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Performance of remedial treatment for cave-in collapse of a subway tunnel

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SYNOPSIS : This paper reports a case study for a cave-in incident which took place during construction of a subway tunnel. The project site is located close to tow rivers,and the ground water level is about 8m below the ground surface. Two major possible cause of collapse were evaluated with regard to the stability condition. The first possibility considered was the appearance of a slickneside and its unfavorable geological structure. The second possibility studied was the effect of blasting vibration. The evaluation was made based on the field records. The remedial treatment has been suggested. The performance of the remedial treatment is monitored by means of insrumentation. Discussions are made on how effectively the treatment was performed at the site. The excavation was stopped and the face was reinforced 6m before the collapse location. A construction method was suggested to maintain the stability for the continued excavation.

1 INTRODUCTION

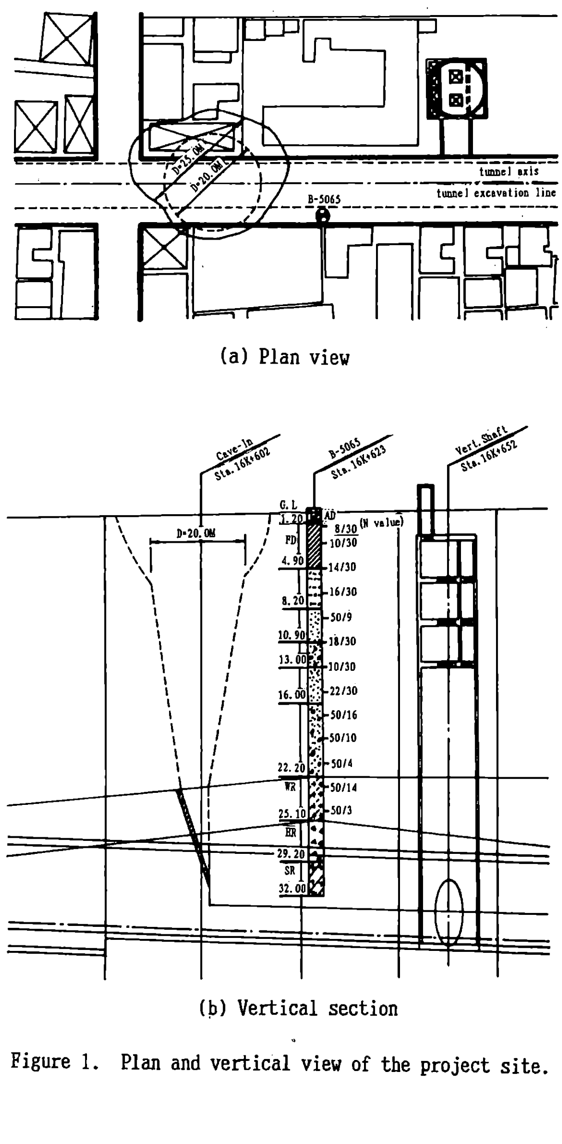
A cave-in incident took place during construction of a subway tunnel. The site is located in the west part of Seoul and close to Han River. The project site includes some commercial and residential areas. Above the tunnel there exists 2 to 5 story old factory buildings and commercial buildings along an old road of 15m width. The boundary of ground subsidence is greater than a circle of approximately 25m diameter.

The site plan is shown in Figure 1 together with the vertical section. The location of the collapse is 16K+602, 50m away from the vertical shaft. Right above the crown of the last facing, there existed a thin layer of weathered granite rock overlain by a thick alluvium deposit. When the facing was collapsed, the works of shotcrete and steel support had been finished 1m behind the facing, and the rock bolt installation had been finished 2m behind the facing. And 17 fore polings had been installed against the crown of the excavated face.

The accident started to break out about 16:00 PM November 27th 1991 and the entire collapse came to an end 4:50 AM the next day. A three story hospital building was buried and neighboring buildings were severely damaged. Detailed circumstances of the incident are summarized in Table 1.

Table 1. Circumstances of the incident.

Time	Circumstances
91.11.27	
10: 40	Excavation via blasting was finished at 16K+602.
11: 30~	Removing the excavated materials was started.
16: 00	A small rock mass dropped.
16: 00~	Removing the excavated materials was stopped. The face was reinforced by shotcrete and retained by the backfill.
21: 00	Further dropping of rock masses with soils took place.
21: 10	Emergency notification was made to the police and the residents.
22: 00~	Gravels in the upper layers dropped. Backfilling was stopped and workers were evacuated.
22: 20	Ground subsidence took place from the tunnel to the surface road
91.11.28	
03: 20~ 04: 50	The 3-story building was buried and neighboring building were severely damaged. Construction equipments were buried.
10: 00~	Rehabilitation work was started for the buried cables and conduits.
23: 00	Soils were backfilled into the cave-in.



