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Performance of linings for shield driven tunnels – A survey on Japanese shield tunnelling

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1. INTRODUCTION

This paper is prepared as a part of the study related to a survey on Japanese Shield Tunneling, which was carried out by the Japanese Society of Soil Mechanics and Foundation Engineering — Committee on "Underground Construction in Soft Ground" (JSSMFE TC-28).

The design of shield tunnel lining commonly determines the strength of the lining members by regarding the earth and water pressures as loads acting on the lining rings supported by the ground so that these lining members are strong enough vis-a-vis the stress generated by such loads. Consequently, it is essential for the modelling process to take the load acting on a tunnel and the state of tunnel support into proper consideration. However, the earth pressure phenomena acting on a tunnel are so complicated that it is virtually impossible to determine the nature of such phenomena solely by theoretical analysis, making measurement of the earth and water pressures acting on an actual shield tunnel extremely important.

In this paper, the existing measuring methods and their problems are discussed first, followed by a review of the characteristics of the earth and water pressures acting on a tunnel.

2. CURRENT STATUS AND PROBLEMS OF MEASURING METHODS

There are 2 methods to measure the earth pressure, ground reaction and water pressure acting on a shield tunnel. One is the direct measuring method using pressure sensors placed behind the segment, while another is the indirect measuring method to estimate the pressure in question by measuring distortions of the segment. In practice, both of these methods are seldom exclusively used and the appropriateness of the measuring results is confirmed by the concurrent use of both methods.

Table 1 Main measuring examples of earth pressure and water pressure

Name of Tunnel	Segment		Shield	Back-Filling		Earth Pressure Gauge			Measuring Period	Remarks
	Outer Diameter (m)	Material	Machine Type	Method	Material	Type	Diameter (mm)	Quantity		
JR Sobu Line: Yanagibashi (Yamaguchi, 1976)	7.06	Special ST	Manual (Compressed Air)	Segment grout hole (post-digging)	Mortar	Special Copper Ring	90 × 100	40/2R	16 months	Entire segment ring used as pressure receiving face
JR Yokosuka Line: Yurakucho (Yamaguchi, 1976)	7.06	RC	Manual (Compressed Air)	Segment grout hole (post-digging)	Mortar	Differential Transformer	60	21/3R	18 months	
JR Yokosuka Line: Hamamatsucho (Yamaguchi, 1976)	7.06	SRC	Mechanical (Compressed Air)	Segment grout hole (post-digging)	Mortar	Differential Transformer	60	18/3R	17 months	
JR Keiyo: Sumidagawa (Ide, 1989)	7.10	RC	Slurry	Segment grout hole (on driving)	Mortar Plastic	Distortion Gauge	30	8/1R	4 months	
NTT Service Tunnel Between Kasumigaseki and Higashi Ginza (Sugano, 1991)	4.55	ST	Slurry	Shield	Plastic	Distortion Gauge	NA	8/1R	5 months	
Chuden Chita Nambu-Taira-machi Line No. 1 (Nishino, 1988)	4.50	RC	Slurry	NA	NA	Distortion Gauge	NA	8/1R	2 years 4 months	
Chuden Chita Nambu-Taira-machi Line No. 2 (Nishino, 1988)	4.50	RC	Slurry	NA	NA	Distortion Gauge	NA	8/1R	1 year 9 months	
Tohoku Shinkansen No. 2 Ueno (Iida, 1985)	12.7	RC	Semi-Mechanical (Compressed Air)	Segment grout hole (on driving)	Mortar	Differential Transformer	50	6/1R	2 years	
Hakodate Airport Water Channel (Abiko, 1983)	6.70	RC	Semi-Mechanical	NA	NA	Distortion Gauge	NA	4/1R	NA	
Unknown (Goto, 1988)	3.55	RC	Slurry	*	*	Distortion Gauge	3	4/1R	1 month	*Primary injection of retempered soil

2.1 Direct Measuring Method

In the case of the direct measuring method, earth pressure gauges (load cells) and piezometers are placed at reasonable interval in the circumferential direction and the obtained values are separated into the earth pressure and water pressure values. Many types of earth pressure gauges are actually available and Table 1 shows those gauges used for main measuring purposes in Japan. These gauges can be classified in terms of the shape and size of the pressure receiving face. In general, the most popular gauge consists of a circular plane with its cross-section bent in accordance with the curvature of the back of the segment. The diameter of the pressure receiving face varies from 30mm to 500mm. In some cases, a special gauge with the entire back of the segment used as the pressure receiving face is used. In this case, the back of the segment is divided every 110cm, each pressure receiving section is supported by steel pipes and the earth pressure is computed based on the distortion of the pipes (Fig. 1).

In many cases, 4 to 8 gauges are placed for a single ring. In special cases, including the case where the detection of the possible impact of close work on a structure is aimed at, a smaller number of gauges are employed. In general, only one ring is subjected to measuring although 2 or 3 continuous rings are subjected to measuring in some cases.

