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OBSERVATIONS OF GROUND MOVEMENTS DURING TUNNEL CONSTRUCTION BY SLURRY SHIELD METHOD AT THE DOCKLANDS LIGHT RAILWAY LEWISHAM EXTENSION—EAST LONDON

TADASHI SUGIYAMA[i], TOSHIYUKI HAGIWARA[i], TOSHI NOMOTO[i], MASAAKI NOMOTO[i], YUTAKA ANO[i], R. J. MAIR[ii], M. D. BOLTON[ii] and KENICHI SOGA[ii]

ABSTRACT

This paper describes monitoring results of ground movements due to slurry shield tunnelling on the Docklands Light Railway Lewisham Extension (hereinafter referred to as DLR). The DLR tunnels (twin bored tunnels) run from Island Gardens Station to Greenwich Station in East London, passing under the River Thames, a distance of around 4.2 km. The construction of the tunnels, which were at shallow depth, needed an extensive appraisal of the potential damage induced by ground movements to the overlying and underground structures. Therefore, in order to effectively minimize the ground movements, a slurry shield machine with a diameter of 5.85 m was employed. Careful control of excavation was carried out during shield tunnel construction. As a result, the slurry shield tunnelling method in Woolwich and Reading Beds by well controlled monitoring was successfully conducted, leading to volume losses less than 1.0%.

This paper focuses on the ground surface movements, particularly the transverse and longitudinal settlement profiles, and volume losses during shield tunnel construction, in relation to the ground conditions. Based on the monitoring results, combined with previous centrifuge model test results and other field monitoring data, practical methods to appropriately predict ground movements due to shield tunnelling are proposed.

Key words: construction process, in-situ monitoring, settlement, slurry shield method, tunnel, volume loss (**IGC:** C7/H5)

INTRODUCTION

In order to improve access from South East London to the Docklands, City Greenwich Lewisham Rail Link Plc (CGL) has commissioned the extension project to the Docklands Light Railway which would run from Mudchute Station on the Isle of Dogs to Lewisham Station, passing under the River Thames. Figure 1 shows the route of the tunnels running from Island Gardens Station on the Isle of Dogs to Greenwich Station. Figure 2 shows a plan view and longitudinal section through the tunnels, together with the ground conditions. Since the tunnels were planned to drive through mixed face conditions, including Terrace Gravel, the clays, sands and gravels of the Woolwich and Reading Beds (WRB), and Thanet Sand, the difficulties in excavating tunnels and controlling the ground movements during tunnel construction had been expected. (The Woolwich and Reading Beds (WRB) are now known as the Lambeth Group but will nevertheless be referred to as WRB in this paper.)

In recent years, the slurry shield tunnelling method has been increasingly used, particularly in Japan. This method, as well as the earth pressure balance shield technique, has been significantly improved, in combination with new computer controlled shield operations (Kurihara, 1998). Therefore, in this project, the slurry shield method with 5.85 m diameter twin tunnels was employed because of the following reasons:

(1) This method is applicable to mixed face conditions and high water pressures. Recently, in Japan, the Tokyo Aqualine Tunnel (56 m in depth and 14.14 m in diameter) which crosses Tokyo bay was successfully conducted by the slurry shield method (Kurihara, 1998). Based on Japanese experience, a preliminary check of the applicability of the slurry shield method to Terrace Gravel, WRB and Thanet Sand had been conducted and confirmed.
(2) It is possible to easily control the tunnel excavation in response to real time measurements.
(3) The slurry shield method was considered to be more economic than the earth pressure balance shield method.

i) Nishimatsu Construction Co., Ltd., Tokyo.
ii) University of Cambridge, Cambridge, U.K.
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During tunnel construction, in addition to sophisticated mechanized slurry shield techniques, including the excavation control and face pressure control, careful and extensive monitoring of ground movements was carried out. The main purpose of the monitoring was that it was possible to promptly detect any unusual conditions and to react accordingly. The monitoring results were rapidly provided to the tunnel machine operators. As a result, the tunnelling work was successfully conducted, leading to volume losses less than 1.0%.

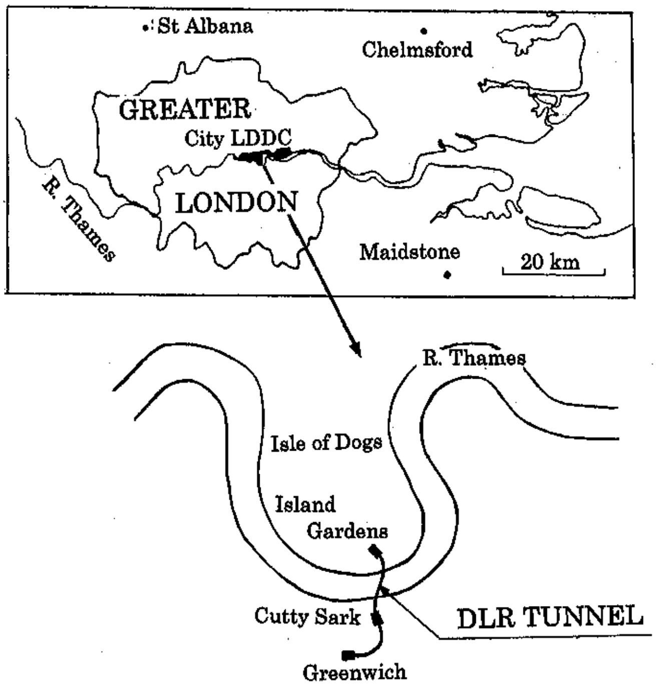

Fig. 1. Route of DLR-LWE TUNNEL

This paper describes results of monitoring of the ground movements, putting particular emphasis on the effects of the shield construction processes (control of excavation, face pressure, grouting etc.) and on the ground conditions. Based on the extensive monitoring results, combined with previous centrifuge model test results obtained by the authors and other field monitoring data, practical methods to appropriately predict ground surface movements due to shield tunnelling are proposed.

SHIELD TUNNEL CONSTRUCTION

Ground Conditions

Ground conditions and the basic soil properties of the ground are summarized in Fig. 2 and Table 1 respectively. The geotechnical strata from surface level along the route of the bored tunnels comprise varying thicknesses of Made Ground, Terrace Gravel, WRB and Thanet Sand. Terrace Gravel is a fluvial gravel bed and consists mainly of sandy gravels, mostly having a grain size from

