Current Underground Construction Technology in Tokyo

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SYNOPSIS : This paper summarizes two topics of the recent case histories of shield TBM tunneling and underground construction technology in the Tokyo metropolitan area. The first topic is to overview the latest construction of Tokyo Metro subway within the central Tokyo area, the Fukutoshin line, using mainly a shield tunneling method. Technical challenges employed for the construction of Fukutoshin line are as follows: an oval shaped cross-section tunnel, a segment fixing expander, a parent-kid shield TBM, a wide length boltless segment and a new refilling material for the station shaft using an industrial waste and clay slurry during TBM driving. Another topic is to introduce the countermeasures against liquefaction of the foundation soil below existing superstructures in the Tokyo metropolitan area using the chemical grouting method. The universal grouting method for the soil improvement just below the existing structures is described and the evaluation procedure of mechanical properties of chemically stabilized soil using a new sensor is indicated. Finally, the systematic long term field and laboratory measurement results of the mechanical characteristics of the chemically stabilized sand with grouting are demonstrated.

Key words : underground construction, shield tunneling method, liquefaction, soil improvement, chemical grouting method

1. Introduction

The infrastructure of mega cities (and new capitals that eventually may become mega cities) requires geotechnical engineers to have distinct expertise, as they have to deal with a variety of geotechnical fields and areas. The high cost of land results in high-density urbanization with high-rise buildings and expensive foundations; in certain communities, the poor population is forced to settle in inconvenient and cheap areas that have high geotechnical risks, such as unstable grounds in steep natural slopes and marshy and flood zones. The congestion in urban areas necessitates the use of underground space; shortage of land forces the use of abandoned industrial districts as residential areas in which there are inherent environmental risks for living on potentially contaminated ground. One should invite geotechnical engineers to share their experience on foundations, slope stability, soft soils, tunnels, excavations, environmental geotechnics, etc.

This paper summarizes two topics of the recent case histories of shield TBM tunneling and underground construction technology in the Tokyo metropolitan area, i.e. one of the mega cities in Japan. The first topic is to overview the latest construction of Tokyo Metro subway within the central Tokyo area, the Fukutoshin line, using mainly a shield tunneling method. Another topic is to introduce the countermeasures against liquefaction of the foundation soil below existing superstructures in the Tokyo metropolitan area using the chemical grouting method.

2. Tokyo Metropolitan Subway

2.1 New subway in central Tokyo

The Fukutoshin Line (subway No.13 in Tokyo), the section extending south from Ikebukuro, with Shibuya as the terminal, was originally planned by the Policy Report No. 7 of the Council for Japan Transportation**.** Subsequently,

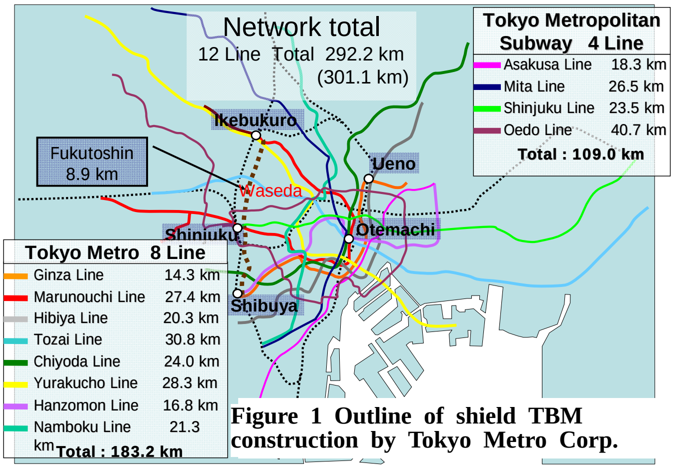


Figure 1 Outline of shield TBM construction by Tokyo Metro Corp.

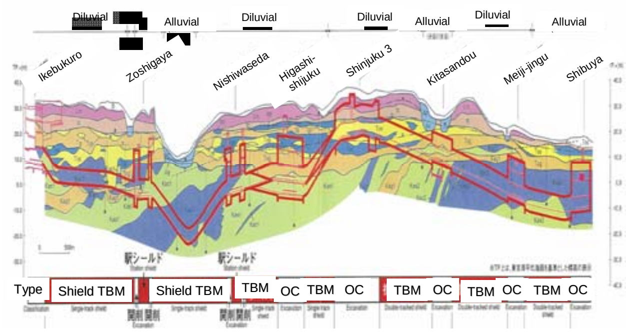


Figure 2 Geological conditions along the Fukutoshin line

Policy Report No. 18 formulated in January, 2000 planned the implementation of a reciprocal through-service with the Tokyu Toyoko Line. As a part of this report, the sections between Shiki and Wakoshi, on the Tobu Tojo Line, and from Wakoshi to Ikebukuro on the Yurakucho Line (the section between Kotake-mukaihara and Ikebukuro has a quadruple track line) are already in operation. The 8.9 km Ikebukuro-Shibuya section has been constructed by the Tokyo Metro Corporation, as shown in Fig. 1. The Fukutoshin Line is made up of the following sections: between Wakoshi and Kotake-mukaihara (common section, shared with the Yurakucho Line), between Kotake-mukaihara and Ikebukuro (quadruple track line, already in use) and between Ikebukuro and Shibuya (recently available for use).

