INVESTIGATION OF SOME SELECTED LANDSLIDES IN 2000 (VOLUME 1)

GEO REPORT No. 129

Halcrow China Limited

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG

SPECIAL ADMINISTRATIVE REGION

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. A charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
October 2002

FOREWORD

This report presents the findings of a detailed study of a series of landslides with total failure volumes ranging from 70 m^3 to 600 m^3, which occurred on 14 April 2000 on the hillside above Leung King Estate, Tuen Mun. An estimated volume of 400 m^3 of outwash material was deposited within the perimeter road of Leung King Estate. No fatalities or injuries were reported.

The key objectives of the detailed study were to document the facts about the landslides, present relevant background information and establish the probable causes of the failures. The scope of the study comprised site reconnaissance, limited ground investigation and laboratory testing, desk study and engineering analysis. Recommendations for follow-up actions are reported separately.

The report was prepared as part of the Landslide Investigation Consultancy for Kowloon and the New Territories in 2000 and the First Quarter of 2001, for the Geotechnical Engineering Office (GEO), under Agreement No. CE 2/2000. This is one of a series of reports produced during the consultancy by Halcrow China Ltd. (HCL).

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1. INTRODUCTION

At sometime before 7:00 a.m. on 14 April 2000 during heavy rain, a series of landslides (collectively registered as GEO Incident No. MW2000/04/018) occurred on the hillside above Leung King Estate, Tuen Mun in the northwestern New Territories (Figure 1 and Plate 1). The larger of these failures are labelled Landslides A to D on Figure 1. This report reviews the site history, geology and geomorphology of the site (defined as the catchment of Landslides A to D) and considers in detail two of these landslides, namely Landslides B and C (Plate 2), which affected the facilities within Leung King Estate. Landslides B and C involved about 600 m^3 and 70 m^3 of material respectively (source and entrained volumes). Both failures developed into channelised debris flows, which came to rest close to the toe of the hillside. The subsequent outwash from these events blocked the drainage at the crest of slope No. 5SE-B/CR26 above Leung King Estate, causing inundation of alluvial outwash debris into the northwestern portion of the estate. It is estimated that about 400 m^3 of material (mostly silty sand with occasional boulders) was deposited within the perimeter road of Leung King Estate (Plates 3 to 6). No fatalities or injuries were reported as a result of the landslides.

Following the landslides, Halcrow China Ltd. (HCL) carried out a detailed study of the failures for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 2/2000.

The key objectives of the detailed study were to document the facts about the landslides, present relevant background information and establish the probable causes of the landslides. The scope of the detailed study comprised site reconnaissance, limited ground investigation and laboratory testing, desk study and engineering analyses. Recommendations for follow-up actions are reported separately.

This report presents the findings of the detailed study, which comprised the following key tasks:

(a) desk study, including a review of relevant documentary records and old topographic maps relating to the history of the site,

(b) aerial photograph interpretation (API),

(c) interviews with concerned persons,

(d) topographic surveys, geomorphological and geological mapping, and detailed observations and measurements at the landslide site,

(e) limited ground investigation of the hillside together with laboratory testing of soil samples collected from the landslide sites,

(f) analysis of rainfall data,

(g) engineering analyses, and

(h) diagnosis of the probable causes of failure.

2. THE SITE

2.1 Site Description

The site is located on a generally southeast-facing hillside on the southern edge of the Castle Peak Firing Range, to the northwest of Leung King Estate, Tuen Mun (Figure 1 and Plate 1). The hillside comprises a northeast to southwest trending main ridgeline, with steep rock cliffs located on its southeast flank. Below the cliffs are minor, generally rounded, spur and valley profiles, approximately perpendicular to the ridgeline. A prominent northwest to southeast trending valley dominates the northeastern end of the site.

In general, the upper hillside is fairly densely vegetated, with the exception of areas of exposed granite rock, with dense bushy scrub and occasional stunted trees. Towards the toe of the hillside, the vegetation changes to dense woodland. Many graves, mostly located on the flanks of the natural drainage line below Landslide B, have been constructed within this area of woodland.

The catchments above the sources of Landslides B and C are relatively small, estimated as approximately 5500 m^2 and 1300 m^2 in plan area respectively.

The source area of Landslide B (Figure 2) is a natural depression (which probably caused convergence of surface runoff flow), located at the head of a minor natural drainage line. The surrounding terrain consists generally of moderately steep sloping natural hillside with slope angles typically varying between 30° and 40°. The inclination of the sparsely vegetated granite cliff located approximately 20 m north of the landslide site varies from 50° to almost 80° in places.

The natural drainage line below Landslide B is generally shallow with a rounded profile along its upper section, becoming more deeply incised towards the toe of the hillside. The drainage line bifurcates at about 60 m above the crest of cut slope No. 5SE-B/CR26. The eastern drainage line is generally poorly defined below this point. About 30 m above the crest of the slope, both drainage lines are intercepted by concrete U-channels which drain into sand traps located at the crest of slope No. 5SE-B/CR26. Stepped channels lead from the sand traps to drainage inlets at the toe of a crib retaining wall (No. 5SE-B/R5), located along the northwestern boundary of Leung King Estate.

The source area of Landslide C (Figure 2) is an oversteep man-made cut slope, about 3 m high at the location of the landslide, formed during mining works prior to 1945. The cut slope is one of a series of mining-related cut slopes that traverse the hillside (see Section 3.2). The inclination of the surrounding moderately densely vegetated hillside is typically 20°, while the inclination of the cut slopes ranges from 45° to about 60°. In addition to the cut slopes, the terrain surrounding the site of Landslide C is extensively altered with many excavations (up to 5 m deep) and mining adits (generally partly or wholly collapsed) located on the surrounding hillside.

The natural drainage line below Landslide C comprises a poorly defined shallow topographical hollow which eventually leads to a 1 m high random rubble retaining wall constructed on a granite rock exposure, some 80 m (in plan) downstream from the source area. Below the exposed rock the drainage line gradually becomes more incised until towards the base of the natural hillside where the bed of the drainage line is about 4 m wide and about 5 m deep. A flagstone spillway has been constructed across the floor of the drainage line along a 30 m section above the man-made features at the toe of the hillside. The spillway is intercepted by a sand trap located at the crest of slope No. 5SE-B/CR26. The sand trap leads down a stepped channel to a drainage inlet at the toe of a crib wall (No. 5SE-B/R5) along the perimeter of the Leung King Estate.

Leung King Estate, a public housing development owned by the Hong Kong Housing Authority (HKHA), is located on a level platform to the southeast of the site.

The only water-carrying services in the vicinity of any of the landslides is the San Wai to Lung Kwu Tan Sewage Tunnels, constructed in 1993 and operated by the Drainage Services Department (DSD). The alignment of the 3 m diameter tunnel (shown on Figure A1, Appendix A) passes in close proximity to Landslide B and C on plan, however, at the location of the landslides the invert level of the tunnel is some 100 m below ground level.

With the exception of Landslides C, C1 and C2 which occurred on unregistered cut slopes formed during pre-1945 mining works, the recent landslides are located on natural hillside. Alluvial outwash from Landslides B and C affected a cut slope (No. 5SE-B/CR26) and a crib wall (No. 5SE-B/R5) located along the northwestern boundary of Leung King Estate (Figure 1).

2.2 Regional Geology

The Hong Kong Geological Survey's 1:20 000 scale map, Sheet 5 Tsing Shan (Castle Peak) Solid and Superficial Geology (Geotechnical Control Office, 1988), indicates that the solid geology of the site is divided into fine-grained granite to the northwest and andesite with tuff and tuffite to the southeast (Figure 3), separated by a faulted boundary. In places, a schist fabric is developed within the granite, while the andesite shows signs of metamorphism in proximity to the fault. Many southwest to northeast trending quartz veins, which contain molybdenite, traverse the hillside. The source areas of both Landslides B and C are underlain by megacrystic fine-grained granite which is Jurassic to Cretaceous in age.

The superficial geology consists of Quaternary debris flow deposits (colluvium) found both as sheet deposits along the toe of the hillside and within the natural drainage lines (Figure 3).

Two prominent fault trends are shown on the geological map as within the landslide site area, i.e. southwest to northeast and northwest to southeast respectively, approximately following the trend of the major ridge line and natural drainage lines.

The Geological Memoir (No. 3), Geology of the Western New Territories, (GCO, 1989) notes that no major mines have been operated within the district but numerous shallow workings of fissure veins, containing economic quantities of wolframite and molybdenite, formerly existed along the eastern slopes of Tsing Shan.

The detailed geology of the sources of Landslides B and C is described in Section 5.6 of this report.

2.3 Maintenance Responsibility

According to the Lands Department, all the 2000 landslides, with the exception of Landslide C4, are located within the Castle Peak Firing Range (Defence Lot 16) which has been handed over to the Hong Kong Garrison. Landslide C4 occurred within unallocated Government land located between the boundary of the Firing Range and the Vesting Order boundary of Leung King Estate. The alluvial outwash resulting from Landslides B and C, affected both Leung King Estate and the adjacent registered slope/retaining wall (see Section 2.4), which are under the maintenance responsibility of the HKHA.

3. HISTORY OF THE SITE

3.1 General

The history of the site has been established from a review of available aerial photographs (detailed observations are given in Appendix A), and a review of relevant documentary information.

3.2 Site Development

The earliest available aerial photographs (1945 and 1949) indicate that much of the hillside in the northeast of the site had been extensively altered by human activities associated with what appears to be shallow strip mining (Figure A2, Appendix A). However, the source area of Landslide B does not seem to have been affected by mining works. The mining was mostly in the form of shallow excavation along mineralised quartz veins (molybdenite and associated minerals) that traverse the hillside, and resulted in a series of discontinuous sub-parallel cut slopes, ranging from about 100 m to over 400 m in length, and trending, east-northeast to west-southwest across the natural hillside. The slope angle of these cut slopes ranges from about 45° to 60°. Based on observations from the photographs and field mapping, these cut slopes are typically up to 4 m high, locally higher. On the aerial photographs, below the cut slopes are irregular pale-toned areas void of vegetation, which are likely to be accumulations of mining waste that had probably been tipped and accumulated downslope of the cut slopes. These areas of mining waste extend up to about 40 m downslope of the cut slopes.

Extensive areas of sheet and gully erosion are visible within the mining waste. Associated with these cut slopes, and generally on the same alignment, were a number of what appeared to be adits excavated into the hillside. Located below the adits were elongated lobes of mining waste. Many of the adits were still visible during the site inspection by HCL. Downhill of the former mining area are two relatively large cut and fill platforms together with a number of smaller cut platforms. Abandoned and partially derelict structures, presumably industrial buildings used for ore processing, were located on the platforms. Various tracks connect the abandoned buildings with the former mining operations. In the 1949 aerial photographs, while the mine appears to have been abandoned, the lack of vegetation on the disturbed ground indicates that work may only have ceased recently at that time (possibly sometime within the previous 5 years). No relevant records are available from the Mines and Quarries Division of the GEO regarding the mining activities in this area.

The API indicates that there were some mining activities pre-1945 within the natural drainage line some 50 m below Landslides B and B1 (Figure A2). However, the terrain directly surrounding the source areas of both of these landslides is apparently undisturbed natural hillside. In contrast, the source areas of Landslides C, C1 and C2 are located on the cut slopes formed during the mining works, whereas the source areas of Landslides C3 and C4 appear to be within disturbed hillside.