(a) Plan view

(b) Vertical section

Figure 1. Plan and vertical view of the project site.

2 GEOLOGICAL CONDITION

The site is located in the conflux zone of Han River and Anyang River. The geological strata are composed of river deposits including sands and gravels of medium to dense state. The ground water level is about 7 to 8m below the ground surface, which is similar to the water level of Han river. There exist a depth of fill soil above the river deposit layer, and layers of weathered granite soils or rocks and soft or hard rocks in varing thickness below. The boring data of the vicinity area are recorded in Table 2 and the locations are showns in Figure 1.

Table 2. Ground profiles of vicinity area.

strata	B-6065(16K+622)	B-5-6-11(16K+500)
fill	0 - 1.2 m	0 - 1.3 m
clayey silt	1.2 - 4.9 m	1.3 - 5.8 m
sand and gravel	4.9 - 22.2 m	5.8 - 23.0 m
weathered granite	22.2 - 25.8 m	23.0 - 35.0 m
soft/hard rock	below	below

The crown of the excavated face is about 28m below the ground surface, so the overlying stratum is the weathered granite rock of approximately 5m in thickness. The base rock is a banded biotite gneiss. The gneissoid pattern slants predominantly in southeast direction and partially in north west direction due to the deformation history. A slickenside plane was discovered at the crown sloping 50° opposingly to the tunnel advancing direction. The appearance of this slickenside could not be foreseen beforehand because the plane develops transversely to the tunnel axis.

3 ORIGINAL DESIGN AND CONSTRUCTION

The tunnel section employed in the project is shown in Figure 2. It is one of the typical subway section suggested for the excavation in a weathered granite rock zone. Followings are the construcion details for the support work in sequence.

(1) Upper and lower part of facing are excavated.
(2) The first wire mesh (ϕ 3 x 50 x 50mm) is installed with the first shotcrete(t=5cm).
(3) Rock bolts (l=3m) are installed; 13 for the upper part and 4 for the lower part.
(4) Steel ribs are installed.
(5) The second wire mesh (ϕ 5 x 100 x 100mm) is installed with the second shotcrete(t=10cm).
(6) The third shotcrete(t=5cm) is poured.

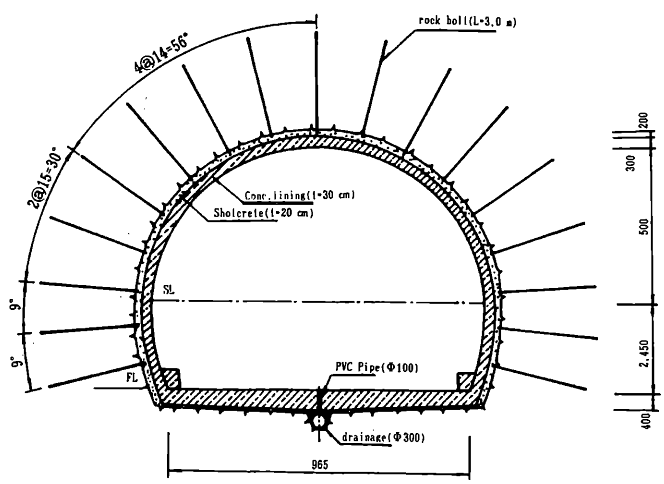


Figure 2. A typical section of the tunnel designed.

4 CAUSES OF COLLAPSE

After the accident additional sets of boring were performed to precisely investigate the geological condition of the site. There found a thin layer of a weathered granite rock approximately 3m in thickness right above the tunnel crown. In the rock strata a slickenside zone with a weak clay seam was discovered sloping 50° to the tunnel axis. Together with this the design and construction data were comprehensively investigated in order to evaluate the possible cause of the collapse. The major possibility considered are concerned with the unfavorable geological structure and the vibration impact due to the blasting.

