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In situ monitoring and analysis of a cut-and-cover tunnel on the Bay-Shore Route

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ABSTRACT

This paper describes the effectiveness of in-situ monitoring and analysis for large-scale cut-and-cover tunnel work in the soft ground which has been reclaimed just recently in the Haneda area of the Bay-Shore Route of Metropolitan Expressway. In spite of efforts to stabilize the earth retaining wall by means of ground improvement of the bottom of excavation, the symptom of heaving was observed. By early detection of such symptom through in-situ monitoring and adequate countermeasures, the work could be completed without incident. As is known from the results of analysis, the ground improvement of the bottom proved to be an effective stabilization measure while ground improvement by arranging piles in a mutually contacting manner was not as effective as expected in enhancing the vertical resistance of the composite ground.

1. INTRODUCTION

The Bay-Shore Route of the Metropolitan Expressway celebrated its opening on December, 1994, connecting reclaimed land along the coast of Tokyo Bay. At Haneda Airport and neighboring areas, a structure with retaining wall or tunnel was planned because of the limited open areas around the Airport and was constructed according to the cut-and-cover method.

This was a large-scale excavation work in soft ground which had been only recently reclaimed and may be quite a rare example. In addition to earth retaining with large-size steel pipes and ground improvement of the excavation bottom, measuring instruments were installed to the earth retaining wall and surrounding ground for field observations.

The excavation work in this section was large in scale: about 120 m long, about 30 m wide, and about 21 m deep. In the course of excavation, the heaving

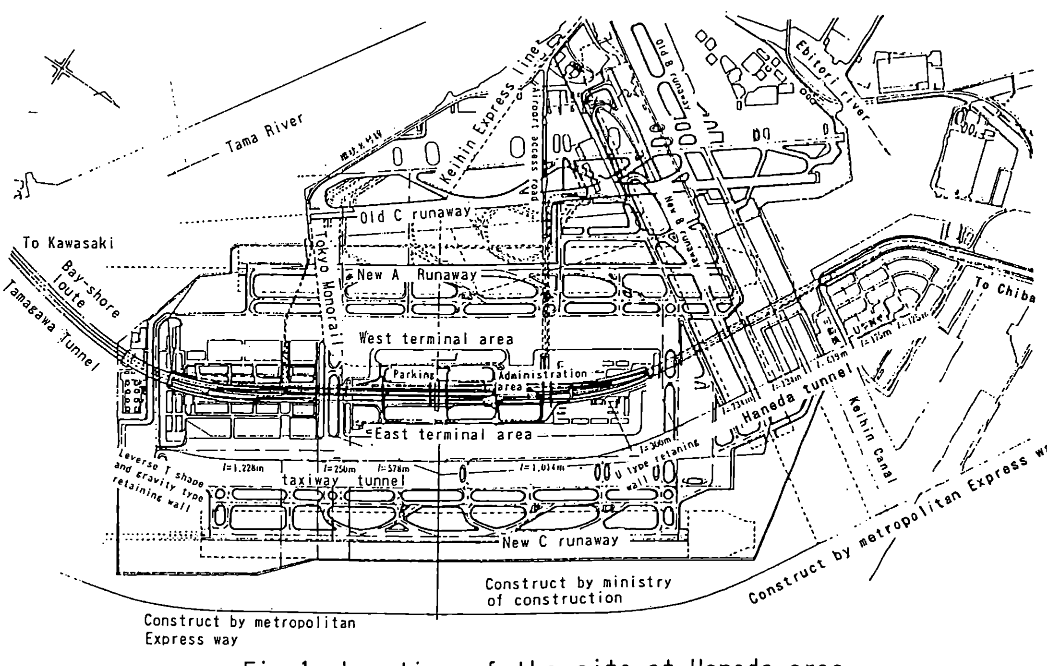

Fig-1. Location of the site at Haneda area phenomenon (upward displacement of the excavated bottom) and a continuous increase in the deformation of the earth retaining wall were observed. Thanks to the monitoring of the behavior by means of measuring instruments and appropriate countermeasures (removal of the soil from the back of walls, water injection in to the excavated side for balancing), this work could be completed without incident.

