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[Report] Example of Construction of Deep Underground Cable Tunnel in Singapore
—EW1 construction section—

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—EW1 construction section —

1. Outline of the Project

1.1 Cable Tunnel

In Singapore, to address future requirements for increase in electricity demand, a transmission line cable tunnel, which extends for 18.5 km in a north-south direction (NS construction section) and 16.5 km in an east-west direction (EW construction section), was planned, and construction was started in October 2012. Completion is scheduled for 2018. In highly urbanized Singapore, utilities such as gas, water supply, telephone lines, etc., are laid underground, and MRT (subway) tunnels are located deep underground due to the congested route network. Since the cable tunnel in question, when viewed from above, intersects with the existing Deep Tunnel Sewerage System (DTSS), it is designed to be located at a depth of 55 m or more, which is deeper than the DTSS. Figure 1 shows an overall location map of the project. It is a design and build project, which comprises six construction sections, where two Japanese companies and three Korean companies are responsible for their respective construction sections: NS1: Samsung; NS2: SKEC; NS3: Hyundai; EW1: Obayashi; EW2: SKEC; and EW3: Nishimatsu Construction-KTC JV.

1.2 Outline of EW1

Table 1 shows the outline of construction of EW1, and Table 2 shows a tunnel route map. EW1 has the length of about 4 km extending from the western end of the East West Line, and a tunnel with a maximum depth of 68 m was constructed from Ayer Rajah Shaft at the western end and Holland Shaft at the eastern

end with two units of shield machines. The two shield machines were started in May and June 2014, respectively, and they were connected with each other in January 2016. Originally, the units were scheduled to connect and be dismantled at North Buona Vista Shaft located in the middle of the two shafts. However, due to delay in progress of the slurry shield on the Holland side, the length of 1,330 m to be excavated by the mud pressure shield on the Ayer Rajah side was extended by 920 m and the tunnels were connected under Holland Village Station. As a result, excavation lengths were 2,250 m by the pressure shield method and 1,640 m by the slurry shield method.

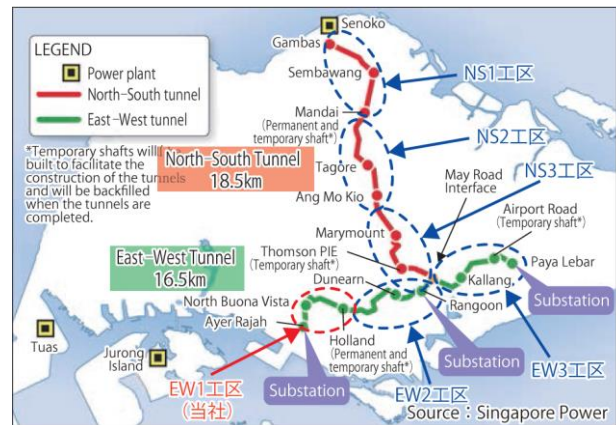


Fig. 1 Overall location map of the project

2. Outline of the Tunnel

2.1 Segment

The tunnel lining comprises axial insertion type 6 split RC segments with an inner diameter of □6.0 m, a thickness of 300 mm, and a width of 1,400 mm (width of 1,000 mm at 140 mR curve section), which were manufactured in a segment factory in Johor Bahru, Malaysia and transported by land. Gaskets integrally formed with water swellable seals were used as a water blocking material, skew

bolts were used for joints, and a cushioning material made of thin plywood was attached between rings for preventing cracks. These specifications were standards in the construction of MRT.

2.2 Excavation of Grounds to be Excavated

The ground of the west side comprises sedimentary rocks called Jurong Formation (maximum unconfined compressive strength: 78 MPa) and the ground of the east side comprises granite called Bukit Timah granite (maximum unconfined compressive strength: 227 MPa), and there was a section comprising composite ground containing a weathered soil layer. Before entering the composite ground, a probe drill mounted inside the shield machine searched the boundary with the weathered soil, and the machine excavated the ground until immediately before the boundary and then changed cutters in the rock ground.

