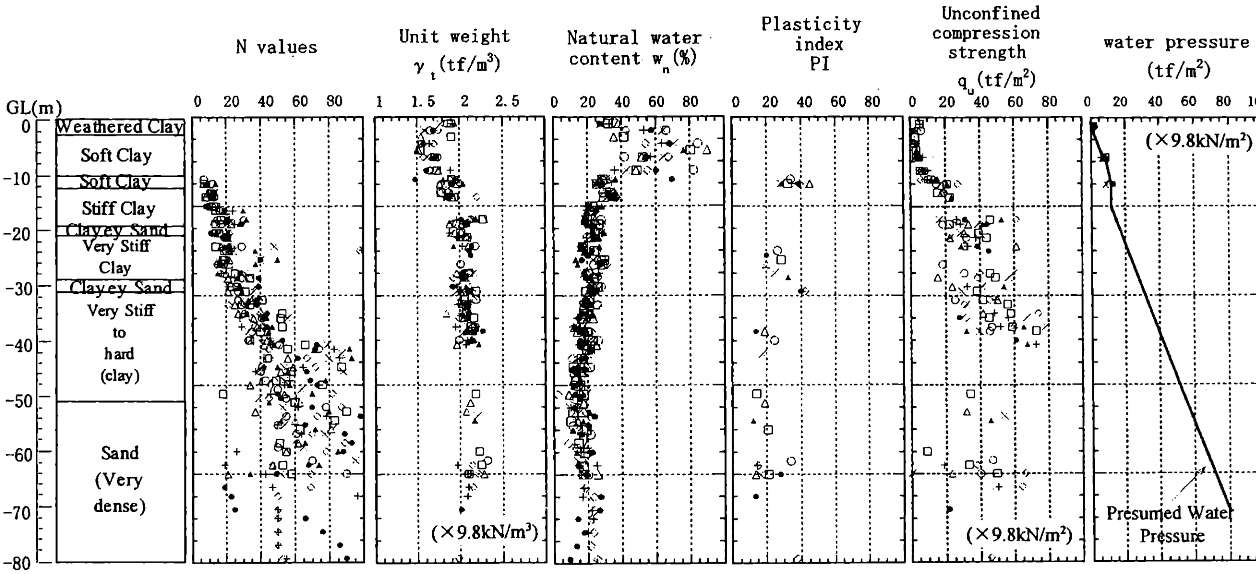

Figure 4. Soil profiles.

stability number N_b proposed by Peck (1973) is about 10, higher than a critical stability number of 7 to 8. There was concern that the soft clay layer may cause engineering problems during excavation work. Below this layer are stiff clay and very stiff clay layers with N-value of 10 to 40.

3 BEHAVIOR OF RETAINING WALL AT THE FIRST EXCAVATION STAGE

When the first excavation (see Figure 2-2) was carried out after driving of bored piles (556 sets) with a diameter of 1000-1500mm and a depth of 57m for the building, the following behavior was observed.

1. Secant pile walls are deformed to cantilever-like deformation as shown in Figure 5, resulting in a 3.5 to 6cm deformation at the top, about twice the initially predicted value (2.5cm), so we had to suspend the excavation work.
2. In spite of the suspension of the excavation work, the lateral displacement rate of secant pile walls showed creep-like deformation of 1.7 - 6.8 mm/day as shown in Figure 6.

Although we were not able to make clear the reason of this behavior, it is inferred that the following factors may have been influential:

1. The ground strength was weakened due to foundation piles driven for the tall building within the area, and due to back filling into the bored pile holes above GL-12m before the first excavation.
2. Accumulated pool-like water due to a heavy rain encountered during first excavation work may have affected the passive resistance of the ground, developing a creep-like deformation of the secant pile walls.

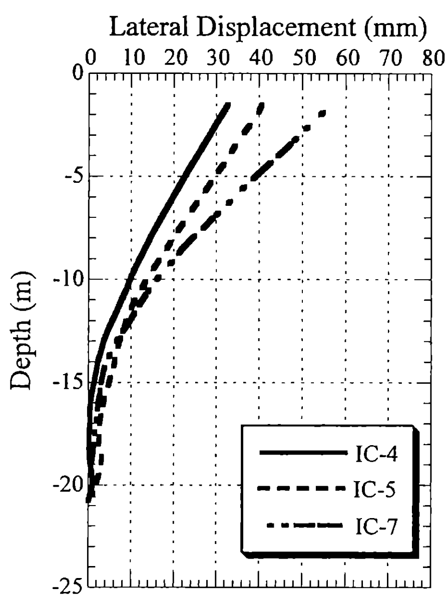

Figure 5. Deformation of secant pile wall at first excavation stage.

