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Study on earth pressure acting upon shield tunnel lining in clayey and sandy grounds based on field monitoring

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ABSTRACT: In this study, firstly, a series of field monitoring data on earth pressure in soft clay, hard clay and sand ground are analyzed respectively. In the soft clay ground, the earth pressure fluctuated with backfill grouting pressure at first, it settled down toward a steady value between (static vertical pressure Pv0 ± cohesion C) and distributed uniformly over the ring finally. In the hard clay ground, the earth pressure was more greatly influenced by the backfill grouting, especially for the lateral earth pressure in hard clay with $2C/p_{v0} \approx 0.5$. In the sand ground, although the earth pressure was also influenced by the backfill grouting, the distribution was relatively uniform than in hard clay ground since the hydraulic pressure accounts for a large portion in earth pressure. The insights obtained from this study can contribute to an improvement of load considerations in shield lining design.

1 INTRODUCTION

In the design code of shield tunnel in Japan (JSCE, 1996), the earth pressure acting upon the segment lining is calculated by the overburden pressure or Terzaghi's loosening earth pressure according to the stratum condition and the overburden height only. However, it is known that the earth pressure is also influenced by the construction conditions (e.g. backfill grouting, position adjusting of shield machine), and the interaction between the ground and man-made structures (e.g. tunnel lining, pile foundation). In most cases, these factors work together and undistinguishable. Therefore, nowadays the mechanical behavior of the earth pressure upon shield tunnel lining has not been clearly clarified yet.

Some researches have been done on this problem in the last decades. After the Terzaghi's theory on loosening earth pressure, Murayama (1968) studied the vertical earth pressure in sandy layers by trapdoor tests. According to the test finding that the sliding surface is similar to logarithm spirals he proposed a formula to calculate the vertical earth pressure. However, the up-to-data shield technology equipped with precise pressure control system at cutter face and simultaneous backfill grouting system makes it possible to build a tunnel without loosening the surrounding ground. Therefore, the actual earth pressure cannot be correctly predicted by conventional methods (Ohta et al., 1997, Hashimoto et al., 1997). Moreover, Suzuki et al. (1996) reported that the maximum loads occurred during backfill grouting in a shield tunnel with large overburden.

In this study, firstly, three cases of field monitoring on earth pressure in soft clay, hard clay and sand grounds are analyzed carefully. And focus is set on the long-term behavior. The results – distributions of earth pressure, axial force and bending moment of lining – are compared with the calculated value by conventional design method respectively. And more than twenty measurement data are summarized and organized according to the strength of the ground to find out some empirical rules of earth pressure. The insights obtained from this study can contribute to an improvement of load consideration in shield lining design.

2 BRIEF DESCRIPTION OF MONITORING SITES

In order to clarify the characteristics of earth pressures acting upon the linings in clayey and sandy grounds, three typical monitoring jobs are chosen from three

Table 1. Descriptions of shields and geology conditions.

Site name	Kadoma (Soft clay)	Osakajo-A (Hard clay)	Osakajo-B (Sand)
Shield type	Mud-soil pressure balanced	Earth pressure balanced	Earth pressure balanced
Segment type	Ductile	RC	RC
Shield diameter	ϕ5300 mm	ϕ5300 mm	ϕ5300 mm
Backfill grouting type	Simultaneous	Simultaneous	Simultaneous
Overburden height	14.09 m	28.2 m	16.8 m
Around soil type	Alluvial clay	Diluvium clay	Sand
SPT-N value	3 ~ 5	8 ~ 9	>50
Unconfined compressive strength	170 ~ 200 kPa	540 kPa	–

types of ground respectively, namely, soft clay, hard clay and sand grounds. Some basic information on the shields and geological conditions are listed in Table 1, and the soil profiles of monitoring sites are shown in Figure 1. Moreover, two comparing monitoring sections were setup at Osakajo-A and Osakjo-B sites to check the influence of backfill grouting. In order to obtain a reliable earth pressure data, a pad type earth pressure cell (Hashimoto et al., 1993), as shown in Figure 2, was adopted in all monitoring jobs. The water pressures were recorded by piezometers from the grouting holes.