2.2 Construction procedure and technical challenges

The construction work on six stations, other than Zoshigaya Station, was being carried out below Meiji Street, which was congested with pedestrians and vehicles. The tunnel connecting Ikebukuro and Shibuya was complete, and the construction work for the opening of the Fukutoshin Line has finished on schedule, June, 2008. As always, safety has been the top priority during the underground construction. Geologically the Fukutoshin Line was mostly constructed in diluvial ground except for a small area of alluvial ground, as shown in Fig. 2. Nine tunnel sections have been built using Shield TBM machines, and the average depth of overburden is around 20 m. The following were the technical challenges faced during the construction of the Fukutoshin Line.

Figure 3 shows the oval-shaped tunnel cross section, which reduces the soil volume due to the slurry-type TBM excavation. Specially designed TBM and tunnel segment linings were employed for this oval-shaped cross section.

In order to construct stable tunnel segments behind the TBM, a specially designed apparatus has been used, as shown in Fig. 4. This apparatus has been successfully

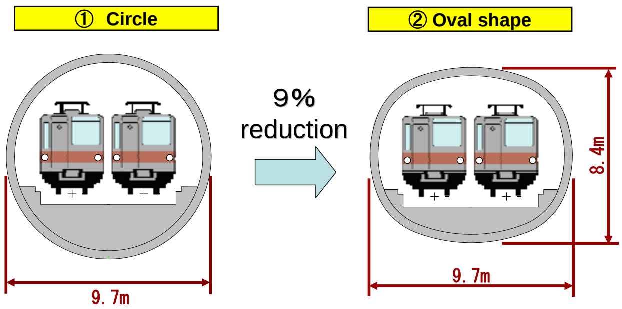


Figure 3 Oval shaped cross section tunnel

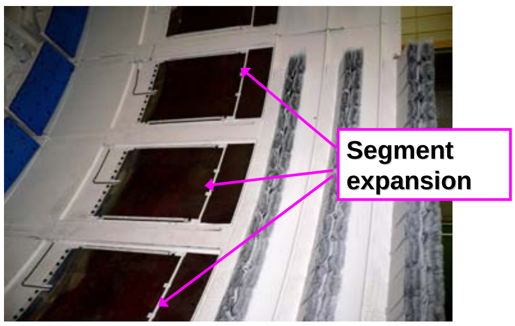


Figure 4 Segment expansion

※ Increase in segment width

1.5 m → 1.6 m

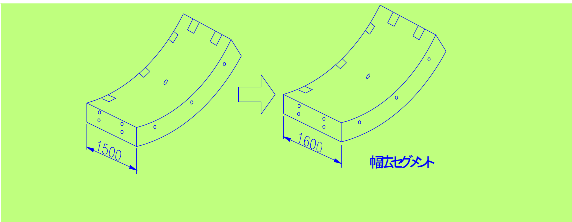


Figure 5 Increase in segment width

Figure 6 Employment of slurry and industrial waste

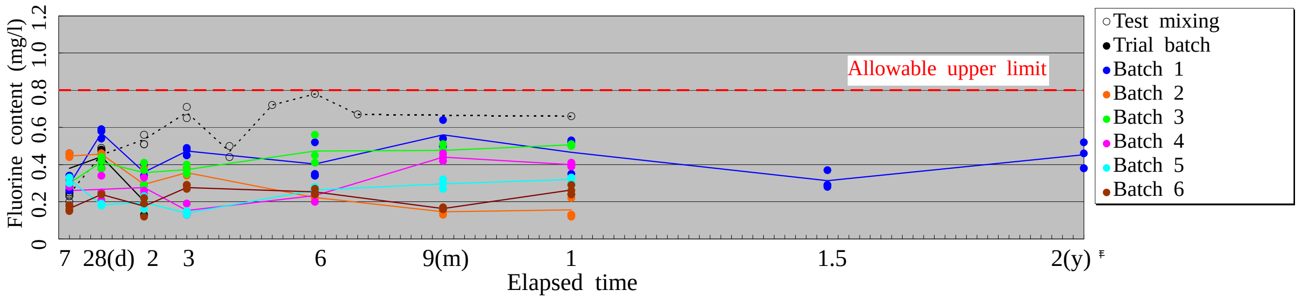


Figure 7 Time-dependent variation of fluorine content leached from filling material

used to expand the segmental linings behind the tail of the TBM during the TBM driving, in order to fix them into the suitable positions.

