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Investigation of a Notable Landslide under Complex Hydrogeological Conditions at Pak Tam Road, Sai Kung, Hong Kong

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ABSTRACT On 8 June 2022, an intense rainstorm triggered multiple landslides in the northeastern area of Hong Kong. One of these landslides occurred at the registered soil and rock cut slope adjoining Pak Tam Road in the Sai Kung East Country Park with significant social consequences and widespread media attention. An investigation of this notable landslide was undertaken to study the probable causes and mechanism of the failure, as well as the hydrogeological conditions of the slope. The landslide was a large-scale rain-induced sliding failure caused by adversely orientated relict joints within the weathered rock profile. Coupling effects of inadequate slope maintenance, steep slope profile and tension cracks rendered the slope particularly vulnerable to landsliding under severe rainfall. Additionally, heavy seepage was observed from some weep holes within the same slope, which suggested the presence of complex hydrogeological conditions. Field mapping and site-specific ground investigation revealed preferential flow paths along a network of soil pipes and relict joints in the groundmass that prompted the subsurface flow and the build-up of a transient perched groundwater table. The landslide highlighted the importance of proper and regular slope maintenance of slopes in Hong Kong, and the need of assessing the site holistically when modifying the site with engineering works. A robust design solution is strongly advocated for stabilising slopes.

KEYWORDS Landslide; Soil Pipe; Hydrogeology; Slope Maintenance

1 INTRODUCTION

On 8 June 2022, around noon, a landslide occurred at the registered soil and rock cut slope (Feature No. 8NW-D/C5) adjoining Pak Tam Road in the Sai Kung East Country Park in Hong Kong, during an intense rainstorm (referred to as "the landslide" and "the slope", respectively). The landslide involved two failures, including a main scar across the full height of the slope and a secondary scar on the upper slope batter nearby, resulting in a total failure volume of about 300 m^3 (Figures 1 and 2). The landslide debris was deposited mainly on the carriageway of Pak Tam Road, leading to the temporary closure of the road for more than a week. This closure disrupted the lives of over 400 villagers in the northeast of Hong Kong who rely on the road for access to Hoi Hai Village and other local villages. The media widely reported the landslide and there were no casualties.

Following the landslide, a detailed landslide investigation was conducted to establish the probable causes and mechanism of the failure (AECOM Asia Company Limited, 2024). As heavy seepage was observed from some weep holes in the intact portion of the slope, the hydrogeological conditions of the slope were studied as part of the investigation. The landslide investigation primarily included topographic surveys, aerial photograph interpretation, detailed geological mapping, site-specific ground investigation and slope stability back-analyses.