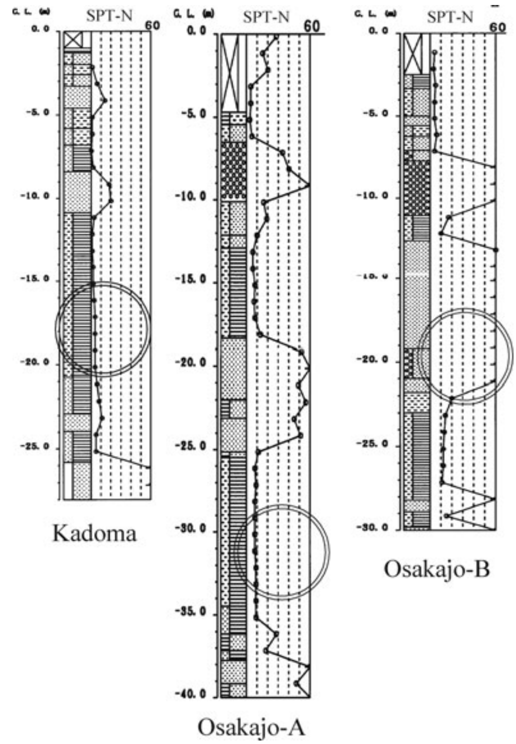

Figure 1. Soil profiles at monitoring sites.

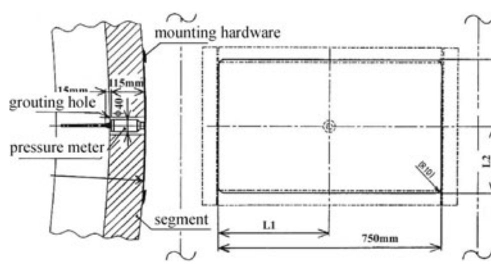

Figure 2. Pad type earth pressure cell.

3 FIELD MONITORINGS OF EARTH PRESSURES ACTING UPON LINING

The time histories of observed earth pressures around the lining in 3 types of ground are shown in Figure 3 ~ 5. The left part of the figures represents the short-term data, and the right part is the long-term one. The earth pressure and water pressure are noted as EP and WP hereinafter. Without mentions, the earth pressure means total earth pressure.

3.1 *In soft clay ground*

Figure 3 shows the changes of observed EPs and WPs at crown, left spring-line, right spring-line and invert of the lining in soft clay ground. After the tail passing, the EPs fluctuated greatly by backfill grouting pressure, and the fluctuation almost disappeared at 7th rings after tail passing, which indicated the extent of influence from grouting holes is about 7 rings in such a ground condition. After that, the EPs and WPs decreased gradually in the first 1 ~ 2 months, and then the WPs remained unchanged while the EPs turned to increased little by little, finally reached a constant status (except R-spring). These final constant values lie between (static pressure $P_0 \pm$ cohesion C). Above phenomena can be explained as following. The excess porewater pressure adjacent to lining, which was generated during shield advancing and backfill grouting, dissipated in the first 1~2 months resulting dropdowns of EPs and WPs, and then the excess porewater pressure in the farer surrounding soil dissipated along with the decoration of soil skeleton, resulted in a very slow buildup of effective EPs upon lining.

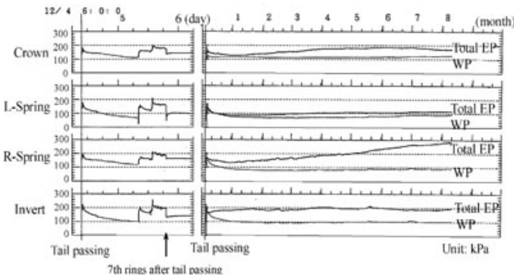

Figure 3. Observed earth pressure acting upon lining in soft clay ground (Kadoma Shield).