The width of the tunnel segment was also increased in order to reduce the number of segments needed and the construction cost, as shown in Fig. 5.

Controlling the amount of soil removed during the construction involved in the shield tunneling and the station shaft excavation has contributed to reducing the impact on the environment. The excess clay slurry and the excavated soil from the slurry shield tunneling was reused with industrial wastes as the filling material for the invert section of the shield tunnel and as the refilling material for the station shaft, as shown in Fig. 6. Air cooled blast furnace slag material was used an industrial waste to be mixed with the excess clay slurry and a gypsum cement material. Air cooled blast furnace material contains a lot of fluorine material and the leaching of the fluorine material is assumed to take place after the installation of the mixed materials into the station shaft. Figure 7 indicates the monitoring results of fluorine content leached from the refilling materials from the installation period, which was conducted in 6 stages of batch installation. The measured values of leached fluorine content were below the allowable upper values defined by the legal regulation in Japan for the period of two years.

3. Long Term Field Monitoring of Chemically Grouted Sand

The purpose of employing chemical grouting method as a supplementary method for underground construction in urban area is to achieve the short-term effects. Hence, there are few case studies on its long term performance. As a result, the duration at which the injected chemical grout material maintained its performance within the soil is not fully understood, since there were no follow-up surveys. From the laboratory experimental results, several researchers reported that there was no long term durability of small sand specimens stabilized with common types of water glass material. On the other hand, the long term good performance of the soil mass stabilized with chemical grouting has been qualitatively reported from the field observation results, where the excavation of chemically stabilized soil mass has been fortunately conducted after the completion of the project. Currently, the application of chemical grouting method is increasingly demanded in cases, where the long term durability of chemically stabilized soil is anticipated to be a countermeasure against the liquefaction of the foundation soil particularly beneath the existing superstructures.

In order to ensure the long term performance of chemically stabilized sand with grouting, the systematic field measurement of the mechanical characteristics of the chemical grouted sandy soil has been carried out by the Japanese Grouting Association for the period of around 10 years, from the beginning of 2000 to the end of 2009. This paper summarizes and discusses the field monitoring test program and the measurement results for these 10 years period performance of chemically stabilized sand with grouting.

