This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 9th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground (IS - Sao Paulo 2017) and was edited by Arsenio Negro and Marlisio O. Cecilio Jr. The conference was held in Sao Paulo, Brazil in April 2017.
Large-scale improvement of converting a Japan’s first subway station constructed by the shield tunneling method into that of an open-cut method: Kiba Station on the Tozai Line

K. Hiromoto, H. Hashiguchi & Y. Arai
Renovation and Construction Department, Tokyo Metro Co. Ltd., Tokyo, Japan

ABSTRACT: Tokyo Metro has constructed subway lines since the 1950s to develop a subway network in the central part of the Tokyo Metropolitan Area. At present, it operates a 200-km network of nine subway lines, which was, for the most part, completed when the entire Fukutoshin Line was opened for service in 2008. Tokyo Metro is now working to enhance safety and raise customer satisfaction by upgrading the quality of service. This paper reports on the contents and the construction methods used to achieve a seamless flow of passengers at Kiba Station on the Tozai Line. This is an example of the improvements being made in the existing subway network to help establish an affluent society in the future.

1 INTRODUCTION

One of the main subway lines among Tokyo’s subway networks, Tokyo Metro’s Tozai Line runs through the center of the metropolitan area, from Nakano Station in the western part of Tokyo to Nishi-funabashi Station in Chiba Prefecture. Opened for service as a 30.8 km trunk line in 1969 (Figure 1).

Ridership during the morning rush hours has risen 200% or more, resulting in chronic train delays. Moreover, passengers on the platform are exposed to increased risk of contact with the cars or falling. Tokyo Metro is addressing this issue by ensuring safety and thoroughly improving service on the Tozai Line. This project includes improvements being made to the Kiba Station, a shield station. When building the Tozai Line, a 1.8-km section from the end of Monzen-nakacho Station to the

Figure 1. Outline of Tozai Line.

(a) Plan view of above-ground and 1st basement level

(b) Longitudinal sectional view

Figure 2. Above-ground, 1st basement level and longitudinal sectional view (existing).
leading end of Toyocho Station was constructed by a shield machine because of soft alluvial clayey soil layer at a depth of about 30–40 meters. Kiba Station is a shield station (inside dia.: 7,240 mm, outside dia.: 7,740 mm, segment width: 800 mm, ductile segments) constructed at the midpoint of this section. At both ends of the station, a shaft was constructed by immersing two under-road caissons (one at each end). The shaft is of a four-level structure (Figure 2).

This paper reports on the contents and the construction methods used to achieve a seamless flow of passengers at Kiba Station on the Tozai Line.

2 CURRENT ISSUES

Since opening for service in 1967, Kiba Station has added ticket gates and made other development efforts to establish one barrier-free route. In recent years, however, the number of passengers using the west ticket gates has become approximately twice as great as the number using the east ticket gates. This is due to ongoing development around the station, including construction of large commercial establishments to the southwest. During the morning rush hours, alighting passengers outnumber boarding passengers in both directions. In addition, two problems discussed below are faced due to the station being a shield station.

2.1 Narrow platform (intricate line of flow)

In the Kiba Station, 187 m of the platform length of 220 m becomes a single-track shield portion. The platform width is 3.0 m on each side.

As there is no passage in the intermediate portion, which connects both platforms, the lines of flow to respective platforms become intricate at both station ends. The platform is so narrow that passenger retention space becomes very small. As alighting passengers are large in number, a long queuing line forms on the platform, which becomes tangled with the boarding passengers.

2.2 Chronic congestion on stairs

The station's stairs and escalators are located only in the shaft portion at both ends of the station, where passengers become concentrated. The long lines that form on the platform up to the center of the station during the morning rush hours do not shrink as trains arrive. Moreover, the stairs and escalators are constructed in such a manner that they double back at each floor, which lengthens movement time. The large numbers of alighting passengers add to the congestion.

3 REVIEW OF THE IMPROVEMENT PROJECT

3.1 Station improvement

A review of the improvement project has focused on enhancing passenger safety by widening the platform, mitigating congestion, and improving convenience by adding elevators and escalators from the platform to the ticket gate.

3.1.1 Widening the platform

The platforms at Kiba Station were constructed in single-track shield tunnels. To widen them, it is necessary to remove the existing shield segments and build a frame body between them.

However, the existing shield was not built to allow later removal, and the segments are ductile segments, making reinforcement difficult. It is therefore difficult to secure the structural safety of the existing shield. In consequence, it was planned
to build a box culvert around the existing shield and to remove the segments later (Figure 3).