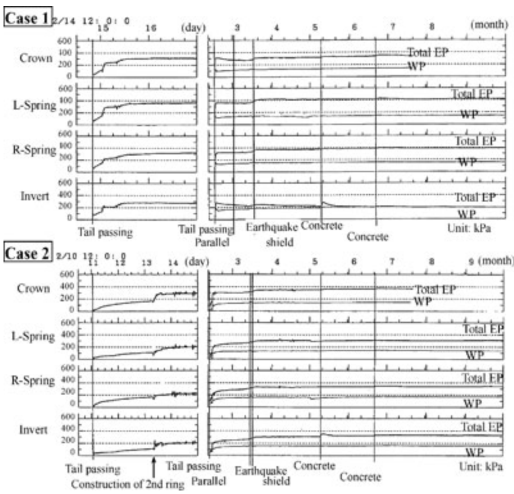

Figure 4. Observed earth pressure acting upon lining in hard clay ground (Osakajo-A Shield).

3.2 *In hard clay ground*

Figure 4 shows the changes of observed EPs and WPs at two compared cases in hard clay ground. In Case 1, a regular backfill grouting was implemented. In Case 2, in order to eliminate the influences (the squeezing force to the ground) of backfill grouting, a special grouting with smaller injection pressure and injection ratio was used, as shown in Table 2. Since the long-term changes in both cases were influenced by the parallel shield passing, earthquake and concrete casting of invert and arcade, and showed increasing trends, the attention was paid to the changes before the 2nd shield passing. In Case 1, after all the EPs primarily climbing up to 300 ~ 400 kPa under the influence of backfill grouting, the EPs at crown and invert turned to decrease. On the other hand, the EPs in Case 2 was smaller (about 200 kPa) at first and then turned to be increased by backfill grouting of next several rings. Comparisons between two cases show that the final differences were very small. The final EPs at the crown in both cases were 370 ~ 380 kPa. The figure shows that the main part of each EP at different location was determined at the first 10 rings, which indicates that the EP largely depends on the backfill grouting in the hard clay ground. And it also can be seen that the settling down of EP only needed several days, much quicker than in soft clay.

Table 2. Backfill grouting of Osaka-A shield

	Case 1	Case 2
Injection pressure	150 kPa	50 kPa
Injection ratio	139%	100%
Grouting material	Standard strength*	Low strength

* Two components grouting material with a gel time less than 10 seconds.

Table 3. Backfill grouting of Osaka-B shield

	Case 1	Case 2
Injection pressure	300 kPa	170 kPa
Injection ratio	135%	100%
Grouting material	Standard strength	Low strength

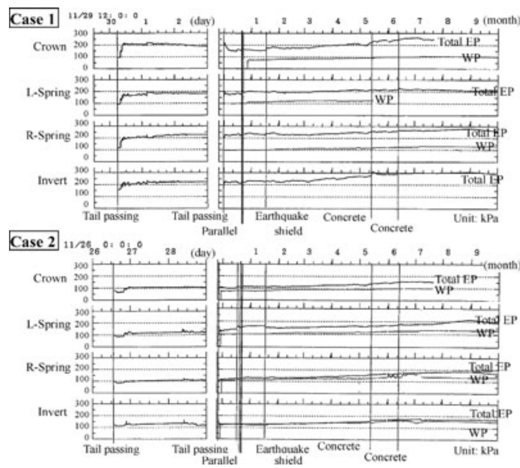

Figure 5. Observed earth pressure acting upon lining in sandy ground (Osakajo-B Shield).

