GEOTECHNICAL ASPECTS OF CURRENT UNDERGROUND CONSTRUCTION IN JAPAN

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**ABSTRACT**

The Japanese national committee for TC28 has produced a committee report on current underground construction activities Japan, including the technologies of braced excavation and tunneling. This lecture is based on the technical papers a construction records available during the last decade. Among the recent technologies related to the braced excavation a tunneling in Japan, the following topics were selected. With braced excavation, proximity construction, excavation machin for braced walls, deep shaft excavation, and groundwater flow preservation measures were selected and introduced, along wi some case records. Statistical data on shield tunneling in Japan has been assembled from questionnaires. Several case records NATM used in urban areas are also introduced.

**Key Words :** Undergoumd construction, Braced excavation, Shield tunnelling, NATM

## 1 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 60m. 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 near various existing structures.

Furthermore, underground construction is now performed over larger areas, and at greater depths. A larger number of braced excavations of areas wider than 10,000$m^2$ and at depths more than 30m are being conducted, where top-down construction was previously used. When constructing the shafts for urban tunnels, deeper excavation, at more than 50m, is required. Ground water is one of the main concerns when performing deep open excavations in alluvial deposits. In the coastal areas of Tokyo, the ground water level in the aquifer has risen to 50m, its maximum, within the last thirty years, ever since ground water pumping was restricted to prevent land subsidence. Countermeasures against high water pressure are crucial both during and after construction. Another aspect related to ground water 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 there is a long underground structure in an acquifer, which might pose an obstacle to ground water flow.

In this lecture, the author discusses recent technologies relating to braced excavation and tunneling 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.

## 2 PROXIMITY CONSTRUCTION

### 2.1 Prediction of the effect on neighboring structures and countermeasures

Most of the braced excavations in Japan are built in congested urban areas, where the deformation and settlement in the ground associated with the construction and the effects of this deformation on neighboring structures must be considered in the design. Table 1 presents brief outlines and lists applicable conditions of the four methods commonly used for predicting the effects of braced excavation on the 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. Modifications for reducing the deflection of the wall and the settlement of the ground behind the wall are performed in the wall and bracing system, e.g., the length

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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 between 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 interacted. |
| 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. |

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-wall 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).

### *2.2* 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 Figures 1 and 2. The soil profile at the site is alluvial sand with a SPT N value of about 8 (GL $\pm 0 \sim -9$m) and soft alluvial clay (GL-9 ~ -24m), and there is a thick Pleistocene deposit below the top alluvial layers. In this construction, a 20,000m$^2$ area with a depth of 20m was excavated. 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, 3D FEM analysis was first 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 Figure 3, together with the soil profile.

The horizontal displacement vectors and contours of the rebound obtained from the 3D analysis are given in Figure 4.

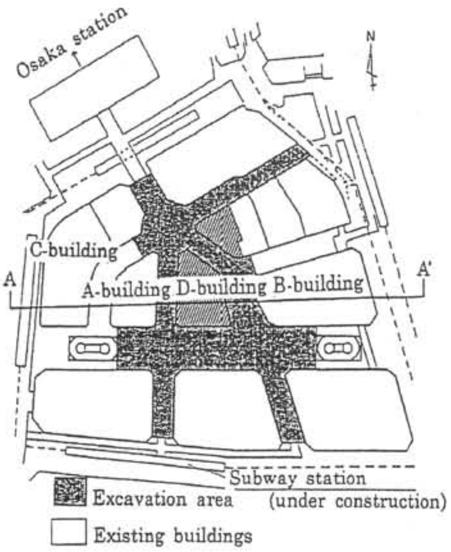



Figure 1. Plan of construction site of large area excavation (Kuroyama, 1996)

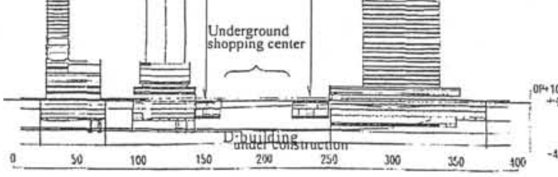



Figure 2. Cross section of construction site (A-A) (Kuroyama, 1996)

