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Geotechnical aspects of the construction of Rinkai Higashi-Shinagawa tunnels in Tokyo

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ABSTRACT: Rinkai Line Higashi-Shinagawa tunnels have been constructed by slurry shield tunnelling method. Detailed in-situ monitoring of surface and subsurface ground movements was carried out to investigate the influence of adjacent structures by shield tunnel excavation. As a result, the ground movements were well controlled, leading to less than 5 mm. And there was no visible damage for the important existing nearby structures. This paper presents an outline of the shield tunnel construction and the monitoring results, especially focusing on the ground movements during shield construction.

1 INTRODUCTION

Rinkai Line is a new railway line of around 12.3 km length, connecting Shin-Kiba Station with Osaki Station through the new Tokyo subcentre on the waterfront (see Figure 1). On March 1996, a section of about 4.9 km between Shin-Kiba Station and Tokyo Teleport Station was put into service in the first phase. As part of the second-phase construction, Rinkai Line Higashi-Shinagawa Tunnels have been constructed by slurry shield tunnelling method. This tunnel project stretches about 983 m from Tennozu Isle Station to Shinagawa Seaside Station.

Main characteristics of this project were that the parallel twin tunnels had very close clearance (the minimum clearance=0.5m) and were located close to some important existing structures such as abutments of bridge and an oil tank. Therefore, careful operation and management of the tunnelling works were carried out. In order to prevent excessive settlements and to ensure the safety of the adjacent existing structures, detailed in-situ monitoring of the structures, surface and subsurface ground movements was planned and implemented. An automatic measuring system was adopted as measure of controlling ground movements, and the optimum control value was established for soil pressure inside the shield chamber for stabilizing tunnel face. Extensometers and inclinometers were adopted for the monitoring of subsurface movements. In addition to these measurements, deformation and section force of segments were measured at very closely excavated sections.

This paper describes an outline of the shield tunnel construction and the results of the ground movement monitoring during the construction period to control. The contractor for the main works was a joint venture of three companies, Nishimatsu Construction Co., Ltd., Mitsui Construction and Aoki Construction.

