ENLARGEMENT OF PRE-DRIVEN SHIELD TUNNEL SECTION BY A NEW METHOD

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1. INTRODUCTION

Tokyo Electric Power Co., Inc. has constructed 275kV main underground power transmission lines radially from the center of Tokyo in order to meet an increasing demand for electricity in Tokyo metropolitan area. The main underground power transmission lines require manholes to connect power cables at intervals of about 700m to 800m. One of these manholes has been built by newly developed Enlargement Shield Tunneling Method (Figs.-1 & 2). The design and the construction methods of this manhole are presented in this paper.

2. SELECTION OF MANHOLE CONSTRUCTION METHOD

Manhole, where power cables are jointed and cable conduits are carried in, serves as an entrance for maintenance and emergency. Since it is sometimes used as a vertical shaft during tunnel construction, the open-cut method is generally adopted. However, environmental conditions of the construction of underground power transmission lines are getting severer, and therefore the open-cut method is recently hardly employed in a highly developed urban area. The manhole presented here is a large-scale structure because of serving for both electric power and telecommunications cables. The construction site is located at an intersection of main roads with heavy traffic of 3500 cars/hour and many underground utilities in a central commercial district of Tokyo. If the open-cut method is adopted for the construction of the manhole under such conditions, most part of the intersection will have to be excavated, and a longer period of construction will be required. In such a case, the local environment and the traffic will be affected, and accidents during construction may occur. To overcome these problems, the application of the newly developed Enlargement Shield Tunneling Method, by which the tunnel is enlarged from inside, has been considered; lining structure, excavation system, face stabilization, etc., are technically examined. As a result, it has been concluded that this method will require less cost and bring about more safety than other method, and that it is technically applicable if the auxiliary work such as ground improvement is employed together. This conclusion has led to the first application of the "Enlargement Shield Tunneling Method" in Japan. Fig.-3 shows the manhole structure when this method is applied and Fig.-4 shows the procedure of the method.

3. GEOLOGICAL CONDITIONS

The geological profile of the site is shown in Fig.-5. The ground to be excavated is composed of diluvial sand layers overlain by an alluvial soil layer of about 9m thickness. There is a thin clay layer at the tunnel springline. Above that layer are alternate layers of coarse to medium sand layers and fine sand and silt layers. Below that layer is a fine sand layer partially containing some shells. Ground water is separated by the clay layer with pore pressures at the excavation level of 1.0E5 Pa in the upper sand layers and of 1.3E5 Pa in the lower sand layer, respectively. Table-1 gives the characteristics of the sand layers. Each sand layer is considered to be susceptible to progressive failure of quick sand due to piping and so on.

4. DESIGN

4.1. DESIGN OF GUIDE RING

Guide rings are attached to the primary segments rings at the both ends of the enlarged part in order to drive the circumferential shield machine without deviating from the driving line on the circle of primary tunnel section. Therefore in order to avoid excessive deformations, causing deviation, of the guide rings, they have been designed under consideration of the following forces; reaction force of the divided segments under condition of unclosed segments ring during the circumferential shield driven, reaction force of circumferential segments under condition of unclosed segments ring and driving forces of the circumferential shield and primary shield. As the ring was considered in the most critical condition, i.e., when removing primary segments for excavating and building the starting base of the circumferential shield, a brasing beam was installed for the missing part of the ring, and the ring was designed as an unclosed ring structure assuming the joints at both ends of the beam as hinge supports.

4.2. DESIGN OF SEGMENTS

The segments used at the enlarged part consist of the primary segments which include normal segments, reinforced segments and divided segments, the circumferential segments, and the secondary segments (Fig.-6 and Table-2). All the segments are made of steel considering workability and cost.

4.2.1. Reinforced Segment

Reinforced segments are used at both ends of the enlarged part. Since the primary segments are removed and circumferential segments do not make a complete ring while driving the circumferential shield, load carried by the primary segments should be supported by some other method. It was assumed that a half-ring arch of the improved ground around the circumferential shield could be formed and transfer the load to the primary segments neighboring at the both ends of the enlarged part (Fig.-7). Based on this assumption, reinforced segments were designed against the load acting on the primary segment ring by analysing the arch with an elastic Finite Element Analysis (FEA).

4.2.2. Divided Segment

The primary segments ring to be removed during the excavation of circumferential shield is divided into 13 by the segments considering workability. Divided segments were designed as a ring with uniform stiffness as in the case of ordinary segment and also were designed as a simple beam supported on the guide rings considering possibility of forming an incomplete ring after removing primary divided segments for circumferential shield base (Fig.-8).

4.2.3. Circumferential Segment

A circumferential segments ring is divided into 50 segments due to advancing step length of circumferential shield and for ease of erecting segments, and there are 4 circumferential segments rings in a longitudinal direction (Fig.-9).

The ring pieces were designed as a simple beam supported by the side plates during the excavation by the circumferential shield. They were also designed as an ordinary ring structure after a complete ring was formed. The side plate was designed in two directions, namely the radial direction (main beam) and the circumferential direction (longitudinal rib). The main beam was designed against side pressure, considered as a simple beam supported by the ring piece and the guide ring. The rib was designed against the driving reaction by the circumferential shield.

4.3. DESIGN OF SHIELD

The circumferential shield and the enlargement shield were used for the enlargement work. Although the closed type excavation method such as slurry shield and soil pressure type shield was considered most suitable for such ground conditions, the open type was selected for both the circumferential shield and the enlargement shield. This is because the enlarged part is short and because the closed type is costly and has technical problems of excavation and mucking.

4.3.1. Circumferential Shield

The circumferential shield machine with a rectangular face advances engaged with grooves on the guide rings attached to the primary segments. In order to erect the circumferential segments at the tail of the machine, the machine has a skin plate, two side plates and no inner plate (Fig.-10 and Table-3). Considering safety and workability, a mechanical excavation system was adopted for upward excavation with a sliding cutter-bit disc and when the machine was stopped the face was retained by shifting the sliding cutter-bit disc over the slitted bulkhead with all openings closed. And a manual excavation system after removing the discs was employed for the downward excavation. Three driving jacks with different strokes are connected to one spreader box in order to have a uniform reaction on the circumferential segments.

4.3.2. Enlargement Shield

The enlargement shield has a doughnut-shaped face and two cylinders connected by web plates. One of them is an outer skin plate (outer cylinder) and the other is an inner plate (inner cylinder) which includes the primary segments (Fig.-11 and Table-4). Manual excavation is performed here. In order to make the machine short, driving jacks of a two stage telescopic type were attached to the hood as forward as possible. Due to narrow enlarged radial width in the tunnel section the machine was not equipped with face retaining jacks but with a structure to attach stop-logs.