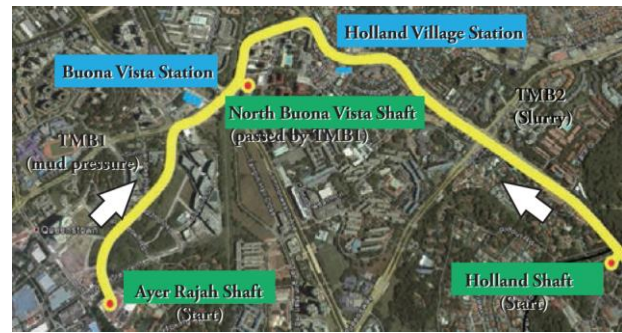


Fig. 2 Tunnel route map



Photo 1 Mud pressure shield

Photo 2 Slurry shield

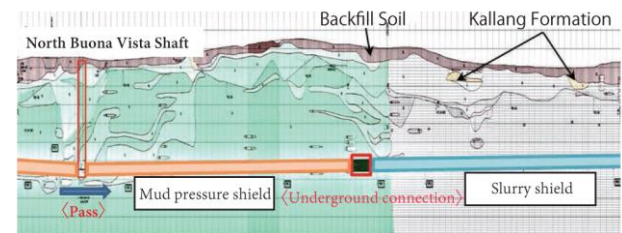


Fig. 3 Longitudinal sectional view around the connection

In the composite ground, earth pressure control was set relatively high, and the slurry shield machine excavated the ground by increasing the viscosity of slurry. Further, even when the cutting face was self-supported, it was required to maintain the earth pressure control high due to concerns about subsidence of the surface of the earth caused by lowering of groundwater level: on average, it was 3.5 bar for the mud pressure shield and 4.0 bar for the slurry shield. Figure 3 shows a longitudinal sectional view around the connection. The connection point was determined to be about 80 m on the

Table 1 Outline of the construction of EW1

Name of construction	SPPA Cable Tunnel EW1
Application of tunnel	Tunnel for ultrahigh voltage (400 kV) transmission lines
Orderer	SP PowerAssets Ltd
Form of contract	Design Built
Contractor	Obayashi Corporation (single)
Construction period	October 2012-November 2016
Place of construction	Ayer Rajah - Holland
Details of construction	<ol style="list-style-type: none"> Shaft, building Start shaft, ventilation building: two places (inner diameter of $\square 14$ m) Intermediate shaft, facility building: one place (inner diameter of $\square 12$ m) Adit Between each shaft and the shield tunnel Shield tunnel TBM1 (mud pressure): construction length L=2.25 km TBM2 (slurry): construction length L=1.64 km RC segment: outer diameter of $\square 6.6$ m Girder height: 300 mm Width: 1.4/1.0 m Minimum curve radius: 140 m Open-cut tunnel 235 m (including a section of 32 m where auxiliary construction method was used in combination with NATM) Entire utility work (ultrahigh voltage cable was constructed separately)

sedimentary rock side from the boundary with the granite.

2.3 Shield Machine

A total of two shield machines namely, one unit of mud pressure shield (Photo 1) and one unit of slurry shield (Photo 2), were used, each having an outer diameter of $\square 6.95$ m and a peripheral dome-type face plate with 6 spokes. TBM1, whose ground consisted mainly of sedimentary rocks of relatively low intensity, employed a mud pressure shield, and TBM2, whose ground consisted mainly of hard granite, employed a slurry shield. Also, the maximum diameter of gravel capable of being taken in was set at 300 mm, and aperture ratios of cutters were set to 24% and 19%, respectively. Roller cutters were mainly employed and scraper bits were provided on the side of each slit opening. In addition, their major equipment and capacities were characterized by a spherical articulation (respond to a curve of 149 mR), a maximum thrust of 48,000 kN, a maximum cutting torque of 7,500 kN-m, and a cutter rotation speed of 5.0 rpm. Soil excavated by the mud pressure shield was conveyed to an excavated earth wagon by primary and secondary screw conveyors and a belt conveyor, and then conveyed to outside of the tunnel. The slurry shield machine discharged excavated soil by means of slurry transportation using a slurry sending-discharging pump. Since most of the ground to be excavated was hard granite, the slurry shield machine employed 19-inch roller cutters. The mud pressure shield employed 17-inch roller cutters, as the ground consisted of sedimentary rocks sandstone and mudstone. Cutters were replaced 40 times for the mud pressure shield, and 75 times for the slurry