The influence of blasting impact was evaluated according to Figure 3 showing the relationship between the peak particle velocity and the distance from the shot hole. The peak particle velocity was calculated based on the following equation.

$$V = K\, W^{0.5}\, D^{-1.5} \tag{1}$$

where, V = peak particle velocity in cm/sec
K = site factor
W = weight of explosive in kg
D = distance from the shot to the dwelling in meter

The site factor K depends on the rock condition, explosive type and blasting condition. The rock considered is the weathered granite, and the explosive is a slurry type. Two types of blasting condition were considered at the roof holes and the cut holes respectively. Values of the charge weight per delay and the site factor for two types are listed in Table 3 together with the distance D considered for the analysis. The distance D considered for the analysis is approximately from the shot hole to the center of the rock strata.

Table 3. Values used for parameters for blasting types.

Type	W	K	D
Cut-holes	2.362 kg/delay	42.0	4.5m
Roof-holes	1.575 kg/delay	18.0	2.5m

With these values used, the peak particle velocity was calculated for the two cases; V = 5.71cm/sec for the cut-holes and V= 6.76cm/sec for the roof holes. According to the standard employed for the subway construction, the calculated level is in the relatively safe range. Furthermore, the field record shows that they reduced the charge weight in consideration of the poor rock quality to about 60% of the originally designed value, and consequently the velocity would have been reduced to

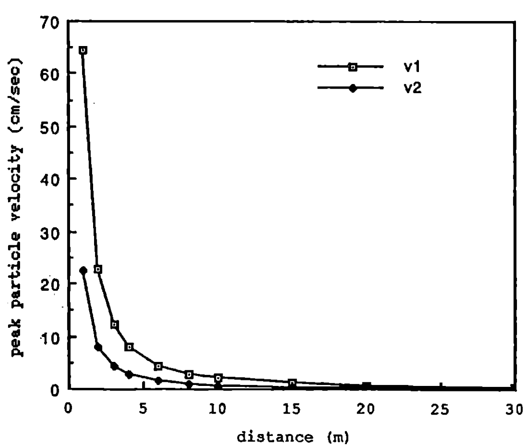


Figure 3. The peak particale velocity and the travel distance.

approximately 77% of the above calculation. From these calculation data, it can be conclusively judged that the blasting impact would not have caused any significant damage to the upper rock strata. However, this can not completely exclude the possibility of the local damage of the rock strata right above the tunnel crown.

Next, a stability evaluation was performed with regard to the geological structure of the site. The safety fator at the failure plane was calculated in consideration of the force equilibrium as follows:

$$F_s = \frac{(W \cos \Psi_P - U) \tan \phi + C}{W \sin \Psi_P} \qquad (2)$$

where, F_s = factor of safety
W = the surcharge force
Ψ_P = the inclination angle between the tunnel axis and the failure plane
U = water pressure
ϕ = the internal friction angle at the failure plane
C = the cohesive force at the failure plane

The average depth of the failure plane was 26m and the ground water depth was 18m. The friction angle of the slickenside was assumed to be 24°. The cohesive force was neglected for the calculation in consideration of the worst condition of geology and the blasting vibration.

Based on the above approach, the calculated safety factor using Eq(2) is 0.44, which is far below 1.0. It is true that the previous assumptions includes some limitations. The actual failure surface is in three dimensional shape. There should exist some resisting force to be mobilized in the upper soil layer. And the neglect of the cohesion in the failure plane might be conservative. If these fators would have been adequately included in the calculation, the safety factor could be more or less increased. However, in consideration of the fact that this type of failure develops from the bottom and progressively to the upper layer, the rock formation at the site is of complex nature, and the blasting impact caused the slickenside more weakened, the calculation approach can be regarded moderately reasonable for evaluating the stability of the tunnel face at the incident moment.

5 DESIGN AND CONSTRUCTION OF REMEDIAL WORK

The collapsed tunnel facing and vicinity grounds are to be sufficiently solidified and water tightened via grouting in order to maintain the stability condition during re-excavation. Large cavity and small voids in the cave-in zone are to be filled with the grout so as to restrain the possible ground settlement during re-excavation and to minimize the shear stress and water pressure to be imposed on the tunnel roof. The adjacent buildings are to be prevented from any damages possibly caused due to the ground subsidence. Other factors considered for the decision was the construction easiness, the construction period and the construction cost.

A remedial treatment has been proposed to meet the above technical requirements out of 4 alternatives. Figure 4 shows the design details of the method. The basic idea is that the cave-in and vicinity are to be filled with cement mortar(Zone A) with the surroundings(Zone B) solidified via cement milk, and the facing and neighboring area(Zone C) are solidified via chemical grouting. The construction details for three zones are as follow:

(1) for Zone A: The injection of cement mortar was performed at intervals of 1.5m in general and 1.0m for the center of the cave-in. The mixing ratio was such that cement : sand : additives = 240kg : 150kg : 1.45kg. The total volume injected was 826.8m³ for 53 holes. The grouting pressure was P_{max} = 18 to 25kg/cm².

