Journal of GeoEngineering, Vol. 1, No. 1, pp. 41-49, August 2006

REHABILITATION OF DAMAGED SOFT GROUND TUNNELS

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ABSTRACT

The paper presents the rehabilitation of soft ground tunnels that were damaged in an accident at the ventilation shaft A of the Pan-chiao Line of Taipei Rapid Transit Systems. Subsoil conditions and the preceding operations at the tunnel-shaft interface are introduced. The accident at shaft A is described and ground settlement data induced by the accident are presented. Procedures taken by the contractor during emergency rescue, damage assessment, and rehabilitation of shaft and tunnels are reported. Possible reasons that caused the accident included drift wood in the ground, cleavage between soilcrete and shaft, and non-verticality of grouting rods. It is concluded that the ground improvement outside the shaft before the shield's arrival played an important role in this accident. The geotechnical engineer should be extremely cautious when breaking the mirror-face on the diaphragm wall under the threat of high groundwater pressure.

Key words: accident, damage, ground improvement, rehabilitation, tunnel.

1. INTRODUCTION

With the rapid growth of population in metropolitan areas, the demand for high-quality mass rapid transit (MRT) systems increases. However, construction areas for infrastructure in big cities are limited. As a result, the MRT facilities, such as soft ground tunnels, are often constructed deeper and deeper into the ground. Accordingly, the risk and difficulties associated with the construction of soft ground tunnels would increase.

Several case histories associated with accidents occurred during soft ground tunneling can be found in the literature. Clough and Leca (1993) reported the excessive ground settlement during the shield tunneling of the Washington D.C. subway extension line. Neyer (1984) introduced the failure of a soft ground tunnel in the suburb of Detroit. Chung *et al.* (1995) described a 25 m-diameter hole was induced by the excavation of Seoul subway No. 2 with the New Austrian Tunneling Method (NATM). So and Endicott (1990) reported that, during the shield tunneling of W-Line of Singapore MRT, about 273 m^3 of soils flowed the tunnel with groundwater at the face. Ju *et al.* (1997b) reported the accident for the construction of Taipei MRT tunnels. Yang and Chao (1997) reported the accident for the construction of ventilation shaft for Hsin-tien Line of Taipei Rapid Transit Systems (TRTS).

In this article, the accident occurred at the ventilation shaft A of the Pan-chiao Line of TRTS is presented. Subsoil conditions and the preceding operations at the tunnel-shaft interface are introduced. The accident at shaft A is clearly described and measured settlement data are presented. Procedures taken by the contractor during emergency rescue, damage assessment, and rehabilitation of shaft and tunnels are reported. Possible reasons that caused the accident are discussed. With the experience accumulated though the case histories, it is hoped that lessons could be learned and the possibility of accident could be reduced for soft ground tunneling in the future.

2. CONSTRUCTION

The construction of Pan-chiao Line Lot CP262 is a part of the TRTS. Figure 1 shows a pair of soft ground tunnels was bored from the crossover at Pan-chiao City, through the ventilation shaft A, to the ventilation shaft B. Two Earth-Pressure-Balance (EPB) shield machines were used for tunnel construction. The EPB shields were made in Japan with an outside diameter 6.24 m and a length of 7.68 m. Shield machines No. 1 and 2 were used for the excavation of down-track and up-track tunnel, respectively. Precast reinforced-concrete segments were used for tunnel lining. The segments were 1.0 m-long with an outside diameter of 6.1 m and an inside diameter 5.6 m. The length of the up-track and down-track tunnel was 1,914 and 1,928 m, respectively. The owner of this project is the Department of Rapid Transit Systems (DORTS) of Taipei City Government. The construction of Lot CP262 was carried out by a joint venture of a famous Japanese corporation and a local engineering corporation.

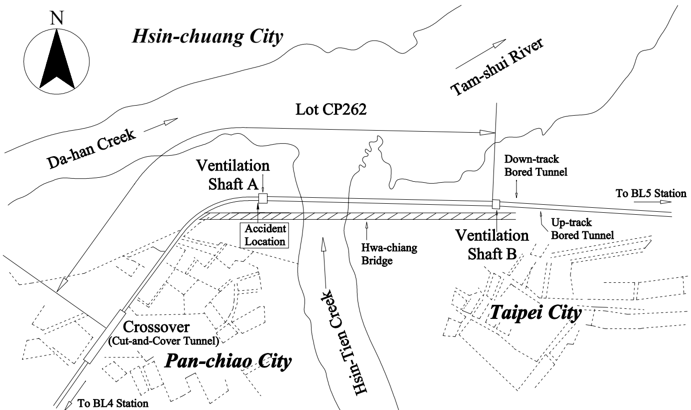

Fig. 1 Location of bored tunnels and shaft A

Manuscript received April 27, 2006; revised June 2, 2006; accepted June 26, 2006.

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Figure 1 shows that part of the tunnel was constructed under the Hsin-tien Creek. The average depth of water of Hsin-tien Creek was 11.1 m. To maintain a safe soil overburden of 12.5 m (about twice the tunnel diameter), the minimum depth of water and soils above the shield should be 23.6 m. In Fig. 2 the overburden above the shield machine was 27.3 m at the ventilation shaft. This was actually the lowest elevation of the bored tunnels. The tunnel west of shaft A had a steep slope of 2.8%, while the tunnel east of shaft A had a flat slope of 0.3%.

Figure 1 shows the ventilation shaft A was only 340 m west of Hsin-tien Creek. The shaft is a 25.9 m-long, 25.4 m-wide subsurface structure. Figure 2 shows the sides of the shaft were made of 1.2 m-thick, 55 m-deep diaphragm walls, which penetrated 3 m into the Ching-mei gravel layer. During the excavation of the shaft, deep-well pumping was executed to reduce the pore pressure in Ching-mei gravel layer. So that no heaving or boiling would occur at the bottom of excavation due to the artesian pressure in the underlying gravel layer.

