Hydrogeological Studies of the Slope Failure at Tuen Mun Highway Chainage 550, Hong Kong

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Abstract

A study is being conducted to understand delayed slope failures caused by hydrogeological regimes and the preliminary findings from this study are summarised in this paper. The slope near Tuen Mun Highway Chainage 550 in Hong Kong which failed about 13 days after a major rainstorm in 1983 was chosen as a case study. Topographical, geological and hydrogeological conditions at the hillside above the failure area are investigated to explore delayed groundwater response and the timing of the landslide. MODFLOW is used to simulate groundwater flow in response to heavy rainfall. The slope comprises igneous rock of different degrees of weathering and is represented by layers of various permeabilities. The hydrogeological model can reproduce reasonably well the delayed groundwater response and the stable hydraulic head behind the slope.

1. Introduction

While most landslides in Hong Kong occur during or immediately after rainstorms, some failures exhibit a delayed response. Delayed failure is an important area of research because risk levels associated with such failures are relatively high, delayed failures being unexpected. A study is being conducted to understand delayed slope failures caused by hydrogeological regimes. The slope near Tuen Mun Highway Chainage 550 in Hong Kong (Figure 1) was chosen as a case study. This 70m high cut slope failed in September 1983. The failure was classified as a deep-seated failure, the rupture surface being 13m deep at maximum, measured normal to the 33° slope surface. The geology of the failed area and a discussion on the possible causes of the failure were presented in the GCO investigation report (GCO, 1984), which provides the main source of information on geology and groundwater for this study. According to the investigation report, there were several aspects of this failure which were puzzling:

(i) A rainstorm occurred on September 9, brought by Typhoon Ellen, but major movements were only noted on September 22 (Figure 2). It was speculated that the failure might be related to Typhoon Ellen, but the mechanisms causing the delayed failure were not clear. Piezometers were installed and monitored daily over several months following the failure.

(ii) The observed piezometer response showed that the main groundwater table in the slope was high and stable and exhibited very little storm response. The cause of the high and stable groundwater level was not known.

(iii) The slope had moved in 1975 during construction and was cut back to 33°, but it failed 8 years later. This suggests that cutting back had not improved its stability to the extent expected. The reason for this was said to warrant further discussion (GCO, 1984).

The aim of this study is to offer some explanations of the failure mechanisms from the perspective of hydrogeology. Hydrogeological conditions at the hillside near Pun Shan Tsuen are examined to explore possible delayed groundwater response. A numerical model is set up for simulating groundwater flow in response to heavy rainfall. The simulated groundwater heads are analyzed to understand the different behavior of the hydraulic head response to rainfall in different parts of the slope. The role of regional hydrogeology in the stability of slopes above the Tuen Mun Highway is discussed.

2. Geological and Hydrogeological Conditions

The study area is dominated by coarse tuff of the Repulse Bay Formation. Locally the area is intruded by fine Tai Po granodiorite (Figure 3). The tuff is completely to highly decomposed. Some areas are overlain by a thin layer of colluvium. The granodiorite is relatively less decomposed (GCO, 1984). Although there are no site permeability measurements available, it is speculated that the weathered fine granodiorite may be less permeable than the weathered course tuff. Figure 3 shows that on the eastern and western sides of the slope, granodiorite is at surface. The lower permeability of the two lateral granodiorite bodies may create channelling, forcing groundwater to flow preferentially in the volume of ground between them. The sea level near the failure area represents the lowest groundwater discharge point. Because of the concave shape of the coastline near the failed slope, a groundwater sink may be created. These geological and hydraulic factors may force groundwater to concentrate in the failure area (Figure 3). Figure 4 shows that on the slope surface there is a geological boundary, separating the slope into an upper section within the coarse tuff and a lower section comprising fine grained granodiorite. The relatively lower permeability of granodiorite forces the groundwater to seep out near the boundary (Figure 5)