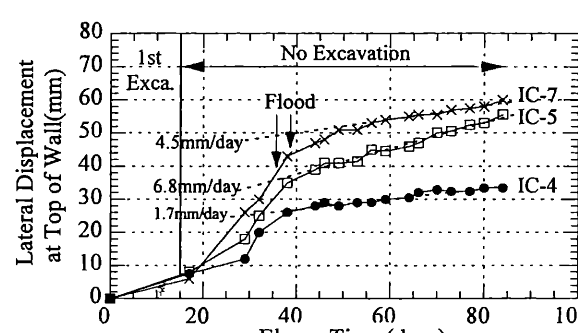

Figure 6. Lateral displacement of wall with elapsed time.

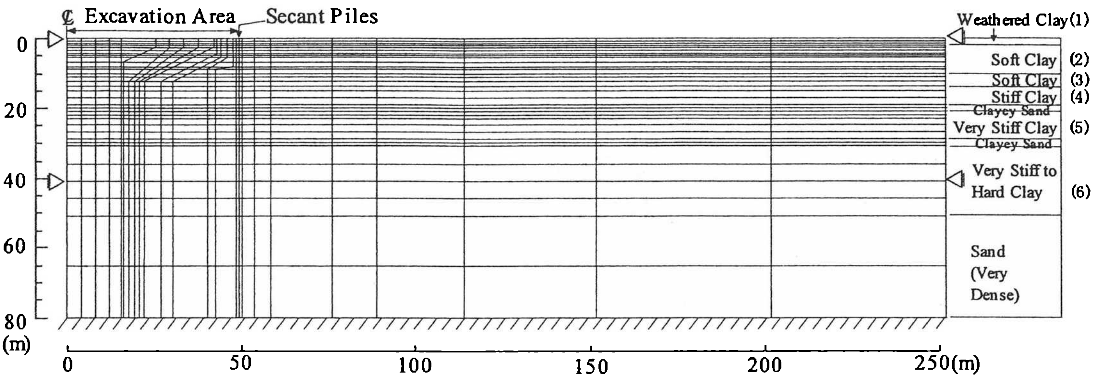

Figure 7. FEM mesh form.

Table 1. Input parameter for elasto-viscoplastic materials.

layer	D	Λ	M	υ	k_x *1)	k_y *1)	K_0	K_i	α *1)	v_0 *2)	λ	e_0	λ_k
(1)	0.035	0.984	3.50	0.412	3.0	1.5	0.70	1.17	6.17	1.0	0.300	0.83	0.300
(2)	0.104	0.977	1.76	0.386	0.5	0.25	0.63	0.65	8.91	1.0	0.475	1.68	0.475
(3)	0.106	0.977	1.76	0.429	3.0	1.5	0.75	0.78	9.58	1.0	0.475	1.48	0.475
(4)	0.043	0.943	2.99	0.385	3.0	1.5	0.63	0.68	6.79	1.0	0.250	0.80	0.250
(5)	0.042	0.945	3.60	0.394	3.0	1.5	0.65	0.97	8.89	1.0	0.260	0.47	0.260
(6)	0.037	0.930	3.66	0.394	3.0	1.5	0.65	0.90	6.54	1.0	0.204	0.64	0.204
					(m/day)	(m/day)					*1)$\times 10^{-3}$	*2)$\times 10^{-7}$	

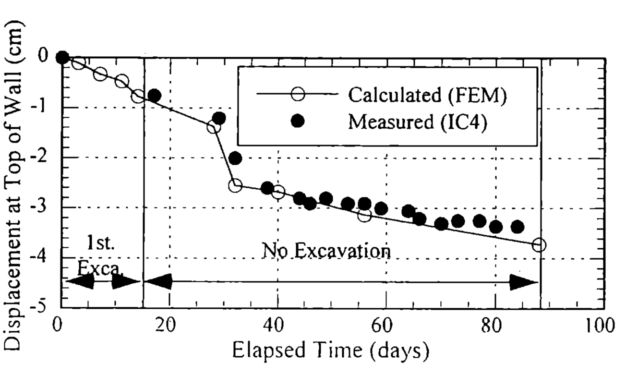

Figure 8. Comparison between calculated and measured value. (Lateral displacement of wall)

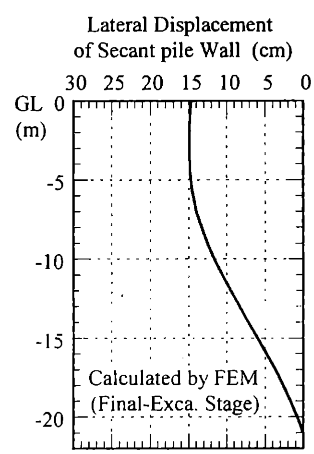

Figure 9. Deformation of secant pile wall. (Final excavation stage)

