Settlement behaviour of a shield tunnel constructed in subsiding reclaimed area

K. Komiya
Chiba Institute of Technology, Chiba, Japan

K. Takiyama
Japan Railway Construction, Transport and Technology Agency, Tokyo, Japan

H. Akagi
Waseda University, Tokyo, Japan

ABSTRACT: This paper focuses on a large settlement behaviour of a shield tunnel, which is constructed in soft alluvial clay strata beneath fresh reclaimed land. The total settlement of the tunnel is more than 0.73 metres and the deformation takes place over more than twenty years after the completion of the tunnel construction. Three dimensional finite element analyses have been carried out to estimate a long-term behaviour of soil and tunnel structure observed in this field. The computed vertical settlement of tunnel structure is compared with the measured data.

1 INTRODUCTION

The Daiba Tunnel, two tubes of 7.5 metres outer diameter and 1439.9 metres long tunnel, was planed as a section of the Keiyo Passenger and Freight Train Line (KTL) Project. The KTL Project was planed by the Japanese government as a new maintenance facility to reduce traffic congestion in Tokyo. The KTL Project was total of about 105 km railway and associated appurtenances construction project in Tokyo bay area. The Japan National Railway (JNR) Corporation put the project into action in September 1972 and the Japan Railway Construction Public Corporation (JRCC), which was the contract constructor of the Daiba Tunnel, started the work on the tunnel in August 1977. In March 1980, the Daiba Tunnel was completely constructed and the JRCC started to monitor the settlement of Eastbound tunnel structure.

In 1986, the government-owned JNR Corporation was privatized. As the process of privatization of the JNR Corporation went forward, the KTL Project had undergone a major overhaul to meet the unique needs of their customers. In 1983, the KTL Project was redesigned expressly for passenger train line and the JNR Corporation suspended the construction of the KTL's dedicated freight train line section between Shinkiba station and Tokyo Kamotsu Terminal station. At that time, the tunnel of 3538.1 metres including the Daiba Tunnel had been completely constructed in this section. The Daiba Tunnel have been left vacant from 1980 for about seventeen years until it repaired in 1997 to use a section of the following new Rinkai Rapid Passenger Train Line (RTL) Project. During the seventeen years, the Daiba Tunnel sunk with a maximum total settlement of more than 0.73 metres caused by consolidation of ground.

In March 1992, the construction of RTL Project started and the KTL Project's abandoned tunnels including the Daiba Tunnel would be reused by this project. In order to make the feasibility of reusing the Daiba Tunnel, it was necessary to estimate future settlement behaviour of ground and the tunnel structure considering events or anomalies occurring at the uncontrolled tunnel structure after the completion of the tunnel construction.

An attempt was made in this study to assess the ability of the three dimensional finite element method to model the long-term behaviour of soil and the Daiba Tunnel observed in this field.

2 FIELD INVESTIGATION OF SETTLEMENT BEHAVIOUR OF THE DAIBA TUNNEL

2.1 *The Daiba Tunnel*

The Daiba Tunnel link between Tokyo Teleport Station and Tenno-zu Station of RTL. The layout of the Daiba Tunnel is shown in plan on Figure 1(a) and longitudinal section on Figure 1(b).

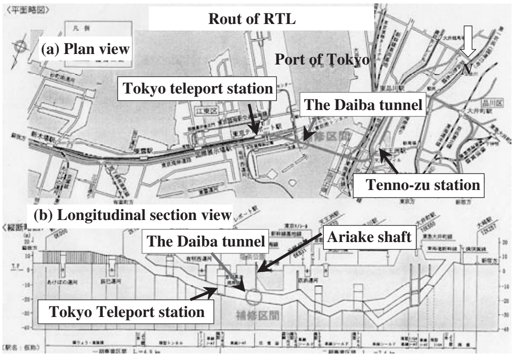

Figure 1. The layout of the Daiba Tunnel.

