GEOTECHNICAL SOLUTIONS FOR UNSTABLE ROCK MASS AT SG. BULOH AREA

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ABSTRACT

This paper presents geotechnical solutions for unstable rock mass at two slopes within a project site at Sg. Buloh area, Kuala Lumpur. The studied area is underlain by granite formation. Rock blasting works had been carried out on the sites, leaving the rock mass with joints, fractures and discontinuities. At both slopes, the rock mass had been subjected to severe blast damage during earthwork operation, raising some concerns on the stability of the blocky and fragmented rock mass. Rock bolts and rock dowels were used to strengthen the rock slopes. One of the rock slopes has a water tank with storage capacity of 8000m^3 located atop of the slope. The problem was aggravated by the overblasting, which encroached into the 6m setback boundary around the water tank. Furthermore, there were proposed car parks and access road immediately next to the toe of the rock slope. An anchored reinforced concrete wall of 10m height was constructed to provide the boundary setback for the water tank. Apart from these, this paper also presents some interesting features of the rock mass during construction after subjecting to blast damage. Rock mapping had been carried out at the rock slope, studying the possibilities of various failure modes for strengthening design. Some highlights on the problems encountered during construction and supervision of the strengthening works are also discussed.

Keywords: Rock slope; Rock bolts; Rock dowels; Soil nails; Shotcrete; Drainage

1. INTRODUCTION

Rock blasting works had been carried out for the rock slopes in a residential development area around Sg. Buloh. Due to the overblasting in a probably uncontrolled manner, the overall stability of the blasted rock slopes become a concern. The first rock slope (Slope A & Slope B) is located at the upper platform, with levels ranging approximately from RL100m to RL106m. The second rock slope (Slope C) is located at the lower platform, with levels ranging from about RL88m to RL100m. Due to the steep slope gradient, a comprehensive geotechnical assessment was therefore carried out to assess the stability of the slopes and to propose possible slope strengthening solutions.

2. GENERAL GEOLOGY OF SITE

The area of study is of granite formation. Figure 1 shows the site location in general geological map, published by the Minerals and Geoscience Department (1976). All the rock slopes on site generally consists of

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of strong, slightly weathered, medium-grained granite.

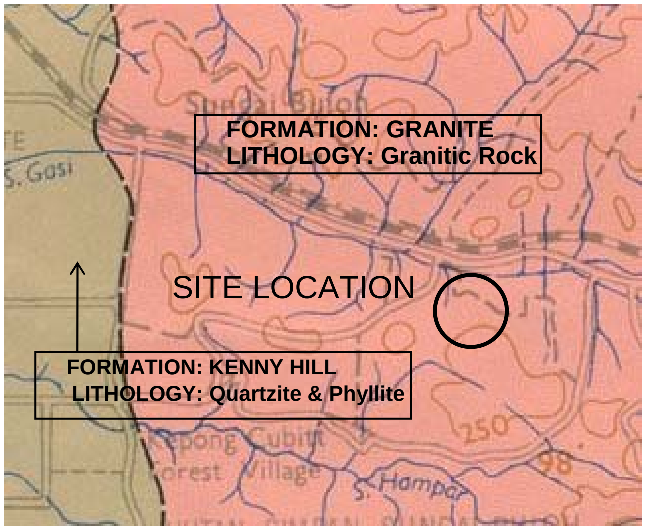

Figure 1: General Geological Map

Figure 2: Unstable Rock Mass at Slope A and Slope B

3. STABILITY OF SLOPE A AND SLOPE B

Slopes A and B are located adjacent to a structure, which was already under construction at that time. Slope A and Slope B, which were formed at the edge of the building platform, were badly damaged by blasting works. A layer of soil is then placed over the overblasted bedrock platform for utilities laying purpose. Figure 2 shows some of the unstable rock mass at Slopes A and B.

The geological mapping using “Line Mapping” method was carried out by recording any discontinuities intersecting a scanline along the slope face at 5m spacing. Photographs of the slope face were also taken during the mapping for stability assessment and planning of the strengthening works.

Kinematic analysis on the geological mapping was performed using rock engineering software (Watts et al., 2003). The kinematic analyses for wedge failure and toppling failure are shown in Figure 3. It is generally summarised that toppling failure for the slopes are more probable compared to wedge failure, particularly on Slope A. For Slope B, some isolated blocky rocks require localised strengthening for stability.