3.1.2 Addition of escalators

Widening the platform alone cannot eliminate the long lines. It is also necessary to add escalators from the platform floor to the ticket gate floor. A decision was made to add escalators after conducting a flow simulation of morning rush hours in which escalators are added as variable parameters (Figure 4).

Figure 4 (b) shows a simulation of the flow during morning rush hours when one escalator is added from the platform floor to the ticket gate floor. It is evident that the passenger line on the platform floor would not be eliminated.

Figure 4 (c) shows a simulation of the flow during morning rush hours when two escalators are added in series from the platform floor to the ticket gate floor. Evidently, the queuing line on the platform floor is eliminated. On the basis of this result, it was planned to add two escalators to carry passengers up from the platform floor to the ticket gate floor.

The planned views after improvement based on the above simulation are shown in Figure 5.

3.2 Contractor contents

In principle, improvement work is to be executed while maintaining normal operation of the revenue line. Therefore, plan is to construct the new structure around the existing shield in advance, and to subsequently remove the existing segments. Since the ground around Kiba Station is extremely soft, the work was planned according to inverted lining method based on using diaphragm walls as the tunnel body. This is being done to reduce to a minimum any effects on neighboring buildings during the excavation work. The work sequence for improvement of Kiba Station is as shown in Figure 6.

3.2.1 Protection of the revenue lines

Prior to construction of diaphragm walls and excavation, the frameworks that are manufactured by curving shape steels are arranged at 800 mm pitch inside the shield, and the steel plates are installed between the frameworks. This protection work serves not only to separate the revenue line area from the work area, but it also prevents entry of fumes from the cutting during removal of ductile segments, prevents cut pieces from falling into the track side, prevents entry of leaking water, provides a support frame for replacing rigid conductor equipment (contact wires), and provides a support frame for lighting and cables.

3.2.2 Preceding footing beams

Use of the inverted lining method is planned to minimize displacement associated with excavation. Since the ground concerned is extremely soft, the formation level will be secured for construction of upper slabs from top down by providing preceding footing beams by providing ground improvement under the inverted lining upper slabs, and by providing inverted lining middle slabs for the second basement level and below the lower slabs.

---

Figure 5. Above-ground, 1st basement level and longitudinal sectional view (planned).
3.2.3 Application of steel-made diaphragm wall
Support in the vicinity of the existing shield becomes necessary for the reinforced-concrete diaphragm walls. It was decided to use a steel diaphragm wall due to revenue line safety considerations, the need to eliminate any work in the vicinity of the existing shield, and the need to secure wide space for removal of existing segments. This eliminated the need to provide support in the vicinity of the existing shield.

3.2.4 Excavation under the existing segments
The plan calls for reducing the work period for excavation under the existing segments to construct the lower slabs in order to ensure the safety of the revenue line and to minimize the period with unstable conditions.

a. Decision not to use steel for the underpinning of existing shield
The use of steel was not recommended because the narrow work space would cause prolonged excavation and installation work periods, and because it would make bar arrangement complicated.

b. Underpinning of existing shield by ground improvement
Ground improvement under the existing shield is to be a yard operation that requires drilling through the existing shield. This work will mean that the ground will be improved and will function as preceding footing beams. In addition, ground improvement will serve to support the existing shield before the start of excavation and will contribute to early stabilization of the existing shield.

Although it is ideal to perform ground improvement totally above and below the existing shield, the plan is to arrange improvement at 6.0-meter intervals in a comb-like pattern. This was decided on because excavation of the improved ground would take a great deal of time.

For advance excavation, trench excavation will be done over the range not subject to ground improvement. Lower slabs will be built as joints for distributing bars. Following excavation will consist of trench excavation of the improved ground, and constructing lower slabs (Figure 7).

4 PREDICTIVE ANALYSIS

4.1 Overview of considerations
The construction flow will be carried out as shown in Figure 6. Currently, the removal of buried objects existing at the position close to the ground surface on the left side of Figure 8 and primary earth retaining work by the CSM method for installation of road decking have been completed, preceding the erection of the diaphragm wall of the main structure.

As the primary earth retaining work involves the excavation of a 60 m extension to a trench (0.8 m
331 wide, 43 m deep, and 3.8 m long) in close proximity to the existing tunnel in soft earth, appropriate risk management is necessary. Thus, predictive analysis was carried out on the Cross-section of the proposal of Figure 8 to assess the effects of the trench excavation on the existing tunnel.