This paper reports on the in-situ measurement results, analytical results and discusses the heaving behavior.

2. OUTLINE OF THE GROUND AND WORK

As shown graphically in Fig. 2, the soil composition in the Haneda area consists of the surplus soil from construction (Bs layer), dredged soil (A_m and Ac_1 layers), an alluvial sand layer (As_1 layer) which was left over from past dredging, and an alluvial clayey soil layer in this order from the ground surface downward. Below the alluvial clayey soil layer, there are diluvial clayey soil and sandy soil layers. The bearing stratum with $N \geqq 50$ was found at Ap - 45 to 60 m or deeper.

Among reclaimed soil layers found in the surface course, A_{ml} and Ac_1 layers are layers reclaimed with soils deposited from dredging between 1971 - 1981. The Bs layer was reclaimed by using surplus soils from construction carried out in 1981 and thereafter.

Table 1 summarizes the soil properties of the Haneda area. The reclaimed soil layers (A_m and A_{c1} layers) in particular are so soft they are often referred to as the "Haneda mayonnaise layer."

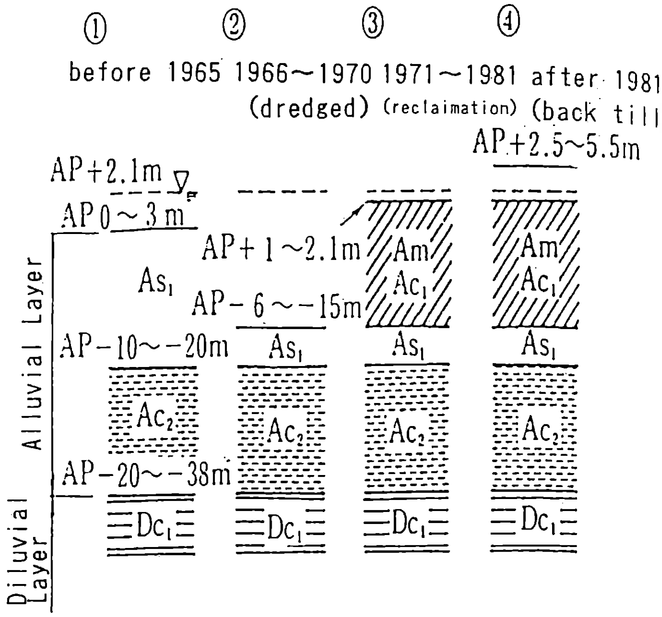

Fig-2.

In this ground, the excavation work proceeded according to the procedure shown in Fig. 3. Fig. 4 shows the standard sectional view of the timbering of a cut. The earth retaining wall was constructed with steel pipe sheet piles (1000 in dia., t = 12 mm, ℓ = 32.8 m) with struts arranged in seven stages. The size of struts in the third and deeper stages was increased to cope with high earth and water pressures.

Ground improvement was made at a depth of 5 m below the excavation bottom by arranging piles in a

Table 1 Soil properties of the ground in the offshore areas, Haneda area (Before ground improvement)