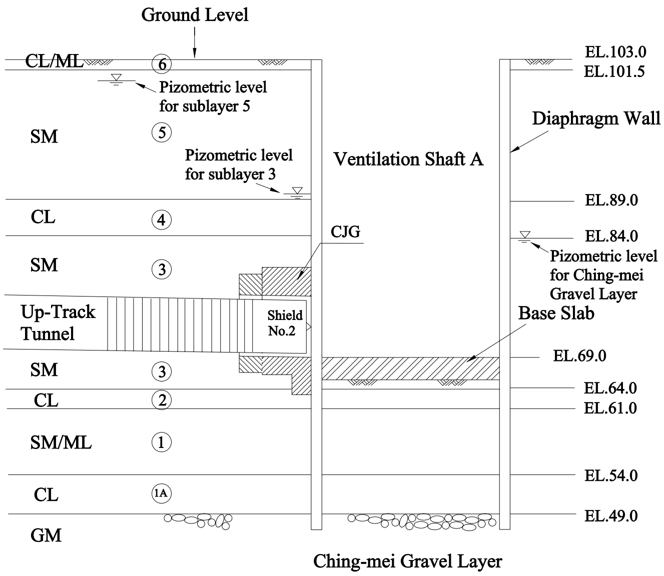

Fig. 2 Subsoil conditions (redrawn after Ju *et al.*, 1997a)

3. SUBSOIL CONDITIONS

Woo and Moh (1990) studied the geological history at the Taipei basin which is an area of Quaternary Holocene alluvial deposits under lain by Tertiary alluvial rock formation. In ascending order, the alluvial deposits consist of the Hsin-chuang Formation (0 to 120 m thick), the Ching-mei Formation (0 to 140 m thick), the Sung-shan Formation (40 to 70 m thick), and a topsoil layer (1 to 6 m thick). Most underground projects in Taipei were located at less than 40 m below ground level; hence most construction works were carried out in the topsoil and the Shun-shan Formation. Hung (1966) proposed that the Sung-shan Formation can be further divided into six sublayers.

A typical soil profile obtained from a boring near the ventilation shaft is illustrated in Fig. 2. In descending order, the sublayers involved with the project are described as follows:

(1) Topsoil and sublayer 6. It comprises soft silty clay with low plasticity (classified as CL/ML). Characteristics of the soil, such as Standard Penetration Test (SPT) N-value, natural moisture content w, void ratio e, liquid limit w_L, plastic limit w_p, specific gravity G_s, and coefficient of permeability k of the soil are summarized in Table 1, if available.
(2) Sublayer 5. Beneath the silty clay is a layer of loose to medium dense silty sand (classified as SM) with a coefficient of permeability, $k = 5.0 \times 10^{-4}$ cm/sec.
(3) Sublayer 4. It comprises soft to stiff silty clay with low plasticity (classified as CL).
(4) Sublayer 3. It comprises silty sand or sandy silt (classified as SM) with traces of shell fragments and organic matters.
(5) Sublayer 2. Beneath the silty sand is another layer of silty clay (classified as CL).
(6) Sublayer 1. It is interbedded with thin strata of sandy silt and silty sand (classified as SM/ML).
(7) Sublayer 1A. A layer of clayey silt (classified as CL) is sandwiched between sublayer 1 and the Ching-mei gravel layer.

Due to the existence of the cohesive sublayer 4, two ground water tables (GWT) were observed in the Sung-Shan Formation. In Fig. 2, the perched GWT in the sublayer 5 is at about 3 m below ground level. However, the pressure head measured during shielding tunneling in the sublayer 3 was located at the elevation of 89.5 m. This was the GWT that caused the accident, and this was the GWT to pay attention to during the rehabilitation of the damaged tunnels.

Underlying the sublayer 1A of Sung-shan Formation is the Ching-mei gravel layer with a SPT N-value greater than 50. Based on the field permeability test, the coefficient of permeability of the gravel was 5.0×10^{-1} cm/sec. The Ching-mei Formation is found to be a confined aquifer with a pressure head changing from 18.5 to 19.5 m below ground level.

Table 1 Subsoil characteristics of Sung-shan Formation

Soil Layer	Soil Type and Classification	Soil Properties
Topsoil and Sublayer 6	Silty clay (CL/ML)	$N = 1 \sim 6$, $w = 23 \sim 28\%$ $e = 0.6 \sim 0.8$
Sublayer 5	Silty sand (SM)	$N = 6 \sim 16$, w = $18 \sim 30\%$ $e = 0.5 \sim 0.8$ $G_s = 2.71 \sim 2.75$ $k = 5.0 \times 10^{-4}$ cm/sec
Sublayer 4	Silty clay (CL)	$N = 6 \sim 10$, $w = 18 \sim 45\%$ $e = 0.6 \sim 1.2$, $w_L = 42 \sim 49$ $w_p = 22 \sim 27$, $Gs = 2.74$ $k = 1.0 \times 10^{-6}$ cm/sec
Sublayer 3	Silty sand or sandy silt (SM)	$N = 13 \sim 35$, $w = 15 \sim 28\%$ $e = 0.4 \sim 0.8$ $G_s = 2.70 \sim 2.74$ $k = 1.0 \times 10^{-4}$ cm/sec
Sublayer 2	Silty clay (CL)	$N = 20 \sim 22$, $w = 34\%$ $e = 0.83$, $w_L = 40 \sim 45$ $w_p = 13 \sim 16$ $G_s = 2.71 \sim 2.74$
Sublayer 1	Silty sand or sandy silt (SM/ML)	$N = 22 \sim 46$, $w = 21 \sim 28\%$ $e = 0.4 \sim 0.8$ $G_s = 2.70 \sim 2.74$ $k = 2.5 \times 10^{-4}$ cm/sec
Sublayer 1A	Clayey silt (CL)	$k = 6.5 \times 10^{-6}$ cm/sec

4. PRECEDING OPERATION AT TUNNEL-SHAFT INTERFACE

To prevent any possible accident during the entrance of shield No. 2 into the ventilation shaft A, several preventive measures were taken at the tunnel-shaft interface. These measures included: (1) ground improvement with the Column Jet Grout (CJG) method; (2) leakage test of the jet-grouted soilcrete; and (3) tail void grouting, which are described as follows. The operation to break of a circular opening (generally termed mirror-face) on the diaphragm wall is also briefly introduced in this section.