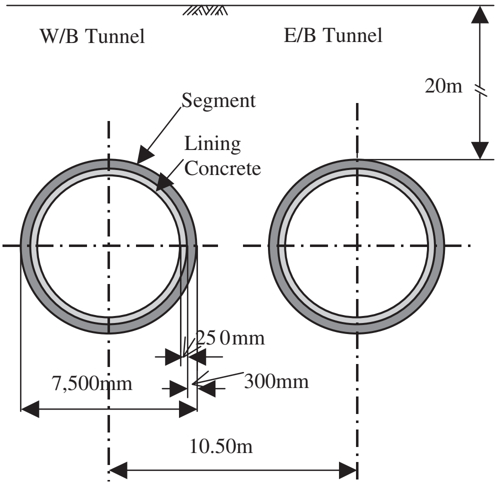

Figure 2. Cross section of the Daiba Tunnel.

Construction of the Daiba Tunnel started from Ariake Shaft using the machine of shield type with slurry pressure balance to stabilise the earth pressure in front of the shield machine. The Ariake Shaft would be rebuilt as a ventilator box of the tunnel after the completion of the tunnel construction.

Figure 2 shows the cross sectional view of the two tubes of Daiba Tunnel. The minimum distance between the two tracks is as small as 3.0 meters at the section where the tunnels cross this study's target area.

2.2 *Soil profile around the Daiba Tunnel*

The soil profile and the location of the tunnel are shown in Figure 3. The Daiba Tunnel was constructed in alluvial cohesive soil deposits layer (Ac1 and Ac2) which is known as Yurakucho Clay layer. The maximum thickness of the Yurakucho Clay layer was approximately 40 metres in the area, and the SPT N value is reported to be close to zero throughout the layer. This layer has originally been located under the seabed in Tokyo Bay.

The shallow sea of about 3 metres depth was filled with construction waste etc from 1962 to 1973, and the reclaimed land was built over the Yurakucho Clay layer. The final thickness of reclaimed embankment is about 10 metres.

Firm diluvium sand and gravel layer, which are known as the Edogawa Sand Layer and Tokyo Gravel Layer, underlies the alluvial Yurakucho Clay layer. These diluvium layer forms tableland of about 15 metres height in the Tokyo Teleport station side, and the tableland is to be thinning the thickness of the Yurakucho Clay layer 25 metres lying over the tableland.

2.3 *Settlement behaviour of ground and the Daiba Tunnel*

The long term settlement due to consolidation of the alluvial cohesive deposits layer (Ac1 and Ac2) occurred by loading from the reclamation of embankment fill. The settlement in this type of ground tended to take place more than several years after the construction. Hence the estimating the deformation of Daiba Tunnel due to large ground settlement and the reinforcement of the lining to resist the tunnel deformation were a major technical challenge to tunnel engineers.

The Japan Railway Construction Public Corporation (JRCC) challenged to calculate ground settlement at Daiba Tunnel using Terzaghi's consolidation theory. This calculation was performed by considering only one dimensional behaviour which was related to the thickness of the alluvial cohesive deposits at some sections of the Daiba Tunnel. They also have monitored the settlement in the field from 1980 to 1997. The measured settlement of the Daiba (Eastbound) Tunnel with time at the monitoring location where the section is 70 metres away from Ariake Shaft is shown in Figure 4. The measured settlement is compared to Terzaghi's consolidation solution as shown in Figure 4. The magnitude of the settlement measured at monitoring location was larger than the settlement which obtained by Terzaghi's consolidation solution.

Figure 5 shows the measured final longitude settlement of the Daiba (Eastbound) Tunnel. It is also compared to Terzaghi's consolidation solution as shown in Figure 5. The maximum measured settlement was about 0.73 metres at the halfway point between Ariake shaft and the diluvium tableland. The transverse measured settlement trough was well-described by the valley curve distribution. On the other hand, the longitude trough of Terzaghi's consolidation solution was similar to the boundary of the underlying diluvium bed because the main parameter controlling the solution was the thickness of the alluvial cohesive deposits over

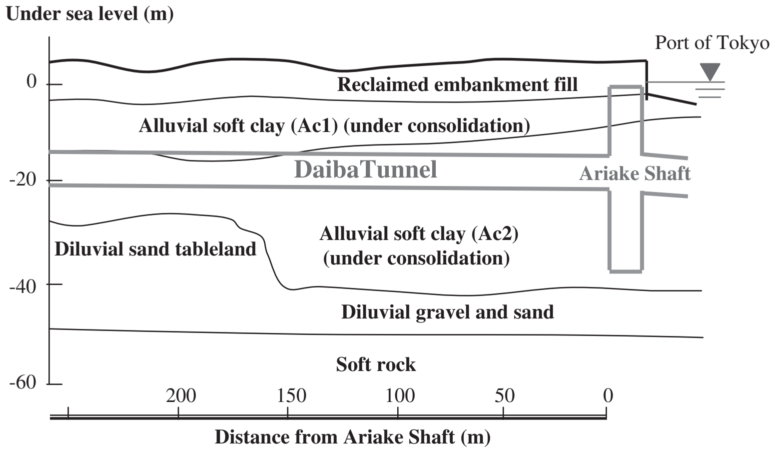

Figure 3. Soil profile and location of the tunnel.