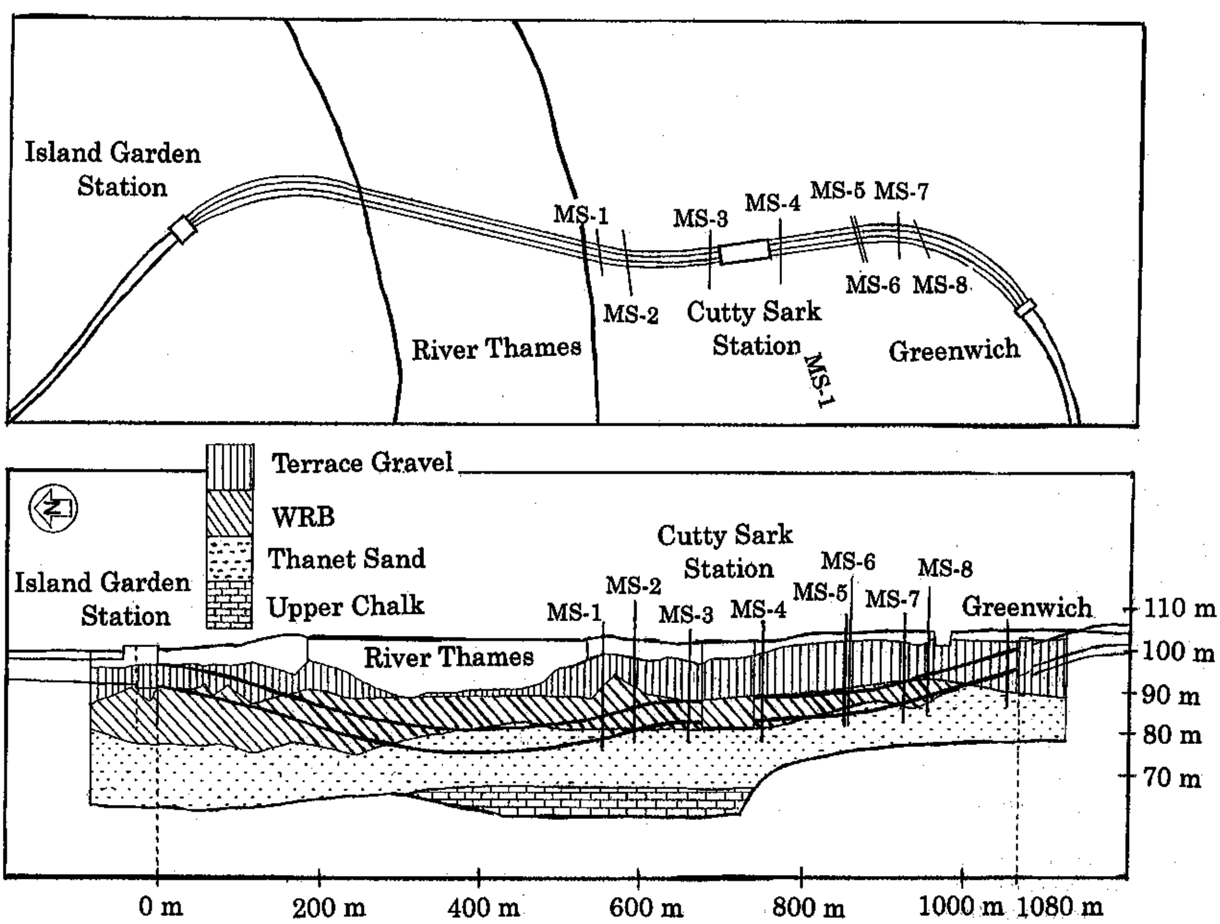

Fig. 2. Schematic geological section along DLR-LWE route

Table 1. Summary of soil properties of the site

Layer		Particle distribution Gravel (%)	Particle distribution Fines (%)	Uniformity coefficient	Permeability k (m/sec)	Water content (%)	Undrained strength s_u (kPa)	N value
Made ground		23.4~47.4	6.6~23.6	17.0~	—	40~80	30~45	3~8
Terrace Gravel	Loose	49.0~94.3	0.0~1.7	2.8~	—	—	—	4~20
	Dense	50.6~80.6	0.0	2.0~	1.0E-02~1.0E-03	—	—	15~90
WRB*	LSS	0.0~1.3	21.7~47.0	—	1.0E-06~1.0E-08	—	—	21~77
	LSC	5.0~23.3	35.7~60.7	—	1.0E-06~1.0E-09	25~90	>200	45~82
	LMC	0.0~0.4	34.3~46.3	2.1~	1.0E-06~1.0E-09	10~30	>250	60~170
	PB	3.0~36.4	20.6~42.0	1.8~	1.0E-06~1.0E-08	10~90	20~140	45~65
Thanet beds		0.0~0.4	1.3~8.3	2.3~	1.0E-06~1.0E-07	20~30	90	16~94

* LSS=Laminated Sits and Sands, LSC=Lower Shelly Clay, LMC=Lower Mottled Clay, PB=Pebble Beds

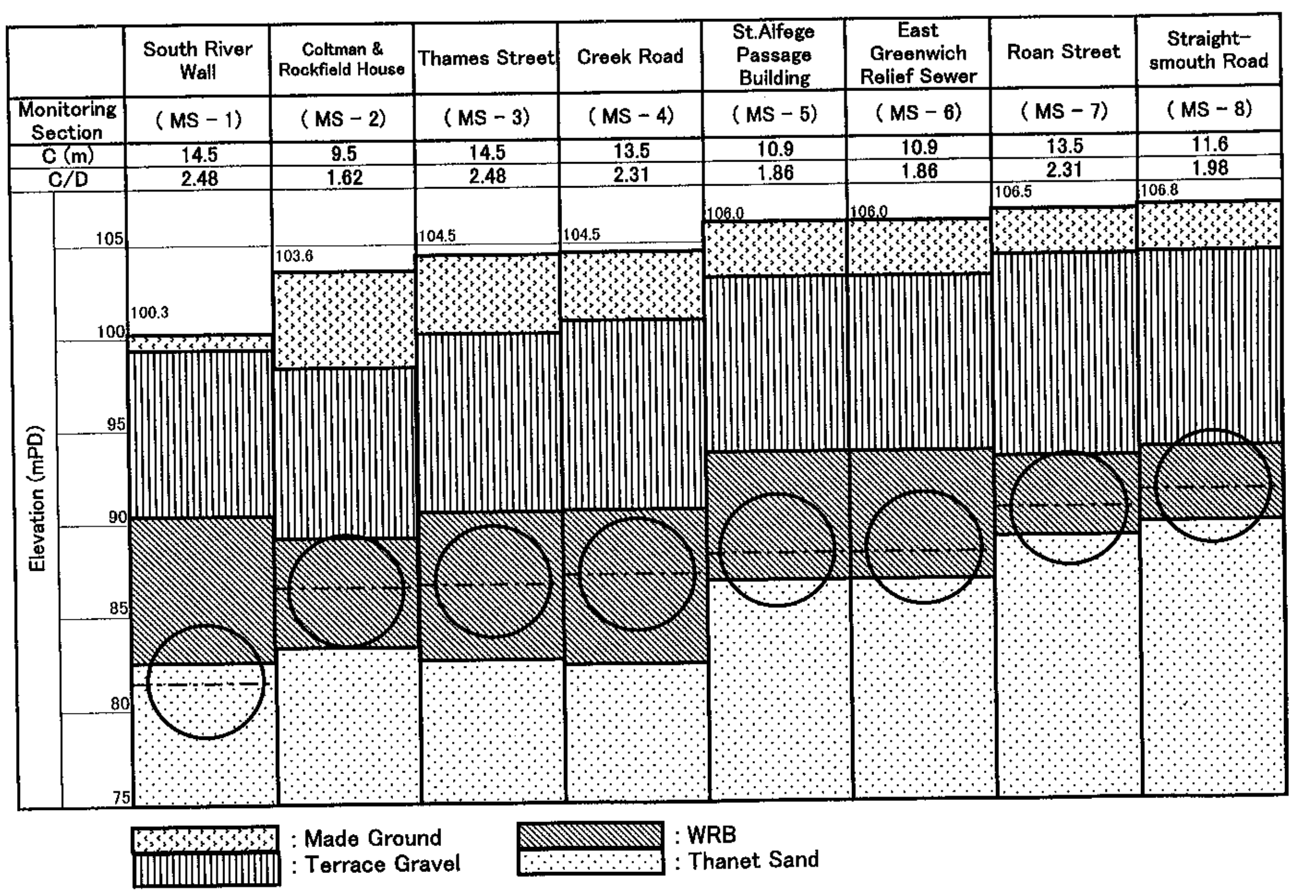

Fig. 3. Ground conditions at monitoring sections

2 mm to 10 mm. The WRB, consisting of various kinds of soils from very stiff to hard clays to dense sands and gravels, is a very complex formation. Stiff to hard clays were the principal strata encountered by the tunnels in the WRB formation. Thanet Sand, consisting of silty fine sand, is a dense sand layer.

Slurry Shield Method

As described above, very mixed face conditions were anticipated. In addition, there were some important overlying and underground structures. Therefore, the slurry shield method was employed to minimize ground movements, taking into account the other advantages as described earlier in the paper. A slurry shield machine with an outer diameter of 5.85 m (D) and a length of 6.82 m (L) was used for excavating two parallel single tunnels of which the concrete segments were 250 mm in thickness and 1200 mm in length. The clear separation of the two tunnels was about 15 m. Of the two tunnels, the southbound route was constructed first, followed by the northbound route. The length of each tunnel was about 1 km and the cover-to-diameter ratio (C/D, C: the depth to tunnel crown) ranged from 1.6 to 2.5. During the tunnel excavation, the tail void was filled immediately by an injection material having a gelling time of about 15 seconds.

Instrumentation

Ground surface movements on a transverse cross-section during shield tunnel construction were carefully measured at eight locations (MS-1 to MS-8) as presented in Fig. 2. The ground strata at the monitoring sections are shown in Fig. 3. The monitoring of the ground movements was carried out using precise levelling survey techniques. On each section ten monitoring points, in a direction perpendicular to tunnel driving and at 5 m intervals, were surveyed.

Based on the extensive monitoring results, transverse and longitudinal settlements trough were obtained and compared with the predicted values. After shield driving, any subsequent settlement is due to long term consolidation; this has been analyzed separately and was found to be generally negligible. Accordingly, only the settlement (short term settlement) which is induced within two weeks after the passing of the shield is presented in the Paper.

DEFINITIONS OF GROUND MOVEMENTS

Transverse Settlement

Construction of a tunnel results in ground movements with a settlement trough developing above and ahead of the tunnel. Analysis of a considerable number of case records (Peck, 1969, O'Reilly and New, 1982; New and O'Reilly, 1991; Fujita, 1989) has demonstrated that the resulting transverse settlement trough immediately after a tunnel has been constructed is well described by a Gaussian distribution curve as:

$$S=S_{max}\exp\left(\frac{-y^2}{2i^2}\right) \tag{1}$$

where S is settlement

S_{max} is the maximum settlement on the tunnel centre line

y is the horizontal distance from the centre line

i is the horizontal distance from the tunnel centre line to the point of inflexion on the settlement trough

The definition is illustrated in Fig. 4. For near surface settlements O'Reilly and New (1982) showed that the dimension i in Fig. 4 was an approximately linear function of the depth z_0 (the depth to tunnel centre line) and broadly independent of tunnel construction method. They assumed that the simple approximate relationship

$$i=Kz_0 \tag{2}$$

can be adopted and the values of the trough width

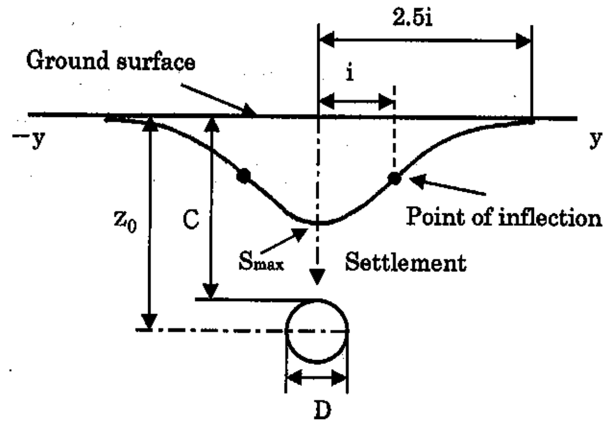

Fig. 4. Definition of settlement profiles of Gaussian form

parameter K for tunnels in clay, and sands or gravels, may be taken as approximately 0.5 and 0.25 respectively. More recently, Mair and Taylor (1997) summarized a wide range of field data observed during tunnelling, including conventional shield tunnelling. They concluded that values of the trough width parameter K for tunnels in clays, and sands or gravels, can be taken as average values of 0.50 and 0.35 respectively, regardless of tunnel size and tunnelling method.