4 PREDICTING ANALYSIS BY FEM

In order to evaluate the displacement of the wall and the slope stability of the shoring system at the final excavating stage, predicting analysis was carried out by employing a soil- water coupling finite element program called DACSAR (Deformation Analysis Considering Stress Anisotropy and Reorientation) based on the Critical State Soil Mechanics concept. The program was developed by Iizuka and Ohta (1987) using an elasto-viscoplastic constitutive model proposed by Sekiguchi and Ohta (1977). Figure 7 shows an analytical mesh form.

As to material parameters for FEM analysis, general values (Muktabhant et al.) for the Bangkok clay and a determination procedure of input parameters in elasto-viscoplastic finite element analysis provided by Iizuka and Ohta (1987) are employed. (see Table.1)

Comparison between displacement of the secant pile wall with time during the first excavating period and observed values are shown in Figure 8. The analytical results agree with measured displacements. The deformation of retaining wall at final excavation stage was predicted as shown in Figure 9. If no shoring system modification was made, a displacement up to 15cm would result and the bending moment of the wall was expected to reach close to the maximum permissible value.

Thus, it was clarified that the stability of shoring system was secured, while the deformation of the wall was larger than the designed value from the results of FEM analysis.

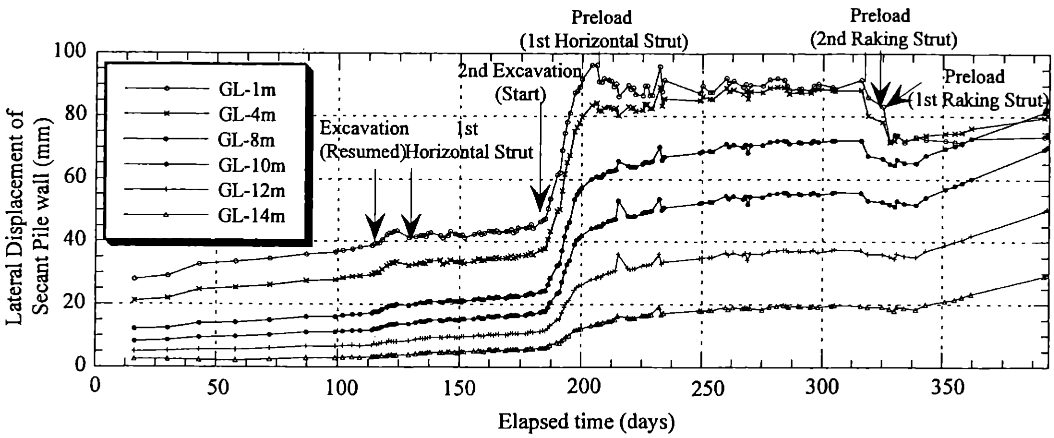

Figure 10. Changing displacement of wall with elapsed time.

5 MODIFICATION OF SHORING SYSTEM

Since the displacement of the retaining wall must be kept small to secure the capacity for the underground car park, we made modifications to the breasting system. After the following countermeasures were performed, the next excavation work was resumed.
1. Horizontal struts were added to near the top of the retaining walls. (See Figure 2-3)
2. The slope of the passive earth buttress in front of walls was reduced from planed 1:2 to 1:3.
3. The faces of the slope were covered with concrete of 10cm thick to prevent the faces from being eroded away by rainwater.

6 RESULTS OF FIELD OBSERVATIONS

6.1 *Displacement of breasting*

Figure 10 shows changing displacement with passing time of a most deformed part of the wall (IC4, see Figure 3). The displacement of the wall increased its pace with a rate of 2mm/day after starting the second excavation, finally reaching about a maximum of 10cm which is 5cm smaller than expected. A depth distribution diagram is shown in Figure 11. As shown in this figure, the shape of deformation is changed between second and third excavation stage. The secant pile wall was deformed in a cantilever-like manner up to the second excavation stage, while the part of about 8m underneath the top of wall was deformed to the front of wall at third excavation.

6.2 *Strut axial force*

Table 2 shows the comparison between measured maximum strut axial forces and designed value. The axial force of struts installed in the road side direction indicate within 75-86 % of the design value, while in the park side direction, the axial force was 50-92 %. The maximum axial force of the horizontal strut was 1764 kN/strut.

