Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
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Settlement of storehouses during the passage of two parallel shields through soft ground

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ABSTRACT : Two parallel earth pressure type shields for construction of an underground railway, having an excavation diameter of 7.35m, passed under three old storehouses which were constructed on soft ground in 1924, a year after the great Kanto earthquake in Japan. The overburden of the shields was 11m and the SPT N-value of the soft ground was equal to 1. Due to the restrictions that during construction settlements of the storehouses should be limited, the method of compensation grouting was employed, which could be carried out from the inside of the tunnels. It was observed that the maximum amount of differential settlements of the storehouses during tunnelling was 14mm. The efficiency of compensation grouting in limiting the settlement of overlying structures as well as the ground behavior during tunnelling is discussed in this paper.

1. INTRODUCTION

Two parallel tunnels for an underground railway were constructed successfully in very soft ground by means of earth pressure type shield without causing any damage to three overlying old storehouses. Fig.1 shows the plan view of the tunnels and the existing structures. The storehouses were constructed on slab foundations which were supported by a number of timber piles of 5m length and 25cm diameter as shown in Fig.2. The bottom of the piles was inserted to a loose sandy layer whose SPT N-value is within 3 to 4. Since the construction in 1924, the area around the storehouses had undergone subsidence up to 3m, most of which occurred before 1971 due to the severe drawdown of water level caused by the large quantity of pumping of underground water for industry use. Consequently, many cracks had appeared on the walls of the storehouses as can be seen in Photo-1, and only a small amount of additional settlement during the construction of the tunnels was estimated to result in possible collapse of the old storehouses.

Although it was difficult to know exactly the stress states in the storehouses, analysis revealed that the amount of differential settlement acceptable to the structures was as small as 15mm. Compensation grouting was therefore undertaken in the ground between the tunnels and the overlying structures as shown in Fig.2 to limit settlements to acceptably small levels. In fact, the settlement of the ground surface was observed to exceed 60 mm in the case where the compensation grouting was not employed. The zone of compensation grouting was 150m long.

The shield machines employed were of 7.35m diameter and 5.85m length, and had 32 jacks with a total capacity of 4800 tons. RC segments of 30cm thickness and 100cm length were lined in the tunnels. A complete ring was made from seven