The volume loss, defined as the settlement trough (per meter length of tunnel), Vs, can be evaluated by integrating Eq. (1) to give

$$V_s=\sqrt{2\pi}iS_{max} \tag{3}$$

The volume loss is usually expressed as a percentage fraction, V_L, of the excavated area of the tunnel, i.e. for a circular tunnel,

$$V_s=V_L\frac{\pi D^2}{4} \tag{4}$$

Longitudinal Settlement

Typically, there are five types of ground displacement caused by shield tunnelling, as depicted in Fig. 5. They are as follows:

Step 1: Preceding settlement

Preceding settlement occurs far ahead of the arrival of the shield machine. In the case of tunnelling in sands, for example, this settlement may be due to lowering of the groundwater level.

Step 2: Deformation of ground at the front of the face

Step 2 is settlement occurring immediately before the arrival of the shield. This settlement is due to the imbalance of any tunnel support pressure with the earth or water pressures at the tunnel face. When pressurized face tunnelling methods are used, Step 2 can be very small if the face pressure is carefully controlled.

Step 3: Settlement during passage of the shield

Step 3 can be appreciable, particularly if the over-cutting edge is of significant thickness, and if there are steering problems in maintaining the alignment of the shield.

Step 4: Settlement due to the tail void

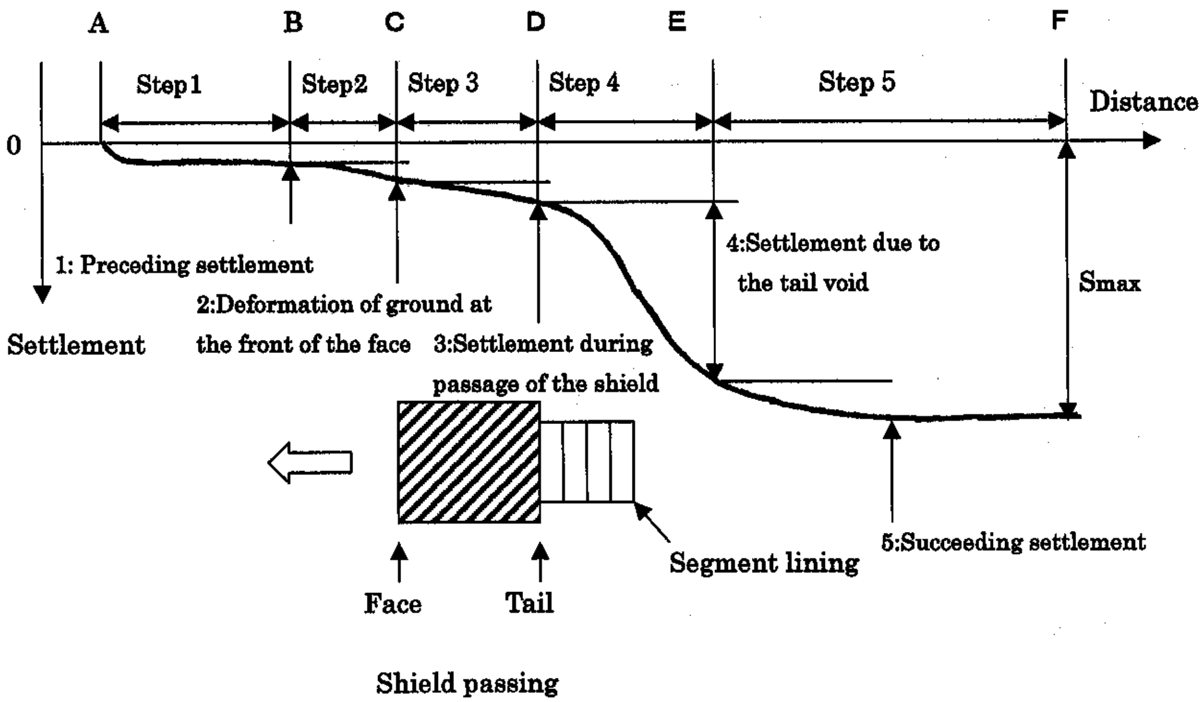

Fig. 5. Illustration of ground displacement caused by shield tunnelling

Step 4 can be minimized by simultaneous or immediate grouting to fill the tail void.

Step 5: Succeeding settlement

Succeeding settlement can be caused by disturbance of the ground due to the shield driving. This can be of importance, particularly when tunnelling in soft clays, where excess pore pressures are generated and those subsequently dissipate to long term equilibrium values. Improvements of the construction method referred to in Steps 2–4 reduce total settlement effectively, as well as decreasing the magnitude of the succeeding settlement.

In this paper, points (A to E) in Fig. 5 correspond to the time of tunnel construction and are defined as follows:

A: 8 days before C (approximately 90 m ahead of C)
B: 1 day before C (approximately 11 m ahead of C)
C: Face
D: Tail
E: 1 day after D (approximately 11 m behind D)
F: 2 weeks after D (approximately 160 m behind D)

The average progress rate of the tunnelling machine was 11 m/day.

Table 2. Monitoring results

		Southbound		Northbound	
Monitoring section	Tunnel depth z_0 (m)	S_{max} (mm)	Volume loss (%)	S_{max} (mm)	Volume loss (%)
MS-1	17.43	5.4	0.85	4.5	—
MS-2	12.43	—	—	4.1	0.70
MS-3	17.43	5.4	0.97	8.4	0.86
MS-4	16.43	6.7	—	—	—
MS-5	13.83	5.5	0.45	8.3	0.64
MS-6	13.83	5.5	0.46	7.7	0.91
MS-7	16.43	6.4	0.54	12.0	0.79
MS-8	14.53	10.6	0.78	—	—

MONITORING RESULTS AND DISCUSSION

Maximum Surface Ground Movements

The maximum ground surface settlements (S_{max}) obtained from each monitoring section are summarized in Table 2. Figure 6 shows the relationship between S_{max} and C/D, together with some previous observations for slurry shields in Japan. It can be seen that all monitoring data of S_{max} are less than 15 mm and the average value is about 7 mm, which shows an excellent achievement in excavation control. Since S_{max} is influenced by various factors such as the shield method, the ground conditions, tunnel depth and size, it is difficult to simply compare these results to other data. However, the values of S_{max} reported in this paper are much smaller than reported in the past (e.g. 20 mm by Hanya, 1977, 40 mm by Fujita, 1989, 20 mm by Nomoto et al., 1995). Nevertheless, according to the recent report by Hashimoto et al. (1996), the measured maximum surface settlement above tunnels in diluvial clay was less than 10 mm for 90 percent of case histories included in a Japanese survey of slurry shield construction. The present monitoring results are reasonably consistent with the findings of Hashimoto et al. In more recent case histories, smaller S_{max} is generally reported. This is because there have been considerable advances in techniques of shield tunnel construction, including good control of tunnel excavation.

The general relationship of settlement with C/D is not clear, as can be seen from Fig. 6, because C/D is in the limited range from 1.6 to 2.5.

Combining Eqs. (2)–(4) gives

$$S_{max}=\frac{0.313\,V_L D^2}{K z_0} \tag{5}$$

From Eq. (5), it can be seen that for a given volume loss and a tunnel diameter, and for a constant K, the maximum surface settlement above a tunnel is inversely proportional to the depth of tunnel (Mair et al., 1993). Figure 7 shows the relationship between S_{max}/D and z_0/D for the measurements described in this paper, together with past shield data in clays (Mair and Taylor, 1997). Although each construction method relating to the past data is different, S_{max} is generally seen to decrease with an increase of z_0, as shown in Eq. (5). On the other hand, the trend of monitoring data for the project described in this paper is unclear because of the narrow range of C/D.