3.3 *In sand ground*

The same as that in Osaka-A shield, here also two cases of comparing measurements were carried out, as shown in Table 3. Figure 5 shows the changes

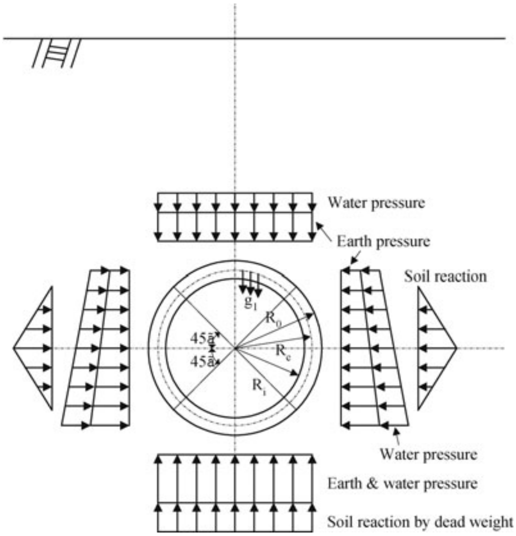

Figure 6. Conventional design method in Japan (effective stress method).

of observed EPs and WPs in both cases. In Case 1, right after the backfill grouting of current ring the EPs was 100 ∼ 150 kPa, then rose up to 200 kPa after the grouting of next 2 ∼ 3 rings, and further increased to 200 ∼ 250 kPa 4 months later. On the other hand, in Case 2, right after the backfill grouting of current ring the EPs were only 80 ∼ 110 kPa, and did not increased significantly after next several rings. The final values were 120 ∼ 160 kPa even 4 months later. The figure also shows that although the long-term EP in both cases influenced by the parallel shield passing and earthquake, and therefore showed an increasing trend, however, the magnitudes did not change a lot from those after the first several rings' backfill grouting. And comparing to the EPs in clayey grounds, the settling down of EPs in the sandy ground was much faster.

4 COMPARISON BETWEEN DESIGN AND OBSERVATION (HASHIMOTO ET AL. 2002)

In Japan, the conventional model and the bedded frame model are main design method for shield lining. In most common situation, such as the original designs of the three tunnels in previous session, the engineer prefers the conventional model for the sake of convenience. We also used conventional model for comparison in this study. Figure 6 is a conceptual figure of conventional model, which considers the EP and WP as well as soil reaction as lining loads.

The comparisons between the observed EP and member forces as well as the calculated values are

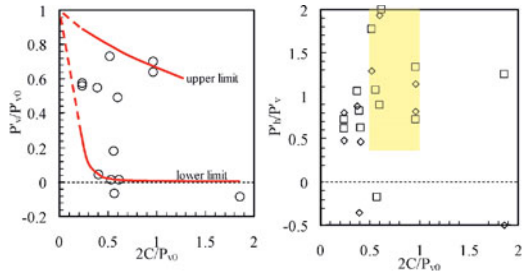

Figure 7. p'_v/p'_{v0} vs. $2C/p_{v0}$ and p'_h/p'_v vs. $2C/p_{v0}$ (clayey ground).

shown in APPENDIX. The EPs over the ring in all ground conditions were relative uniform over the ring by comparing with the design values. Consequently the axial force and the bending moment were smaller than the design ones. As mentioned in previous session, the EP was greatly influenced by the backfill ground in relative hard ground, and it is not difficult to image that the backfill grouting can flow into the tail void quickly and cover the whole ring with a uniform pressure. Therefore, the uniform distribution of EPs is directly related with the backfill grouting.

5 RELATION BETWEEN EARTH PRESSURE AND GROUND CONDITION AND ITS INTERPRETATION

Besides aforementioned three measurement data, we also collected more than twenty data (long-term data) from other monitoring jobs in Japan, including the shield tunnels for subway, sewer, power-supply and communication, and summarized them according to the strength of the ground. The authors would like to emphasize that based on these limited data it is still difficult to execute a quantitative analysis. Therefore in this session a qualitative analysis on the behavior of EPs in various ground conditions is given out.