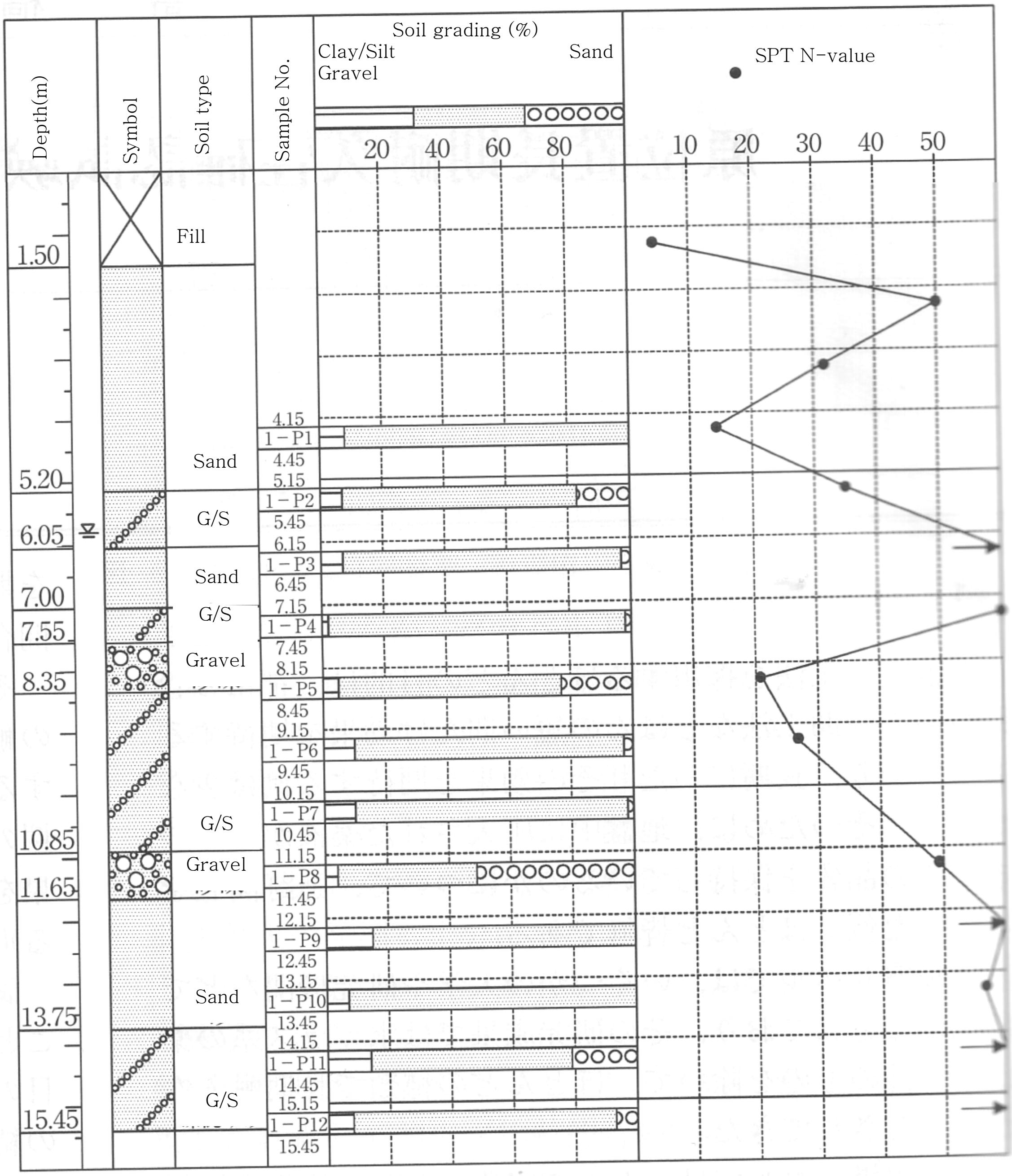


Fig.8 Soil profile, soil composition and SPT N-value of test field

3.1 Preliminary survey and testing program

The field test site is located at Ibaraki prefecture along the coast of the Pacific Ocean, north of Tokyo. The soil layer profile, the soil grading and the SPT N-value measurement results are indicated in Fig.8. The sandy soil layer prevails over the whole test field zone. Small gravel grain is mixed with the sand layer and a large amount of gravel is partly found. According to the grain-size analysis results, the soil composition is mainly that of sandy soil and fine grained material content is less than 10%, but in certain parts of the layer, the gravel content is between 25 and 50 %. The N value ranges within 14-17 down to GL-5.0m and is found to be comparatively loose. Below GL-5.0m, N value is increased to be 20-50 and the soil is at dense condition. The underground water level is GL -5.7m to -5.8m and the seasonal fluctuation of the depth is 5.0 to 6.1m. The initial permeability constant is 1.2×10^{-2} (cm/s) on average and is an almost homogeneous within the test site.

Three types of silicate grouting materials, “Type A: inorganic alkaline material”, “Type B: organic alkaline material” and “Type C: neutral and acid material” were employed dependent on the type of hardening material. A small amount of “Type D: silica colloid material” was injected just for the lateral loading test at the 5 years measurement. The injecting method adopted was a “double-packer method”, which guarantees a high quality outcome that ensures the uniform