4.4. DESIGN OF AUXILIARY CONSTRUCTION METHOD

4.4.1. Selection of Auxiliary Construction Method

This enlarged shield construction in saturated sand layers with open face excavation required ground improvement as an auxiliary work. As a result of comparison and examination of such methods as chemical grouting, compressed air method, ground water lowering and ground freezing, chemical grouting was selected for the ground improvement. This was because it was superior to other methods from the viewpoints of sufficient improvement effectiveness of the saturated sand layers with the clay layer, less influence on safety and the environment, and less cost. Compressed air method was also used together for perfect safety.

4.4.2. Range and Target Values of Ground Improvement

The improved range by chemical grouting is shown in Fig.-12. The target values of cohesion and coefficient of permeability are determined as $C \geqq 4.0E4$ Pa and $k \leqq 1.0E-4$ cm/sec, respectively, based on stability analysis of the face during the excavation of the circumferential shield and the enlargement shield, achievements of chemical grouting for the previous shield works, and advices of the experts for chemical grouting. Following is the result of the face stability analysis.

As for the face stability during the excavation by the circumferential shield, stresses of the half ring type structure of improved soil on which ground load acted as shown in Fig.-7 was estimated by elastic analysis. Necessary strength was obtained according to Mohr-Coulomb failure criterion, and it was found that the critical stress occurred at the inner surface of the arch structure for a 3m thick arch (major principal stress σ_1 =2.0E5 Pa and minor principal stress σ_3 =0.0 Pa) and that the necessary cohesion was C=4.6E4 Pa for the internal friction angle of ϕ=40 degree. Since this analysis was based on a plane strain model of the longitudinal section through the center of the tunnel, it would be rather safer if three dimensional dome effect was taken into account.

Stability of the face during the excavation of the enlargement shield was checked against a sliding wedge along a logarithmic spiral surface using the theory of Murayama, Honorary Professor of Kyoto University. The maximum height of 4.4m of the doughnut-shaped face was adopted here. The loosened vertical ground pressure on the slilding wedge was estimated by the loosened zone inside the improved range considering an overburden pressure on the upper surface of the improved range and a surcharge on the ground surface. As a result, in case of 3m improved width, a safety factor of 3.7 against sliding failure was obtained for the improved cohesion and the friction angle of C=4.0E4 Pa and ϕ=40 degree, respectively.

Although piping was suspected because of the existence of the sand layers with high pore pressure and a low uniformity coefficient, it was considered to be overcome by the chemical grouting, and a two dimensional FEA of steady seepage flow was carried out to confirm it. The result is shown in Table-5. It was concluded that no piping would occur, because, even in case 1 where the improving effect was assumed a lower limit, Justin's critical velocity of seepage flow Vc=2.5E-2 cm/sec (equivalent to Darcy's average velocity) for D5-D10=0.08-0.15mm of this site is ten times larger than the seepage velocity of the FEA result. Furthermore, safety margin against piping could be expected to increase, because of the increased cohesion due to the ground improvement and experienced judgement from little water leakage through the face.

4.4.3. Plan of Chemical Grouting

Chemical grouting method was selected by carrying out experimental grouting at the site under the conditions such that the saturated sand layers with high pore pressure could be improved much more than the designed values entirely and that the primary segments would not be affected. As a result, "Permeation Grouting with low pressure" (Fig.-13) was adopted since it was effective, of high workability and economical. Based on the results of the experiments, enough rough grouting beforehand was considered necessary, for the loosened ground behind the segments and for the coarse to medium sand layer. Work procedure is as follows: 1. rough grouting behind the segments, 2. the first step of descending permeation grouting with low pressure, 3. rough grouting for the loosened ground caused by primary shield advance, 4. rough grouting for the coarse to medium sand layer, 5. the rest of descending permeation grouting with low pressure continued. "Cement bentonite" (Table-6) and "Non particle silicate of long geling time" (Table-7) were used as materials for the rough grouting and the permeation grouting, respectively.

4.5. PLAN OF CONSTRUCTION MANAGEMENT

4.5.1. Plan of Shield Driving

Construction was controlled on the items and the values shown in Table-8. Strain gages were installed on the guide rings, the primary segments (normal segments and reinforced segments) and the circumferential segments so that the strains could be measured continuously as the shield advanced. At the same time, the ground behavior was also monitored by underground displacement gages to check the stability during excavation.

4.5.2. Plan of Chemical Grouting

Control values of chemical grouting used in this construction are indicated in Table-9. Table-10 shows investigations and tests carried out to assess the effectiveness of the ground improvement.

5. CONSTRUCTION WORK

Fig.-14 shows the achieved time table.

5.1. ACHIEVEMENTS OF SHIELD ADVANCE

Achievements of shield advance are shown in Table-11. The advance of the circumferential shield and the enlargement shield was almost satisfactory with regard to construction control items in Table-9. The followings are the problems which occurred during the construction.

1. Found behind the primary segments were the backfilled layer while the primary shield advance and compound cemented layers of the rough grouting and the permeation grouting. Each of them was harder than expected and had a thickness of 10 to 20 cm. Disc driving force of the circumferential shield was found insufficient since actually required was a force of 14E5 N beyond its capacity of 5E5 N. Here, these hardened layers were excavated by using a pick hammer before the disk of the shield machine was driven. However, these things should be taken into account in the future design.
2. During the advance of the enlargement shield, there occurred a maximum deformation of 50mm at the outer cylinder and that of 30mm at the inner cylinder due to the existence of the hardened layers behind the primary segments, the layers' nonhomogeneity for the advancing direction and low stiffness of the machine for the driving force. It should be investigated in the future how to design the stiffness of the enlargement shield and the backfill materials behind the primary segments.

It could be seen from the measurements in Fig.-15 that the primary segments deformed with changing stresses when removing the lower segments for constructing the starting base of the circumferential shield. After the circumferential shield started, there had been few change observed until the construction was completed.

5.2. GROUND IMPROVEMENT ACHIEVED BY CHEMICAL GROUTING

Table-12 shows the achievements of the chemical grouting, and Figs.-16 & 17 show the results of the investigation and the experiment to assess the effectiveness of the ground improvement. There is a part where the improvements are found insufficient, but the ground is in general improved well and satisfies the designed values.

6. CLOSING STATEMENT

As reported in this paper, enlargement shield tunnel construction was the first practice in its design and execution in Japan. We have successfully accomplished with safety in driving through water bearing and unstable sand with cooperations from all sources. Followings will be the major problems awaiting settlement with which we were confronted on applying this new tunneling method.