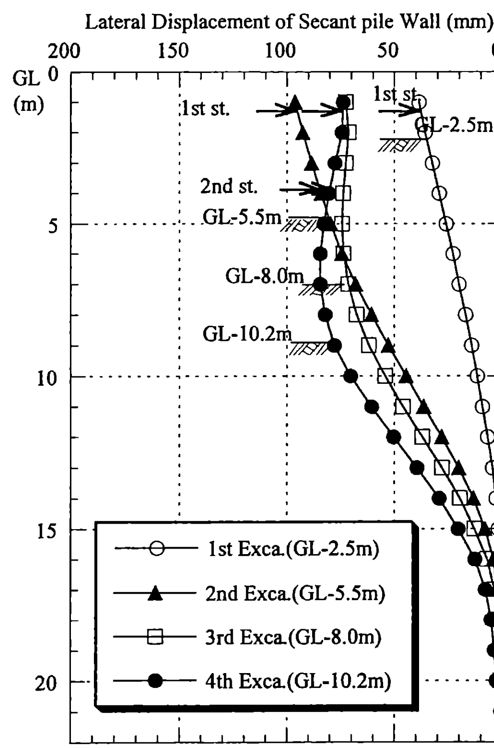

Figure 11. Measured deformation of wall (IC-4).

Table 2. Maximum axial force of struts.

Strut	Designed (kN/set)	Observed (kN/set)	
		Road side	Park side
1st. raking	1372	1176	686
2nd. raking	2352	1764	2156
1st. horizontal	-	-	1764

6.3 *Ground surface settlement of surrounding areas*

Figure 12 shows the ground surface settlement of surrounding areas (park side) due to excavation works. The maximum surface settlement was about 11cm near the secant wall in park side direction. This value is similar to the maximum displacement of the

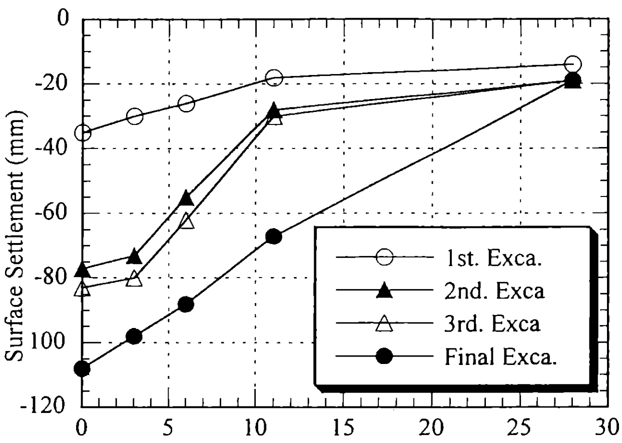

Figure 12. Ground surface settlement. (Park side)

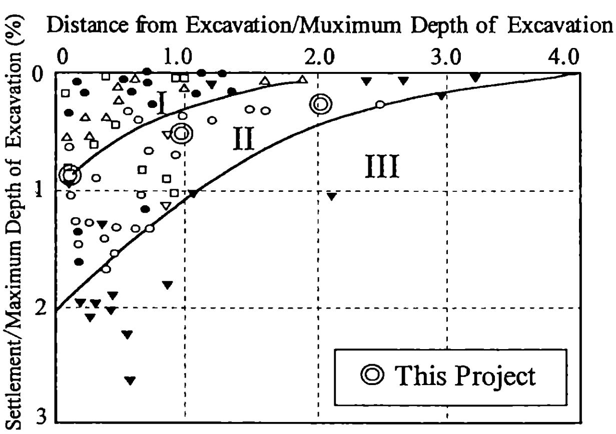

Figure 13. Settlement of surrounding area.
(after Peck (1969))

wall. In the different direction (road side), the maximum settlement was 4cm. Interpreting data based on the Peck (1969) resulted in Figure 13.

The park side direction with bigger subsidence rate is plotted in the II region. According to Peck (1969), II region indicates the following case, 1) very soft clay ground, 2) clay layer is deposited up to nearby final excavation depth. In the case of the excavation works, ground conditions are estimated to be in II region of Peck' evaluation method. It is considered that the ranges of settlement effects on the surrounding areas are predicted by using Peck's evaluation method.

7 CONCLUSION

The planned Island construction method was required the following changes because of bigger-than-expected displacements of the first excavation.

1. Using horizontal struts together with raking struts.
2. Easing the slope of the surface to about 1:3 from 1:2.
3. Protecting the surfaces with 10cm thick concrete.

These measures were effective in securing the decrease of wall displacement and the stability of the face of the slope.

Also back analyses as well as field observation methods have made clear that the deformation and stability of shoring system can be secured. We were therefore able to complete safely the final excavation stage without building the third raking struts planned in the original shoring system.

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