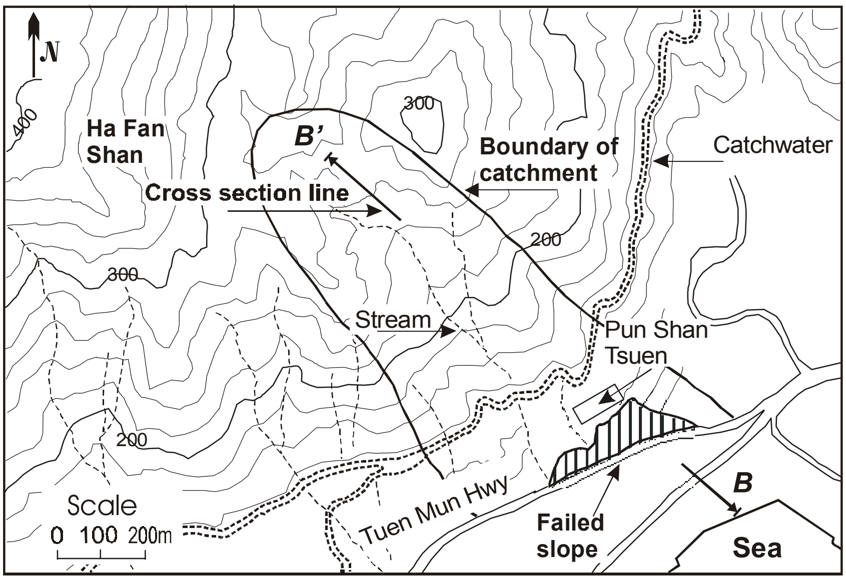

Figure 1 Location of the failure area at Tuen Mun Highway chainage 550 (modified from GCO, 1984)

Figure 1 shows that there is a stream on the western side of the slope. An approximated boundary of the stream system can be located by examining the topography of the area. The catchment area is estimated to be about 0.4 km^2. The elevation of the uppermost boundary of the catchment is about 280mPD. The lowest groundwater discharge point is at sea level, which is about 1mPD. The horizontal distance between the upper boundary of the catchment and the cutting slope is more than 1200m. The topography is believed to have a general control on the overall groundwater flow velocity and patterns.

It appears from Figure 1 that the slope is on a spur and that no streams drain directly towards or across the failed slope. If groundwater catchment and surface water catchment coincide, the stream channel is also the center of the groundwater system. However, the groundwater flow system may not coincide with the surface water flow system. As described in the GCO investigation report, three strips were cut into the slope surface for material inspection (Figure 4). The cross section A-A’ has the deepest bedrock profile. The bedrock gradually becomes shallower towards the two sides of the slope (GCO, 1984). Because of the deeper weathering profile, the center of the groundwater system may shift to the east and the main groundwater flow direction may be right through the center of the cut slope (see Figure 1).

It is likely that the main recharge to the groundwater system is the rainfall in the catchment. There is an unlined water supply tunnel below the slope (Figure 5) and a catchwater above the slope (Figure 1). However, the possible recharge from the two sources was ruled out by the GCO investigation. According to the GCO report

(1984), the Water Supply Department regularly tested their tunnels for leakage and did not detect any water loss from this tunnel. It was also mentioned that the catchwater above the slope was inspected carefully after the failure but no major cracks that could have provided sufficient water to initiate such a large failure were found. The report did not mention the possible leakage from other sources such as the sewage system beneath the Pun Shan Tsuen Village. Although the conclusions made by GCO regarding the leakage from other sources warrant further discussion, only recharge from rainfall is considered in this preliminary study due to lack of information.

