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CONSTRUCTION WORK FOR SUBWAY TUNNELS BY SHIELD OR CUT-AND-COVER METHODS AND PROTECTION METHOD FOR STRUCTURES ADJACENT TO THE TUNNELS

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SUMMARY

This paper discusses how to protect the principal structures adjacent to subway tunnels being constructed by the shield or cut-and-cover methods by referring to some actual examples.

CONSTRUCTION METHODS FOR SUBWAY TUNNELS

There are several construction methods for subway tunnels, such as cut-and-cover method, shield method, NATAM method, mountain tunnel method, caisson method, etc. if roughly classified. Normally, we employ the cut-and-cover method for subway stations and the shield method for the tunnels between subway stations because of economic reasons and of reducing the road traffic disturbances to a minimum during construction works.

In the shield method, a tunnel is made by excavating a lateral bore hole similarly to the construction of a mountain tunnel. The shield method is characteristic in that cylindrical steel shells are used for strutting with earth supporting purpose.

There are several kinds of shield methods such as open type shield, pneumatic shield, blind type shield, slurry type shield, earth pressure type shield, etc. depending on the method of suppressing earth pressure and water pressure at the face in front of the cylindrical shells being driven.

The shield method has been developed with the times and improved in the direction to reduce the influence into adjacent ground but its construction costs tend to increase.

MECHANISM OF THE EFFECT OF SHIELD METHOD ON ADJACENT GROUND

The effect of shield method on adjacent ground

Though it is difficult to make the ground settlement due to shield method completely zero, it is now possible to construct tunnels by the shield method with the smallest ground settlement by selecting the kind of shield method the most suited to the ground conditions and by carrying out the work very carefully as a result of the technological development in recent years. The effect of the shield method on the adjacent ground can be analyzed as follows:

(a) Elastic and elasto-plastic deformation of natural ground due to the short time release of ground stresses at the face and tail void as shown in Fig. 1.

(b) If the face is incorrectly supported, an earth volume more than required is taken into the shield and the adjacent natural ground may be disturbed.

(c) Since the shield machine is pushed forward by a great driving force, the adjacent natural ground is affected by the force and disturbed.

(d) If the filling material for tail void is insufficient in both the quantity and quality, then this may cause the settlement of ground surface.

(e) If groundwater level is changed by pneumatic method and air interruption, then this may become the cause of settlement of consolidated ground surface.

(f) The shield tunnel is generally made by connecting the shield segments together as shown in Fig. 2, and thus the rigidity of this structure is small and the tunnel section is deformed even though the load is being applied in accordance with design and calculations, by which the tunnel section is deformed and adjacent natural ground is affected.

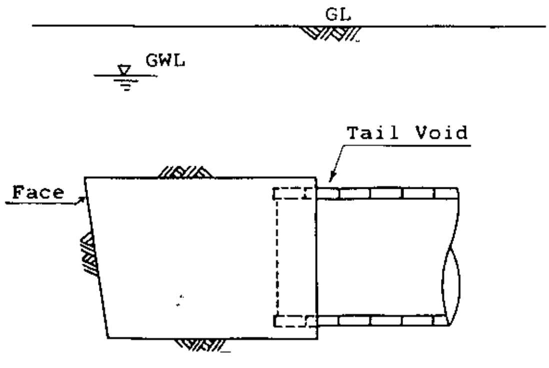

Fig.1 Shield Machine

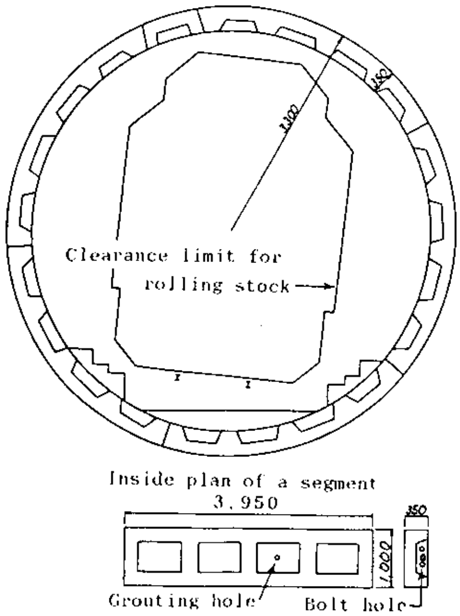

Fig.2 Reinforced Concrete Segment

Range of ground surface settlement due to shield method

The range of ground surface settlement of the lateral section due to the shield method is considered to be as shown in the Fig. 3.

Ground surface settlement due to the shield method normally increases very gradually and reaches to the final value. Where earth covering is small or blind type shield method is adopted, front rising on ground surface sometimes occurs.

It is very difficult to determine the final settlement of ground surface and the range of ground surface settlement of alluvial clayey layer. These values are normally affected by the soil conditions of ground, groundwater level, shield diameter, earth covering, type of shield method, workmanship in construction work, etc. The final settlement and the range of ground surface settlement are often calculated by using the finite element analysis method.