5.1 *In clayey ground*

The relation between p'_v/p'_{v0} and the normalized strength $2C/p_{v0}$ is plotted in Figure 7(a), where p'_v is the effective vertical EP at the crown, p'_{v0} is the effective overburden pressure, p_{v0} is the total overburden pressure, and C is the cohesion. It is well known that in natural clayey ground, there has such an empirical relation as $q_u(=2C) = 0.3 \sim 0.4p_{v0}$. When the ground is unconsolidated with a very small $2C/p_{v0}$ (<0.3), the tail void is easy to collapse immediately after tail passing, and cannot be easily filled by the backfill grouting in time. Therefore, the surrounding soils will yield and most of the overburden weight will act upon the tunnel ($p'_v/p'_{v0} \to 1$). And when the ground is consolidated or a little over-consolidated ($2C/p_{v0} \approx 0.5$), the grouting material can fill the tail void in time before collapse. On the other hand, when the stiffness of the grouting material and surrounding ground are close, their interaction becomes active. Consequently, p'_v/p'_{v0} depends on the grouting pressure and varies largely according to the construction conditions. Furthermore, when the ground is stiff enough ($2C/p_{v0}$ becomes large), the shrinkage of grouting material during hardening will be larger than the deformation of surrounding ground. The backfill grouting has minor effect on the EP, resulting in a small p'_v/p'_{v0}. The data in Figure 7(a) indicate those kinds of phenomenon. For a better understanding, two boundary lines are drawn in the figure, and for the areas lacking of data broken lines are drawn.

The relation between p'_h/p'_v and the normalized strength $2C/p_{v0}$ is shown in Figure 7(b), where p'_h is the effective horizontal EP at the spring line, and p'_v is the effective vertical EP at the crown. It is found that when $2C/p_{v0}$ is small (<0.3), $p'_h/p'_v = 0.45 \sim 0.8$. Considering the coefficient of lateral earth pressure at rest (K_0) of clayey ground is about 0.5 and the location of spring line is deeper than the crown, such values of p'_h/p'_v are rational. When $2C/p_{v0} \approx 0.5$, the value of p'_h/p'_v scatters in a wide range, implying that the circumferential distribution of earth pressure in such ground also depends on the backfill grouting and other construction conditions.

5.2 *In sandy ground*

The relation between p'_v/p'_{v0} and the equivalent SPT (Standard Penetration Test) N value – which is regarded as a strength index – is plotted in Figure 8(a). By applying the Terzaghi's loosening earth pressure to an assumptive shield tunnel, we can get a curve of loosening earth pressure against SPT-N value, as the dot line in Figure 8(a). However, due to the influences in construction process, the EP acting on the lining will vary above or below the theoretical line. When the ground condition is relatively good and the loosening earth pressure is small, a proper backfill grouting can reduce the EP dramatically, otherwise a large grouting pressure may remain on the lining. The former phenomenon often occurs in the sand ground mingled with cohesive silt/clay. The data with SPT-N between 40~90 in Figure 8(a) are just some vivid instances. Furthermore, when the ground becomes very dense sand or gravel, similar to the clayey ground with large $2C/p_{v0}$, the shrinkage of grouting material may larger than the deformation of surrounding ground, and the EP consequently becomes very small, such as the date with SPT-N > 100 in the figure.

Although there is no enough observed data in the SPT-N < 40 ground, we can make an estimation from the knowledge of soil mechanics. When the soil is very loose, the dependency on construction shall be weakened (the same as the soft clay ground), and then the

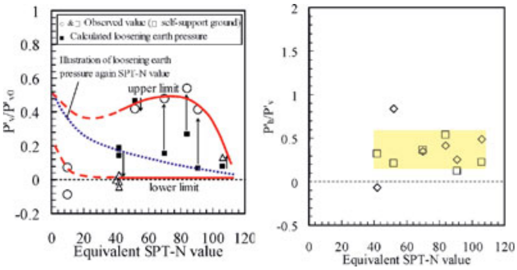

Figure 8. p'_v/p'_{v0} vs. SPT-N and p'_h/p'_v vs. SPT-N (sandy ground).

distribution of EP will get close to the theoretical loosening earth pressure, as the broken line in Figure 8(a). For a better understanding, the authors also draw two boundary lines in the figure.