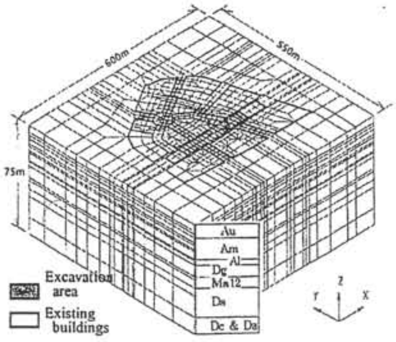
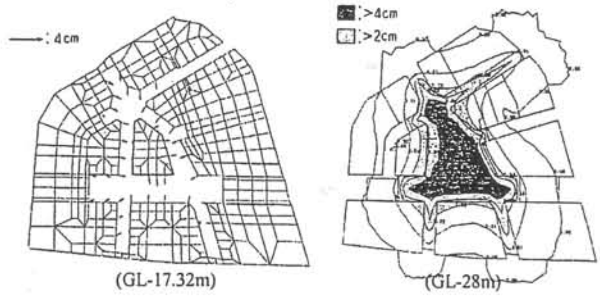



Figure 3. FEM mesh for 3D analysis and soil profile (Kuroyama, 1996)




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

Figure 4. Results of 3D FEM analysis (Kuroyama, 1996)

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 4cm within the excavated area. The rebound of the A-building was expected to be larger than those of the other buildings. A comparison between the 3D and 2D analyses demonstrated that the horizontal movements obtained from the 3D analyses were 50% smaller than those from the 2D, while the magnitude of the rebound in the 3D was greater than in the 2D by 70%. 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.

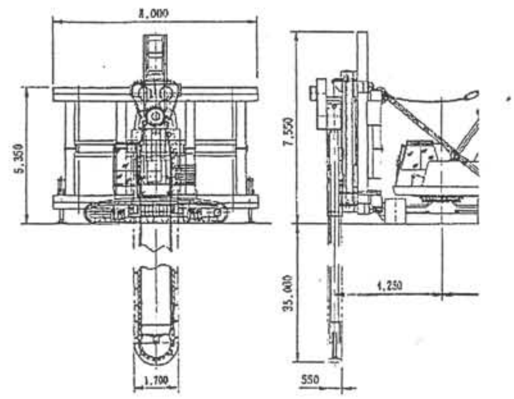



Figure 5. Trench Cutting Re-mixing Deep Wall(Takahasi,1998)

## 3 EXCAVATION MACHINES FOR BRACED WALL CONSTRUCTION

To meet various demands in excavation projects, a variety of excavation machines have been adopeted, which can build braced walls with high accuracy and performance. Table 2 shows the excavation machines used for braced wall construction in urban areas of Japan. Figure 5 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. Figure 6 also shows a recently developed machine, called the "Swing Arm Taisei Twincutter," which can make braced walls under buried structures.

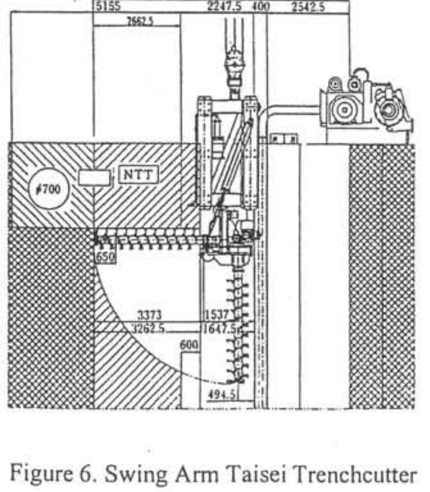



Figure 6. Swing Arm Taisei Trenchcutter (Miyamae,1997)

Table 2. Excavation machines used for braced wall construction

| Type of excavation | Excavation machine | Name of wall | Wall construction method<br>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 |

## 4 DEEP SHAFT EXCAVATION

### 4.1 Recent trends in deep excavations

Figure 7 presents the 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, and shows the continuous increase in wall length since 1978. Walls longer than 50m and 100m were first constructed in 1980 and 1987, respectively. Almost all braced walls longer than 50m were constructed using the diaphragm wall excavation method. Only circular type diaphragm walls were used for excavations deeper than 50m. 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 140m and 82m, respectively (Koizumi, 1998).