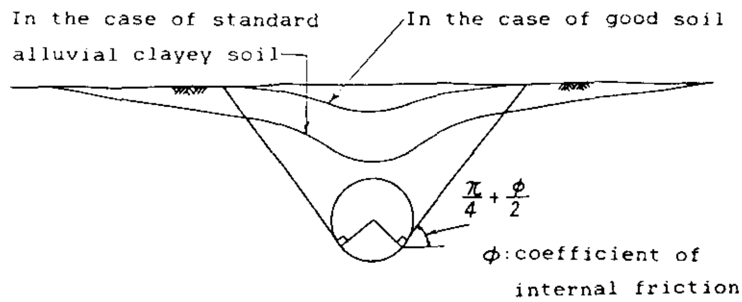

Fig.3 Figure of Ground Level Settlement

Concepts of reducing the influence of shield method upon the adjacent ground

The first important point for reducing the effect of shield method upon the adjacent ground is how to maintain the stability of the face at the front during shield excavation and driving.

The second point is how to excavate the earth, the amount of which should be just same as the volume of shield machine.

The third point is how to fill up the tail void at the same time with the driving of shield machine, which is created by the difference between the outside diameter of shield machine and the outside diameter of tunnel which is created by the segment assembly.

It is almost impossible to completely clear the three points stated above by using the present state-of-the-art though this may depend upon the soil conditions of ground and the selection of shield machine.

Therefore, the principal means of reducing the influence of the shield method upon the adjacent ground is to reinforce the adjacent ground by chemical grouting before shield driving and also to reinforce the adjacent loosened ground immediately after the shield driving.

A SINGLE TRACK SHIELD TUNNEL PASSING NEAR THE FOUNDATION PILES OF THE SHINKANSEN BRIDGE DIRECTLY BELOW THE ROAD BRIDGE PILES

Site conditions

This site is located near the Mt. Asuka through which the Subway No. 7 is currently being constructed at a good pace by the Teito Rapid Transit Authority (hereinafter referred to as the Authority), and two shields for subway's single track were safely completed last year.

On this site, as shown in Fig. 4 and Fig. 5, various important structures are interconnected and intersected with each other in a very complicated manner. At first, there is a street 36 m wide with a very heavy traffic volume at all times. On this street, a sole streetcar system in Tokyo is still being operated. In this vicinity, the street crosses over the Shakujii River 11.5 m wide by the Oji Bridge. And the foundation piles of the bridge piers of the Shinkansen Line are located close to the subway tunnel.

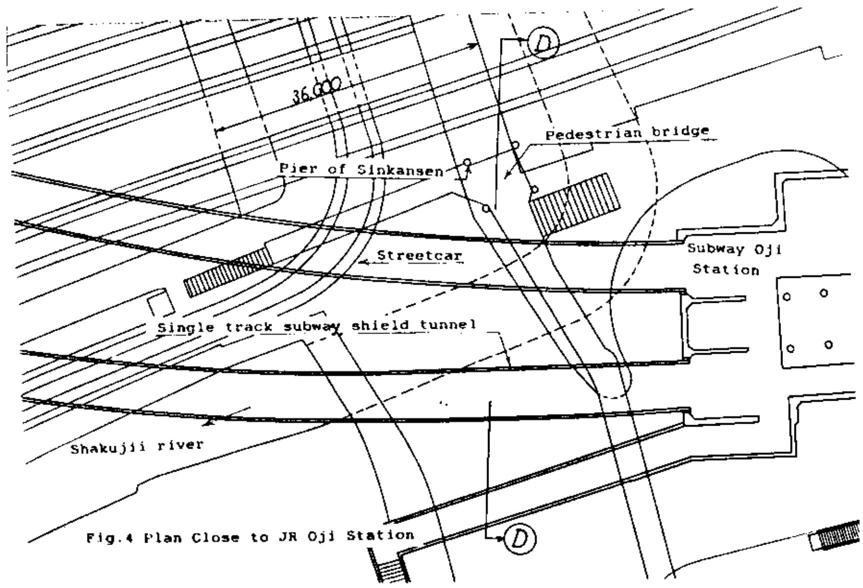

Fig.4 Plan Close to JR Oji Station

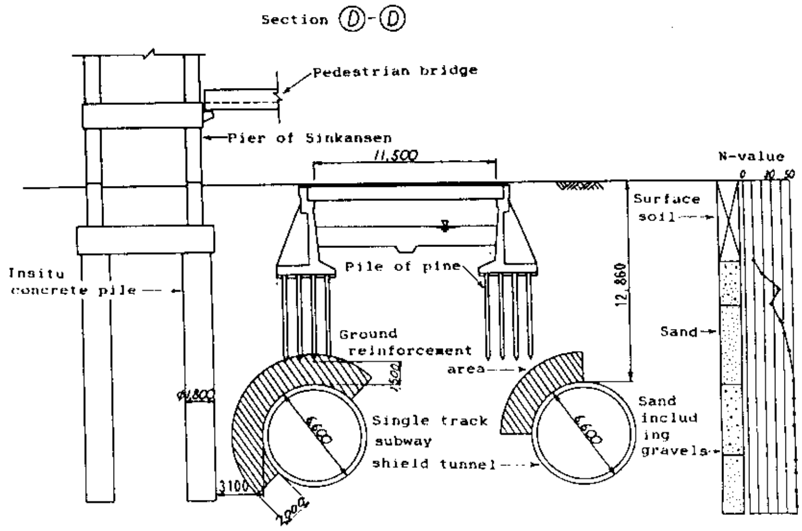

Fig.5 Section Close to JR Oji Station

Seven JR lines are running in parallel to and close to the Shinkansen Line.

The subway tunnel was constructed directly below said structures and adjacent to the foundation piles by the shield method.