In 1982, site formation works for Leung King Estate commenced. By 1984, the site formation works, including the series of cut slopes that form the northwest boundary of the estate, were substantially complete. Construction of the Leung King Estate buildings commenced in 1985 and was completed by 1991. Construction of a DSD pumping station, tunnel and a private access road along the southwest boundary of the site commenced in 1991 and was completed by 1993.

The vegetation cover of the hillside, especially on the lower portion, is noted as generally becoming denser with the passage of time. At least 10 separate hillfires affecting various parts of the hillside were identified from both the API and Fire Services Department (FSD) records, though the FSD records do not give a precise location of the fires. The most recent hillfires that affected the source areas of Landslides B and C are visible in the 1991 and 1985 aerial photographs respectively (see Figure A4).

3.3 Past Instabilities within the Site

In 1995, GEO compiled the Natural Terrain Landslide Inventory (NTLI) based on interpretation of high-level aerial photographs dating from 1945 to 1994 (Evans et al, 1997; King, 1997). The only NTLI features in the study area are two separate clusters of landslides identified along the ridgeline to the north of the site (NTLI Reference Nos. 4 to 7 and 179 to 181 on Figure A1) and a single open hillside failure (NTLI Reference No. 9 on Figure A1) located about 20 m southwest of the source of Landslide B. The detailed review of the low-level aerial photographs of the site by HCL showed that both of these clusters were in fact areas of surface erosion and not landslides, and that the single open hillside failure (No. 9) probably occurred within colluvium and had an estimated volume of less than 50 m^3.

In 1998, GEO commissioned Scott Wilson (Hong Kong) Ltd. (SW) under Agreement No. CE 39/98 to carry out a review entitled Large Landslide Study (SW, 1998). No large landslides were identified within the site during SW's review.

The GEO landslide database indicates no reported landslides within the site.

Figure 4 shows the location of all observed landslides within the site based primarily on observations from low-level API, together with the NTLI and field observations. The observed landslide scars are labelled with a two digit number representing the year when the scar was first observed, followed by a sequential letter. The history of instabilities within the individual catchments associated with the natural drainage lines, namely DC1, DC3, DC4 and DC6 (Figure A1), are described below (no failures were recorded in the catchments associated with drainage lines DC2 and DC5):

(a) Catchment associated with DC1:
The API identified two failures (91B and 98A) that appear to have occurred within the source area of Landslide A. While Landslide 91B appears to be minor (source volume probably < 50 m^3), Landslide 98A resulted in a channelised debris flow that travelled about 160 m (Figure 4). The source volume of Landslide 98A (based on API) was probably between 100 m^3 and 200 m^3. Three relatively minor open hillside failures (91A, 91C and 91D) were observed on the 1991 aerial photographs, located close to the southern boundary of the site. Based on the API the source volumes of these failures were probably < 50 m^3 and the resulting debris did not travel far. A minor failure (labelled 94A on Figure 4), with an estimated volume of < 50 m^3, occurred on the northeast boundary of the catchment. The debris became channelised within a minor subsidiary channel, probably terminating at the junction of the subsidiary and the main natural drainage line, some 100 m below the source of the landslide.

(b) Catchment associated with DC3:
Two minor open hillside shallow landslides (failure volume assessed by API to be probably < 50 m^3) were identified on the open hillside about 30 m southwest of the source of Landslide B (NTLI Reference No. 9 and 86A). Based on the aerial photographs, both failures appear to be within colluvium and the debris from the failures did not become channelised.

(c) Catchment associated with DC4:
From 1945 to 1986 many of the aerial photographs show areas of disturbed ground (identified as pale-toned patches without vegetation cover) on the cut slopes formed during the mining works in the vicinity of Landslide C. It was generally not possible to determine whether the disturbed ground was a result of surface erosion or minor shallow failures of the slopes. The irregular slope profile of the cut slopes, visible both on site and on the aerial photographs, suggests that they have been affected by a series of minor landslides and/or erosion over the years. As described in Section 3 above, the mining waste on the hillside was prone

to both sheet and gully erosion over the years. Two minor failures, Landslides 82A and 82B, that were visible on the northeast flank of the natural drainage line (DC4) located downhill of Landslide C, probably occurred within the mining waste. Landslide 85A, estimated from the aerial photographs to have a source volume of < 100 m^3, occurred just above the cut slope No. 5SE-B/CR26 and is likely to be related to the slope formation works which had been completed recently at the time of failure.

(d) Catchment associated with DC6:
A series of relict landslide scars (49A to 49G) located on the northeast flank of DC6 are visible in the 1949 photographs. The scars are located at the heads of minor natural drainage lines generally between 10 m and 50 m above the main drainage line below. Recent Landslides D, D3 and D4 occurred in close proximity to these earlier failures, and were probably of similar volumes (between about 50 m^3 and 200 m^3). On the southwest flank of the drainage line, twin adjacent minor open hillside landslides (91E and 91F) were visible on the 1991 aerial photographs. Based on the API, the source volume of these failures is probably < 50 m^3 and the resulting debris did not travel far.

In general, few landslides or relict landslide scars are visible in the early photographs (viz. 1945 to 1980). Up to 1980, only 7 landslide scars were observed, all occurring prior to 1949 and were located within the catchment associated with DC6. However, from about 1982 onwards, there appears to be a significant increase in the total number of landslides observed (36 nos. post-1982). All the relatively large failures, with estimated source volumes of > 50 m^3, and all the observed channelised debris flows occurred after 1982. The reason for this apparent increase in propensity of failures is not known. It should be noted that the years 1945 to 1980 are covered by 10 sets of photographs, while the years 1981 to 2000 are covered by 16 sets of photographs. Other landslides may have occurred earlier, but have become obscured by the time of later photography. This is evidently the case for several landslides recorded within the site, the scars of which are very difficult to identify only 5 to 10 years later.

It is noteworthy that only three failures (viz. NTLI References Nos. 9, 82A and 82B), all estimated to be < 20 m^3 in volume, were observed to have occurred as a result of the two major rainstorms that occurred in May and August 1982 and no landslides were observed after the major rainstorm of June 1983. A review of the rainfall distribution maps for these rainstorms indicated that the study area was not located close to the areas of maximum rainfall. However, the May 1982 rainstorm resulted in between 250 mm and 300 mm of rain in the general area of the site.

With the exception of a well defined debris lobe below Landslide A, no distinct debris lobes were identified within the site during either the API or field mapping.

Further discussion on the geomorphological hillside processes relating to the landslides is given in Section 5.5.

3.4 Previous Studies

In January 1981, Scott Wilson Kirkpatrick & Partners (SWKP) prepared a geotechnical report on the site formation of Tuen Mun New Town Area 1 (SWKP, 1981) for Tuen Mun New Town Development Office of Public Works Department. The report contains a Terrain Unit Map covering the site of Leung King Estate and an area of hillside to the northwest, including the source areas of Landslides B and C, which were classified as GM2–74. This refers to the following terrain unit descriptors:

(a) Landform Description (GM2) – Narrow to broadly rounded ridges and spurs and broadly rounded dissection slope interfluves below the steep rocky escarpment slopes of the main range.

(b) Surface Form (7) – Steep, broadly rounded to planar dissection slope interfluves, planar to concave colluvial and dissection slopes and planar slopes to drainage.

(c) Material Type (4) – Residual soils on weathered granite, generally pale yellowish brown silty sand, brownish white clayey sand to sandy silt – completely weathered granite.

In the GCO's GASP Report No. III, West New Territories (GCO, 1987), produced at a scale of 1:20 000 for regional appraisal and outline strategic planning purposes, the areas where Landslides B and C subsequently occurred were identified as zones of general instability associated with predominately colluvial and insitu terrain respectively. The Generalised Limitation and Engineering Appraisal Map contained within the report designates the source areas of both landslides, and much of the surrounding terrain, as a zone of constraints for development. The map also records an area of severe gully erosion at the location of the former mining works. The source areas of both Landslides B and C were assigned Class IV (i.e. with high geotechnical limitations) in the Geotechnical Land Use Map (GCO, 1987).

In 1998, the Territory Development Department commissioned SW under Agreement No. CE37/98 to carry out a planning and development study of a potential site near San Wai Court, Tuen Mun. As part of the assignment, a report entitled "Natural Terrain Landslide Hazard Assessment" (SW, 1999) was prepared. The area assessed covered the locations of Landslides A, B and C (Figure 4). The objective of the study was to undertake a systematic assessment of the hazards posed from landslides to downhill facilities, for a potential housing development located to the southwest Leung King Estate. The final report contains a series of maps, including API, geomorphology, slope angle and drainage and field inspection maps. The following summarises the key findings of SW's report, in the vicinity of the recent failures:

(a) The API map shows the locations of landslides and areas of erosion identified by SW. Since areas of erosion and landslides were marked with the same legend, it is not possible to distinguish between the two. Most of the failures identified during the API for this study by HCL (Figure A1, Appendix A) were recorded by SW. However, of the two channelised debris flows identified as part of this report (94A and 98A on Figure A1) only one, 98A, was marked on the SW map. Also, the API by SW did not identify the area of disturbed terrain resulting from the pre-1945 mining activities in the vicinity of Landslide C.

(b) The map on geomorphology shows certain parts of the hillside classified (in some cases inaccurately) as particular terrain units; the three terrain units used are areas of erosion, boulder accumulation and rock outcrop. However, the areas in the vicinity of Landslides B and C were not classified.

(c) The drainage map shows the locations of the natural drainage lines within the site, including those below the source areas of Landslides B and C, i.e. the locations of the 2000 channelised debris flows.

(d) The slope angle map shows local inclinations (typically at 50 m spacing) which were presumably taken from contours. No attempt was made to relate the slope angle to other parameters in order to produce a terrain susceptibility map.

(e) Six "representative potential failures sites" were identified from the API and field mapping, with a range of postulated failure mechanisms. Only one of these sites, Site 1 (see Figure 4) was within the study area for this report, located at the head of natural drainage line DC1. No landslide occurred at Site 1 during the 2000 rainstorm. The locations of the 2000 landslides were not identified as potential failure sites by SW.

(f) Debris flow runout distances were predicted for the six potential failure sites based on estimated landslide volumes. The estimates were based on empirical formulae proposed by Corominas (1996) and Lau & Woods (1997). The calculated runouts for Site 1 with predicted failure volume of 400 m^3 above Leung King Estate terminated some 300 m northwest of the boundary of the estate.

Using the same empirical formulae used by SW, HCL calculated the debris flow runout distances for Landslides B and C. The actual runout distance of Landslide B was 361 m, compared to 79 m and 205 m predicted using the Corominas and Lau & Woods equations respectively. The actual runout distance of Landslide C was 137 m, compared to 25 m and 68 m predicted using the Corominas and Lau & Woods equations respectively.

The conclusion of the SW report is that the risk of debris from natural hillside landslides (assumed to mean open hillside failures) affecting the western boundary of the site (the hillside to the northwest of Leung King Estate) is considered to be low. The report recognized that channelised debris flows may potentially affect the site and recommended that debris/rock traps and mesh nets be constructed in two of the streamcourses located to the south of Leung King Estate.