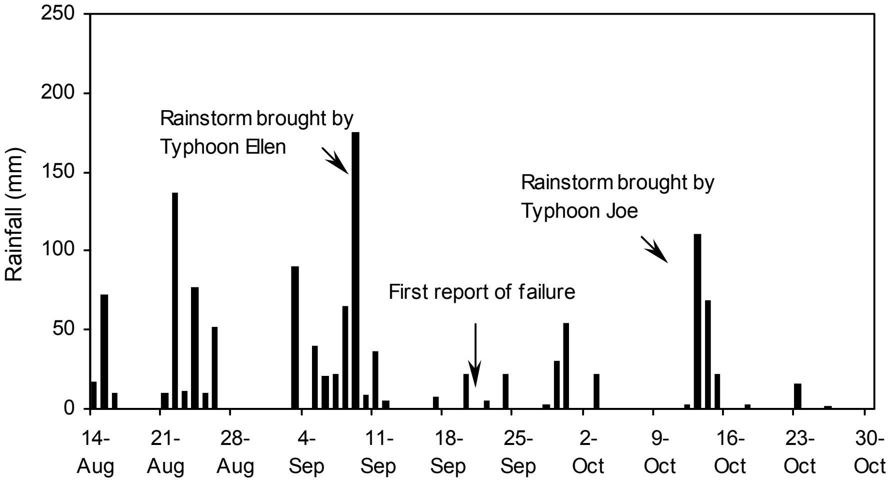

Figure 2 Daily rainfall from August 14 to October 30, 1983

Piezometers were installed and monitored daily over several months following the failure. The results showed that the groundwater in the slope was high and stable and exhibited very little storm response, as demonstrated by the water level change of less than 1 m in response to the rainstorm caused by Typhoon Joe in mid-October, 1983. The groundwater table observed in this period is presented approximately in Figure 5, which shows that the groundwater level at the slope crest was observed to be about 90 m. Beneath the slope, the groundwater table was shallower with seepage occurring at the slope face below the second berm. Several of the existing horizontal drains below the second berm were constantly issuing moderate amounts of water throughout the rainy season. At various locations above the third berm are large surface depressions. Internal erosion pipes were seen in some of the depressions and seepage was observed from these internal erosion pipes during heavy rain.

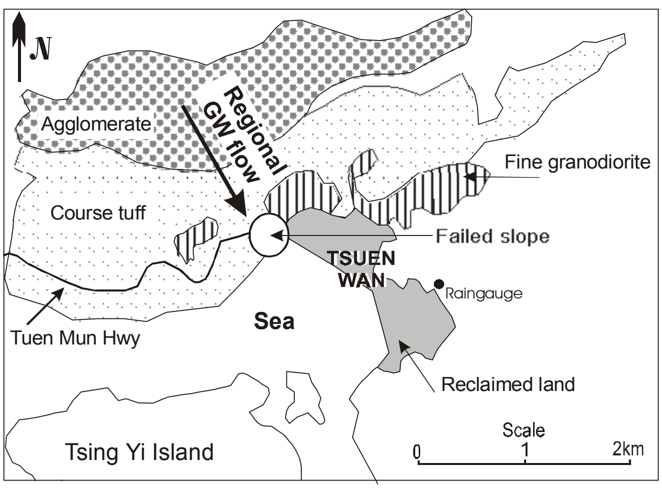

Figure 3 Simplified regional geological map of the study area (modified from GCO, 1984)

3. Numerical Modelling of Groundwater Flow

3.1 Model Configuration and Hydraulic Parameters

A two-dimensional cross sectional model is chosen for this study. The location of the section *B-B'* is chosen to pass approximately through the deepest weathering profile, as shown in Figure 1. The selection of the vertical thickness is important because the flow pattern may be distorted if the aquifer thickness is not properly represented. The aquifer thickness is assumed to be controlled by weathering. As shown in Figure 5, some boreholes at the fifth berm of the slope encountered Grade IV rock at a depth of 35 m and below that the rock is Grade III or less weathered. It can be reasonably believed that the actual thickness of the aquifer is much greater than 35m due to existence of fractures in Grade III rock. In the model, the vertical thickness of the model at the left is 110 m and at the right is 200m, as shown in Figure 6 (Note that Figure 6 has been vertically exaggerated by a factor of 2). The material below that is assumed to be impervious and is represented by an impermeable boundary. In terms of weathering, the profile may be divided into four zones of different degrees of weathering (Table 1). For the purpose of better describing the vertical variation of the groundwater head in the cross section, each zone is divided up to 4 layers and all together 11 model layers are used (Figure 6).