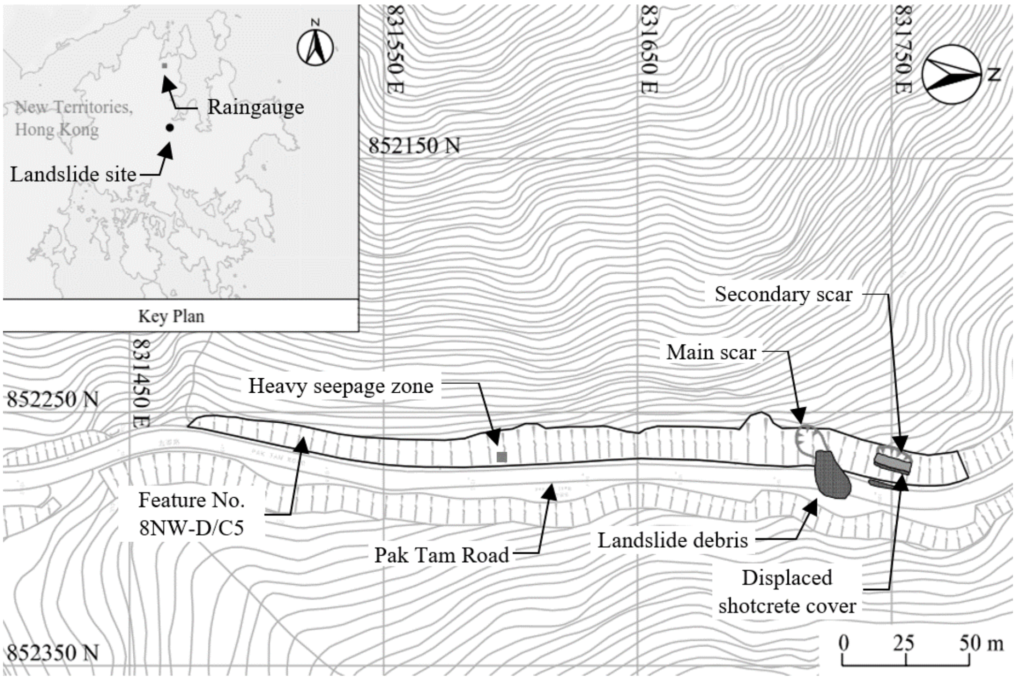

Figure 1. Location plan of the landslide site

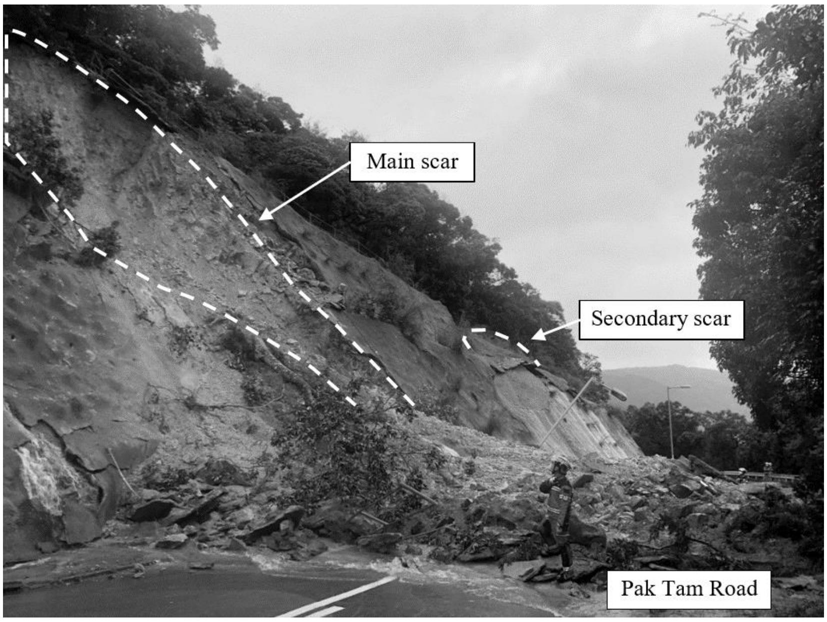

Figure 2. General view of the landslide

2 SITE DESCRIPTION

The landslide occurred on an east-facing slope with dimensions of about 300 m long, 20 m high and 50° inclined on average (Figure 3). The slope is mostly covered by shotcrete with occasional stone pitching cover and exposed rock outcrops. It has two batters and a surface drainage system comprising U-channels, stepped channels and downpipes at the crest, mid-slope and toe. Above the crest of the slope is a natural hillside with a gradient of about 28°, and at the toe of the slope is Pak Tam Road, a two-lane, two-way carriageway descending towards the north. No distinct drainage lines are descending to the landslide site, but within the natural hillside above the slope, a major drainage line descends at the south end of the slope, and a pair of ephemeral drainage lines converge at about 250 m and 75 m, respectively, to the south of the landslide site.

3 SITE HISTORY AND PAST INSTABILITIES

The earliest aerial photographs taken in 1945 show that the area where the landslide occurred was mostly a natural hillside with light to moderate vegetation. The slope was formed between 1964 and 1974 in relation to the construction of Pak Tam Road and was largely completed by 1975. There were two landslides reported in 2005 and 2020, respectively. The 2005 landslide occurred on the natural hillside just above the crest of the slope and had a failure volume of about 20 m^3. The 2020 landslide involved two failure scars in the northern portion of the slope with a failure volume of about 17 m^3 (Figure 4). The main scar was right below the slope berm, while the secondary scar was situated on the upper slope batter, coinciding with the secondary scar of the 2022 landslide. These landslide scars were later repaired by applying shotcrete with weep holes.

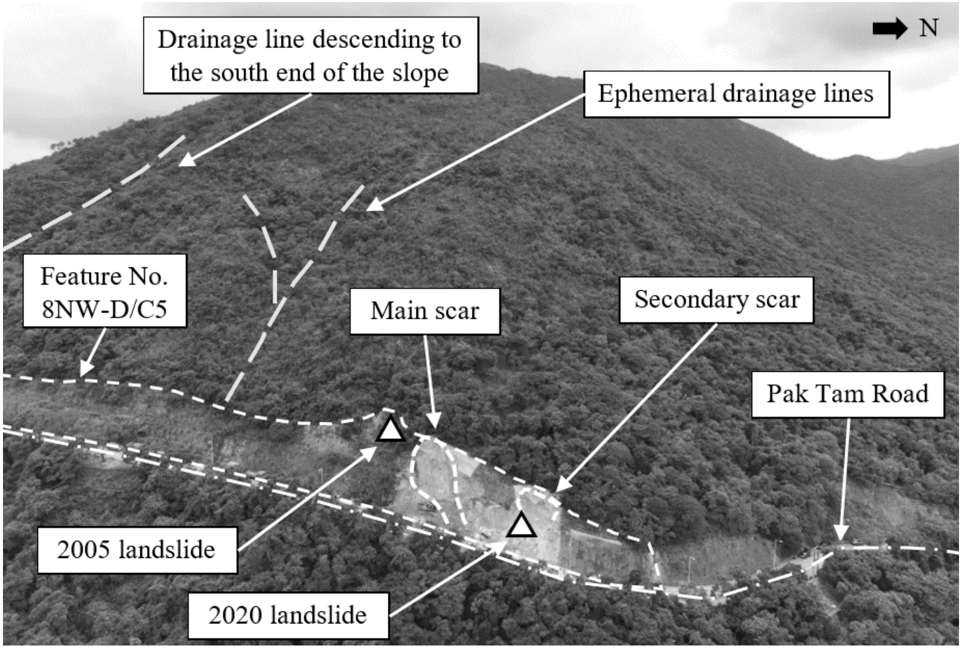

Figure 3. Aerial view of the slope

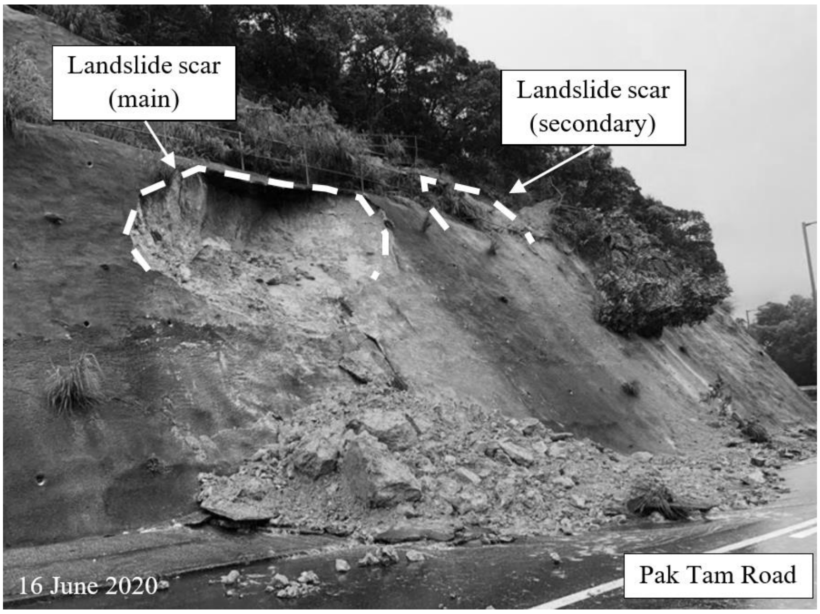

Figure 4. Landslide in 2020

4 GEOLOGY OF THE LANDSIDE SITE

Site-specific ground investigation reveals that the geology of the landslide site is primarily composed of weathered tuff overlain by a mantle of colluvium. The colluvium layer was up to 2 m thick and consisted of angular to sub-angular fine to coarse gravel and cobble- to boulder-sized rock fragments. Beneath the colluvium layer, there was a 9 m-thick layer of saprolitic soil, which comprised completely to highly decomposed coarse ash crystal tuff (C/HDT) and highly to moderately decomposed coarse ash crystal tuff (H/MDT) towards the base of the unit. This formed a less permeable layer to the strata of colluvium and C/HDT above. Bedrock, which was moderately to slightly decomposed coarse ash crystal tuff (M/SDT), was found at various depths ranging from 4 m to 11 m below the ground level. A depressed rockhead surface was encountered in drill hole Nos. DH5 and DH10, forming a subsurface valley feature (Figure 5).