3.1 Strengthening Works

Strengthening works were carried out at certain identified areas for Slopes A and B. Rock bolts and rock dowels had been used to strengthen the unstable block mass. Horizontal subsoil drains were also used to drain away the water in the rock mass. The details of the strengthening works are summarised in Table 1.

Scaling and removal of debris and rock fragments were carried out prior to the strengthening works. This is to expose the rock face to show other secondary geological features for detailed assessment of the stability. A preliminary pull out test for rock bolt was performed in accordance to BS8081:1989. The result of the pull out test is shown in Figure 4. The displacement measured at the maximum test load 465kN was 4.5mm confirming that the constructed rock bolt can perform satisfactorily.

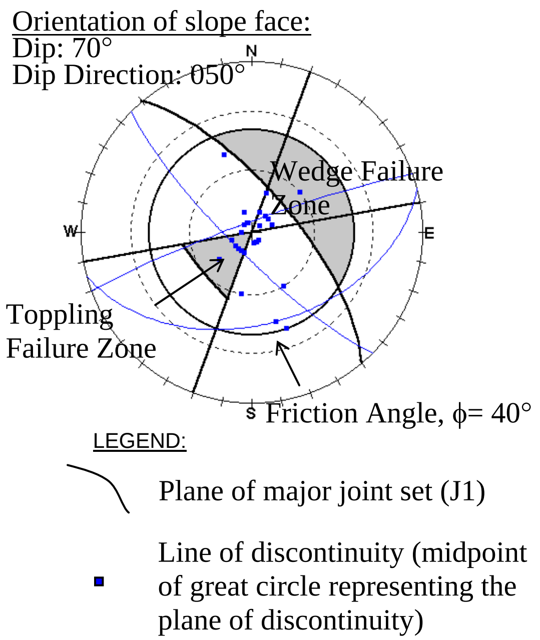

Figure 3: Stereonet Plot for Slope A

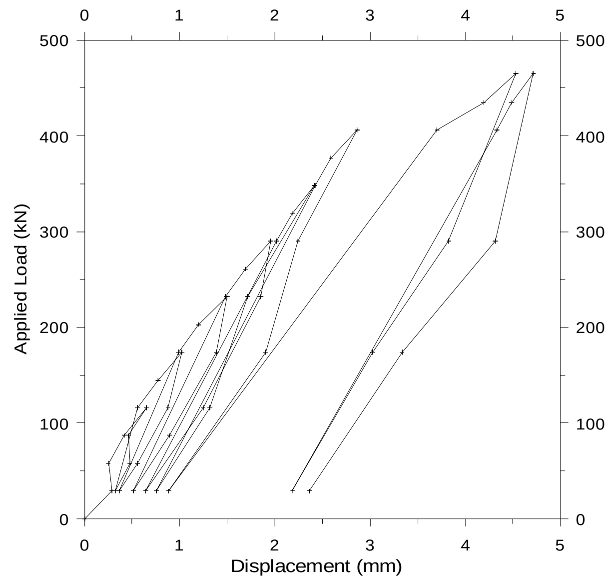

Figure 4: Pull Out Test Results of Rock Bolt

Table 1: Details of Strengthening Works at Slopes A and B

Strengthening Works	Details
Permanent Rock Bolt (Working Load 200kN)	ϕ 80mm diameter hole drill Galvanised high yield steel bar of 40mm diameter. Free Length = 1m, Bond/Fixed Length = 3m
Permanent Rock Dowel	ϕ 80mm diameter hole drill Galvanised high yield steel bar of 40mm diameter. Dowel length = 3m

Corrugated high density polyethylene (HDPE) pipe is used for the pre-grouted segment of the bond/fixed length of the rock bolt. Its main purpose is to provide the primary corrosion protection for the galvanized steel bar, in which the grout and the galvanized layer will form the secondary corrosion protection layer. The steel bar was pre-grouted in corrugated pipe. It was found that pre-grouting in a horizontal configuration was not effective due to the voids found trapped within the space between the corrugated pipe and the steel bar. Therefore, it was later decided to pre-grout the pipe in a vertical arrangement. Figure 5 shows vertical grouting of the corrugated pipe. During grouting the pipe from bottom up, constantly vibrating the HDPE pipe had been carried out to prevent trapping of air voids.

BS8081:1989 demands that shrinkage of cementitious grout should preferably be less than 5%. Otherwise, debonding on the reinforcement can occur, which creates a potential corrosion hazard for the steel bar and also possibly jeopardise the pull out performance of the rock bolt. Therefore, non-shrink grout had been used by adding expansive mixtures to ordinary Portland cement for pre-grouting and grouting up the holes of both rock bolts and rock dowels.