4.1 Ground Improvement with CJG Method

In Fig. 2, the triple-jet CJG grouting was conducted for soils outside the diaphragm wall several weeks before shield No. 2 approached its arrival shaft. The double-jet Jumbo-jet Special Grout (JSG) method was commonly adopted for soil improvement at a depth less than 25 m. For this case, the more powerful and more expensive CJG method was selected because soil modification was to be executed from 22.99 to 38.33 m below ground surface. The high pressure 39.2 MPa (400 kgf/cm^2) water jet could cut native soils under high overburden pressure more effectively.

Figure 3 shows the arrangement of ground improvement outside shaft A before the shield machines arrived. For the up-track tunnel, a total of 40 CJG columns were grouted in 5 rows. Each soilcrete column had a diameter of 1.8 m. The centers of columns were arranged in triangular formation with a center-to-center spacing of 1.54 m. The 40 overlapped columns were supposed to form a rectangular soilcrete block next to the tunnel-shaft interface. Since the width of the soilcrete block 6.27 m was slightly less than the length of the EPT shield (7.68 m), an additional 3.0 m-long CW1 grouting zone shown in Fig. 3 was conducted. The composition of CW1 grout is indicated in Table 2. The CW1 fluid is an inorganic fast-set silicate sodium grout additive. It is added to the CW1 grout to enhance the watertight characteristics of the improved ground. The permeation grouting with the CW1 grout could effectively fill the voids among soil particles. With this arrangement, the shield machine can be fully protected by the improved ground when waiting to enter the shaft.

4.2 Leakage Test of Soilcrete

Before breaking the mirror-face on the diaphragm wall for the shield machine to enter the shaft, it was of critical importance to conduct leakage tests for the jet-grouted soilcrete. Bore holes were drilled at the mirror-face location, through the diaphragm wall, into the soilcrete body to the desired length. Then the valve attached to the bore hole was open to measure the rate of inflow of groundwater through the improved ground into the work shaft. For the first test, bore holes shown in Fig. 4 were drilled just penetrating the diaphragm wall. Inflow of water was measured at holes No. 3, 4, 5, and 13. The maximum rate of inflow measured was 70 l/min. The water was assumed to flow through the cleavage at the soilcrete-wall interface. Then chemical grouting with OHA agent was conducted at the soilcrete-wall interface. OHA agent is made of methylene diphenyl diisocyanate (MDI). For this project, the OHA agent was grouted into the cleavage in the ground under pressure. When encountering water, the OHA agent will foam and expand in about 4 minutes to fill all the voids in the cleavage and to block the flow channel of groundwater. For the second test, bore holes were drilled 1.1 m into the soilcrete and inflow was measured at holes 3, 4, 5, 8, and 9. The maximum rate of inflow was 170 l/min. It was obvious that some flow channels existed. As a result, one additional row of CJG columns shown in Fig. 3 was grouted. For tests 3 and 4, the bore holes were penetrated 3.1 m into the soilcrete. Following the construction of the extra row of CJG executed from 24.99 to 40.33 m below ground surface, no more groundwater inflow was observed.

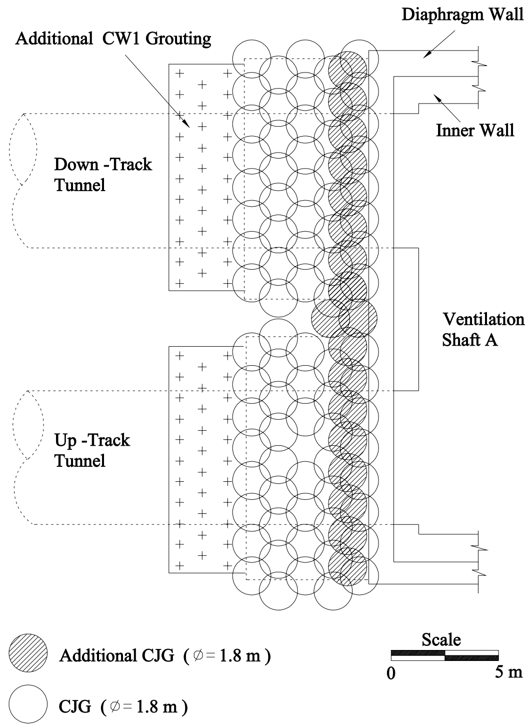

Fig. 3 Ground improvement at shaft A before accident

Table 2 Composition of CW1 Grout (400 l)

Fluid A		Fluid B	
Water glass (l)	75	CW1 Fluid (kg)	22
Water (l)	125	Water (l)	191
Total (l)	200	Total (l)	200

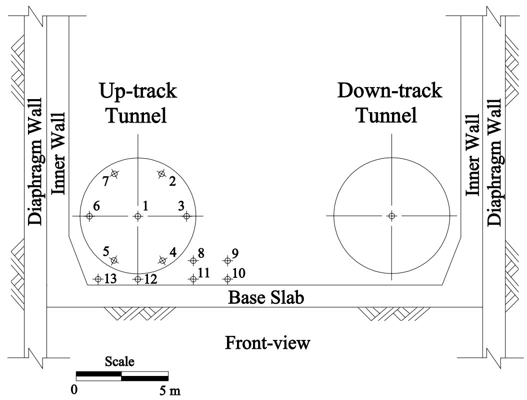

Fig. 4 Borehole location for leakage tests

4.3 Tail Void Grouting

When the shield machine was pushed into the improved ground and approached the diaphragm wall, groundwater might intrude the tail void between the reinforced concrete lining segment and the soilcrete. For this project, the composition of the backfill material, a mixture of fluid A and B, used to fill the tail void is indicated in Table 3.