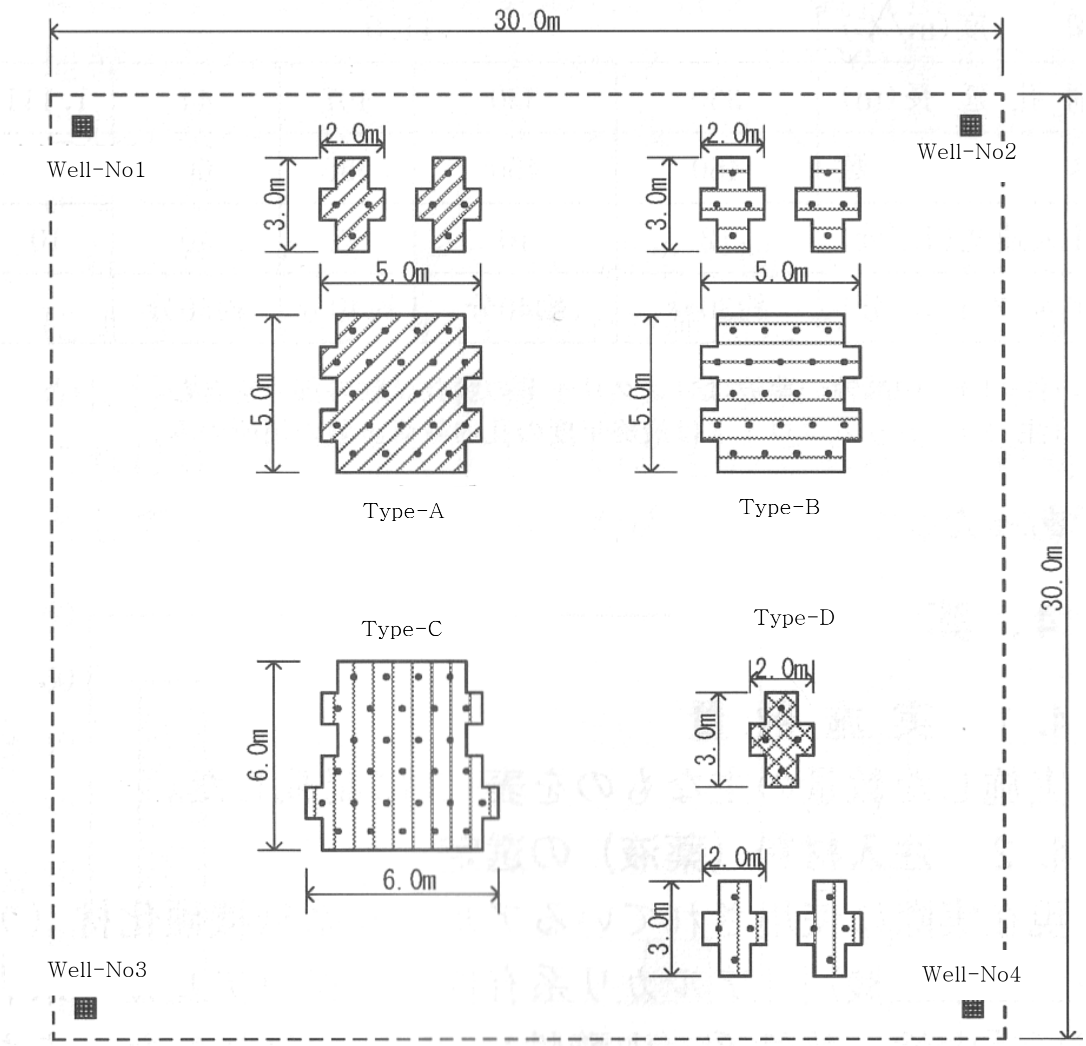


Fig.9 Plan of the field test site

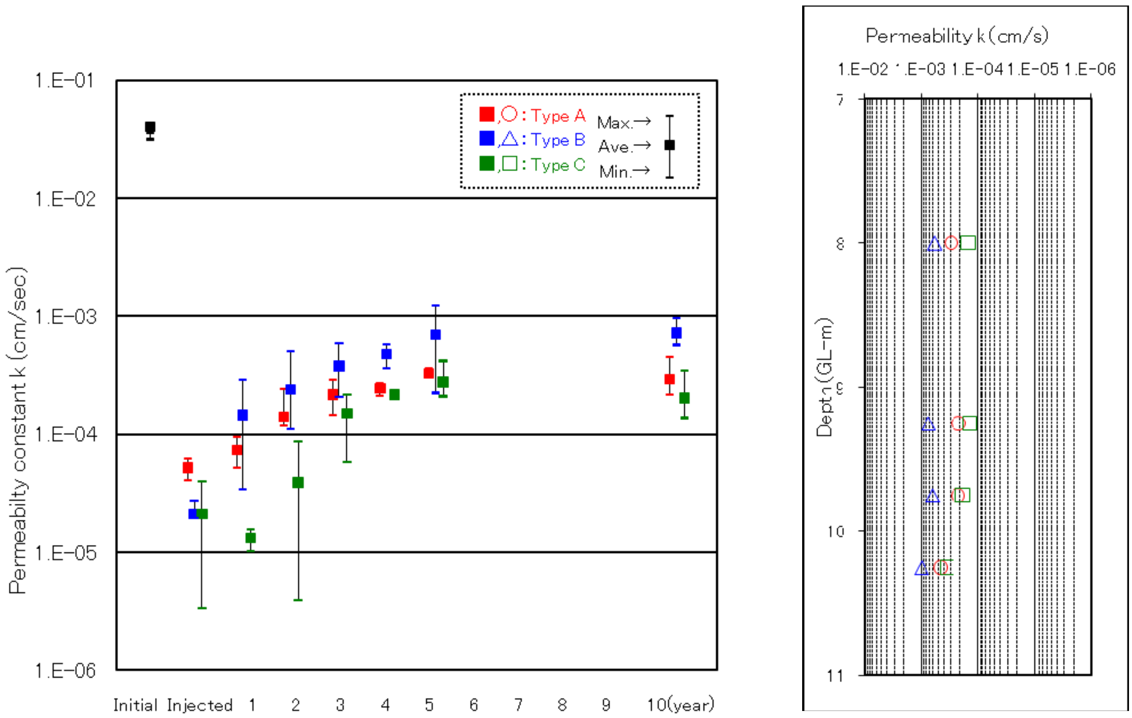
improvement of the target sand layer as much as possible. However, the commonly used CB (Cement-Bentonite material) grouting to improve the resistance against the leakage of injected material was omitted.