(2) for Zone B: The injection interval of cement milk was 1.5m in longitudinal and transverse directions. The mixing ratio was such that cement : water = 301.5kg : 905.0kg. The total volume injected was 1028.5m³ for 166 holes. The grouting pressure was P_{max} = 15 to 25kg/cm².

Adjacent to the existing buildings, a chemical grouting was performed supplementarily to mitigate the ground settlement. The total volume injected was 471m³ for 144 holes. The grouting

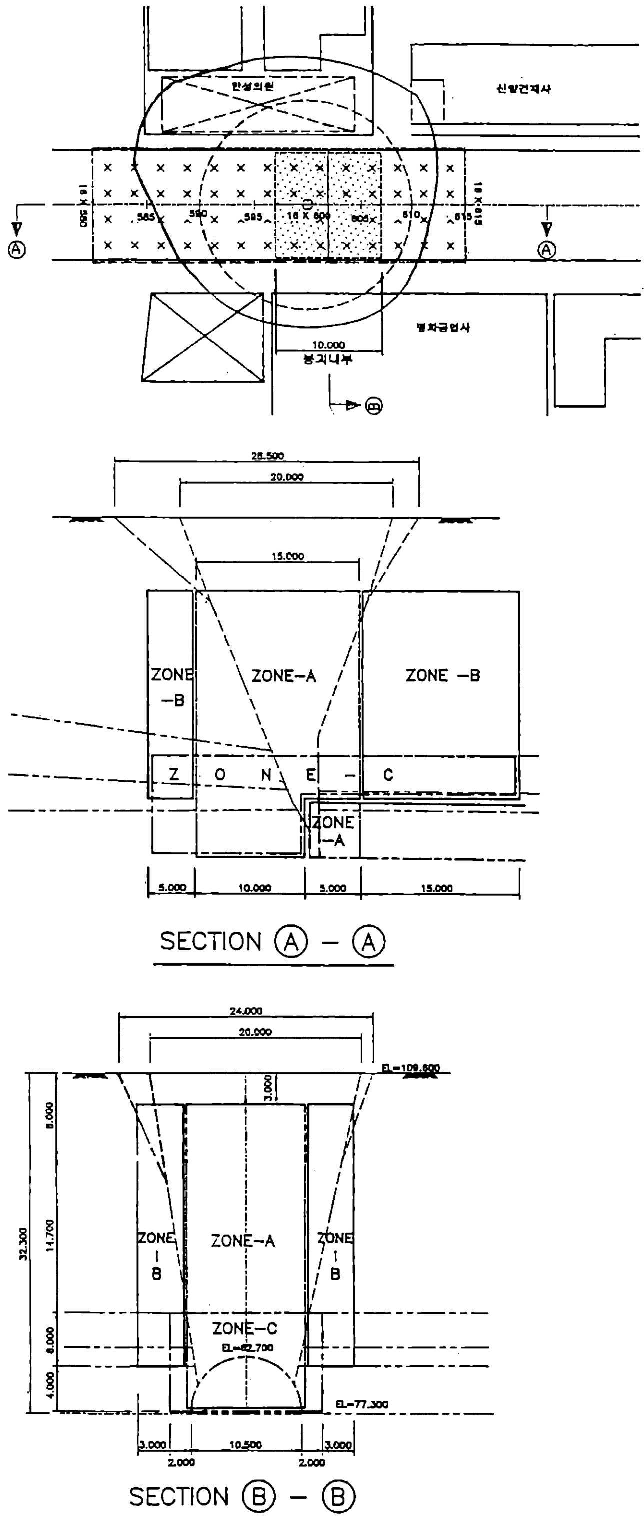


Figure 4. Design details of the remedial treatment plan.

pressure was P = 5 to $10 kg/cm^2$.

(3) for Zone C: A chemical grouting was injected for this region. The grout was composed of two solutions: A solution of silicate and water(mixing ratio = 250kg : 250kg) and B solution of cement and water(mixing ratio = 250kg : 336kg). Total volume grouted was $546.4 m^3$ for 117 holes. The grouting pressure was P_{max} = $20 kg/cm^2$.