4. DESCRIPTION OF THE LANDSLIDES

4.1 General

Oblique aerial photographs of Landslides B and C, together with minor Landslides B1, C1, C2 and C3, are shown in Plates 1 and 2. Photographs showing the detailed features of the landslides, including their respective debris trails, are presented as Plates 7 to 44. Plans and sections for Landslides B and C are presented as Figures 5 to 11 and Figures 12 to 15 respectively. Details of the volumes of both landslides and their respective debris trails are given in Appendix B.

4.2 Landslide B

The total volume of Landslide B (Plates 7 to 10) is estimated to be about 600 m^3 (source volume 450 m^3 and entrained volume 150 m^3). The failure occurred at the head of natural drainage line DC3 below an exposed granite cliff (Figure 2 and Plate 11). Figure 5 shows a plan of the landslide. The source area of the failure was essentially eye-shaped, with a concave west flank (Plate 7), a slightly curved and stepped floor (Plate 8) and a near-vertical relatively linear eastern flank striking down dip to the floor of the landslide (Plate 9). The source was about 13 m wide, 30 m long with a maximum depth of about 3 m as measured normal to the surface of rupture. The inclination of the main scarp ranged from about 85° (on the eastern flank) to about 60° below the crown and about 30° on the western flank. The inclination of the floor of the landslide was typically 32° to 50° (Figures 5 and 6 and Plates 9 and 10).

The exposed geology of the landslide is shown on Figure 5, and cross-sections through the landslide site are shown in Figures 6 and 7 (Sections A-A and B-B). The material exposed on the surface of rupture comprised a near continuous mantle of colluvium, up to 2 m thick, underlain by a weathered sequence of granite ranging from residual soil on the western flank to slightly decomposed granite in the centre of the floor of the landslide. Two prominent quartz veins and a minor fault cut the granite and are exposed on the surface of rupture. The geomorphology and geology of the landslide is described in detail in Section 5.

The surface of rupture generally follows a series of undulating joints orientated sub-parallel with the ground surface (Plate 8). These joints are probably sheeting fractures resulting from unloading as part of general hillside development as discussed in Section 5. The joints are generally dilated, with sediment infills and showing signs of downhill movement (Plates 12 to 14), which is discussed in detail in Section 5.6.1. The exposed surfaces of all but the basal sheeting joint are often highly fractured, generally forming an orthogonal pattern (Plate 15), though at two locations a well developed partially radial pattern was observed (Plate 16). The nature of the jointing and fracturing is described in detail in Section 5.6.1.

The release surface along the western flank of the landslide (Plate 7) consists of a curved detachment surface within the underlying colluvium, residual soil and CDG. There is no evidence that this surface follows any pre-defined geological structure. A convex break of slope exposed on the surface of rupture running generally parallel to and between 1 m and 3 m below the western main scarp, probably marks the boundary of CDG and residual soil (Figures 5 and 7 and Plate 7).

The release surface of the eastern main scarp follows a series of sub-vertical joints sets (Plates 17 and 18). The rock mass exposed on the surface of rupture contains dilated and generally sediment-infilled joints.

At the crown of the landslide, while the jointed rock mass has failed, the overlying colluvium has remained insitu, thereby forming a cave-like structure that extends about 1.5 m into the hillside above (Plate 19). It is likely that tree roots within the colluvium layer effectively reinforced the material, thereby preventing the roof from collapsing.

A total of seven possible erosion pipes were exposed on the surface of rupture. No seepage was observed from any of these pipes during inspections by HCL, though the presence of minor local erosion below a number of the pipes indicates possible post-failure water flow (Plate 20). The pipes were located within the following material types:

(a) small diameter (about 20 mm) rounded pipes within the colluvium, residual soil and CDG exposed in the main scarp (Plate 20);

(b) irregular flat openings (about 20 to 30 mm) within the sandy clay infilled joints (Plate 21); and

(c) a relatively large (about 60 mm) rounded pipe (one number) located at the intersection of two joints (Plate 22).

No tension cracks or other signs of distress were observed above the main scarp of the landslide.

It is pertinent that during the initial inspection of the source area by Fugro Maunsell Scott Wilson Joint Venture (FMSW) on 3 May 2000 (HCL had not been engaged at the time), active seepage was observed emanating from the location of the sheeting joints. It should be noted that 105 mm of rain was recorded at raingauge No. N07 the day before. While no seepage was noted in the many site inspections between August 2000 and November 2000 undertaken by HCL, a minor seepage from the sheeting joints was noted in the centre of the landslide on the 24 October 2000 (Plate 23 and Figure 20), some four days after a heavy rainstorm.

Two separate lobes of debris were retained on the surface of rupture of the landslide (Figure 5 and Plate 10). The larger lobe, estimated to be about 50 m^3 in volume, rested on the eastern side of the surface of rupture and consisted predominantly of angular cobble and boulder-sized fragments of granite and quartz in a matrix of yellowish brown silty sand. Two partially intact blocks of largely intact material were visible within the debris. The vegetation on these intact blocks was noted to match that at the crown of the landslide. It is thought likely that these blocks may have became displaced sometime after the initial failure. A second lobe, with an estimated volume of 5 m^3, was located close to the western flank of the landslide. The debris consisted almost entirely of remoulded yellowish brown sandy silt derived from CDG and residual soils exposed on the western flank of the landslide. Five isolated partially intact blocks of displaced material, consisting of colluvium held together by vegetation, were located within the debris.

A minor failure, Landslide B1, is located to the east of Landslide B (Plate 24). The source volume of the landslide is estimated to be about 30 m^3. The failure occurred at the head of a minor, poorly defined topographical hollow to the east of DC3. Figure 5 shows a plan of the landslide. The failure consisted of a shallow western flank, about 0.5 m deep, and a steep 2 m deep eastern flank. The source was about 6 m wide, 7 m long with a maximum depth of about 2 m, measured normal to the surface of rupture. The inclination of the surface of rupture ranged from about 70° on the main scarp below the crown to about 40° on the floor of the landslide. Two minor recent tension cracks extend about 1 m to the east from the source of the landslide (Figure 5). Both cracks were open up to 30 mm wide and showed no signs of vegetation growth on their surfaces, indicating that they were probably formed at the same time as the landslide.

The exposed geology of the landslide source is shown on Figure 5, and a cross-section through the landslide site is shown in Figure 8 (Section C-C). The material exposed in the main scarp comprised a continuous layer of bouldery colluvium and a small patch of CDG was exposed on the floor of the landslide. Two erosion pipes, both about 20 mm in diameter, were exposed on the eastern scarp of the landslide within the colluvium. The geology of the landslide is described in detail in Section 5.6.

A small lobe of debris, estimated to be about 2 m^3 in volume was retained on the floor of the landslide. The debris consists of cobbles and boulder-sized blocks of sub-angular granite and quartz fragments in a remoulded matrix of yellowish brown sandy silt.

Directly below Landslide B1, a strip of abraded ground with a minor eroded channel, representing a zone of transportation, leads downhill to the eastern flank of Landslide B. At this point the debris from Landslide B1 merges with, and becomes essentially indistinguishable from the debris derived from Landslide B. A discussion of the relative timing of Landslides B and B1 is included in Section 8.1.1.

4.3 Debris Trail from Landslide B

Details of the debris trail below Landslide B are shown in Figures 9 and 10, and a section along the natural drainage line from the source area of Landslide B to Leung King Estate showing the relative zones of erosion, deposition and transportation is shown in

Figure 11. Plates 25 to 31 show details of the debris trail. Details of the erosion and deposition volumes along the debris trail are presented in Appendix B.

The debris from Landslide B was initially funnelled into an incised channel close to the head of natural drainage line DC3 (Chainages 40 to 65). Minor deposits of fine debris, consisting of silty sand with gravel, were deposited on both flanks of the channel some 3 m above the stream bed. Substantial erosion has occurred on the base of the channel within the highly decomposed granite forming the substrate. The complex curved and locally deeply incised form of the eroded channel indicates that it was likely formed by high velocity surface water flow, rather than by entrainment during the debris flow.

About 90 m^3 of debris was deposited in a 40 m long measured section (Plate 26), below the eroded channel described above (Chainages 65 to 95). At the time of inspection, the debris on the western flank comprised unsorted cobbles in a silty sand matrix with occasional boulders, while the debris on the eastern flank generally comprised flow sorted cobbles and boulders. A linear zone of transportation, indicated by flattened vegetation and an abraded ground surface, run along the western flank of the channel on the edge of the debris deposit. The morphology of the debris, consisting of graded debris on the western flank and flow sorted debris within the eastern channel bed and traces of graded debris on the upper eastern flank, could have resulted from the formation and later breaching of a debris dam. Figure 16 presents a schematic representation of the likely sequence of development and breaching of the dam. Landslide debris most likely initially formed a dam within the natural drainage line between about Chainages 65 and 95, corresponding to a travel angle of about 27°. This probably reflected a reduction in the momentum of the debris as the pre-existing channel locally widened, from about 4 m to about 8 m, and became shallower, from about 22° to between 6° and 16°. The gradient of the distal end of the dam increases to about 22°. Overtopping of the blocked drainage line resulted in surface water flow over the temporary dam thereby forming the zones of transportation found on the western flank. The eastern side of the dam probably failed later, with the subsequent flow sorting the debris and depositing cobbles and boulders in the stream bed (Plate 26). The volume of the original pre-breach dam is estimated to have been between 300 m^3 and 350 m^3, which would have been a substantial proportion of the source volume of Landslide B.

Between Chainages 95 and 210 is a series of zones of erosion and transportation. Generally erosion is restricted to an area where the substrate is colluvium. Along this section the drainage line ranges from between 5 m and 8 m wide, with an average gradient of about 22°. From Chainage 135 onwards the drainage line curves around a 100° bend, causing undercutting of the western flank of the drainage line and resulting in a series of minor failures within the colluvium forming the western flank of the drainage line (Plate 27). The debris from these minor failures partly infilled the drainage line at Chainage 145 (Plate 27). On the flanks of the drainage line, minor deposits of debris mark the super-elevation of the debris flow. The differential in height of the debris reaches a maximum of 12° at about Chainage 140. This super-elevation suggests that the velocity of the debris flow, which was probably partially impeded by existing boulders within the drainage line, was about 8 m/s. It was not possible to measure super-elevation accurately at any other location along the drainage line.

Below a minor waterfall located on a prominent concave break of slope that traverses the hillside is a large zone of deposition (Chainages 210 to 240), with an estimated debris volume of about 130 m^3 (Plate 28). The distal end of the debris deposit is offset from the main drainage line and follows a slight topographical hollow. The gradient of the drainage line drops from about 18° to about 8° below the waterfall, and its width increases from about 5 m to over 15 m. The debris generally comprises loose boulders and cobbles underlain by unsorted cobbles and boulders in a silty sand matrix. This may indicate that the surface of the original debris deposit was affected by post-failure washout removing the fines.

From about Chainages 225 to 285 the debris flow becomes confined to a linear incised channel. The channel is eroded, up to 2 m in places, into the colluvium substrate, generally about 3 m wide with a gradient of about 8°. Boulders within the colluvium, up to about 2 m in size, have been exposed (Plate 29). Mature vegetation on the flanks of the channel indicates that the channel was at least partly formed sometime prior to the recent failure. Below about Chainage 250, weathered volcanic rock is visible in the floor of the channel underlying the colluvium.