Transverse Settlement

Figure 8 shows the change of the surface transverse settlement troughs associated with shield construction at the monitoring section of MS-5 (St Alfege Passage, Southbound). Each point from A to F is consistent with

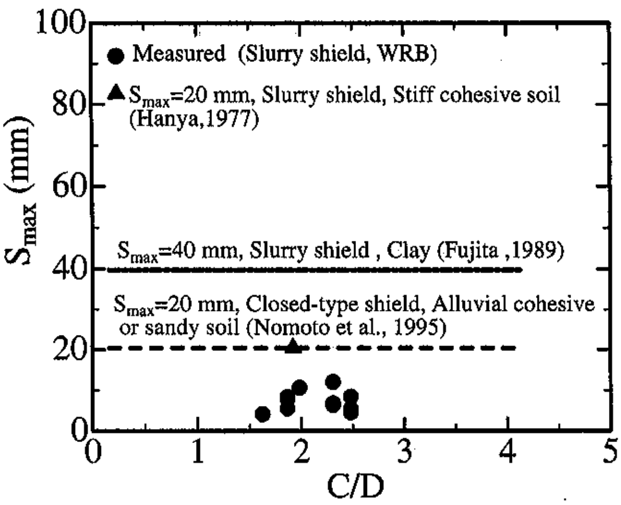

Fig. 6. Relationship between S_{max} and C/D, compared with data reported in the past for closed face tunnelling shields in Japan

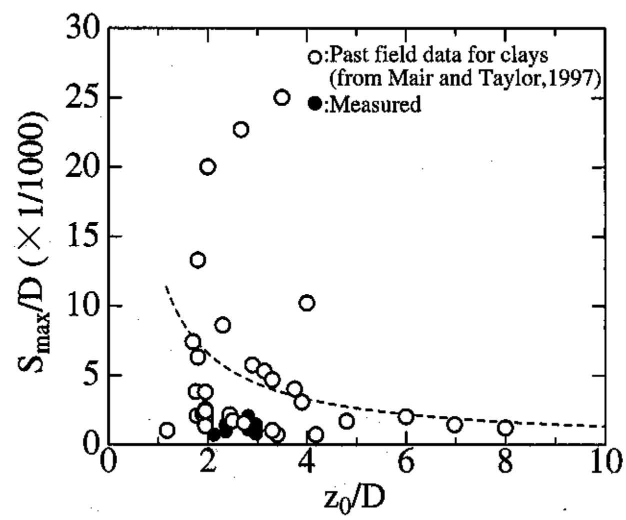

Fig. 7. Relationship between S_{max}/D and z_0/D, compared with data from other projects

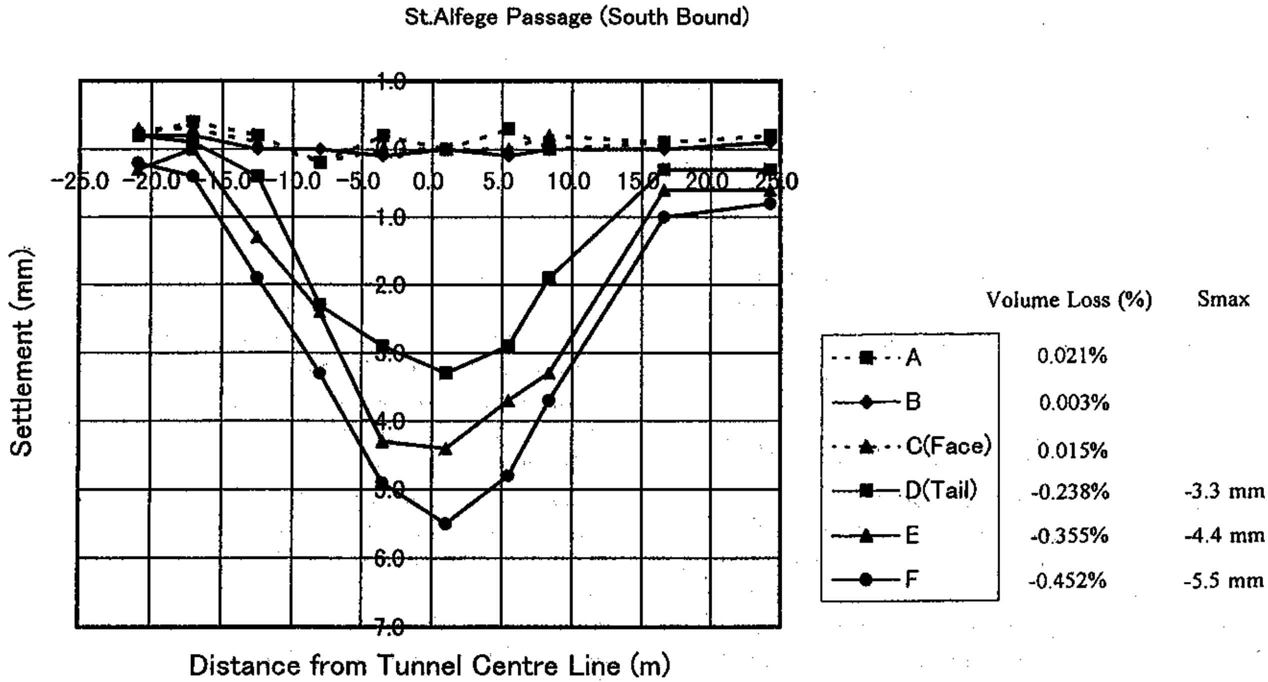

Fig. 8. Transverse settlement troughs (S vs. y)

those used in Fig. 5. Since surface settlements ahead of C (tunnel face) were very small, transverse settlement troughs began to be clearly discernible from point D. After that, as the shield tunnel proceeds, the settlement of every point on the transverse section increases, indicating an increasing amount of volume loss.

Figure 9(a) shows normalized transverse settlement troughs, compared with a predicted trough. The predicted trough is based on Eq. (1), assuming $i=Kz_0$ where $K=0.41$ and $z_0=13.8$ m. The value of $K=0.41$ had been derived before tunnel construction, by using an equation given by Selby (1988). This equation for tunnels in clays and sands was derived by Selby (1988) by taking account of the thickness of the different strata, based on O'Reilly and New (1982), so that for a two-layered system

$$i=0.43z_2+1.1m+0.28z_1 \tag{6}$$

(for a tunnel in clay overlain by sand)

as shown in Fig. 10(a). In the case of MS-5, since the layer of Made Ground and Terrace Gravel is 9.1 m and the

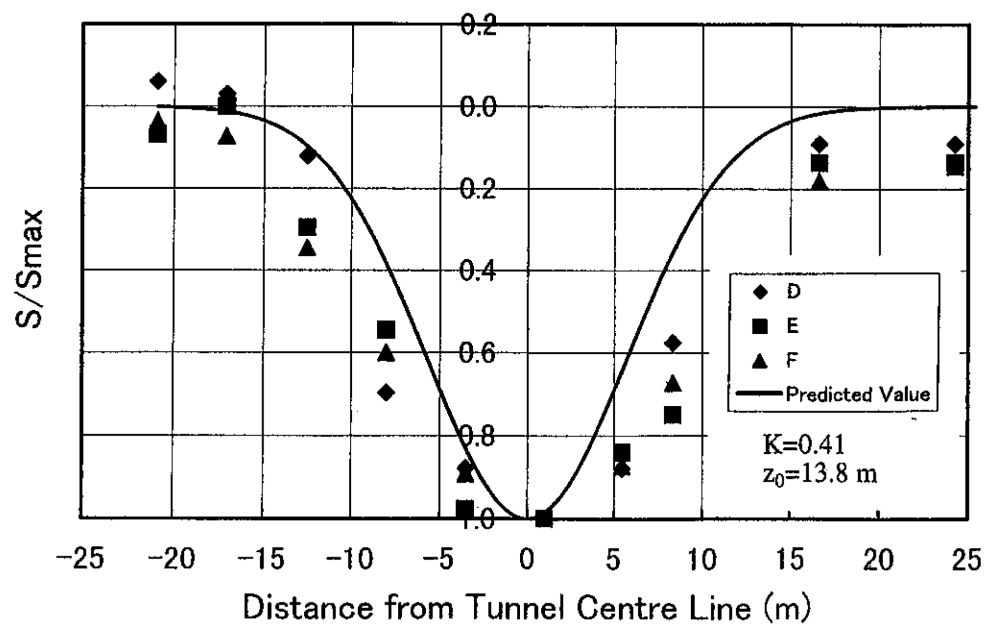

Fig. 9(a). Transverse settlement troughs (S/S_{max} vs. y)