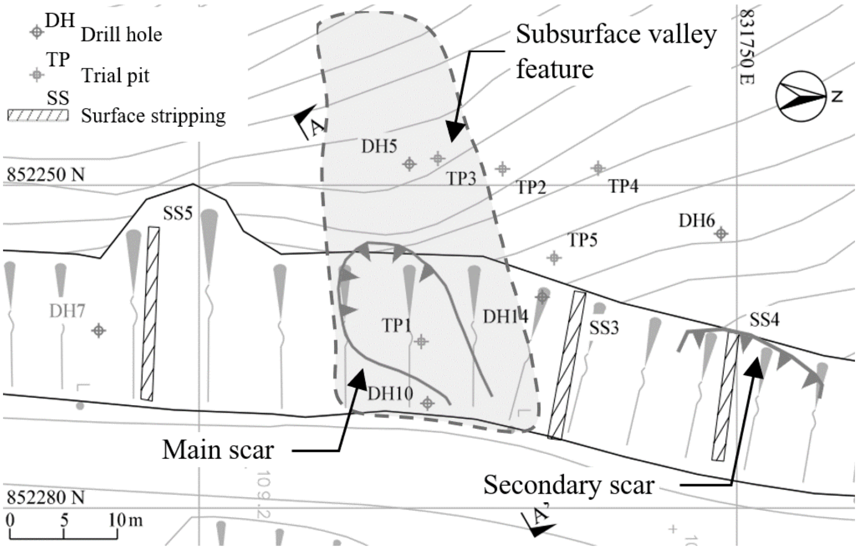

Figure 5. Subsurface valley feature

5 POST-LANDSLIDE OBSERVATIONS

5.1 General

The landslide occurred in the northern portion of the slope with distinct main and secondary scars. The slope profile was locally, steeply inclined, at about 60° to 70°, and covered with shotcrete and unplanned vegetation. Several field inspections had been conducted between June and October 2022 to examine the landslide scars. After the landslide, an unmanned aerial vehicle (UAV) was deployed to capture photogrammetric data of the site and identify any obvious signs of distress.

5.2 Landslide Scars

The main scar involved the full height of the slope and could be divided into upper and lower parts (Figure 6). The upper part was spoon-shaped and about 10.5 m wide, 15 m long, and up to 2.5 m deep. It encroached onto the natural hillside above and straddled across the intermediate slope berm. The lower part of the main scar was an extension of the upper part, but with a shallower extent. It was rectangular-shaped and about 4.7 m wide, 9.4 m long and 1.7 m deep. The main scarp was steeply inclined at about 75° with two discrete erosion gullies. The source floor was inclined at about 40° to 50° dipping out of the slope. The exposed colluvium, forming much of the main scarp, comprised angular gravel and cobble, and occasional cobble- to boulder-sized rock fragments within a silty sand matrix. Below the colluvium was a layer of C/HDT, comprising stiff to hard sandy, clayey silt with blocks of H/MDT. The source floor exposed adversely orientated relict joints on C/HDT, partly forming the basal slip plane, dipping between 25° to 50° out of the slope.

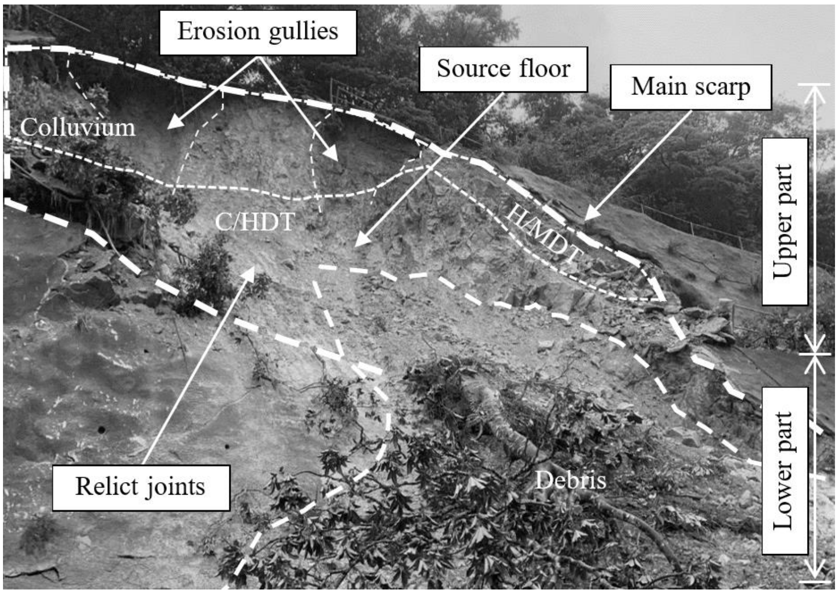

Figure 6. Main scar

The secondary scar was located approximately 20 m north of the main scar and involved a re-activated failure of the previous landslide in 2020 (Figure 7). The landslide was shallow with limited movement of the displaced shotcrete cover. The scar had a rectangular shape of about 12.5 m wide, 7.5 m long and 0.5 m deep. Colluvium formed the main scarp and C/HDT formed the source floor. The material characteristics of colluvium and C/HDT were similar to those described in the main scar, although no relict joints were observed on the source floor.

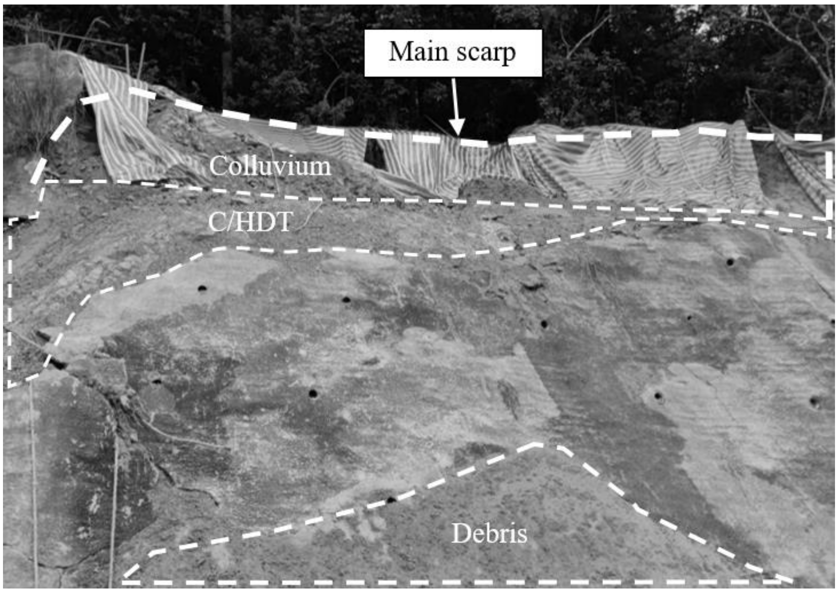

Figure 7. Secondary scar

Tension cracks were observed above the landslide scars and generally orientated in a northwestern to southeastern direction. The tension cracks were measured to be about 3 m to 12 m long with a vertical displacement of up to 0.2 m, exposing colluvium (Figure 8). The growth of moss and the general weathered appearance of the exposed soil surface suggest that part of the tension cracks was formed before the landslide.