As the earth pressure measured by these methods has the inherent problem described next, the limitations in terms of both the interpretation and application of the measured earth pressure must be properly understood.

① At ground where arching occurs, the earth pressure recorded by gauges on the tunnel wall in fact originates from the relative displacement between the pressure receiving face of the gauges and the neighbouring tunnel wall(s). There are several types of such relative displacement.
- Relative displacement between the section near the main beam and the longitudinal rib and the intermediate section of the rib.
- Relative displacement between adjoining rings.
- Relative displacement caused by the difference between the rigidity of the pressure receiving face and the rigidity of the segment wall.

To avoid these problems, it is necessary to understand the conditions of localized changes by enlarging the pressure receiving face or by installing many earth pressure gauges in both the cross-sectional and longitudinal directions to establish the average value of localized changes in earth pressure. Some cases have adopted an increased number of rings where earth pressure gauges are installed or an increased number of gauges. These solutions, however, are not only costly but also tend to disrupt the tunneling work. The case listed in Table 1 uses retempered soil for primary injection to avoid the problems associated with small diameter earth pressure gauges.

② The earth pressure recorded by an earth pressure gauge is the earth pressure acting on the pressure receiving face at a right angle, making it impossible to estimate the actual distribution form of the earth pressure, including the tangential element. This problem occurs regardless of the type or location of the earth pressure gauge. As it is difficult to directly infer the distribution form and size of earth pressure, this inference must be made by assuming a theoretical distribution form of earth pressure.

③ The presence of back-filling material between the ground and the earth pressure gauge as described earlier largely affects the measurement value of earth pressure depending on the actual back-filling method and material used.

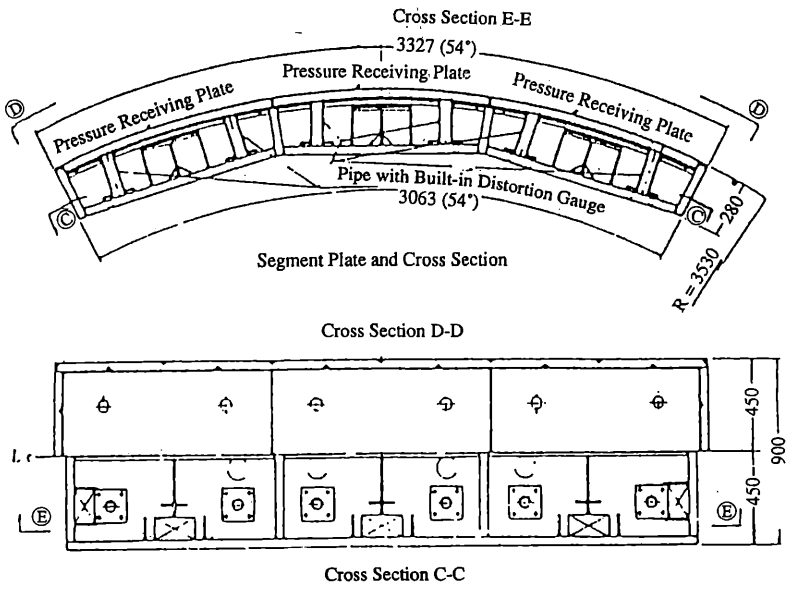

Fig. 1 Steel ring-type earth pressure gauge

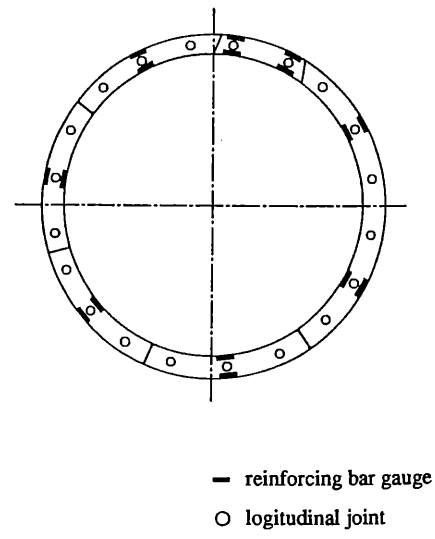

Fig. 2 An example of reinforcing bar gauges layout

2.2 Indirect Measuring Method (Measurement of Lining Distortion)

The earth pressure or water pressure can be estimated by comparing the sectional force computed based on the segment distortion and the sectional force computed from the assumed earth pressure or water pressure. This method is not necessarily used to obtain the earth pressure or water pressure but can be used to confirm segment safety.

In such a case, the measurement of distortion is conducted using reinforcing bar gauges inserted in a concrete segment or distortion gauges mounted to the main beam and other sections of a steel segment. In many cases, one gauge is placed for each segment piece in the circumferential direction (Fig. 2). In special cases, 2 or 3 gauges are used to determine the segment characteristics. A total of 6 to 12 gauges, more than the number of earth pressure gauges, are installed per ring. Two or 3 rings are often measured together in view of the differences of the generated sectional force depending on the locations of the segment joints.