1. Development of closed type enlargement shield machine: It is necessary to develop a closed type enlargement shield machine without requiring any ground improvement for its further application to various soil conditions with more safety and less cost.
2. Evaluation process for the effectiveness of the ground improvement: It is necessary to establish the evaluation process how to check the effectiveness of the ground improvement, more easily and reliably in order to secure both safety and low cost. This is because a successful application of the open type shield in driving through unstable grounds as reported here greatly depends on the magnitude of the ground improvement.

This method will get more popular to apply in pursuing low cost, social demands for the environment and safety in a densely populated urban area. Based on this experience, this method is expected to be improved much more in the future by developing various systems depending on the construction scale and geological conditions.

Reference

K.Tsutaya et. al.,"Experimental Works to Spread the Section on Shield Tunnelling", Memoria Proceedings/ Comptes Rendus, I.T.A., Caracas, Venezuela, June 1984.

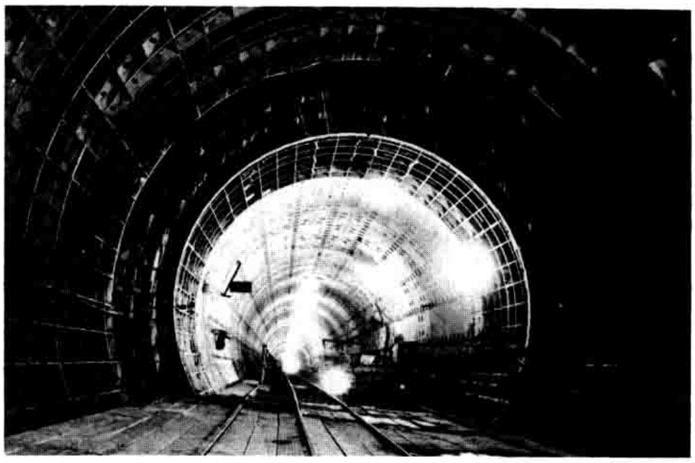


Fig.-1 Enlarged Portion of Shield Tunnel (Manhole)

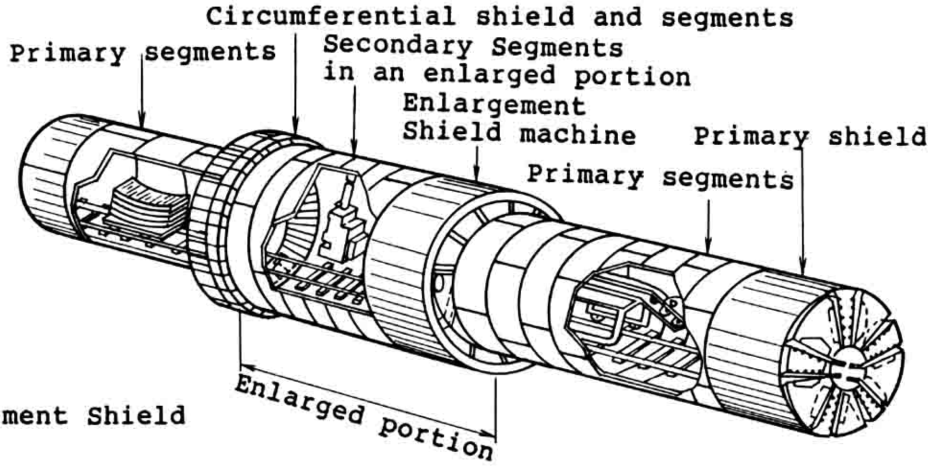
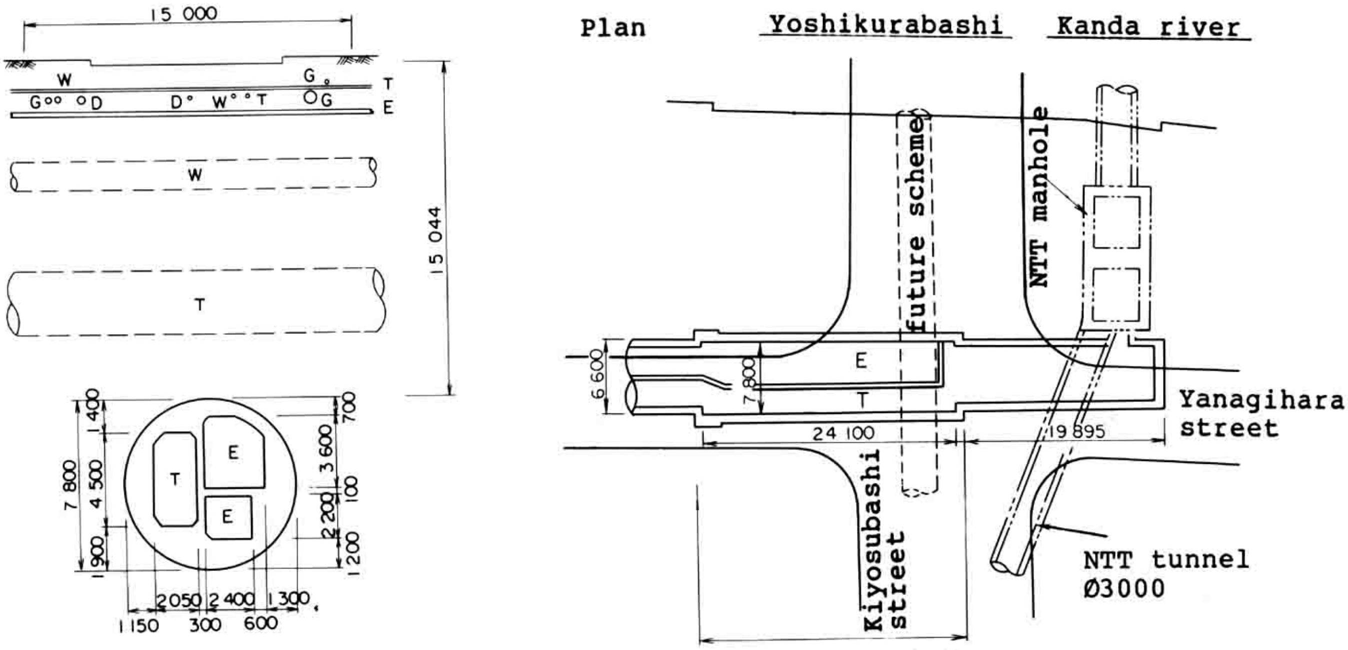