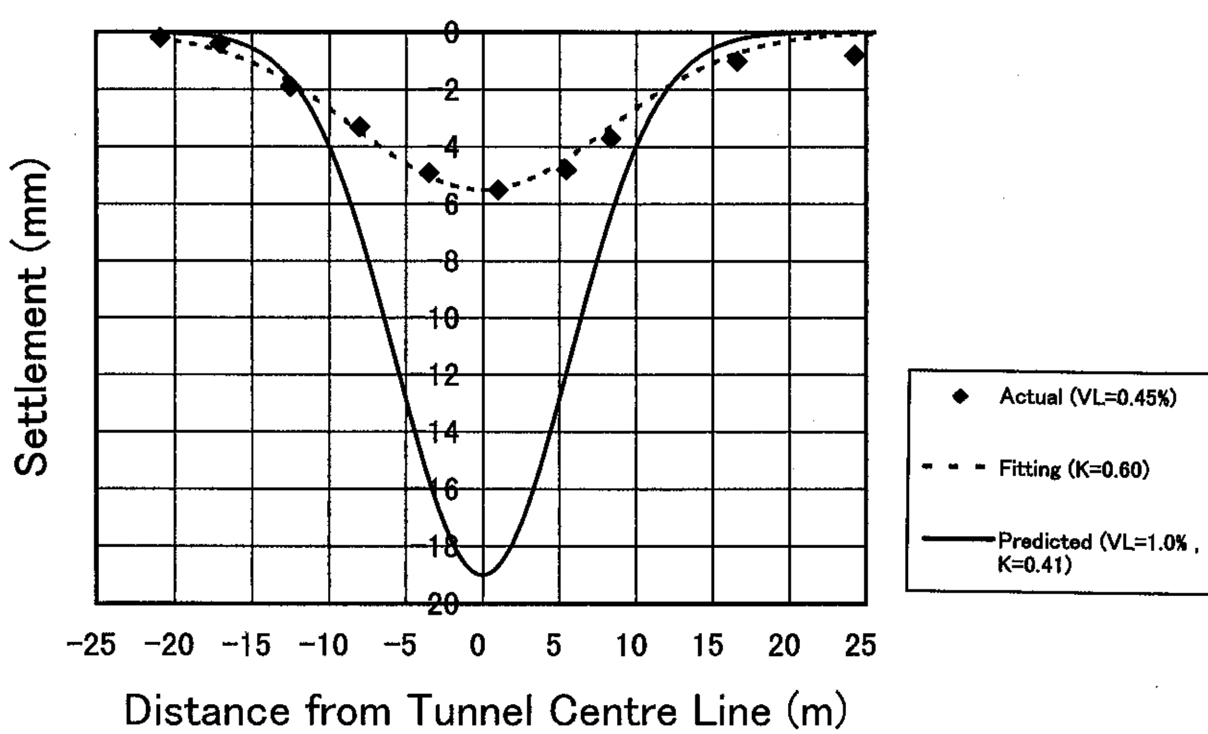

Fig. 9(b). Transverse settlement troughs (S vs. y) at final stage

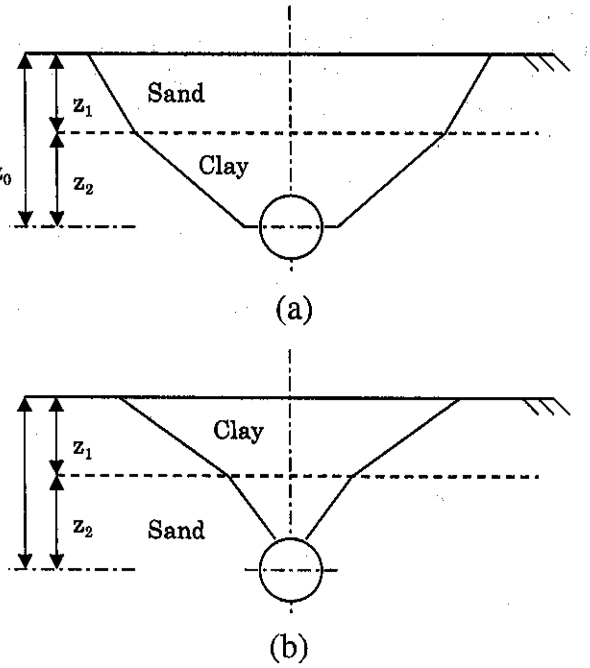

Fig. 10. Proposed behaviour of two-layer soils (Selby, 1988)

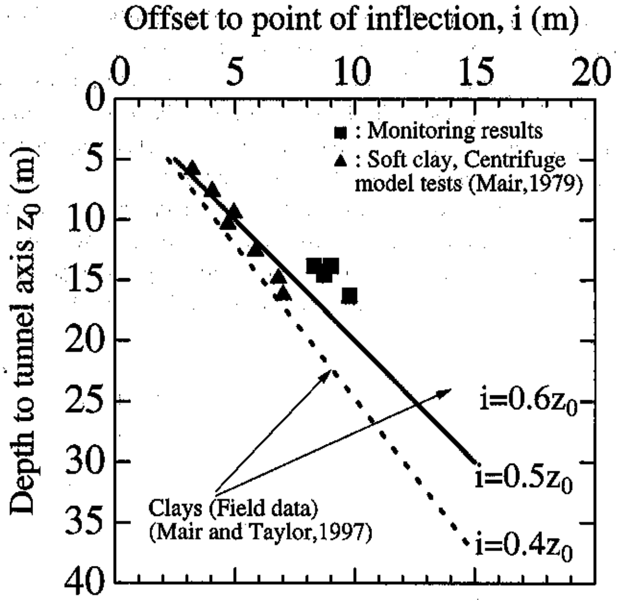

Fig. 11(a). Variation in surface settlement trough width parameter with tunnel depth for tunnels in clays

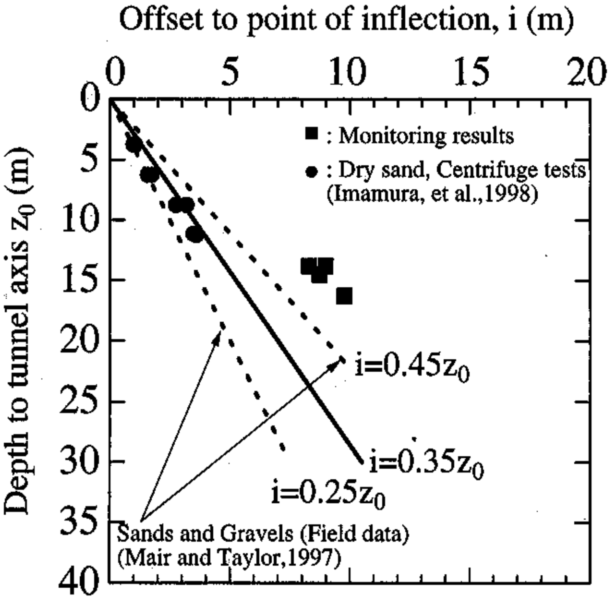

Fig. 11(b). Variation in surface settlement trough width parameter with tunnel depth for tunnels in sands and gravels

layer of WRB (predominantly clay) is 4.6 m, the values of z_1 and z_2 are 9.2 m and 4.6 m respectively. By substituting these values into Eq. (6), $i=5.65$ m was obtained. By substituting this i value and $z_0=13.8$ m into Eq. (2), the trough width parameter of $K=0.41$ was obtained as the predicted value.

It is sometimes found that the transverse settlement troughs based on measured results are wider than the predicted one, regardless of the construction process. Centrifuge model studies by Grant and Taylor (1996) indicated that in the case of a tunnel in soft clay overlain by sand, the surface settlement profile is wider than that determined by assuming constant values of K of 0.5 for the clay stratum and K of 0.3 for the sand, which are suggested by current design practice. Also, recent field observations of surface settlement profiles above stratified soils where the tunnel is in sand overlain by clay layers, as shown in Fig. 10(b), indicated wider profies than would be obtained if the tunnels were only in sands (e.g. Ata, 1996).

Figure 9(b) shows a direct comparison between the settlement trough at point F of MS-5 and the predicted one. The magnitude of volume loss V_L, defined in Eqs. (3) and (4), was assumed to be 1.0 percent as the predicted design value based on recent experiences with slurry shield machines in clays. The best fit Gaussian distribution curve estimated from the monitoring results is also shown in Fig. 9(b), and this corresponds to the value of $K=0.6$. The transverse settlement trough based on the monitoring results is reasonably represented by the Gaussian distribution curve. The predicted and observed troughs show a large difference, mainly because the observed volume loss (and hence S_{max} value) is much smaller than that assumed for the predicted trough. Also, the i value based on monitoring results is about 50% larger than the predicted value.

Figure 11 shows relationships between z_0 (depth to the tunnel axial) and i based on the plots of field measurements given by Mair and Taylor (1997). Figures 11(a) and (b) show the results for tunnels in predominantly clays and the results for tunnels in predominantly sands and gravels respectively. In both Figures, the solid line shows a mean line obtained from past field data (including shield tunnels) around the world and the dotted lines show the upper and lower bounds. Centrifuge results in soft clay by Mair (1979) and centrifuge results in dry sand by Imamura et al. (1998) are also plotted in Figs. 11(a) and (b) respectively; both sets of data are plotted at the equivalent prototype scale modelled by the centrifuge tests. In addition, the monitoring results from the DLR tunnels described in this paper are plotted for comparison. Data obtained from 4 monitoring sections are plotted on both Figures 11(a) and (b). At some monitoring sections, large enough settlement troughs to derive i values were not obtained because only very small settlements were measured.

Figures 11(a) and (b) show that both sets of centrifuge results are consistent with the results from past field data in spite of different conditions, including tunnel size and construction method. This is consistent with the conclusion of Mair and Taylor (1997) that the width of the surface settlement trough, based on field measurements, is independent of construction method. The monitoring results from the DLR tunnels nearly fall on the line of $i=0.6z_0$ (upper bound line for clays) in Fig. 11(a). Some of the settlement data for the monitoring sections were obtained from measurements on buildings rather than from settlements of the ground surface. The inherent stiffness of the buildings relative to the ground could explain the apparently wider settlement trough than usually obtained for a 'greenfield site', as described by Mair and Taylor (1997).

Generally, the settlement behaviour recorded at the monitoring sections is typical for tunnels in predominantly clays, even though the ground profiles showed mixed face conditions, including Terrace Gravel above the WRB. This is because the shield was mostly driven through the clays of the WRB, as shown in Fig. 3. Accordingly, for design calculations, superposition of movements from one layer to another, as proposed by Selby (1988) in Fig. 10(a), is not appropriate since smaller i values are predicted compared with those observed. In reality, i values are expected to be strongly dependent both on the ground depth and on the type of soil strata through which the tunnels are driven, as clearly shown by the difference between Figs. 11(a) and (b).