Soil classification	Physical properties		Mechnical properties			
	Natural water content W_n	Wet density ρ_t	Unconfined compressive strength q_u	Consolidation yield stress P_c	Coefficient of volume compressibility m_v	Compression index C_v
A_m	Varying greatly within a range of W_n = 50 - 180%. Mostly at W_n = 130%.	Varying approximately within a range of ρ_t = 1.3 - 1.5 g/cm^3	Varying approximately within a range of q_u = 0.1 - 0.5 kgf/cm^2 and increasing with increasing depth	Approximately equal to the effective overburden pressure. In the normal consolidation condition	$m_v = 10^{-0.92(\log P + 1.0)}$	C_v = 70cm^7/day
A_{c1}	Varying greatly within a range of W_n = 40 - 120%. Mostly at W_n = 80%.	Varying approximately within a range of ρ_t = 1.35 - 1.65 g/cm^3 and increasing with increasing depth	Similar to the A_m layer. Varying approximately within a range of q_u = 0.1 - 0.5 kgf/cm^2 and increasing with increasing depth	Approximately equal to the effective overburden pressure. In the normal consolidation condition		
A_{c2}	W_n = 60 - 80% at AP-20 m or above and W_n = 80 -100 % at AP - 20m or deeper	ρ_t = 1.55 - 1.65 g/cm^3 at AP-20 m or above and ρ_t = 1.45 - 1.50 g/cm^3 at Ap-20 m or deeper	Varying approximately within a range of q_u = 0.8 - 2.0 kgf/cm^2 and increasing with increasing depth	About 5 tf/m^2 higher than the effective overburden pressure. In the over-consolidated condition	$m_v = 10^{-0.91(\log P + 0.87)}$	Normal consolidation: C_v = 100cm^7/day Over consolidated: C_v = 700cm^7/day
D_{c1}	Within a range of W_n = 40 - 65%, decreasing as the depth increases	Varying approximately within a range of ρ_t = 1.65 - 1.75 g/cm^3 and increasing with increasing depth	Varying approximately within a range of q_u = 1.5 - 2.5 kgf/cm^2 and increasing with increasing depth			

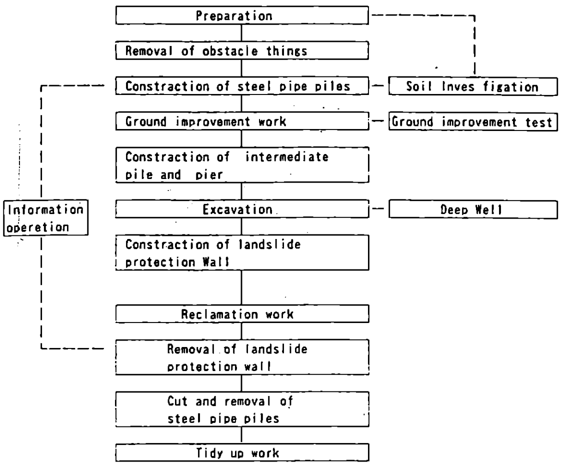

Fig-3. Work Procedure

mutually contacting manner according to the DJM method (1000 in dia.) as shown in Fig.5. The space to the earth retaining wall (steel pipe sheet piles) was filled through ground improvement according to the JMM (1200 in dia.) method.

3. IN-SITU MONITORING

3.1 Outline of the in-situ monitoring

Fig. 6 shows the layout plan for the measuring instruments. Measurement of the earth retaining wall was mainly measurement of the deformation with insertion type inclinometers which were installed at six points. Strut load was measured at two points in the transverse direction and one in the vertical direction. Differential settlement gauges were installed, to monitor heaving, at two points in the middle of the excavated area. In addition to these, supplementary measurements of earth and water pressures and the settlement of the ground surface and the head portion of the earth retaining wall were carried out.

3.2 Result of field observations

Excavation proceeded carefully and smoothly up until the sixth phase. Then, the upward displacement of the excavation bottom different in its characteristics from rebound due to the removal of the excavation load was observed (Fig. 7). Though deformation of the earth retaining wall was more or less suppressed in the bottom improvement section as shown in Fig. 8, the wall developed deformation over toward the excavated side. As the seventh phase of excavation proceeded, the rate of deformation rose, causing a temporary stop in the excavation work. In spite of this, the deformation did not slow down.

We considered this to be a symptom of heaving, and carried out removal of the soil from the back of walls and water injection into the excavated area as emergency measures. As a result, the bottom of the excavation ceased its upward displacement and the earth retaining wall stopped deforming.