Table 3 Composition of Backfill Grout (1 m^3)

Fluid A		Fluid B	
Cement (kg)	255	Water glass (kg)	2.5
Bentonite (kg)	38	Water (l)	53.5
Water (l)	849		

4.4 Break of Mirror-Face

Figure 2 shows the shield machine No. 2 stopped when the fish-tail on the cutter-disc touched the diaphragm wall. For this case, the cut open of a mirror-face on the diaphragm wall was divided into 2 stages. Since the CJG soilcrete had been compressed, sheared and disturbed during the passing-through of the shield, leakage test of the soilcrete was conducted again in stage one. When no groundwater inflow was confirmed, 2/3 thickness (0.8 m) of the diaphragm wall was chipped off with a crawler-type hydraulic breaker. Since the soilcrete was disturbed once more by the stage one wall-breaking operation, leakage test was conducted once again before breaking the final 1/3 thickness (0.4 m) of the diaphragm in stage two. The leakage test used the holes 10 and 11 shown in Fig. 4. The complicated and careful process mentioned above indicated the tremendous danger and risk involved with the break through of mirror-face on the diaphragm wall.

5. ACCIDENT AT VENTILATION SHAFT A

The accident at ventilation shaft A occurred on July 16, 1995. Figure 5 shows, at that time, shield No. 1 for the down-track tunnel had departed shaft A and was advancing toward shaft B in Taipei City. Shield No. 2 for the up-track tunnel was waiting for its entrance into shaft A. The manual chip-off of the final 0.4 m-thickness of the diaphragm wall was in progress. Figure 6 shows, the lower-right corner of diaphragm wall was removed and the CJG soilcrete can be observed in the shaft.

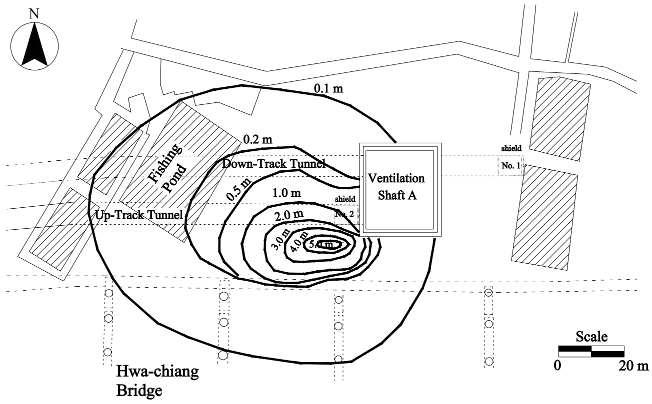

Fig. 5 Contours of ground settlement near ventilation shaft A (redrawn after Ju *et al.*, 1997a)

Unfortunately, at 1:30 a.m., large amount of groundwater leaked into the shaft at the up-track mirror-face at the 6-o'clock position as illustrated in Fig. 6. The initial rate of inflow was about 200 m^3/hr, and increased to 400 ~ 700 m^3/hr three hours later. The contractor tried to seal the groundwater inflow by piling up soil bags at the point of piping but in vain. Soils flowed into the shaft with intruding groundwater. The piping and associated loss of ground caused serious ground settlement near shaft A. The maximum surface settlement was about 5.0 m. The picture in Fig. 7 shows the significant ground subsidence near shaft A after the accident. The contours of ground settlement were illustrated in Fig. 5. The fishing pond indicated in Fig. 5 cracked and was badly damaged. Most importantly, the down-track and up-track tunnels already completed were severely damaged due to ground displacement and tunnel settlement.

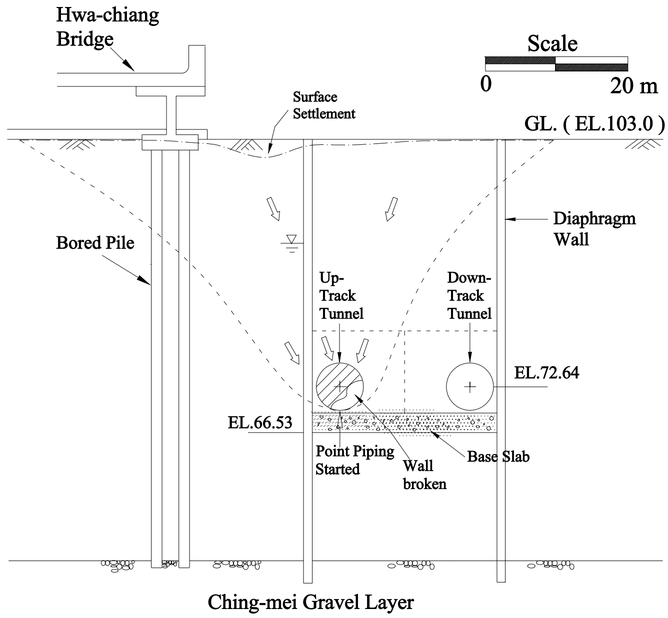

Fig. 6 Location of piping point and estimated ground movement

Fig. 7 Emergency fill for ground settlement near shaft A

In the up track tunnel, failure of the curved bolts between the lining segments was observed. Some segments shifted with each other. Groundwater seeped though the cracked lining segments. The tunnels and shaft were soon flooded, and finally both shield machines were submerged. For safety reasons, all personnel were evacuated from the submerged area. Flooding quickly expanded westward through the tunnels toward the crossover. At 4:30 a.m. of July 16, 1995, the depth of water in shaft A was about 2.5 m. Fortunately, the nearby Hwa-chiang Bridge was not severely affected because it was supported with pile foundations as indicated in Fig. 6.

The operation to restore the damaged tunnels was a very long and expensive process. The entire operation was divided into three phases: (1) emergency rescue; (2) damage assessment; and (3) rehabilitation of the shaft and damaged tunnels. These operations are described in the following sections.

6. EMERGENCY RESCUE

The purpose of the emergency rescue was to stop any further ground settlement as soon as possible. Under the supervision of DORTS, major emergency rescue procedures taken by the contractor are described as follows.

6.1 Balance of Water Pressure

The ground subsidence was induced by piping of soils into the shaft. The inflow of mud was driven by the tremendous pressure head in sublayer 3 indicated in Fig. 2. Under such a high hydraulic gradient, the groundwater was pushed to seep through some fissures and entered the shaft. To stop the mud inflow, it would be necessary to balance the water pressure inside and outside of the ventilation shaft.