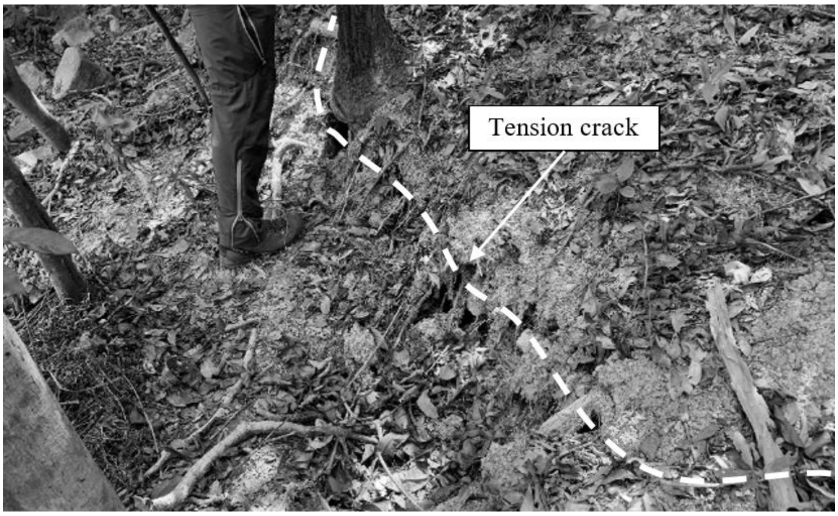

Figure 8. Tension cracks above the landslide scars

5.3 Landslide Debris and Failure Volume

The debris contained highly saturated colluvium, soil and angular rock blocks of partially weathered tuff, and uprooted shrubs and trees. The landslide caused damage to concrete berms, shotcrete covers, surface channels and handrails, with fragments of these materials scattered among the debris on the carriageway.

To better understand the extent of the landslide, a change detection analysis was carried out. This involved comparing the 3D point cloud model generated from the photogrammetric data of the UAV survey, with the pre-failure slope profile captured by the Light Detection and Ranging (LiDAR) survey in 2020 (Figure 9). The analysis indicated that the main and secondary scars had a failure volume of about 250 m^3 and 47 m^3, respectively, resulting in a total failure volume of about 300 m^3. These results were consistent with the field observations and mapping results.

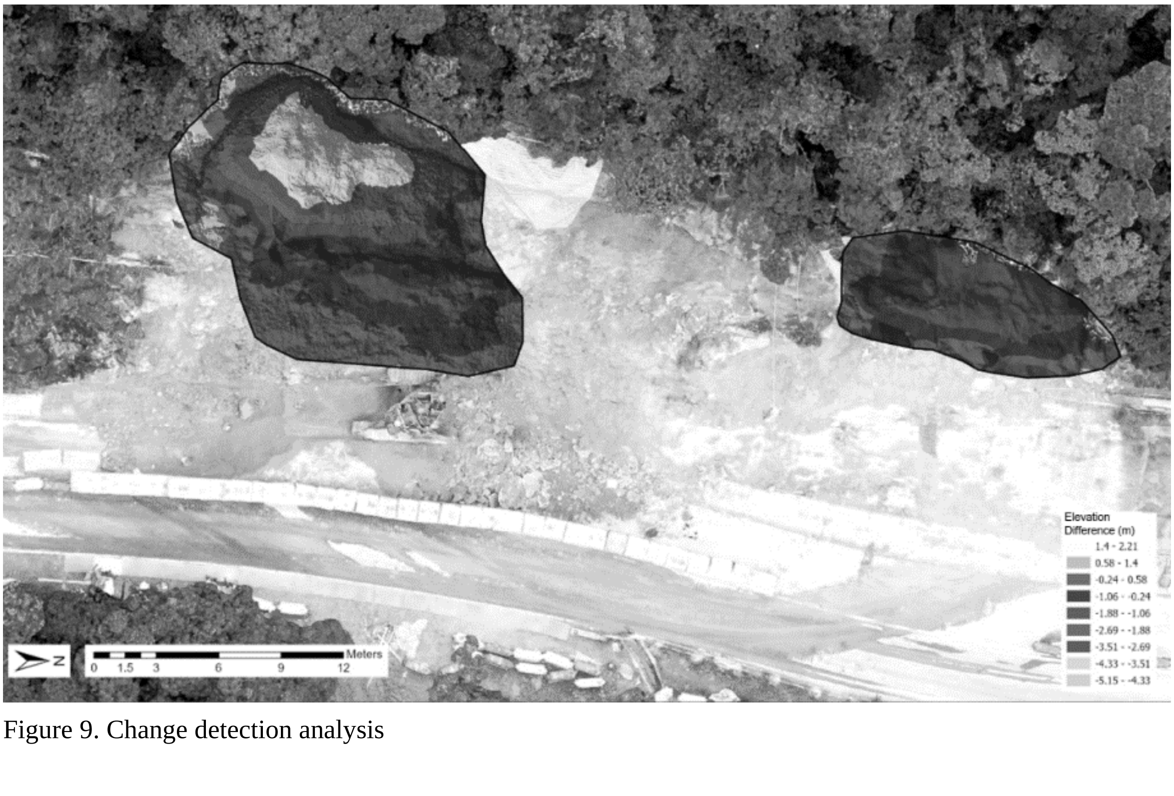

Figure 9. Change detection analysis

5.4 Slope Drainage Provisions and Surface Protection

Site inspections revealed several defects on the slope, including extensive blockage of surface channels, catchpits and weep holes, growth of unplanned vegetation on the slope surface, and cracking and spalling of the shotcrete cover. The blockage of the surface drainage system led to significant surface overflow from the intermediate slope berm during the rainfall (Figure 10). Trees and shrubs growing in the surface channels, catchpits and shotcrete cover indicate that the undesirable conditions had been developing for a long time, possibly in the order of years.