Specifically, as shown in Figure 8, as the distance between the slurry wall and the shield tunnel of the operating line will be 2880 mm at its closest point, attention was paid to the horizontal behavior for the transverse direction of the tunnel during trench excavation. The analysis was carried out using the elastic-perfectly plastic 3D FDM method of the Mohr-Coulomb model (analysis code: FLAC).

4.1.1 Study conditions
The ground conditions are shown in Table 1. Each of the values set are based on the result of on-site geological survey.

Construction conditions and structural conditions are shown in Tables 2 and 3.

![Figure 8. Cross section of the proposal.](image)

### Table 1. Ground conditions.

<table>
<thead>
<tr>
<th>Stratum type</th>
<th>Elevation (m)</th>
<th>N-value (N&lt;sub&gt;SPT&lt;/sub&gt;)</th>
<th>γ (kN/m&lt;sup&gt;3&lt;/sup&gt;)</th>
<th>E (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>ν</th>
<th>c (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>φ (deg)</th>
<th>K&lt;sub&gt;0&lt;/sub&gt;</th>
<th>case 1</th>
<th>case 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>TP+0.32</td>
<td>–</td>
<td>18.0</td>
<td>1.0</td>
<td>0.40</td>
<td>–</td>
<td>0.0</td>
<td>0.67</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>Yuc</td>
<td>TP-1.28</td>
<td>0.3</td>
<td>18.0</td>
<td>2.7</td>
<td>0.45</td>
<td>1.8</td>
<td>0.0</td>
<td>0.82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yus</td>
<td>TP-4.48</td>
<td>2.0</td>
<td>19.0</td>
<td>1.4</td>
<td>0.40</td>
<td>0.0</td>
<td>28.0</td>
<td>0.67</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ylc-1</td>
<td>TP-6.35</td>
<td>0.0</td>
<td>16.2</td>
<td>2.0</td>
<td>0.45</td>
<td>22.4-60.9</td>
<td>0.0</td>
<td>0.82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ylc-2</td>
<td>TP-15.18</td>
<td>2.0</td>
<td>17.5</td>
<td>5.4</td>
<td>0.45</td>
<td>90.6</td>
<td>0.0</td>
<td>0.82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nac</td>
<td>TP-25.48</td>
<td>4.4</td>
<td>16.7</td>
<td>16.6</td>
<td>0.45</td>
<td>111.5</td>
<td>0.0</td>
<td>0.82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Btg</td>
<td>TP-31.88</td>
<td>47.9</td>
<td>20.0</td>
<td>33.5</td>
<td>0.30</td>
<td>0.0</td>
<td>42.0</td>
<td>0.43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Toc</td>
<td>TP-40.38</td>
<td>23.3</td>
<td>18.0</td>
<td>16.3</td>
<td>0.35</td>
<td>155.0</td>
<td>0.0</td>
<td>0.54</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tog</td>
<td>TP-47.86</td>
<td>112.0</td>
<td>20.0</td>
<td>78.0</td>
<td>0.30</td>
<td>0.0</td>
<td>42.0</td>
<td>0.43</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 2. Conditions of primary earth retaining work (CSM).

<table>
<thead>
<tr>
<th>Excavation dimensions</th>
<th>2.4 m long × 0.8 m wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance between outer surface of tunnel wall and slurry wall</td>
<td>2880 mm (A Line)</td>
</tr>
<tr>
<td>Mud level</td>
<td>GL-0.75 m</td>
</tr>
<tr>
<td>Specific gravity of mud</td>
<td>1.5</td>
</tr>
<tr>
<td>Surface of construction surface</td>
<td>GL ± 0.0 m</td>
</tr>
<tr>
<td>Construction conditions</td>
<td>At the time of slurry wall trench excavation (before construction)</td>
</tr>
</tbody>
</table>