As indicated in the section D-D of Fig. 13, the clearance from the shield was 1.5 m to the tip of the foundation piles of the Oji Bridge Abutment and 3.1 m to the side of the foundation piles of the Shinkansen Bridge.

Geology in the vicinity is sand including gravel and is well compacted ground with the N-value of larger than 50.

Subway shield tunnel work

Shield method adopted was of pressurized slurry shield method, and the work was carried out with extra precautions when the shield was passing adjacent to other existing structures. For protecting the existing structures, ductile cast iron was used as segment material instead of reinforced concrete in order to increase the rigidity of tunnel cross section, and also the surrounding ground was reinforced by chemical grouting. Grouting for reinforcing the surrounding ground was performed from the inside of completed shield tunnel immediately after shield driving. As shown in Fig. 5, the range of reinforcing grouting is the range facing the structures to be protected, and the protecting thickness is 2 meters along the periphery of shield cross section.

Grouting material is mainly the sodium silicate, and the grouting ratio is 10 to 20%, as standard, of earth volume within the range of reinforcement depending on geological conditions.

According to the present view of the reinforcement by chemical grouting, the influence of shield driving upon the adjacent ground is considered to be extremely small as long as the shield work is performed carefully by means of pressurized slurry shield method. However, with respect to the filling of tail void, even though the filling is performed simultaneously with the shield driving, the bearing capacity of the ground will be weakened until the grounding material is hardened. Basic concept of protecting adjacent structures by grouting from tunnel inside is to harden the adjacent ground by chemical grouting before the looseness made during shield driving in the ground adjacent to the shield is spread further to the surrounding ground.

SUBWAY TUNNEL WORKS BY CUT-AND-COVER METHOD ADJACENT TO THE FLAT FOUNDATION WITH NO PILES OF VIADUCT OF EXPRESSWAY

Outline of the site conditions

As shown in Fig. 7 and Fig. 8, the planwise situation of this construction site is that the subway tunnel crosses the Kanda River below the Edogawa Bridge while making a steep curve near the bridge. A viaduct of an expressway is located nearby, and its bridge piers are spaced at 60 m on centers across the river diagonally. In addition, there is a construction plan for widening the width of the existing Edogawa River and building a water intake of the Kanda River Diversion Channel in this area. And the subway work has to be carried out while considering these plans and partially performing the work simultaneously with these projects.

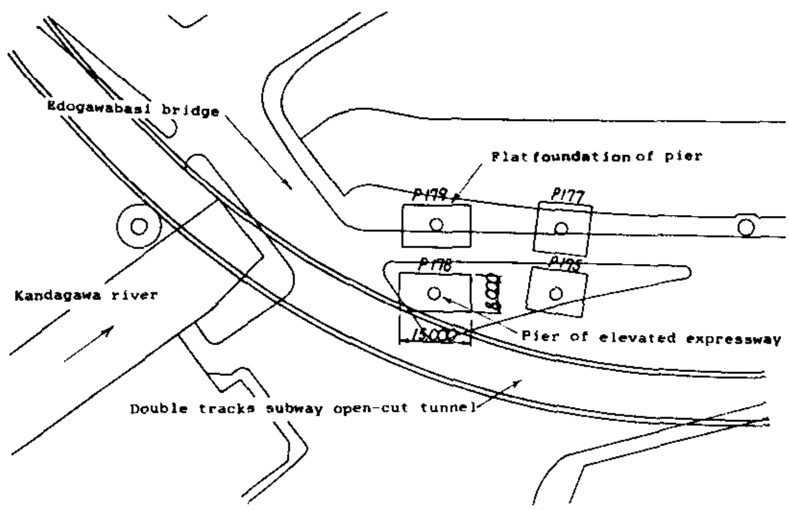

Fig.7 Plan of Kandagawa River Crossing

According to the Kanda River diversion channel plan, two waterway tunnels with the internal width of 7.5 m and height of 7.2 m respectively will be constructed below the road running along the Kanda River for increasing the discharge of the Kanda River twice, and its intake is planned to be built near the Edogawa Bridge. The stream of the Kanda River is bent by 35 degrees at the point of the Edogawa Bridge and moreover river width is narrower here by the bridge. Because of this, it was planned to completely replace the Edogawa Bridge with the bridge width enlarged from 20 m to 30 m in accordance with the street widening plan.

The construction work for Edogawa Bridge across the Kanda River must be carried out while satisfying all the requirements stated above.

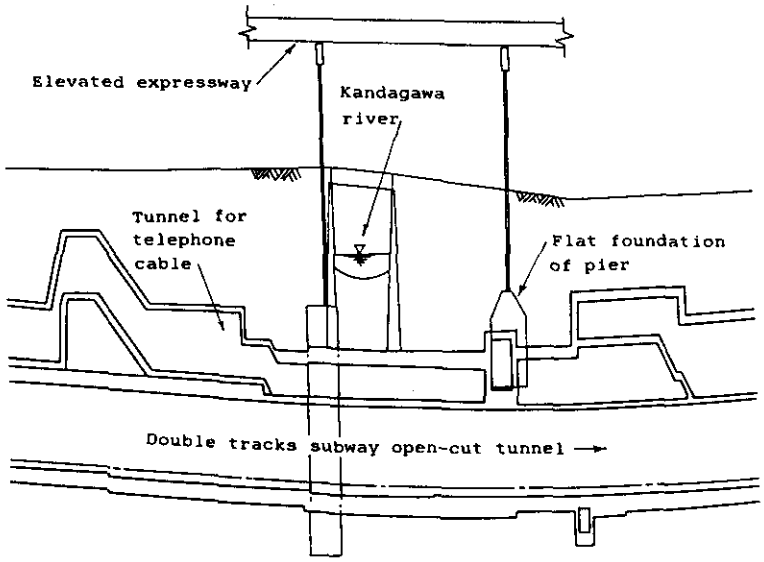

Fig.8 Profile of Kandagawa River Crossing

Construction of the viaduct piers of expressway

The bridge pier P178, which is constructed adjacent to the cut-and-cover work for subway, forms a gantry type structure with the pier P179 and supports one end of a continuous beam for three spans.