The data so obtained have the following inherent problems, making awareness of the limitations of data interpretation or application essential.

① It is difficult to uniformly determine the load distribution which generates the sectional force and, therefore, the measured sectional force generated simply supplements the theoretical assumption.

② As in the case of the direct measuring method, it is difficult to determine the relative displacement between the segment rings or the impacts of back-filling operation.

③ Since the distortion to be generated depends on the presence of segment joints and minor structural differences in the segments, it is necessary to calibrate the distortion gauge under the load condition of combined axial force and bending moment. This calibration requires the actual assembly of 2 or 3 rings, making it a large-scale test. Therefore, this calibration is not normally conducted.

In some cases, the earth pressure and water pressure of the neighbouring ground are also measured. According to this method, the earth pressure acting on the tunnel is estimated from the in-situ earth pressure of adjoining areas. Having said this, it is difficult to obtain the earth pressure which directly acts on the tunnel from the measured values of in-situ earth pressure. Furthermore, the common use of a boring hole to obtain this data makes it difficult to establish data around the lower half of the tunnel.

3. MEASUREMENT EXAMPLES AND THEIR APPRAISAL

The phenomena of earth pressure and water pressure acting on a shield tunnel are very complex and difficult to quantify. Here, 12 cases of actual measurement with relatively good results are cited from existing literature in view of analyzing these phenomena (Table 2). As shown in Table 3, these cases were first classified in terms of the soil conditions and shield type, etc. and the analysis of such features as the impacts of back-filling, stabilization timing of earth pressure, comparison with the theoretical soil or water pressure and the impacts of close work, etc. was conducted.

3.1 Changes with Passing of Time and Impacts of Back-Filling

Back-filling has substantial impacts on the earth pressure acting on a shield tunnel and the ground reaction to it. Such impacts can be substantiated by the measurement results at actual sites.

Table 2 Subject tunnels for analysis

Symbol	Tunnel Name (Author)	Segment Type	Segment Outer Diameter (m)	Approximate Earth Cover (m)	Grouting Timing
A	JR Yanagibashi (Yamaguchi, 1976)	Special Steel	7.06	23	2 ~ 3 Rings Rear of Shield Tail
B	JR Yurakucho (Yamaguchi, 1976)	RC	7.06	31	2 ~ 3 Rings Rear of Shield Tail
C	JR Hamamatsucho (Yamaguchi, 1976)	RC	7.06	25	2 ~ 3 Rings Rear of Shield Tail
D	JR Ueno No. 2 (Sugano, 1991)	RC	12.55	19	Nearly Simultaneous with the Shield Advancing
E	JR Ecchujima (Mori, 1988)	RC	7.20	12	Simultaneous with the Shield Advancing
F	JR Sumidagawa (Ide, 1989)	RC	7.10	13	Simultaneous with the Shield Advancing or After Digging
G	JR Shin Hacchobori (Maeda, 1989)	RC	8.10	22	NA
H	JR Kyobashi* (Shimizu, 1989)	RC	7.20 - 11.97	25	Instant
I	Kasumigaura Headrace (Ito, 1991)	RC	5.10	16	NA
J	NTT Service Tunnel (Sugano, 1991)	Steel	4.55	38	Simultaneous with the Shield Advancing
K	Chuden Service Tunnel No. 1 (Nishino, 1988)	RC	4.50	14	Simultaneous with the Shield Advancing or After Digging
L	Chuden Service Tunnel No. 2 (Nishino, 1988)	RC	4.50	12	Simultaneous with the Shield Advancing or After Digging
M	Unknown (Goto, 1988)	RC	3.55	8	Special
N	Unknown (Kamemura, 1982)	RC	3.25	3	2 ~ 3 Rings Rear of Shield Tail

* Multi-Circular Cross-Section

In practice, the impacts of back-filling are measured for most tunneling work. In terms of the back-fill grouting method, at Tunnels A, B and C where grouting was conducted from 2~3 rings rear, the actual grouting commenced more than 24 hours after the shield tail segment ring moved out of the tail. Upto this point, the measured earth pressure rose little but a noticeable rise of the earth pressure occurred at the time of grouting, followed by gradual stabilization (Fig. 3). In contrast, at those tunnels where simultaneous or nearly simultaneous grouting with the shield advancing was conducted, high earth pressure due to grouting was recorded immediately after the segment ring pulled-out of the tail, followed by a gradual decline. In the case of Tunnel M, grouting caused a sudden, large increase of the earth pressure when then gradually declined over a few days, stabilizing at a certain level (Fig. 4). This trend of decline lasted for several months, however, in the case of Tunnels K and L (Fig. 5). At Tunnel D where nearly simultaneous grouting was opted for, the increased earth pressure after grouting remained stable at a high level.

