Geotechnical Aspects of Underground Construction in Soft Ground, Kusakabe, Fujita & Miyazaki (eds)
© 2000 Balkema, Rotterdam, ISBN 90 5809 1 066

A case of deep braced excavation for subway in Tokyo

T. Okada
Tokyo Subway Construction Corporation, Japan

T. Hoshino
Tokyo Metropolitan Government, Japan

S. Sakuma & S. Kohda
Hazama Corporation, Tokyo, Japan

ABSTRACT: In the excavation work (about G.L.-37.0m) to construct the Higashinakano station of Metropolitan subway route No.12 in Tokyo, the displacements of the diaphragm wall and the axial forces of the steel struts and another data were monitored.

In this paper, these monitoring data are compared with the original design. And especially the displacements of the diaphragm wall below middle slab part (t=2.0m) are investigated.

1 INTRODUCTION

The metropolitan subway route No.12 consists of a radial part (from Shinjuku to Hikarigaoka, 13.9km) and a belt line part (from Nishishinjuku to Shinjuku, 28.8km). It is under construction as a new traffic network possible to transfer at many stations of existing JR lines, subway lines, and private lines. Now the radial part is starting business and a belt line part is under construction (Figure 1).

This construction site is placed at the middle of the radial part under Yamate-dori (one of the main streets of Tokyo). We constructed Higashinakano station and two shield tunnels(ϕ5,540,L=910m×2) and a pump shaft between Higashinakano Station and Nakanosakaue Station (Figure 2).

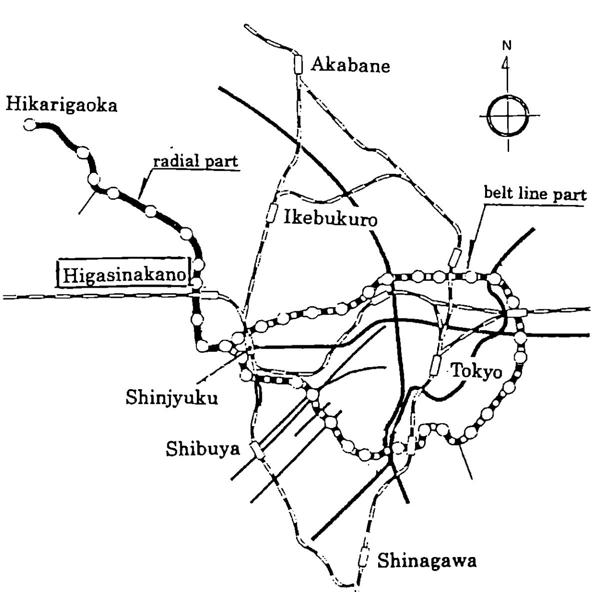


Figure 1. Metropolitan subway No.12.

This paper is the report about cut-and-cuver work for construction of Higashinakano Station.

2 CONSTRUCTION SITE

2.1 *Ground conditions*

The construction site is located on the East Side of Musashino Plateau. The ground is composed of Kanto loam layer (Lm), Musashino gravel layer (Mg), Tokyo layer (Toc), Tokyo gravel layer (Tog), and Edogawa layer (Eds). Eds layer includes a thin clayey sand layer. This clayey sand has 1.5m thickness at the bottom of the reinforced underground diaphragm.

Lm layer: Volcano ash matter clay formation.

Mg: Clayey gravel and gravel (under 20mm except for a few 60mm).

Toc: Clayey and sandy clay.

Tog: ϕ5~50mm gravel and ϕ100mm gravel.

Eds: Mainly hard silt and fine sand.

The underground water level is about G.L.-10.0m for Mg and about G.L.-17.0m for Eds, so the ground water level is not so high(Figure 3).

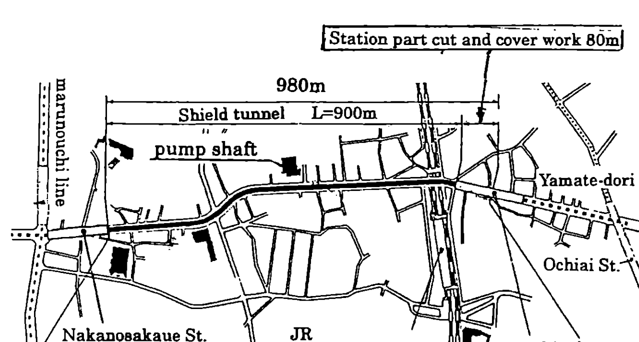


Figure 2. Higasinakano station construction site.