Fig.-2 Enlargement Shield



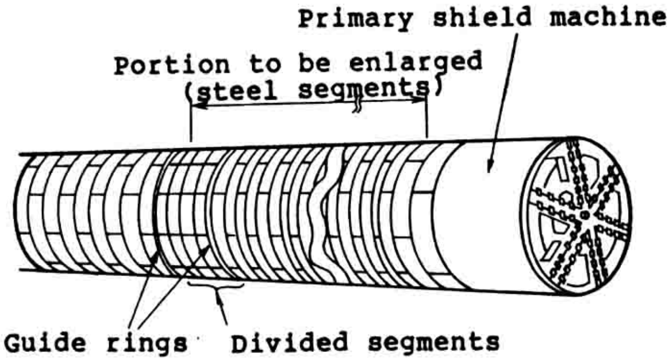
Structure of manhole by Enlargement Shield Tunneling Method

Symbol of utilities
- E: Electricity (Tokyo Electric Power Co., Inc.)
- T: Telecommunications (Nippon Telephone & Telegram Co.)
- G: Gas
- W: Water supply
- D: Drain

Fig.-3 Structure of Manhole by Enlargement Shield Tunneling Method

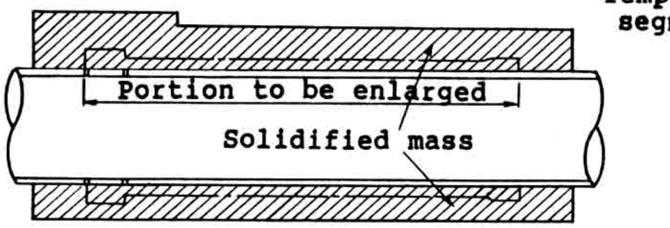
1. Primary shield pre-driving:

In the portion to be enlarged, divided segments of 450mm width are used between two guide rings for the circumferential shield at the beginning of the portion to be enlarged during the construction of primary shield tunnel.



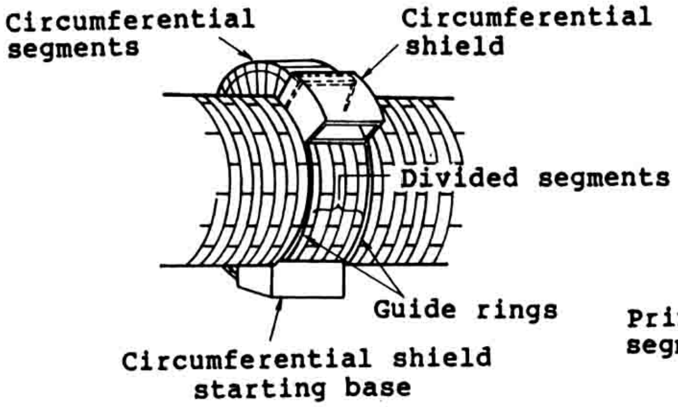
2. Ground improvement around the portion to be enlarged:

The ground, through which the enlargement shield and the circumferential shield are driven, is solidified utilizing chemical grouting beforehand.



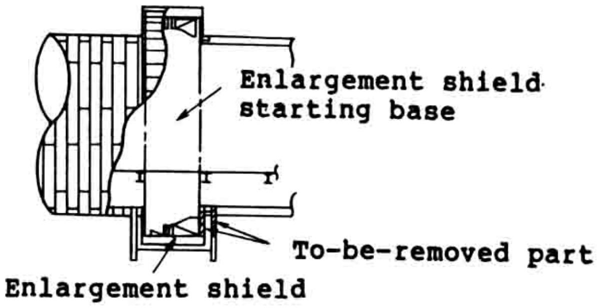
3. Construction of the base to assemble an enlargement shield (circumferential shield driving):

The circumferential shield placed on the guide rings is driven erecting the circumferential segments in order to construct a starting base for the enlargement shield.



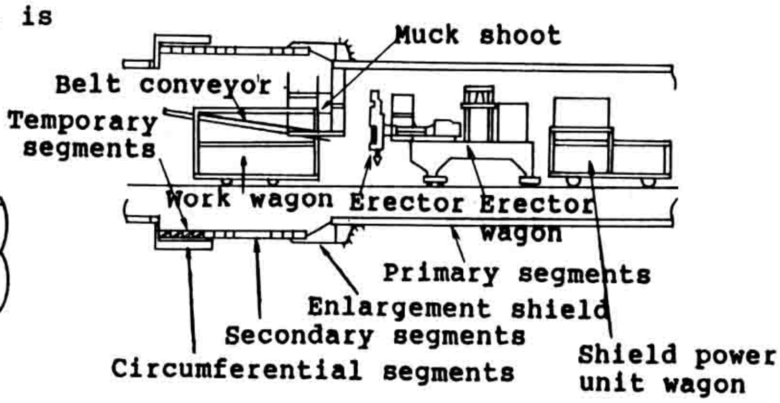
4. Assembling the enlargement shield:

Divided enlargement shield is conveyed to be built up into the base constructed by the circumferential shield.



5. Enlargement shield driving:

Enlargement shield is driven repeating the excavation by the enlargement shield, the erection of the secondary segments and the removal of primary segments.



6. Completion of enlargement:

Enlargement shield is completed upon retaining the tunnel face and removing the enlargement shield jack, after enlarged portion is excavated.