Each bridge pier will be supported independently by a flat slab foundation. Foundation for P178 is a flat slab 8 m wide, 15 m long and 3.6 m thick without foundation piles. When this bridge pier was being designed, the subway tunnel was also being designed so that both the design parties had discussions and it was decided to cut off one corner of the flat slab for the purpose of making the work easier for the subway tunnel to be constructed later.

Geology on the site

Strata in this vicinity comprises surface soil from the ground surface to the depth of 4 m, sand including gravel from the depth of 4 to 9 m, medium sand from 9 to 15 m, and fine sand from 15 to 25 m.

Ingredient of the sand including gravel has 77% of gravel content, and the maximum particle size of gravel is 25 mm. Ingredient of medium sand is that the size of sieve which passes 60% of total weight is 0.5 mm. The size of sieve of the fine sand is 0.2 mm comparatively. Uniformity coefficient for both the medium sand and fine sand is 2.6 so that the particle size is considerably uniform. This sand with the uniform particle size gave a great effect on the working method for earth supporting wall as explained later. N-value is 30 to 50 for gravel layer, 50 for sand layer with the amount of penetration of 15 cm, so that the layer is very compact. Groundwater level is 10 m below the ground surface.

Selection of execution method

In selecting the execution method for construction, the most important item to be taken into account is to absolutely prohibit the loosening of stratum which is supporting the bridge piers of expressway. When carrying out the cut-and-cover method adjacent to the foundation of bridge pier, the selection of the method of building earth supporting wall is very important. Generally, a wall type diaphragm is the safest method. The digging in slurry for creating the wall type diaphragm has many good achievements of not loosening adjacent ground especially when there is no extra load in adjacent ground. In the case of this site, an extra load of 30 tf/m^2 was being applied from the superstructure of bridge to the bottom of foundation of bridge pier. Therefore, in the case of excavation method using no casing such as wall type diaphragm method, it was not sure whether the slurry water pressure was able to maintain the wall surface of excavated hall against the earth and extra load. Therefore, employed was the Benoto method using the casings. Results of the Benoto method used in this field will be explained later.

Design of subway tunnel

Because of the situation near the construction site, there was no room for placing a temporary earth supporting wall between the space for subway tunnel and the foundation of viaduct piers of expressway, so that the earth supporting wall was designed also as the permanent structure of the tunnel. If the thickness of the earth supporting wall built by the Benoto method was 80 cm, then the thickness that could be utilized as the thickness of tunnel side wall was only 20 cm.

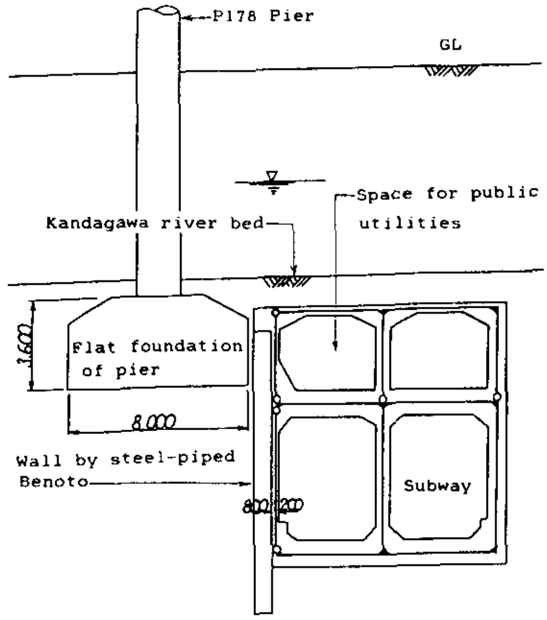

Fig.9 Section at the Point of P178 Pier

Basic concept of the structural calculations is as indicated in Fig. 9. According to this concept, the lateral pressure at the bridge pier side is all supported by the Benoto piers. The side wall with the thickness of 20 cm at the bridge pier side is not considered as a monolithic structure with the Benoto piles. In other words, earth pressure of side wall at the bridge pier side is all borne by the Benoto piles. Contact point between the side wall 20 cm thick and a thick floor slabs is considered as a hinge construction. This means that the side wall 20 cm thick will support the reaction from the floor slabs but not bear the bending moment from the floor slabs. This will be an example of the design of structure affected by the method of executing excavation.

Execution of earth supporting wall

According to the Benoto method, earth is dug out with a circular grab within steel pipe casing, and the casing is oscillated and driven into the predetermined depth. Thereafter, a reinforcing bar cage is inserted into the casing, and the casing is pulled out while concrete is being poured into the cage. The casing is normally made of many steel pipes each being 6-meter long. The earth supporting wall is normally built starting from the top of a road. In this case, the foundation of bridge pier is made about 10 m below the road surface because of the influence of the diversion channel plan as shown in Fig. 10. Thus, it was difficult to verify the accurate position on the road surface and the working accuracy of Benoto pier was 1 to 2 cm per ten meters. Therefore, the foundation of bridge pier was dug out by using a temporary earth supporting wall, and the work was carried out by placing the Benoto excavator on the road surface.