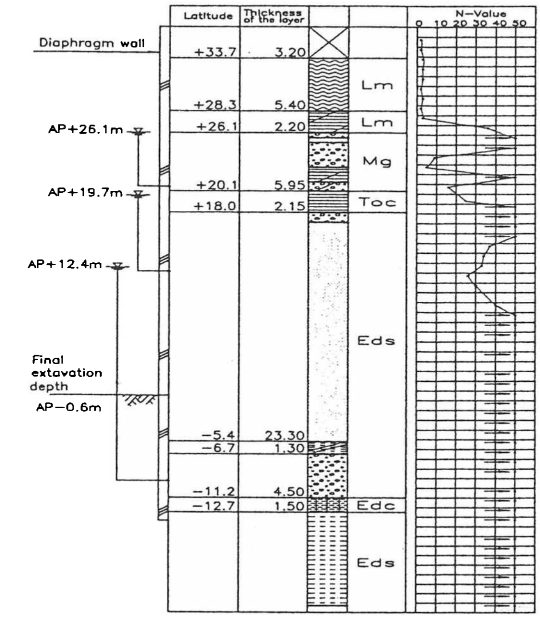


Figure 3. Soil profile at the excavation site.

2.2 *Building site and excavation work*

The underground diaphragm (t=1.2m) is constructed to a depth of G.L.-50.5m by the reiforced concrete using mini-cutter (BC30). Under the slab by top down method (t=2.0m), the steel H-section piles (L=22.0m) by bore-hole method are constructed inside the RC diaphragm wall(Figure 4).

For the original design five steel struts are constructed above the inverted slab and four steel struts are constructed until the G.L.-38.0m excavation.

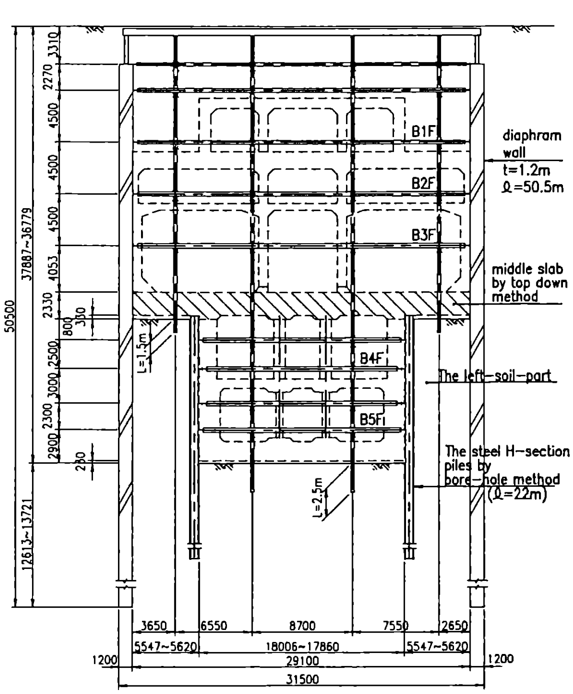


Figure 4. The typical cross section of the excavation.

This structure is composed of 5 floors, B1F is the concourse, B2F and B3F are highway facilities of Metropolitan expressway public corporation in the future, B4F is the equipment room of a ventilator, B5F is platform and tracks. Between B3F and B4F a concrete slab (t=2.0m) is constructed by the top down method.

3.OUTLINE OF MEASUREMENTS

To monitor and ensure the stability of the excavation during construction, instrumentation was installed with inclinometers, strain gauges, earth pressure cell, manometers and thermometers.

3.1 *Inclinometer*

To measure the horizontal displacements of the wall, 16 inclinometers were installed inside the diaphram from G.L.-3.5m to G.L.-48.5m (@3.0m), and 7 inclinometers were installed on BH method wall from G.L.-25.3m to G.L.-43.3m(@3.0m). The displacement is calculated from the angle of inclination on the supposition the bottom of the diaphram wall doesn't move. The displacements are automatically measured 6 times a day.

