SOILS AND FOUNDATIONS Vol. 44, No. 1, 1-24, Feb. 2004
Japanese Geotechnical Society

GEOTECHNICAL ASPECTS OF CURRENT UNDERGROUND CONSTRUCTION IN JAPAN

HIROKAZU AKAGI[i]

ABSTRACT

The Japanese National Committee for TC28 has produced a committee report on current underground construction activities in Japan, including the technologies of braced excavation and tunneling.

This report is based on technical papers and available construction records from the past decade. In the field of braced excavation, the topics of proximity construction, excavation machines for braced walls, deep shaft excavation, and groundwater flow preservation measures have been selected and introduced, along with some case records. In the field of tunneling, statistical data on tunneling in Japan has been assembled from published data and questionnaires. Several case records using current shield tunneling and urban NATM methods are described. Finally, damages to underground structures due to the 1995 Hyogoken-Nanbu (Kobe) Earthquake are summarized, and new earthquake-resistant designs for shield tunneling and other technologies are introduced.

Key words: excavation, tunnel, underground structure (**IGC:** H0/H5/K6/K9/K10)

INTRODUCTION TO BRACED EXCAVATION IN JAPAN

Almost all of the large cities in Japan, including Tokyo and Osaka, are located on alluvial plains in coastal areas. These plains are formed of less-firm deposits from the last two million years of the Quaternary period. In the shallow portion of the alluvial plains, soft soils are widely prevalent. These soils, called "alluvial soils," are composed of sediments from the end of the Holocene epoch to the Recent epoch, and form very thick, soft layers, which at their deepest are 60 m. Alluvial soils are quite soft and weak, especially the normally consolidated clay, which is one of the most common deposits found in the alluvial soils.

Although soft soils are difficult to build on, the demand for building in Japan's urban areas has continued to increase. Underground structures play a crucial role in the redevelopment of urban areas. In very congested and highly populated areas, it is impossible to avoid performing underground construction close to various existing structures.

Furthermore, underground construction is now performed over larger areas, and at greater depths than before. Where top-down construction was previously used, now a larger number of braced excavations of areas wider than 10,000 m^2 and at depths more than 30 m are being conducted. Excavation deeper than 50 m is required to construct the shafts for urban tunnels. Groundwater is one of the main concerns when performing deep open excavations in alluvial deposits. Within the last thirty years, the aquifer groundwater level in the coastal areas of Tokyo has risen to 50 m, its highest ever; historically, this trend began with the imposition of restrictions on groundwater pumping, intended to prevent land subsidence. Countermeasures against high water pressure are crucial both during and after construction. Another aspect related to groundwater is the effect of underground structures on the water flow in aquifers. Measures need to be taken to prevent the possible change of the flow regime when, for example, a long underground structure threatens to obstruct groundwater flow.

In this section, the author discusses recent technologies related to braced excavation in Japan, including proximity construction, excavation machines for braced walls, and deep shaft excavation. Groundwater flow preservation measures are selected and introduced, together with some case records.

PROXIMITY CONSTRUCTION

Most of the braced excavations in Japan are built in congested urban areas. In such settings, deformation and settlement in the ground associated with construction and the potential effects of this deformation on neighboring structures must be considered in the design. The causes of ground displacement due to braced excavation are as follows.

1) Vertical and lateral displacement of backfill due to the deformation of retaining wall.

[i] Professor of Civil and Environmental Engineering, Waseda University, Tokyo, Japan (akagi@waseda.jp), Chairman of the Japanese National Committee for TC28, JGS.
Manuscript was received for review on February 26, 2003.
Written discussions on this paper should be submitted before September 1, 2004 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

Table 1. Methods used for predicting effects of braced excavation on neighboring structures

Outline of method		Applicable condition
FEM analysis modeling excavation and existing structures.	Construction sequences, that is, removing soil and bracing, are simulated in the analysis, so that the whole behavior including braced wall, ground and existing structures can be predicted in the analysis.	Interaction among the bracing frames, existing structure and ground is important in the overall behavior.
	Deflection of wall is predicted separately using different method, e.g., spring beam model. The deflection is used as input values in the FEM analysis.	Size and rigidity of the existing structures are relatively small compared to the scale of excavation, so that the existing structures do not affect the behavior of the braced wall significantly. But the ground and the existing structure mutually interact.
Structural analysis on existing structures, e.g., beam on springs for pile foundations, in which deflection or load caused by excavation is considered.	Ground deformation predicted separately by the other method, e.g., FEM, is assumed the same as the deflection of the structure and input into the structural model.	Size and rigidity of the existing structures are small enough to assume that they do not affect the behavior of the ground deformation caused by the excavation.
	Change of stresses in the ground due to the excavation is predicted separately by the other method, e.g., FEM or elastic analysis, and applied to the structural model of the existing structure.	Size and rigidity of the existing structure are large compared to the scale of construction work, so that the movement of ground may not cause significant movement of the structure.

2) Surface settlement due to draw-down of the groundwater level.

3) Rebound of excavation bottom due to unloading.

However, these problems are mainly controlled by an in-situ monitoring system.

Table 1 presents a brief outline and lists of applicable conditions for the four methods commonly used for predicting the effects of braced excavation on nearby structures. If designers can predict the undesirable effects a braced excavation will have on the existing structures before the construction is undertaken, the design conditions can be modified to reduce deflection of the bracing wall and the settlement of the ground behind it. Such modifications might involve the length and flexural rigidity of the wall and the number and axial rigidity of the struts.

However, on special occasions, some countermeasures that use ground improvement techniques must be employed. A buttress-type wall made using the Deep Mixing Method (DMM) can increase the stability and rigidity of the wall, and this DMM can be effectively used for large area excavations (Uchiyama, 1999). Creating stiff layers as kinds of struts at the level of embedment using DMM or jet grouting is also effective for reducing the deflection of the wall at greater depths, especially when excavating in thick clay deposits (Tanaka, 1997).

Settlement Prediction from Laboratory Model Test Results (Okahara et al., 1993)

Figure 1 demonstrates the prediction of ground displacement using laboratory model test results. A large model (2 m width × 5 m length × 4 m depth) and dry sand were used in this study. The retaining wall was displaced with and without the pile groups in the backfill. The slip lines and soil deformation observed are shown in Fig. 2. From these experimental results, the following was concluded.

1) 3D displacement is smaller than 2D displacement.

2) Ground loss at the surface is equal to 0.5 ~ 0.8 × (Area of wall deformation).

3) The distance of influence remains almost constant during the increase in wall deformation.

4) When the slip line crosses the pile, the displacement of the pile is in the horizontal direction.

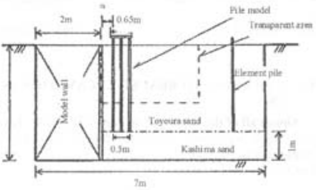


Fig. 1. Soil container used in the experiment

Statistical Analysis of Field Measurement Results (Sugimoto, 1986)

Figure 3 shows the relation between wall deformation and excavation depth obtained from around 150 case records of braced excavation in Tokyo, Japan. Wall deformation is found to be independent of excavation depth, since the soil properties, geometrical condition and wall rigidity are different for each case. If the excavation factor α_c defined in Eq. (1) is adopted, the surface settlement can be successfully predicted from the unique relation between α_c and surface settlement, as shown in Fig. 4.

$$\alpha_c = \frac{B \cdot H}{\beta_D \cdot D} \qquad \beta_D = \sqrt[4]{\frac{E_s}{EI}} \tag{1}$$

In this equation, B is the excavation width, H is the excavation depth, D is the embedment length of wall, E_s

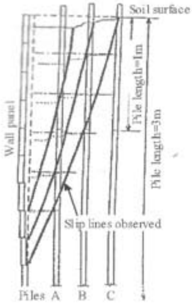


Fig. 2. Slip lines and soil deformation observed in the experiment

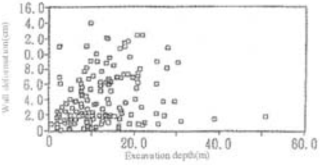


Fig. 3. Relation between wall deformation and excavation depth

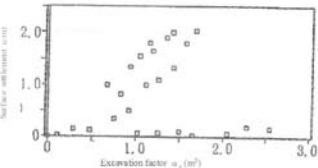


Fig. 4. Relation between α_e and surface settlement

is the deformation modulus of soil, and EI is the flexural rigidity of the wall.