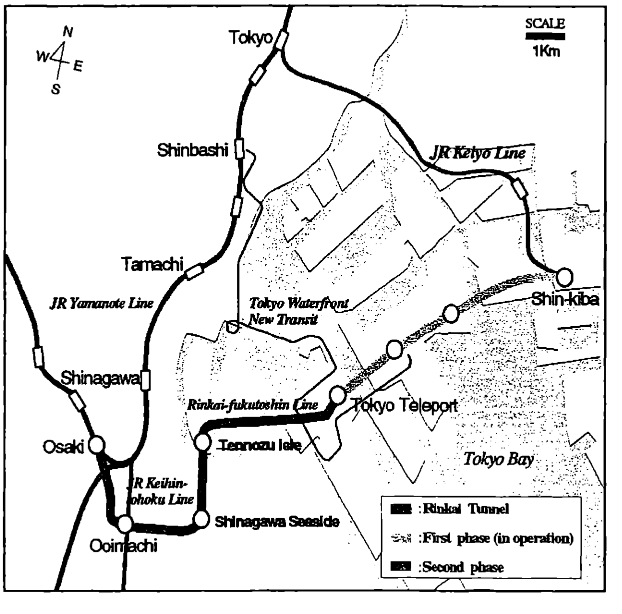
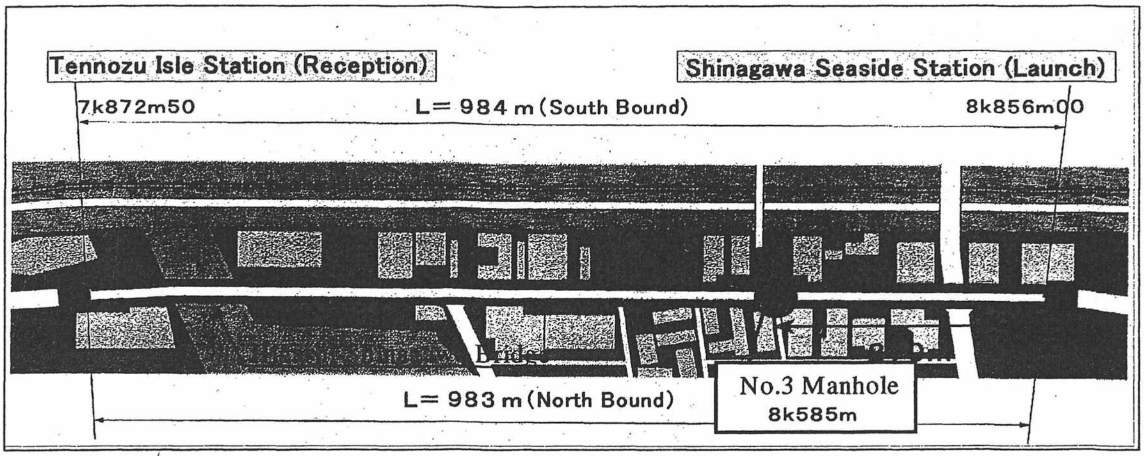
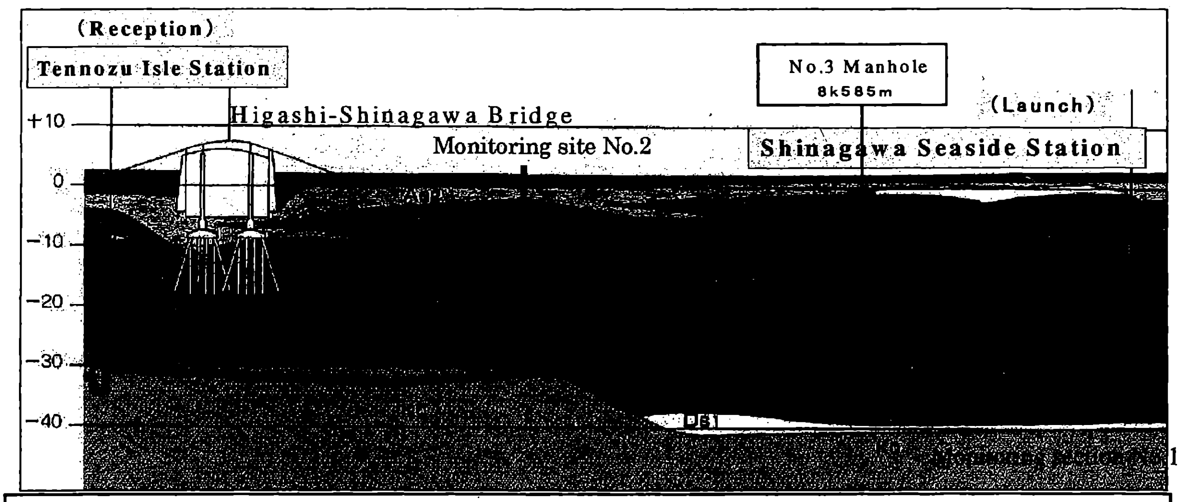


Figure 1 Route of Rinkai Higashi-Shinagawa tunnels



(a) Tunnel alignment



Alluvial layers:As2: Sand layer Ac1: Clay layer Ac2:Clay layer Ag2:Gravel layer Ag1:Gravel layer
Diluvial layers Ds3:Gravel layer Dg1:Gravel layer Dg2:Tokyo-gravel layer Ds2:Sand layer Ko:Clay rock

(b) Schematic geological section

Figure 2 Tunnel alignment and schematic geological section along the Rinkai Higashi-Shinagawa tunnel route

2 GROUND CONDITION

The ground conditions along the tunnel route are summarized in Figure 2. The soil conditions in the tunnel section are roughly divided into two types: alluvium and diluvium. For the first 300 m from the launching shaft, the tunnelling machine passed through very soft clay layer (Ac1), which had N-value less than 4. After that, the tunnels were mostly driven through stiff sand layer (Ds2) (N-value=35～50). The tunnel site generally consisted of various kinds of soil layers from very soft to hard clays.