3.2 *Strain gauge*

To monitor the stress in the diaphragm wall the straining gauges were attached to reiforced bars. On 10 points (inside and outside) of the daphragm wall used as permanent structure the strain gauges were installed. And the monitored stress was compared with the design value.

To monitor the axial force in the struts at every stage struts, the strain gauges were installed on the struts. One pair of strain gauges was installed on both side of the web of the steel H-section and we converted the measured values into the axial force.

3.3 *Thermometer*

To monitor the axial force of the steel struts at every stage correctly, the temperature of the steel struts was measured by the thermometer.

4.RESULTS OF MEASUREMENT

4.1 *Behavior of the wall*

The excavation took almost a year. By the end of the 4^{th} excavation, the maximum horizontal displacement occurred at the top of the diaphragm wall. From the 5^{th} excavation the top of the diaphragm moved towards the soil face. Then the maximum displacement occurred at almost G.L.-25.0m until the last excavation step. And the maximum horizontal displacement is almost 13.0mm. Under the concrete slab made by the top down method the horizontal displacement occurred little. On the other side the maximum horizontal

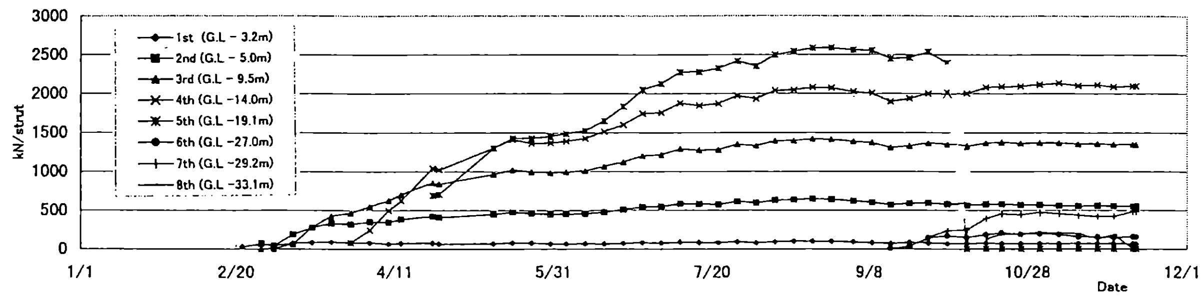


Figure 5. The result of the axial force stage struts at every stage.

displacement of the steel H-section piles by bore-hole method is about 10mm.

4.2 *Axial force of the steel struts*

The axial force of the steel struts is showing a tendency toward large values, as the position of the strut is deeper above the slab (middle slab) top down method. But under the middle slab that axial force isn't showing the above-stated tendency (Figure 5).