The injection for each grouting material was basically conducted on a test field site soil mass plan of 5m×5m, as shown Fig. 9. The grouting depth was selected to be GL-6.0m to -11.0m. This depth was determined because the water table level was about GL-5.8 m and the upper end of the injection area had to be deeper than the water table. The ratio of the grouting material volume against the soil volume was 43%. The spacing between the injection holes adopted was 1.0 m pitch, injection step length: 33 cm, injection rate: 12 l/min, gel time: 30 to 50 min, the injection pressure: 1 to 1.5 MPa on average and the injection was controlled at a fixed injection rate of 144 l/step length.

In-situ permeability tests, lateral loading tests (LLT) and standard penetration tests (SPT) within a bored hole were conducted to obtain the permeability and strength characteristics of the grouted sand. These measurements have been performed once a year for the initial five years and at the end of 2009. At the ten years measurement at the end of 2009, large diameter tube sampling (Diameter, 136mm) were also carried out. Unconfined compression tests and quantification analysis of silicate content using Inductive Coupled Plasma method (ICP) were conducted on tube sampled specimens.

3.2 Field measurement results and discussion

The measurement results for this ten years period are demonstrated and discussed as follows. Figures 10(a) and (b) indicate the time-dependent variation of in-situ permeability constant values and their vertical distribution obtained from 10 years measurement. The in-situ permeability is reduced by around 3 orders with the injection of the grouting material and then increases gradually each year. Particularly, Type C: neutral and acid material values have been kept to be smaller than those of other grouting materials, which demonstrate the effectiveness of removing the sodium ions from the silicate materials, as in Type C. The average of in-situ permeability constant values obtained from 10 years measurement is around 2.0×10^{-4} to 7.2×10^{-4} (cm/s) and the distribution of in-situ permeability is almost constant throughout the depth for each grouting material. Therefore, the cutoff capacity of the chemically stabilized soil mass

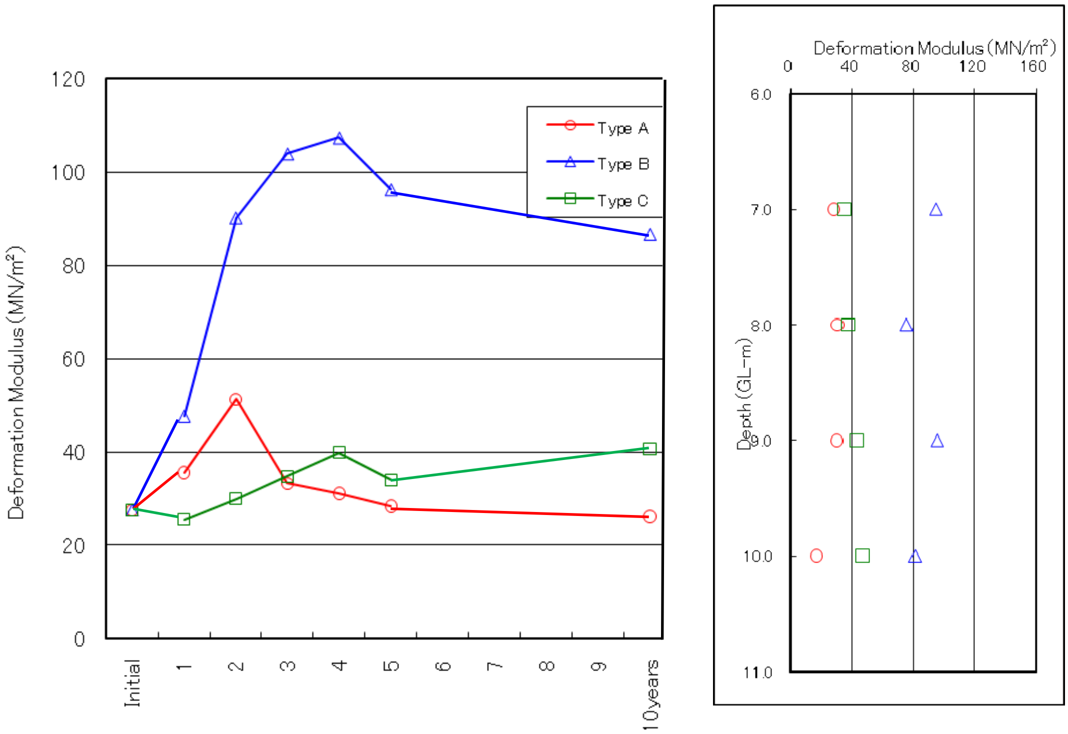


(a) Time-dependent Variation of In-situ permeability (b) Vertical Distribution of In-situ permeability