The four deep-wells employed to lower the GWT for shaft excavation was used for water supply. At 7:00 a.m. of July 16, 1995, DORTS required the contractor to pump water into the shaft. In 5 hours, approximately 42,000 m^3 of water was poured into the shaft. At the end of pumping, the thickness of water in the shaft was 20.97 m as indicated in Fig. 8. The water level in the shaft was kept at the elevation of 89.5 m.

6.2 Monitoring of Ground Settlement

An instrumented array consisted of surface settlement indicators and inclinometer casings was established at the settlement zone to closely monitor the behavior of ground deformation.

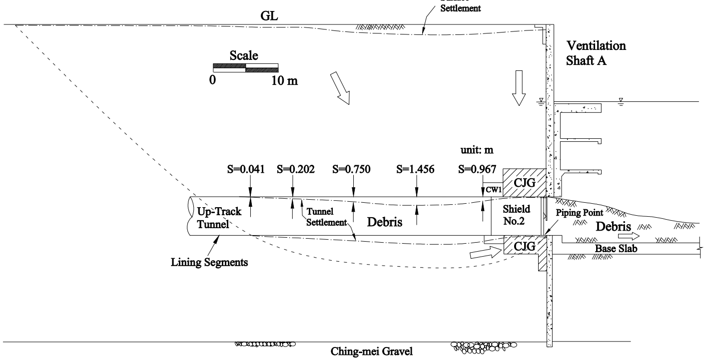

Fig. 8 Settlement of damaged tunnel and estimated ground movement

To prevent the subsidence and tilting of the nearby Hwa-chiang Bridge, settlement points were also set up on the piers of the bridge.

6.3 Fill of Surface Subsidence

The settlement cone shown in Fig. 5 was quickly filled with graded soils as indicated in Fig. 7. The total amount of fill used was about 4,850 m^3.

6.4 Ground Stabilization by Grouting

After the shaft was filled with water, monitored data indicated that the fill over the settlement zone continued to subside. The contractor decided it was necessary to conduct grouting for the disturbed ground. Figure 9 shows a total of 18 grouting-holes were arranged for A, B, C, and D zones. Grouting depth started from 20 m up to 5 m below ground level. The LW grout injected into the ground consisted of cement, water, and water glass. The LW grout was expected to fill the voids in the soils thus stabilize the disturbed ground.

6.5 Building Protection

To protect the nearby Hwa-chiang Bridge and the fishing pond, two rows of steel sheet piles were driven as illustrated in Fig. 9. The stiffness of the sheet pile was expected to limit the extent of ground relaxation. A total of 255 pieces of 13 m-long sheet piles were driven to fabricate the 102 m-long cut-off wall.

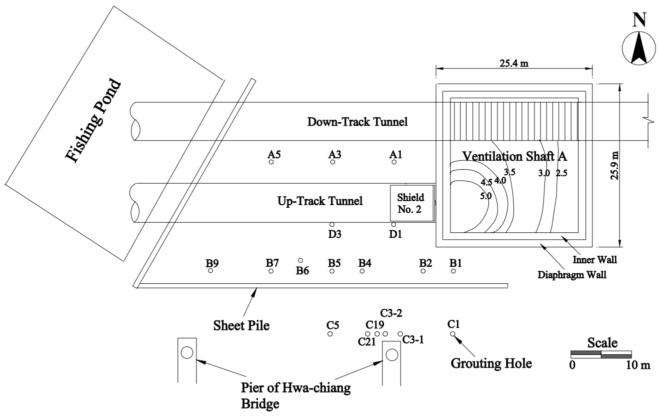

Fig. 9 Emergency treatment with grouting and sheet piling

7. DAMAGE ASSESSMENT

After conducting the emergency rescue procedures, the ground around the shaft gradually regained it stability. Before taking any rehabilitation action, it was necessary to evaluate the degree and extent of damage near the tunnels. The assessment results would be used to select appropriate methods for tunnel restoration.

7.1 Investigation of Disturbed Ground

During the accident, large amount of soils flowed into the tunnels and shaft with groundwater. The soils around the up-track tunnel were severely disturbed. To assess the extent of ground relaxation, subsurface exploration was conducted along the tunnel alignment after the accident. Split spoon samples of soil were collected and physical properties of the disturbed ground were reported. The SPT N-values collected after the accident were compared with the N-values obtained before the accident. At a specific location, a decrease of SPT N-value implied the soils there might had been disturbed or even washed away during the accident.

7.2 Investigation of Damaged Tunnels

During the accident, some lining segments of the tunnels were badly damaged by the excessive settlement. It was also necessary to evaluate the extent of tunnel settlement to define the zone of segment replacement. Investigation was conducted by drilling bore holes from ground surface to detect the amount of tunnel settlement and the integrity of the segments in the ground. The condition of sedimentation in the shaft and tunnels was explored by professional divers.

The settlement of the damaged up-track tunnel is illustrated in Fig. 8. The maximum measured settlement was 1.456 m. Shield machine No. 2 and its associated facilities were submerged and totally damaged. From ring 780 to 818, 39 lining rings were badly damaged and needed replacement. About 3,300 m^3 of soils flowed into the tunnel and about 1,500 m^3 of soils flowed into shaft A. The thickness of sedimentation in the shaft was indicated in Fig. 9. In the ventilation shaft, the submerged electrically, mechanical, communication and illuminating equipments were all seriously mutilated.

7.3 Geometry of Ground Movement

Based on the settlement curve of ground surface, settlement of damaged tunnel, and the location of the piping point, the longitudinal geometry of ground movement can be estimated as shown in Fig. 8. In the figure, the loss of ground at the piping point caused the soils under the tunnel to be eroded or even washed away. The tunnel and overburden soils subsided accordingly. The estimated transverse geometry of ground movement is indicated in Fig. 6. It is obvious that the surface settlement trough is not symmetrical with the up-track tunnel. Greater surface settlement was observed on the Hwa-chiang Bridge side. It is most probably that the wall friction between the diaphragm wall and soil helped to reduce the ground subsidence on the down-track tunnel side.