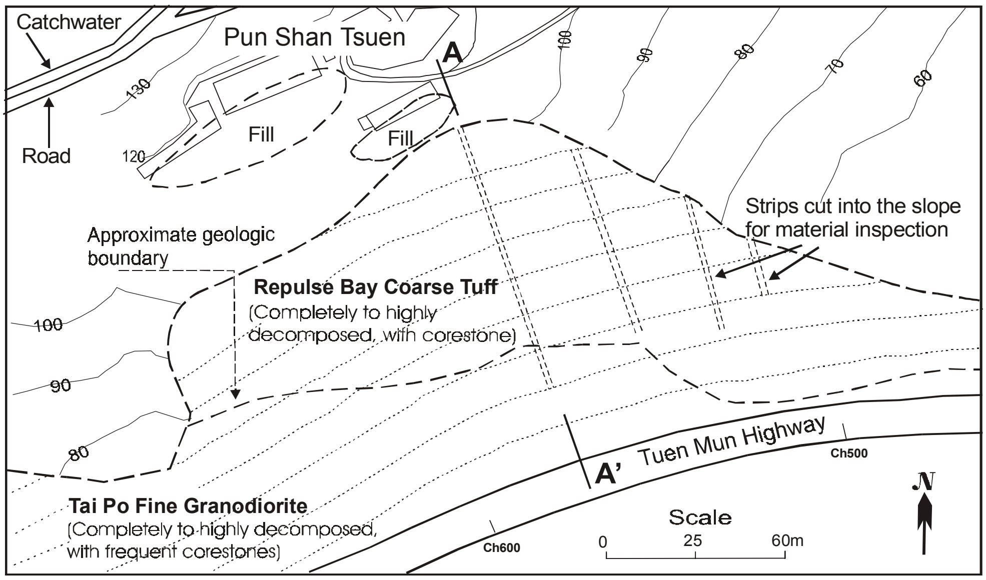

Figure 4 Detailed geological map of the slope (modified from GCO, 1984)

Among the 11 layers, the top layer is always unconfined and the rest are convertible between confined and unconfined, depending on the location of the water table. Specific storage is used to calculate the rate of change in storage if the layer is fully saturated, otherwise, specific yield is used. There is no aquifer parameter information available for this site and all the data are from literature about other similar sites in Hong Kong. The hydraulic conductivity values are based on GEO (1993) and GEO (1996). The specific storage and specific yield values are based on Leach and Herbert (1982). The details of the parameters used in the model are listed in Table 1. Vertical conductivity is taken as 20% of the horizontal conductivity. The model is divided into 122 columns, each column representing a horizontal distance of about 10m. The system is represented by 1342 cells. For this preliminary study, only saturated flow is considered. MODFLOW, a finite-difference code developed by the USGS (McDonald and Harbaugh, 1988), is used for this study.

Table 1 Hydraulic parameters used in the cross sectional model

Zone	Layer	Hydraulic Con. (m/s)	Specific yield	Specific storage
Zone 1, completely decomposed or colluvium	1 & 2	*1.8×10^{-5}	0.01	0.0001
Zone 2, highly decomposed	3 & 4	*1.6×10^{-5}	0.005	
Zone 3, slightly Decomposed	5	5.7×10^{-6}	0.001	
	6 & 7	1.2×10^{-6}		
Zone 4, slightly decomposed to fresh rock	8 & 9	1.2×10^{-7}		
	10 & 11	1.2×10^{-8}		