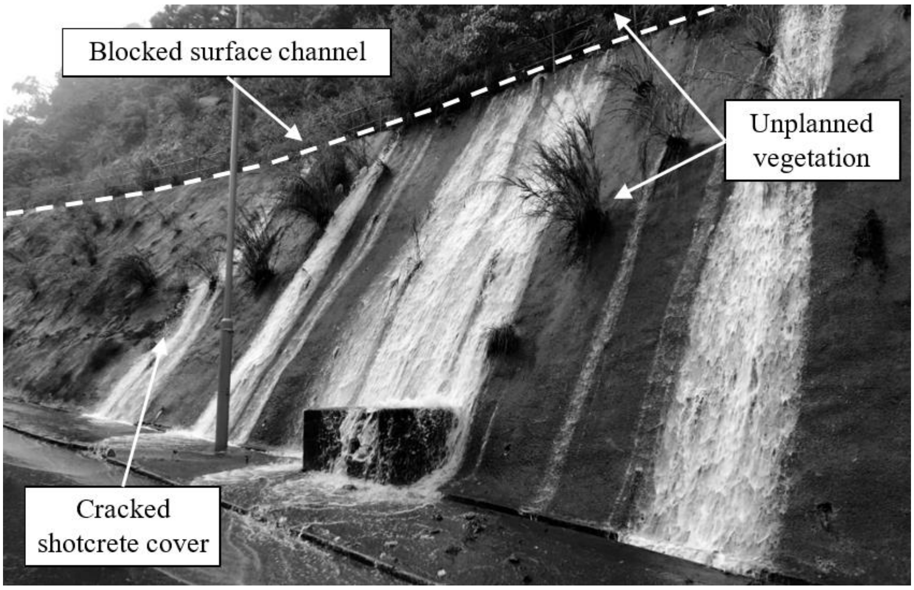

Figure 10. Surface overflow from the intermediate slope berm

5.5 Heavy Seepage Zone

During the time of the landslide, a zone of heavy seepage was observed in the intact portion of the slope, which is about 120 m south of the landslide site (Figures 1 and 11). The localised nature of the heavy seepage indicates it could be related to specific hydrogeological conditions of the slope.

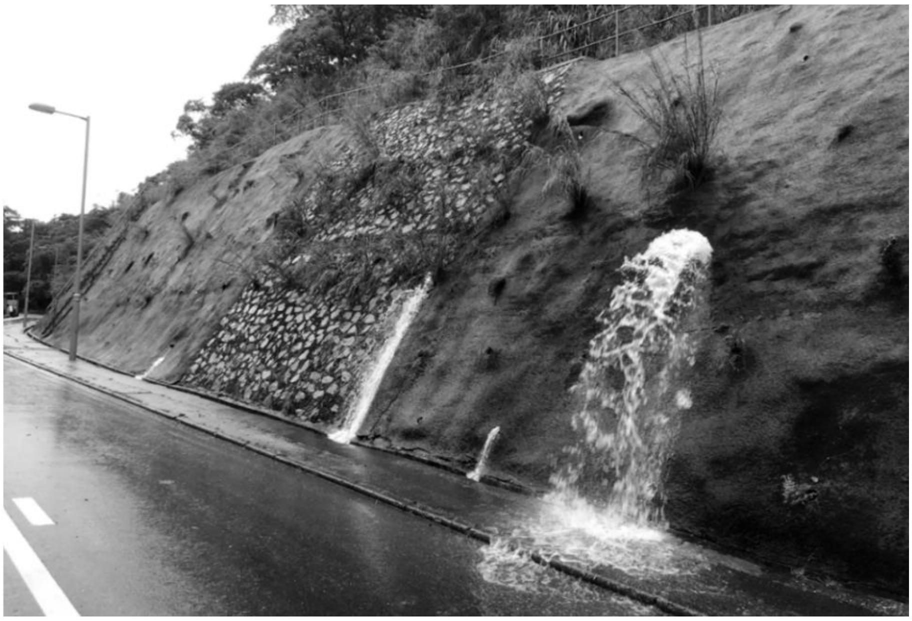

Figure 11. Heavy seepage in the intact portion of the slope

6 SLOPE STABILITY BACK-ANALYSES

Slope stability back-analyses were conducted to evaluate the likely rise of the groundwater table that triggered the failure. The analyses employed the computer program SLOPE/W, which uses the limit equilibrium method to calculate theoretical factors of safety at various levels of the groundwater table. This method considers both rigorous force and moment equilibriums (Morgenstern-Price approach). Shear strength parameters of the ground materials were derived from the single-stage isotropically consolidated undrained triaxial compression tests on the soil specimens collected from the site-specific ground investigation. The adopted shear strength parameters are summarised in Table 1.

Table 1. Shear Strength Parameters adopted in Slope Stability Back-analyses

Soil Type	Bulk unit weight γ (kN/m^3)	cohesion c' (kPa)	Angle of shearing resistance ϕ' (°)
Colluvium	19	2	32
C/HDT	19	8	35
H/MDT	19	10	38

According to the results, the factor of safety of the soil mass that failed in the landslide would be less than unity when a perched groundwater table had risen to 1.8 m above the H/MDT layer, i.e. 0.8 m above the rupture surface (Figure 12). These findings are based on the shear strength parameters listed in Table 1. The site-specific ground investigation shows that the H/MDT layer is relatively less permeable than the colluvium and C/HDT strata. This allows the perched groundwater table to rise above H/MDT and reach the soil mass of C/HDT.