Prediction of Effect on Existing Structures Using Numerical Method

If the interaction among the retaining wall, struts and soil is not to be neglected, the calculation of the sequential unloading of initial stress due to excavation is required using the finite element method. When no struts are employed, the only element introduced into the finite element model is the forced displacement of the wall (Fig. 5).

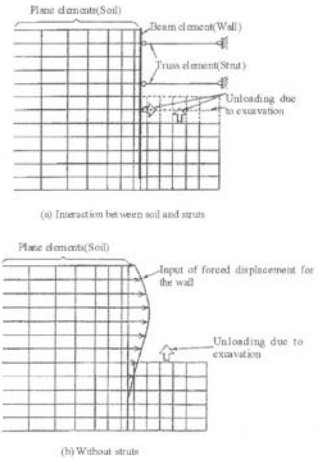


Fig. 5. Prediction of effect on existing structure using FEM

If the effect of an existing structure on soil behaviour is neglected, the soil displacement obtained from finite element analysis is employed around the existing underground structure. If the existing structure is assumed to be fixed during excavation, only the distributed loads are acting on the existing underground structure, which are obtained from elastic theory, i.e. the Boussinesq solution (Fig. 6).

A Case Record of Proximity Construction (Kuroyama, 1996)

A large area excavation was performed in order to construct a new building (D-building) and an underground shopping complex at Osaka station, which is fronted by high-rise buildings, as shown in Figs. 7 and 8. The soil profile at the site is alluvial sand with an SPT N value of about 8 (GL $\pm 0 \sim -9$ m) and soft alluvial clay (GL $-9 \sim -24$ m); below those is a thick Pleistocene deposit. In this construction, an area of 20,000 m^2 area was excavated to a depth of 20 m. The underground walls of the surrounding buildings were utilized as retaining walls for the excavation work. Because of the complicated plan view required to perform this excavation, a preliminary 3D FEM analysis was used to comprehend the overall behavior of the buildings at the site. Then, the behavior of various sections was scrutinized using 2D FEM. Anisotropic elasticity was used as the constitutive model of the soils. The FEM mesh used for the 3D analysis is shown in Fig. 9, together with the soil profile.

The horizontal displacement vectors and contours of the rebound obtained from the 3D analysis are given in Fig. 10. It was predicted that the excavation would induce a horizontal movement of the surrounding buildings towards the new building, and cause rebounds of more than 4 cm within the excavated area. The rebound of the A-building was expected to be larger than those of the other buildings. The horizontal movements obtained from the 3D analyses were 50% smaller, while the magnitude of the rebound was 70% greater than those of the 2D analyses, respectively. As the inclination and stresses of the pile foundation of the A-building were predicted to be sufficiently below the allowable values from the 2D analysis with a detailed mesh, the construction was completed safely using minimal countermeasures.

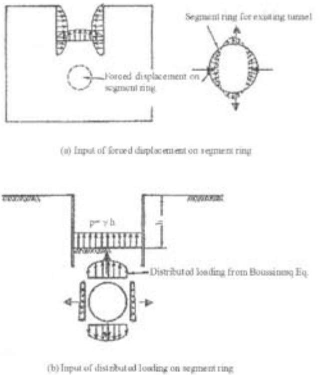


Fig. 6. Simplified prediction of effect on existing structure

Fig. 7. Plan of construction site of large area excavation (Kuroyama, 1996)

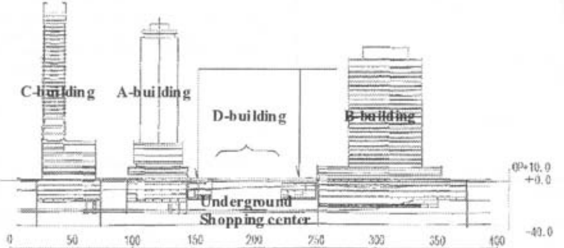


Fig. 8. Cross section of construction site (A-A' section)

Soil Improvement Associated with Braced Excavation

Figure 11 shows buttress-wall type soil improvement, conducted in Tokyo Bay area (Uchiyama et al., 1999). The excavation area was 70 m × 50 m and the depth was 9 to 11 m. The soil condition was soft clay, with an N value of 0 ~ 2 and a natural water content greater than the liquid limit w_L. Measurement results of the buttress improvement are shown in Fig. 12. The maximum wall displacement at the bottom was around 20 mm without buttress and 12 mm with the buttress. The wall deformation was reduced almost by half with the buttress improvement.

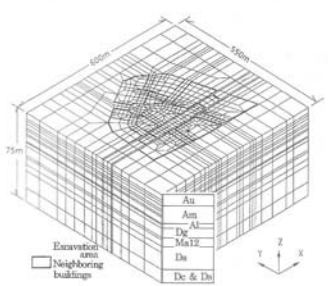
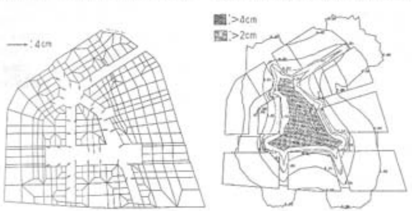


Fig. 9. FEM mesh for 3D analysis and soil profile (Kuroyama, 1996)

Figure 13 shows a pre-injected underground beam constructed in a soft clay deposit in Kanagawa prefecture, near Tokyo (Tanaka et al., 1997). In order to reduce the soil deformation and protect the existing structure from damage, the excavation bottom was improved by jet grouting. The area of excavation was 34 m × 13 m and the excavation depth was 12.7 m. The soil condition was soft clay with an N value of 1 ~ 2. The thickness of the beam was 1.5 m. As shown in this figure, the pre-injected beam remarkably reduced the wall deformation at the bottom.

A self-supporting wall with soil improvement constructed in saturated fine sand at Chiba prefecture is shown in Fig. 14 (Ishii and Abe, 2000). The excavation area was 51 m × 77 m and its depth was 7.1 m. The soil condition was fine sand with an N value of 5 ~ 10. The



(a) Horizontal displacement at GL-17.3m (b) Contour of rebound at GL-28m

Fig. 10. Results of 3D FEM analysis (Kuroyama, 1996)

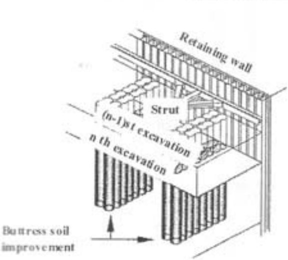


Fig. 11. Buttress wall type soil improvement

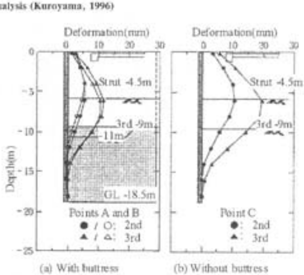


Fig. 12. Measurement results of buttress improvement

maximum deformation at the excavation bottom was around 12 mm without any struts.

EXCAVATION MACHINES FOR BRACED WALL CONSTRUCTION

To meet various demands in excavation projects, a variety of excavation machines have been adopted to build braced walls with high accuracy. Table 2 shows the excavation machines used for braced wall construction in urban areas of Japan. Figure 15 shows a recently developed machine used in the trench cutting re-mixing deep wall method, in which a chainsaw type cutter is inserted into the required depth and moved transversely, forming a soil-cement mixing wall of equal thickness (Takahashi and Amakusa, 1998). Figure 16 shows another recently developed machine, called the "Swing Arm Taisei Twin Cutter," which can make braced walls under buried structures (Miyamae et al., 1997).

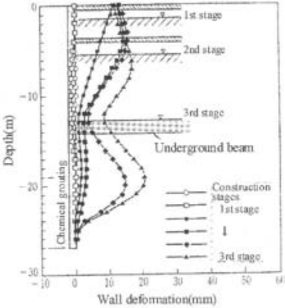


Fig. 13. Pre-injected beam

DEEP SHAFT EXCAVATION

Recent Trends in Deep Excavations

Figure 17 presents a history of excavation projects in Japan, showing the relationship between the length of braced walls and the year in which construction commenced (JGS, 2001). It gives the history of braced wall construction methods, and shows the continuous increase in wall length since 1978. Walls longer than 50 m and 100 m were first constructed in 1980 and 1987, respectively. Almost all braced walls longer than 50 m were constructed using the diaphragm wall excavation method. Only circular type diaphragm walls were used for excavations deeper than 50 m. The deep excavations were all done in order to place deep shafts in urban tunnel construction, except for the foundations of bridges and in-ground LNG tanks. The longest braced wall and deepest excavation in the history of open excavation in Japan are 140 m and 82 m, respectively (Koizumi, 1998).