From Chainage 310 onwards, the channel becomes poorly defined causing the debris to spread over a wide (about 20 m) section of gently inclined hillside with a gradient of about 7° (Plate 30). An elongated levee of unsorted cobbles and boulders in a silty sand matrix marks the western extent of this deposit, while a smaller lobe of rounded, sorted cobbles and boulders forms a sheet deposit over a broad, almost level area (Plate 31). The rounded nature of this material suggests that it consists mostly of material entrained from previously transported and abraded colluvium substrate rather than ‘first-time’ debris from the source of Landslide B. A series of poorly defined, partly anastomosing channels cross the area.

The toe of this deposit (Chainage 360) is taken as the distal end of the debris flow. The horizontal travel distance of the debris flow is approximately 360 m from the crown of Landslide B to the toe of the debris, ignoring the subsequent outwash. The travel angle of the debris flow, determined after Wong & Ho (1996), corresponding to the inclination of the line that joins the distal end of the debris flow (disregarding the outwash material) and the crest of Landslide B, is about 17°. For the debris volume involved, this is at the lower end of typical values that have been recorded in Hong Kong, indicating a fairly mobile failure (Wong & Ho, 1996). One explanation for such a mobile failure could be as a result of a sudden breach of a debris dam by impounded water (as deduced in this case), resulting in a debris flow involving fluidised debris. With reference to the alert criteria proposed in Ng et al (2000) for understanding hazard assessment of natural hillsides for proposed developments, the debris of this channelised debris flow terminated about 160 m beyond the point where the angular elevation of the channel bed decreased to 15° or less.

The material deposited below Chainage 360, including the debris that inundated Leung King Estate, is most likely outwash material transported by subsequent overland flow. The extent of the outwash material and its effects on Leung King Estate are described in Section 4.6 below.

4.4 Landslide C

The total volume of Landslide C is estimated to be about 70 m^3 (source volume 60 m^3 and entrained volume 10 m^3). The failure occurred on an existing oversteep cut slope inclined at about 60° and about 4 m high, that was formed during mining activities prior to

1945 (Section 3.2 and Figure A2). Figure 12 shows a plan of the landslide. The relatively shallow failure was essentially spoon-shaped, indicating a partially rotational form of failure. The source was about 11 m wide, 7 m long with a maximum depth, measured normal to the surface of rupture, of about 1.5 m (Plates 32 to 33). The inclination of the surface of rupture ranged from about 70° to 80° on the main scarp, to about 30° on the floor of the landslide.

The exposed geology of the landslide is shown on Figure 12 and a cross-section through the landslide site is shown in Figure 13 (Section E-E). The material exposed on the surface of rupture comprised a mantle of colluvium about 1 m thick over the main scarp of the landslide, underlain by weathered granites ranging from CDG on the western flank to moderately to highly decomposed granite on the eastern flank of the landslide.

An intact but displaced block remains below the eastern main scarp of the landslide (Plate 33). The estimated volume of the mass is about 2 m^3 and it has been displaced by about 150 mm. Two lobes of debris were retained on the floor of the landslide. The smaller lobe, estimated to be about 1 m^3 in volume, is located close to the eastern flank. A larger dumb bell shaped lobe, estimated to be about 3 m^3, covers the lower part of the failure and part of the drainage line below. The debris in both lobes comprises yellowish brown silty sand with angular cobble- and boulder-sized fragments of granite and quartz.

Two erosion pipes, about 60 mm and 80 mm in diameter, are exposed in the main scarp (Figure 12). A V-shaped erosion feature below one of the pipes (Plate 34) is likely to have been formed by post-failure water flow, though no seepage was observed during HCL's inspections. An inspection of the hillside above the source, did not identify any signs of distress or tension cracks.

4.5 Debris Trail from Landslide C

Details of the debris trail below Landslide C are shown in Figure 14, and a section along the natural drainage line from the source of Landslide C to Leung King Estate is shown in Figure 15. Plates 35 to 40 show details of the debris trail. Details of the deposition, transportation and erosion volumes along the debris trail are presented in Appendix B.

The channelised debris flow from Landslide C initially entrained the relatively loose mining waste that had been end-tipped downslope of the cut slope, forming an incised eroded channel (Plate 35). The channel is about 3 m wide and about 1.5 m deep, with a gradient of about 32°. The mining waste consists of angular cobbles and boulders of quartz and granite in a gravelly sand matrix. Traces of molydbenite are common within the debris. The debris trail crosses a second cut slope at about Chainage 45. Below this second cut slope the gradient of the hillside locally decreases to about 14°. A significant proportion of the debris from landslide C, with an estimated volume of 40 m^3, was deposited in an elongated lobe (Plate 36) along this section between Chainages 45 and 60. The base of the debris deposit bifurcates, with one lobe of debris accumulating within a minor topographical hollow to the east of the main debris flow (Plate 2). The debris generally consists of cobbles and boulders of granite and quartz in a gravelly sand matrix.

From Chainages 60 to 100, the debris flow eroded a near continuous entrained shallow channel within the colluvium and mining waste substrate, flanked by parallel levees of debris

(Plate 37). The natural drainage line is poorly defined along this section and only about 2 m wide with a gradient reducing from about 28° in its upper part to about 18° towards the base. The debris generally consists of silty gravelly sand with occasional cobbles of granite and quartz. The continuous debris lobe terminates at Chainage 106 where a 1 m high, 7 m long man-made random rubble retaining wall is located, which is founded on exposed granite rock (Plate 38). The channel behind the wall is infilled with colluvium and landslide debris forming an almost level area with a gradient of less than 5°. Below the wall, there is a sharp increase in gradient to about 32°, locally greater than 40°, as the drainage line travels over the exposed rock. No debris was deposited along this section.

Beyond Chainage 115, the drainage line becomes better defined and is about 4 m wide with a gradient of about 20°. Discontinuous lobes of debris are deposited within the drainage line. The debris consists of silty sand with occasional angular cobble-sized rock fragments. The material does not appear to have been sorted, indicating that it was not deposited as a result of outwash. Obvious signs of debris transportation (flattened vegetation and debris smears) are clearly visible along much of the drainage line.

Below the 65 mPD contour at about Chainage 170, the gradient of the drainage line increases abruptly from 20° to about 37° as it travels across an area of exposed granite. Beyond this point only occasional small deposits of debris, generally silty sand, are found within the natural drainage lines. This material appears to be flow sorted and is probably alluvial outwash from the debris above.

At about Chainage 185 a minor failure, Landslide C3 (Plate 39), is located below the area of exposed rock above the eastern flank of the channel. The failure volume of the landslide is estimated to be about 40 m^3. The failure occurred within what is probably mining waste. The majority of the landslide debris was deposited directly below the source area. The debris is composed of silty gravelly sand with occasional angular cobbles of quartz and granite.

A second minor failure, Landslide C4 (Figure 2 and Plate 40), is located above the eastern flank of the channels at about Chainage 290. The failure volume of the landslide is estimated to be about 5 m^3, and, similar to Landslide C3, occurred within what appears to be mining waste. All the debris from the failure was deposited directly below the source.

The horizontal travel distance of the channelised debris flow is approximately 137 m from the crown of Landslide C to the toe of the debris, at Chainage 165, ignoring the subsequent alluvial outwash. The travel angle of the debris flow, determined after Wong & Ho (1996), corresponding to the inclination of the line that joins the distal end of the debris flow (disregarding the outwash material) and the crest of Landslide C, is about 26°. For the volume of material involved, this debris flow was not particularly mobile.

With reference to the alert criteria in Ng et al (2000), the debris terminated some 40 m uphill of the point where the angular elevation of the channel bed decreased to 15° or less.

The material deposited below Chainage 170, including the debris that inundated the perimeter road of Leung King Estate, is most likely alluvial outwash transported by subsequent overland flow. The effect of this material on Leung King Estate is described in Section 4.6 below.

4.6 Observations Made Following the Landslides

At about 6:40 a.m. on 14 April 2000, flooding at San Wai Court (Figure 2) was reported. At about 7:00 a.m., "muddy" water was observed flowing down the southwest portion of the Leung King Estate perimeter road and through Tuen Mun Primary School and San Wai Court, at the southern corner of the estate. San Wai Court was flooded with 0.5 m deep "muddy" water. At around 8:00 a.m., the water flowing down the road became "clear". A bridge at the western corner of the estate (Figure 2) was blocked with vegetation and soil debris. No information is available on the timing and condition of discharge from the three stepped channels at slope No. 5SE-B/CR26. Therefore, the actual timing, pulses (if any) and duration of Landslides B and C could not be determined.

The landslide incident was reported to the Housing Department (HD) Management Branch by the Leung King Estate Management Office at about 2:00 p.m. on 14 April 2000. The Term Geotechnical Consultant of HD, Maunsell Consultants Asia Ltd. (MCA), carried out an emergency inspection at about 4:00 p.m. on the same day.

At the time of MCA's inspection, debris was being removed from the northwest portion of the Leung King Estate perimeter road by HD's Contractors (Plate 3). The debris had blocked the concrete U-channels that intercept natural drainage lines DC3A and DC3 at the toe of the hillside below Landslide B (Figure 2 and Plate 4), and the channelised section of natural drainage line DC4 at the toe of the hillside below Landslide C (Figure 2 and Plate 5). Further debris had overflowed from the surface drainage channels onto the perimeter road and the adjoining playground. An area of approximately 2 hectares was covered with an estimated 300 m^3 of outwash material (Figure 2 & Plate 6). Examination of the landslide debris located adjacent to the blocked channels showed it to be composed of unsorted sub-rounded to angular gravel and cobbles in a sandy to silty clay matrix, with sizeable boulders over 300 mm diameter. No inspection of the landslide scars was carried out by MCA.

The GEO was informed of the landslide at 5:25 p.m. on 14 April 2000. The Mainland West Division of the GEO carried out an inspection on 18 April 2000. According to the GEO Incident Report (GEO Incident No. MW2000/04/018), the incident affected open space and carparks, caused the blockage of one road lane and displaced a parked vehicle causing minor damage. No inspection of the landslide scars was carried out by the GEO on that day. The GEO Incident Report noted the time of failure as 8:00 a.m. on 14 April 2000.

The landslide investigation consultant, FMSW engaged by the GEO, carried out an inspection of the landslides on 17 April 2000 (HCL had not been engaged as Landslide Investigation Consultants at this stage). Most of the observations of the landslides were made from the top of Leung Kit House and Leung Chun House located at the northwest side the Leung King Estate (Figure 2).

On 2 May 2000, the Planning Division of the GEO carried out a preliminary inspection of five failures that occurred above natural drainage line DC6 (Figure A1, Appendix A).

The inspection noted that there was no debris present in the natural drainage line that could pose an immediate hazard to the development below.

The FMSW landslide investigation team carried out a second, and more detailed, inspection on 3 May 2000, which included preliminary mapping of the source areas and debris trails of Landslides B and C. Active seepage was observed to be emanating from the basal rupture surface of Landslide B (it should be noted that 105 mm of rainfall was recorded at raingauge No. N07 on 2 May 2000). No seepage was observed from Landslide B1 or Landslide C at that time.