8. REHABILITATION OF SHAFT AND DAMAGED TUNNEL

The long and complicated procedures involved with the rehabilitation of shaft A and damaged tunnels are described in the following sections.

8.1 Pumping of Sediments from Shaft A

Before rehabilitate the damaged tunnels, the soils and water in ventilation shaft A must be removed. With the help of six divers, 4 inch-diameter steel pipes were used to suck the soil-water mixture from the bottom of the shaft. The mixture was pumped to a sedimentary tank at the ground surface. The soil sediments obtained from the tank was dumped by trucks. The water separated from the tank was circulated back to the shaft to balance the water pressure outside the shaft. It should be noted that, during the pumping of sediments, the water level in the shaft must be kept at the elevation of 89.5 m to prevent any piping from happening. The sediment removal operation started on August 4 and was completed on August 24, 1995.

8.2 Plugging of Mirror-Face with Hydrocrete Wall

The plugging of mirror-face in the shaft with a hydrocrete wall was proposed by the contractor due to the following reasons:

(1) When the accident happened, a third of the 0.4 m-thick diaphragm had already been chipped off. The strength and water-blocking ability of the remaining diaphragm wall became insufficient.
(2) Pumping of water from shaft would decrease the water pressure in the shaft. The strength of the retaining wall should be strong enough to resist the groundwater pressure outside the shaft.
(3) The New Austrian Tunneling Method (NATM) was selected for the rehabilitation of the damaged tunnels. The compressed-air method would be used as an auxiliary technique. The hydrocrete wall should be tight enough to prevent leaking of compressed-air from the tunnel.

Based on the considerations mentioned above, the highly water-tight, high strength (designed compressive strength 23.5 MPa) hydrocrete was selected as the plug material. Hydrocrete is the special concrete that can be constructed in water. Figure 10 shows the hydrocrete retaining wall was constructed in the shaft before pumping water form the shaft. It should be noted that the placement of hydrocrete in a submerged shaft was actually quite complicated. Due to the scope of this article, it was only briefly described here.

8.3 Grouting under Damaged Tunnel

During the accident, large amount of groundwater and soils flowed into the tunnels and shaft. Figure 8 shows the up-track tunnel settled significantly because the soils under the tunnel was eroded or even washed away. If the voids and loose pockets in soils under the tunnel were not properly treated, during the following construction, the tunnel might settle more as a result of insufficient bearing capacity. Grouting holes were drilled at two sides of the tunnel down to the elevation of 69.0 m, which was the bottom elevation of the tunnel. The CB (cement + bentonite) grout injected under the damaged tunnel included 250 kg of Portland cement, 50 kg of bentonite, and 900 liter of water. Grouting operation was stopped when the injection pressure was 49 kPa (0.5 kgf/cm^2) over the ground water pressure, which implied the soil voids were properly filled.

8.4 Tunnel Voids Filled with CB Grout

In the next stage of rehabilitation, CJG grouting would be carried out around the damaged tunnel. To prevent the leakage of CJG grout through the damaged segments into the tunnel, the voids in the damaged tunnel were injected with CB grout. During the damage assessment it was found that, from ring 780 to ring 818, 39 segment rings were badly damaged. The contractor decided to seal the tunnel at ring 775 and 776 with LW grouting and a special set of obstruction form. The grouting wall at Ring 775 and 776 and the hydrocrete wall next to the shield machine No. 2 could effectively seal both ends of the damaged tunnel. After the two ends of the tunnel were blocked, CB grout was injected through the grout pipes shown in Fig. 10 to fill the voids in the damaged tunnel.

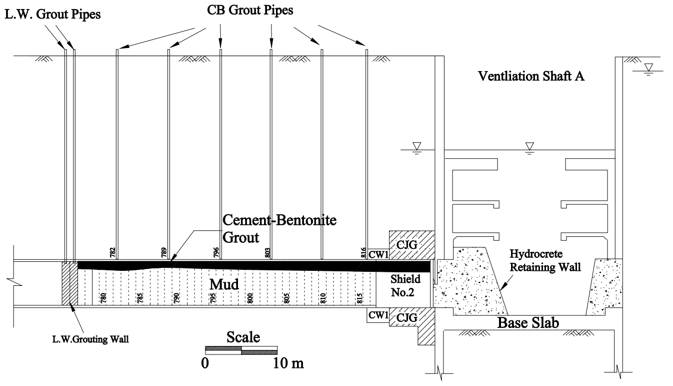

Fig. 10 Damaged tunnel filled with cement-bentonite grout

8.5 Jet Grouting around Damaged Tunnel

During the accident, the soils around the tunnel were severely disturbed. During the re-excavation of tunnel with NATM, the surrounding loose soils might collapse after removing the damaged lining segments. To increase the self-standing ability of surrounding soils and to enhance the safety of construction, jet grouting was conducted for soils around the damaged tunnel. For the up and down-tracks, a total of 192 2.3 m-diameter soilcrete columns were fabricated by cross-jet-grouting and another 276 1.8 m-diameter columns were grouted with column-jet-grouting. Figure 11 shows the protective grouting zone extended from 4.5 m above the tunnel to 6.5 m below the tunnel.