3.2 Construction Problems

Several problems were encountered during the implementation of strengthening works. It was found that drilling of holes in highly fractured rock was difficult due to the frequent collapse of drill hole and loss of flushing air pressure in the fracture. Therefore, it is almost impossible to advance the drill hole in the fractured rocks. Finally, all fracture rock pieces were removed using either labour or machinery.

Figure 5: Vertical Grouting of Corrugated Pipe

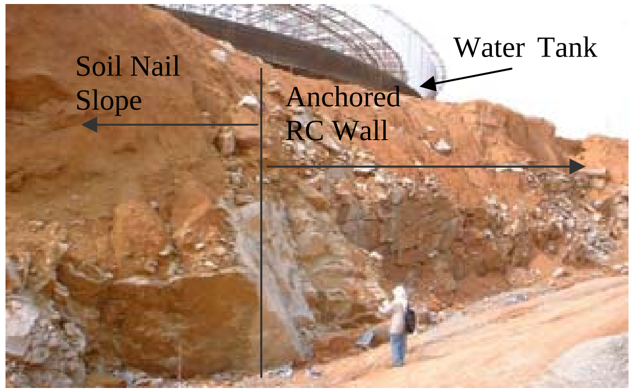

Figure 6: Slope C Before Strengthening Works

Another problem encountered was excessive grout loss. The natural fractures, joints, fissures existed within the rock mass were further opened up during uncontrolled rock blasting, the volume of grout loss was significant. Some of such cases can be overcome by stage grouting, in which the previous stage of grouting will narrow down the joint openings and the flow path of the subsequent grouting operation.

It was mentioned earlier that a layer of topsoil was laid on the platform. During the ongoing of construction, the soil slope was eroded and gullies started to form. To avoid further erosion, the soil slope on top of Slope A was covered with shotcrete. The soil slope on Slope B, facing the main access road was also later turfed to avoid further erosion.

4. STABILITY OF SLOPE C

Slope C is located adjacent to a water tank. The top 3m of the slope is backfilled with granitic residual soil, while the lower portion of the slope is granitic rock with varying degrees of weathering.

Geological mapping was carried out along vertical face of the rock slope. From site observation, the rock slope surface has very poor fragmentation without any smooth face. No traces of the pre-splitting lines were observed. Figure 6 shows the condition of Slope C before the rectification works. The visual inspection suggested that falling of loose and fragmented rocks from the slope was highly probable.

4.1 Subsoil Condition

Three boreholes, namely BH-7, BH-7A and BH-7B were respectively sunk at the designated location and the offset locations adjacent to the water tank. Figure 7 shows the designated location of the borehole. The summarised results of the boreholes are shown in Table 2.

4.2 Anchored RC Wall and Soil Nailed Slope

The exposed rock slope face had been overblasted to about 4m away from the nearest outer perimeter of the water tank. This setback was not sufficient as the authority required a 6m setback around the water tank for maintenance access. Furthermore, there were proposed car parks just adjacent to the boundary line at the lower platform. (See Figure 7). However, the toe of the slope at that time had also encroached into the proposed car park. Thus, a retaining structure was required to retain the 6m perimeter road and, at the same time to provide the necessary space for the car parks.

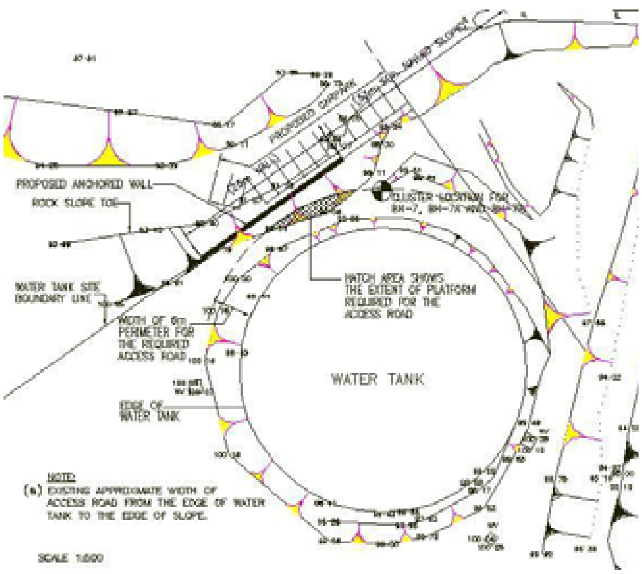

Figure 7: Layout Plan of Slope C Showing Borehole, RC Wall and Soil Nailed Slope Location