3 TUNNEL CONSTRUCTION

3.1 Summary of Tunnel Construction

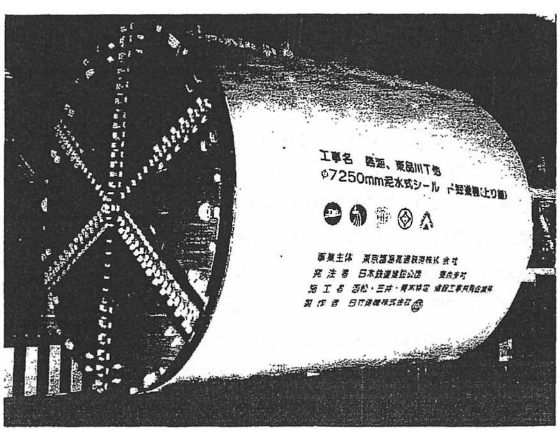


Figure 3 General view of Slurry shield machine

The slurry shield method was employed to maintain face stability and minimize ground movements. Two slurry shield machines with an outer diameter

(D) of 7.25 m and a length (L) of 8.36 m were employed to excavate the two parallel single tunnels. Figure 3 shows a general view of the shield machine for the south bound rout in this project. Of the two tunnels, the south bound route began to construct first. After 100 rings excavation of the south bound route, the construction of the north bound route had started. The cover-to-diameter ratio (C/D, C being the depth to tunnel crown) ranged from 3 to 4. An excavation speed of the tunnelling machine averaged 11 m/day.

Simultaneous grouting method was employed. The grouting unit features two injection pipes so as to ensure filling the tail void at low injection pressure because of preventing any damage from the adjacent structures. The unit is also equipped with simultaneous freezing system capable of reducing the cleaning work of the injection pipes.

3.2 Slurry control

Table 1 shows standard control values of slurry mud for each excavation layer. During shield construction, a full-time engineer for slurry control was selected. The engineer controlled the slurry mud at real time and was swiftly restored in case of unusual situations. Theoretical upper and lower limit values of face pressure were carefully set in advance based on the past performances and geological surveys. These values are reexamined in line with actual in-situ conditions during the shield construction.

Table 1 Standard control values of slurry mud

Item	Standard Control Value	
	Diluvium	Alluvium
Density	1.15~1.25	1.20~1.30
Viscosity	20~36 sec	
Filtration	<35 ml	
Bleeding	<5 %	
Sand Content	<10 %	<5 %
PH	7.0~11.5	

4 INSTRUMENTATION

In order to obtain precise and reliable experimental data, two sites were selected to monitor the ground response due to the shield excavation (see Figure 2(b)). The first site (No.1) is the near the starting shaft and the second one (No.2) is at the point before passing beneath the Higashi-Shinagawa bridge. Locations of extensometer to monitor subsurface movements at No.2 section are presented in Figure 4. Control values, including

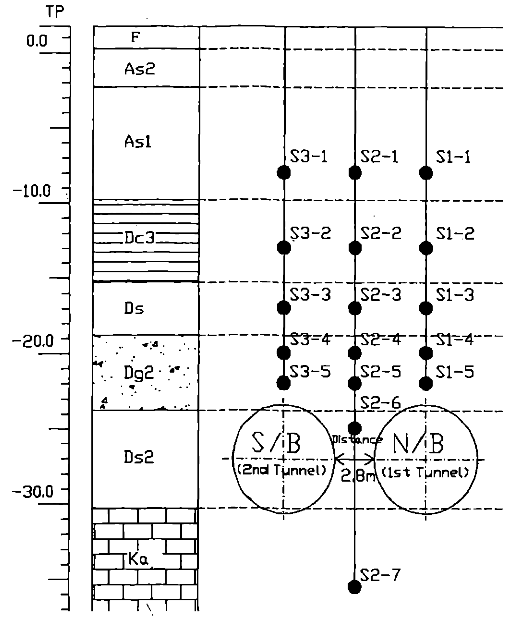


Figure 4 Ground instrumentation (Monitoring site No.2)

Table 2 Monitoring results of ground surface settlement

Monitoring Point		Smax(mm)	
		MV	CV
Higashi-Shinagawa Bridge		4.7	9.0
Tennozu Embankment	Right	1.6	2.2
	Left	1.5	9.0
Aqueduct		2.4	6.0
No.3 Tunnel		2.9	7.0

*MV:Measured value, CV:Control value

soil pressure inside the chamber were optimized so as to make the displacement the smallest by trail and error.