(*For the lower part of slope, where the superficial material was removed during the construction of Tuen Mun Highway, values are assumed to be the same as the values for layer 5)

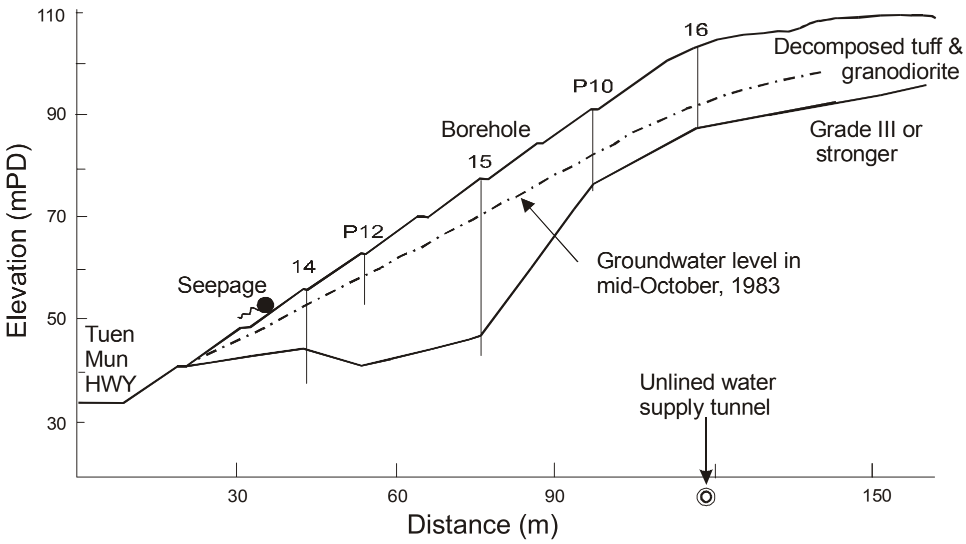

Figure 5 Simplified geological and hydrogeological section through A-A' in Figure 4 (modified from GCO, 1984)

3.2 Boundary Conditions and Recharge

The ridge near Ha Fan Shan (Figure 1) is considered as a groundwater divide and is represented by a no-flow boundary. The sea below Tuen Mun Highway is considered as a constant head boundary. Only recharge from infiltration is included in the model. The ground from the reclamation area near the Tuen Mun Highway to the slope area above the catchwater is covered by roads, buildings, chunam, or shortcrete and may have negligible infiltration (Lerner, 1986). In the model, the recharge from rainfall is added only to the upper part of the slope where the natural ground surface is not covered. The infiltration coefficient is a very important parameter but is usually difficult to estimate. It is believed that the recharge coefficient for water supply purposes should be different from slope stability studies. For water supply, the groundwater resources are considered on a yearly basis and the recharge coefficient includes evaporation and transpiration losses. In a particular case study for water resource evaluation, Rushton and Ward (1979) used 30% of the rainfall as annual groundwater recharge. Groundwater studies related to slope stability, however, concern the short-term response of the groundwater to rainstorms. The evaporation and transpiration losses are usually ignored. The Geotechnical Manual for Slopes (GEO, 1997) suggested that 50% of the rainfall is produced as runoff and 50% as infiltration. In the model, recharge is taken as 50% to the rainfall during the modelling period between August 14 to October 30.