shield. Due to the frequent cutter replacement work, particularly the progress of excavation by the slurry shield was delayed. A pneumatic facility and equipment for conducting injection from inside of the machine were provided as auxiliary facilities for cutter replacement. When it was not possible to decrease the amount of influent groundwater to a specified value only with compressed air, it was controlled by means of a reverse grouting method to be described later. First, water stop injection from inside of the shield machine was planned, but it was not carried out in the construction.

2.4 Shaft and Adit

A circular start shaft had the diameter of 14 m, and a circular intermediate shaft had the diameter of 12 m. A diaphragm wall was built in the upper part of the weathered ground and used as the main body and, a base rock layer at the lower part of the shaft was subjected to blasting excavation, shotcrete process, and rock bolting, and then lining reinforced concrete was constructed. Lining of the adit was conducted in a similar manner. Also, a recharge well was installed as an auxiliary method for constructing the shafts and the adit, and a cement-based material was injected into rocks.

3. Excavation Performances of Shield Tunnels

Major excavation performances of shield tunnels are given below. Measures for improving workability during cutter bit replacement and measures for recovering delay in process will be explained in detail in Chapters 4 and 5.

3.1 Mud Pressure Shield

3.1.1 Excavation Data

Average excavation speed was 20.4 mm/min. Backfill grouting was conducted by means of co-injection from the tail of the shield machine, which was standard in Singapore, and average injection ratio was 106%. An add-in material consisted mainly of foam, and as other ingredients, water was contained, and bentonite was used as needed.

3.1.2 Issues and Measures during Construction

·The tunnel in question passed under the tunnel of MRT Circle Line, Buona Vista underground station, and Buona Vista elevated station of East West MRT Line. Changes in the states of the tunnel and underground station were automatically measured, and they were constantly monitored by monitoring staff for the excavation management unit.

·The section located 200 m from the starting point had ground consisting of highly abrasive igneous rock, which caused not only partial wear of the cutters but also damage of the center cutter housing. In order to smoothly take in excavated earth at the center of the faceplate, an opening was made and a water injection line was modified so that a discharge port would not be clogged.

· In the middle of the construction, a chip-insert type was used as a centre cutter, and its range of use was broadened to the outer periphery side, which was successful in reducing the frequency of cutter replacement.

3.2 Slurry Shield

3.2.1 Excavation Data

Average excavation speed was 8.6 mm/min. During the excavation, the pressing force of the cutters was mainly monitored. In ground

with many cracks, excavation speed increased up to 15 mm/min., decreased to 5-6 mm/min when the ground was hard and further decreased to 2-3 mm/min. when excavation efficiency lowered due to wear of the cutters. Average backfill grouting rate was 107%.

3.2.2 Issues and Measures during Construction

·Due to hitting and vibration of cutter fixing brackets, boards of the housing gradually sank and deformed, fixing bolts loosened, and fixing metal fittings dropped off. They were taken care of by conducting clad welding and abrasive operation with respect to the housing during replacement of cutters.

· In the section where ground had many cracks, rocks of large mass were taken in, causing frequent clogging of the sludge drainage pipe between the shield machine and the crusher on following truck No. 1. A window for taking out mass of rocks was installed on the sludge drainage pipe in order to reduce time for recovery from clogging, and a baffle plate was added on the slit section for secondary crushing on the faceplate.