Work of Benoto piles was started from a place on the road far from the bridge pier for the purpose of performing the work for trial. The first Benoto pile was successfully completed after work for 18 hours. During the placement of the second pile adjacent to the first one, the work was smoothly progressed up to the pushing down. But in the stage of casing pull-out, the first pipe 3 m long was pulled out without any problem, but the oscillating pull-out of the casing became gradually difficult thereafter, and at last the expensive casing was abandoned in the earth.

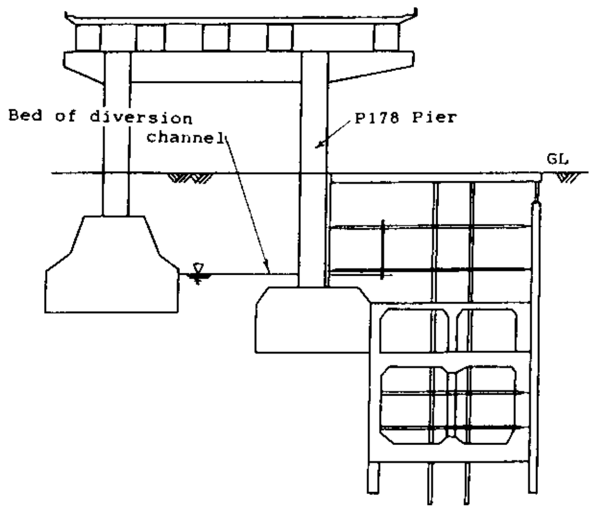

Fig.10 Working Diagram at the Point of P178 Pier

For the third pier, the work was more carefully managed and was smooth up to the pushing down; in the pull-out stage, however, two pipes were barely pulled out and the remainder was abandoned again in the earth. Hot discussions were held with respect to the causes, and it was finally judged that the main cause was the phenomenon of so-called compaction by sand, because the sand had the uniform particle size with the uniformity coefficient of 2.6. Also, boiling phenomenon of sand was slightly recognized at the bottom of casing during excavation. This was believed to be caused because some sand around the casing slightly loosened by the oscillation and pushing down of the casing for Benoto pier and the sand was shifted to the inside of the casing. The first Benoto pier for trial was successful because the ground was not in loosened state in the first stage but, after the second stage, the ground became loose around the pier by the oscillation imposed by the first pier. Those were our conclusion. Based on the new situation, a revision to the work method was reviewed. Finally it was decided to use steel pipes instead of casings since the pushing down was no problem, and the joints of the steel pipes should be welded in the field and no pipes would be pulled out. Hereinafter, this method will be called "steel-piped Benoto Method." Even for the steel-piped Benoto method, it was decided to reinforce the vicinity of two-meter range by chemical grouting before the execution of the Benoto method in order to reduce the degree of loosening the adjacent ground during pushing down operation. In addition, even after pushing down the Benoto pipe, the ground loosened even slightly was reinforced again by chemical grouting. In order to respond to the sand boiling phenomenon, the inside of steel pipe was filled up with a bentonite liquid, and the excavation and concrete pouring were performed in the bentonite liquid.

By the method outline above, the steel-piped Benoto method were constructed at a rate of one pile per day. The vertical accuracy was very good or about 1/600.

Measurement of the anomaly at the viaducts and piers of expressway

For assuring successful construction work performed adjacent to other structures, it is the most important to quickly and accurately measure the abnormal situation, if any, of the adjacent structures.

Horizontal movement of bridge piers should be first measured and theoretically this can be achieved by calculations incorporating traverse. However, in actual phase of work, it is very difficult to create fixed traverse points in the field where cut-and-cover work is being performed in a wide range. Moreover, traverse surveying techniques capable of obtaining the results of millimeter unit are required. In addition, there are other problems such as a time-consuming procedure in arranging and calculating surveying data.

Therefore, as the method of measuring the horizontal movement in this field, survey points were set on the bridge pier columns, the survey point was sighted by transit, and the movement was determined from the difference from the fixed point on the line-of-sight. If this measurement by sighting is performed in two directions almost perpendicular to each other, the horizontal movement can be determined very accurately.

With respect to the ground settlement, the purpose can be fully achieved by the measurement using levels. However, because of the advantage of continuous recording at all times, water level type leveling gauges were installed. Automatic recording type differential transformers were installed for the ground settlement.

Strain gauges were attached to the upper girders and gantry type structures of bridge piers for fully measuring the anomaly without fail.

Disturbances to works by embedded utility pipes

Telephone cable lines were embedded in 4 rows and 3 steps at the place where the steel pipe of Benoto had to be placed adjacent to bridge pier, and these cables were obstructive to the work. A lot of time was spent for relocating these cable lines, and the process of the subway work was greatly affected so that the work of upper slab concrete for reverse slabbing was carried out first by leaving a space for the cable lines which could not be relocated quickly. Thereafter, steel-piped Benoto method was applied to the remaining space and the excavation for the soil under the upper slab was started.

Effect of the work on highway bridges

The steel-piped Benoto piles constructed from the road surface was carried out from January to March, 1972, and caisson work crossing the Kanda River was carried out from November 1971 to March 1972 at the place about 17 m away from the foundation of bridge pier.