Fig.10 In-situ permeability test results

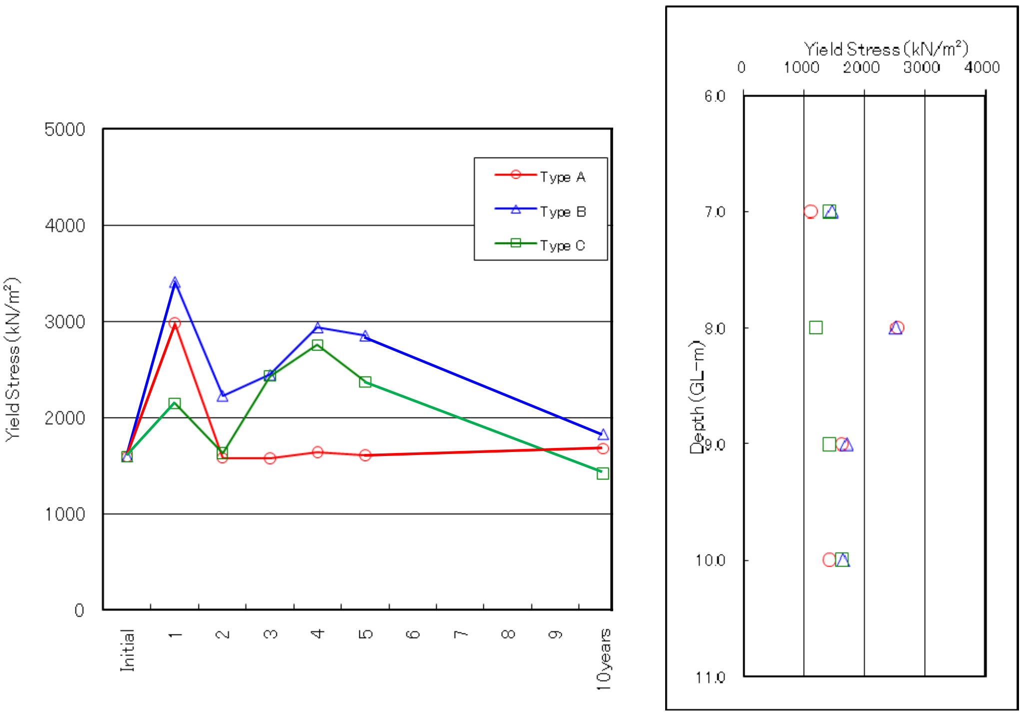
remains to be kept by the injection of silicate materials for these 10 years period.

Figures 11(a) and (b) demonstrate the time-dependent variation and the vertical distribution of the deformation



(a)Time-dependent Variation of Deformation Modulus (b)Vertical Distribution of Deformation Modulus at 10 years

Fig.11 Deformation Modulus from LLT tests



(a) Time-dependent Variation of Yield stress (b) Vertical Distribution of Yield stress at 10 years

Fig.12 Yield stress from LLT tests

modulus obtained from LLT tests within a bore hole. The deformation modulus of the chemical injected materials experienced a comparatively small change except for the Type B: organic alkaline material. In the case of Type B material, the deformation modulus increases up to the value, three times of the initial value particularly for the period from the first year to fourth year due to the development of chemical reaction. Figures 12(a) and (b) show the time-dependent variation and the vertical distribution of the yield stress from LLT tests. The values of yield stress for all the grouting materials become larger than their initial values during the first year. However, in the second year, the Type A material values were decreased to almost the same as the initial value, while both of the Type B and Type C material values increase to about 1.5 times of the initial values. At the time of 10 years measurement, the yield stress values for all the grouting materials returned to their initial ones.