### Table 3. Structure of shield tunnel.

<table>
<thead>
<tr>
<th>Material</th>
<th>Ductile cast iron steel (FCD450)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment outer diameter</td>
<td>7740 mm</td>
</tr>
<tr>
<td>Segment girder height</td>
<td>250 mm</td>
</tr>
<tr>
<td>Segment width</td>
<td>800 mm</td>
</tr>
<tr>
<td>Segment joint bolt</td>
<td>φ35 (JIS B1186 (B type))-5 bolts/1 piece</td>
</tr>
<tr>
<td>Ring joint bolt</td>
<td>φ28 (SS400)-57 bolts</td>
</tr>
<tr>
<td>Skin plate thickness</td>
<td>9 mm</td>
</tr>
<tr>
<td>Main girder thickness</td>
<td>10 mm</td>
</tr>
<tr>
<td>Vertical rib thickness</td>
<td>15 mm</td>
</tr>
<tr>
<td>Plate thickness at joint between segments</td>
<td>12 mm</td>
</tr>
<tr>
<td>Plate thickness at joint between rings</td>
<td>10 mm</td>
</tr>
</tbody>
</table>

331
4.1.2 Examination technique

The analysis model is a simulation using the solid elements of the ground and the shell elements of the tunnel. The analysis area is taken as the depth of the Tokyo Gravel Strata (Tog), which is approximately 4 times the excavation depth. The lower boundaries of the model were made with fixed vertical $z$ and variable horizontal $x$. The side boundaries were made with fixed horizontal $x$ and variable vertical $z$, as shown in Figure 9.

The shield tunnel model represents longitudinal stiffness as follows: cylindrical shell elements with axial bending equivalent with that of a $\varphi 7.74$ m cylindrical tube equipped with earthquake resistant ring joints, and with a thickness equivalent to that incorporates that axial bending.

Also, since the vertical bending stiffness is sufficiently large compared with longitudinal stiffness, the vertical deformation of the tunnel is not taken into consideration.

The analysis includes that the rings used are made with girders that have large vertical stiffness.

The analytical steps are shown in Figure 10. Analysis was performed on the three steps of STEP 1 to STEP 3.

Step 1 takes initial stress and analyzes weight before introducing the planned shield tunnel, and evaluates the loosening of the ground during the construction of the tunnel. In Step 2, the construction of the shield tunnel has been simulated and its tunnel displacement is included as initial values. In Step 3, the excavation work for the primary earth retaining in the slurry wall is simulated to find the behavior of the existing tunnel with the trench filled with muddy water.

In Step 3, the destressing created by the primary earth retaining in the slurry wall is assumed to be the lateral pressure equivalent, but in order to grasp the effect on the tunnel from the difference of the coefficient of the lateral pressure, they were calculated in two different ways. In Case 1, calculation is made according to the elasticity theory of Poisson’s ratio $\nu$ (set value). In Case 2, the $N$ value in the tunnel plans is between 4 and 8 for the coefficient of lateral earth pressure 0.6 of cohesive soil, which is quoted from the “Standard Specifications for Tunneling”.

Using the above calculations, the load distribution (destressing minus mud water pressure) applied to the slurry wall is shown in Figure 11.

4.2 Study results

The horizontal displacement distribution due to the difference in lateral pressure is shown in...
Figure 11. Distribution of loads on the slurry wall.

Figure 12. Water displacement around tunnel.

Figure 12. Also, the occurrence of the area of plasticity of the ground mentioned in Case 1 is shown in Figure 13.

The reason that the displacement became very small in Case 1 is considered to be because the destressing and the hydraulic pressure of the mud acting on the slurry wall are nearly identical.

In Case 2, the hydraulic pressure of the mud is greater than the destressing so, in the analysis, the tunnel moves away from the slurry wall.

5 CONSTRUCTION RESULTS

As one of the construction results of the primary earth retaining work, analysis target, the calculations of the distance from the tracks and from the track center to the platform edge before/after
the primary earth retaining work is shown in Figure 14. Figure 14 (a) shows the horizontal displacement of the tracks, and as Figure 14 (b) shows that the change in distance from the track center to the platform edge is almost nothing, it is determined that the tunnel displacement is miniscule. The results of this construction support that the assumptions of Case 1, which were made prior to construction, that there is a very small effect on the conditions of the tunnel were valid.

6 CONCLUSION

This construction work currently in progress to mitigate the congestion stemming from the increasing demand in this shield station. Also, this construction method is considered to be applicable when constructing new stations or branch vertical shafts in an existing shield tunnel section. Here at Tokyo Metro, to improve transportation safety and reliability, and customer service, we are implementing eagerly large-scale projects in addition to the one mentioned in this paper. We hope to realize quick and high quality renovation works capitalizing on the experience and techniques gained in this project and others, such as technology developed in building new lines and renovating existing ones.

Finally, we hope that this paper may serve as a reference in the future for large-scale renovation of other underground structures.

REFERENCES