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

### 4.2 A case record of a deep shaft construction project (Maeda, 1997)

In deep shaft constructions, chemical grouting is sometimes used to produce a less permeable layer as a countermeasure against heaving or boiling due to high ground water pressure. In the construction of the launching shaft for shield tunneling, which was used for the flood control reservoir, chemical grouting was used, as shown in Figure 8. A circular type diaphragm wall with an inner diameter of 28.2m and a depth of 59.9m was constructed in the ground, under soil conditions shown in the figure. The ground

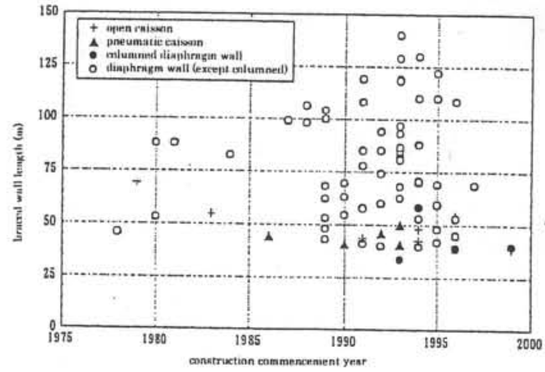



Figure 7. Chronological data on length of braced wall, (JGS, 2001)

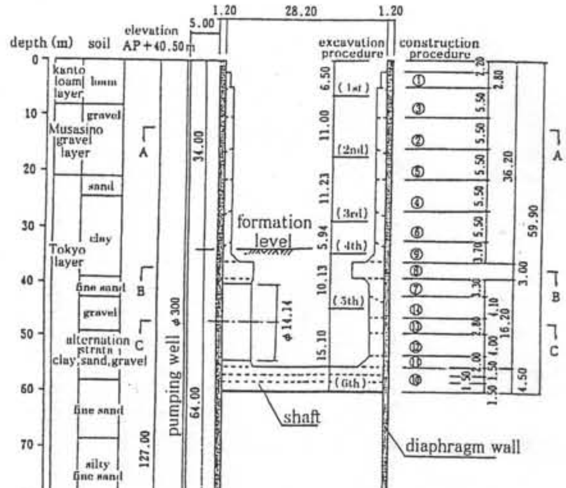
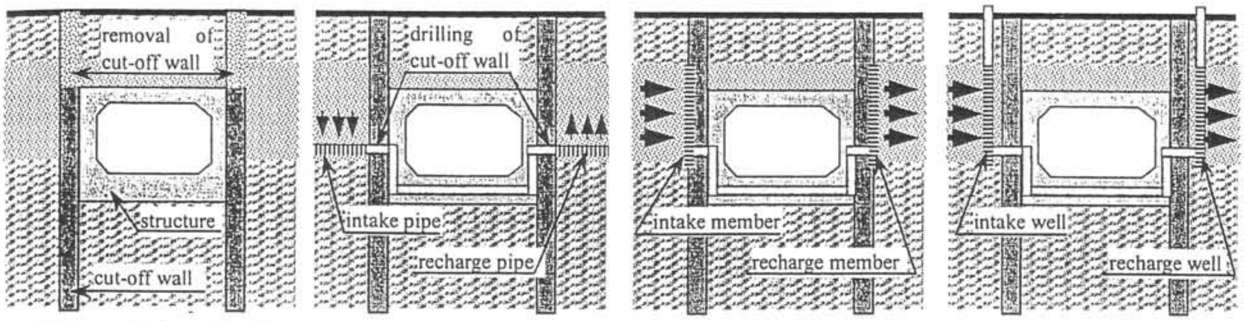



Figure 8. Cross section of deep shaft and soil condition, (Maeda, 1997)




(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 (c) Intake and recharge wells

Figure 9. Groundwater flow preservation using intake and recharge method (RCGG, 2001)

water level in the upper and lower sand layers were GL-10m and 20m respectively. As a countermeasure against heaving, the wall was built to be 98m in length, and chemical grouting of 5m 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 ground water problem was observed and the construction project was completed safely.