In recent years, tunnels at very shallow depth, i.e. $C/D<2.0$, have been increasingly constructed because of severe limitations of space in urban areas. In addition, the tunnel size has in many cases become larger, up to around 15 m. In such cases, the ratio C/D, which is a parameter commonly referred to by Japanese tunnel engineers, may be another factor influencing the point of inflection i (as proposed by Clough and Schmidt, 1981). Figure 12 shows relationships between $i/(D/2)$ and C/D plotted in a similar way to Fig. 11, although the data is now plotted on a log-log scale. The following relationships have been derived from Fig. 12:

(a) Clays (Past field data, Mair and Taylor, 1997, Centrifuge data by Mair, 1979 and Monitoring results)

$$\left(\frac{i}{D/2}\right)=1.5\left(\frac{C}{D}\right)^{0.8} \qquad \text{(7a)}$$

$$|r|=0.94$$

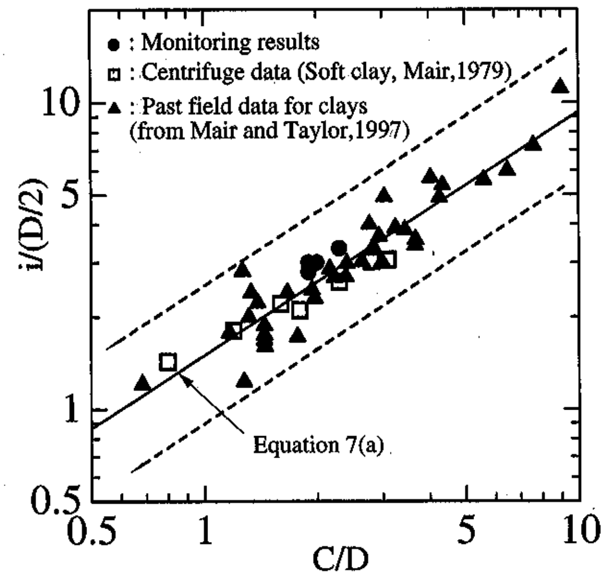

Fig. 12(a). Variation in trough width parameter with cover-to-diameter ratio for tunnels in clays

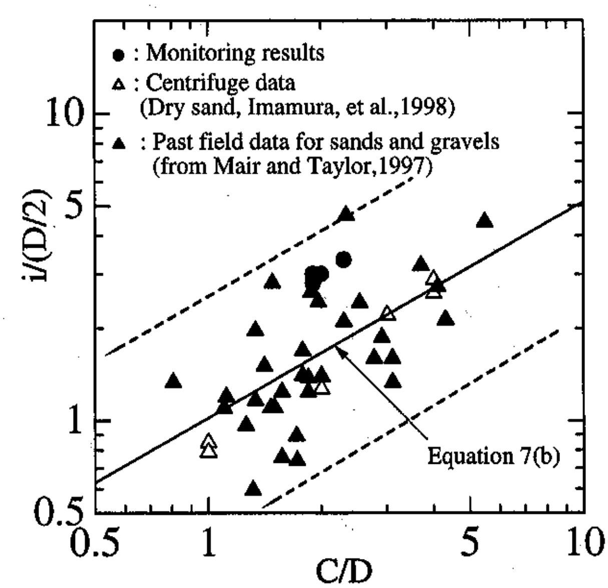

Fig. 12(b). Variation in trough width parameter with cover-to-diameter ratio for tunnels in sands and gravels

where r: Correlation coefficient

(b) Sands and gravels (Past field data, Mair and Taylor, 1997, Centrifuge data by Imamura et al., 1998 and Monitoring results)

$$\left(\frac{i}{D/2}\right)=\left(\frac{C}{D}\right)^{0.7} \qquad \text{(7b)}$$

$$|r|=0.62$$

On the basis of the Correlation coefficient (r) for the above equations, the data can be reasonably approximated by the proposed power functions of C/D, although the data for tunnels in sands and gravels exhibit somewhat more scatter than in the case of clays. The monitoring data presented in this paper are in reasonable agree-

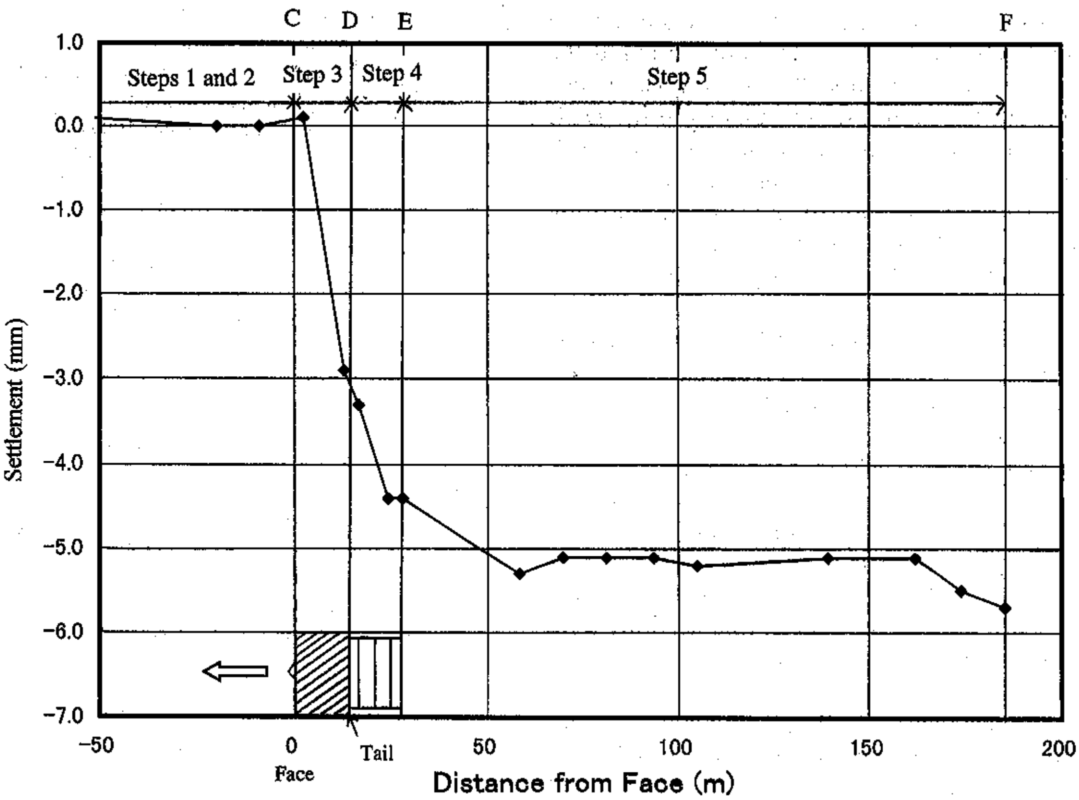

Fig. 13. Measured surface settlement during tunnel driving (Monitoring results at MS-5)

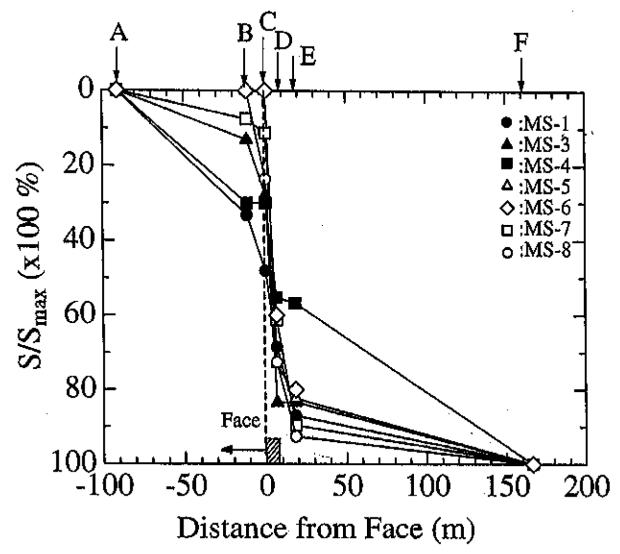

Fig. 14(a). Proportion of settlement during development of longitudinal settlement troughs (Southbound)

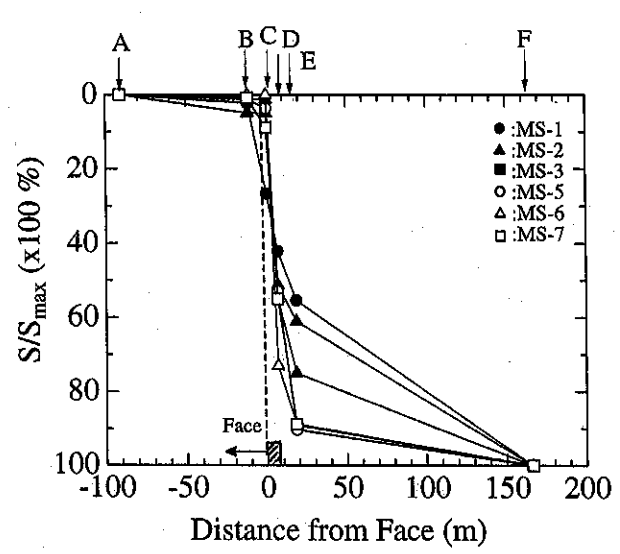

Fig. 14(b). Proportion of settlement during development of longitudinal settlement troughs (Northbound)

ment with the field data for clays in Fig. 12(a), in a similar manner to that shown in Fig. 11(a). In the cases of very shallow tunnels and large diameter tunnels, Fig. 12 and Eqs. (7a) and (7b) may be more useful for tunnel engineers to predict i, because of tunnel diameter having significant effects on i as well as the ground depth.