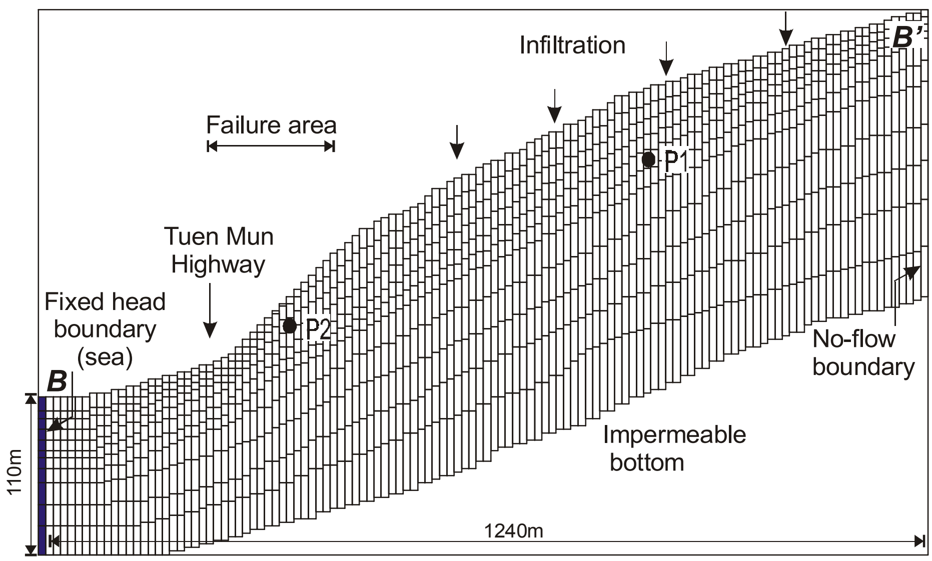

Figure 6 Mesh system and boundary conditions for the cross-sectional numerical model through *B-B'* in Figure 1 (vertical exaggeration is two times of horizontal)

The model is first run for a steady state to provide the initial water level for the transient flow from August to October. In the steady state model, a recharge of 0.003m/day (equivalent to 30% of the average rainfall from May to July) is taken as the recharge. The head values of steady state analysis were then taken as initial hydraulic head for the transient model. A total of 78 days simulation was done in the transient model by taking high recharge (50 % to the rainfall shown in Figure 2).

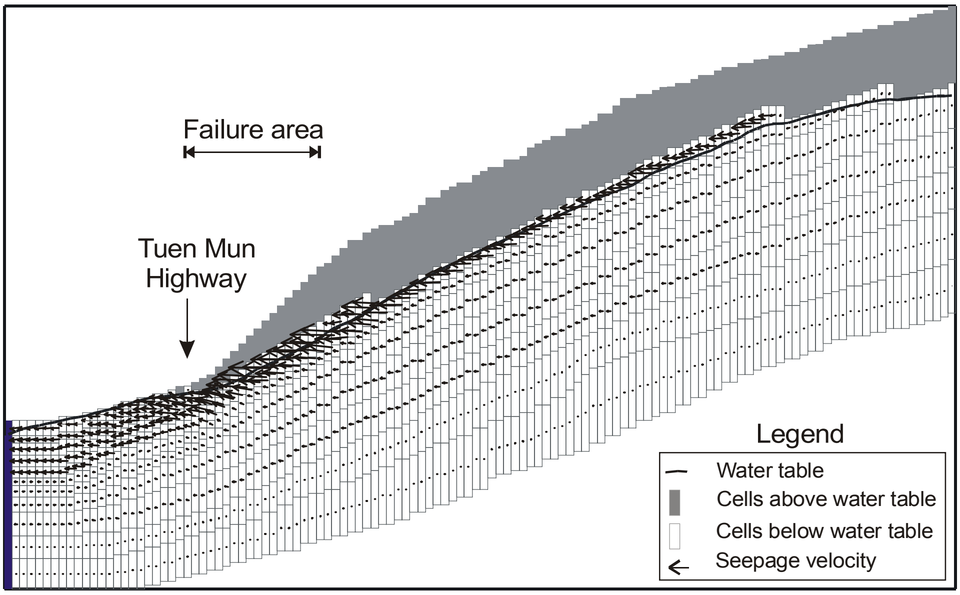

Figure 7 Simulated steady state groundwater table and seepage velocity distributions (vertical exaggeration=2x)