4.7 Urgent Repair Works

Urgent repair works carried out by HKHA's contractors comprised the removal of landslide debris from the perimeter road and erection of four 6.5 m high, temporary safety barriers along the perimeter road, below the outlets of the natural drainage lines DC3A, DC3, DC4 and DC5. During the emergency inspection, MCA recommended that the Leung King Estate perimeter road be totally fenced off during red and black rainstorm warnings, and sandbag barriers be erected around the catchpit/sand traps along the perimeter road until they could be replaced by proper safety fences.

5. GEOMORPHOLOGY, GEOLOGY AND HYDROGEOLOGY OF THE LANDSLIDE SITES

5.1 General

The ground conditions at the site were determined using information obtained from desk and field studies. The desk study included a detailed API supplemented by a review of the relevant documentation. The field studies included detailed post-failure geomorphological and geological mapping as well as limited ground investigation.

5.2 Previous Ground Investigation

The only previous ground investigation work within the site was carried out some 50 m downhill of the source areas of Landslides B and C. The majority of the boreholes were carried out as part of the development of Leung King Estate in the early 1980s (SWKP 1981, 1985a). These existing boreholes generally confirmed the local geology as shown in Figure 3. The locations of the boreholes are shown on Figure 17.

5.3 Current Ground Investigation

Limited ground investigation work was carried out at the locations of Landslides B and C by the GEO's ground investigation term contractor, Vibro (HK) Ltd., between October 2000 and November 2000. The results of the GI are reported in the Final Field Work Report for Leung King Estate (Vibro, 2000).

The ground investigation at Landslide B comprised four trial pits, one trial trench and three surface strips, whilst the ground investigation at Landslide C comprised four trial pits and two surface strips. The findings of the ground investigation are described in detail in Section 5.6.

5.4 Laboratory Testing

During the GI, bulk samples in the trial pits and jar samples of the sediment infill from various locations across the surface of rupture of Landslides B were taken. Classification and index tests were carried out at the Public Works Central Laboratory of Civil Engineering Department (Table 1).

Atterberg limits and PSD tests were carried out in accordance with GEO Report 36 (Chen, 1994). PSD tests of the colluvium matrix at Landslide B indicate a uniform deposit, with between 50% and 53% of sand and gravel, between 28% and 30% silt and between 19% and 20% clay. The Plasticity Index ranges between 30% and 37%. PSD tests of the colluvium at Landslide C also indicate a generally uniform deposit, with between 69% and 75% of sand and gravel, between 18% and 24% silt and 7% clay. It should be noted that cobble- and boulder-sized rock fragments are also present within the colluvium deposits.

The mining waste encountered around Landslide C comprised predominantly of a silty sand, with between 61% and 81% of sand and gravel, between 18% and 32% silt and between 1% and 7% clay. The Plasticity Index ranges between 24% and 36%.

The sedimentary infills taken from the open joints at Landslide B ranged from a sandy silt to a silty clay, with between 15% and 60% of sand and gravel, between 25% and 44% silt and between 11% and 35% clay. The Plasticity Index ranges between 21% and 33%.

5.5 Geomorphology

The main geomorphological features of the site, interpreted from the 1949 and 1999 aerial photographs, are shown on Figure A1 in Appendix A. Figure A2 shows details of the mining activities in the vicinity of Landslide C (based on the 1949 aerial photographs). Figure A3 shows the details of the geomorphology and interpretive observations in the vicinity of Landslide B (based on the 1973 low-level aerial photographs). Figure A4 shows the extent of hillfires identified from available aerial photographs.

The landslide site is in the most part located on a southeast-facing hillside at the eastern boundary of the Tsing Shan Range. A series of generally minor northwest to southeast trending natural drainage lines, labelled DC1 to DC6 on Figure A1, are located within the site. Subsidiary drainage lines are labelled DC1A, DC1B, etc. on Figure A. A single prominent valley (DC6) dominates the northeastern end of the site; the valley has a similar orientation to the smaller drainage lines to the southwest. While the drainage lines are generally shallow (2 m to 3 m deep) and have a rounded profile in the upper section of the hillside, they become more incised (4 m to 5 m deep) below a concave break of slope (Figure A1) that follows the faulted contact between the granite and tuffs (Figure 3). The average slope angle across this break of slope changes from about 20° above to about 10° below.

Exposures of rock are present within the following locations within the site (Figure A1):

(a) as steep cliffs generally located below ridgelines,

(b) along irregular southwest to northeast trending bands that traverse the hillside which tend to follow contours, and

(c) along the beds of natural drainage lines.

Colluvium deposits were identified over much of the hillside. These deposits generally occur as:

(a) lobes of bouldery, often angular, material visible below the cliffs (talus deposits),

(b) elongated deposits following the natural drainage lines,

(c) boulder fields and areas of hummocky ground on the open hillside, and

(d) a near continuous mantle that covers the lower section of the hillside.

Prominent areas of surface erosion, originally observed in the 1949 aerial photographs and generally still visible on the 2000 aerial photographs, are located adjacent to the ridgeline that forms the northwest boundary of the site (Figure A1). Other smaller areas of erosion appear to coincide with the alignment of abandoned footpaths, formed prior to 1949, that traverse the hillside (Figure A1).

Both the recent and relict landslides have generally occurred in hillside with similar geomorphological landform characteristics. Typically the landslides are found within the following site setting:

(a) below steep exposed rock cliffs,

(b) on hillside with an inclination of between 30° and 40°,

(c) at the heads of natural drainage lines that are incised into superficial deposits or weathered rock, and

(d) associated with colluvium deposits located both within the drainage line and as continuous sheet deposits towards the toe of the hillside.

The reasons for the apparent increase in the propensity of landsliding in this hillside over recent years (see Section 3.4) are not certain. This may possibly be explained by a gradual deterioration of marginal stable areas of hillside, which may have been related to distress caused by the severe rainstorms in 1982 and 1983. Such cycles of deterioration, each resulting in phases of landsliding, could have been ongoing for hundreds, or possibly thousands of years. However, while there is field evidence that such deterioration initiated Landslide B, there is no obvious evidence that other sections of the hillside were similarly affected.

5.5.1 Landslides B and B1

The source area of Landslide B is within an elongated hollow at the head of a minor natural drainage line (DC3) between two rounded spurs (Figure A3). About 20 m above the landslide is a steep granite cliff that rises some 90 m to the ridge-line above. Elongated lobes of bouldery and hummocky ground extend from the foot of the cliff downhill along both flanks of the natural drainage line. These lobes are colluvial deposits derived from the exposed granite cliff above, as established by API together with site observations and post-failure ground investigation (Section 5.6). Many angular boulders, up to 2 m in size, are located below the cliff at the head of the colluvium lobe.

A series of parallel sub-horizontal west to east trending persistent photogeological lineaments (probably sheeting joints) traverse the exposed cliff face. Sparse vegetation is visible along these lineaments. A single curved lineament, possibly a fault, with a broadly northwest to southeast trend appears to intersect the main scarp of Landslide B (Figure A3 and Plate 11). Due to the steepness of the cliff, access was not possible to confirm the orientation of the fault. Site measurements of the orientation of the sheeting joints exposed towards the base of the cliff, indicate that they have a similar orientation to the sheeting joints exposed on the surface of rupture of Landslide B (see Section 5.6.1). Dense vegetation growth is visible along much of the length of this curved lineament, indicating possible seepage. Staining, most likely as a result of seepage or surface water flow, is visible on the cliff face.

On the 1973 aerial photographs, a thin incised gully, with a semi-circular pale-toned area above, is visible within the source of Landslide B (Figure A3). This feature is interpreted to be a natural spring, possibly associated with the fault described above.

The source of landslide B1 is at the head of a second minor topographical hollow to the east of DC3. The failure occurred within an area of colluvium that extends from the cliff above (Figure A3). There is no evidence that the hillside in the vicinity of the sources of either Landslides B or B1 has previously been modified by human activities.

Although there was no obvious large relict landslide scars visible from the available aerial photographs, the extensive colluvial deposits associated with the natural drainage lines and found covering much of the lower portion of the hillside, indicate that the site had been prone to landsliding.

5.5.2 Landslide C

The terrain surrounding the source of Landslide C, as described in Section 3.2 of this report, was in the main formed as a result of human activity (i.e. mining) as opposed to geomorphological processes (Plate 42).

5.6 Geology

The regional geology of the site is described in Section 2.2 and shown in Figure 3. Geological plans and sections for Landslides B are shown in Figures 5 to 8, whilst those for Landslide C are shown in Figures 12 and 13 respectively.

5.6.1 Landslide B

Exposed on the floor of the landslide is a core of strong, light grey speckled black fine-grained slightly decomposed granite. Towards the eastern flank the granite becomes moderately decomposed, while towards the western flank the granite becomes gradually more decomposed, eventually grading into residual soil (Figure 5 and 7). The main scarp of the western flank exposes up to 0.5 m of colluvium overlying the residual soil (Plate 7). The colluvium is composed of stiff orange brown clayey sandy silt with occasional sub-angular gravel and cobble-sized fragments of granite and quartz. A smaller strip of colluvium, composed of angular cobbles and boulders of granite in a silty sand matrix, is exposed on the eastern flank of the landslide (Plate 17 and Figure 18). The surface of rupture is generally within and predominantly controlled by the slightly to moderately decomposed granite.

There is much evidence of disturbance within the rock mass exposed at Landslide B, including open sediment infilled joints (Plate 18), offset joints (Plate 13), offset quartz veins (Plate 12) and fractured rock (Plate 15 and 16). The sediment infill varies from a soft to firm brown slightly silty clay with occasional sand grains common on the exposed floor of the landslide, to medium dense sand with angular rock fragments close to the original ground surface. The apertures of the open joints are typically 5 mm to 20 mm wide, but locally up to 150 mm wide. This disturbed fabric is interpreted as evidence of past movement of the ground, which may be attributed to progressive opening-up of the rock mass and intermittent movements, probably during heavy rainfall.

The jointing within the slightly to moderately decomposed granite exposed on the floor and eastern flank of the landslide, is generally closely-spaced becoming very closely-spaced in places. Five separate joints sets (J1 to J5) were identified (Figure 19) as described below:

(a) J1 - these adversely orientated sheeting joints are the most dominant discontinuities involved in the landslide. They are exposed over much of the floor of the landslide, where a series of sub-parallel joints, typically spaced about 0.2 m apart, produce a slab and stepped geometry to the translational surface of rupture through the weathered granite. The joints are undulating with a mean orientation of about 42°/162°, with the dip typically ranging from 30°

to 62°, and dip direction ranging from 119° to 177°. The joints are generally open, often infilled with sediment, and show indications of past downhill movement (Figure 18 and Plate 13). A detailed characterisation of the sheeting joints is presented later in this section.

(b) J2, J3 and J4 - based on the measured data, these comprise a series of sets of similarly orientated joints which form the eastern release surface (Figure 18 and Plate 17) and are also exposed along the floor of the landslide. The joints are generally persistent over at least 8 m, open, generally infilled with sediment and are occasionally offset, indicating possible downhill movement. Typical orientations of the joints sets are 90°/223° (J2), 67°/253° (J3) and 70°/234° (J4).