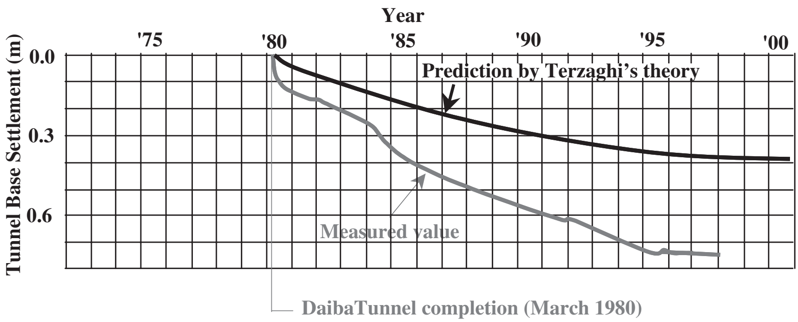

Figure 4. Settlement at Daiba Eastbound Tunnel base where the section is 70 metres away from Ariake Shaft.

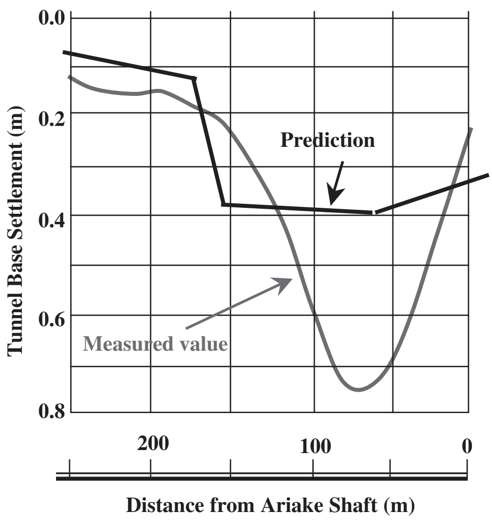

Figure 5. Longitudinal displacement of tunnel.

the diluvium bed. The maximum calculated settlement is much smaller than the measured one.

3 THREE DIMENSIONAL FINITE ELEMENT ANALYSIS OF SETTLEMENT BEHAVIOUR OF GROUND AND THE DAIBA TUNNEL

Many factors of the settlement behaviour of Daiba Tunnel constructed in the subsiding reclaimed area are closely linked to the interaction between the soil and the tunnel. The nature of this problem is three dimensional and the magnitude of settlement of the ground and the tunnel are related to characteristics, three dimensional boundary condition of the soil and the tunnel structures. Because of the complex boundary conditions of this problem, the use of the finite element method is one of the popular methods to investigate the deformation behaviour of the soil and the tunnel.

The long term ground subsidence around tunnel is directly related to the excess pore pressures generated during the reclamation of embankment fill. Therefore, coupled soil-pore water analysis is necessary to assess the time-dependent deformation of the soil and the tunnel.

With above considerations in mind, a three dimensional coupled soil-pore water analysis was conducted to simulate the settlement behaviour of the cohesive soil ground and the Daiba Tunnel. The three dimensional finite element used in this study is the eight-nodded trilinear solid element.

3.1 *Finite element modelling*

Cam-Clay model (Schofield, A.N. & Wroth, C.P. 1968) was used to model the stress-strain behaviour of the alluvial cohesive soil deposits. The input parameters used in the analysis are listed in Table 1. Most of the input parameters were determined from the results provided by standard geotechnical tests on samples obtained at various depths. Other input parameters, which were not able to be determined from these tests, were calculated by the method suggested by Iizuka et al., (Iizuka et al. 1985).