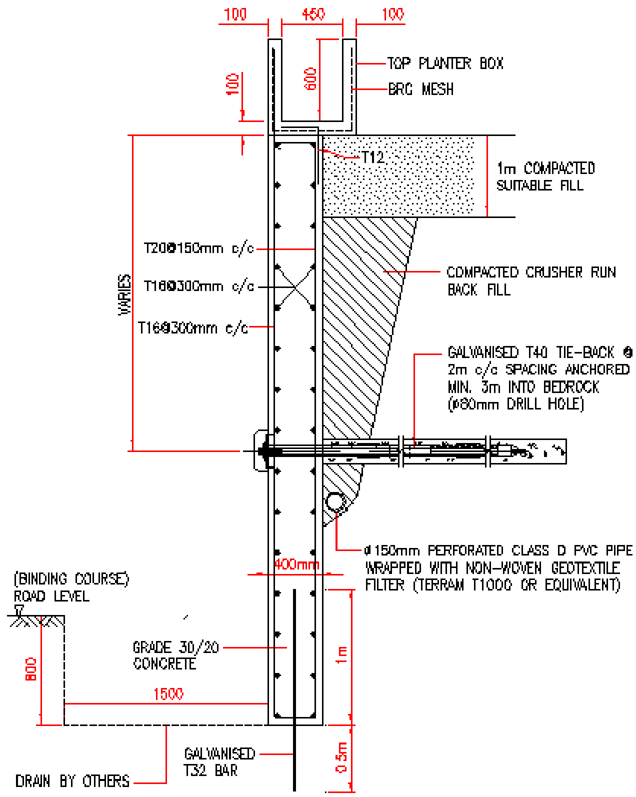

Figure 8: Schematic Diagram of Anchored RC Wall

Table 2: Summary of Boreholes

Borehole	Soil Description	SPT-N (Below ground level)
BH-7	Medium dense, silty sand.	10 (1.5m – 1.95m)
BH-7A	Medium dense to very dense, silty sand.	13 (1.5m –1.95m) >50 (4.5m – 4.84m)
BH-7B	Medium dense to very dense, silty sand.	13 (1.5m –1.95m) >50 (4.5m– 4.88m)

The proposed retaining system for Slope C was a reinforced concrete anchored wall of about 10m height. The original design of the reinforced concrete wall was to socket the wall base into the bedrock and anchored by a series of tieback anchors at certain levels. The Finite Element Method (FEM) was employed for the soil-structure interaction analysis using computer program, PLAXIS. A ϕ150mm perforated subsoil pipe wrapped with geotextile had also been provided to drain away potential groundwater seepage in the rock mass. The schematic RC anchored wall details can be referred in Figure 8.

Immediately adjacent to the rock slope as mentioned above was a residual soil slope with varying degrees of weathering. As the soil slope of 8m height had also encroached into the proposed car park, the soil slope needed to be trimmed back and strengthened. Soil nails of 12m length had been proposed to strengthen the soil slope after trimming it to a gradient of 4V:1H. The vertical spacing and horizontal spacing of the soil nails was 1.6m and 1.4m respectively. The elevation view of soil nailed slope before and after strengthening works is shown in Figure 9 and Figure 10.

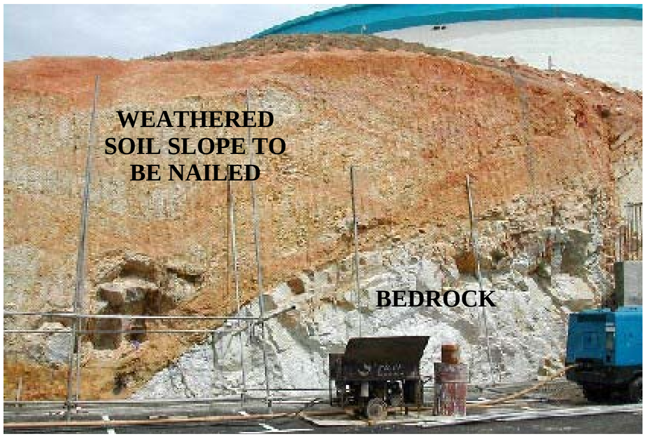

Figure 9: Slope Before Strengthening Works

Figure 10: Completed Soil Nailed Slope

4.3 Construction Sequence

The construction sequence as planned involved rock hacking to trim the rockhead encroaching into the car park boundary. Rock slope trimming was required from the existing slope toe to the boundary line as shown in Figure 7. A 0.5m deep trench had initially designed to socket the wall base on to the rock. The tieback anchors were then installed at required levels. The RC wall was later constructed to next level, where another tieback anchor will be installed, until reaching the platform level of the water tank. Well graded crusher run had been specified as the backfill material of the RC wall due to its effective drainage and ease of compaction in the narrow backfilling space.