3.3 Modelling Results and Analyses

Figure 7 shows the initial water level and seepage velocity distributions from the steady state model. It represents the average groundwater situation in the summer when there is no rainstorm. It can be seen that the water level is usually below the slope surface and there is seepage discharge from the ground only near the bottom of the slope. Figure 8 shows the transient ground water level and seepage velocity distributions on 22nd September when the major failure occurred. The water level near the Tuen Mun Highway is very high. In the failure area, seepage velocity is the greatest and oriented almost normal to the slope surface in an outward direction, which represents the worst hydraulic conditions at the time of failure. It is interesting to note that, although the thickness used in the model is more than 110m, Figures 7 and 8 show that groundwater is active only in the first 6 layers with permeability greater than 10^{-6} m/s.

The simulated groundwater head at two locations P1 and P2 is selected for discussion. P1 is in the upper part of the slope and P2 at the failure area (see Figure 6). The response of groundwater head to recharge is very different, as shown in Figure 9. It is seen that at the upper slope the hydraulic head shows a quick response to rainfall and therefore fluctuates significantly while the head at the failure area is quite stable and shows a delayed response. As shown in Figure 9, after the heaviest rainfall brought by Typhoon Ellen on 9th September, the hydraulic head behind the failure area reached the highest value of 67m on 15th September and remained almost the same level until around 22nd September. There is no much rain in the period when the groundwater level behind the slope remains high. It is seen that the model can reproduce the stable hydraulic head at the lower part of the slope and has the potential to simulate the delayed effect. Some sensitivity analysis (Jiao and Lerner, 1996) can be conducted to understand influence of parameter uncertainty on the groundwater level and delayed response. This will be a topic for further study.

The location of P2 is roughly corresponding to that of Borehole 15 in Figure 5. The observed hydraulic head in Borehole 15 in mid-October, 1983 was about 70mPD with a variation of less than 1m in response to the rainstorm brought by Typhoon Joe. The corresponding simulated head is about 62mPD and shows a rise of more than 2m in response to the rainstorm, although it is delayed by about 5 days, as can been seen in Figure 9. It appears that the observed head was even higher and more stable than the simulated one. This may imply that, in addition to rainfall infiltration, the groundwater system has some other recharge sources. A more definitive conclusion requires detailed studies on the water-carrying services and further hydrogeological investigation in this area.

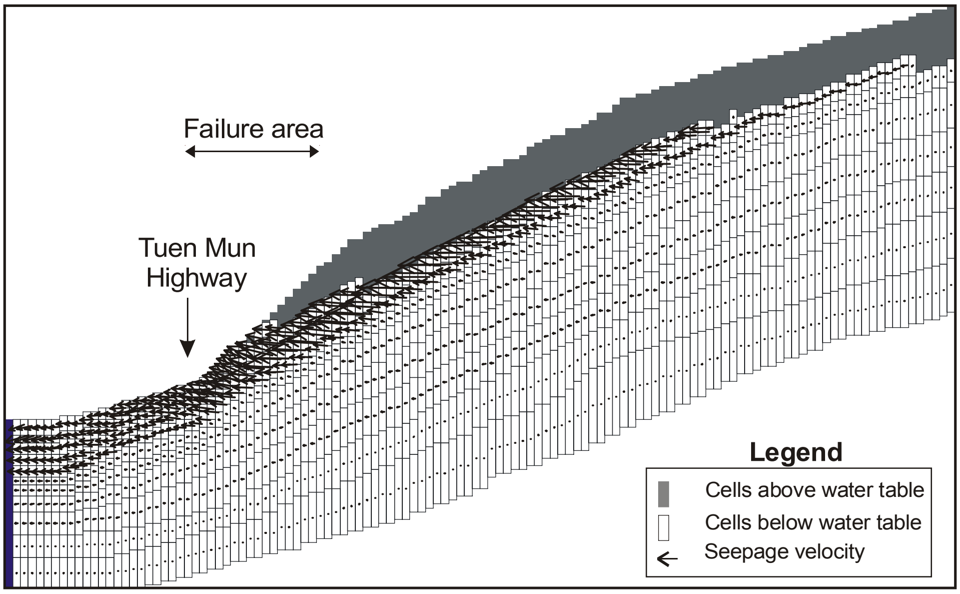

Figure 8 Simulated groundwater velocity distribution at the time of failure (Sept. 22, 1983) (vertical exaggeration=2x)