The initial distribution of the preconsolidation pressure was obtained based on the results of standard oedometer consolidation tests. The initial distribution of the water content was obtained from the samples taken, and the unit weight of the soil was estimated by assuming that the soil is fully saturated. The ground water table was assumed to be at the ground surface and initially hydrostatic. The coefficient of lateral earth pressure was assumed to be 0.55 corresponding to $Kn = 1 - \sin\phi'$.

Table 1. Input Parameters used in the FEM Analysis.

Embankment fill	$E = 5000.0\ kN/m^2$
	$\nu = 0.38$
	$kv = 1.5 \times 10^{-8}\ m/s$
	$\gamma t = 1.6\ kN/m^3$
Clay Ac1	$\lambda = 0.191$
	$\kappa = 0.017$
	$kv = 2.0 \times 10^{-9}\ m/s$
	$e_0 = 1.25$
	$M = 1.20$
	$\nu = 0.333$
Clay Ac2 (upper layer)	$\lambda = 0.456$
	$\kappa = 0.052$
	$kv = 1.3 \times 10^{-9}\ m/s$
	$e_0 = 2.48$
	$M = 1.20$
	$\nu = 0.333$
Clay Ac2 (bottom layer)	$\lambda = 0.404$
	$\kappa = 0.048$
	$kv = 1.0 \times 10^{-9}\ m/s$
	$e_0 = 2.16$
	$M = 1.20$
	$\nu = 0.333$
Tunnel structure	$E = 4.74 \times 10^5\ kN/m^2$
	$\nu = 0.300$

The coefficient of permeability at various depths was estimated by the results of standard oedometer consolidation tests. Values of coefficient of horizontal permeability were not measured in the field. They were assumed to be ten times those of vertical permeability, as it is common that the horizontal permeability is larger than the vertical.

In this analysis, the reclaimed embankment fill was modelled as an isotropic elastic body. It has been recognized that the long-term subsidence after the reclaiming is caused by excess pore pressures in the alluvium cohesive soil deposits induced by weight loading of the embankment fill. Providing body forces inside the embankment fill elements simulated the initial vertical load applied to the alluvial cohesive soil deposits.

The Daiba Tunnel structure, two tunnels of 7.5 metres outer diameter, was modelled using three dimensional isotropic elastic solid elements. The Elastic modulus of the tunnel structure was known by the JRCC design code.

In order to be deformed with ground settlement, the Daiba Tunnel was provided with expansion joints as shown in Figure 6. The elastic rubber of the joint is flexible enough to endure elongation of 54 mm. However, in 1984, the loads of some joints were increased due to the settlement of the tunnel structure and the elastic rubbers of the joints extended more than the acceptable 54 mm.

As a consequence, some rubbers broke and ground water drained into the tunnel through some joints from 1984. Since reinforcement of the existing Daiba Tunnel should have to be done for the reuse in the KTL Project, these breakages were repaired as shown in Figure 7 and drainage of ground water stopped in 1997.

In this analysis, the boundary between the tunnel lining and soil was assumed to offer no drainage unless the area of the broken-down joints. And the boundary

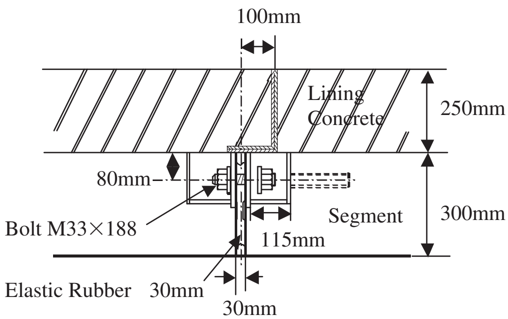

Figure 6. Expansion joint system.

conditions adjacent to the broken-down joints were turned to drainage from 1984 to 1997.

Value of Young's Modulus of the tunnel structure after the reinforcement was not in the JRCC design code and it could not be measured in the field. In this analysis, it was assumed to be ten times the value of Young's Modulus before the reinforcement. In practice, it was determined by trial and error, so that the computed was compatible with the monitoring data.

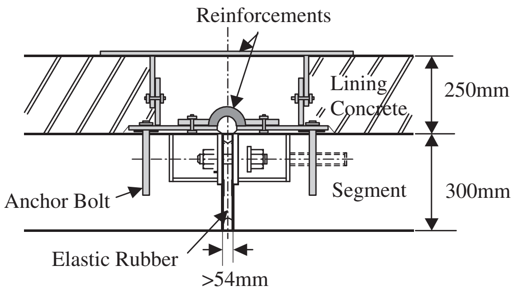

Figure 7. Reinforcement of expansion joint.