## 5 GROUNDWATER FLOW PRESERVATION MEASURES

### 5.1 Interruption of groundwater flow and countermeasures

Because of environmental issues such as noise, vibrations, and preservation of the landscape, urban planners sometimes decide to construct transportation systems like 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 the flow in the aquifers at the site, causing the rise and drawdown of groundwater levels both upstream and downstream. This change of groundwater regime leads to various environmental problems, such as the leakage of water into basements. The change also causes the death of plants upstream, and land subsidence and the drawdown of water in wells 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 upstream is drained through pipes laid in or beneath the structure and is recharged downstream. The following procedure is used. Figure 9 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.

### 5.2 A case record on groundwater flow preservation measures (Ueda, 1999)

A semi-underground box-culvert type highway was constructed using cut-off walls, which penetrated through an aquifer. Since the rise and drawdown of the groundwater level upstream and downstream from the aquifer was expected due to the interruption of the groundwater flow, the following measures were used in the construction (see also Figure 10):

- 3m and 3.5m of the upper portions of the cut-off walls were removed at the upstream and downstream sides of the acquifer respectively.
- Underdrains were built in the longitudinal direction of the box culvert at the upstream and downstream sides of the acquifer.
- 200mm diameter siphon pipes were transversely installed beneath the box culvert every 50m.
- Discharge at the downstream side was conducted through infiltration pits.

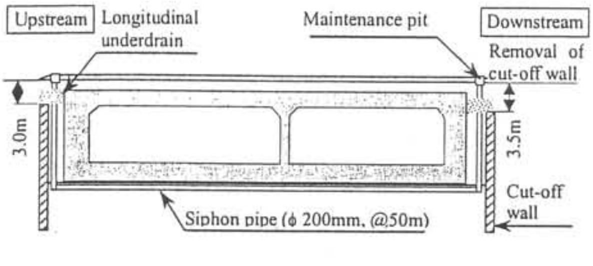



Figure 10. Schematic diagram of a groundwater flow preservation method (Ueda, 1999)

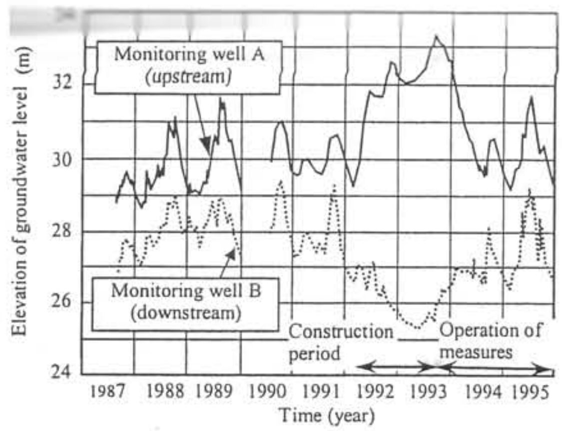



Figure 11. Variations of the groundwater level with time (Ueda, 1999)

The groundwater level was monitored for five years prior to construction, since 1987.

Variations of the groundwater levels monitored at the upstream side (Well A) and downstream side (Well B) are shown in Figure 11. During the construction period, the water level in Well A became elevated, and that in Well B became gradually lower. The difference in the groundwater level between the two wells increased from about 2m to 7m at maximum. However, after the measures to preserve the groundwater flow were adopted in 1993, the difference in the groundwater level gradually decreased. The groundwater levels at the two wells returned to nearly their original levels prior to construction.

## 6 ' INTRODUCTION TO TUNNELING CONSTRUCTION IN JAPAN

During the last decade, the shield tunneling method has been used under difficult conditions, and NATM has been used in softer ground in urban areas. Therefore, this 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) Technology 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 the 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 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 alluvial layers, it has become one of the most commonly used urban area tunneling methods.