The differences observed in the measured earth pressure were presumably caused not only by the different soil conditions and different shield employed but also by the different back-filling materials. In the case of open-type shield tunneling, the common practice is to drain the groundwater while tunneling and mortar or rapidly hardening mortar is used for grouting. When the grouting pressure is too high, the mortar may find its way to the face or leak through the tail seal, making it necessary to employ a low grouting pressure. Under a low grouting pressure, the active earth pressure is conveyed to the segment and creeping of the grouted materials seldom occurs, leading to a steady earth pressure which is increased due to the grouting pressure. Closed-type shield tunneling, where the groundwater pressure is retained during tunneling, has been gaining popularity in recent years. The back-filling materials are mainly plastic materials by 1.5 shot grouting using a high grouting pressure. The initial excessive grouting pressure is believed to gradually decline due to the consolidation and/or creeping of the materials and neighbouring ground.

Table 3 Types of shield and soil conditions of tunnels

	Predominant Soil Condition Near Tunnel			
	Loose Sandy Soil	Dense Sandy Soil	Soft Cohesive Soil	Stiff Cohesive Soil
Open-Type	———	A, B, D	C	———
Closed-Type	I	G, H, J, L	E, F, M, N	K

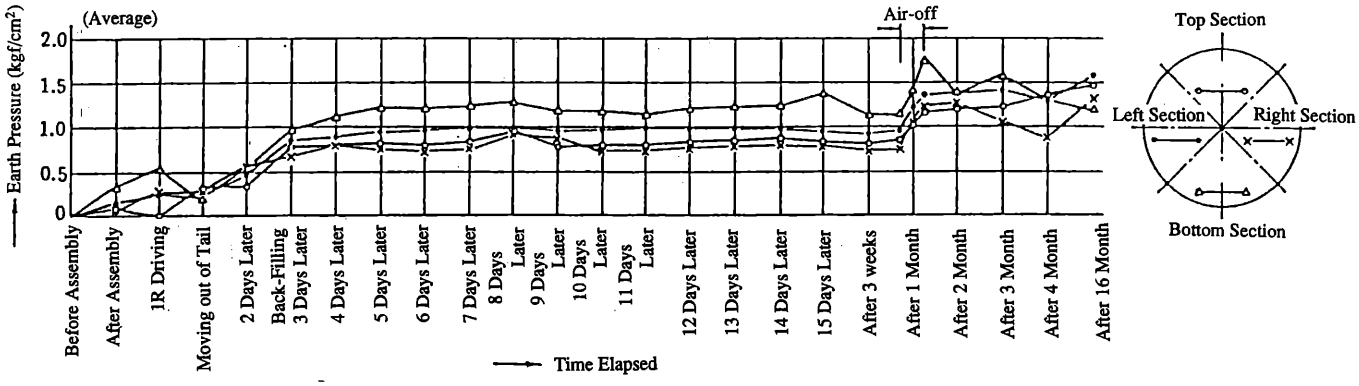

Fig. 3 Changes of actual earth pressure with passing of time (tunnel A) (Yamaguchi, 1976)

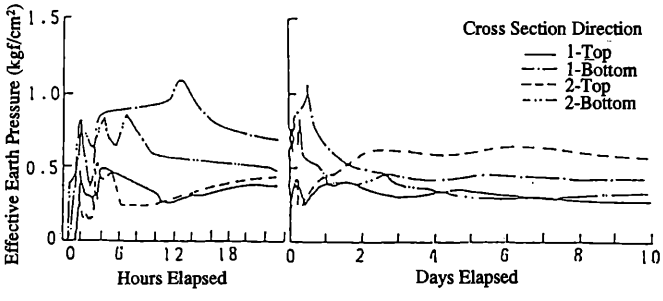

Fig. 4 Changes of earth pressure with passing of time (tunnel M) (Goto, 1988)

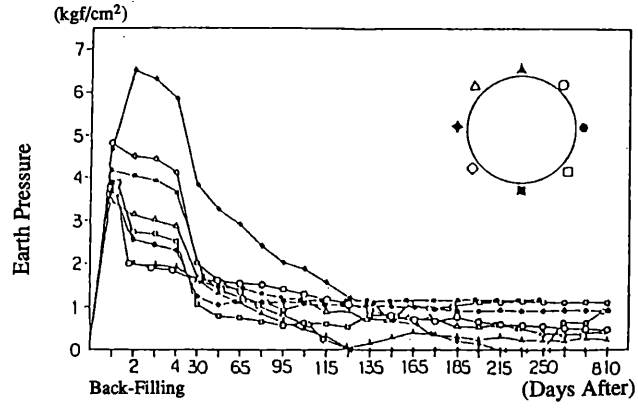

Fig. 5 Changes of earth pressure with passing of time (tunnel K) (Nishino, 1988)

In the case of Tunnel E involving soft cohesive soil, the earth pressure gauges sharply reacted to the grouting for the neighbouring shield tunneling work (Fig. 6).