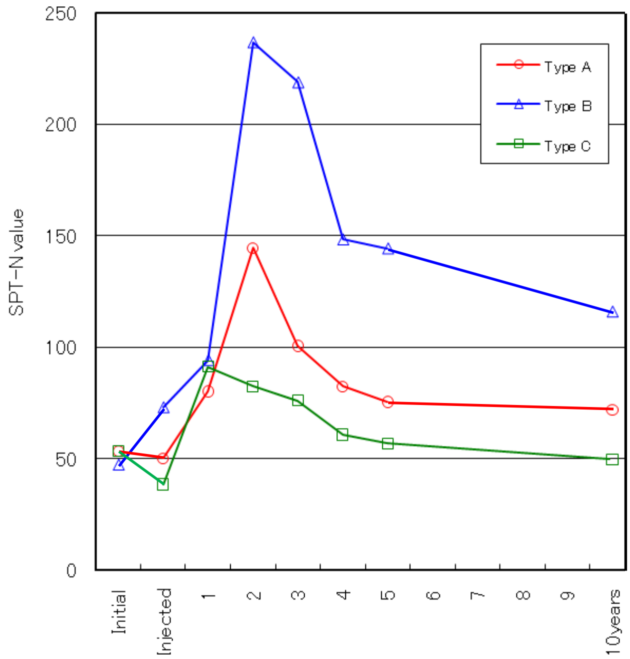


Fig.13 Time-dependent variation of SPT-N value

Figure 13 indicates the time-dependent variation of SPT-N value for all the grouting materials. These are the average of SPT-N values measured at 5 depths for each grouting material. SPT N-value increases greatly over time more than the value obtained at just after the injection. As for the Type B: organic alkaline material, the results indicate twice the value of N compared with other injected materials immediately after grouting and has been proven to have high strength characteristics. The SPT-N value for Type C: neutral-acid material increases at initial stage and decreases gradually to the initial one at ten years measurement.

SPT-N value is known to be greatly influenced by the local soil grading characteristics, i.e. if a hard gravel grain exists just below the base of drop hammer, the quite great N-value may be obtained. Comparing Figures 11, 12 and 13, quite similar time-dependent variations of LLT and SPT results are obtained. Therefore, these time-dependent variations of strength and deformation characteristics of chemically stabilized sand with three types of grouting materials are assumed to be representing the field time-dependent behavior.

Table 1 summarizes the unconfined compression test results on tube sampled specimens, whose ages are 10 years old. The unconfined compression strength q_u values of Type A and Type C specimens are much smaller than those of Type B

Table1 Unconfined Compression Strength q_u (at 10 years)

	Type A	Type B	Type C
	q_u(kN/m^2)	q_u(kN/m^2)	q_u(kN/m^2)
Sample 1	19.4	172.6	50.3
Sample 2	10.9	83.0	—
Sample 3	25.2	96.0	—
Average	18.5	117.2	50.3

Table 2　Silica content（at 10 years）

	Silica content (mg/g)	Silica increment (mg/g)
Type A	6.64	4.78
Type B	12.40	10.54
Type C	8.17	6.31
Initial silica content	1.86	−

specimen. The average q_u value of Type B specimen is greater than 100(kN/m^2) and is assumed to have enough shear strength against liquefaction. These q_u values of chemically stabilized specimen are correspondent to the in-situ LLT and SPT results shown in Figs. 11, 12 and 13.

Table 2 demonstrates the silica content values of sampled chemically stabilized sand specimens with three types of grouting materials. Silica increment is the difference between the silica content value of chemically stabilized specimen and the initial silica content value obtained from the specimen without chemical grouting. This increment value represents the silicate grouting material content, remaining within the soil skeleton at 10 years measurement. The increment value of Type B is much greater than those of two other types of grouting material. Comparing Tables 1 and 2, silica increment values for three types of grouting materials are correspondent to the q_u values, i.e. in-situ LLT and SPT results. The greater the remaining silica increment values within the soil skeleton, the greater the strength and deformation characteristics of chemically stabilized sand with grouting. From the view point of cutoff capacity of chemically stabilized sand, around 5(mg/g) of silica increment values are enough to secure the permeability constant value of the order of 10^{-4}(cm/s).

4. Concluding Remarks

This paper summarizes two topics of the recent case histories of shield TBM tunneling and underground construction technology in the Tokyo metropolitan area. The concluding remarks are summarized as follows.

1) Technical challenges employed for the construction of Fukutoshin line have been successfully achieved, including an oval shaped cross-section tunnel, a segment fixing expander, a parent-kid shield TBM, a wide length boltless segment and a new refilling material for the station shaft using an industrial waste and an excavated soil during TBM driving.

2) The average of in-situ permeability constant values obtained from 10 years measurement is around 2.0×10^{-4} to 7.2×10^{-4} (cm/s) and the distribution of in-situ permeability is almost constant throughout the depth for each grouting material. The cutoff capacity of the chemically stabilized soil mass remains to be kept by the injection of silicate materials for these 10 years period.

3) In the case of Type B: alkaline organic material, the deformation modulus obtained from LLT test increases up to the value, around three times of the initial value particularly for the period from the first year to second year due to the development of chemical reaction. At the time of 10 years measurement, the yield stress values for all the grouting materials returned to their initial ones.

4) The average q_u value of Type B specimen is greater than 100(kN/m^2) and is assumed to be enough shear strength against liquefaction.

5) The greater the remaining silica increment values within the soil skeleton, the greater the strength and deformation characteristics of chemically stabilized sand with grouting. From the view point of cutoff capacity of chemically stabilized sand, around 5(mg/g) of silica increment values are enough to secure the permeability constant value of the order of 10^{-4}(cm/s).

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