Some effect was measured on the inclinometer at the highway bridge pier in January 1972. Many surveys and discussions were made on the cause of this effect. And it was concluded that setting down of caisson was slightly delayed and adjacent ground was loosened; effect of compressed air spread far than expected due to the properties of lower sand layer; and sand around casing moved from the bottom of casing to the inner side during trial work for

Benoto piers. The results of measurement played an important role in changing the working method thereafter and were useful for the construction management by measurement.

Chemical grouting work

In the original planning for ordering the construction work in this work section, the chemical grouting was planned mainly for reinforcing the base of revetment and the penetrated portion of cofferdam work. However, by the construction management by measurement, the range of chemical grouting was further enlarged and reinforced based on the recognition of the anomaly at the highway bridge pier.

As the idea of the enlargement and reinforcement, influencing range of caisson work was reinforced; water cutoff reinforcement were performed at the rear side of column-type diaphragm wall; disturbance due to the construction of steel-piped Benoto method was reinforced; and the bearing capacity of the ground supporting the flat foundation of viaduct of expressway was reinforced.

Chemicals mainly used in the chemical grouting was sodium silicate, but cement bentonite and acrylamaide-based chemicals were also used partially. But grouting materials such as acrylamaide-based ones cannot be used now from the viewpoint of environmental conservation.

Completion of work

When the steel-piped Benoto piles adjacent to the foundation of highway bridge piers were safely constructed, the whole work was almost completed successfully. Thereafter, struts and reverse slabbing were poured quickly and the tunnel work was fully completed safely.

SINGLE TRACK SHIELD TUNNELS CROSSING BELOW A ROAD TUNNEL

Outline of the construction site

As shown in Fig. 11 to Fig. 13, this project is for constructing two single track subway shield tunnels in parallel crossing diagonally below an existing road tunnel. Directly above the road tunnel, there was a high level road in parallel to the tunnel and the foundation of the road bridge piers were constructed monolithically with the road tunnel. Clear distance between the road tunnel and subway shield was only 3.2 m, and this site needed extra precautions for making the effect of subway construction on the road tunnel to a minimum.

Geology on the site

Geology of the layer through which the shield will be driven was a diluvial sand layer including thin solidified clayey layers. Groundwater was not much and the ground was in good condition, and the lower surface of the road tunnel supporting the bridge piers of high level road is also supported by the ground as a flat slab foundation having no foundation piles.

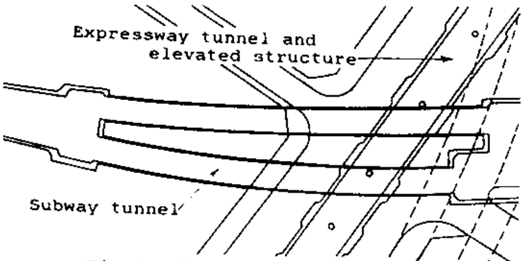

Fig.11 Plan of Subway Tunnel Crossing under Expressway Tunnel

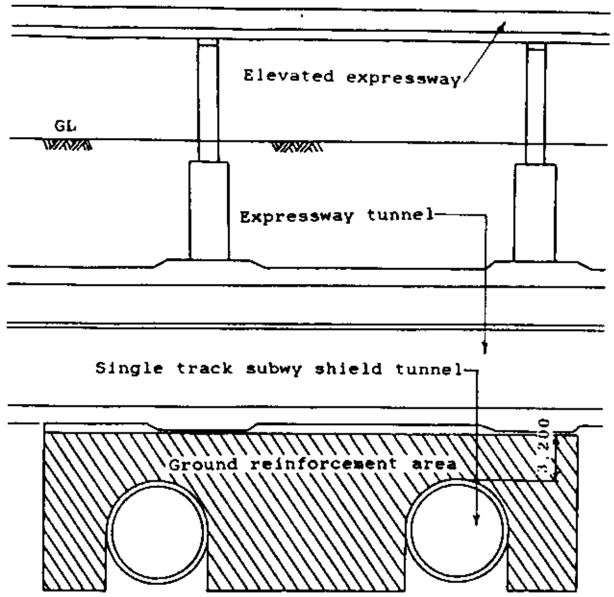

Fig.12 Section of Subway Tunnel

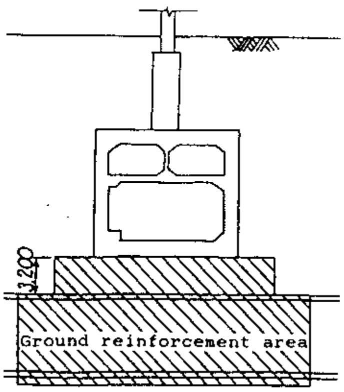

Fig.13
Section of Expressway Tunnel

Execution of shield work

The shield machine first passed the tunnel at the upper side of Fig. 13 from the left to the right, was then diverted to another direction within the premises of railway station provided in advance by cut-and-cover method, and then passed the tunnel at the lower side from the right to the left. The layer through which the shield passed, had few spring water but compressed air of about 0.2 to 0.5 kgf/cm^2 was applied for safety purpose and the shield was driven by hand and mechanical excavation.

Protection of road tunnel and the bridge of high level road

Protection for road tunnel and others was provided mainly by the ground reinforcement using chemical grouting. Range of protection grouting was as indicated in Figs. 12 and 13.