In order to meet the requirements for further deep excavation, new construction technologies for maintaining wall shapes and withstanding high groundwater pressures are being developed by various organizations in Japan.

A Case Record of a Deep Shaft Construction Project (Maeda, 1993)

In deep shaft constructions, chemical grouting is sometimes used to produce a less permeable layer as a counter-

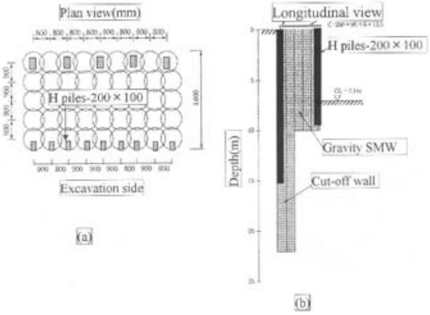


Fig. 14. Self supporting wall improvement

Table 2. Excavation machines used for braced wall construction

Type of excavation	Excavation machine	Name of wall	Wall construction method Structual Wall
Auger excavation	Multi-axes (single-axis) augur	Soil-Cement mixing wall	In-site mixing, Steel H
		Steel pipe sheet pile wall	In-site mixing, Steel pipe
Trench excavation	Bucket type excavator Rotary cutter with vertical multi-axes or horizontal multi-axes	RC-diaphragm wall	Concrete replacement, Reinforcement
		Steel box diaphragm wall	Concrete replacement, Special steel H
		Slurry setting wall	Slurry cement replacement, Steel H
		Mud setting wall	Mud mortar replacement, Steel H
Row excavation	Chainsaw type excavator	Trench cutting re-mixing deep wall	In-site mixing, Steel H or Special steel H

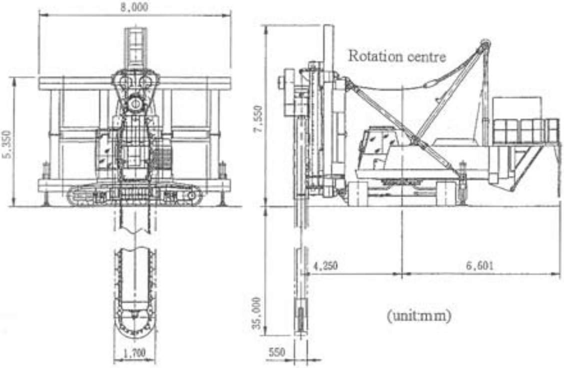


Fig. 15. Trench cutting re-mixing deep wall machine (Takahashi, 1998)

measure against heaving or boiling due to high groundwater pressure. In the construction of a launching shaft for shield tunneling, which was used for a flood control reservoir, chemical grouting was used, as shown in Fig. 18. A circular type diaphragm wall with an inner diameter of 28.2 m and a depth of 59.9 m was constructed in the ground, under soil conditions shown in the figure. The groundwater level in the upper and lower sand layers were GL-10 m and 20 m respectively. As a countermeasure against heaving, the wall was built to be 98 m in length, and a chemical grouting of 5 m in thickness was applied at the bottom of the wall, so that the self-weight of the soil in the embedment could resist the uplift force from the lower sandy layer.

After confirming the efficiency of the grouting, i.e., confirming that the targeted permeability of the grouted portion was $k \leqq 2.5 \times 10^{-7}$ m/s, a well was used inside the shaft, and the excavation work was performed. Throughout the entire construction process, including the excavation and tunneling, no groundwater problems were encountered, and the construction project was completed safely.

GROUNDWATER TREATMENT AND CONSERVATION

The background of this topic is the remarkable rise of underground water levels in urban areas in the past three decades, since the 1960s when the government started restricting pumping up the groundwater. The demand

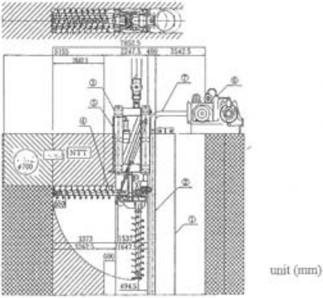


Fig. 16. Swing Arm Taisei Trench Cutter (Miyamae, 1997)

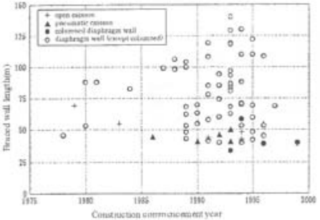


Fig. 17. Chronological data on length of braced wall (JGS, 2001)

for underground construction of greater depths and larger areas coupled with increased environmental awareness has made environmental issues with regard to underground water more important than before. Therefore, underground water conservation is required during and after construction.

Figure 19 summarizes countermeasures during construction for the treatment and conservation of underground water. The methods are classified into three categories, i.e. drainage, cut-off of groundwater and others.

A Case Record of the Deep Well Method: Kawasaki Manmade Island (Nakagawa, 1999)

The first case record concerned with groundwater treatment employed the deep well method for the construction of a manmade island within Tokyo Bay, as shown in Fig. 20. Large-scale excavation using a diaphragm wall was conducted, i.e. thickness of wall = 2.8 m and wall depth = 119 m. To conduct the dry excavation, the water head of the topsoil layer was lowered to the target level of 71.6 m from sea level, using four deep wells. In order to avoid heave of the excavation bottom, the water head of the second layer was made at 45.6 m using 14 deep wells and the third layer water head was made at 35.8 m using 12 deep wells. This case raises two important safety points for construction. The first is the importance of specifying the target draw-down groundwater level. The second is the need for sufficient water treatment capacity within the system.

A Case Record of the Deep Well Method with Recharge of Groundwater (Miyake et al., 1999)

This second case record employed the deep well method with a recharge of groundwater in Tokyo. The wall excavation was carried out to a depth of 28 m from the ground surface using the SMW (Soil Mixing Wall) method, as demonstrated in Fig. 21. The deep well method was employed to avoid excavation bottom heave. Groundwater was pumped up from the first soil layer to draw down the water head to the target of −19 m and from the second layer to −9 m. Simultaneously, the recharge of groundwater from the first and second layers was conducted for the third layer.

The aims of this recharge are as follows. On the one hand, it conserves the groundwater environment; on the other, it secures the exit or drainage route of groundwater from the pumped up layers. In other words, this deep well method satisfies dual purposes, i.e. pumping up and recharging.

A Case Record of Underwater Construction (Nakajima et al., 1998)

The third case record is of underwater construction, employed for a subway station in Tokyo, as demonstrated in Fig. 22. The depth of the diaphragm wall excavation was 27 m and its width was 20 m. An old expressway structure existed just adjacent to the excavation. A soft peat soil layer was embedded in the alluvial clay layer.

During the first stage, dry excavation was carried out. Then, the tap water was retained within the retaining wall, in order to avoid excavation bottom heave. Underwater excavation was conducted by using bucket and rotary excavators. A 3 m bottom layer of concrete was poured. The bottom concrete was sufficiently compacted to secure water cut-off capability between the wall and the bottom concrete. Heavy steel structures were installed to resist the buoyancy force exerted under the concrete. Finally, the drying up and construction of the subway station structure was conducted.

GROUNDWATER FLOW PRESERVATION MEASURES

Because of environmental issues such as noise, vibrations, and preservation of the landscape, urban planners sometimes decide to construct transportation systems

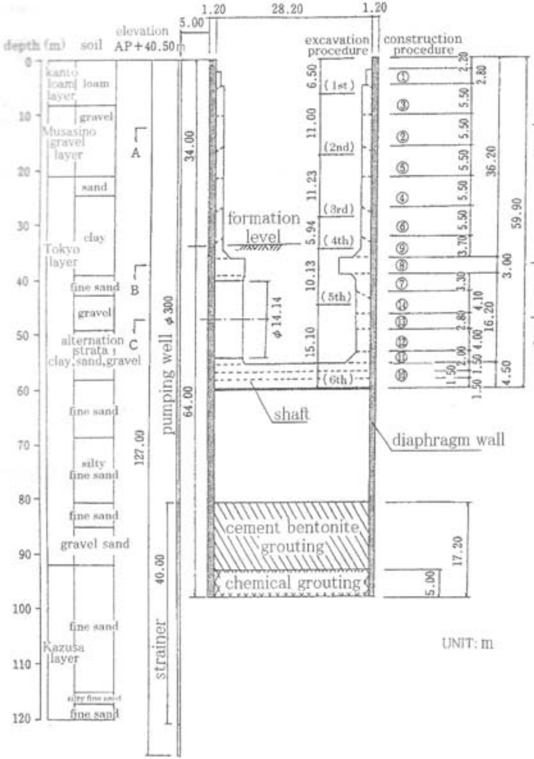


Fig. 18. Cross section of deep shaft and soil condition (Maeda, 1997)

such as railways or highways underground. However, these long underground structures may cause other environmental problems. If the underground structure is constructed by the cut-and-cover method, long and deep braced walls are normally built and are left at the site, even after construction is completed. This long deep wall and the underground tunneling itself may interrupt flow in aquifers at the site, causing a rise of groundwater level in the upstream and a draw-down in the downstream. This change of groundwater regime leads to various environmental problems, such as leakage of water into basements. The change also causes death of plants in the upstream, and land subsidence and draw-down of water in wells in the downstream.

