INVESTIGATION OF SOME SELECTED LANDSLIDES IN 1999 (VOLUME 3)

GEO REPORT No. 122

Fugro Maunsell Scott Wilson Joint Venture

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
December 2001

EXPLANATORY NOTE

This GEO Report consists of two Landslide Study Reports on the investigation of selected slope failures that occurred in 1999. The investigations were carried out by Fugro Maunsell Scott Wilson Joint Venture (FMSW) for the Geotechnical Engineering Office as part of the 1999 Landslide Investigation Consultancy.

The LI Consultancies aim to achieve the following objective through the review and study of landslides:

(a) establishment of an improved slope assessment methodology,

(b) identification of slopes requiring follow-up action, and

(c) recommendation of improvement to the Government's slope safety system and current geotechnical engineering practice in Hong Kong.

The Landslide Study Reports prepared by FMSW are presented in two sections in this Report. Their titles are as follows:

The Landslip Investigation Division of the Geotechnical Engineering Office worked closely with the LI Consultants and provided technical input and assistance to the landslide studies.

SECTION 1:
DETAILED STUDY OF THE 24 AUGUST 1999 LANDSLIDE AT TSING YI ROAD TSING YI ISLAND

Fugro Maunsell Scott Wilson Joint Venture

This report was originally produced in September 2000 as GEO Landslide Study Report No. LSR 6/2000

FOREWORD

This report presents the findings of a detailed study of the landslide (GEO Incident No. MW 1999/8/36) that occurred on 24 August 1999 at slope No. 10NE-B/C254 above Tsing Yi Road at the southwestern corner of Tsing Yi Island. Debris from the landslide completely blocked the two-lane road at the toe of the slope, trapping a taxi and a lorry in the northbound carriageway, but there were no casualties.

The key objectives of the detailed study were to document the facts about the landslide, present relevant background information and establish the probable causes of failure. The scope of the study generally comprised site inspections, limited ground investigation, desk study and analysis. Recommendations for follow-up actions are reported separately.

The report was prepared as part of the 1999 Landslide Investigation Consultancy (LIC), for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 101/98. This is one of a series of reports produced during the consultancy by Fugro Maunsell Scott Wilson Joint Venture (FMSW).

Y.C. Koo
Project Director/Fugro Maunsell Scott Wilson
Joint Venture

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1. INTRODUCTION

A landslide (GEO Incident No. MW 1999/8/36) occurred at slope No. 10NE-B/C254 above Tsing Yi Road at the southwestern corner of Tsing Yi Island, at about 8:00 a.m. on 24 August 1999. Debris from the landslide completely blocked the two-lane road at the toe of the slope, trapping a taxi and a lorry in the northbound carriageway (Plate 1 and 2). There were no casualties resulting from the landslide.

Following the landslide, Fugro Maunsell Scott Wilson Joint Venture (FMSW), the 1999 Landslide Investigation Consultants (LIC), commenced a study of the failure on 25 August 1999 for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED) under Agreement No. CE 101/98. This is one of a series of reports produced during the consultancy by FMSW.

The study was carried out by a team comprising members from Fugro (Hong Kong) Ltd. only, so as to avoid a conflict of interest with Scott Wilson who form part of the 1999 LIC joint venture and were responsible for the original design of the failed cut slope.

The key objectives of the study were to document the facts about the landslide, present relevant background information and establish the probable causes of failure. The scope of the study generally comprised site inspections, limited ground investigation, desk study and analysis. 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 relating to the history of the site,

(b) aerial photograph interpretation (API),

(c) topographical survey, geological mapping and detailed observations and measurements at the landslide site,

(d) limited ground investigation works,

(e) analysis of rainfall records, and

(f) diagnosis of the probable causes of the landslide.

2. THE SITE

2.1 Site Description

A location plan and general views of the landslide are shown in Figure 1 and Plates 1 to 4 respectively. A more detailed site plan and typical cross-sections through the landslide site are shown in Figures 2 to 4.

Slope No. 10NE-B/C254 is a 25 m to 50 m high soil cut slope, with occasional rock outcrops, that comprises between 4 and 8 batter slopes, which stand at angles of between 33° and 35° to the horizontal. Slope No. 10NE-B/C254 forms one of a series of cut slopes constructed in association with the formation of Tsing Yi Road South in the late 1970's. The concerned slope is 240 m long and is located at the southwestern corner of Tsing Yi Island. The landslide occurred at the mid- to lower-portion of the northern end of the slope (Figure 2).