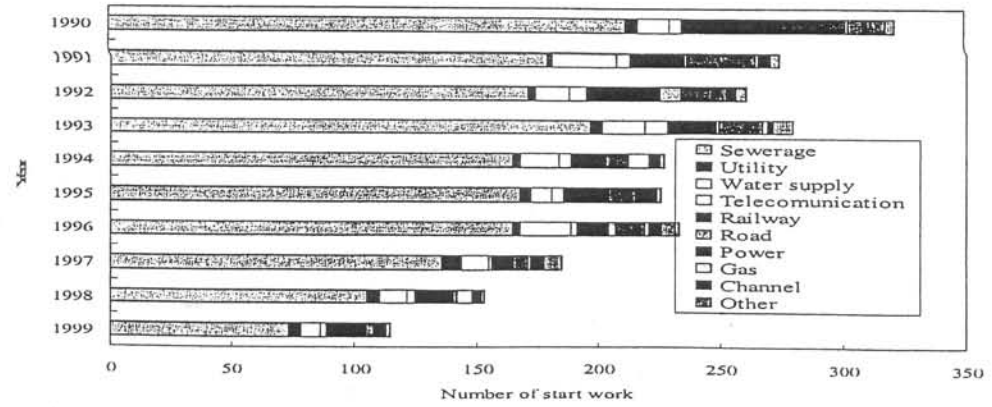



Figure 12. Number of shield tunneling and their usage

## 7 SHIELD TUNNELING METHOD

The statistical features of the shield tunneling projects from 1990 to 1999 can be summarized as follows, based on the survey administered by the Shield Tunnel Association in Japan.

### 7.1 Number of shield tunneling projects performed, and their usage

Figure 12 chronologically lists the number of shield tunneling projects performed, and the usage of the shield tunnels 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.

### 7.2 Shield type

Figure 13 demonstrates the shield types reported in the case records. According to this figure, it can be understood that almost all of the constructions projects adopted the closed type shield machine, that is, the closed type shield machine was used in 97.0% of the projects, the open type shield machine, 2.6%, and other machines, 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%).

### 7.3 Ground type

Figure 14 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 than other types. For gravelly ground, the earth pressure balance machines with mud are more suitable than other machines. These results represent the compatibility of the type of closed shield machines used with the various soil types.

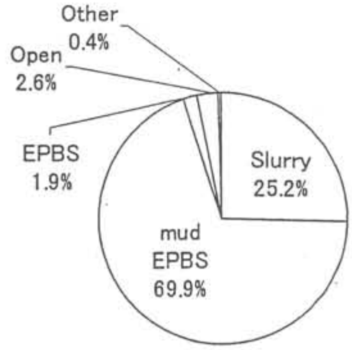



Figure 13. Shield types reported in the case records

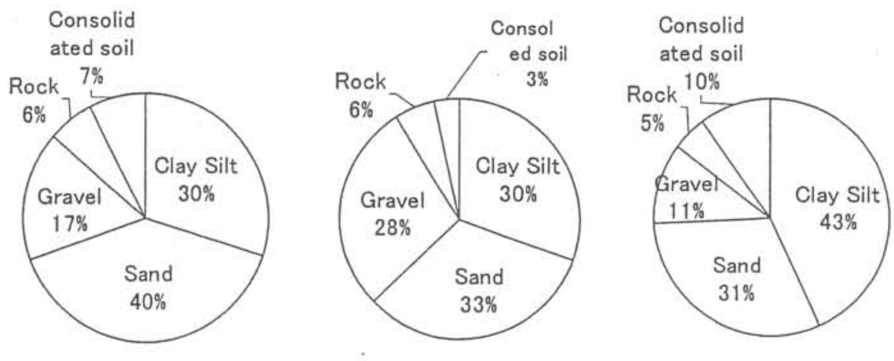
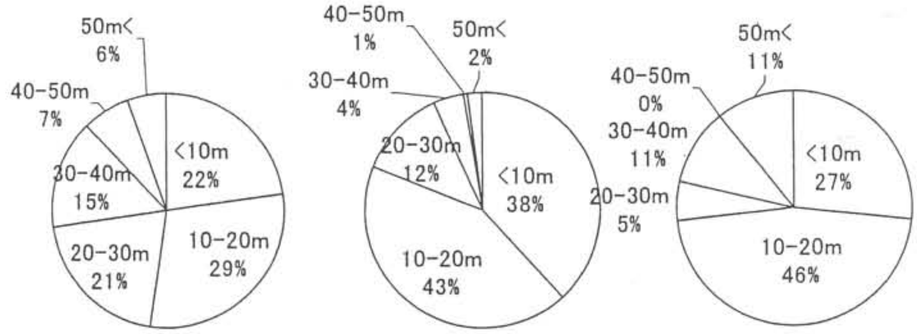
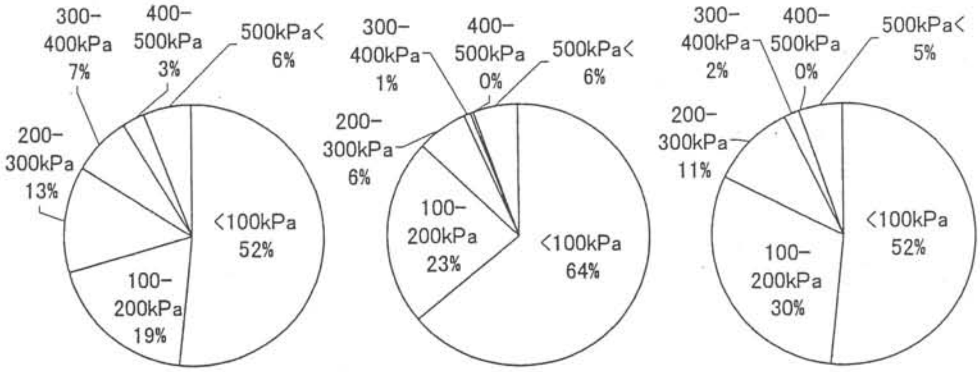