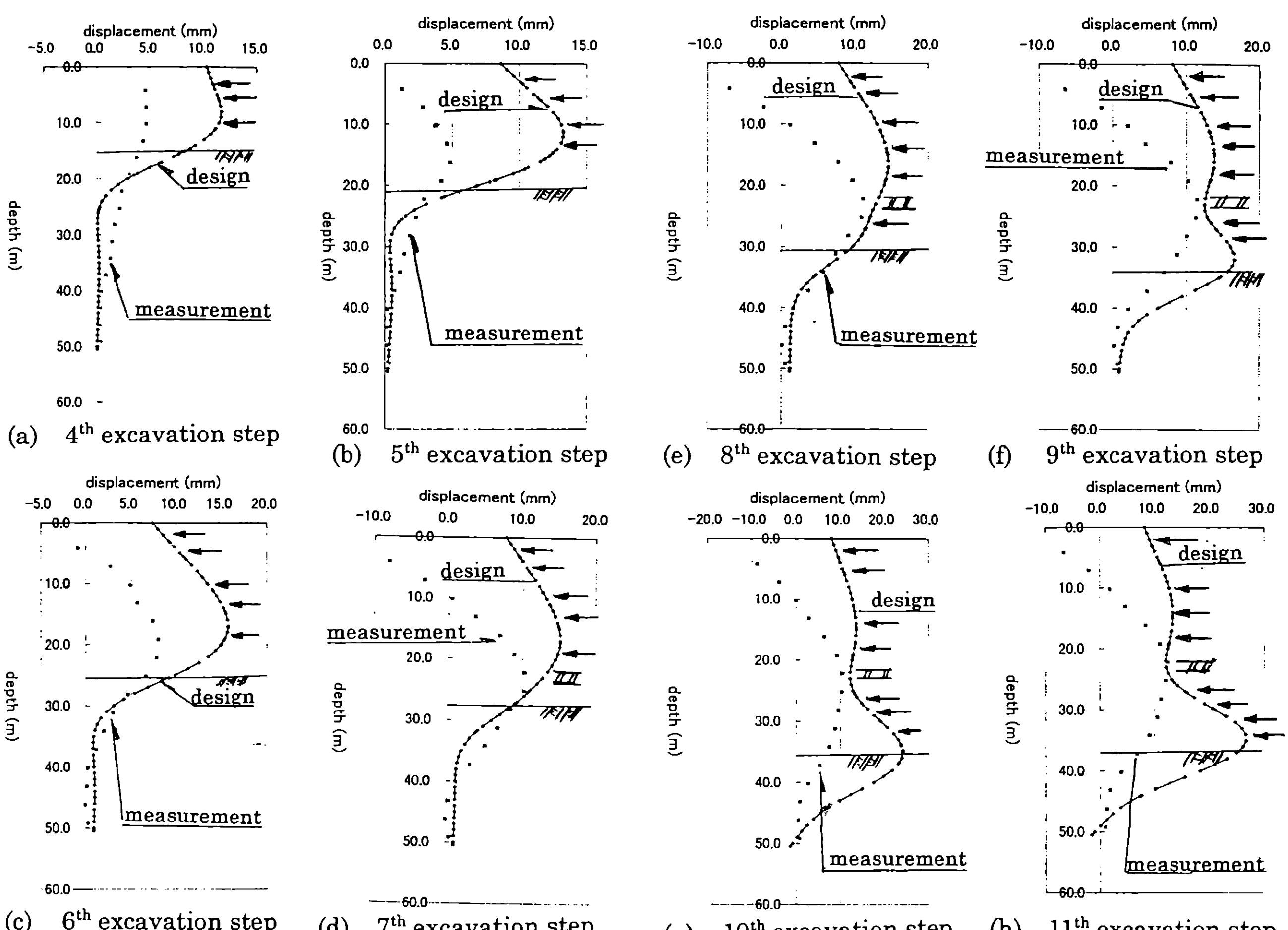
5.COMPARISON OF DESIGN AND MEASUREMENT RESULT

5.1 *Outline of Design*

We designed those temporary structures using the elasto-plastic method for the diaphragm wall. The excavation step has 11.

And the steel struts under the middle slab are modeled as the series springs calculated from the steel strut and soil. The steel H-section piles by bore-hole method is designed by the conventional method.

One of the distionctions of this excavation work is the double diaphragm wall. There isn't the



(a) 4th excavation step
(b) 5th excavation step
(e) 8th excavation step
(f) 9th excavation step
(c) 6th excavation step
(d) 7th excavation step
(g) 10th excavation step
(h) 11th excavation step

Figure 6. Comparison about the displacement of diaphragm wall at 4th –11th excavation step

Table 1. Excavation steps.

Step	Excavation		Strut	
	Date	G.L.-(m)		G.L.-(m)
1	2/2	4.31		
2	2/14	6.58	1st	3.31
3	2/28	11.08	2nd	5.58
4	3/18	15.58	3rd	10.08
5	4/8	20.08	4th	14.58
6	5/14	25.463	5th	19.08
7	9/7	28.263	Slab by top down method	24.133
8	9/15	30.763	6th	27.263
9	10/3	33.763	7th	29.763
10	10/20	36.063	8th	32.763
11	11/30	37.963	9th	35.063

authorized design method for this case. Therefore we designed the diaphragm wall by the elasto-plastic method which is ordinary method for the deep excavation work in Japan. We should comfirm wheter the elasto-plastic method expresses the displacement of the case like this.