5 MONITORING RESULTS AND DISCUSSIONS

Table 2 shows the monitoring results of the existing structures near the Higashi-Shinagawa bridge. The values of Smax (maximum surface settlement) were well controlled, showing the values much less than the control values. The values of Smax at the southbound route were less than 5 mm. No major damage to the structures was also observed during visual inspection.

Figure 5 shows the development of surface settlement above the centreline of the tunnel during the passage of the tunnel machine. The value of Smax at surface level was 2.5 mm and very small settlements at the tunnel face were observed

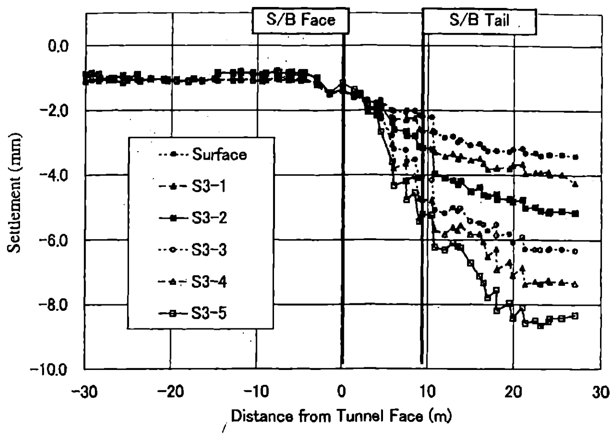


Figure 5 Longitudinal ground surface settlement (Monitoring point B)

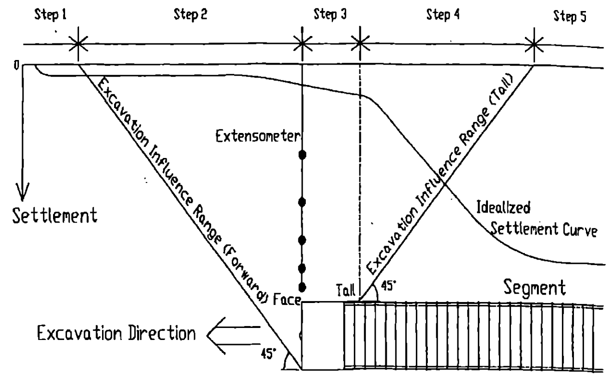


Figure 7 Idealized stages of settlement with respect to time (after Sugiyama, et al.(1999))

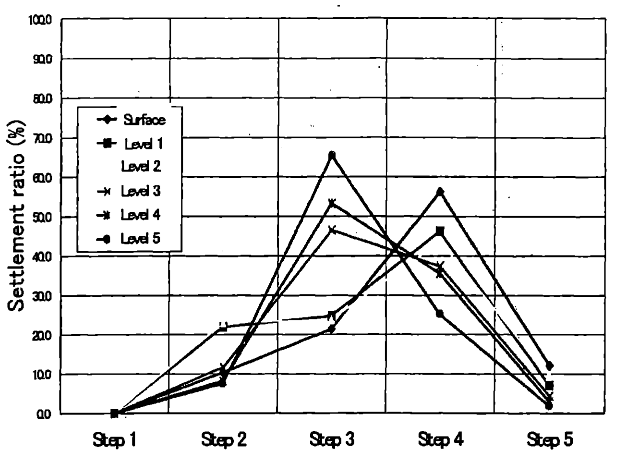


Figure 6 Settlement ratio based on construction process (south bound S3)