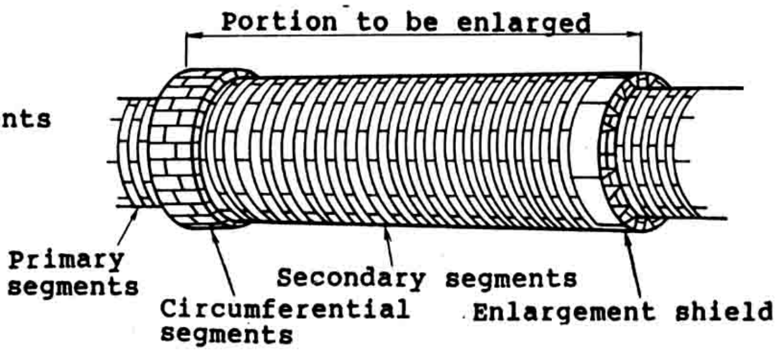
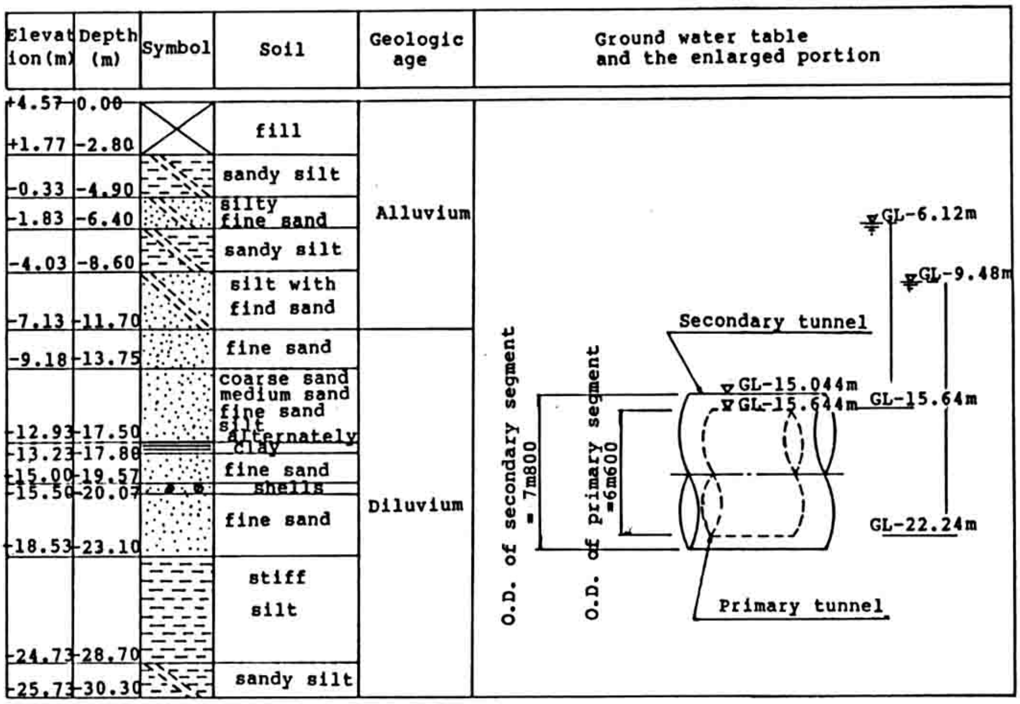


Fig.-4 Procedure of Enlargement Shield Tunneling

Fig.-5 Geological Profile



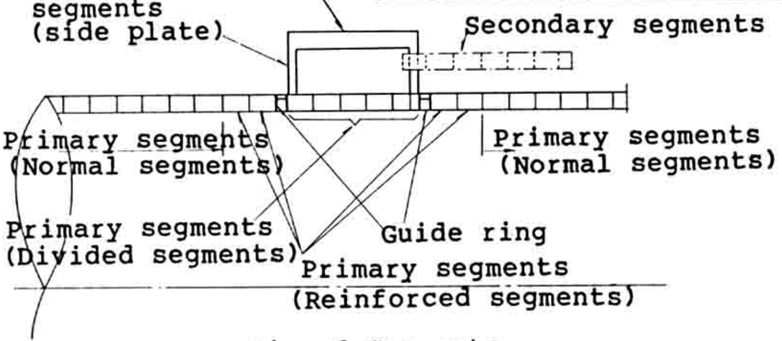


Fig.-6 Segments

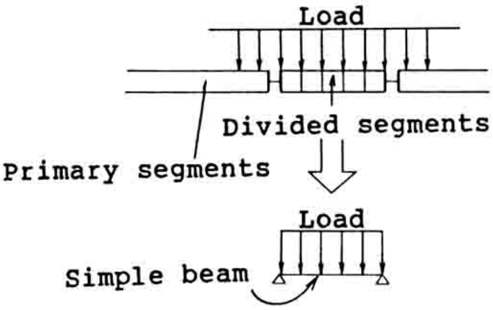


Fig.-8 Design Model of Primary Segment (Divided segment)

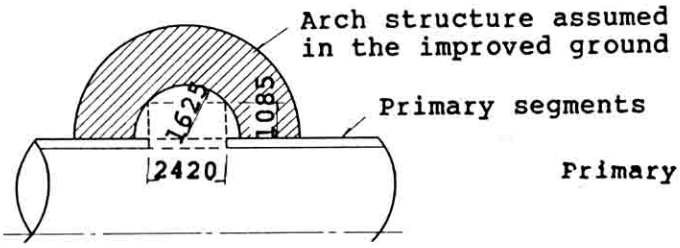


Fig.-7 Model of Designing Primary Segment (Reinforced segment) and Analysing Circumferential Shield Face Stability