Secondary grouting was conducted for both Tunnels B and E, causing a sharp increase of the earth pressure. In the case of Tunnel B, the large stress which had emerged in several areas was reduced, indicating that the secondary grouting of high liquidity material helped to level the stress distribution over the entire ring (Fig. 7).

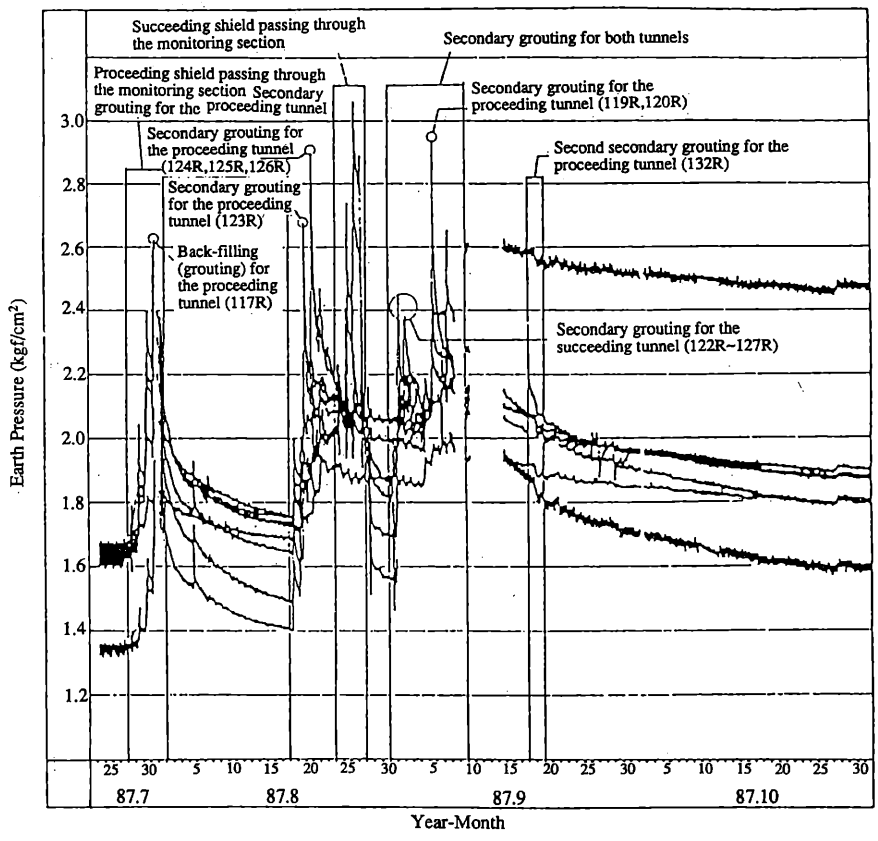

Fig. 6 Change of underground soil pressure with passing of time (tunnel E) (Mori, 1988)

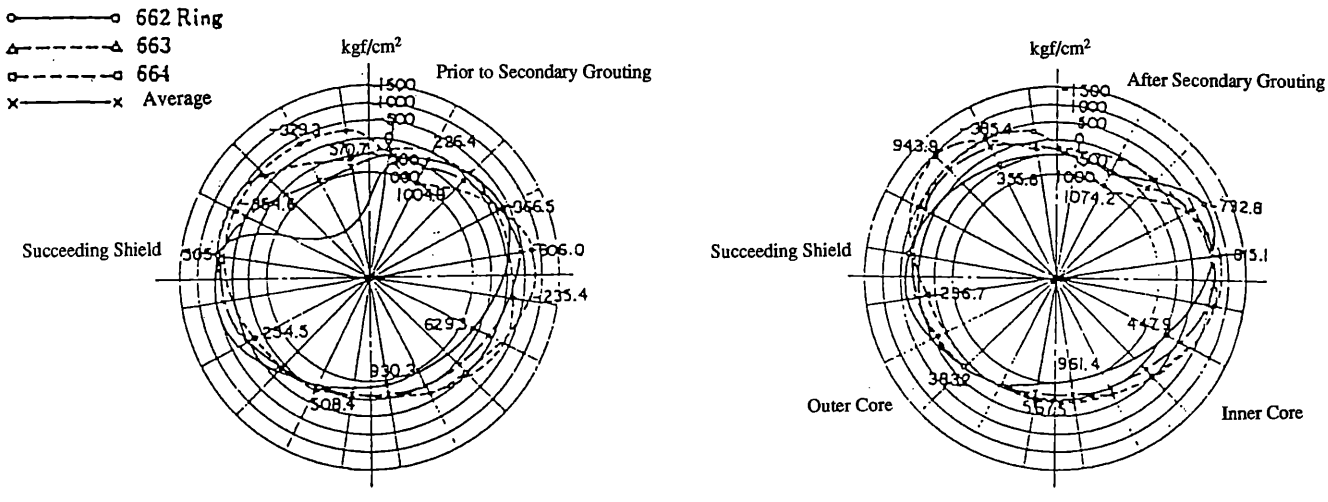

Fig. 7 Impacts of secondary grouting (tunnel B) (Yamaguchi, 1976)

Some cases suggest the occurrence of uneven earth pressure due to back-filling. As such uneven earth pressure levelled over a period of 9 days, the earth pressure appeared to stabilize as well as level in one or 2 weeks (Fig. 8). The different soil conditions did not show as conspicuous impacts on the earth pressure as the grouting method. In short, the earth pressure shows an abrupt change due to grouting or secondary grouting but tends to stabilize at a certain level after a few days or few months.