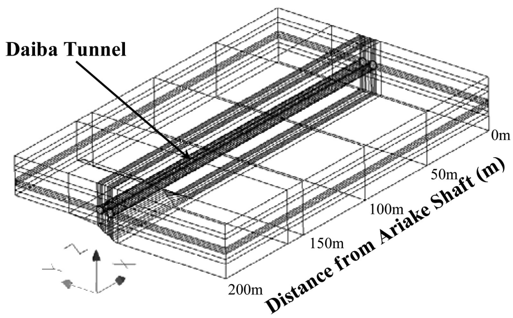

Figure 8. Finite element meshes.

4 NUMERICAL RESULTS

Coupled soil-pore water finite element analysis during period of twenty-seven years after the construction of Daiba Tunnel was performed. The three dimensional finite element meshes using numerical analysis is shown in Figure 8. In the analysis, in order to estimate the magnitude of the ground settlement for the period from the time of the completion of reclaiming fill to the time of starting the field measurement, the initial time for the analysis was set on 1973. Since the field measurements conducted after the completion of Daiba Tunnel in 1980, the seven years elapsed from the initial time of analysis to the time starting the field measurement.

The calculated and measured settlement of Daiba (Eastbound) Tunnel is shown in Figure 9. The result of finite element calculation was well similar to the measured settlement.

5 CONCLUSIONS

In this paper, a field measurement of the behaviour of tunnel structure was carried out for the twin shield tunnel in alluvial clay deposits at the subsiding reclaimed area in Tokyo. And the ability of the three dimensional coupled soil-pore water finite element method to model the long-term behaviour of soil and tunnel structure observed in this field.

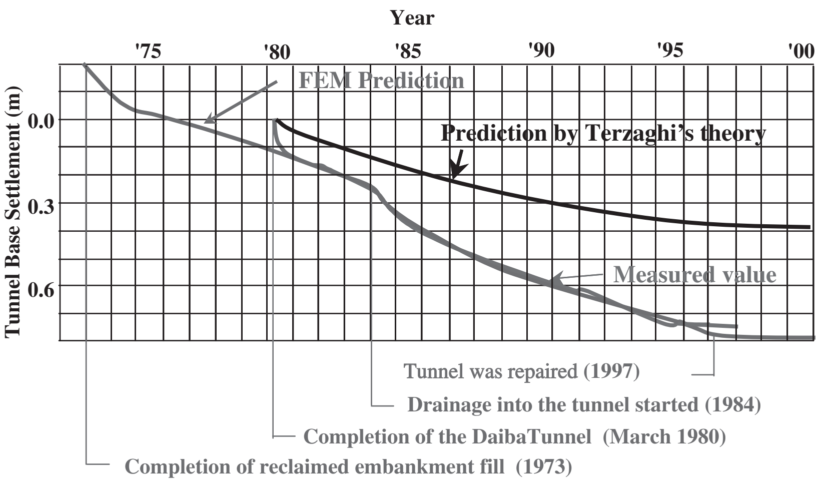

Figure 9. Comparison between the measured value and result of FEM simulation at the Daiba Eastbound Tunnel base where the section is 70 metres away from Ariake Shaft.

The results from the field trial showed that over 0.73 metres settlement at the tunnel base caused by the consolidation of the cohesive soil around the tunnel due to the reclamation of embankment fill. This magnitude of the settlement was larger than the prediction using Terzaghi's consolidation theory.

Finite element analysis of the field trial was performed to simulate the long-term settlement of tunnel after the reclamation. Adopting the finite element modelling technique used in the field simulation, it was possible to obtain a similar trend of the long-term settlement of tunnel observed in the field trial.

REFERENCES

Iizuka, A., Ohta, H. & Yoshimine, T. 1985, *Input parameters of the elast-visco-plastic finite element method*, Proc. of 20th annual meetings on Japanese Society on Soil Mechanics and Foundation Engineering, p.973–974 (in Japanese).

Schofield, A.N. & Wroth, C.P. 1968, *Critical State Soil Mechanics*, McGraw-Hill.