4. Measures for Preventing Inflow of Groundwater by Reverse Grouting Method

4.1 Large Amount of Groundwater In-flow during Cutter Replacement

In January 2015, nine months after the mud pressure shield started, when scheduled replacement of cutters was carried out, groundwater of 1,300 L/min. with a chamber pressure of 2 bar came in, which we had not experienced within the construction section. The upper limit groundwater inflow amount according to analysis of lowering of groundwater level and

subsidence of the surface of the earth was 200 L/min, and lowering of groundwater inflow amount was needed, since the operation was impossible even under the upper limit pressure of 3.5 bar specified by a public agency.

First of all, polyurethane was injected into the ground around the shield machine, which had been reported to be effective in other construction sections, but groundwater inflow amount remained unchanged and thus water was considered to come from the front. The construction work was expected to be suspended for a few months if ground improvement were to be conducted from the ground. Amid continued discussion, it was decided to experimentally introduce a reverse grouting method using back-filling material, which was conceived of in a shield work in Ohio, USA, in concurrence with preparation work on the ground.

4.2 Principle of Reverse Grouting Method

Figure 4 shows a conceptual diagram of the reverse grouting method. In this method, first, water is injected into a chamber at a pressure higher than a natural water pressure (at the site in question, initially, +0.5 bar, and then +1.0-1.5 bar) to induce flow of water toward cracks in the surrounding ground (i.e., leakage of mud water) and, at the same time, cement grouting material is discharged to outside of the shield to let it naturally flow toward the cracks to fill them. Water injection pressure rises as the cracks are clogged, and water injection amount and grouting injection amount are decreased while carefully monitoring the clogging condition. Injection of water and grouting is continued until the injection speed is decreased to about 10 L/min.

When the injection pressure begins to rise with the low flow volume, the above process is repeated in another injection hole. When the procedure for areas, which are scheduled to be taken care of, is finished, necessary time is taken for curing, groundwater inflow amount is checked, and reinjection is conducted if necessary.

At the site, natural water pressure was basically 3.5-4.0 bar and grouting injection pressure was set to 5.0-5.5 bar, and injection holes provided on the outer periphery of the rear body of the shield machine were used for injecting grout until water injection pressure increased in each of the injection holes. Water injection and grout injection took at least 2-3 hours and took one shift at the longest, and curing time was set to be 8 hours after finishing grouting work.

4.3 Effect of Reverse Grouting Method

Injection amount (amount of leakage of mud water) for making pressure of water injected greater than natural water pressure of 4.0 bar by +0.5 bar at a point where groundwater inflow amount was 1,300 L/min. was about 500 L/min. immediately before starting grouting. After the first grouting, an effect that the groundwater inflow amount was decreased to 825 L/min. at 2 bar was observed, and it decreased to 200 L/min. at 1.5 bar after second and third grouting. Cutter replacement work was started under compressed air, and it was finished without an increase of the groundwater inflow amount, which had been a concern.

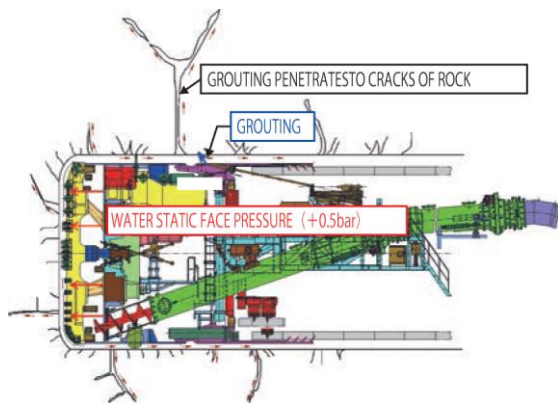


Fig. 4 Conceptual diagram of reverse grouting

The reverse grouting method was conducted also in the subsequent cutter replacement operations when groundwater inflow amount exceeded the established groundwater inflow amount (40-200 L/min. depending on areas), and it became possible to conduct most of the work without compressed air. However, there was a case where groundwater inflow amount could not be decreased to or less than the specified value in the Jurong Formation of the mud pressure shield section. Excavation was continued and the cutter was overused under the situation, but in granite of the slurry shield section, groundwater inflow amount could be lowered to the target amount each time even though compressed air was used several times.