3.2 Comparison with Theoretical Earth Pressure Values

The long-term earth pressure acting on the shield tunnel lining is an important element in tunnel design and a comparison was made between the measured values and design values (theoretical values) for some of the tunnels. At Tunnel C where the soil was a soft cohesive soil, although the whole overburden pressure was not actually measured, the earth pressure tended to increase with the passing of time after the air off. The stress of the reinforcing bars also continued to rise (Fig. 9 and Fig. 10). Similarly at Tunnel M where the soil was also a soft cohesive soil, the recorded earth pressure was similar to the loosening earth pressure while the water pressure well corresponded with the hydro static pressure distribution computated based on the groundwater level. At Tunnel F, the computated whole overburden pressure based on the assumed unit volume weight of the ground of 1.7 tf/m^3 was almost identical to the actual value. Given the fact that Tunnel M is located in intercalated layers of sandy silt and sandy clay, both classified as soft

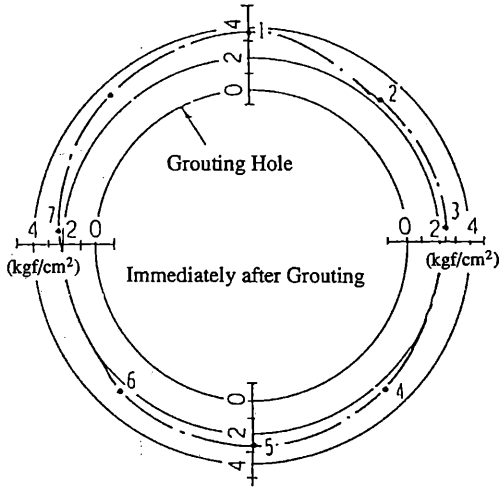

Fig. 8 Impacts of back-filling (tunnel J) (Sugano, 1991)

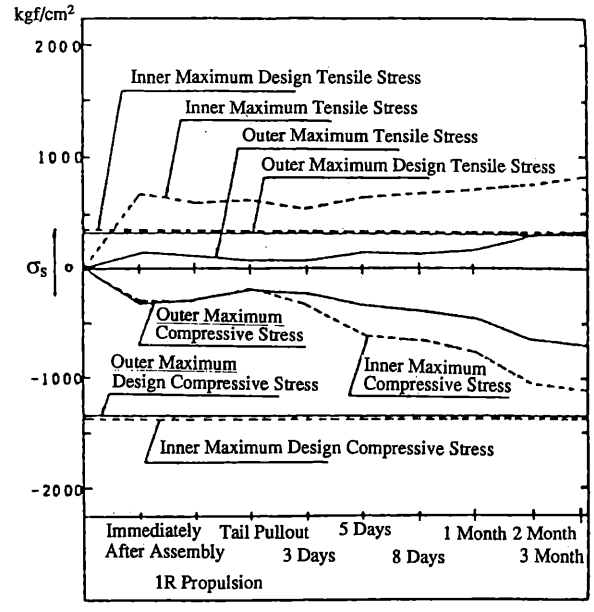

Fig. 10 Changes of reinforcing bar stress (tunnel C) (Yamaguchi, 1976)