As the method of grouting, the work was performed from the inside of the road tunnel for the portion directly below the road tunnel. Work for the vicinity of shield was performed from the vertical shaft and the side face of station structure constructed in advance by cut-and-cover method. Also, in order to reinforce the ground near the shield loosened immediately after the passing of shield, the secondary grouting was performed from the inside of the shield.

Chemicals were the materials including urea and water glass, and grouting was applied with the grouting ratio of 20% to the ground of 2,300 m^3 as object of grouting. Urea-based chemical grouting is not used at present.

Measurement of anomaly

Anomaly measuring instruments were leveling gauges and inclinometers attached to the inside of road tunnel and inclinometers attached to the piers of road bridge for automatic measurement. In addition, the survey points at the road tunnel and high level bridge piers were observed by levels.

Results of construction

By the initial protection grouting, the road tunnel was floated upward by 4.5 mm. But the road tunnel settled down by 6.5 mm after the passing of two shields. An inclinometer installed inside the tunnel indicated 2'30" but these were all less than allowable limit values and the work was completed without any problem.

DOUBLE TRACK SHIELD TUNNEL PASSING DIRECTLY BELOW FOUNDATION PILES OF A BUILDING

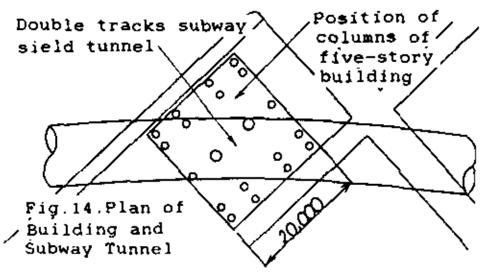

Fig.14.Plan of Building and Subway Tunnel

The construction site was in Jinbocho in downtown Tokyo and the work was performed from 1982 to 1983. Positional relation between the building and tunnel is as shown in Fig. 14. This proposed building has almost a square plan section with 20 m for each side. And the shield has to be driven directly below the building in the diagonal direction of square building plan. The building was 6-storied with foundation piles but it had no basement floors. The shield tunnel was for double track subway with the earth covering of 23 m and the minimum clearance of 3.7 m from the tip of foundation pile to the tunnel. While negotiation were being made with the owner of the building, it was decided to start the construction at the same time for both the building and subway tunnel.

Geology on the site

According to the geology on the site, the top soil was about 2 m deep from the ground surface, there was a soft silt layer about 13 m deep with the N-value of less than 5.0, and thereafter the cohesive soils of Tokyo Layer were present with N-value of 10 to 15 and coefficient of permeability of 10^{-6} cm/sec. thereby forming a stable impermeable layer. There was Tokyo gravel layer underneath the cohesive soils, and this layer contained cobble with the diameter of 100 mm maximum and had the coefficient of permeability of 10^{-3} cm/sec, and layer thickness of 3 to 5 m. This layer was a bearing layer for large buildings in the same area.

Prediction of the settlement of building

Trial calculations were made by using the techniques of finite element method for predicting the settlement of the building after a space of tunnel section was created underground with the properties of ground and building structure remaining as same as before. The result indicated that the building will be settled down relatively by 22 mm and even 45 mm at the central part of the building absolutely.

Factors to be considered when determining the allowable settlement for a certain building are the foundation structure of the building, superstructure, purpose of use of building, surrounding situation, experience in the past and so forth. In the case of this building, the relative settlement of 23 mm was considered to be the limit, and the allowable settlement of 12 mm was established with the safety factor of 2.

Measures for preventing the settlement of building

In order to maintain the relative settlement of 12 mm, the following measures were taken:

(a) To increase the strength of walls and underground beams connecting pier heads of the new building to increase the rigidity of the building as a whole.

(b) To increase the pile diameter of foundation piles of the building and to disperse the load from the tip of pile.

(c) For verifying the geology in the field in advance and for completely performing the reinforcing work such as soil stabilization, to perform the pilot shield tunnel work 2.95 m in diameter by means of compressed air method before carrying out the shield tunnel method for main lines.

(d) To perform reinforcement by chemical grouting for the ground within the range shown in Fig. 15.

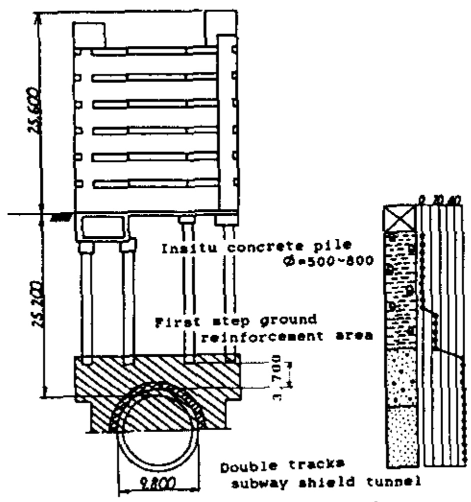

Fig.15 Section of Subway Tunnel

(e) To carefully perform and review the tail void grouting, its method, material, hardening time, etc.

(f) To drive the shield with the face supporting jacks applied and to slowly and gradually perform the excavation and driving.

(g) After passing the shield of the main line, to perform the secondary grouting from the position inside the tunnel as shown in Fig. 15 thereby restricting the subsequent settlement of adjacent natural ground.