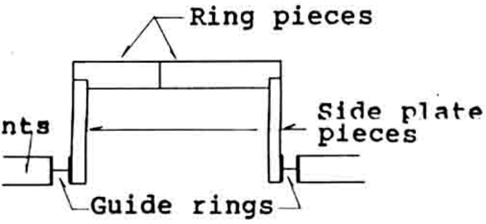


Fig.-9 Details of Circumferential Segments

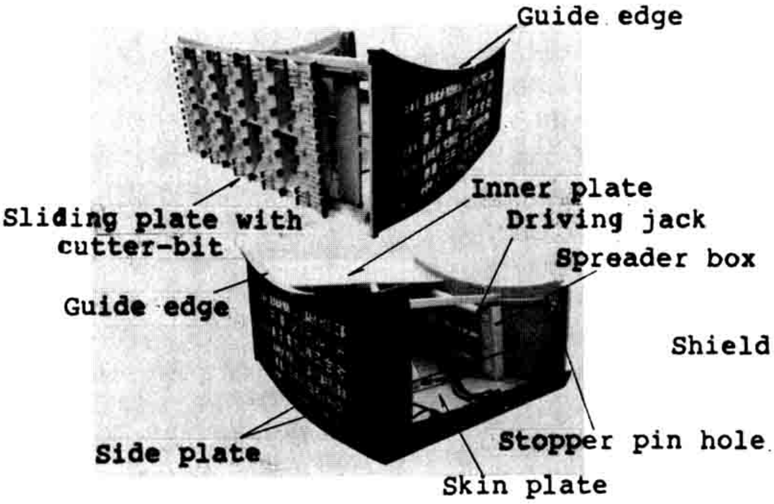


Fig.-10 Circumferential Shield Machine

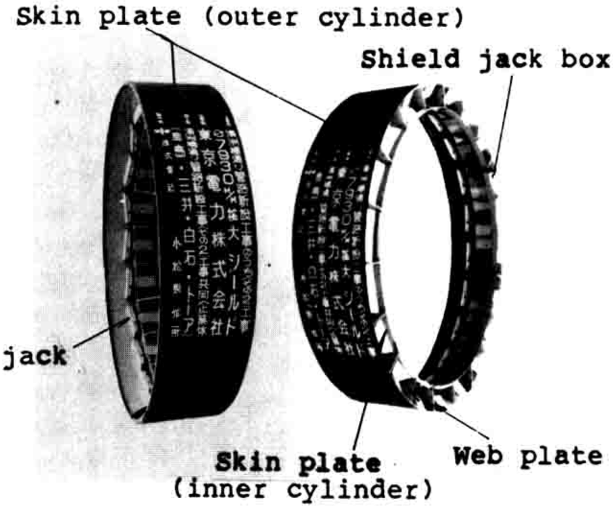


Fig.-11 Enlargement Shield Machine

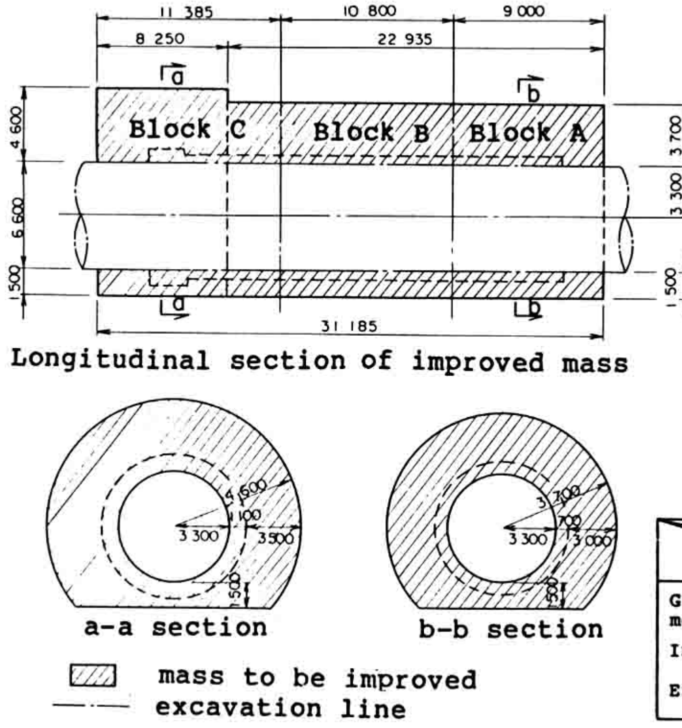


Fig.-12 Improved Mass

Note:
1. Chemical grouting in Block A
2. Chemical grouting in Block B
3. Chemical grouting in Block C
4. Installation for air compressing
5. Clearing after chemical grouting
6. Conveying circumferential shield, enlargement shield, erector and wagon
7. Assembling circumferential shield, erector and wagon
8. Setting circumferential shield
9. Demolition of circumferential shield
10. Assembling enlargement shield
11. Preparation for shield advance
12. Demolition of enlargement shield
13. The face sealing and extra grouting
14. Demolition of erector and wagon
15. Excavation for circumferential shield starting base
16. Driving circumferential shield
17. Driving enlargement shield

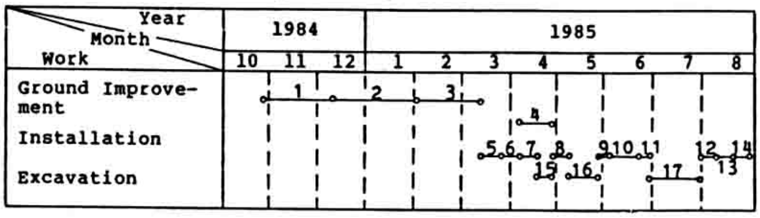


Fig.-14 Achieved Time Table

1. Alignment of grout holes:
The nipple is attached to the skin plate of the segment by welding at a desired grout hole interval of "e" on which a ball valve is located. After that, the skin plate is bored using a special drilling machine.

2. Descending drilling:
After all the grout holes are provided, the preventer is placed to carry out the first descending drilling (depth is 50 to 70 cm).

3. Grouting:
The drilling rod is pulled out after descending drilling is completed; then the valve is operated for grouting.

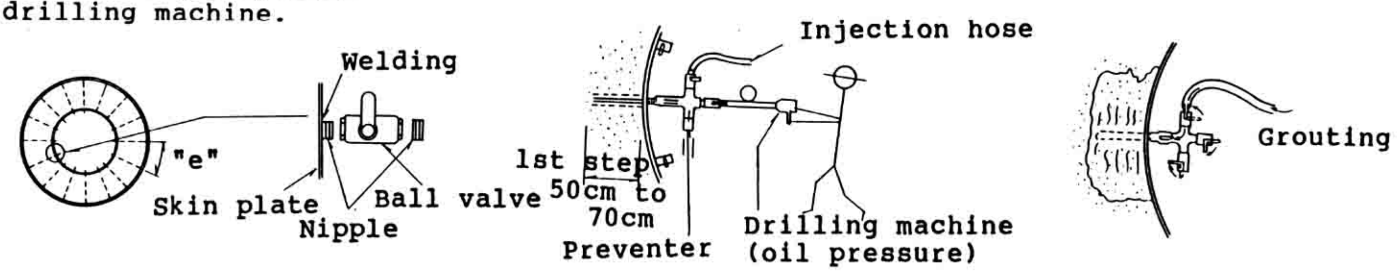
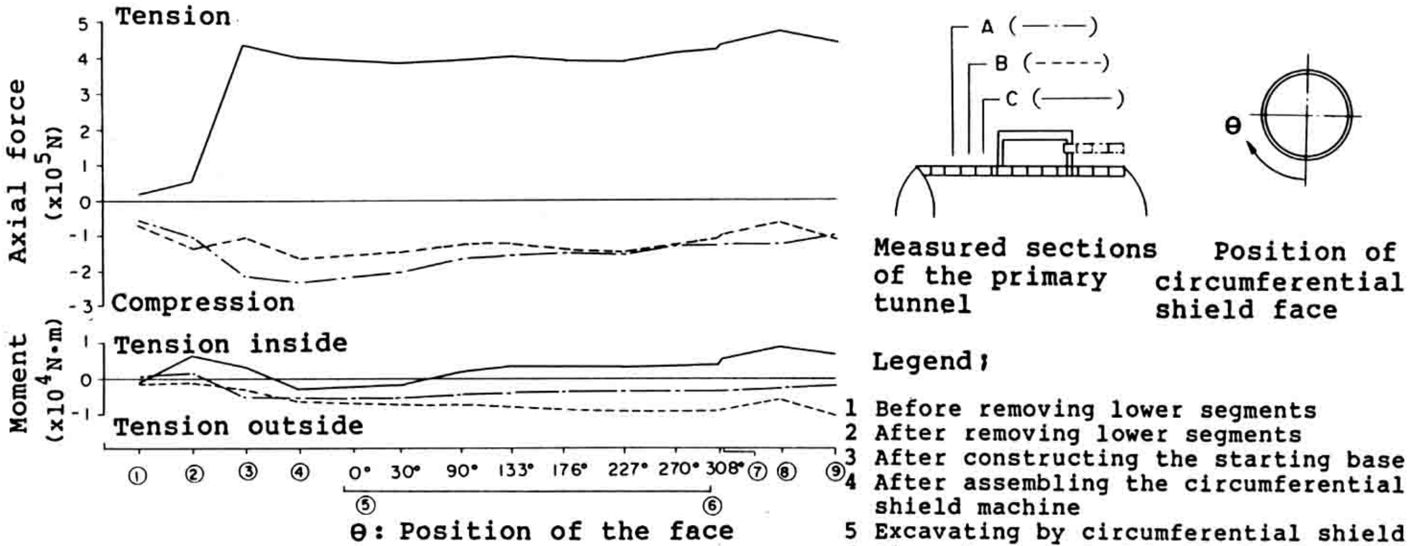