In order to prevent these problems associated with groundwater flow interruption, groundwater flow preservation measures must be employed. With these measures, the interrupted water collected in the upstream is drained through pipes laid in or beneath the structure and is recharged to the downstream. The following procedure is used. Figure 23 illustrates the four construction stages of this intake-recharge method:

(a) After the completion of the underground structure, parts of the cut-off walls are removed, so that transmissivity through the aquifer can occur.

(b) After the completion of the underground structure, horizontal holes are drilled in the cut-off wall, and intake and recharge pipes are installed into the aquifer from the holes.

(c) During the construction of the braced wall, the functions of intake and the recharge of water are provided in the wall.

(d) Intake and recharge wells are installed outside the walls; some space is therefore required for this installation to take place.

A Case Record of a Prefabricated Pervious Wall System for Groundwater Flow Preservation (Hashimoto et al., 2001)

Figure 24 shows the prefabricated pervious wall system that was adopted for subway construction in Kyoto. The subway underground structure was constructed using braced excavation by a SMW wall. The depth of excavation was 15.5 m and the width was 10 m.

When the SMW wall excavation was conducted, the prefabricated pervious walls were placed at the upstream and downstream sides of the underground water flow.

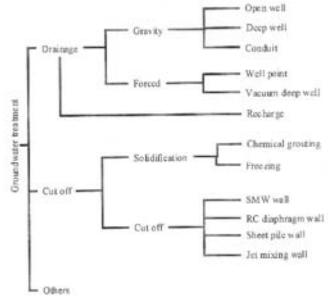


Fig. 19. Underground water treatment

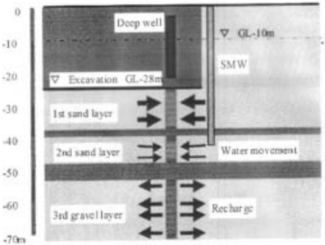


Fig. 21. Deep well method with recharge of groundwater

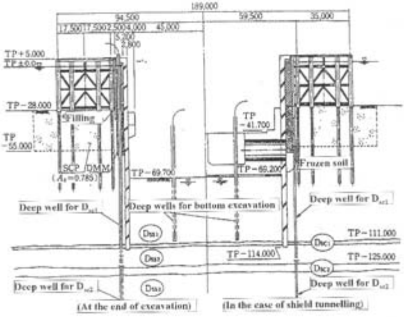


Fig. 20. Deep well method in Kawasaki manmade island

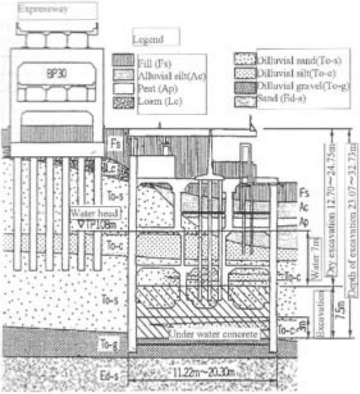
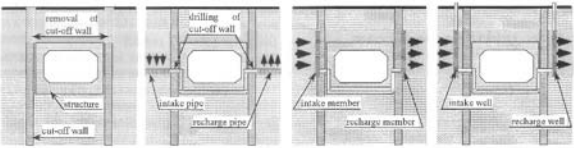


Fig. 22. Under water excavation



(a) Removal of cut-off wall (b) Intake and recharge pipes installed in drilled hole (c) Intake and recharge functions provided in cut-off wall (d) Intake and recharge wells

Fig. 23. Groundwater flow preservation using intake and recharge method (RCGG, 2001)

Then, the pipes for water flow were connected to both sides. The braced excavation was completed without any disturbance of groundwater condition.

A Case Record of Removal of Walls for Groundwater Flow Preservation (Ueda, 1999)

A semi-underground box-culvert type highway was constructed using cut-off walls, which penetrated through an aquifer. Since a respective rise and draw-down of the groundwater levels upstream and downstream of the aquifer were expected consequences of the interruption of groundwater flow, the following measures were used in the construction (*see* also Fig. 25):

—3 m and 3.5 m of the upper portions of the cut-off walls were removed at the upstream and downstream sides of the aquifer, respectively.

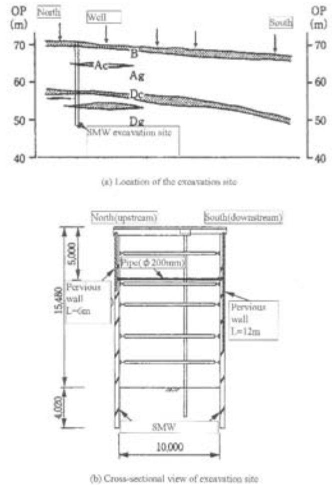
—Under-drains were built in the longitudinal direction of the box culvert at the upstream and downstream sides of the aquifer.

—200 mm diameter siphon pipes were transversely installed beneath the box culvert every 50 m.

—Discharge at the downstream side was conducted through infiltration pits.

Beginning in 1987, the groundwater level was monitored for five years prior to construction.

Variations of the groundwater levels monitored at the upstream side (Well A) and downstream side (Well B) are shown in Fig. 26. During the construction period, the water level in Well A became elevated, and that in Well B became gradually lower. The difference in the ground-



(a) Location of the excavation site

(b) Cross-sectional view of excavation site

Fig. 24. Prefabricated pervious wall system

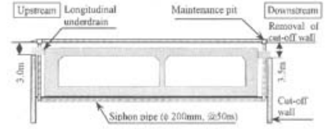


Fig. 25. Groundwater flow preservation by wall removal method (Ueda, 1999)

water level between the two wells increased from about 2 m to 7 m at maximum. However, after measures to preserve the groundwater flow were adopted in 1993, the difference in the groundwater levels gradually decreased. The groundwater levels at the two wells returned to nearly their original levels prior to construction.

INTRODUCTION TO TUNNELING CONSTRUCTION IN JAPAN

During the last decade, the shield tunneling method has been used under difficult conditions, and NATM has come to be used more widely in softer urban-area ground than before. Therefore, this committee report will also include case records on NATM in urban areas.

The characteristics of advancing shield tunneling technology during the last decade are as follows:

1) Technologies such as the Multi Faced shield, non-circular shield, and cross-section enlargement shield have been developed, and these technologies are being used to overcome problems with the construction of infrastructures in urban areas.

2) The shield tunneling method has been used to construct road tunnels and underground channels, such as the Trans Tokyo Bay Highway project, Metropolitan new subway line and the Metropolitan Area Outer Discharge Channel project.

The characteristics of the advancing NATM technology during the last decade are as follows:

1) The excavation process and countermeasures against face stabilization have been greatly improved.

2) NATM has been employed for road tunnels, railway tunnels and underground channel construction in urban areas.

Since NATM can be used in diluvial soil layers containing alluvial soil, it has become one of the most commonly used urban area tunneling methods.

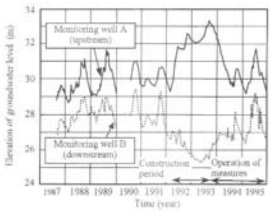


Fig. 26. Variations of the groundwater level with time (Ueda, 1999)

SHIELD TUNNELING METHOD

The statistical features of the shield tunneling projects from 1990 to 1999 can be summarized as follows, based on a survey administered by the Shield Tunnel Association in Japan. Three case records of shield tunneling projects are introduced and two types of numerical methods to analyze the TBM movement and soil deformation during shield tunneling are demonstrated.