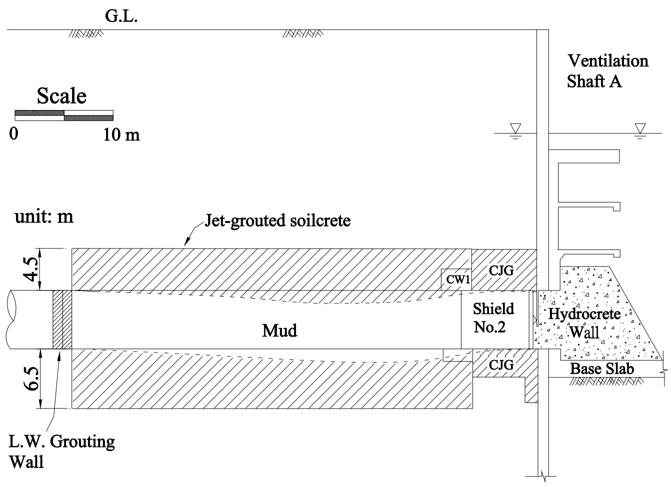

Fig. 11 Jet grouting around the damaged tunnel

8.6 Pumping of Water from Shaft A

Figure 12 shows the pumping of water from shaft A in eight stages with two 4 inch-diameter steel pipes. It should be mentioned that as the water was pumped out of the shaft, the pressure head in the shaft would decrease. The water pressure inside and outside the shaft became unbalanced stage by stage. The groundwater outside the shaft might intrude the ventilation shaft again. For this reason, the pumping operation was conducted with extreme care. For the first stage, water level was lowered from EL. 89.5 to EL. 87.0 m. At the end of the stage, pumping stopped, and the water level in the shaft, ground water table in the ground, and ground settlement were closely monitored. Only when all monitored data were assured to stay in the safe range, the next stage of pumping would proceed. The pumping of water was carried out in eight stages as shown in Fig. 12, until all water in the shaft was removed.

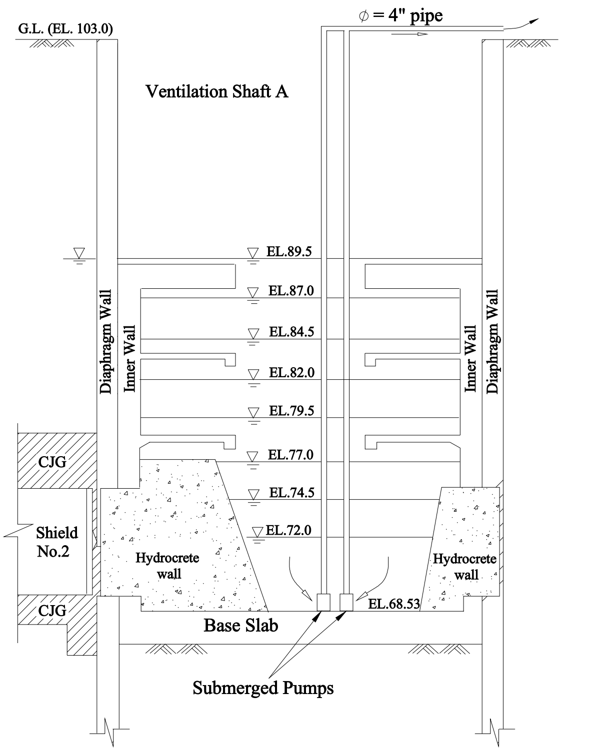

Fig. 12 Pumping of water from impounded shaft in eight stages

8.7 Ground Freezing at Tunnel-Shaft Interface

Ground freezing was conducted to freeze the soils and water surrounding shield machine No. 2. As the rehabilitation work approached the mirror-face, it would be necessary to disassemble the shield machine, chip-off the hydrocrete wall, and replace damaged lining rings. Without any groundwater resistant measure, piping might happen again. As a result, the contractor decided to freeze the ground at the tunnel-shaft interface.

Whittaker and Frith (1990) reported that ground freezing is a stable and effective method for groundwater control, though the method needs more advanced technique and more expenditure. The frozen soil is highly water-tight and can be closely bonded to reinforced concrete and many other materials. For this project, the brine circulation system was selected for ground freezing. The circulations of −25°C $CaCl_2$ brine carried the heat and energy from the ground. The measured ground temperature was 24°C. The circulation of brine would slowly cool down the ground temperature to the design value of −12°C. Only when the soils at the tunnel-shaft interface were properly frozen, the hydrocrete wall can be removed.

Figure 13 shows, to freeze the soils, 62 vertical freezing pipes were installed with a horizontal spacing of 0.8 m. To measure and control the variation of ground temperature during the freezing stage and maintenance stage, eight vertical thermal sensor pipes (S1 to S8) were installed. In Fig. 13, ADD1 and ADD2 were the extra freezing pipes added to assure the cut-off of water flow at the soil-wall interface. ADD3 and ADD4 freezing pipes were added because the verticality of nearby freezing pipes measured with an inclinometer was not satisfactory.