Figure 14. Soil types in the case of closed face type shield machine




Slurry type | Earth pressure with mud type | Earth pressure type

Figure 15. Overburden in the case of closed face type shield machine




Slurry type | Earth pressure with mud type | Earth pressure type

Figure 16. Groundwater pressure in the case of closed face type shield machine

### 7.4 The overburden depth of tunnel and maximum groundwater pressure

Figures 15 and 16 indicate the overburden depth of the tunnels and the maximum groundwater pressure for each closed type shield machine, respectively. It was found that the ratio of construction records with more than a 20 m overburden depth was 49% for the slurry type shield machine, 19% for the mud EPBS, and 23% for the EPBS. The slurry type shield machine is the better type of machine for constructing deeper tunnels.

From Figure 16, it can be seen that the ratio of construction records with more than a 200kPa maximum ground water pressure is 29% for the slurry shield, 13% for the mud EPBS, and 18% for the EPBS. The 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 on the use of 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 a more than 200kPa 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 reported.

## 8 NATM IN 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 was seldom used in alluvium soils, because there are very few cases reportiong its use with alluvium in the case history.

Moreover, it is assumed that the unconfined compressive strength $q_u$=0.1MPa and the elastic modulus $E$=10MPa level are the application lower bounds, as a physical property value of the ground. However, it is not employed in the case of the difficult groundwater condition.

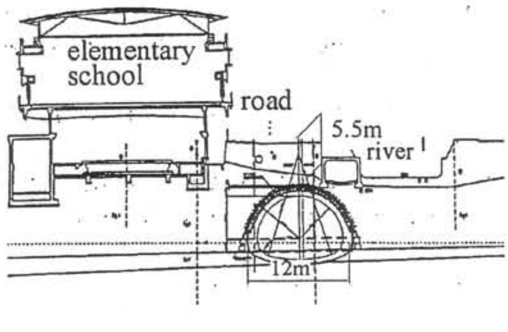



Figure 17. Shin-Kobe tunnel cross section

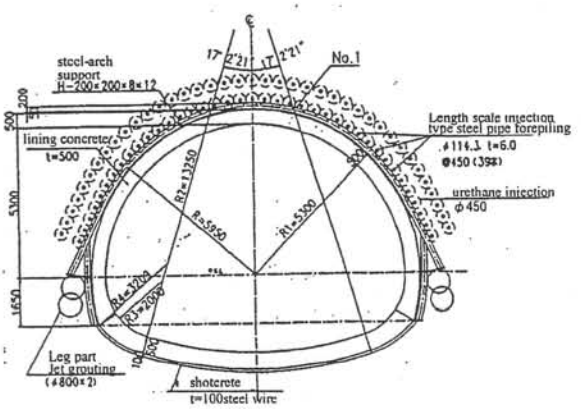



Figure 18. Shin-Kobe tunnel execution scheme

## 8.1 Development of new technology

When the ground is soft, the auxiliary method with Urban NATM is indispensable, because it stablizes the tunnel face (crown and face). Moreover, the auxiliary method is often adopted because of the necessity of controlling the subsidence at the ground surface.