6 RE-EXCAVATION WORK WITH MONITORING

The evacuation of the water and soils in the tunnel was performed from the vertical shaft. The water had been pumped during 8 days, and the soils had been removed during 9 weeks. During the excavation the buried pay loader and other equipments were found. The dump truck collected is the one fallen from the ground surface. The detailed work process for re-excavation is shown in Table 4.

Table 4. Work process for re-excavation.

Time	Works Processed
91.11.27	Cave-In collapse broke out.
92.08.11~12.22	Grouting was performed forremedial treatment.
92.11.03~11.11	Pumping of the water was performed.
92.11.21	Removing the dumped soils was started.
93.01.13	Boring tests were performed at the site.
93.01.29	The excavation work was stopped at 16K+608.
93.02.14	Reinforcement work was done for the face.

The site investigation was performed to check the effectiveness of the grouting treatment in the cave-in area. Table 5 shows the N values determined at the cave-in site for the original ground, dumped fill and the treated fill after grouting. The data indicates the ground of the collapsed zone gained significant stiffness with N values mostly greater than 40. The measured permeability for the treated ground was in the range of $1.0x10^{-4}$ $\sim 2.0x10^{-5}$cm/sec.

Table 5. N values determined at the cave-in site.

depth(m)	original ground	dumped fill	treated fill
0 - 5	22 - 27	5 - 30	28 - 34
5 - 10	8 - 10	7 - 50	41 - 50
10 - 15	>50	18 - 50	43 - 50
15 - 20	>50	18 - 50	38 - 50
20 - 25	>50	18 - 50	>50
25 - 30	>50	>50	>50

Figure 5 shows the ground water level changed from the moment of the incident including the pumping record. The curve indicates it took about one month until the water level reached to about -10.0m. And it began to rise as the grouting work started and increased about 0.8m. This trend of rise is also shown in the curve of slime level. This indicates the grouting slime was accumulated in the tunnel to raise the ground water level. The pumping work was continued for 8 days to lower the ground water level about in 2.0m per day. During the pumping, there was no drop of water head detected in the grouted zone.

Next, the excavation work started. Any settlements did not occur at the ground surface and the deformation of the tunnel was about 2mm at most until the excavation stopped at 16K+608, 6m away from the collapse location. The shotcreted face began to deform significantly with cracks shown when drilling was executed into the face. The drilling was done for grouting the face vicinity using a crawler driller with air blown in. The forward thrust of the face was measured to be about 2.0cm. The cracks and deformations took place when the ground water spouted from the drilled hole. The drilling work was stopped, and the face was reinforced with the wire mesh and shotcrete of approximately 50cm in thickness.

The major potential cause of the thrust would be the local

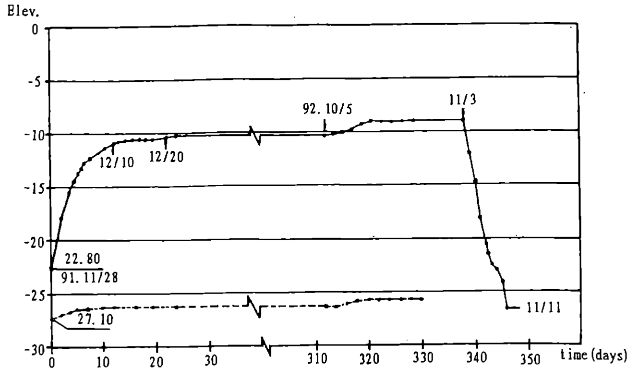


Figure 5. Levels of the ground water and the slime.

instability of the treated zone. A large scaled instability can not also be eliminated. As a counter measures the pipe roof method is being under consideration to take the role of a bridge sustaining the overburden of the potential wedge. The pipe roof would be used as a temporary support for excavating the soils in the tunnel. The ground above the pipe roof should be reinforced via grouting before removing the support.

7 CONCLUSIONS

A case study for a cave-in collapse of a subway tunnel has been reported. The stability condition of the collapsed tunnel face was evaluated with regard to the geological structure and the blasting impact. The remedial treatment employed for the project was introduced with a discussion on how effectively performed at the site.

The excavation of the soils in the tunnel could not be continued from 6m before the collaps location because the excavation face experienced a significant deformation with cracks shown. The excavation work was stopped and the face was reinforced for the following construction. The pipe roof method has been under consideration for a counter measures as a temporary support for excavating the soils in the tunnel. Special considerations should be paid so as to maintain the stability to sustain the overbunden during the excavation.

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