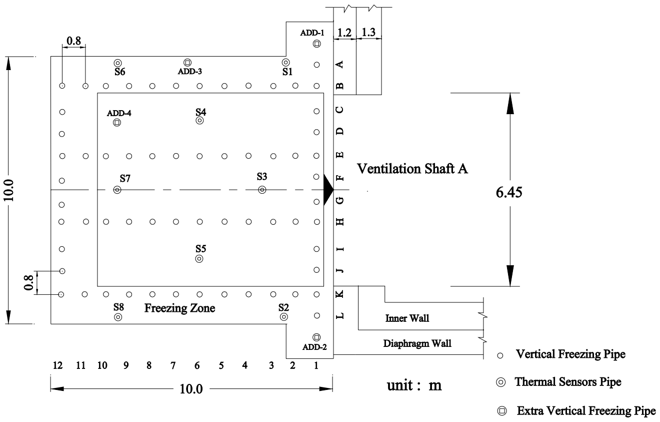

Fig. 13 Top-view of freezing pipes and thermal-sensor pipes

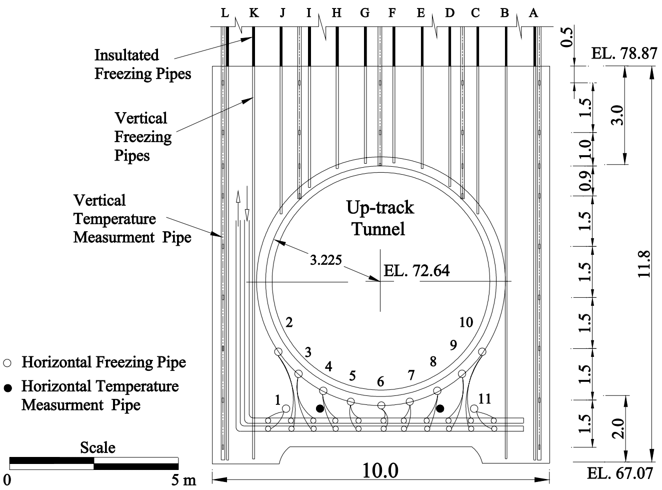

Fig. 14 Front-view of freezing pipes and thermal-sensor pipes

Since the shield machine remained in the up-track tunnel, it was difficult for all vertical freezing pipes to penetrate the steel shield. Figure 14 shows eleven horizontal freezing pipes were installed from the shaft to freeze the soils under the steel shield. The horizontal freezing pipes penetrated the hydrocrete wall, through the diaphragm wall, into the soilcrete. The contractor reported that the moisture content of soil was 25%. With the circulation of the −25°C brine, it took about 90 days to glaciate a 1.4 m-diameter frozen-soil column. Based on the temperature monitored with the thermal sensors, the operation of ground freezing had been quite successful.

8.8 Tunnel Re-Excavation with NATM

To resist groundwater intrusion, compressed-air was applied during the re-excavation of the up-track tunnel. An air-lock was established in the tunnel near the crossover. The air pressure applied to depress the groundwater pressure increased form 29.4 kPa (0.3 kgf/cm^2) to 190 kPa (1.94 kgf/cm^2).

Figure 15 shows the re-excavation of the up-track tunnel was divided into five steps. In step one, the CB grout filled in the tunnel was excavated. Then the damaged lining segments near the crown and the CJG soilcrete above the crown were chipped off with manual or crawler-type hydraulic breaker, and a 50 mm-thick shotcrete layer was sprayed. After placing the steel wire mesh, another 150 mm-thick shotcrete layer was sprayed.

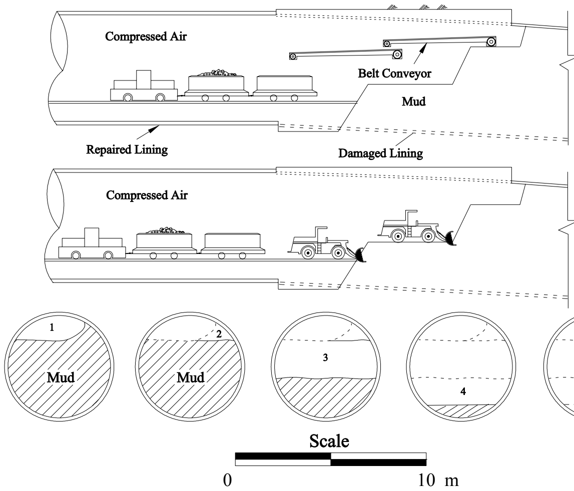

Fig. 15 Sequence of re-excavation in damaged tunnel

After assembled the new ring of lining segments, the void between the segment and shotcrete was backfilled. The same sequence of procedures was carried out for step 2 to 5.

With the efforts of the DORTS and the contractor, the rehabilitation of the damaged tunnels was successfully completed on June of 1997, which was almost two years after the accident. Pan-chiao Line of the Taipei Rapid Transit Systems was completed and started its operation on August 31, 2000.

9. DISCUSSION AND CONCLUSION

It is important to discuss why did the accident happen? What were the possible reasons that caused the event? In fact, with only limited evidence at hand, it was difficult to conclude what was the main reason that induced the accident. However, several possible reasons that might be responsible for the accident are discussed as follows.

(1) Drift wood in the ground

Ventilation shaft A was constructed only 340 m from the river bank of Hsin-tien Creek. Shell fragments and organic matters were found in soil samples obtained from the alluvial deposits. Pieces of drift wood were observed in the CJG soilcrete chipped-off at the tunnel-shaft interface. The drift wood shown in Fig. 16 would block the high-pressure jet to cut native soils and might have adverse effects on ground improvement.

Fig. 16 Drift wood obtained between diaphragm wall and shield machine

(2) Cleavage between soilcrete and shaft

Jet grouting was conducted outside the work shaft before shaft excavation. Mana and Clough (1981) reported that, during the excavation and bracing of the retaining structure, the retaining wall would yield laterally toward the excavation. If the CJG soilcrete outside the wall did not yield laterally the equal amount, a narrow cleavage would exist between the soilcrete and the diaphragm wall. For this case, the groundwater inflow rate of 70 l/min measured during the leakage test for the up-track tunnel indirectly justified the existence of the cleavage between the soilcrete and shaft. For this project, before the accident, the measured maximum lateral displacement of the diaphragm wall was 69.6 mm at the depth of 21 m. After the accident, the maximum wall displacement toward the excavation increased to 87.5 mm at the depth of 20.5 m.

(3) Non-verticality of grouting rod

In design, all grouting rods were assumed to be vertical. However, due to the variation of resistance that might be encountered in the ground during drilling, the grout rod may not be kept totally vertical at all times. For jet grouting at 22.99 to 38.33 m below ground level, a slight deviation of the grouting rod from vertical would induce a significant horizontal shift of the soilcrete column formed in the ground. If a few grouting rods were slightly tilted during grouting, discontinuity might exist in the soilcrete block, even though the soilcrete columns were designed to be overlapped. For the local contractor, the high-precision bubble levels are commonly used for verticality control. A few jet-grouting facilities made in Japan have built-in inclination control for its grouting rod. However, how to control the verticality of the grouting rod remains a challenge for the ground modification contractor.

Based on the discussion above, it may be concluded that the ground improvement outside the shaft before the shield's arrival played an important role in this accident. The geotechnical engineer should be extremely cautious when breaking the mirror-face on the diaphragm wall under the threat of high groundwater pressure. The accumulation of case histories in the literature would help the geotechnical engineer to better understand the problem and hopefully to learn how to prevent it.

ACKNOWLEDGMENTS

The writers wish to acknowledge the Department of Rapid Transit Systems, Taipei City Government for providing valuable information. Special thanks are extended to the National Science Council of the Republic of China government (NSC 86-2211-E-009-007) for the financial assistance that made this investigation possible.

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