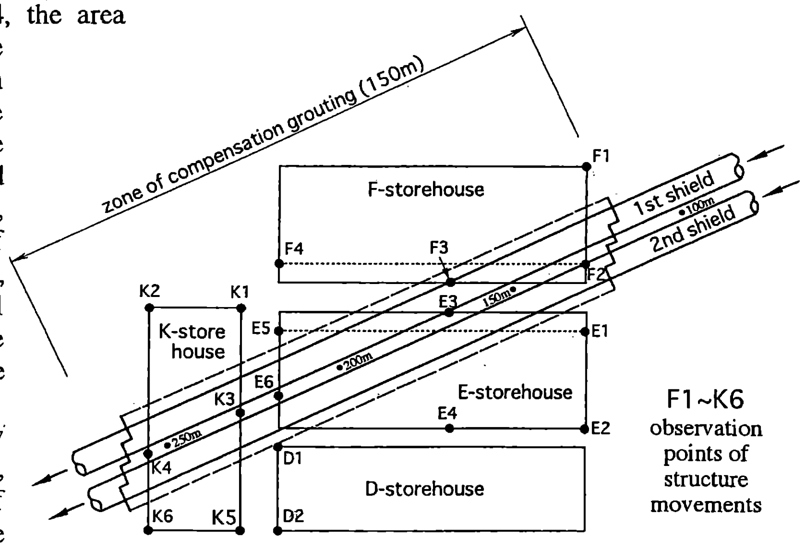

Fig.1 Plan location of two parallel shield tunnels and existing storehouses

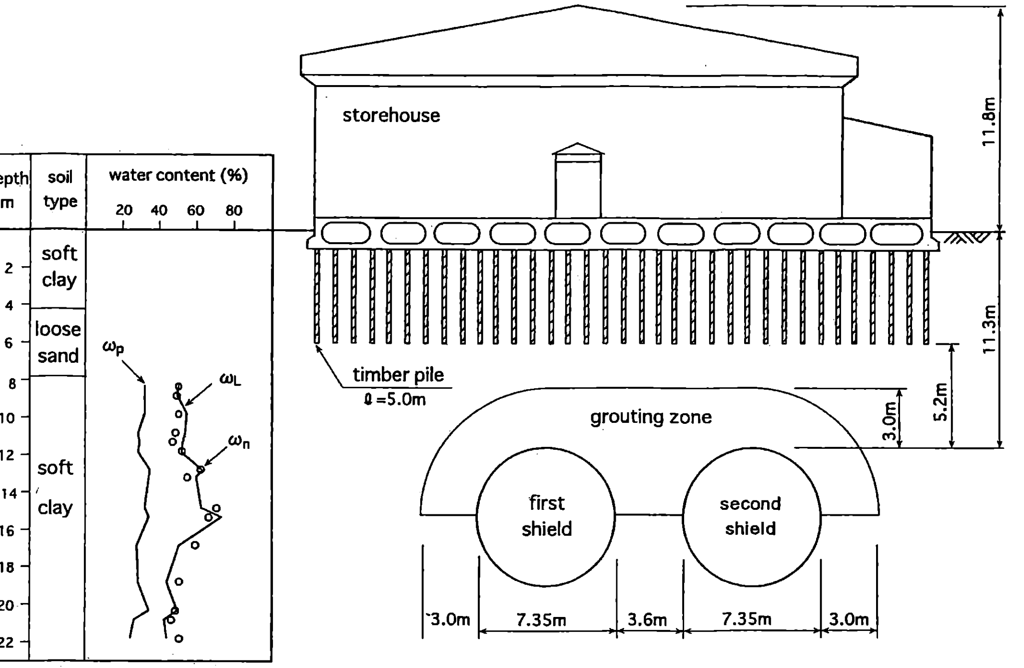

Fig.2 Typical cross section

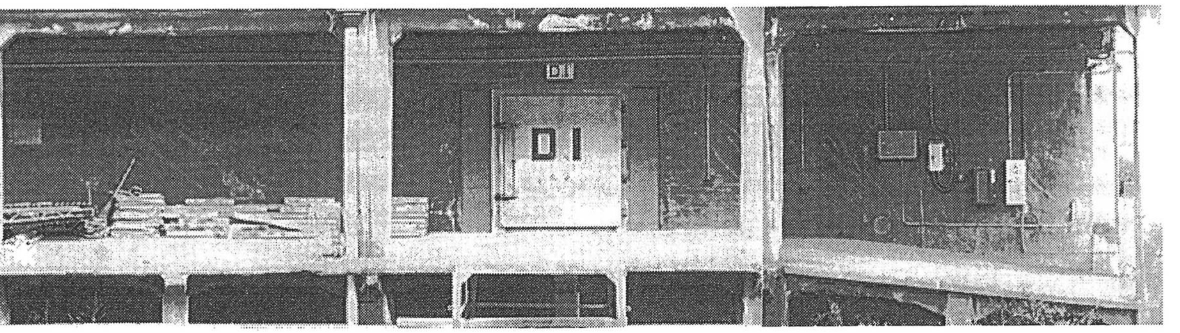

Photo-1 Cracks on the walls of D-storehouse

segments. The two shields passed under the three storehouses with a time lag of one month, leaving a parallel space of 3.6m only. During the tunnel excavation, the tail void was filled immediately by an injection material having a gelling time of 5 seconds. The injecting pressure was controlled at 400 kPa.

2. COMPENSATION GROUTING

The aim of compensation grouting is to limit the movements of overlying structures within a specified level during the construction of the tunnels. The details of compensation grouting are illustrated schematically in Fig.3. After the tunnel excavation and the setting up of segments, grout holes were drilled as quickly as possible into the ground from the segments by means of double tube

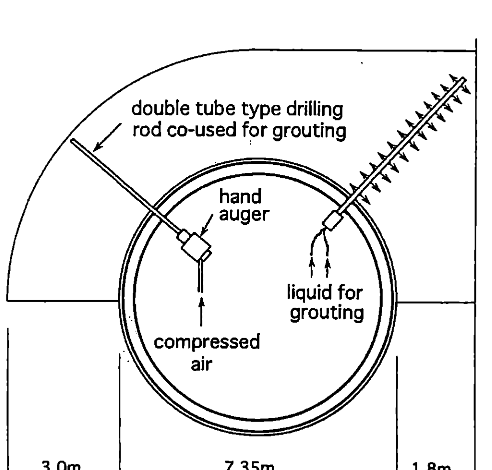

Fig.3 Details of compensation grouting

type drilling rod driven by compressed air, and then grouting was performed from the inside of the tunnel. There were two types of grout hole arrangement adopted in the rings; the first one was 9 holes in one ring (A-type ring) and the second one was 4 holes in one ring (B-type ring). These two arrangements of grout holes were employed alternately in the rings of the tunnels.