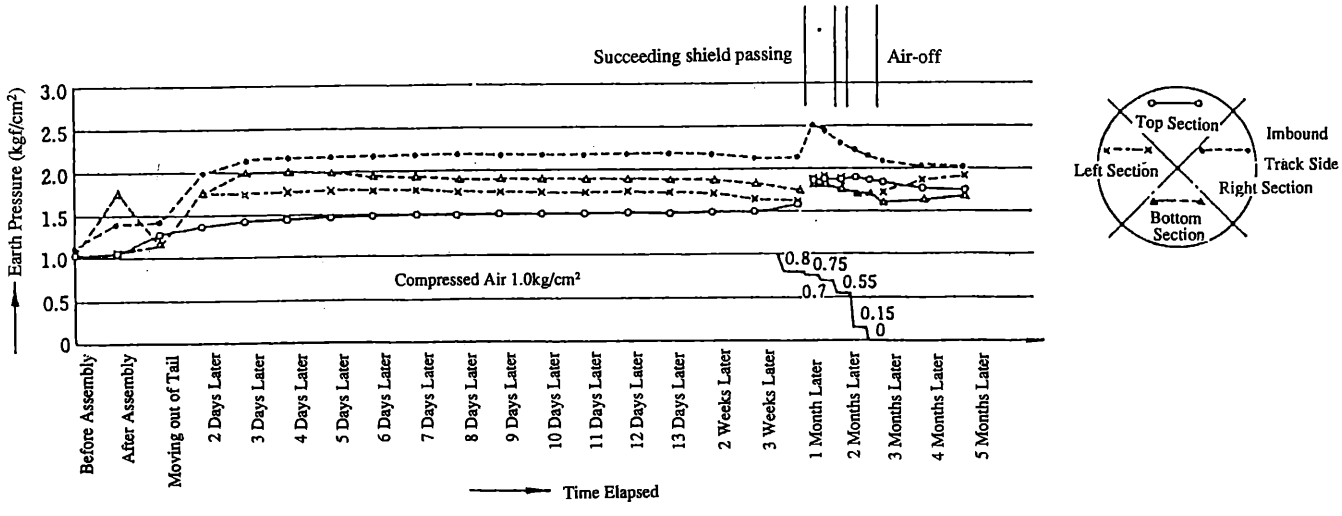

Fig. 9 Changes of earth pressure with passing of time (tunnel C) (Yamaguchi, 1976)

cohesive soil, the ground must have a sufficiently large internal frictional angle to produce the state of loosening earth pressure. The measurement results of Tunnels C and F suggest that even if a smaller earth pressure than the whole overburden pressure is temporarily recorded, creeping of the ground and other factors tend to gradually increase the earth pressure to approximately the whole overburden pressure.

At Tunnel K where the soil is a stiff cohesive soil, the recordings of the earth pressure gauges and those of the piezometers are almost equal, suggesting an extremely low acting earth pressure (Fig. 11).

With regard to sandy soil, a comparison between the theoretical earth pressure and measured earth pressure was conducted for Tunnel I located at loose ground. The recorded earth pressure was small, even compared to the loosening earth pressure, while the water pressure was found to well correspond to the distribution of the hydro static pressure. The water pressure accounted for some 80 - 90% of the total pressure (Fig. 12).

At Tunnels A and B which employed the open-type shield tunneling method and where the soil is a dense sandy soil, the recorded earth pressure was almost equal to or slightly smaller than Terzaghi's loosening earth pressure (Fig. 13). In the case of Tunnel J where closed-type shield tunneling was conducted, the actual earth pressure value was slightly smaller than the common theoretical value which took the loosening earth pressure and ground reaction into consideration. The portion of the effective earth pressure in the recorded value was extremely small at the converted earth column height of 2m equivalent. In the case of Tunnel L with similar conditions, the pore water pressure was said to account for 87% of the total pressure recorded by the earth pressure gauges, implying that the effective earth pressure was less than Terzaghi's loosening earth pressure.

All the above observations appear to indicate that the earth pressure acting on a tunnel in sandy soil has a ceiling of Terzaghi's loosening earth pressure regardless of loose or dense ground and that the water pressure plays a dominant role for deep tunnels.

3.3 Impacts of Close Work

In the case of Tunnel N, the shield tunneling took place almost diagonally above an existing tunnel. In the case of Tunnels F and G, parallel shield tunneling was conducted in an overlapping period. The deformation and earth pressure on the preceding shield segment were measured to determine the impacts of the succeeding shield tunneling on the preceding tunnel. According to such data for Tunnel N, when the face of the new shield approached 2 -3m of the preceding tunnel, both the earth pressure and bending moment began to increase with the tunnel cross-section being squeezed. The recorded values reached the maximum point when the shield reached the face of the preceding shield work. The earth pressure by this time had increased by 0.5 kgf/cm^2 compared to its original level (Fig. 14). It gradually declined during the passing of the shield, subsequently returning to the original level. A sudden decline of the earth pressure was recorded when the tail passed the measuring point but later moved back towards the original level with the passing of time. Data during the approach by the succeeding face indicated that the preceding shield tunnel was pressurized by the succeeding shield. Because back-fill grouting at the 2-3 rings rear of shield tail for Tunnel N, ground reaction was released during the passing of the tail section but the ground was gradually arrested by the subsequent grouting. While no such detailed observation was conducted for Tunnels F and G, it was obvious that the earth pressure acting on the segment of the preceding tunnel increased at the approach of the succeeding shield.

The earth pressure acting on the crown of the tunnel was also measured although the fluctuations due to the approach and passing of the succeeding shield were smaller than those in the lateral direction. Lateral squeezing of the tunnel cavity cross-section was observed at Tunnel F where an increase of the earth pressure on the other side of the succeeding shield was also reported, indicating an increased ground reaction to pressing by the succeeding shield.

In short, the impacts of close work vary depending on the relative locations of the measuring points of the preceding tunnel vis-a-vis the succeeding shield. These impacts also include impacts caused by grouting. It is necessary to establish the earth pressure and ground reaction which reflect the passing of time in order to include such impacts in the design.