(c) J5 - this is a set of persistent planar joints that dip into the hillside (Figure 19 and Plate 13). The joints have a typical orientation of 45°/340°. Within the slightly decomposed rock exposed on the floor of the landslide, the joints are generally tight, though they are dilated and sediment infilled towards the original ground surface where exposed on the eastern flank.

In addition to the above five joints sets are two sets of fractures (F1 and F2). These sets are not developed on the basal sheeting joint surface but are visible on the upper surfaces that show signs of downhill movement:

(a) F1 - moderately persistent north to south striking fractures, which are open and infilled with 4 mm to 10 mm of sediment infill, and

(b) F2 - west to east striking fractures which are generally persistent over only 0.1 m to 0.3 m, and generally terminate against the F1 fracture set.

The fracture sets form an orthogonal pattern on the floor of the landslides as shown on Figure 21 and Plate 15. At two locations, other fractures form a partial radial pattern as shown on Figure 21 and Plate 16. Both patterns may be the result of tensile stresses induced in the upper slab as it has been displaced slightly and lost contact over most of the surface during past incremental movements. The radial fractures may reflect point contact on adjacent sets of sliding sheeting joints. The grid look of F1/F2 fractures may reflect general doming and sagging of the upper slab. The extensive nature of these fractures is reflected by the presence of infill, as detailed inspection of the fractures indicates that the fracture aperture is approximately equally open in all orientations.

The roughness of the sheeting joint was characterised on a 0.1 m by 0.1 m grid on the basal surface of rupture exposed in the centre of the floor of the landslide using an 80 mm and a 420 mm plate (Plate 8). Contoured data are plotted on the stereoplots in Figure 22. These data indicate roughness angles of up to about 12° for the failed section, and 28° for the unfailed section, based on measurements taken using the 420 mm plate (Richards & Cowlands, 1982). It should be noted that these are extreme values (for the limited number of measurements) and that the operating roughness value will depend on the dimensions of the sliding slab and the strength of the asperities as discussed later in Section 7. Sections along the centre line were plotted (Figure 23), and the roughness angle (i) was determined for both the unfailed (where the block above was still insitu) and failed sections of the sheeting joint. The roughness angle varied from 9° to 10° for the failed section, and 17° to 18° for the unfailed section, for the 420 mm and 80 mm plates respectively (Figure 23). The wavelength of the surface was about 2 m, with an amplitude of up to 0.15 m.

Rock to rock contact during dilation was observed at the site as illustrated in the sketch of the west face of Trial Trench LKTT1 (Figure 20), and discussed further below. The sketch also illustrates the apparent downhill movement over the sheeting joints, the development of sediment infills within the open joints and the alternating rock to clay and rock to rock contact along the asperities.

The ground investigation identified that the hillside to the west of the landslide was formed of a layer of colluvium, at least 1 m thick divided into a 0.5 m thick upper bouldery layer and 0.5 m thick lower finer layer, overlying CDG or residual soil. The CDG shows few relict joints and there is no evidence of downhill movement. The depth of weathering of the granite in this area is at least 2 m. Surface strip LKSS1 shows that uphill towards the cliff, the colluvium becomes noticeably coarser, ranging from a sand with occasional gravel and cobble-sized rock fragments at the start of the strip (145 mPD), to angular granite boulders up to 2 m in size below the cliff at the top of the strip (165 mPD). A similar sequence is found along surface strip LKSS2 above the crown, and along LKSS3 to the east of the landslide.

Trial pit LKTP3 located on the hillside between the source areas of Landslides B and B1 was the only trial pit to expose moderately decomposed granite. Unlike the rock mass exposed on the surface of rupture of Landslide B, the rock mass did not show the same signs of downhill movement. Trial pit LKTP4 located on the surface of rupture of Landslide B1 exposed a thin layer of bouldery colluvium above at least 2.5 m of CDG. No discernible relict structures are visible within the CDG.

5.6.2 Landslide C

A plan showing the geology of Landslide C is shown in Figure 12, and a geological cross-section through the landslide site is shown in Figure 13.

The exposed geology in the main scarp of Landslide C consists of moderately to highly decomposed granite becoming CDG along the western flank overlain by colluvium along the northern section of the main scarp. Mining waste is exposed at the toe of the failure and within the drainage line below.

The weathered granite is generally moderately weak to moderately strong becoming weak along the western flank, light brown mottled light grey and fine-grained. A single joint set was recorded dipping out of the surface of rupture. The joint set is adversely orientated with a typical orientation of about 60°/172°. The upper section of the surface of rupture scar appears to have been controlled by, and follow, this joint orientation. Two prominent quartz veins cut the granite, both dipping into the hillside at about 62°/340°. In addition, a number of irregularly trending thin quartz veins cut through the weathered rock.

The colluvium has a maximum thickness of about 0.8 m and consists of cobbles and boulders of granite and quartz in a silty sand matrix close to surface, becoming a silty sand with occasional gravel-sized fragments at the base.

The mining waste consists of loose angular cobbles and boulders of quartz and occasional granite in a sandy gravel matrix. Traces of molybdenite are common within the mining waste (Plate 41). The Hong Kong Geological Survey 1:20,000 Scale Map also shows molybdenite, associated with quartz veins, in the general area of Landslide C (Figure 3).

Two erosion pipes are exposed on the main scarp. The larger pipe (80 mm) is within the colluvium and is partially infilled with silty sand. The smaller pipe (60 mm) is within the CDG and shows signs of erosion caused by surface water flow directly below the opening of the pipe (Plate 34), indicating that significant post-failure water flow from the pipe took place.

No seepage was noted on the surface of rupture, however, the exposed main scarp of the landslide was noted to be consistently moist, even 6 months after the failure.

Two surface strips (LKSS4 and LKSS5) carried out as part of the post-landslide GI were used to expose the cut slopes that traverse the hillside and sections of hillside above and below (Plates 42 and 43). It proved difficult to distinguish between topsoil, colluvium and mining waste in these strips. The two trial pits, LKTP6 and LKTP8 located below the cut slope on which Landslide C occurred, exposed 1.4 m and over 3 m of mining waste respectively. The mining waste was generally loose, orange brown sandy silt with some sub-rounded to sub-angular fine to coarse gravel, cobbles and boulders of quartz and rock fragments. Insitu density tests were carried out within the mining waste indicating a range of dry density from 1.28 Mg/m^3 to 1.36 Mg/m^3.

5.7 Groundwater Conditions

Information on the groundwater regime in the vicinity of the landslides is limited. The closest existing borehole is No. 846D, which is located some 50 m southeast of Landslide B, and was drilled as part of the GI for Leung King Estate (SWKP, 1981). The response zone of the piezometer was between 8 m and 13 m below ground level. The piezometer was monitored over a 27 month period from February 1981 to April 1983 and indicated a ground water level between 6 m and 9 m below ground level.

At Landslide B, a minor seepage was noted to be emanating from the sheeting joints during an inspection of the site by FMSW on 3 May 2000. A minor seepage was also noted during an inspection by HCL on 28 October 2000 following a minor rainstorm that occurred two days previously.

The following geological conditions are thought to have a major influence on the groundwater conditions at the source of Landslide B:

(a) The presence of a persistent photogeological lineament, interpreted as a geological fault, visible on the exposed granite cliff above Landslide B (Section 3.2). The fault, which is not accessible, is liable to concentrate subsurface groundwater flows towards the location of the landslide site. A possible spring, located in the source area of Landslide B that is visible on the 1973 aerial photographs (see Section 3.2 and Figure A3), may be connected to this fault.

(b) The likely increased permeability of the rock mass due to the presence of open joints and erosion pipes.

(c) The presence of quartz veins dipping into the hillside (with a typical orientation of 50°/000°) and cutting across the rock mass. This may result in a zone of decreased permeability thereby forming a partial dam causing an increase in groundwater levels in the uphill rock mass.

(d) The geomorphological setting of the landslide, within a topographical depression below an exposed granite cliff. Minimal infiltration would occur on the exposed rock forming the cliff, resulting in concentrated surface water flows over the colluvium deposits and disturbed rock mass below the cliff and potentially high local concentrated direct infiltration.

A significant influence on the groundwater conditions in the vicinity of Landslide C is thought to be the former mining works. The presence of the various cut slopes formed in the disturbed hillside would tend to direct and concentrate surface water flow, while the many mining related excavations (Section 3.2) on the hillside would provide areas susceptible to ponding, thereby enhancing local infiltration. In addition, the extensive deposits of loose mining waste dumped on the hillside would tend to increase surface infiltration. However, as there was little mining waste identified above Landslide C, it is considered that the above is not a major contributory factor to the 2000 landslide there. Located some 20 m above the source area of Landslide C is a large 2 m wide and at least 4 m deep adit excavated at an angle of about 30° from the horizontal into the hillside. During a site inspection on 28 October 2000, this adit was seen to be filled with at least 3 m depth of water. This would tend to significantly affect the local groundwater level close to the adit and may influence the groundwater condition at the location of the landslide. A possible indication of a locally high groundwater table is that the main scarp of Landslide C was still moist several months after the failure.

6. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge No. N07 is located at Tuen Mun Technical

Institute, Tsing Wun Road, about 2 km south of Landslides B and C (Figure 1). The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the GEO.

As there are no eye-witnesses to Landslides B and C, the exact times that the landslides occurred are not known. “Muddy” water (assumed to be the result of failure of one or more of the several landslides that occurred above Leung King Estate), was first noticed flowing down the southwest portion of the perimeter road at about 7:00 a.m. on the morning of 14 April 2000. At about 08:00 a.m., the water flowing down the road became “clear”. For the purposes of rainfall analysis, it has been assumed that the landslides took place at 7:00 a.m. on 14 April 2000, approximately the same time that ‘muddy’ water was first observed flowing down the southwest portion of the perimeter road.

The daily rainfall recorded by raingauge No. N07 for one month preceding the landslides is presented on Figure 24. The daily rainfall figures show that the landslide occurred on a day of fairly heavy rainfall following a mostly dry spell of 10 days. The hourly data for 13 April 2000 and 14 April 2000 (also shown in Figure 24) indicates an intense peak rainfall between 03:00 hours and 07:00 hours of up to 66 mm/hour on 14 April 2000. In addition, the rainfall records indicate that there was 151.5 mm of rainfall after 07:00 hours on 14 April 2000.

Isohyets of rainfall between 07:00 hours on 13 April and 07:00 hours on 14 April 2000 for the whole of Hong Kong (Figure 25) show that the peak rainfall prior to the failures was centred at the landslides site, with a cumulative rainfall of over 250 mm.

Table 2 presents the estimated return periods for the maximum rolling rainfalls recorded at raingauge No. N07, corresponding to selected durations based on historical rainfall data recorded at the Hong Kong Observatory (Lam & Leung, 1994). The maximum rolling 4-hour rainfall between 03:00 hours and 07:00 hours on 14 April 2000 was the most severe (in terms of return period), with a corresponding return period of about 6 years. Whilst it is acknowledged that this simplified method of rainfall analysis does not necessarily give the true return period for a particular site, as several contributory factors are not taken into account (Wong & Ho, 1996b), it does, nonetheless, provide an indication of the likely relative severity of the various rainfall characteristics assessed.