Number of Shield Tunneling Projects Performed and Their Usage

Figure 27 chronologically lists the number of shield tunneling projects which have been performed, and the usage of the shield tunnels which have been built. Surprisingly, the number of shield tunnels being built decreased from 322 in 1990 to 115 in 1999. From the point of view of usage, sewage tunnels constituted 69.1% of the total shield tunnels built; railway tunnels, 9.8%; water supply tunnels, 6.4%; electric power supply tunnels, 5.4%; telecommunication lines, 2.0%; utility tunnels, 1.9%; underground channels, 1.8%; gas supply tunnels, 1.1%; road tunnels, 0.7%; and other kinds of tunnels, 1.8%, respectively.

Shield Machine Type

Figure 28 demonstrates the type of shield machine reported in the case records. From this figure, it can be seen that almost all of the shield tunnel construction projects in Japan adopted the closed type shield machine, that is, the closed shield-type machine was used in 97.0% of the projects, the open type shield machine in 2.6%, and other machines in 0.4%. Among the closed-type shield machines, the slurry-type shield machine constituted 25.2%, and the EPBS, 71.8% (earth pressure balance type with mud, 69.9%, and earth pressure balance type, 1.9%).

Soil Type

Figure 29 shows the soil types for each closed shield-type machine. From this figure, it can be seen that the earth pressure balance machines are used more frequently in clay and silty ground. For gravelly ground, the earth pressure balance machines with mud are more suitable than other machines. These results represent the compatibilities of different types of closed-type shield machines with different soil types.

The Overburden Depth of Tunnel and Maximum Groundwater Pressure

Figures 30 and 31 indicate the overburden depth of the tunnels and the maximum groundwater pressure for each closed type shield machine, respectively. From construction records of projects with more than a 20 m overburden depth, it was found that the slurry-type shield machine was used in 49%, mud EPBS in 19%, and EPBS in 23%. The slurry-type shield machine is the favored type of machine for constructing deep tunnels.

From Fig. 31, it can be seen that of construction records for projects with more than 200 kPa maximum groundwater pressure, the slurry shield was used in 29%, mud EPBS in 13%, and EPBS in 18%. Tunneling within ground with a higher groundwater pressure was carried out more often with a slurry-type machine.

However, the actual numbers of construction records

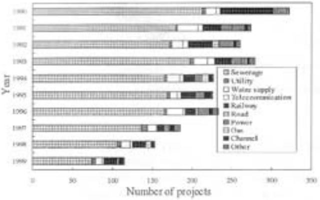


Fig. 27. Number of shield tunneling and their usage

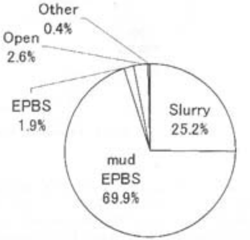
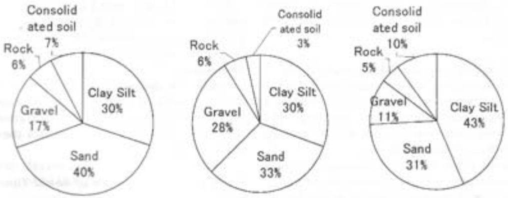


Fig. 28. Shield types reported in the case records



Slurry type Earth pressure with mud type Earth pressure type

Fig. 29. Soil types in the case of closed face type shield machine

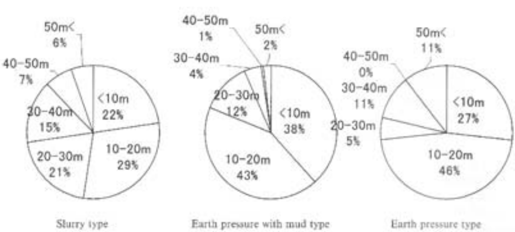


Fig. 30. Overburden in the case of closed face type shield machine

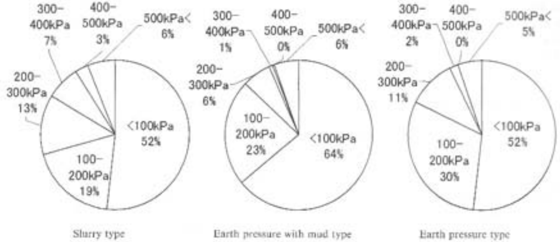


Fig. 31. Groundwater pressure in the case of closed face type shield machine

using slurry shield machines and mud EPBS machines under a depth of more than 20 m were 348 and 386, respectively. The actual numbers of construction records on slurry shield machines and mud EPBS machines used under more than 200 kPa groundwater pressure were 215 and 265, respectively. The differences between the actual numbers of construction records are not so great, when considering the difference in the numbers of shield tunneling projects using slurry type machines and EPBS machines reported.

A Case Record of Triple Faced TBM (Yahagi, 2001)

The first case record is of subway tunneling in Tokyo. The length was 6.1 km, where triple-faced TBM and a machine with eccentric cutters were employed. The special TBMs were driven successfully within very soft alluvial clay, with an N value of $0 \sim 1$.

Figure 32 demonstrates the special machine with triple faces, which was employed in the construction of underground station and of the train parking place. In this case, the triple faced TBM employed in the excavation of the subway station was transported with sliders through the connection shaft. Then, the machine was reused for the excavation of train parking space.

The special earth pressure balanced type machine with eccentric face cutters is shown in Fig. 33, which is able to excavate any shape of tunnel cross section, i.e. elliptic, rectangular or circular.

A Case Record of Parent-and-kid TBM

The second case record is of shield tunneling in the railway project along Tokyo Bay Coast. The length was 12.3 km, a parent-and-kid TBM was driven, and difficult proximity construction was performed in a congested urban area.

Figure 34 demonstrates the parent-and-kid TBM, which consists of two types of machines. The larger parent machine was used for the underground station and the smaller kid machine was used for the ordinary train track tunneling.

Finite Element Simulation of Shield Tunneling (Komiya et al., 1999)

The aim of this numerical procedure is to reproduce the machine movement and soil deformation during shield tunneling. The procedure consists of two phases: the first

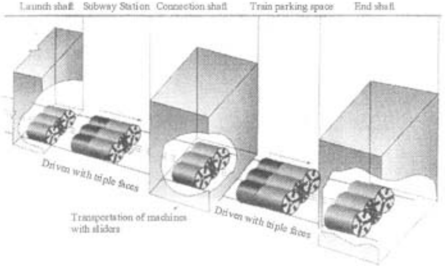
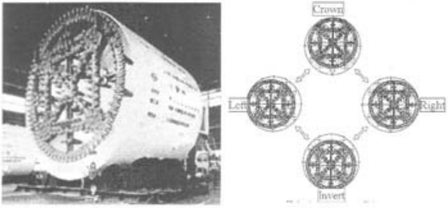


Fig. 32. Shield machine with triple faces



(a) Total view of machine (φ9,600 mm) (b) Movement of eccentric cutters

Fig. 33. EPB machine with eccentric cutters

is the rearrangement of finite element mesh and the second is the employment of excavating elements, which represent the excavation at the cutting face, as shown in Fig. 35.

The numerical simulation results are demonstrated in Fig. 36, using the 3-D finite element method. These figures show the three-dimensional view of the shield machine movement and the soil displacement obtained at the subsurface plane 1 m above the machine crown. The machine moves in the up-left hand direction. The soil moves upward at the face of the machine and downward at the tail clearance behind the machine.

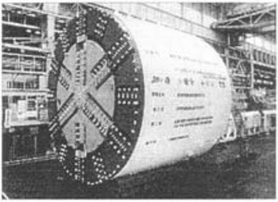
Shield Machine Movement Simulation Using Static Equilibrium (Sugimoto and Sramoon, 2001)

This numerical procedure consists of the back analysis of soil properties and the prediction of shield machine movement during tunneling. The static equilibrium and the geometrical relations among the TBM itself, the excavated area, the tail clearance, the cutting directions and soil loosening area at the tunnel crown are described and solved to predict the machine movement, as shown in Fig. 37.

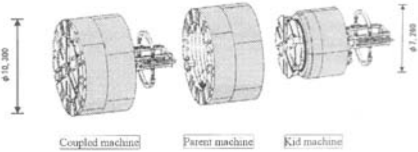
NATM IN URBAN AREAS

The background of this issue is that NATM has been nominated as the standard tunneling method by the Japanese Society of Civil Engineers, JSCE in 1986. Since then, auxiliary methods for stabilizing the cutting face have been investigated. Consequently, the range of applications of NATM has been extended to soft soil conditions, frequently found in Japanese urban areas.