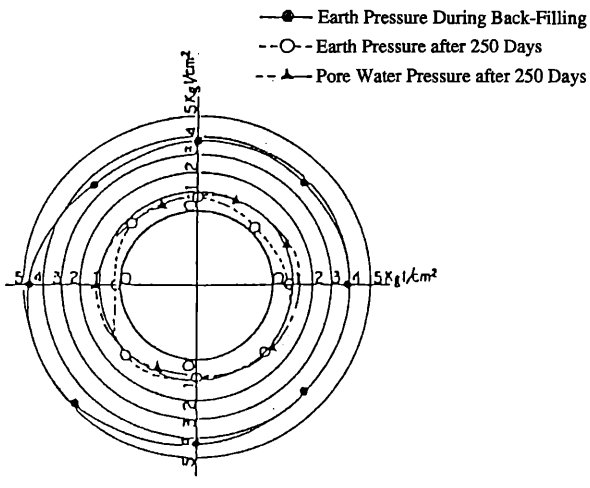

Fig. 11 Cross sectional distribution of total pressure and water pressure (tunnel K) (Nishino, 1988)

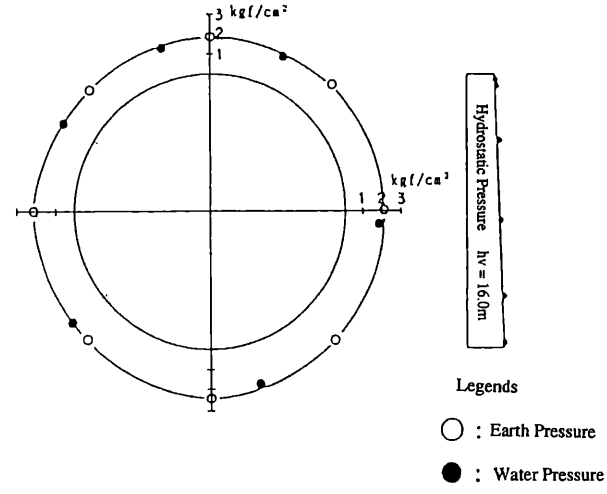

Fig. 12 Cross sectional distribution of total pressure and water pressure (tunnel I) (Ito, 1991)

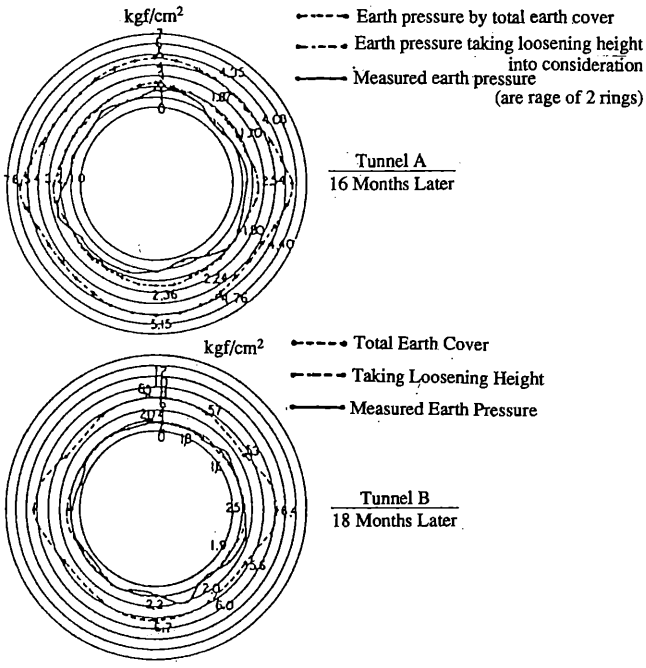

Fig. 13 Cross sectional distribution of earth pressure (tunnel A & B) (Yamaguchi, 1976)

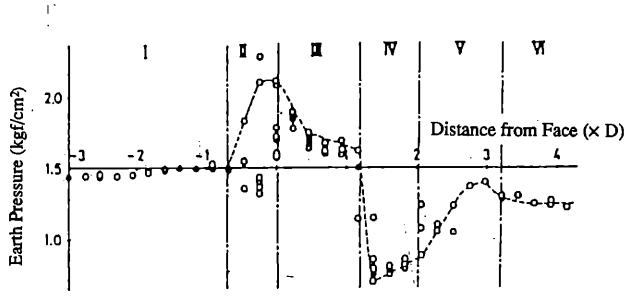

Fig. 14 Changes of earth pressure with passing of time (tunnel N) (Kamemura, 1982)

4. CONCLUDING REMARKS

In addition to being very complicated, the earth pressure phenomena at a tunnel are difficult to measure at a real tunnel, demanding the utmost care in interpreting the obtained data. Furthermore, measurement of the earth pressure is generally costly. The available measurement data are quite limited and are not sufficient to fully elucidate the nature of earth pressure phenomena at a tunnel. A highly rational method to design shield tunnel lining should eventually be established provided that relevant measurement data is continuously accumulated in the future.

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