5.2 *Comparison about the displacement of wall*

For all steps the displacement of the diaphragm by the design exceeded the measured displacement predicted. We supposed that the overestimate of the active earth pressure and the underestimate of the horizontal ground reaction coefficient cause this difference. The design couldn't represent the behavior of the diaphragm wall from 5th excavation. This difference is caused by as follows(Figure 6). At the top of the diaphragm wall the displacement on the design is twice times as large as the measurement at 1st excavation step. Therefore the displacement of the top of the diaphragm wall is restricted to the back face. On the other side the design couldn't represent the displacement of the diaphragm wall under the slab top down method. We describe this part as the left-soil-part the following. We investigate the difference of the displacement of

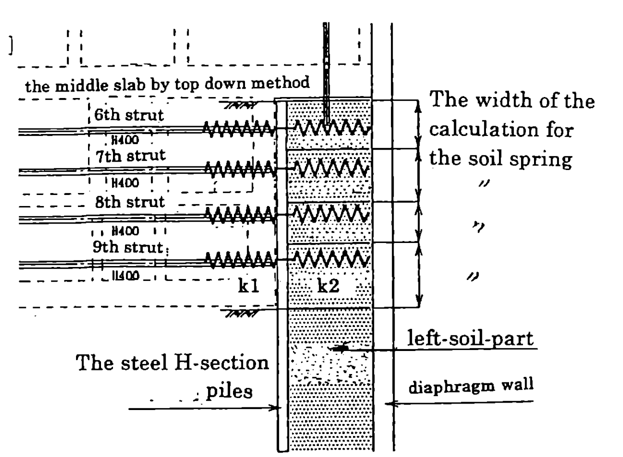


Figure 7. The series linear spring of the original design.

the design and measurement at 6.INVESTIGATION OF THE BEHAVIOR AT THE LEFT-SOIL-PART.

5.3 *Comparison of the axial force of the steel struts*

Compared with the design of the axial force of 3rd and 4th and 5th strut exceed the measurements. Especially the axial force of the 5th strut exceeded the allowance of the waling's shear force for a while, then we had to reinforce the waling and monitor the change of the axial force carefully. The measured axial force of the struts under the slab top down method was less than the design.

6.INVESTIGATION OF THE BEHAVIOR AT THE LEFT-SOIL-PART

6.1 *Original design of the diaphragm wall*

On the original design we designed by the model of elasto-plastic method. And the model about the left-soil-part was described by the series linear spring calculated from the steel strut and horizontal ground reaction coefficient ($k=1/(k1+k2)$; k1:the spring of the strut spring,k2:the spring of the left-soil-part)(Figure 7).

6.2 *Another techinique for the elasto-plastic method*

Another techniques for the elasto-plastic method to represent the displacement of diaphragm wall at the left-soil-part.

We tried to represent the displacement of diaphragm wall at the left-soil-part using elasto-plastic method by another technique as follows (Figure 8).

Technique 1: Increase the flexural rigidity for the diaphragm wall about the left-soil-part.

We used the flexural rigidity for the diaphragm wall about the left-soil-part to add the soil($Es \cdot Is = 61{,}000 \times 1/12 \times 1.0 \times 5.5^3 = 850{,}000$ kN/m^2).

Technique 2: The suppositional depth of the excavation.

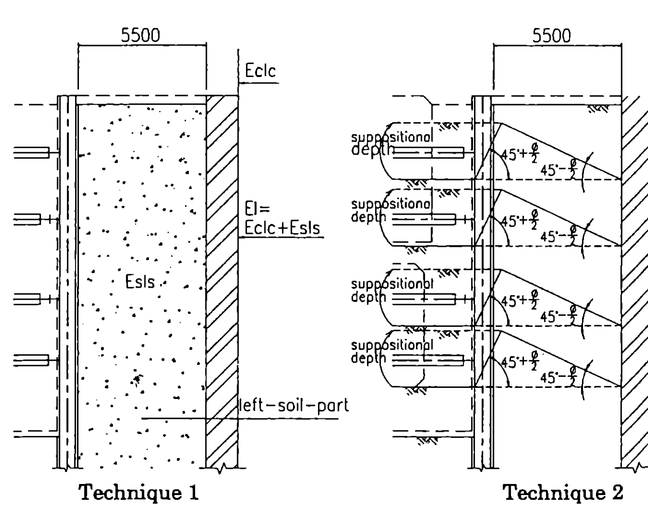


Figure 8. Another techniques for the elasto-plastic method.