A comparison of the pattern of the rainfall preceding the landslides with that of selected previous major rainstorms affecting the area, as recorded by raingauge No. N07 since its installation in the mid-1980’s, is shown in Figure 26. The rainfall comparison shown on Figure 26 indicates the rainfall preceding the 14 April 2000 landslides was not particularly severe for the landslide site. However, it should be noted that as raingauge No. N07 was located about 2 km from the site, it is probable that the rainfall recorded may not necessarily represent that at the steep hillside above Leung King Estate.

7. THEORETICAL STABILITY ANALYSIS

7.1 Introduction

Landslide B was predominantly a translational failure involving sliding on a pre-existing wavy and rough, persistent sheeting joint through mainly Grade III granite. There is abundant evidence of precursor movement involving dilation of the rock mass and subsequent infilling. The landslide occurred at a time of moderately heavy rainfall. There would have been almost 100% runoff from the cliffs above the site of the failure for a continuous period of several hours, and it is also likely that a geological fault channelled subsurface water flow towards the site of Landslide B.

Analysis has been carried out to identify the likely conditions at the time of failure, taking due account of the geotechnical setting and likely shear resistance along the failure plane.

7.2 Conditions at Failure

The investigation has identified that the failure primarily involved sliding on a persistent wavy joint running roughly parallel to the ground surface, though locally more steeply. Such persistent joints can be regarded as essentially frictional with shear strength derived from a "basic" friction plus a roughness component which forces the failing 'slab' to lift over asperities. In doing so, work is done which is reflected as an additional shear strength component. The roughness component will depend upon the size of the sliding block (scale effect) and strength of the wall rock.

Many of the joints in the failure zone were infilled with sediment, predominantly clay. There was also patchy clay infills along the series of parallel sheeting joints along which the failure took place. Seepage was noted from the sheeting joints after the failure and it is likely that the failure was initiated by cleft water pressures developed in these open joints.

The section selected for sensitivity analysis is Section A-A as shown on Figure 6. A two-dimensional analysis is considered valid considering the presence of persistent steeply dipping release joints striking parallel to the dip of the failure plane in the eastern flank and weak material in the west flank. The closely jointed nature of the Grades III and IV rock above the failure plane means that zero tensile strength is a reasonable assumption for the main scarp. For analysis, effective friction angles have been calculated for different assumed water levels above the maximum depth of failure, given a Factor of Safety of 1.0. Results are presented in Figure C in Appendix C. For the purpose of analysis, the water level was assumed to be parallel with the hillside surface, although in view of the variable geology the actual groundwater profile was likely to have been much more complex.

7.3 Discussion

Figure C1 indicates that at the time of failure the effective friction angle was probably somewhere between 32° (dry state) and 61° (piezometric surface at ground surface). Considering that the effective friction angle of kaolinitic clay, even undisturbed, tends to be lower than 32°, it seems unlikely that the abundant clay infill contributed to the joint controlled shear strength.

Assuming rock to rock contact at the asperities at the time of failure, the possible contribution of basic friction and roughness can be assessed for this case. There are two main models in considering the strength of rock joints. Hencher & Richards (1989) proposed an approach whereby samples of joint are tested to derive a basic friction angle. Following corrections for dilation, the basic friction angle represents that of a planar but naturally textured joint and the probable additional strength due to dilation at the field scale is then added to derive the field shear strength. The second model is that of Barton & Bandis (Barton, 1990). The basic friction angle is that of a saw-cut surface, which is typically 6° or so lower than the basic friction angle derived by testing natural joints following the method of Hencher & Richards. The effect of roughness (corrected for scale and stress level) is added based on an assessment of the joint roughness coefficient (JRC) following an empirical relationship.

Both methods have been used here as presented in Appendix C and shown on Figure C1.

7.4 Hencher & Richards' Approach

Hencher & Richards (1982) reported the results of direct shear testing on natural joints through granite of various weathering grades. They showed that, once corrected for dilation of individual samples, the basic friction was fairly consistent at about 38°. It can be seen from Figure C1 that a rise in water level of about 0.8 m above the deepest part of the failure would be enough to cause the landslide if there were no roughness component in the field shear strength.

Measurement and observation in the field, however, demonstrates the roughness and waviness of the sheeting joints. Over the failed portion (Figures 22 and 23), the extreme of variation of dip from the mean orientation downhill was 12° for a 420 mm plate and 19° for an 80 mm plate. If the roughness contributed an equivalent of 19° to the friction angle because of dilation effect ($\phi_b + i = 57°$), it can be seen from Figure C1 that the piezometric pressure would have needed to be close to the ground surface. Because of the long thin slab length and relatively weak nature of the rock asperities, as well as the fact that the rock mass was already probably displaced (i.e. losing intimate contact except at major waves of asperities), the effective i value was probably lower, probably less than 12° (i.e. $\phi_b + i < 50°$). On this basis, the rise in the water level was 1 m to 2 m above the maximum depth of failure.

7.5 Barton's Approach

Calculations of joint shear strength based on the Barton-Bandis model, as described in Barton (1990), are presented in Appendix C and plotted on Figure C1. Based on field observations and comparison with published roughness profiles for JRC, values for 100 mm length of surface would be about 8 to 12 typically. Using these values, as well as an estimated wall compressive strength of 20 MPa, a likely effective normal stress and assuming a ϕ_b value (saw cut) of 32°, effective friction angles of greater than 55° are calculated. That would have required water pressures rising almost to ground level prior to failure. However, Barton (1993) advises reducing JRC and Joint Compressive Strength (JCS) for scale effects and this would reduce the effective friction value to between 42° and 44°. In that case, the failure could be explained by a rise in water pressure of just over 1 m. Barton (1990) suggests, however, that an additional i value might be added (presumably to account for major waves of asperities - the JRC relating to relatively minor roughness) but gives no guidance as to what to allow for. If an additional 12° was allowed (as measured by the 420 mm plate), then the effective friction angle would become 54° to 56° which would require a rise in water table of about 2.5 m above the surface of rupture to trigger failure.

8. DIAGNOSIS OF THE PROBABLE CAUSES OF THE LANDSLIDES

8.1 Landslide B

8.1.1 Mode and Sequence of the Landslide

The mode of Landslide B involved principally translational failure along a series of persistent, rough and adversely orientated sheeting joints. A persistent set of subvertical joints provided the eastern release surface, forming the deepest part of the landslide. The western release surface consisted of an essentially circular rupture surface through weak residual soils and CDG. The presence of isolated relatively intact rafts of material with vegetation, located within the source area and overlying the main debris lobes, indicates possible occurrence of minor secondary failures after the initial failure.

The mode of Landslide B1 involved a shallow sliding failure within the colluvium, essentially following the colluvium/CDG interface.

There are few indications from the field mapping of the relative sequence of failure of Landslides B and B1. It was observed on site that the eroded channel formed below Landslide B1 appeared to be truncated by the eastern scarp of Landslide B. A likely interpretation of this observation is that Landslide B1 occurred prior to Landslide B. It is possible that the debris from Landslide B1 eroded a channel at the base of Landslide B, thereby removing toe support from the hillside above.

A significant proportion of the landslide debris appears to have initially been deposited some 30 m downhill from the source of the landslide, effectively damming the natural drainage line. The dam was probably breached subsequently, with concentrated surface water flow remobilising the debris and resulting in further entrainment of material from the drainage line below. No evidence was found to indicate the time duration between dam formation and breaching but it was unlikely to have been significant.

8.1.2 Probable Causes of the Landslide

The close correlation between the relatively heavy rainstorm on the night of 13 April 2000 and early morning of 14 April 2000, and the assumed probable timing of Landslide B suggests that the failure was likely to have been triggered by rainfall.

Transient elevated groundwater pressures within the rock mass due to direct infiltration and subsurface seepage via a preferential flow path provided by the possible geological fault exposed in the cliff above probably caused the landslide. On 3 May 2000, at the time of the initial inspection of the landslide and later after periods of intense rainfall, seepage flows were noted from the basal sheeting joints exposed in the scar.

The existence of open, sediment-infilled and displaced joints is evidence of progressive hillside deterioration and past movement of the ground. Such deterioration may have occurred in a series of intermittent phases initiated by successive periods of heavy rainfall. In each phase, groundwater pressures would have developed sufficiently for the peak shear resistance to be overcome temporarily. Movement would have been downhill together with some riding up over asperities (i.e. dilation). In doing so, ground water and cleft water pressures would have been lowered enough for the movement to have been halted. Subsequent flow of water through the open joint system was at low velocity, but sufficient to transport and deposit clays locally. It is possible that concentrated subsurface seepage over many years via the geological fault exposed in the cliff above, may have contributed to the general deterioration of the source area of the landslide.

The failure mechanism of the landslide with complete detachment was largely controlled by the presence of undulating, open and partially sediment-infilled adversely orientated sheeting joints within the hillside. Detailed characterisation of the sheeting joints demonstrates that the shear strength along the joints is controlled by the roughness of the joint (where rock to rock contact is maintained) and not by the shearing resistance of the infill.

Theoretical stability assessment based on back analysis of the failure, assuming probable shear resistance of the sheeting joints obtained from joint characterisation, indicates that an increase in groundwater pressure which is equivalent to a groundwater level probably between 1 m and 2 m above the surface of rupture would be required to initiate failure.

Other factors that probably contributed to the landslide are summarised below:

(a) the presence of voids in the near-surface ground, both open joints and erosion pipes, leading to increased infiltration and promoting the development of cleft water pressure,

(b) the presence of colluvium on the hillside above the landslide site, resulting in increased surface infiltration and subsurface groundwater flow,

(c) the topographical hollow where the landslide was located would tend to concentrate both groundwater and surface water flows,

(d) the presence of quartz veins dipping into the hillside and cutting the rock mass. These may have resulted in a zone of reduced permeability, thereby forming a partial subsurface dam and causing an increase in ground water levels in the uphill rock mass, and

(e) the recent hillfires might have resulted in the removal of vegetation cover and charring of the topsoil. The root system could have decayed and created rootholes which would have enhanced water ingress and promoted deterioration of the hillside. However, it is not possible to establish a direct correlation between the incidence of hillfires and natural hillside landslides under this study.

While a number of the factors described above also most probably contributed to Landslide B1, the most significant cause of the landslide was probably transient elevated groundwater pressures due to the development of a perched water table at the colluvium/CDG interface as a result of surface infiltration.

Much of the landslide debris appears to have been initially deposited within the natural drainage line some 30 m downhill from the source, corresponding to a travel angle of about 27°. At some later time, this "dam" of debris was largely remobilised following 'dam break', becoming channelised and developing into a mobile wet debris flow. The mobility of the debris flow may have been exacerbated by the large volume of water built-up behind the debris prior to the dam breaching and subsequently concentrating surface water flow down the drainage line.

8.2 Landslide C

8.2.1 Mode and Sequence of the Landslide

The mode of Landslide C involved principally a translational sliding failure along a series of adversely orientated joints within the moderately to highly decomposed granite forming the main scarp, becoming a partially rotational slide in the CDG below.

The landslide debris travelled down a minor topographic hollow before becoming channelised on reaching the drainage line below, where it developed into a wet debris flow. The morphology of the landslide debris suggests that the failure occurred essentially in a single stage. However, the debris was significantly eroded by post-failure surface water flow.