4.4 Grout Composition

In the reverse grouting method, a back-filling material consisting of solution A made of cement bentonite (CB) and solution B made of silicate soda was used. Since grout is discharged from exploration drilling holes provided radially on the rear body of the shield machine, gel time was set to be relatively long, such as 15-20 seconds, so that the insides of the holes would not be clogged. For the mud pressure shield, injection holes were added on the front body when the machine went

through the intermediate shaft and during maintenance of the machine.

Incidentally, adjacent EW2 was reported to have conducted, in addition to a similar reverse grouting method, reverse grouting by pouring only solution A into the chamber for stabilizing the cutting face. Also, it was reported that, at a location where subsidence of the ground occurred, microfine cement slurry was poured into the chamber and decreased groundwater inflow amount to almost zero.

5. Reduction of Excavation Process by Means of Underground Connection

5.1 Background to Determination of Underground Connection Point

As described above, predetermined progress was not achieved by the slurry shield on the Holland side. Therefore, in order to compensate for the delay, excavation was conducted also from the opposite side of the slurry shield tunnel section after the excavation reached North Buona Vista Shaft, so that the two shield machines were connected underground to reduce the total mining time. In consideration of the progress of the two shield machines, at first, the mud pressure shield was decided to pass the shaft, and then the length of excavation from the opposite side was gradually increased to 500 m and then to 800 m. Finally, the excavation length of the mud pressure shield was decided to dock under Holland Village station of MRT Circle Line where impact on the ground settlement was considered to be small.

The boundary between the Jurong layer and Bukit Timah granite was located near the eastern end of the station building. In this

construction section, since Bukit Timah granite had less groundwater inflow than the other, there was a plan to shift the junction location further to the east. However, the mud pressure shield, which was configured for Jurong Formation, was provided with 17-inch cutters, which means that it could have broken easily while excavating Bukit Timah granite. Therefore, it was eventually decided to connect the excavated tunnels in the Jurong Formation under the station building. Figure 5 shows a plan view of the underground connection point.

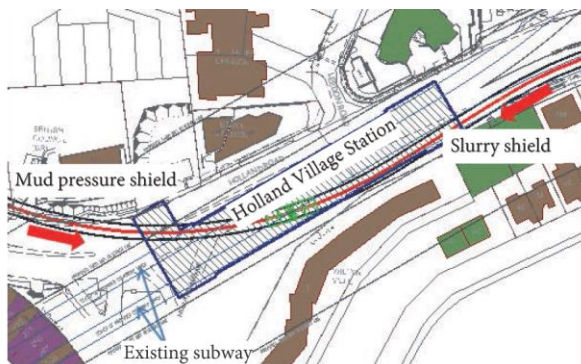


Fig. 5 Plan view of the underground connection

5.2 Segment Layout

After the slurry shield entered the Jurong Formation, the faceplate and the chamber clogged frequently, preventing the shield machine from excavating in accordance with the target schedule. Therefore, the connection point was moved to the eastern side of the station building (close to the slurry shield side). Consequently, this caused an excess or deficiency in quantity of tapered segments, but this was adjusted by exchanging segments with the adjacent construction section.

5.3 Process for Arrival of Shield Machine

First, the slurry shield arrived at the intended connection point. The cutting face was self-supported and stable (visual RQD of

30) while it had many slight cracks, and groundwater inflow amount was less than 10 L/min. RQD was expected to be within the wide range of 0-60 based on the result of soil investigation in the vicinity, and therefore it was difficult to predict whether the ground condition was good or bad when excavation was further proceeded. For this reason, the place whose cutting face was stable and had less in-flow underground water compared with cutting grounds which had been observed by that time was determined as the docking/connection point.