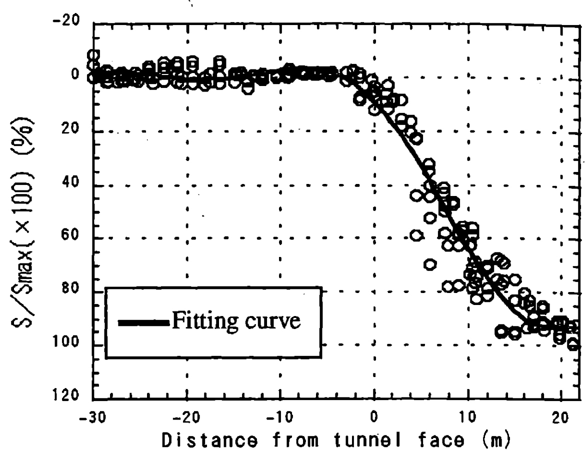


Figure 8 Relationship between surface and sub-surface settlement and tunnel progress

irrespective of the ground level from the surface. One of the reasons for this is believed to be due to well and careful face control. The values of sub-surface total settlement more increase as the distance between the tunnel crown and the monitoring point becomes larger. An increasing ratio of the settlement during passage of shield was also larger at sub-surface points.

Figure 6 shows a comparison of settlement ratio by the depth for the south bound (S3) as shown in Figure 4. Step 1 to Step 5 in Figure 6 show five types of ground displacements by the shield tunneling, as depicted in Figure 7. They are as follows:

Step 1 Preceding settlement
Step 2 Deformation of ground at the front of the face
Step 3 Settlement during passage of the shield
Step 4 Settlement due to the tail void
Step 5 Succeeding settlement

Settlement ratio of Step 2 is very small. This trend means that controlling of slurry pressure at the tunnel face was appropriate. Consolidation settlement (Step 5) was also few because the tunnel drove through the diluvial sand layer at this region. Combined settlement ratio of Step 3 and Step 4 are around 80 %. This tendency is well consistent with the monitoring results based on recent mechanized closed face shield (Mair(1996), Mair and Taylor(1997) and Sugiyama et al.(1999)). Further research to reduce settlement during the shield passage is needed.

Figure 8 shows the relationship between the

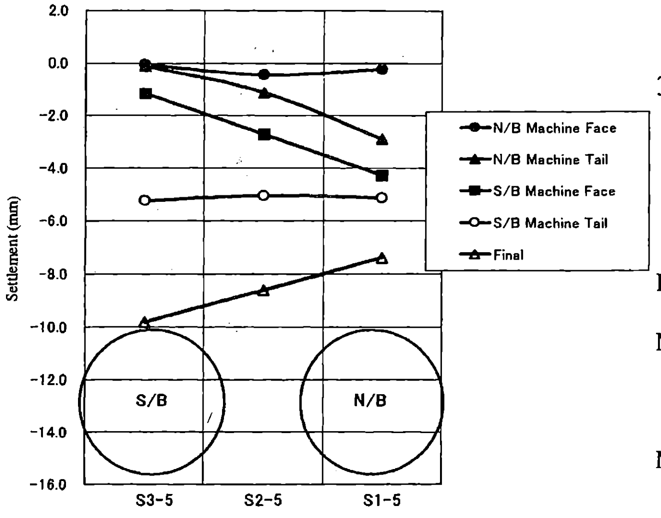


Figure 9 Measurement of ground surface and sub-surface settlements

normalized monitoring settlement of track and the distance from the cutting face to the observational points of surface and sub-surface. The longitudinal surface and sub-surface settlement curves were able to be fitted as a single fitting curve. About 10 % of the total settlement along railway track was found to occur just above the cutting face.

Figure 9 presents ground movements just above the tunnel crown associated with the tunnel progress. At this monitoring point, the clearance between the southbound tunnel and the northbound tunnel is about 2.8 m. The maximum total settlement after passage of the northbound tunnel (Final settlements) was 9.8 mm just above the northbound tunnel, not above the centerline between the southbound tunnel and the north bound tunnel. These final settlements were less than predicted.

6 CONCLUDING REMARKS

From the in-situ monitoring results of the ground movements in the slurry shield tunneling in soft clay, the following conclusions were deduced.

1. It has been shown that, by careful and well controlled tunnel excavation, detailed monitoring of the ground movements and rapid data processing, it is possible to achieve well control ground movements to values less than 5 mm.

2. Both the settlements above the tunnel face and the settlement due to the tail boil were significantly reduced and the ratio of ground movements during the shield passage was the main part of the longitudinal settlement.

3. About 10 % of the total longitudinal settlement at surface and sub-surface positions was found to occur just above the tunnel face and the longitudinal surface and sub-surface settlement curves were shown as a single fitting curve.

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