Sequence of the execution of simultaneous construction works

(a) Pilot tunnel work
(b) Work of cast-in-place foundation piles for building
(c) Work of chemical grouting for reinforcing the ground
(d) Start of work for the structural portion of building in parallel to the shield work
(e) Groundwater level dropping work by well point
(f) Execution of subway double track shield tunnel by compressed air method
(g) Tail void grouting
(h) Secondary grouting work
(i) Placement of invert concrete within tunnel
(j) Interior and exterior finishing work for building

Results of work

Settlement of the building occurred when the face of shield was driven for a distance of about 30 m from the central part of building, but its relative settlement was only 3 mm, and all the works were completed safely.

DOUBLE TRACK SHIELD WORK ADJACENT TO AN EXISTING DOUBLE TRACK SHIELD TUNNEL

Outline of the site

Near the Kudan in Tokyo, new subway double tra shield tunnel work was performed in parallel to and adjacent to an existing subway double track shield tunnel in 1983. Sizes of the two tunnels and the positional relation at the closest point between them are as shown in Fig. 16. This kind of relation was maintained almost for a distance of about 420 m in the parallel adjacency.

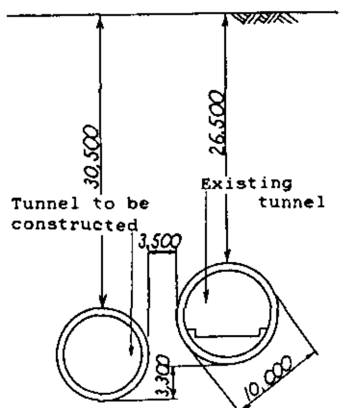

Fig.16 Relative Position of Two Double Tracks Sield Tunnel

Geology on the site was the loam layer from the top to the depth of about 10 m. Subsequent layers were the alternating diluvial layers of sand and clay.

N-value was generally high, and was higher than 50 at the depth of 35 m from the ground surface at place where the tunnel was to be driven. Groundwater pressure was about 1.2 kgf/cm^2 at the location for tunnel.

Policies for executing the work

Though the shields were very closely located to each other, the geological situation on the site was very good, fortunately, so that the policies were mainly to carry out the work very carefully for the new shield tunnel by using pressurized slurry shield method. Otherwise, main things performed as the safety measures for this work were the detailed observation of anomaly at the existing tunnel and adjacent ground and so forth.

Prediction of anomaly at the existing tunnel

The prediction of the anomaly at the existing tunnel was performed by the finite element method. As the stress condition accompanied with the driving of the new shield, the sum of earth pressure and water pressure and the difference from the slurry pressure were used. As the results of the anomaly calculations, the settlement of 2.5 mm at the top of tunnel and horizontal displacement of 2.6 mm at the side portion were calculated.

Also, it was verified from the calculations that the structure of the existing tunnel was sufficiently safe against this kind of displacement.

Method of measuring the anomaly at the existing tunnel

The anomaly at the tunnel was checked and measured by installing leveling gauges inside the tunnel as shown in Fig. 17, attaching strain gauges inside the tunnel segments, and measuring the displacement in inner dimensions of tunnel by using measuring instruments. The leveling gauges were of type capable of automatically recording the relative movement. Additionally, four basic level points were measured by transit and even the absolute settlement was checked.

Measurement of anomaly in the surrounding ground

The following measurements were performed for the surrounding ground as the shield driving progressed:

(a) Settlement of ground surface
Metal surveying rivets driven to the road surface were measured by leveling.

(b) Underground horizontal movement
An insertion type high-precision inclinometer was inserted into a measuring pipe embedded underground, and the movement of the measuring pipe was determined.

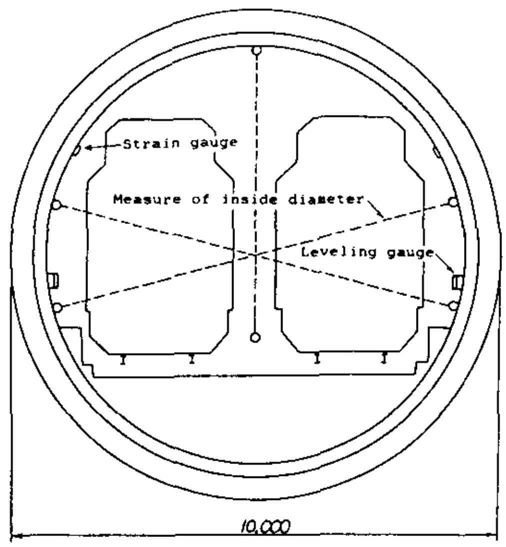

Fig.17 Deformation Check Points of the Existing Tunnel

(c) Underground vertical movement
Movement was measured by anchor type leveling gauges for each layer.

(d) Pore water pressure
High-precision pore water pressure gauges were used, and the pressure was converted into electric signals and continuously recorded above the ground.

Results of measuring anomaly

Scale of leveling gauges inside the existing tunnel was about -0.5 to +1.3 mm, and the measured values were within the range of errors affected by train vibration and temperature.

Strain gauges on the inner surface of tunnel did not indicate significant stress increases.

Settlement of the ground surface was measured to be about 3 mm. Settlement of underground points was 6.5 mm maximum at the measuring points directly above the tunnel. Horizontal movement of underground points were observed to be 1.7 mm in the direction of pushing outward at the time of shield driving.

Because of good ground conditions and careful execution of shield work, this project was completely safely as scheduled.

REFERENCES

(1) Standard Specifications for Tunnels (Shield Section) and Commentaries, March 1983, Institute of Civil Engineers.

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