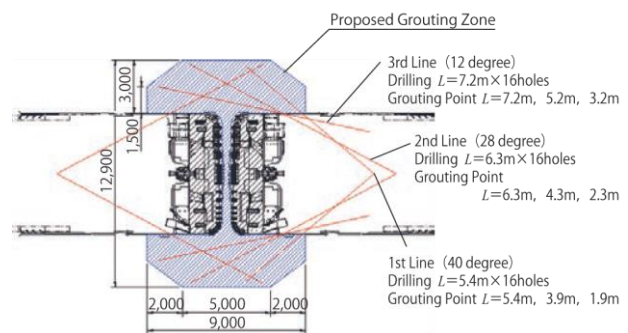


Fig. 6 Schematic drawing of improvement range of the connection point

The slurry shield maintained the cutting face pressure by feeding slurry until the mud pressure shield arrived at the docking/connection point. In order to prevent excessive excavation, the mud pressure shield arrived by decreasing cutter rotation speed and excavation speed.

After the mud pressure shield arrived at the specified docking/connection point, earth inside the chamber was discharged and checked. The upper part collapsed with excavated earth accumulated by being pressed toward the slurry side. Further, since intermittent fall of earth was observed, slurry was fed again from the slurry shield side in order to maintain the cutting face pressure.

After a dry spray machine was carried into the tunnel and prepared, slurry was discharged, and spray concrete using prepacked material (28-day compression strength: 50 N/mm³, bending strength: 6 N/mm³) was sprayed at a thickness of 50 mm and thereby suppressed the fall of earth. Eventually, an articulation jack of the mud pressure shield was stretched and separation between the roller cutters and the cutter bits of the slurry shield (their roller cutters had already been removed) was decreased to 50 mm. The hollow on the upper part was filled with cement-based grout after the shield machines were dismantled, and water stop iron plates were installed.

5.4 Survey Accuracy for Connecting Shield Machines

During the shield excavation, daily inspection of control points and automatic tracking survey of the shield machines were conducted by a total station. In each tunnel, a control point was subjected to gyro survey after completion of the initial boring and before arrival of the shield machines. Owing to the results of these surveys, error in connection of the shield machines was 5 mm in the horizontal direction and 20 mm in the vertical direction.

5.5 Grouting at Connection Point

There was also a plan to decide a docking/connection point in advance and conduct ground improvement from the ground, but it turned out difficult to prepare an improvement zone with accuracy by obliquely boring the ground at a depth of 70 m. Therefore, it was decided to improve the ground from inside of the machine after primary dismantling of the shield machine

when ground improvement was necessary. In order to prepare an improvement zone in the ground near the docking/connection point, it is easier to do so from inside of the machine, because boring distance can be shorter and boring and injection can be conducted at an optimum elevation angle, and it was also easy to conduct additional boring and injection. Figure 6 shows a schematic drawing of improvement range of the connection point.

A period of ground release at the docking/connection point was determined to be 2 months, and the upper limit groundwater inflow amount was set to be 60 L/min. based on the analysis result.

Actually, groundwater inflow amount after arrival of the two machines was as low as 10 L/min. and further, it decreased to a few L/min., which was difficult to measure due to the aforementioned reverse grouting and supplemental injection of an aqueous material. Therefore, boring & injection operation, which had been planned to be conducted as needed, was not conducted.

5.6 RC Lining

RC structure was employed for lining of the underground connection point including inside of the shield machine. Thickness of RC lining was set to be 240 mm without taking into consideration the outer shell of the shield machine left behind. Spraying was conducted on the ground from which the faceplate of the shield machine was removed at a thickness of 50 mm, and water stop iron plates were welded on the entire ground to connect it with the outer shell of the shield machine. The total length of 18 m was subjected to four times of concrete placements: the lower part: once, and the upper arch: about 6 m x 3 spans.

All reinforced steel & steel formwork materials were carried in by a storage battery locomotive after dismantling of the shield machines was completed and before removal of underground rails and crossties were started. Concrete was transported by means of pressure feeding from the ground of the intermediate shaft to a 2-m³ bucket in the

tunnel, then it was delivered by a 5-t forklift for about 900 m from there to a fixed concrete pump installed before the docking/connection point, and then it was subjected to pressure feeding and placement.

Concrete with design intensity of 60 MPa had slump of 230±30 mm and coarse aggregate of 10 mm, and coagulation time was 4 hours.

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