The auxiliary method, which contributes to Urban NATM development, can be classified as follows:

-Fore piling; Jet grout, Long steel pipe injection forepiling, Pre-lining

-Leg piling; Foot pile(horizontal jet grout)

-Face reinforcement; Face reinforcement bolt and pile

## 8.2 Challenge to shallow overburden

Shin-Kobe tunnel is a road tunnel, and the overburden is very shallow, i.e. 4-11m. The typical cross section is shown in Figure 17. In this figure, a road and a river are situated just over the tunnel, and an elementary school is located on

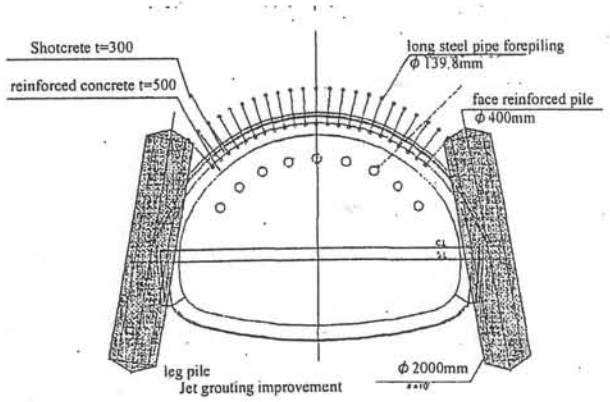



Figure 19 Mito tunnel: typical cross section

the left. Geotechnical features are classified into an alluvial layer, and there is a gravel layer 1.5-3m from the surface. The soil condition is represented as $E$=80MPa 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, the leg horizontal jet grouting methods, and preloading shells, as shown in Figure 18.

The *Trevi tube* is a steel pipe of 12m length, whose lap length for each shift is 6m; 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 the leg horizontal jet grout of $\phi$ 800-1000mm was performed to improve the soil body, whose unconfined strength was about $q_u$=10MPa, as measured seven days after the injection. According to the measurement result, an initial subsidence of 3-5mm was observed at the ground surface, and the final subsidence was 12mm, observed around 15m behind the face.

The Mito tunnel is a road tunnel. The excavating cross sectional area exceeds 100 $m^2$, and the over-burden depth is also shallow, i.e. 6.6-12.8m. The road and lifeline 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 Quaternary Diluvium Epoch, and the ground is composed of the cohesive soil layer and grits layer. The cohesive soil layer appears at the tunnel face. The SPT N value is around eight on the average, and the elastic modulus of the soil is about $E$=48MPa. The most important problem in this project was to reduce the influence on the ground surface as much as possible.

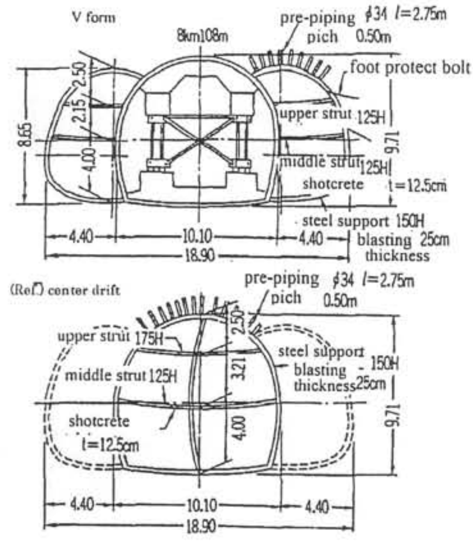



Figure 20. Narashino-dai tunnel station: large tunnel cross section

The crossing jet method was adopted as a countermeasure in order to conduct the large diameter pre-leg improvement, as shown in Figure 19, which was a kind of the soil mixing method with a high-pressure jet.