It is to be pointed out that the general pattern of ground response to compensation grouting was a temporary heave followed by a settlement. Hence, grouting was undertaken twice in each hole. In the case of the first grouting, the grouting ratio (the volume percent of grouting material with respect to the volume of grouting zone) was determined as 15 % and 20% for the first and the second tunnels respectively, while in the case of the second grouting, the grouting ratio was controlled based on the observed movements of the overlying structures. The total grouting ratio, however, was determined to be less than 30%. Both the first and the second groutings were carried out under a pressure of 500 kPa. The grouting material was a water glass type with a short gelling time.

3. GROUND RESPONSE TO TAIL VOID INJECTION AND COMPENSATION GROUTING

To demonstrate the validity of compensation grouting as a protective measure against settlement, a set of instruments including earth pressure cells, pore water transducers, and vertical extensometers were installed in the area near the F-storehouse. The layout of the instruments are shown in Fig.4 and Fig.5, together with the schedule of the main activities of tunnel construction.

An example of the ground movements observed during the passage of the shields is shown in Fig.6, where the lines C10~C12 represent the vertical displacements at various subground points between the two tunnels. The main features of the ground movements which can be drawn from Fig.6 can be listed as follows. The first one is the fact that the ground was subjected to marked settlements after the passage of the tails of the first and the second shields. It is obvious that the rate of ground settlement was much higher in the case of the passage of the second shield as compared to the case of the first shield. The other feature is that the compensation grouting restrained the development of ground settlement and allowed the ground to heave. However, the ground heave was a temporary phenomenon, after which a subsequent ground settlement took place. The implication is the fact that the amount of ground heave caused by compensation grouting was much greater than that of the subsequent settlement, and this difference covers the

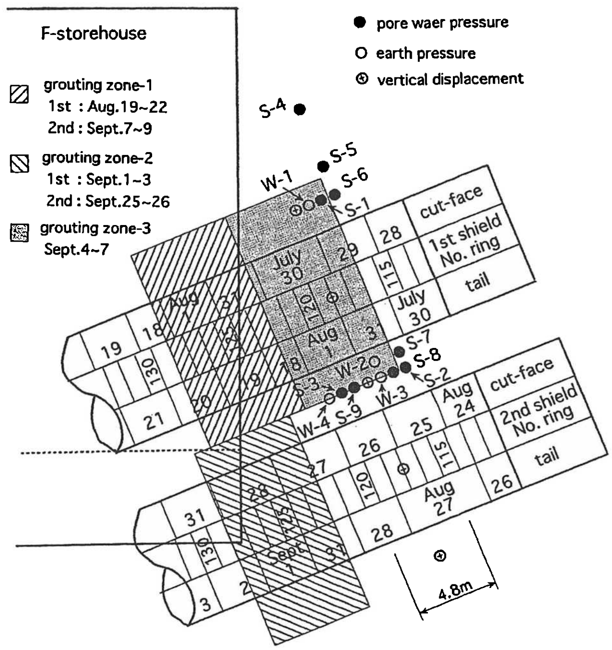

Fig.4 Layout of instruments and advancing schedule of shields

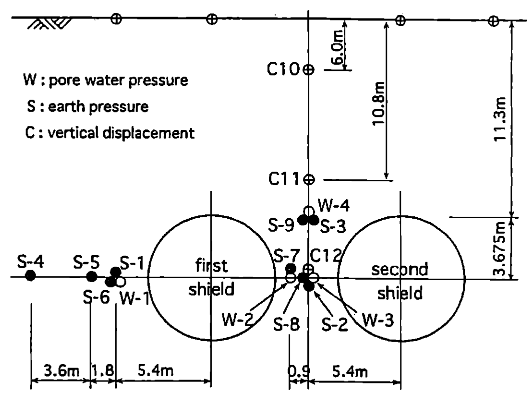

Fig.5 Location of instruments

previously occurred ground settlement.