The relation between p'_h/p'_v and the equivalent SPT-N value is shown in Figure 8(b). It is found that p'_h/p'_v scatters between $0 \sim 0.6$, indicating that the circumferential distribution of earth pressure in sandy ground also depends on the backfill grouting and other construction conditions.

The shapes of the boundary lines in Figure 7(a) and 8(a) have the same characteristics: two ends are narrow while the middle is wide. This kind of shape clearly tells us that when calculating the EP in the ground with medium stiffness, more attention should be paid to the backfill grouting and other construction conditions than those in soft or very hard ground. It should be pointed out that, however, the backfill grouting must be considered when dealing with the settlement in soft clay ground (Hashimoto et al. 1999).

6 CONCLUSIONS

In order to clarify the mechanical behavior of earth pressure acting upon shield tunnel lining in various ground conditions, saying soft clay, hard clay and sand grounds, a series of field monitoring has been carried out. By analyzing the observations carefully, following conclusions were obtained.

1. The earth pressure is influenced by the injection of backfill grouting at $7 \sim 8$ rings away in the case of simultaneous backfill grouting.
2. In the soft clay ground, the earth pressure fluctuates with the backfill grouting in the early phase. However, it settles down to a steady value between (static pressure $P_{v0} \pm$ cohesion C) finally, regardless of the backfill grouting pressure.
3. In the hard clay and sand ground, the initial earth pressure that builds up gradually by backfill grouting remains in the long-term earth pressure. In other words, the earth pressure depends on the backfill grouting to such an extent that sometimes the

earth pressure will be larger than the prediction by Terzaghi's loosening earth pressure.

4. The settling down of earth pressure in soft clay ground needs several months, while those in hard clay and sand grounds only need several days or even several hours.
5. The distributions of earth pressure are more uniform than predictions by conventional design method in all ground conditions. In other word, the bending moment is apt to be overestimated, especially in the sandy ground where hydrostatic pressure plays a dominant role in earth pressure. Therefore, it is suggested that when designing the lining, the influence of backfill grouting should be taken into consideration.
6. By analyzing more than twenty monitoring data, it is found that in the clayey ground, if $2C/q_u < 0.3$, a large portion of the overburden will act upon the lining. If $2C/p_{v0} \approx 0.5$, the magnitude and distribution of earth pressure depend largely on the backfill grouting. And in the sandy ground, if the equivalent SPT-N value lies between $40 \sim 80$, the magnitude and distribution of earth pressure also depend largely on the backfill grouting.

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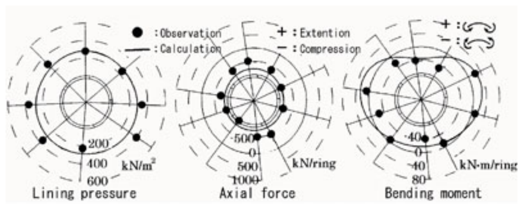
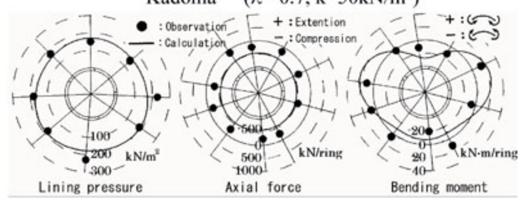
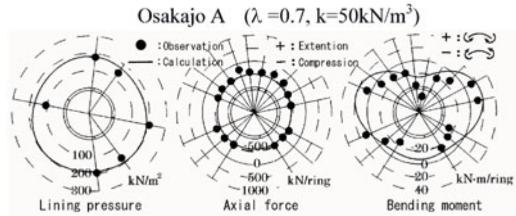
APPENDIX



Kadoma (λ =0.7, k=50kN/m^3)



Osakajo A (λ =0.7, k=50kN/m^3)



Osakajo B (λ =0.6, k=50kN/m^3)

Figure A1. Comparison of observed and designed values of earth pressure and member forces (Hashimoto et al, 2002).