The soil nailing works were carried out concurrently with the construction of anchor RC wall. Prior to the soil nailing, the soil slope was trimmed to the designed gradient of 4V:1H. The soil nails and nail heads were then installed, followed by shotcreting to protect the slope from surface erosion. Sufficient weephole drains were provided to prevent building up of pore pressure behind the shotcrete.

4.4 Construction Matters/Problems

The construction works started with rock hacking and clearing of debris material left from the previous earthwork. Initially, a 200 series pneumatic hacker was mobilised, but later replaced with a 300 series pneumatic hacker to speed up the hacking works. However, when solid rock was encountered, the 300 tonnes hacker encountered difficulties in the rock hacking. A driller rig was later mobilised to pre-drill and fracture the rock before hacking was performed.

Due to the difficulties encountered and poor efficiency as expected in rock hacking, the 0.5m socket trench for the RC wall was brought to review. In the design review, alternative dowel bar system was used to replace the socket trench and provide the shear resistance to the wall base. The dowel bar system includes drilling and installation of a series of ϕ32mm high yield steel bar at 1m intervals along the toe of the wall. The dowel bar was installed into the ϕ40mm pre-drilled hole and grouted with high strength epoxy grout.

During the construction of the RC wall, the contractor proposed to leave a ϕ150mm PVC pipe in the wall to facilitate the subsequent installation of anchors in which its progress was slower than the RC works. However, it was later discovered that the ϕ150mm hole was too large and would cause high bending moment at the steel bearing plate when the anchors are in the working condition. An analysis was carried out using FEM to assess the bending moment induced in this case. It was later decided to weld an additional steel plate (thickness 25mm) to the steel bearing plate (250mm x 250mm x 25mm) to improve the bending capacity of the bearing plate. The revised anchor head details is shown in Figure 11. The subsequent level of anchors were then installed before casting the next segment of the RC Wall.

Backfilling for the anchored RC wall was carried out in a few stages. The first backfill of crusher run was performed after the first levels of anchors had been installed. Due to the narrow space behind the wall, compaction could be properly carried out by spraying water while spreading the crusher run to achieve self-compaction. The subsequent crusher run backfill was carried out with vibratory compaction as the RC wall progressed to its full height. A layer of suitable cohesive fill (1m thick) was laid on top of the crusher run backfill to avoid direct ingression of surface runoff into the backfill of the wall which will potentially increase the water pressure behind the wall.