The measurement results are as follows. The preceding displacement 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 (17mm).

### 8.3 The challege of tunneling in soft ground

The Narashino-dai tunnel is a railway tunnel with an extension of 2.36km, and is located in Kita-Narashino on the Toyo Rapid Transit line, which connects Tokyo and Chiba Prefecture. The geotechnical features of the ground were the unconsolidated soft Narita sand beds with groundwater, whose elastic modulus was $E$=21～59MPa and cohesion was 0～0.03MPa. The uniformity coefficient was small, and it was assumed that tunnel collapse would take place due to the quicksand phenomena.

The CRD method (Cross Diaphragm Method) was developed in order to solve this problem. This method follows the variation of 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 steel pipe pillar. After the side drifts on both sides were completed, the entire tunnel section was completed, as shown in Figure 20. The supplementary methods adopted were the deep well tunneling method and a chemical grouting of 3m thickness around the tunnel.

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

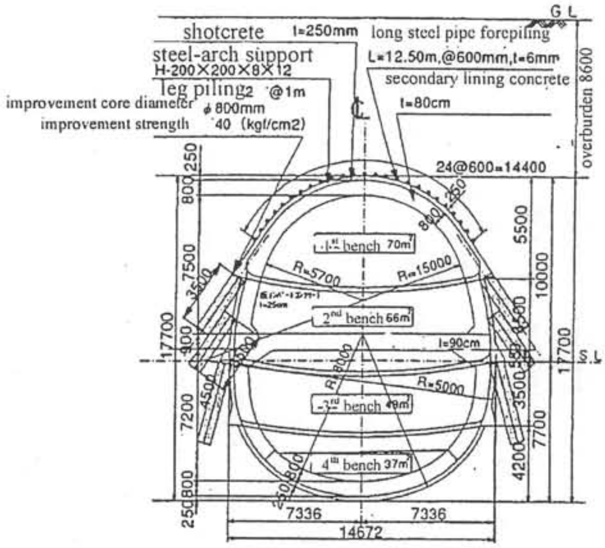



Figure 21. Oume tunnel : Initial 2 layer structural section

### 8.4 The challenge of building 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 excavating cross sectional area was 230m$^2$ on average. The geotechnical features to be excavated were the riverside terraces, which were composed of a terrace boulder layer and fine-grained Kanto loam layer.

The excavation method initially planned was the two steps bench and upper lining bridge support system, as shown in Figure 21. However, it was predicted from the analysis of the results of measurements during the first and the 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 the method, which uses a temporary bearing pile, as shown in Figure 22. With regard to the auxiliary method, the horizontal jet grouting and the foot pile method were adopted.

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

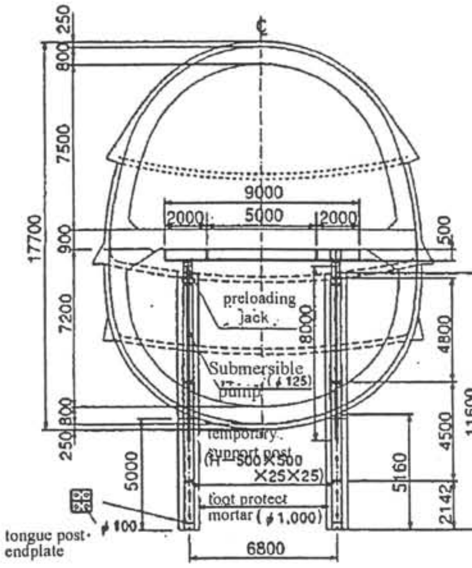



Figure 22. Oume tunnel temporary support post (Final design)

## 9 CONCLUDING REMARKS

This paper summarized the geotechnical aspects of current underground construction activities in Japan during the last decade. In the first half of this paper, the proximity construction, excavation machines for braced wall, deep shaft excavation, and groundwater flow preservation measures were selected among the recent technologies, and introduced along with several case records. Then, statistical data on shield tunneling in Japan was collected from the survey that had been administered. Several case records on NATM used in urban areas were also introduced.

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