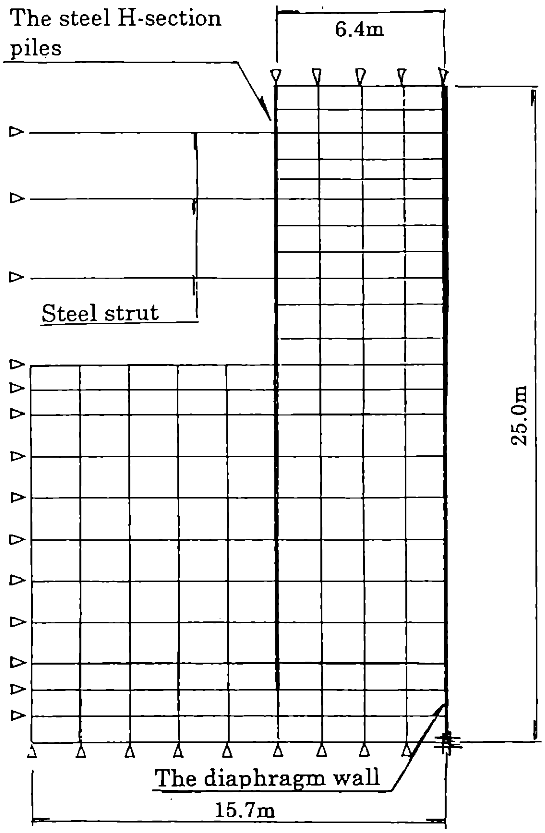


Figure 9. Finite Element Method model for the 10th excavation step.

From the 8th excavation step, the depth of the excavation is supposed the cross point between the line of the angle the active collapse and of the positive collapse.

However we couldn't suppress the convex shape of the displcement of the diaphragm wall. Therefore about the left-soil-part the linear springs couldn't suppress the displacement of the diaphragm wall.

6.3 *Finite Element Method analysis of the left-soil-part for check*

In order to check the difference of the displacement at the left-soil-part we analyzed the displacement using the linear finite element method model as follows (Figure9).

Analysis condition:

Force; The increment of the active earth pressure for the every excavation

Modulus of deformation of the Eds layer; 61,000 (kN/m^2) (The maximum value of the test) and 122,000(kN/m^2)(Twice of the maximum value).

Area of the analysis; a half of the width of the excavation.

Boundary condition; refer to Figure 9.

The comparison of the displacement at 10th excavation step about the left-soil-part. This figure shows that the displacement of E=122,000(kN/m2) fitts to the measured displacemnet of the diaphragm wall. It means that the modulus of deformation

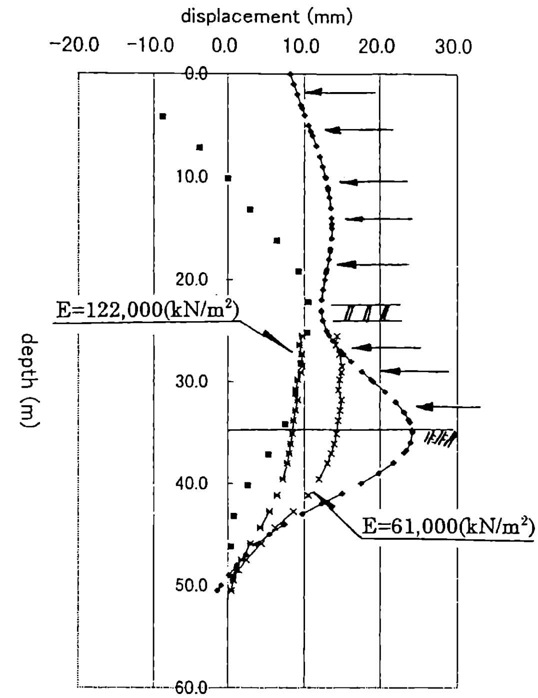


Figure 10. 10th excavation step about the left-soil-part

should be estimated larger in surrounding the rigid body.

7.CONCLUSION

Now on the design of the deep excavation work, the model of elasto-plastic method is used most generally. But the model doesn't work for double diaphragm walls such as this case. In this paper we discussed about only the displacement using the simplest finite element method model. But finite element method model can describe well cases such as this, better than models of elasto-plastic method.

Now the construction costs are showing a tendency to reduce increasingly so the composite underground structure will be constructed more and more. And the excavation soil's volume will be reduced at the trend. Therefore double diaphragm walls such as this case will increase in the future.