"Urban NATM" is used as a generic name for tunnels constructed using NATM in urban areas. The ground in urban areas is often composed of alluvium, diluvium, and Tertiary Era soil, as well as other types of soil. NATM has seldom been used in alluvium soils, as there



(a) Total view of machine



(b) Coupling and De-coupling of Parent and Kid machine

Fig. 34. Parent and Kid TBM

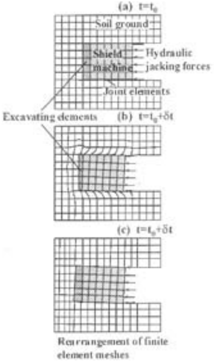


Fig. 35. Advance of the shield machine simulated by using excavating elements

are very few cases reported.

It is assumed that the unconfined compression strength $q_u = 0.1$ MPa and the elastic modulus $E = 10$ MPa level are the lower bounds of physical ground properties suitable for its application. Urban NATM is not employed in the case of difficult groundwater conditions.

Statistical Data of NATM in Urban Areas

Figure 38 indicates the classification of NATM usage in urban areas. NATM is frequently employed in construction of highways and water courses.

The types of auxiliary methods used with urban NATM are summarized in Fig. 39. Among 29 case records published in this decade, 23 NATM constructions adopted fore piling and groundwater treatment.

Figure 40 classifies the soil types in the cases of NATM. In most cases, NATM was employed in non-tertiary deposits or diluvial soil deposits.

Development of New Technology

When the ground is soft, an auxiliary method to stabilize the tunnel face (crown and face) is indispensable for Urban NATM. Moreover, auxiliary methods are often adopted because of the necessity of controlling subsidence at the ground surface.

The auxiliary methods that have contributed to Urban NATM development can be classified as follows:

—Fore piling: Jet grout, Long steel pipe fore-piling with

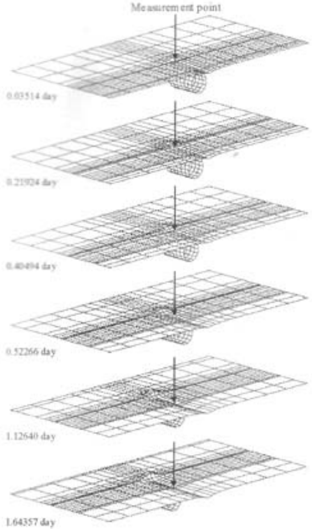


Fig. 36. Three dimensional views of the computed settlement (1 m above the crown of the machine)

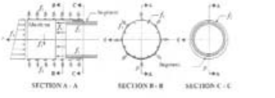


Fig. 37. Machine movement simulation using static equilibrium

injection, Pre-lining
—Leg piling: Foot pile (horizontal jet grout)
—Face reinforcement: Face reinforcement bolt and pile.

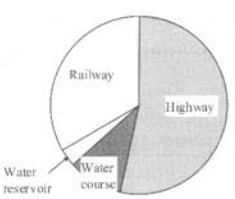


Fig. 38. NATM usage in urban area

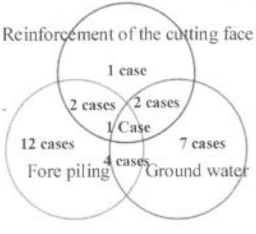


Fig. 39. Auxiliary method employed in NATM

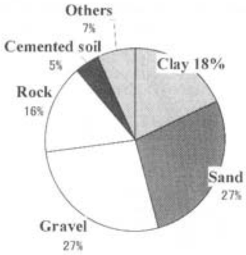


Fig. 40. Soils types in NATM

Technical Challenge to Shallow Overburden

Shin-Kobe tunnel is a road tunnel, and the overburden is very shallow, i.e. 4-11 m. A typical cross section is shown in Fig. 41 (Iwata et al., 2000). In this figure, a road and a river are situated just over the tunnel, and an elementary school is located on the left. Geotechnical features include an alluvial layer and a gravel layer at 1.5–3 m from the surface. The soil condition is represented as $E = 80$ MPa as an elastic modulus.

The umbrella method was adopted as an auxiliary method, and the tunnel support was provided with long steel pipe fore piling (*Trevi tube*, widening type), wing ribs, leg horizontal jet grouting methods, and pre-loading

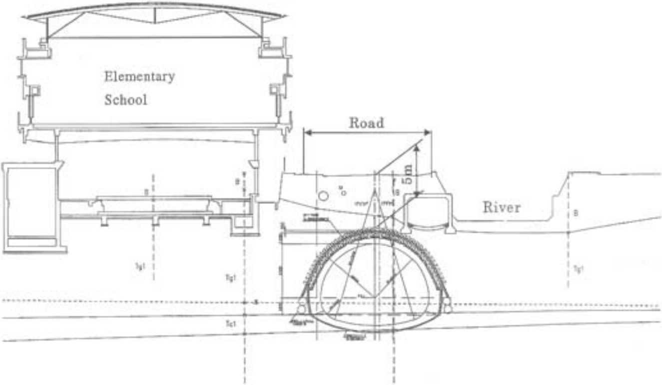


Fig. 41. Shin-Kobe tunnel cross section

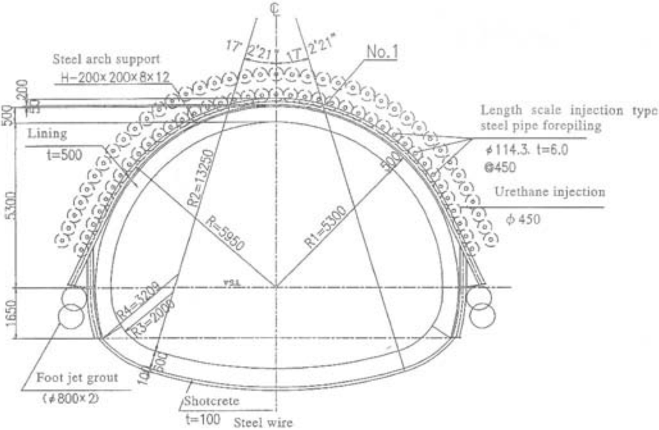


Fig. 42. Shin-Kobe tunnel execution scheme

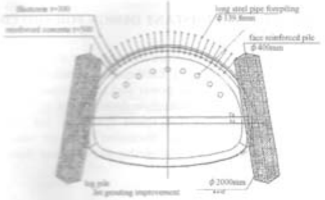


Fig. 43. Mito tunnel: typical cross section

shells, as shown in Fig. 42. The *Trevi tube* is a steel pipe of 12 m length, whose lap length for each shift is 6 m; the range of the placing angle is 180 degrees in the upper region, and the injection material is urethane. A half excavation was carried out and leg horizontal jet grouting of φ800-1,000 mm was performed to improve the soil body, resulting in an unconfined compression strength of about $q_u = 10$ MPa, measured seven days after the injection. According to the measurement result, an initial subsidence of 3-5 mm was observed at the ground surface, and the final subsidence was 12 mm, observed around 15 m behind the face.

The Mito tunnel is a road tunnel (Ohmori et al., 2000). The excavated cross sectional area exceeds 100 m^2 and the overburden depth is also shallow, i.e. 6.6-12.8 m. The road and lifeline structures are situated just above the tunnel, and residential houses and buildings are located within the proximity. The geological age of the ground is classified into the unconsolidated Quarternary Diluvium Epoch, and the ground is composed of a cohesive soil layer and grits layer. Cohesive soil layer appears at the tunnel face. The SPT N value is around eight on average, and the elastic modulus of the soil is about $E = 48$ MPa. The most important problem in this project was to reduce the influence on the ground surface as much as possible.

The crossing jet method was adopted as a countermeasure in order to conduct large-diameter pre-leg improvement, as shown in Fig. 43, which was a kind of soil-mixing method with a high-pressure jet.

The measurement results are as follows. The preceding surface subsidence was observed approximately 20 m ahead of the tunnel face, and a 7 mm surface settlement took place at the tunnel face position. The final surface settlement observed at around 25 m behind the tunnel face was a little bit larger than the predicted value, i.e. 17 mm.

Technical Challenge to Tunneling in Soft Ground

The Narashino-dai tunnel is a railway tunnel with an extension of 2.36 km, and is located in Kita-Narashino on the Toyo Rapid Transit line, which connects Tokyo and Chiba Prefecture (Narazaki et al., 1995). The geotechnical features of the ground included unconsolidated soft

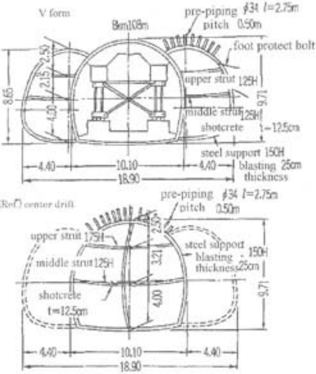


Fig. 44. Narashino-dai tunnel station: large tunnel cross section

Narita sand beds with groundwater, whose elastic modulus was $E = 21 \sim 59$ MPa and cohesion was $0 \sim 0.03$ MPa. The uniformity coefficient was small, and it was assumed that tunnel collapse would take place due to quicksand phenomena.