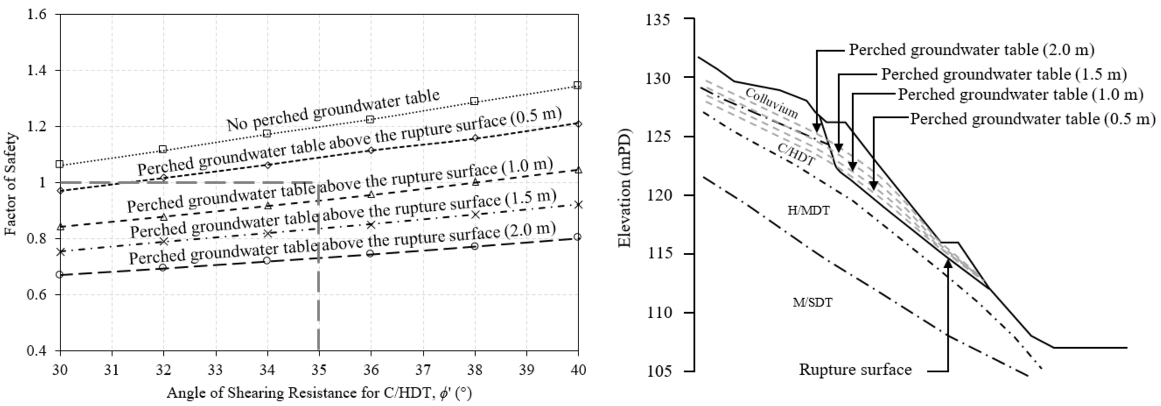

Figure 12. Summary of slope stability back-analyses

The slope stability back-analyses indicate that the slope could fail if a transient perched groundwater table builds up above the H/MDT layer. The loss of shear strength of C/HDT eventually led to the collapse of the soil mass, with a rupture surface developed along saprolitic soil.

7 DIAGNOSIS OF PROBABLE CAUSES AND MECHANISM OF THE FAILURE

The landslide involved a sliding failure, in which the rupture surface exploited daylighting relict joints in the weathered tuff. In light of the mode of failure and the close correlation of the time of the incident with the rainfall, the landslide is considered rain-induced. The presence of adversely orientated relict joints and steep slope profile, coupled with various slope defects, could have rendered the groundmass particularly vulnerable to landsliding.

The defective surface drainage and damaged shotcrete cover could have enhanced direct infiltration to the landslide site (Figure 13). The blocked U-channels in the slope crest failed to divert surface runoff from the upslope catchment but aggravated water ingress to the underlying soil through the prevalent cracks on the shotcrete cover. This, in turn, caused local erosion and elevated pore water pressures. The existing tension cracks along the slope crest could have further facilitated direct infiltration into the slope. With the presence of abundant water sources, subsurface flow could have developed in the permeable layers of colluvium and C/HDT, and converged towards the landslide site under a setting of locally depressed rockhead. Due to unplanned vegetation blocking the weep holes, groundwater pressure behind the shotcrete cover could not be relieved from the slope properly. This could have provided the hydrogeological conditions favourable to the rapid development of a transient perched groundwater table above the less permeable layer of H/MDT to the stratum of C/HDT. This is supported by the results of the slope stability back-analyses.

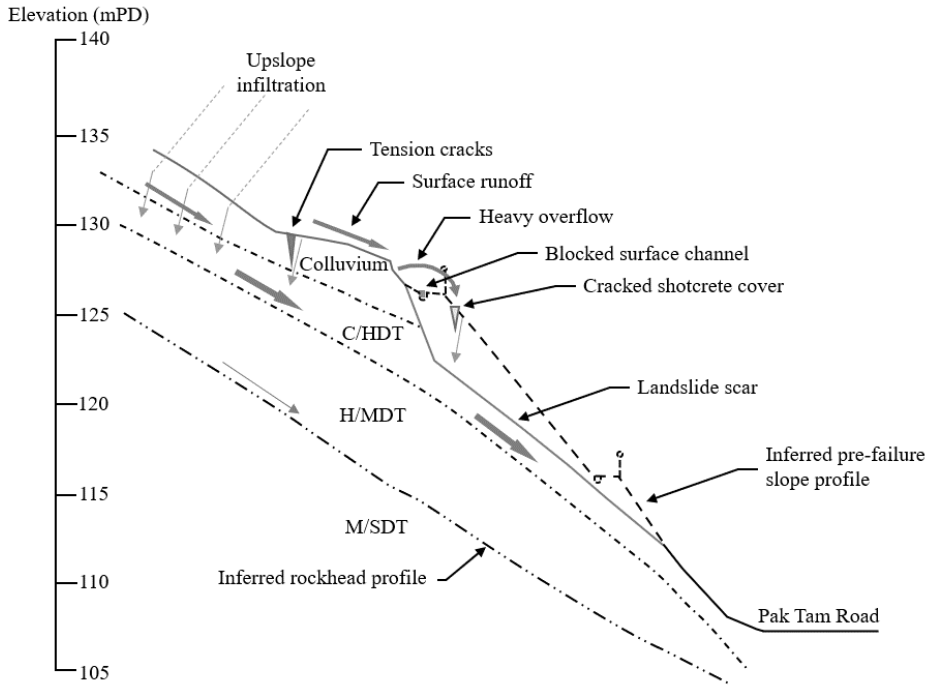

Figure 13. Cross-section A-A' of the main scar

Inadequate slope maintenance is one of the key contributory factors to the landslide. Coupling effects of slope defects, such as cracked shotcrete cover, blocked surface channels and weep holes, could have exacerbated a sizeable failure even under a moderately intense rainstorm (Ho & Lau, 2010; Lo et al., 1998; Wong & Ho, 2000). The growth of unplanned vegetation in weep holes and surface channels would impede the intended slope drainage functions and cause significant spalling and cracking of the hard surface cover. Apart from hampering the surface protection function, this could provide a pathway for direct water ingress, possibly promoting landsliding. The landslide reaffirmed the importance of proper and regular slope maintenance to retard slope deterioration and upkeep the conditions of prevailing slope surface protection and drainage provisions (Cheung, 2021; Geotechnical Engineering Office, 2023; Lee et al., 2018).

Given the highly heterogeneous ground conditions in weathered rock profiles, those subtle adverse geological features may be difficult to detect and assess in the design stage. Hence, a pragmatic approach to managing the landslide risk is to use more robust design solutions, such as soil nails with prescriptive raking drains, which are less sensitive to uncertainties or undetected weaknesses (Ho et al., 2016; Pun et al., 2019).

8 HYDROGEOLOGICAL CONDITIONS AT HEAVY SEEPAGE ZONE

8.1 Geomorphology

The heavy seepage zone was observed from weep holes in the intact portion of the slope, some 120 m south of the landslide site (Figure 11). The catchment upslope has an area of about 1.1 hectares and is characterised by planar to gently concave, moderately steep, densely vegetated natural hillside with no prominent drainage lines but bounded by broad and rounded spurs. The catchment forms a shallow, broad depression that converges groundwater flow towards the heavy seepage zone. Intermediate rock outcrops in the natural hillside facilitate concentrated surface runoff to the catchment.