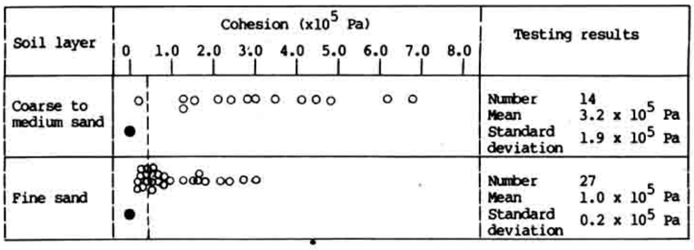
Fig.-13 Permeation Grouting with Low Pressure



Legend;
1 Before removing lower segments
2 After removing lower segments
3 After constructing the starting base
4 After assembling the circumferential shield machine
5 Excavating by circumferential shield
6 Circumferential shield arrived
7 Back-filling
8 Circumferential segments completed
9 Back-filling

Fig.-15 Diagram of Sectional Forces of The Main beam at The Bottom of The Primary Segment

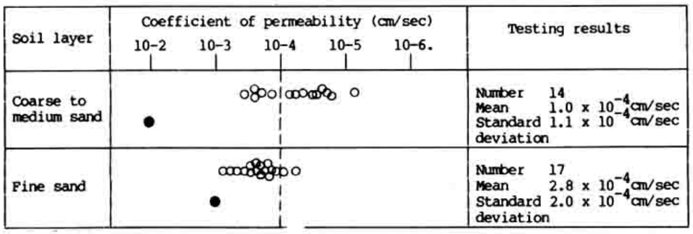
(a) Cohesion by triaxial compression test



Coarse to medium sand: Number 14, Mean 3.2 x 10^5 Pa, Standard deviation 1.9 x 10^5 Pa

Fine sand: Number 27, Mean 1.0 x 10^5 Pa, Standard deviation 0.2 x 10^5 Pa

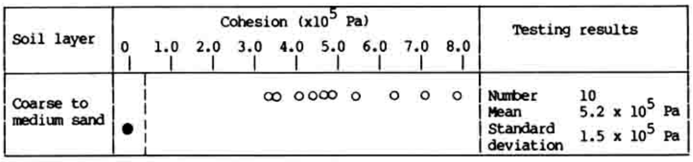
(b) Coefficient of permeability by laboratory test with triaxial test specimen



Coarse to medium sand: Number 14, Mean 1.0 x 10^{-4}cm/sec, Standard deviation 1.1 x 10^{-4}cm/sec

Fine sand: Number 17, Mean 2.8 x 10^{-4}cm/sec, Standard deviation 2.0 x 10^{-4}cm/sec

(c) Cohesion by lateral load test in a borehole



Coarse to medium sand: Number 10, Mean 5.2 x 10^5 Pa, Standard deviation 1.5 x 10^5 Pa

Legend ○ : Testing result of improved soil specimen
● : Property of pre-improved ground
---- : Designed property of improved ground

Fig.-16 Results of The Tests

Table-2 Specifications of Segment

Segment (mm)		Height	No. of pieces in a ring	Thickness of main beam	Width
Primary segment (D=6600m)	Normal Segment	300	7	16	450
	Divided Segment	300	13	16	450
	Reinforced segment	300	7	22	450
Secondary segment (D=7800mm)		300	8	16	450
Circumferential segment (D=8640mm)		300	50	19	931 1110

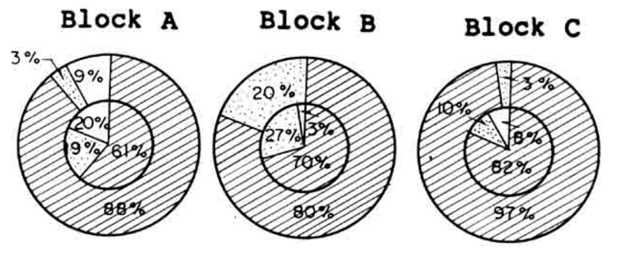
note: D stands for the outside diameter of the segment.

Table-3 Specifications of Circumferential Shield Machine

Item		No.	Specification
Main body	Length x width x height	1	2695mm x 2556mm x 1169mm
	Skin-plate thickness x material	1	25mm x SS41
	Tail seal	1	One stage of wire brush
	Driving velocity	1	0.0 to 2.97 cm/min
Jack			(x10^5N) (mm) (mm/st) (Pa)
	Driving jack (outer)	2	4.2 x Ø140 x 620 x 2.8x10^7
	Driving jack (middle)	2	4.2 x Ø140 x 590 x 2.8x10^7
	Driving jack (inner)	2	4.2 x Ø140 x 560 x 2.8x10^7
	Sliding disc jack	1	4.9 x Ø140 x 420 x 3.5x10^7
Power unit	Oil pump for driving	1	FGS 2.74 l/min x 2.7x10^7Pa
	Motor for driving	1	2.2 kW x 4P
	Oil pump for sliding	1	FGV15 16.6 l/minx3.4x10^7Pa
	Motor for sliding	1	11 kW x 4P

Table-6 Mix Proportion of Cement Bentonite (per cubic meter)

Cement (kg)	Bentonite (kg)	Water (lit.)	Strength of homo-gel (Pa)	Remarks
400	40	857	σ_{28}=12.7x10^5	For rough grouting



Note Inner circle for coarse-medium sand
Outer circle for fine sand
▨ Much reaction
░ A little reaction
☐ No reaction

Fig.-17 Rate of Reaction by Using Phenolphthalein in Each Block

Table-1 Characteristics of Sand Layer Excavated

Item \ Layer	Coarse sand layer	Medium sand layer	Fine sand layer
D10(mm)	0.14	0.20	0.14
D30(mm)	0.21	0.31	0.21
D60(mm)	0.58	0.43	0.28
Content of silt and clay (%)	5	6	5
N-value	N=30	N>50	N>50
Coefficient of permeability*(cm/s)	-	-	2.4x10^{-3}
- Do - ** (cm/s)	1.2x10^{-2}	1.7x10^{-2}	6.0x10^{-3}
Cohesion (CD test) (Pa)	-	1.5x10^4	0.0
Friction angle (CD test) (degree)	-	38	43

note: * in-situ test ** estimated from D20

Table-4 Specifications of Enlargement Shield Machine

Item		No.	Specification
Main body	Outside diameter x length x no.of pieces	1	D7930mm x 1765mm x 7
	Skin plate thickness x material		40 mm x SS41P
	Tail seal		Carbonate SBR (super weatherproofed rubber)
	Driving velocity		0 mm/min to 25 mm/min
Jack	Shield jack	20	(x10^5N) (mm/st) (Pa) 7.8 x 550 x 3.3x10^7 (two stages of jack)
Power unit	Oil pump	1	FGV15 16.6 l/minx3.3x10^7Pa
	Motor	1	11 kW x 4P

Table-5 Results of Two Dimensional Steady Seepage Flow Analysis by F.E.M.