The CRD method (Cross Diaphragm Method) was developed in order to solve this problem. This method follows the variations in cross-sectional shape and minimizes the loosening of the ground. The excavation of the station, with a maximum cross sectional area, was divided into three sections. The central pilot tunnel was constructed using the CRD method. The rigidity of this tunnel was improved by using a secondary lining and a steel pipe pillar. After the side drifts on both sides were completed, the entire tunnel section was completed, as shown in Fig. 44. The supplementary methods adopted were the deep well method and a chemical grouting of 3 m thickness around the tunnel.

A surface settlement of 20 mm was observed when the central pilot tunnel was excavated. The amount of the final settlement was 25 mm, which was smaller than the permissible value (30 mm).

Technical Challenge to Build a Special Tunnel Structure (Two Layer Structure)

The Oume tunnel, located in the Ken-oh-do expressway, is a special tunnel structure with two layers (double decks), whose excavated cross sectional area was 230 m^2 on average (Haruyama et al., 2001). The geotechnical features to be excavated were the riverside terraces, which were composed of a terrace boulder layer and a fine-grained Kanto loam layer.

The two-staged bench cut and the upper lining supported by the bridge system was planned to be employed for the initial excavation, as shown in Fig. 45. However, it was predicted from the analysis of the measurement results during the first- and second-stage bench excavation that the effects of soil deformation on the existing ground surface structures induced by the third stage bench excavation would be severe. Therefore, the builders decided to change the excavation system to a method using a temporary bearing pile, as shown in Fig. 46. With regard to the auxiliary method, the horizontal jet grouting and the foot pile method were adopted.

An initial surface settlement of 3 ~ 5 mm was observed when the first bench was excavated. The final surface settlement was 10 ~ 15 mm, when the second bench excavation was completed.

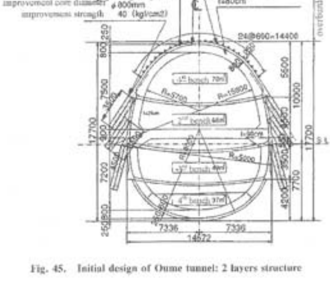


Fig. 45. Initial design of Oume tunnel: 2 layers structure

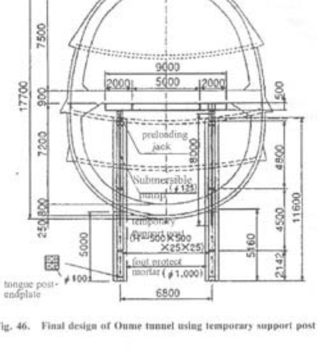
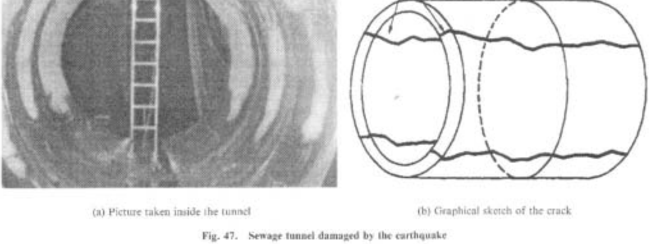


Fig. 46. Final design of Oume tunnel using temporary support post

EARTHQUAKE-RESISTANT DESIGN FOR SHIELD TUNNELING

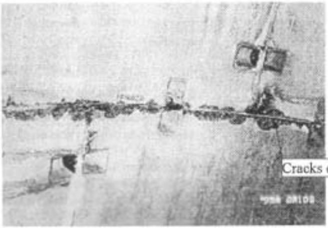
The underground structure damages due to the 1995 Hyogoken-Nanbu (Kobe) Earthquake included tunnel cracks in sewage, electric power and telecommunication lines. From these experiences, a new concept of earthquake-resistant design for shield tunneling has been created. Below, following discussion of these issues, some practical examples of earthquake-resistant design are demonstrated.



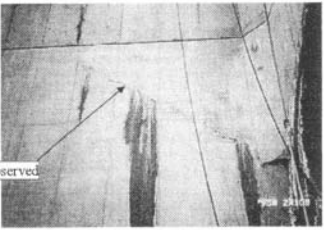
(a) Picture taken inside the tunnel

(b) Graphical sketch of the crack

Fig. 47. Sewage tunnel damaged by the earthquake



(a) Cracks observed at the tunnel crown



(b) Cracks observed at the tunnel shaft wall

Photo 1. Electric power line tunnel damaged by the earthquake

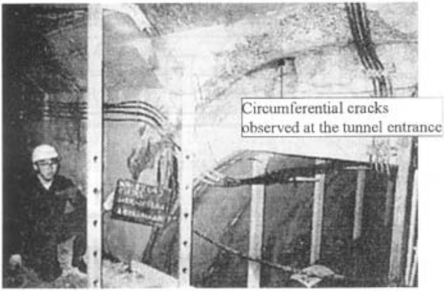


Photo 2. Telecommunication line tunnel damaged by the earthquake

Damages due to the 1995 Hyogoken-Nanbu (Kobe) Earthquake (JSCE, 1998)

Figure 47 shows a picture of a damaged sewage tunnel and a graphical sketch of the observed cracks. The tunnel diameter is 3.5 m and the overburden is 9 ~ 14 m. The soil condition is alluvial sand and clay deposit. Longitudinal and radial cracks were observed in the secondary lining.

The partial fracture of a concrete segment and small cracks in a tunnel shaft wall for the electric power line tunnel are shown in Photo 1. The tunnel diameter is 4.8 m. The soil condition is sand and gravel with an N value of 30 ~ 50.

Photo 2 shows a picture taken of a damaged telecommunication line tunnel. Circumferential cracks were observed at the entrance of the tunnel. The tunnel diameter is 4 m and the overburden is 10 m. The soil condition is diluvial soil.

New Concept of Earthquake Resistant Design for Shield Tunneling (JSWA, 2001)

From experiences of the 1995 Hyogoken-Nanbu (Kobe) Earthquake, the magnitudes of design values for seismic input loading during large-scale earthquake have been discussed among the academic societies in Japan.

Consequently, a concept of two stages of earthquake-induced vibrations has been introduced. The first stage vibration is just within the elastic range of the structure. The second stage vibration induces the fracture stage of the structure.

This concept has been introduced into civil engineering structures and is expected to be extended to underground structures, including shield tunnels.

Practical Technology for Earthquake Resistant Design (JSCE, 2001)

Figure 48 shows a flexible segment, which reduces the

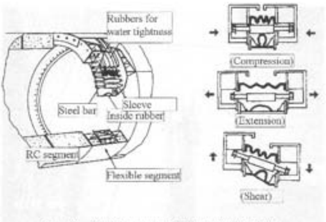


Fig. 48. Flexible segment between segment rings

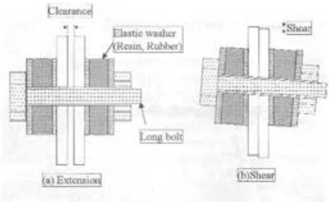


Fig. 49. Elastic joints

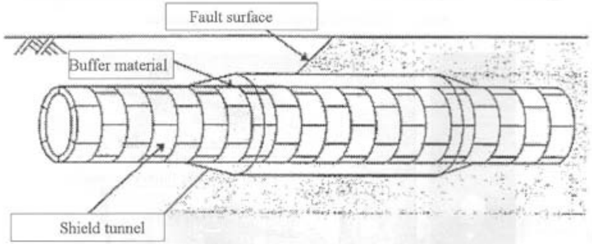


Fig. 50. Buffer material used for absorbing fault displacement

acting forces due to earthquake, ground settlement and temperature variation. Flexible segments adapt to the compression, extension and shear displacement between the other segments.

The reduction of joint rigidity is aimed at the elastic joints, as shown in Fig. 49. Joint rigidity controls the total rigidity of the tunnel. Reducing the joint rigidity is effective in decreasing the acting force within the primary lining. Long bolts with elastic resin or rubber washers are employed for this purpose. The elastic joints also adapt to the extension and shear displacement between the other segments.