Longitudinal Settlement

Figure 13 shows a typical example of monitoring results of longitudinal settlement (MS-5, Southbound). Very small settlement at the tunnel face (point C) was observed, probably because the face pressure was carefully and well controlled. The settlement due to the tail void (Step 4) was only about 1.5 mm, which was about 50% of the settlement that occurred during passage of the shield (Step 3). One of the reasons for this is believed to be due to the immediate grouting to fill the tail void, which is an effective means of minimizing ground movements.

Figure 14 shows the change of non-dimensional settlement ratio (S/S_{max}) at each reference point as the shield progresses. Figures 14(a) and (b) show the results of the southbound and northbound tunnels respectively. It can be seen that most of the construction settlement is associated with passage of the shield machine (i.e. between points C and D: Step 3). Only relatively small settlements occurred at Steps 2 (B to C) and 4 (D to E). This is different from recent observations that most of the construction settlement is associated with the tail void (Nomoto et al., 1995). It is inferred that small settlement ratios obtained at Steps 2 and 4 are a consequence of good control of tunnel excavation, for which significant face support was achieved, and immediate grouting using high quality grout. On the other hand, settlement during passage of the shield was to some extent inevitable, despite circulation of high quality slurry. Control and minimizing of the settlement at Step 3 remains the next problem to be overcome.

The settlement above the tunnel face was found to be $0.20S_{max}$ for the southbound tunnel and $0.07S_{max}$ for the northbound tunnel. A mean value of all monitoring sections of $0.14S_{max}$ is in good accordance with $0.11S_{max}$, which was reported for shield tunnelling in alluvial cohesive soil by Nomoto et al. (1995).

It is apparent from Fig. 14 that results for the northbound tunnel exhibit less scatter and smaller settlement ratios at the tunnel face, compared with those for the southbound tunnel. This can be attributed to the fact that the northbound tunnel was constructed after the southbound tunnel. When constructing the northbound tunnel, valuable experience based on performance from the southbound tunnel could be effectively used to achieve an even greater degree of control.

Attewell and Woodman (1982) suggested the cumulative probability curve to be reasonably valid for the longitudinal settlement trough, as shown in Fig. 15. They found that the surface settlement directly above the tunnel face generally corresponds to about $0.5S_{max}$ for tunnels constructed in stiff clays without face support. However, in this slurry shield method, for which there is significant face support, as can be seen in Fig. 14, the surface settlements directly above the tunnel face are only $0.14S_{max}$. Therefore, the major source of surface settlement is further back from the face and this leads effectively to a translation of the cumulative curve in accordance with the recent recommendation by Mair and Taylor (1997), as shown in Fig. 15.

Figure 16(a) shows a comparison between the monitoring results at MS-5 (Southbound) and a translated cumulative probability curve. The translation of the curve was determined so that the settlement at the tunnel face is equal to $0.14S_{max}$, as a mean value from the monitoring results, instead of $0.5S_{max}$. In a similar way, Figs. 16(b) and (c) show field results for a slurry shield in sand (Ata, 1996) and centrifuge model results in dry sand (Imamura et al., 1998) respectively. Settlement ratios at the tunnel face in Figs. 16(b) and (c) were $0.25S_{max}$ and $0.03S_{max}$. It is found from Fig. 16 that the major part of the longitudinal surface settlement trough is well predicted by translation of the cumulative probability curve. In particular, based on Fig. 16(a), a tunnel face settlement of about $0.14S_{max}$ is suggested for settlement predictions due to slurry shield tunnelling in stiff clay. In addition, the assumption of translation of the cumulative probability curve is supported by field and centrifuge data, irrespective of the ground conditions.

Volume Loss

Figure 17 shows the change of the volume loss with time due to shield driving at MS-5. The general trend of the curve is similar to the longitudinal settlement curve in Fig. 15.

Figure 18 shows the relationship between the *i* value

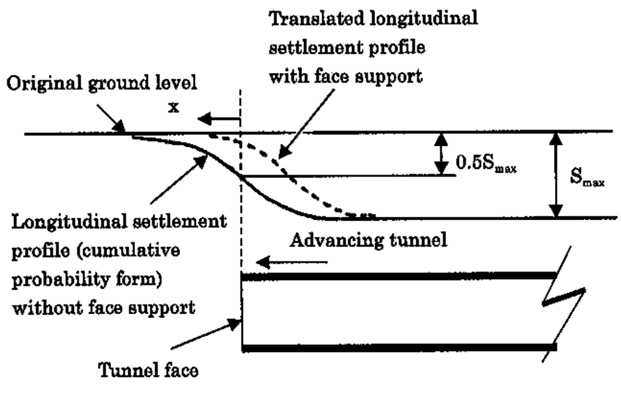

Fig. 15. Longitudinal surface settlement troughs with and without face support (From Mair and Taylor, 1997)

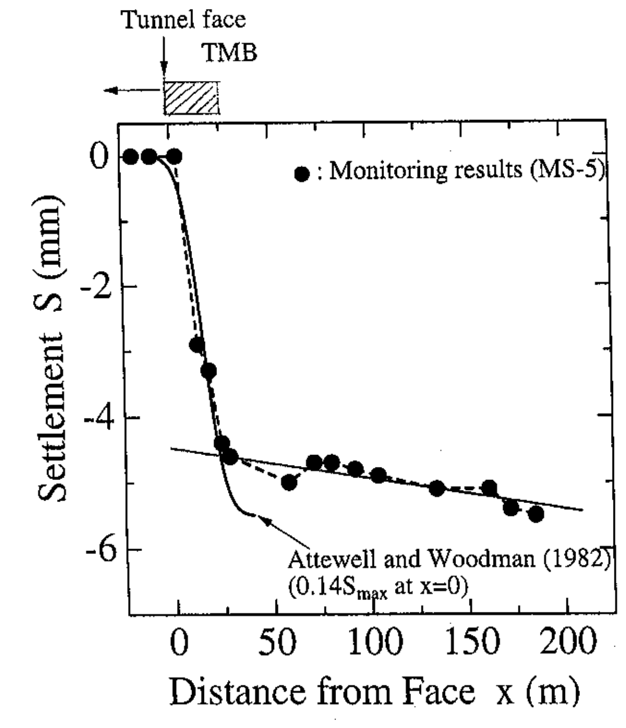

Fig. 16(a). Comparison of the translated cumulative probability curve and monitoring data

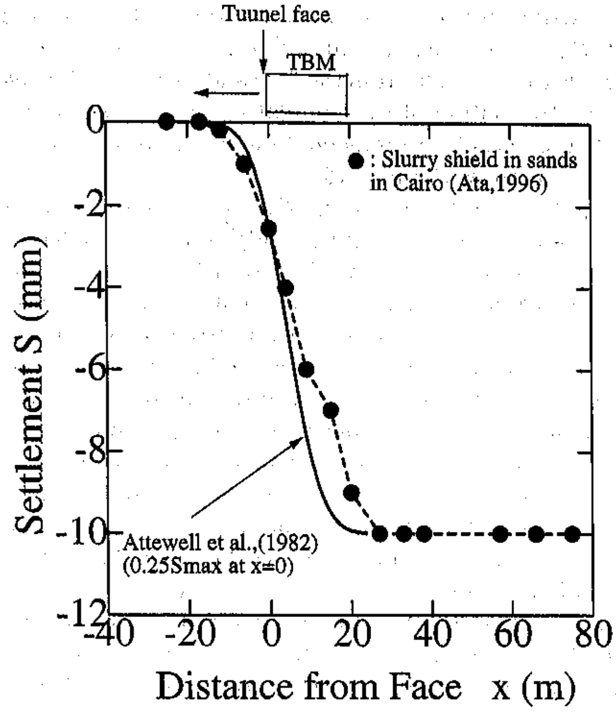

Fig. 16(b). Comparison of the translated cumulative probability curve for field data by Ata (1996)