Case	Coef. of permeability(cm/s)			Numerical results		
	Improved up to 2m behind segment	Improved up to 2m to 3.7m behind segment	Un-improved	Quantity of flow from the face (lit/min)	Hydraulic gradient	Darcy's average velocity (cm/s)
1	4.0x10^{-4}	5.0x10^{-3}	5.0x10^{-2}	62	7 to 9	0.3x10^{-2} to 0.4x10^{-2}
2	4.0x10^{-6}	3.0x10^{-4}	3.0x10^{-3}	0.87	10 to 12	0.4x10^{-4} to 0.5x10^{-4}

Table-7 Mix Proportion of Non Particle Silicate of Long Geling Time (per cubic meter)

Special silicate (lit.)	Hardener (lit.)	Water (lit.)	Remarks
400	25	275	For permeation grouting with low pressure (PSG-III)

Table-8 Control Item of Shield Advance

Work	Control item		Content	Method		Target values
Excavation	Retaining face	Strength of the ground	Cohesion ($C \geqq 4 \times 10^4$ Pa) Friction angle ($\phi \geqq 40^\circ$)	Phenolphtalein reaction rate (monitoring)		Over 70% for coarse-medium sand, 100% for fine sand
		ground water	Sweating of face Little piping	Monitoring		Not available
	Soil excavated	Quantity	Target value [1.27m^3/set] (6.84m^3/ring)	Counting wagons		[1 wagon] (5–6 wagons)
		Quality	Checking alkali	Phenolphtalein reaction (monitoring)		Not available
	Cutter-sliding force		Jack driving force [less than 4×10^5 N]	Measurement by a pressure gage		[less than 2.8×10^5 Pa]
Advance	Normal advance		Deviation of the shield	Monitoring the clearance to primary segment		No deviation
	Driving force		Jack driving force [less than 2×10^6 N] (less than 1×10^7 N)	Measurement by a pressure gage		[less than 2.2×10^7 Pa] (less than 2.1×10^7 Pa)
Compressing air	Air pressure		The consumption air	Quantity of the in-air	Measurement by a flow-meter	More than 10 m^3/min
				Consumption in the ground	- Do -	Less than 40 m^3/min
Backfilling	Quantity		Designed quantity of grouting	Counting batches		More than designed quantity of grouting
	Grouting pressure		Allowable pressure	Measurement by a pressure gage		Less than 3.0×10^5 Pa
	Material		Geling time	Experimental measurement of a geling time		60 seconds
Measurement	Streeses of guide ring, primary segment and circumferential segment		Target stress	Measurement by strain gages		Less than designed values (1st step) Less than allowable stresses (2nd step)
	Settlement		Settlement of the ground surface and gas pipes	Level measurement		less than 5mm
			Underground settlement	Measurement by underground displacement gage		

Note: [ ]; while the circumferential shield advancing
( ); while the enlargement shield advancing

Table-9 Control Values of Chemical Grouting

Item	Content
Injection pressure	Less than 5×10^5 Pa for permeation grouting with low pressure Less than 5×10^5 Pa for rough grouting
Injection velocity	8 lit./min
Quantity of grouting	Up to 8×10^5 Pa of injection pressure and 40% of max. injection rate for rough grouting Up to 5×10^5 Pa of injection pressure and 40% of injection rate for permeation grouting with low pressure
Geling time	Long geling time of 15 min. to 25 min
Deformation of segment	Deformation of main beam to be less than 7mm
Quantity of seepage	Check with ø19 mm rod drilling for 1st step to 6th step (1st step to 8th step in the portion of circumferential shield) Check with ø34 mm rod drilling for the final step
Ground heave	Less than 10mm of ground heave (including settlement of gas pipes)
Quality of ground water	pH <8.6 Under 10 ppm. of consumed $KMnO_4$

Table-11 Achievements of Shield Advance

Item	Circumferential shield	Enlargement shield
Duration	14 days (8/5/85 to 21/5/85)	36 days (24/6/85 to 29/7/85)
Average excavation	4 sets per day (45cm a set)	2 rings per day (45cm a ring)
Average excavated volume	5 m^3 per day	14 m^3 per day
Average excavation time	2h per set	5h per ring
Average time to assemble segments	1h per set	1.5h per ring
Pressure of compressed air	Max. 9×10^4 Pa Min. 3×10^4 Pa	6×10^4 Pa

Table-10 Content of Investigations and Tests to Assess Effectiveness of Ground Improvement

Item	Content
Experimental excavation	The experimental excavation through a 0.4m x 0.4m square hole cut out from the skin plate is carried out with 0.7m to 1.1m depth in order to observe improved conditions, investigate quantity of seepage and carry out a phenolphtalein test.
Investigation of quantity of seepage	Quantity of seepage is investigated through a hole drilled up to 2.0m to 2.5m depth with ø76mm boring.
Laboratory test	The sample core is taken from a hole drilled up to 2.0m to 2.5m depth with ø116mm boring to carry out a triaxial compression test (CD), laboratory permeability test with triaxial test specimen and phenolphtalein test.
In-situ test	In-situ lateral load test is carried out in a hole drilled up to 2.0m to 2.5m depth with ø65mm boring.

Table-12 Achievements of Chemical Grouting

	Rough grouting (cement bentonite)			Permeation grouting (non particle silicate)	Extra grouting (non particle silicate)
	Back filling	Disturbed portion due to excavation	Coarse to medium sand layer		
Quantity (m^3)	42.4	52.6	83.3	1442	145
	178				
Soil volume to be grouted (m^3)	3502				
Grouting rate (%)	5			41	4
Grouting pressure	Max. 5.4×10^5 Pa ; Min. 3.3×10^5 Pa				