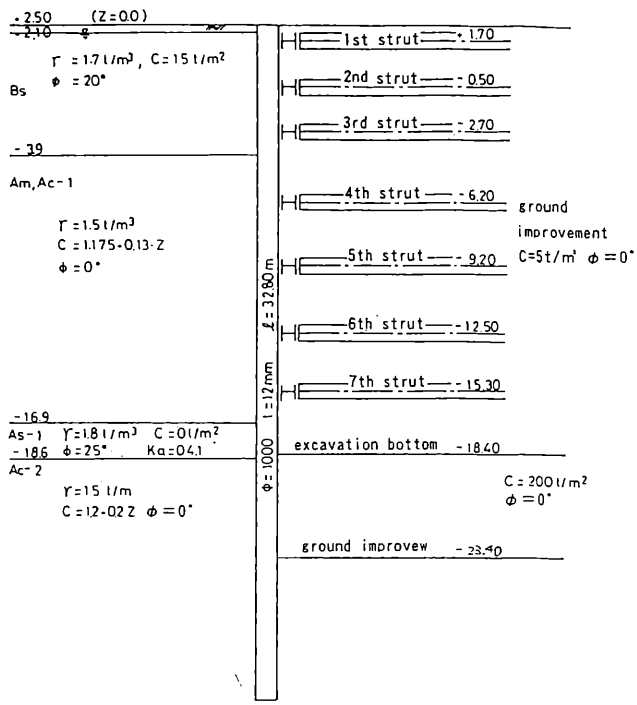

Fig-4.Standard sectioanl view

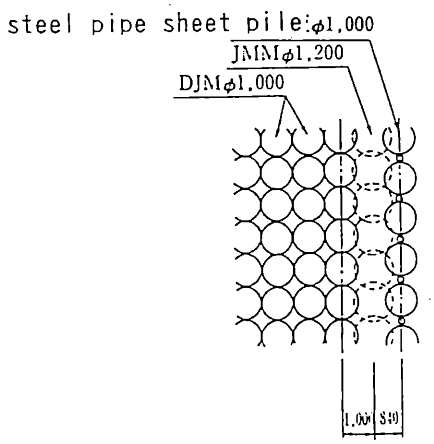

Fig-5.Detaile configration of the soil improvement

4. ANALYSIS OF DEFORMATION

An FEM analysis was made on the deformation behavior of the bottom and earth retaining wall by referring to the field observation results. The analysis was a two-dimensional non-linear (bi-linear) analysis to study the plastic deformation of the ground by simulating the excavation steps sequentially. To study the effectiveness of the bottom improvement, an analysis was made of two cases, one with improvement and the other without improvement. For the case with improvement, the deformation of the earth retaining wall toward the back side due to bottom improvement was designated as forced displacement.