8.2.2 Probable Causes of the Landslide

The landslide occurred in a steep 3 m to 4 m high soil cut slope formed during the pre-1945 mining works. The cut slope stands at an angle of between 50° and 60°. The close correlation between the rainfall and the reported time of failure suggests that the landslide was triggered by rainfall.

Transient elevated groundwater pressures within the soil mass probably caused the landslide. Direct infiltration is considered to be the principal source of water ingress into the hillside. Factors contributing to direct infiltration above the source of the landslide are:

(a) the presence of colluvial deposits and mining waste on the hillside above the landslide site,

(b) the presence of mining excavations allowing local ponding of water,

(c) the presence of cut faces on the hillside above the landslide, tending to concentrate surface runoff towards the site of the failure,

(d) the presence of an exposed granite cliff above the hillside which would have tended to concentrate ground water flow,

(e) the presence of erosion pipes within the near-surface ground material promoting concentrated subsurface water flow, and

(f) the recent hillfires might have resulted in the removal of vegetation cover and charring of the topsoil. The root system could have decayed and created rootholes which would have enhanced water ingress and promoted deterioration of the hillside. However, it is not possible to establish a direct correlation between the incidence of hillfires and natural hillside landslides under this study.

9. DISCUSSION

9.1 Landslide B

Based on the detailed post-failure field mapping and GI, there is extensive evidence of precursory movement of at least the upper 4 m of the rock mass surrounding Landslide B. The evidence of past movement comprises dilation of the rock joints and subsequent infilling of the open joints with sediment. Such deterioration probably occurred in recurrent phases, probably initiated by successive periods of heavy rainfall. After the 2000 failure, seepage was noted to be emanating from some of the wavy sheeting joints exposed in the surface of rupture. It is likely that cleft water pressures, which developed in these open joints, contributed to initiating the latter phases of deformation. During each initiating rainstorm, as the water pressure in the joint system increased to a critical level, the available shear resistance would have been overcome and movement was triggered. Because of the waviness of the joints, downhill movement would have been accompanied by dilation of the rock joints with corresponding increase in volume of voids and permeability along the sheeting joint. As a result, there would have been rapid dissipation of the elevated water pressures leading to increased effective stress and frictional resistance, thereby arresting further movement.

There is no real evidence of the time frame for this progressive deterioration. However, the fracturing of the rock slab above the eventual failure surface can be interpreted, at least in part, as being due to prior movements and this has implications on the likely timing of the previous phases of movement. The movements caused a mismatch of the rough surfaces with "draping" of the upper slab over the (now) mismatched lower rock surface. Where only point contacts supported the load, radial fractures developed in the rock slab. Elsewhere in the slab, tensile stresses due to bending led to the development of parallel and orthogonal sets of fractures. All of the upper fractures were subsequently infilled with predominantly clay sediments, probably washed over the surface by relatively low velocity throughflow. It is postulated that the whole process, from initial movement (when the effective friction angle was probably about 60º - see Appendix C), to the final detachment in 2000 (by which time the friction angle had probably reduced by about 10º), probably took hundreds, or possibly thousands, of years.

The observation of progressive hillside deterioration described above is based on subsurface data obtained after the landslide. It is considered unlikely that field mapping prior to the failure would have identified such deterioration given the dense vegetation and colluvial cover over much of the area.

The API did not identify a significant history of instability, in the form of relict landslide scars, either adjacent to the source area of Landslide B or within the natural drainage line below. There was, however, indirect evidence of previous instability associated with the natural drainage line as indicated by the presence of significant accumulations of colluvium both within the drainage line and covering much of the lower section of the hillside. This landslide incident demonstrates the hazard that significant landslides may occur within natural drainage lines that show little or no direct previous evidence of past instability in the form of landslide scars over a period of tens of years in aerial photographs.

A noteworthy characteristic of Landslide B was the interpreted formation of a debris dam located some 30 m below the source area of the landslide. Assuming the dam represents the initial debris lobe resulting from the failure, the debris had an original travel angle of about 27°. The dam appears to have later been breached, presumably due to damming of water behind the debris, resulting in the formation of a wet channelised debris flow, the distal end of which corresponds to a final travel angle of about 17°. It is likely that the eventual mobility of the debris was significantly increased due to the formation, and subsequent breaching, of this dam. With reference to the "alert criteria" as defined in the Natural Terrain Hazard Study: Interim Guidelines in Ng et al (2000), the toe of the debris is located about 160 m beyond the point where the hillside of the bed of the natural drainage line has decreased to 15° or less. The debris mobility was also much greater than that predicted in pevious natural terrain hazard assessments (Section 3.4).

9.2 Landslide C

Landslide C occurred on an unregistered cut slope that was likely to have been formed during mining works carried out on the hillside sometime prior to 1945. It is likely that the objective of the mining was the extraction of molybdenite or associated minerals, as significant traces of the mineral were found within the mining waste that covers much of the hillside. While no records of these specific mining activities were identified, the mining waste visible on the 1945 aerial photograph appears to be relatively fresh, suggesting that the mining possibly occurred during the Japanese occupation of Hong Kong, sometime between 1941 and 1945.

Landslide C provides an example of a landslide that, while initially appearing to be a natural hillside failure, actually occurred on an unregistered (but registerable) man-made slope on a disturbed hillside. Evidence of both the original mining works and subsequent instability of the disturbed hillside, relating to the formation of the cut slopes and accumulation of mining debris on the hillside, are clearly identifiable both from the API and during field inspections. The natural hillside landslide hazard assessment by SW (1999) did not appear to have recognised the evidence of previous mining works. This may have been related to the large scale at which the assessment was carried out, and the fact that the location of mining works was at the boundary of SW's study area.

10. CONCLUSIONS

It is concluded that the landslides that occurred within the hillside above Leung King Estate in the early morning of 14 April 2000 were triggered by rainfall. The actual timing of the failures of the landslides is uncertain.

The geomorphological setting of most of the natural hillside landslides within the site is similar, in that the failures are located generally directly below exposed rock cliffs at the heads of natural drainage lines on steep hillsides (with a gradient of between 30° and 40°) that have a history of instability.

Elevated ground water pressure in adversely orientated sheeting joints was probably a primary cause of Landslide B. Progressive deterioration of the hillside involving opening up of discontinuities and the presence of erosion pipes would have promoted local infiltration. The presence of a geological fault as mapped in the cliff above the landslide site probably increased subsurface seepage via preferential flow paths. The rough sheeting joints, largely forming the surface of rupture, were partially infilled with sediment. However, it was the asperity contact of the rough joints (i.e. rock to rock contact), and not the infill material, that primarily controlled the operational shear strength of the jointed weathered rock in this instance.

The principal cause of Landslide C was probably the build-up of transient elevated water pressures within an unregistered (but registerable) oversteep cut slope which was formed during pre-1945 mining works. The presence of colluvium and mining waste, depressions and adits formed during the mining works which resulted in ponding, and the presence of erosion pipes in the soil mass, would have enhanced water ingress into the ground and were probably contributory factors in the failure.

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LIST OF TABLES

Table 1 - um ry of catio Index Te ul s

Labor tory Sample No.	Material Type	Sample Locat on	D th below Ground Level (m)	Sample Type	Pa ze stribution				L quid Lim (%)	Pla imi (%)
					Gra (%	nd (%)	S (%)	C ay (%)		
1	Clay	--	--	Bulk	3	37	25	35	80	33
2	Clay	--	--	Bulk	10	50	29	11	36	21
3	Clay	--	--	Bulk	6	35	33	26	64	29
4	Clay	--	--	Bulk	1	14	44	41	68	33
5	Colluvium	LKTP1	1	Bulk	13	40	28	19	64	30
6	CDG	LKTP1	3	Bulk	11	46	26	17	65	31
7	Colluvium	LKTP2	1	Bulk	5	45	30	20	67	37
8	CDG	LKTP2	3	Bulk	8	53	31	8	52	33
9	HDG	LKTP3	1	Bulk	21	44	24	11	51	27
10	CDG	LKTP4	2	Bulk	6	57	27	10	47	27
11	Colluvium	LKTP5	1	Bulk	14	61	18	7	38	25
12	Colluvium	LKTP6	1	Bulk	8	61	24	7	37	24
13	CDG	LKTP6	2	Bulk	20	38	27	15	61	33
14	Fill	LKTP7	1	Bulk	7	54	32	7	52	36
15	CDG	LKTP7	3	Bulk	5	57	34	4	52	39
16	Fill	LKTP8	1	Bulk	27	54	18	1	40	24
17	Fill	LKTP8	2	Bulk	14	53	30	3	47	34

Legend:

G omple ly Decompos Gr ni e G ghly Decomposed Gr nite LKTP1 Trial Pit No. 1

Note: Visual descriptions of Soil ples 1-17 are summaried as follows:

Sample - Brownish orange, slightly gravelly, sandy CLAY
Sample 2 - Brownish yellow, slightly gravelly, sandy SILT
Sample 3 - Brownish yellow, slightly gravelly, sandy SILT
Sample - Brownish yellow, slightly gravelly & sandy SILT
Sample - Brownish orange, slightly gravelly, sandy SILT
Sample 6 - Brownish yellow, slightly gravelly, sandy SILT
Sample 7 - Brownish yellow, slightly gravelly, sandy SILT
Sample - Brownish orange, slightly gravelly, sandy SILT
Sample - Brownish orange, slightly gravelly, sandy SILT

Sampl 0 Brownish or nge, sl htly gravelly, sandy SILT
Sampl 11 - Yellowish brown, c ayey, very silty, gravelly SAND
Sampl - Brownish yellow, c ayey, very silty, gravelly SAND
Sampl 13 - Brownish yellow, s ightly ravelly, sandy SILT
Sampl - Yellowish grey, slightly gravelly, sandy SILT
Sampl - Yellowish grey, sl htly gravelly, sandy SILT
Sampl - Yellowish grey, slightly clayey, very silty, very gravelly SAND
Sampl - Yellowish grey, slightly clayey, very silty, gravelly SAND

Table 2 - Maximum Rolling Rainfall at GEO Raingauge No. N07 and Estimated Return Periods for Different Durations Preceding the Landslides of 14 April 2000

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	9	6:05 on 14.4.2000	< 2
15 minutes	24	06:05 on 14.4.2000	< 2
1 hour	75	06:45 on 14.4. 2000	2
2 hours	113	07:00 on 14.4.2000	3
4 hours	172	07:00 on 14.4.2000	6
12 hours	234	07:00 on 14.4.2000	5
24 hours	258	07:00 on 14.4.2000	3
2 days	258	07:00 on 14.4.2000	2
4 days	259	07:00 on 14.4.2000	2
7 days	259	07:00 on 14.4.2000	< 2
15 days	375	07:00 on 14.4.2000	< 2
31 days	375	07:00 on 14.4.2000	< 2

Notes :

(1) Return periods were derived from Table 3 of Technical Note No. 86, using Gumbel's equation (Lam & Leung, 1994).

(2) Maximum rolling rainfall was calculated from 5-minute data.

(3) The use of 5-minute data for durations between 2 hours and 31 days results in better data resolution but may slightly over-estimate the return periods using Lam & Leung (1994)'s data, which are based on hourly and daily rainfall for these duration.

(4) Assumed time of failure at 07:00 on 14 April 2000.