As can be seen in Fig.6, there were three stages labeled as Step-1, Step-2, and Step-3 in which the ground was subjected to settlement. Considering the curve shape in the said portions, it is possible to fit the observed data with the following hyperbolic equation

$$St=S0-\frac{t}{a+bt} \tag{1}$$

where, St and t represents vertical displacement and time respectively, and S0 is initial displacement, while a and b are constants. Therefore, the rate of ground displacement dSt/dt is given by

$$\frac{dSt}{dt}=\frac{-a}{(a+bt)^2} \tag{2}$$

If dSt/dt at t=1 day is referred to as settlement coefficient, fs, then fs is expressed as

$$fs = \frac{a}{(a+b)^2} \qquad (3)$$

It is to be pointed out that Step-1 in Fig.6 represents the ground behavior against the tail passage of the first shield, and Step-2 shows the ground response to the the tail passage of the second shield after the end of the first grouting in Zone-1 in the first tunnel, and Step-3 denotes the post-grouting behavior of the ground. The values of a, b and fs corresponding to each step can be used to characterize the ground behavior against tunnelling, which are summarized in Table-1, along with the correlation between 1/b and fs in Fig.7. It is apparent that 1/b indicates the ultimate amount of relative settlement.

Table-1 Summary of coefficents of a, b, and fs

step	coeffi-cents	position		
		c10	c11	c12
1	a	0.464	1.244	1.256
	b	0.045	0.045	0.061
	fs	1.784	0.749	0.724
	1/b	22	22	26
2	a	0.129	0.189	0.095
	b	0.059	0.051	0.039
	fs	3.69	3.259	5.280
	1/b	17	20	26
3	a	0.698	0.464	0.38
	b	0.029	0.028	0.024
	fs	1.321	1.917	2.33
	1/b	34	36	42

It is clear from Fig.7 that the ultimate relative settlements of ground subsurface are almost of the same order for Step-1 and step-2, implying that the grouting performed in Zone-1 had little contribution to the ground strength. This may relate to the fact that Zone-1 was about 4m farther from the observation line. The settlement coefficient, fs, in Step-2, however, is much greater than that in Step-1. In addition, the point close to the tunnels has a higher value of fs, especially the point C12 which was between the two tunnel center lines. These tendencies indicate that the extent of the ground disturbance was more remarkable in the case of the second shield passage. On the other hand, the settlement coefficient, fs, in Step-3 reduces significantly in comparison with that in Step-2, and

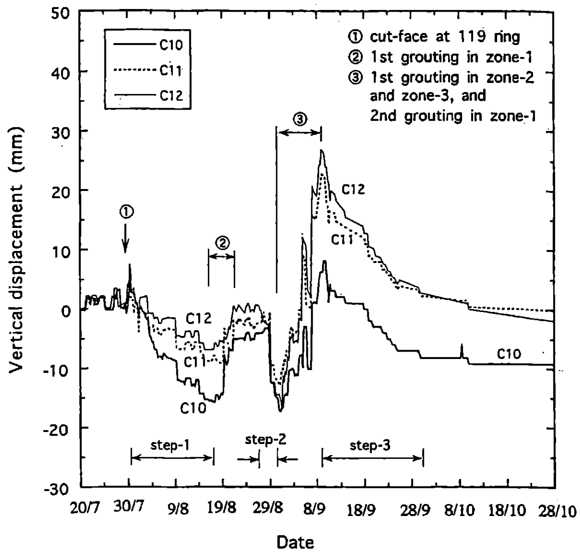

Fig.6 Ground movement observed during tunnelling

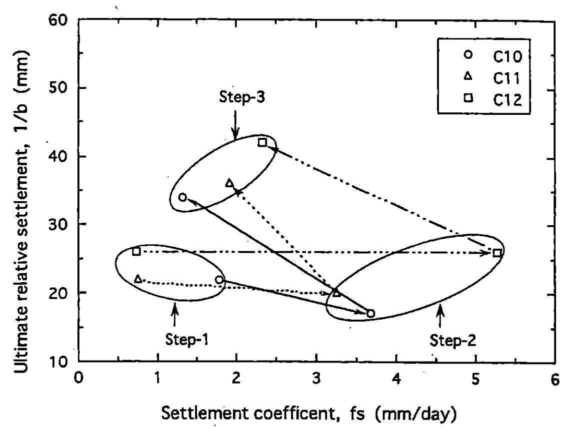

Fig.7 Relation between **1/b** and **fs**

almost become equal to that in Step-1. Consequently, it can be said that the compensation grouting undertaken in Zone-3, which was just bellow the observation line, played a role to strengthen the ground.