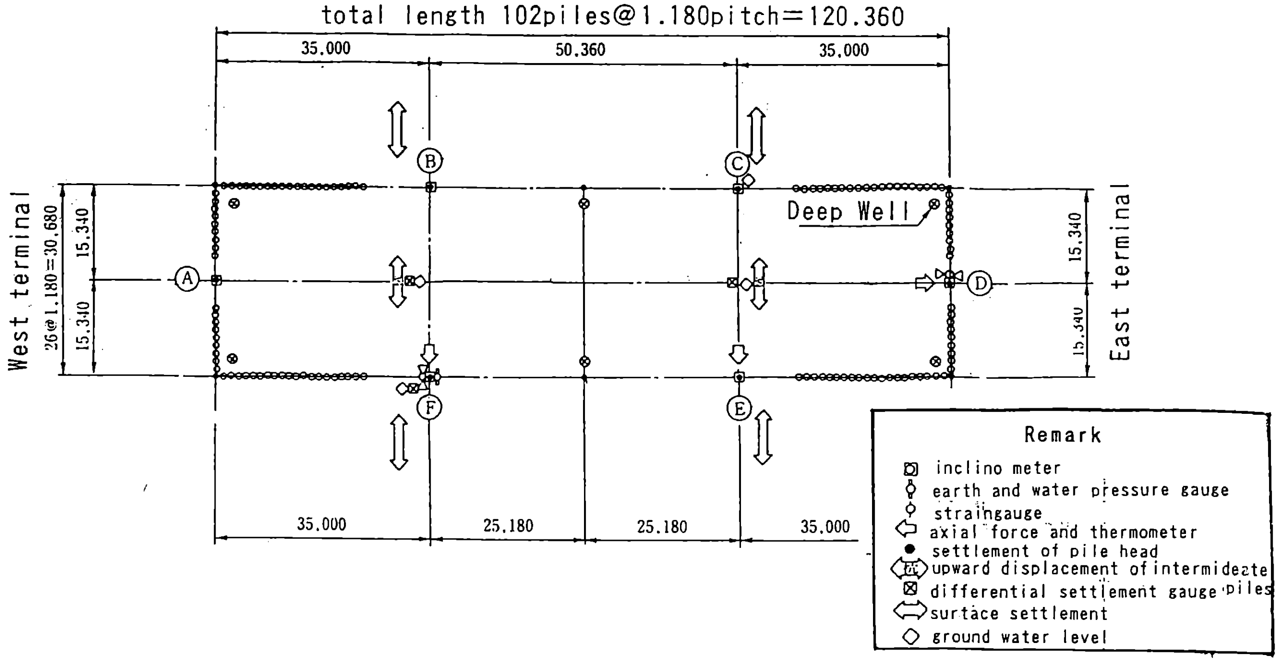

Fig-6.Layout plan for the measuring instruments

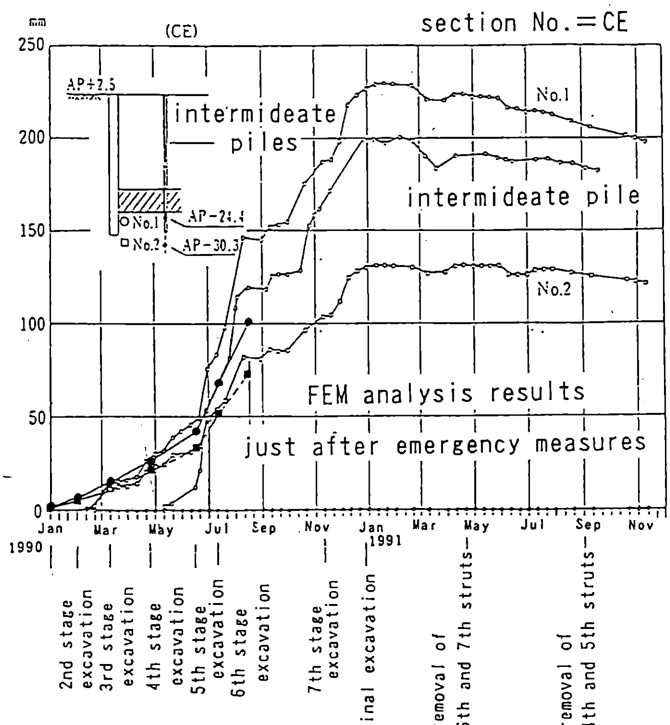

Fig-7.Ground heaving during excavation

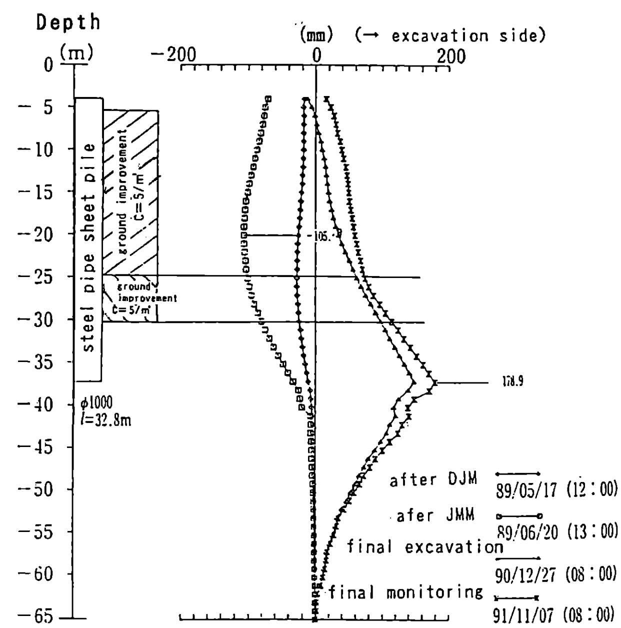

Fig-8.Deformation of the retaining wall

As a result of analyses, the earth retaining wall was found to deflect as a whole toward the excavated side if no bottom improvement had been made. On the contrary, if the bottom improvement had been made, the upper portion of the wall deflected toward the excavated side with the lower portion deflecting toward the back side. And the amount of deflection decreased by about 1/3. (Fig. 9)

Concerning upheaval of the bottom, on the other hand, a sudden increase in sixth and subsequent phases could not be brought nearer to the measured values even when the coefficient of deformation was decreased. This fact may indicate that the ground under the improved bottom had collapsed. (Fig. 7)

Also analyzed were emergency measures; removing the soil from the back of walls and water injection into the excavated area. As far as the result of this analysis is concerned, water injection proved more effective in suppressing upward displacement the excavated bottom than removal of the soil from the back of walls.

5. DISCUSSION

Effective suppression of deformation of the earth retaining wall by bottom improvement could be analytically verified. However, in spite of bottom improvement, the so-called heaving phenomenon (upward displacement of the ground) occurred.

Principal causes may be as follows:

(1) Since bottom improvement was made by arranging piles in a mutually contacting manner and as a consequence the ground could not be considered to be composite ground, sufficient resistance to deformation could not be expected.

(2) Accordingly, there was an increase in the force released through excavation, which induced deformation of the earth retaining wall toward the excavated side. It is highly possible that this is the cause of the decrease in the strength of the natural ground.

(3) Another cause may be the fact that the vertical resistance developed due to bottom improvement through the arrangement of piles in the mutually contacting manner was not as great as the one obtained with the composite ground.

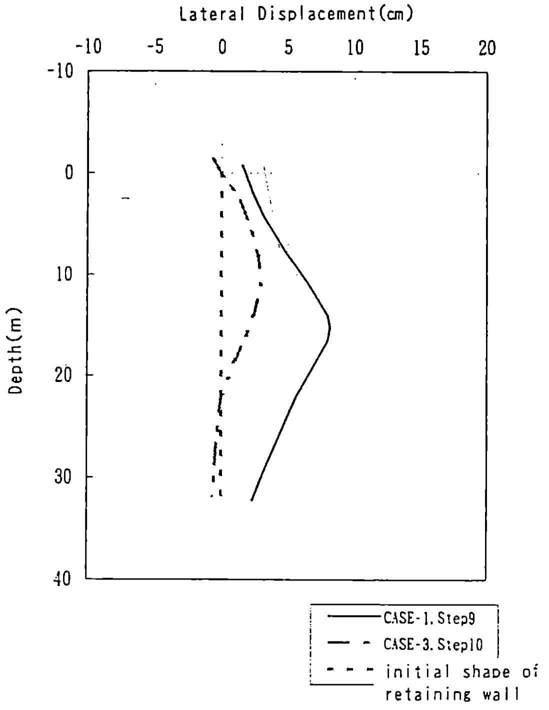

Fig-9. Results of FEM Analysis

6. CONCLUSIONS

The excavation work on this site could be completed without the occurrence of any severe accidents by understanding early on the deformation behavior of the earth retaining wall and ground through in-situ measurement and by taking the appropriate countermeasures on the basis of measurement results.

For large-scale excavation in soft ground in the future, a construction method utilizing information technology similar to the one employed for this site will contribute to the safe completion of excavation. Invaluable data concerning bottom improvement methods in excavated areas was also obtained and should be fully utilized in design and construction in the future.

We would like to express our deep gratitude for the cooperation extended by the many persons concerned during the preparation of this paper.