8.2 Ground Investigation

Apart from the landslide site, the site-specific ground investigation was also carried out in the intact portion of the slope and the natural hillside above. A vertical drill hole and a trial pit at the heavy seepage zone encountered CDT with coarse gravel and cobble-sized rock fragments and abundant relict joints daylighting out of the slope. These relict joints have similar orientations as found in the main scar to the south (Figures 14 and 15). They could have facilitated infiltration and provided pathways for groundwater flow in saprolitic soil. An area of localised depressed rockhead was also mapped at the heavy seepage zone.

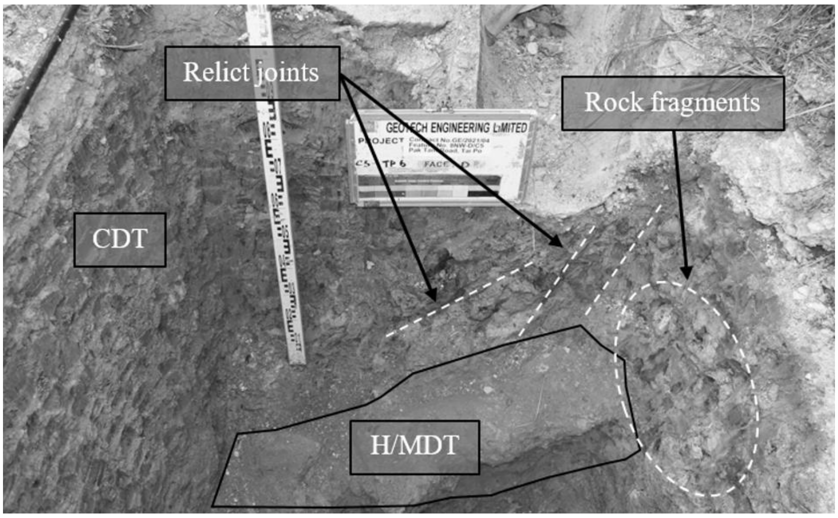

Figure 14. Ground condition in the trial pit

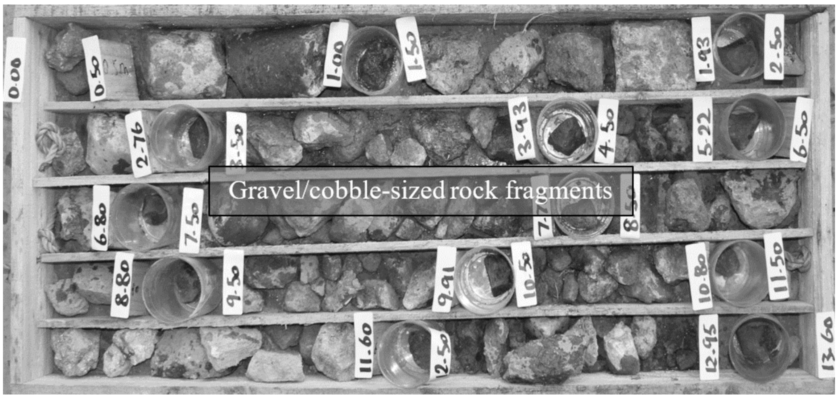

Figure 15. Samples of rock cores from the vertical drill hole

Surface stripping of the shotcrete covers was conducted to expose soil pipes in the heavy seepage zone. The soil pipes were measured up to 250 mm in diameter and 1.5 m deep, connecting to weep holes (Figure 16). These pipes were typically located in CDT that was characterised by sandy clayey slit and close to a less permeable HDT interface below. This formation created preferential flow paths for groundwater to be discharged from the slope, and the water was observed flowing from the weep holes, as shown in Figure 11.

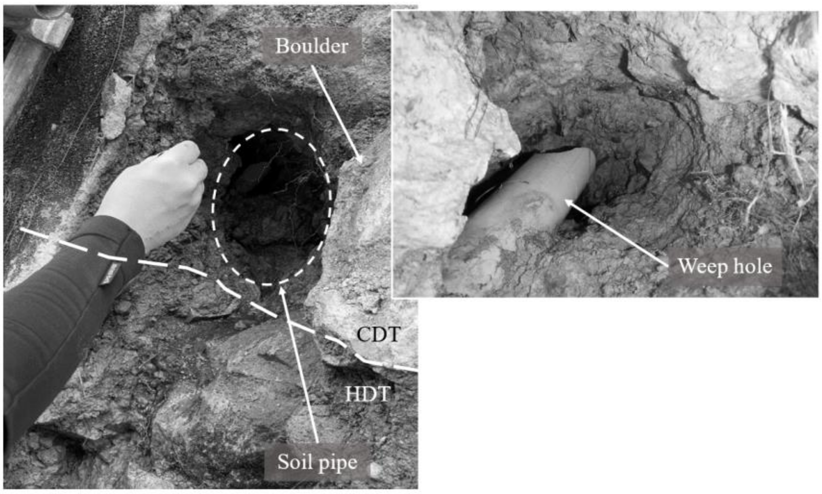

Figure 16. Soil pipe exposed from the surface stripping

The large trajectory of water flowing from the weep holes suggests that the water was subjected to a transient build-up of a high-pressure head due to the impermeability of the shotcrete cover and the blockage of surrounding weep holes by unplanned vegetation, causing the back up of groundwater in the slope. The soil pipes could have developed progressively from pre-existing relict joints and eroded headward, forming a relatively large opening.

Permeability tests were carried out in drill holes in the natural hillside above the slope. It was determined that the permeability of colluvium and CDT was about 10^{-5} m/s, while HDT was about 10^{-7} m/s, indicating an interface of contrasting permeability. During the permeability test in one of the drill holes, water was observed flowing from the soil pipe, about 35 m downslope of the drill hole. A dye test was carried out in the drill hole to confirm the source of the flowing water (Figure 17). The test result found that the dye flowed to the soil pipe in about ten minutes, equivalent to a rate of about 10^{-4} m^3/s.