Fig.8 shows the changes in pore water and lateral earth pressures in the ground when the first shield was passing through 116~128 rings. The initial values of the ground stresses prior to tunnel excavation were observed as

Depth m	σh kPa	σv kPa	$\sigma h/\sigma v$	Uw kPa
GL-11.8m (crown)	134	201	0.67	102
GL-15.0m (center)	163	262	0.62	132

in which the values of σv are calculated in terms of unit weight. As the shield advanced, the earth pressures started to increase and reached the maximum values with the passage of the shield tail, after which they started to decrease. The maximum observed values of the earth pressures were almost the same with each other at points S-6 and S-8 which were in the same elevation with the tunnel center lines, and were equal to 262 kPa. The absolute increment was 100 kPa which is only 25% of the injection pressure of 400 kPa.

In comparison with the earth pressures, the pattern of the pore water pressures generated in the ground during the tunnelling were quite different. As can be seen in Fig.8, the pore water pressures at W-2 and W-3 which were installed in the location of 118 ring showed a marked increase when the cut-face arrived in 120 ring. As the shield advanced, the pore water pressures increased continuously and obtained their peak values when the cut-face reached 125 ring (with the tail at 119 ring), giving a time lag as long as 19 hours. The increment in the pore water pressures with respect to the initial values are as follows.

Point	initial kPa	peak kPa	increment kPa
W-2	132	234	102
W-3	132	193	61

After reaching peak values, both the earth and the pore water pressures started to decrease and finally returned to the levels which are almost the same as the initial values, implying that the tail void injection induces no additional pressures that remained in the ground.

Fig.9 illustrates the behavior of the ground stresses when compensation grouting was being undertaken in the first tunnel. The grouting was carried out when the shield excavation stopped. Because the grouting was performed in the upper half zone over the tunnel as shown in Fig.2, the most sensitive response to grouting was observed in S-9 and W-4. It is to be noted that S-9 was installed in the location of 119 ring, while W-4 was in 121 ring. When the portion of 123 ring was being grouted, it was observed that there existed a time lag based on the location of the peak values of S-9 and W-4. The earth pressure cell S-9 reached its peak value earlier than the pore water transducer W-4 did, and the difference in time was 8 hours. A similar phenomenon was also observed in the case where the portion of 124 ring was being grouted. The time difference, however, was 5 hours. The grouting made after August 21 was observed to have little

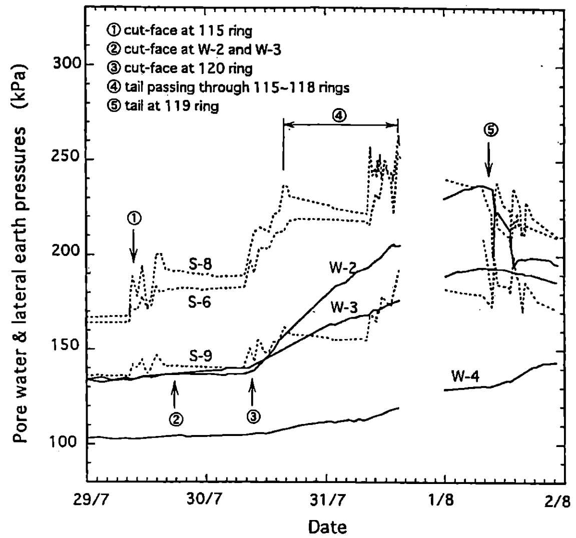

Fig.8 Changes of pore water and earth pressures during the passage of the first shield

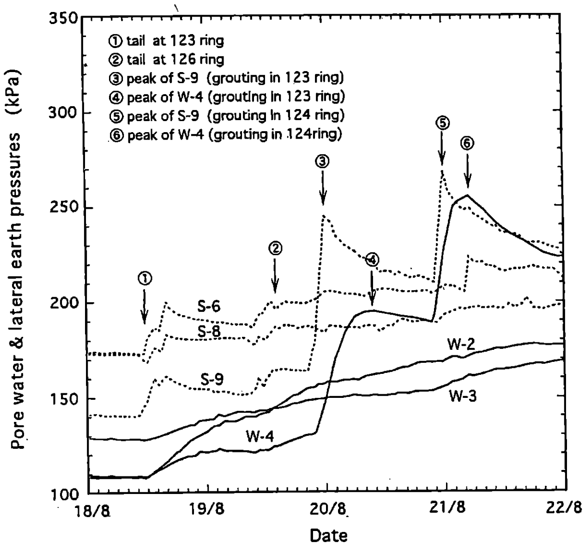

Fig.9 Changes of pore water and earth pressures during compensation grouting (Zone-1)

effect on S-9 and W-4, and these two sensors temporarily showed the following steady values as of August 25.