Soft buffer material around tunnels and shafts has been suggested for absorbing the induced displacement by the fault, as demonstrated in Fig. 50 (Kawashima, 1994). Injection of the soft buffer material, i.e., air mortar, between primary and secondary linings, as shown in Fig. 51, has been indicated to be effective in the actual event of earthquake.

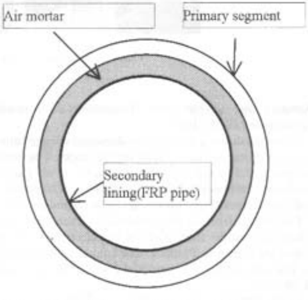


Fig. 51. Reduction of secondary lining damage by injecting flexible material

CONCLUDING REMARKS

This committee report has summarized the geotechnical aspects of current underground construction activities in Japan during the last decade.

In the first half of this paper, proximity construction, excavation machines for braced walls, deep shaft excavation, and groundwater flow preservation measures were selected among the recent technologies, and introduced along with several case records. They have been summarized as follows:

1) During the last decade, a number of deep and large-scale braced excavation projects have been conducted.
2) New excavation machines have been developed especially for handling difficult conditions.
3) Environmental issues concerned with underground water have become important and good case records have accumulated.

In the second half of the paper, statistical data on shield tunneling in Japan collected from a survey was presented. Several case records on shield tunneling and NATM used in urban areas were also introduced. Earthquake-resistant design for underground structure was also discussed. These issues may be summarized as follows:

1) During the last decade, the number of projects using shield tunneling has decreased remarkably and the method has only been used under difficult conditions.
2) NATM has become more widely used for softer ground than before in urban areas.
3) Severe experiences in the event of the 1995 Hyogoken-Nanbu (Kobe) Earthquake have enhanced the Japanese earthquake-resistant design for underground structures.

ACKNOWLEDGEMENTS

The original draft for this committee report was prepared by the members of the Japanese National Committee for TC28, supported by the Japanese Geotechnical Society. The author is very grateful to them for their co-operation.

The names of the committee members supporting this report are:

Braced excavation group: Mr. Shumpei Sasaki, Mr. Yusuke Ishii, Dr. Jiro Takemura, Mr. Masamichi Aoki, Mr. Nobuaki Kosaka, Mr. Kenji Ogata and Mr. Kazushi Fukada.

Tunneling group: Prof. Kazuhito Komiya, Prof. Mitsutaka Sugimoto, Mr. Tohru Yasuda, Mr. Hideo Watahiki, Dr. Hiromasa Igarashi and Mr. Osamu Yoshino.

REFERENCES

1) Haruyama, K., Harada, H., Teramoto, T., Sudoh, T. and Katoh, T. (2001): Design and construction of under-pinning using temporary piling in double-layer tunnel, *Proc. Tunnel Engrg.*, JSCE, **11** (in Japanese).
2) Hashimoto, T., Arimoto, H., Sugimura, T. and Ido, S. (2001): Case records on countermeasures against groundwater flow preservation—Case records in Kyoto and Kobe—, *The Foundation Engrg. and Equipment "Kisoko"*, **29**(11), 41–43 (in Japanese).
3) Ishii, Y. and Abe, S. (2000): Self-supporting walls with soil improvement using cement material, *The Foundation Engrg. and Equipment "Kisoko"*, **28**, 64–66 (in Japanese).
4) Iwata, M., Fukuya, Y., Hagiwara, T. and Yokoo, A. (2000): Shallow overburden NATM tunneling and measurement in urban areas, *Proc. Tunnel Engrg.*, JSCE, **10** (in Japanese).
5) JGS (2001): Learning from troubles: Technologies in design and construction of deep underground structures, *JGS Committee on Technologies in Design and Construction of Deep Underground Structures*, 27–31 (in Japanese).
6) JSCE (1998): Report on the damage due to the 1995 Hyogoken-Nanbu (Kobe) Earthquake, *Tunnels and Underground Structures*, 67–76 (in Japanese).
7) JSCE Kansai Branch (2001): Guideline for earthquake-resistant design for shield tunnel, 129–139 (in Japanese).
8) JSWA (2001): Standard segment for shield tunneling, *Japan Sewage Works Association*, 399–425 (in Japanese).
9) Kawashima, K. (1994): Earthquake-resistant design for underground structure, *Kajima Publication*, 160 (in Japanese).
10) Koizumi, A. (1998): The technical subjects of underground space utilization, *J. of Construction Management and Engrg.*, JSCE, (588/VI-38), 1–20 (in Japanese).
11) Komiya, K., Soga, K., Akagi, H., Hagiwara, T. and Bolton, M. D. (1999): Finite element modeling of excavation and advancement processes of a shield tunneling machine, *Soils and Foundations*, **39**(4), 37–52.
12) Kuroyama, Y., Nishikawa, T., Hino, M., Yoshikawa, T., Morikawa, S., Shimamura, S. and Hamano, T. (1996): Information oriented construction of large-scale excavation works on neighboring tall buildings, *The Foundation Engrg. and Equipment "Kisoko"*, **24**(3), 62–67 (in Japanese).
13) Maeda, H. (1993): Deep shaft for shield tunneling in construction of Kandagawa, underground regulating reservoir beneath Loop Road No. 7, *The Foundation Engrg. and Equipment "Kisoko"*, **21**(7), 76–82 (in Japanese).
14) Miyake, T., Kohsaka, N. and Ishikawa, A. (1996): Technical issues and countermeasures in recharge method, *The Foundation Engrg. and Equipment "Kisoko"*, **24**(2), 60–67 (in Japanese).
15) Miyamae, M., Matsumoto, T. and Suzuki, K. (1997): Diaphragm wall method to be employed beneath existing structures, *The Foundation Engrg. and Equipment "Kisoko"*, 87–90 (in Japanese).
16) Nakagawa, O. (1999): Countermeasures against ground water in Kawasaki manmade island, *Case Records Concerned with Underground Structures and Groundwater*, Japanese Geotechnical Society, 13–24 (in Japanese).
17) Nakajima, S., Asai, T. and Honda, Y. (1998): Subway station excavation adjacent to express highway, *Tunnels and Underground*, **29**(1), 25–31 (in Japanese).
18) Narazaki, M., Okuda, Y. and Tosaka, T. (1995): Tunneling in urban areas using CRD method, *Tunnels and Underground*, **26**(5), 15–25 (in Japanese).
19) Ohmori, M., Yokoo, A., Takahashi, H., Ohya, H. and Arikane, M. (2000): Design and tunneling with large cross-sectional pre-leg improvement, *Proc. Tunnel Engrg.*, JSCE, **10** (in Japanese).
20) Okahara, M., Kimura, Y., Watarai, M. and Inoue, A. (1993): Experiment and analysis on ground displacement due to the braced wall deformation, *Proc. 28th Annual Meeting for JSSMFE*, 2003–2006 (in Japanese).
21) Research Committee for Groundwater and Geo-Environment (RCGG) (2001): Groundwater flow preservation measures, JGS (in Japanese).
22) Sugimoto, M. and Sramoon, A. (2001): Modeling of shield machine movement based on actual driving records, *Proc. JSCE*, (673/III-53) (in Japanese).
23) Sugimoto, T. (1986): Prediction of maximum surface settlement due to braced excavation, *Proc. JSCE*, (373), 37–44 (in Japanese).
24) Takahashi, H. and Amakusa, M. (1998): A machine for diaphragm wall construction using soil cement diaphragm wall (TRD method), *The Foundation Engrg. and Equipment "Kisoko"*, 31–35 (in Japanese).
25) Tanaka, S., Miura, T. and Murayama, S. (1997): Behavior of an

earth retaining structure reinforced by soil improvement, *Tsuchi to Kiso*, 45(10), 17–22 (in Japanese).

26) Uchiyama, N., Katsura, Y. and Kamon, M. (1999): Case studies of buttress-wall type ground improvement in strutted excavations, *Proc. Int. Symp. on Geotech. Aspects of Underground Construction in Soft Ground*, IS-Tokyo '99, 573–580.

27) Ueda, T. (1999): Measures of groundwater preservation in underground construction at Nerima site Gaikan Highway, *Case Records Concerned with Underground Structures and Groundwater*, Japanese Geotechnical Society, 41–52 (in Japanese).

28) Yahagi, S. (2001): Current technology and prospects for shield tunneling methods, *J. of Civil Engrg*. (in Japanese).