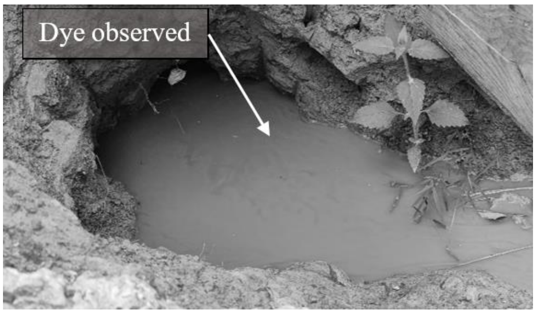

Figure 17. Observations on the soil pipe during the dye test

The permeability measured in the groundmass is unlikely to convey groundwater through the slope at the rate observed in the soil pipe. It is considered credible, with support from the observations in trial pits and surface stripping, that the drill hole intersected a network of soil pipes and relict joints which were directly connected to the weep holes at the heavy seepage zone.

The interface of ground materials with relative permeability contrast is a common location for soil pipe formation (Nash & Dale, 1984; Fletcher, 2004). The inherent characteristics of the weathered rock, dense blocky nature of clasts, and relict joints in CDT and HDT locally facilitated the initiation of soil pipe formation through preferential pathways. The influences of the upslope catchment and groundwater flow are important factors as the direct infiltration of rainfall does not suggest such a rapid response of heavy seepage via perching in the soil mass, and where groundwater response is rapid, a local area or feature of high hydraulic conductivity may have been intercepted.

8.3 Discussion

A ground model in Figure 18 illustrates how groundwater flows through the heavy seepage zone. It is postulated that direct infiltration and surface runoff from the upslope hillside could have converged on the catchment bounded by broad and rounded spurs. The surface water infiltrated into the groundmass and flowed along the network of soil pipes and relict joints preferentially, heading to the zone of heavy seepage under a setting of depressed rockhead. Some weep holes were blocked by unplanned vegetation, which caused the groundwater to back up and exit at nearby open weep holes at a greater pressure, resulting in a large trajectory observed at the site. These hydrogeological conditions could have caused the slope susceptible to internal erosion and landsliding.

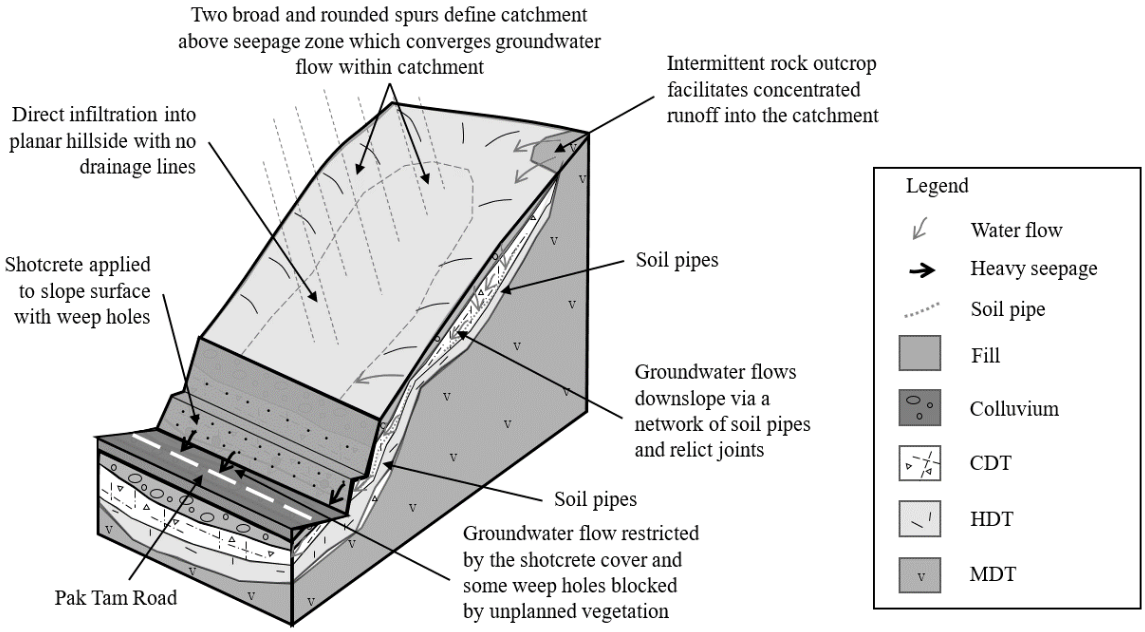

Figure 18. Conceptual ground model of the heavy seepage zone

The investigation of the heavy seepage zone revealed that the local site setting and geomorphology are conducive to active groundwater flow downslope via upslope catchment, permeable surficial soil strata, and network of soil pipes and relict joints developed at interfaces of permeability contrast. When this natural system is modified by slope formation with a shotcrete cover, it is crucial to ensure adequate subsurface drainage measures are provided for and maintained to prevent the build-up of excess groundwater pressures and consequent deterioration of slope stability.

Design models of simple homogeneous and isotropic masses with given hydraulic conductivity may not accurately represent the actual ground conditions in Hong Kong. Refinements in models might be warranted to incorporate geological features affecting hydrogeology (Hencher, 2006; 2010). The heavy seepage observed at the slope demonstrates the importance of assessing the site holistically, including geomorphology, site setting, hydrogeology, and potential adverse effects, when modifying the site with engineering works.

Following the landslide, the slope was stabilised by a robust design solution, in the combination of soil nails and raking drains, under the Landslip Prevention and Mitigation Programme. The stabilised slope remains intact after several rainstorms, including the record-breaking rainstorm in September 2023. This demonstrates the robust design solution is less sensitive to local adverse geological and groundwater conditions in comparison with unsupported cuts.

9 CONCLUSION

The landslide occurred within a steep, shotcrete-covered soil and rock cut slope with adversely orientated relict joints in the weathered tuff over a locally depressed rockhead surface. The failure was probably caused by the build-up of a transient perched groundwater table above the less permeable layer of H/MDT following the intense rainfall. Tension cracks were found above the landslide scars, which led to enhanced infiltration and water ingress. Defects in the slope drainage system and surface cover could have also contributed to the landslide. This incident highlighted the importance of proper and regular slope maintenance to the practitioners in Hong Kong. The study of hydrogeological conditions at the heavy seepage zone revealed that local site setting and geomorphology are conducive to active groundwater flow downslope. This reiterates the importance of assessing the site holistically, including geomorphology, site setting, hydrogeology, and potential adverse effects, when modifying the site with engineering works. Robust design solutions that are less sensitive to local adverse geological and groundwater conditions are strongly advocated for stabilising slopes.

DISCLAIMER

The authors declare no conflict of interest.

AVAILABILITY OF DATA AND MATERIALS

All data are available from the author.

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