Point	initial kPa	Aug. 25 kPa	increment kPa
S-9	134	207	73
W-4	102	185	83
(S-9)-(W-4)	32	22	-10

On the other hand, the second shield passed through the observation area in the schedule shown in Fig.4. On October 31, all the pressure meters indicated relatively constant values and some of them are summarized in Table-2.

As can be seen in Table-2, the increment in effective lateral earth pressure was equal to 77 kPa and to 16 kPa at the crown and at the center, respectively. The increment at the crown was about 5 times as great as that at the center, which can be considered as the effect of grouting on ground modification.

Table-2 ground stress states

Point	initial kPa	Oct. 31 kPa	increment kPa
S-9	134	247	113
W-4	102	138	36
(S-9)-(W-4)	32	22	77
S-8	163	188	25
W-3	132	141	9
(S-8)-(W-3)	31	47	16

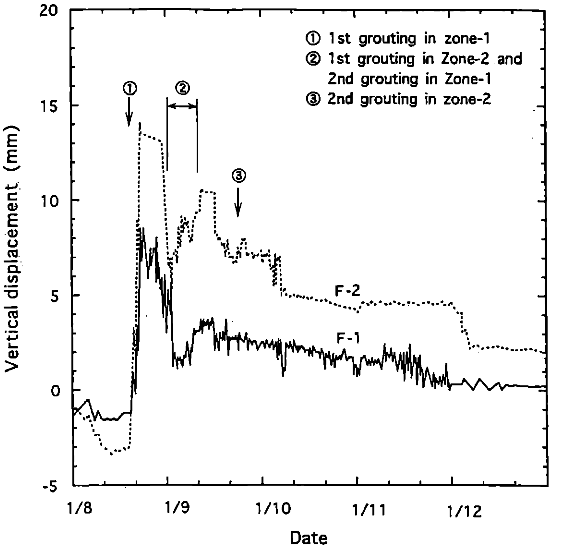

Fig.10 Movement of F-storehouse during tunnelling

4. STOREHOUSE MOVEMENTS DURING TUNNEL CONSTRUCTION

The movements of the storehouses were monitored automatically by means of electrolevels attached to the slab foundations. The plan layout of the observing points are shown in Fig.1. During the construction of the tunnels, compensation grouting was carefully controlled in response to the detailed observations of the structure movements. Generally, in each tunnel the first compensation grouting was carried out within two days after the passage of the shield tail. In addition, the second compensation grouting, if necessary, was undertaken to limit the storehouse movements within the specified level.

An example of the storehouse response to tunnelling is illustrated in Fig.10, in which the movements of F-storehouse at two points (F1 and F2) are plotted against time. It is to be noted that the cut-face of the first shield advanced to 123 ring on July 31 and the second compensation grouting in Zone-1 was finished on September 9, while the second shield advanced to 124 ring on August 27 and the second compensation grouting in Zone-2 was ended on September 26. Initially, the storehouse settled slightly as the first shield approached and passed through it. However, the subsequent compensation grouting resulted in heaving of the structure, followed by the occurrence of ground settlement at the end of grouting. As can be seen in Fig.10, the movement of F-storehouse was more remarkable at F2 point as compared to that at F1 point. The patterns of displacement at F1 and F2 points, however, were similar to each other so that the structure was subjected to a less amount of differential settlement. The maximum magnitude of F-storehouse movement during the tunnelling was observed to be 14.1mm and 14.0mm in terms of absolute and differential displacement respectively, and both of them were less than the specified level of 15mm.

After the two shields had passed under the overlying three storehouses, no more cracks were observed on the walls of the D-storehouse in Photo1. The ultimate displacements of the storehouses were observed to converge within the limitation of ±5 mm, with corresponding differential settlements within the limitation of ±10 mm.

5. CONCLUSIONS

Two shield tunnels with excavation diameter up to 7.35m were successfully constructed in very soft ground without causing any damage to overlying three old storehouses, each structure being particularly sensitive to settlements. The technique of compensation grouting undertaken from the inside of the tunnels had been proven to be very effective in limiting settlements and distortions of overlying structures. The key element of the success in the construction of the shield tunnels were the careful controlling of compensation grouting in response to the detailed observations of the structure movements.