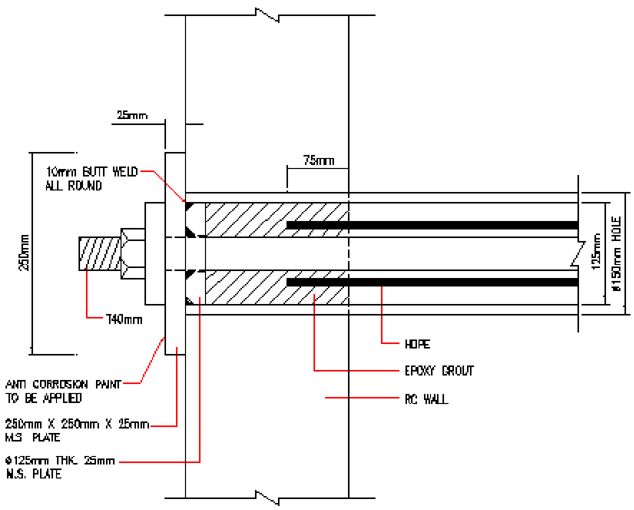

Figure 11: Anchor Head Details

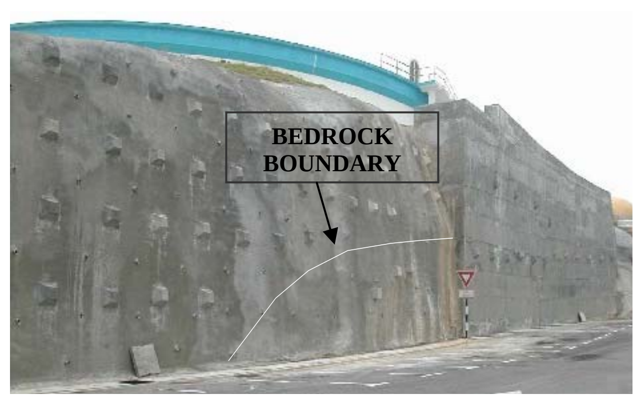

Figure 12: Front View of Completed Anchored Wall and Soil Nailed Slope

The soil nailed slope surface was covered with shotcrete after the installation of soil nails. Weepholes were provided at a regular interval to facilitate the drainage of groundwater particular along the soil and rock interface. Horizontal drains were also installed at the toe of the slope. Figure 12 shows the completed view of anchored RC wall and soil nailed slope.

4.5 Groundwater Discharge Monitoring

It was observed that after the installation of the subsoil pipe drain behind the anchored RC wall, groundwater started to discharge from the pipe even before the construction of RC wall was completed. Water discharged from weepholes on the shotcrete surface adjacent to the RC wall was also observed.

The construction of the RC wall and soil nailed slope was completed at the end of October 2002. It was observed that rain was frequent and heavy for the month of November. A site visit was specifically arranged on November a day after a heavy downpour. It was found that the discharged water from the subsoil pipe, horizontal drain and weepholes was of substantial amount as shown in Figure 13 and Figure 14. Water was also seen seeping out from the rock slope near by. It could therefore be conceded that groundwater perched up and flowed over the bedrock surface when the groundwater table built up.

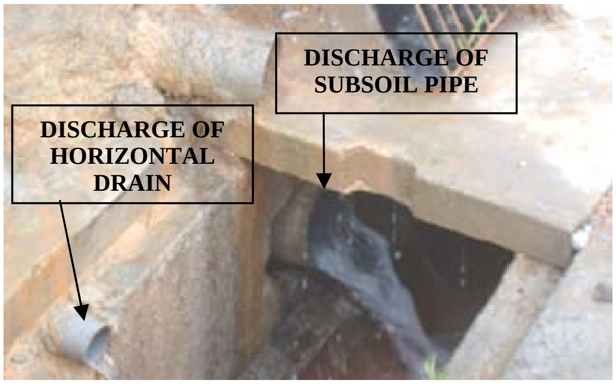

Figure 13: Discharge of Weepholes After Heavy Downpour

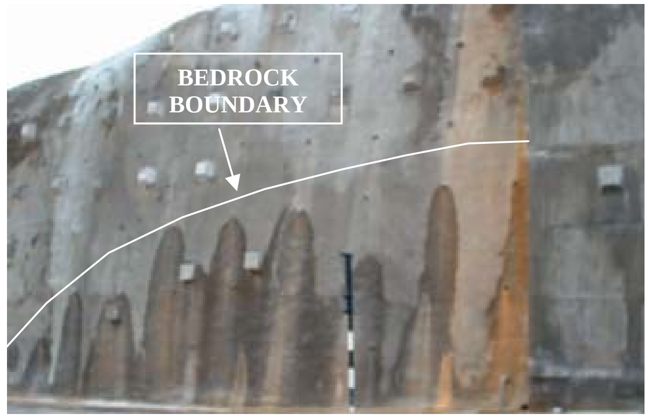

Figure 14: Discharge of Weepholes After Heavy Downpour

5. CONCLUSION

The lessons learnt from this case history can be summarised as follows:

a. In a rock blasting operation for development project, due considerations shall be taken to prevent excessive blasting which may cause fractures or fragmentation to the finished rock mass.
b. Geological mapping is very important in strengthening design of rock slopes. Economical and effective design can be achieved by identifying the mode of instability and then applying appropriate strengthening treatment.
c. During pre-grouting of the rock bolts, it is important to ensure that no air void is trapped in the corrugated pipe and sufficient vibration shall be applied to the vertically arranged grout column.
d. Where rock fractures are created by the uncontrolled blasting, care shall be taken in the hydro-geology aspect, in which infiltrated surface water will tend to flow in the rock fractures and discharge at the lowest fractures.
e. For shotcrete surface, it is crucial to provide sufficient weephole drains to relieve pore pressure built up underneath the shotcrete surface.
f. Proper and sufficient drainage shall be provided at the soil/rock interface, where groundwater tend to perch and flow along the bedrock interface. If this drainage is not properly control, the outcome can be catastrophic.

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