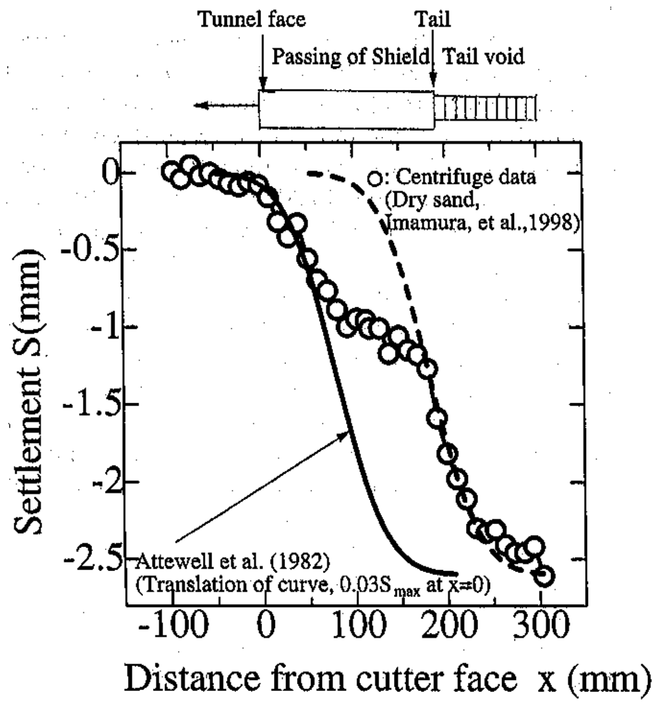

Fig. 16(c). Comparison of the translated cumulative probability curve and centrifuge data by Imamura et al. (1998)

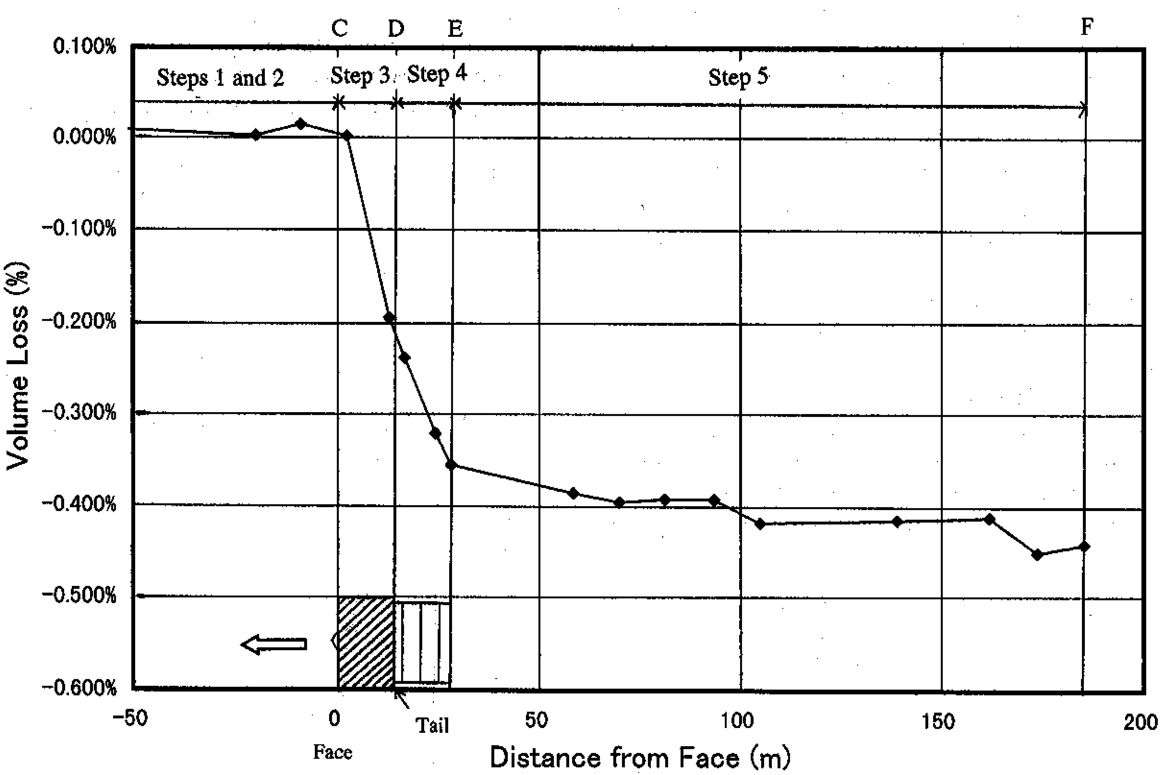

Fig. 17. Volume loss vs. time plot (MS-5, Southbound)

and the magnitude of volume loss at some sections for the southbound tunnel. As also shown in Table 2, the values of volume loss at all sections were significantly less than 1.0 percent which was initially predicted. Figure 18 also shows very little change of i during the shield construction. It is evident that the values of i at the surface stay practically constant, independent of construction stage (and volume loss) from point D to point F. Similar observations are reported by Mair (1979) and by Grant and Taylor (1996) for different stages of centrifuge model

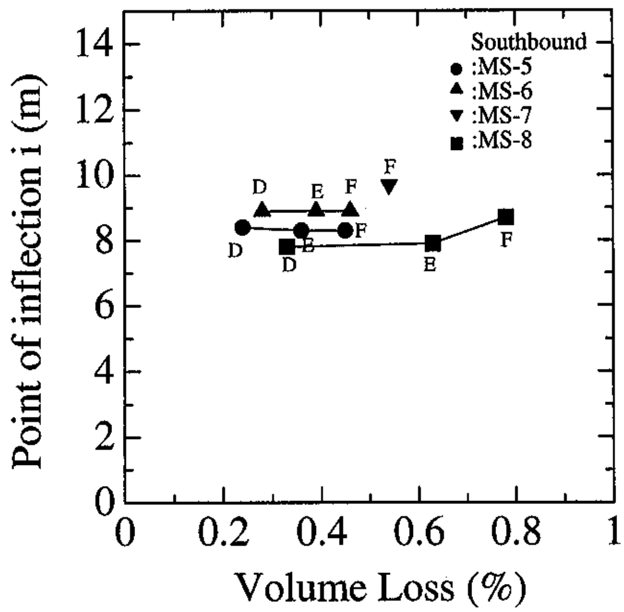

Fig. 18. Relationship between volume loss and *i*

tests.

The average value of volume loss at point F is about 0.7 percent. The scatter in the data is probably a consequence of the variations of the ground conditions, as shown in Fig. 3. In spite of such mixed face conditions, smaller volume losses were measured compared with the predicted value of 1.0 percent. This is due to the high degree of settlement control that was achieved by good construction techniques, including the careful control of face pressure and immediate grouting of the tail void.

The above discussion can be briefly summarized as follows:

1. Compared with predicted values and recent field observations, both S_{max} and the volume loss values were much smaller.
2. Settlements both at the tunnel face and after the tail void were also smaller.
3. The i values were larger, leading to wider and shallower transverse settlement troughs.
4. Based on the extensive monitoring results for the DLR project (D=5.85 m, z_0=15 m, C/D=2), tunnel face settlement of $0.14S_{max}$, a trough width parameter K of 0.60, and a volume loss of 0.7% are suggested for settlement predictions due to slurry shield tunnelling in stiff clays for similar conditions to those as experienced on this project.

CONCLUSIONS

The following conclusions may be drawn from data presented in this paper:

1. Sophisticated mechanized slurry shield tunnelling method in the WRB on the DLR project by careful and well controlled tunnel excavation has been successful in controlling ground movements to values less than those predicted.
2. The maximum ground surface settlements (S_{max}) obtained were less than 15 mm at all monitoring sections and the average value was about 7.0 mm, which was an excellent achievement. This was smaller than past and recent reports of slurry shield case histories in Japan, and is attributed to good control of ground movements. Although S_{max} appeared to decrease with increasing depth of tunnel, any general tendency of S_{max} to decrease with increasing C/D ratio was not clear because of the narrow range of C/D on this project.
3. The transverse settlement troughs were reasonably represented by Gaussian distribution curves. However, the values of point of inflection i have been shown to be wider than those predicted. Therefore, the actual transverse settlement troughs were wider and flatter. This may have been due to the fact that some of the settlement measurements were made on buildings, and their stiffness could have a significant influence on the settlement trough.
4. Surface settlement behaviour of monitoring sections even with mixed face conditions could be interpreted as behaving as if the tunnels were in predominantly clays. This is probably because the tunnels were mostly driven through the WRB clays.
5. Two kinds of practical design charts to appropriately predict surface transverse settlement troughs due to shield tunnelling are proposed for ground conditions such as clays or sands and gravels. They are as follows:

   (a) Linear function of the depth: $i=Kz_0$
   Clays: K=0.5
   Sands and Gravels: K=0.35
   (b) Power function of C/D: $(2i/D)=\alpha(C/D)^{\beta}$
   Clays: α=1.5, β=0.8
   Sands and Gravels: α=1.0, β=0.7
   Equation (a) is appropriate for most tunnels, as concluded by Mair and Taylor (1997). Equation (b) is relevant to very shallow tunnels, particularly if the tunnels are of large diameter.
6. The vital importance of the ground conditions and a detailed knowledge of the construction process to evaluate the ground movements was reconfirmed.
7. Compared with the previous closed face shield data, both the settlement above the tunnel face and the settlement due to the tail void were significantly reduced.
8. The average surface settlement directly above the tunnel face was found to be $0.14S_{max}$. The main part of the longitudinal settlement before and after the tunnel face could be reasonably estimated by a translation of the cumulative probability curve based on Attewell and Woodman (1982), as recommended by Mair and Taylor (1997).
9. Good control of the tunnel excavation resulted in the volume losses being satisfactorily controlled to a mean value of 0.7%, and being smaller than 1.0% at all locations.

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