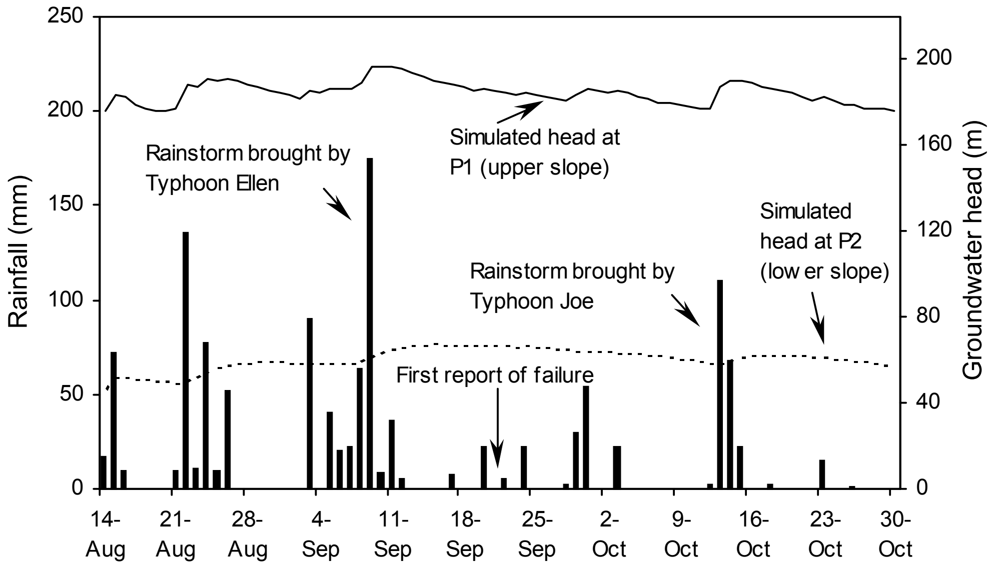

Figure 9 Comparison of rainfall with simulated hydraulic heads in upper and lower parts of the cross-sectional model in Figure 6

4. Discussion and Summaries

The Tuen Mun Highway Chainage 550 slope studied here appears to be subject to delayed groundwater hydraulic pressure response to rainfall due to the regional hydrogeological regime. The observed water level behind the slope increased after 13 days after a major rainfall in September 1983. The numerical model showed that the groundwater head behind the slope increased to a maximum 6 days after the rainstorm and then remained a very high level for about 7 days. Due to delayed response, groundwater level remained high in a period without much rainfall. In addition to the delayed response, the model also reproduced the stable hydraulic head response near the failure area. It seems that the numerical model can reproduce reasonably well the observed behavior of the groundwater. The performance of the model could be improved with more reliable aquifer parameters and recharge coefficient.

The influence of main groundwater aquifers on slope failure can be best understood in the frame of a large hydrogeological system. This is very different from the traditional geotechnical slope stability studies in which usually only a small area near the failure is examined. If the controlling factor for slope stability is regional groundwater, stability is not likely to be improved by simply cutting back the slope. In the case studied here the slope moved in 1975 and was cut back, but it failed 8 years later. In fact, when the regional groundwater conditions are considered, slope stability may be worse after a slope is cut back. This is because, after being cut back, the main groundwater level is shallower and therefore the influence of groundwater on stability is more significant. Drainage may be a more effective measure than cutting back.

Acknowledgement

The study is supported by Committee on Research and Conference Grants (CRCG) at the University of Hong Kong.

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