The failed slope is densely vegetated with shrubs and mature trees of maximum height about 10 m. No rigid surface protection had been applied to the slope, except for two localised areas of shotcrete cover in the northern portion. Surface drainage channels traverse the slope at a horizontal spacing of about 5 m to 10 m. These discharge into cascade channels that descend the slope at either end and at about the mid-point of the feature. The cascade channels at either end of the feature mark the boundary of adjoining features, with slope No. 10NE-B/C262 to the north and slope No. 10NE-B/C255 to the east (Figure 1). Horizontal drains have been installed along the toe and above the first batter of the northern portion of the slope. Above the crest of the slopes lies moderately sloping (35°) ground, with locally steeper sections (40° to 50°), that rises to a north-south trending ridgeline standing at an elevation of about 70 m above Tsing Yi Road.

2.2 Maintenance Responsibility

According to the "Systematic Identification of Maintenance Responsibility of Slopes in the Territory" (SIMAR) project undertaken by the Lands Department, the maintenance responsibility for slope No. 10NE-B/C254 lies with the Highways Department (HyD).

2.3 Water-carrying Services

There are no water-carrying services present within the failed slope, but a 600 mm diameter fresh water main runs along the footpath at the toe of the slope (Figure 2).

2.4 Site History

2.4.1 General

The site history has been established from an API, together with a review of documentary records. The salient information from the API is summarised below and illustrated in Figures 5 and 6. A more detailed account of the API is presented in Table 1.

Desk study records show that slope No. 10NE-B/C254 was formed in association with the construction of Tsing Yi Road between 1978 and 1980, under PWD Contract No. 524/77. Scott Wilson Kirkpatrick and Partners (SWKP) designed the road as detailed in their design submission, dated October 1980 (Section 2.4.4).

2.4.2 History of Development

Prior to the late 1960's, the area surrounding the landslide site comprised mountainous terrain unaffected by human development. The prominent north-south trending ridgeline identified on site can be seen in the aerial photographs, below and to the west of which lie a number of natural drainage lines that descend the slope in the vicinity of the landslide site. Lobes of bouldery colluvium cover much of the hillside (Figure 5).

Evidence of development in the area surrounding the landslide site can be observed in the 1968 aerial photographs. Excavations for three single-storey structures and an access track have been cut into the hillside in association with quarrying works to the west of the 1999 landslide. The excavations for the middle and most northerly structures (hereafter referred to as the northern and southern structures), when plotted in relation to the alignment of Tsing Yi Road, were located in the mid- to upper-part of the northern portion of slope No. 10NE-B/C254 (Figure 5). The associated cut slopes are steep (60° to 70°), with an estimated maximum cut height of 5 m to 6 m. The excavation for the northern structure has been formed in bouldery colluvium and lies upslope of the location of the 1999 landslide. The excavation for the southern structure lies below a lobe of bouldery colluvium, with a moderately sharp convex break of slope between the two (Figures 4 and 6).

In the 1976 aerial photographs, the area towards the base of the convex break of slope above the excavation for the southern structure has an irregular/stepped surface profile. Two arcing lines can be seen in this area of the hillside, which could be possible tension cracks (Figure 6). The upper line is about 15 m to 20 m long and occurs within the lobe of bouldery colluvium above the convex break of slope. The lower line is more extensive, being about 50 m long and lies at the base of the convex break of slope. Another arcing line about 25 m long, and a localised area of erosion is evident above the northern structure. Post-failure inspection of these areas identified a pre-existing tension crack at the location of the lower arcing line above the southern structure (Section 3.3), but could not locate any surface distress in the other areas.

In December 1978, slope No. 10NE-B/C254 and the adjoining slopes have been cut to their present profiles, but the road along the toe of the slopes has not been formed. The northern portion of slope No. 10NE-B/C254 has been formed through an area of bouldery colluvium below a rocky outcrop. At the toe of the slope in this location is an accumulation of coarse bouldery material. This suggests the possibility of surface instability during excavation.

The 1980 aerial photographs show that road works are largely complete. A comparison between the 1978 and 1980 aerial photographs shows that locally the cut for the northern portion of slope No. 10NE-B/C254 does not extend to the crest drainage channel, such that an area of quasi-natural terrain exists between the top of the cut slope and the crest channel. This is corroborated by a photograph of the slope taken during construction (Plate 5). Furthermore, the excavations for the two structures identified in 1968 were not completely removed during the formation of slope No. 10NE-B/C254, such that areas of fill have been used to make up the slope profile (Figures 3 & 4 and Plate 5). Over the lower northern portion of slope No. 10NE-B/C254, a number of small regularly spaced light coloured strips are visible. These features have been identified on site as horizontal drains, which suggest the presence of high groundwater conditions.

A major landslide, about 70 m wide by 60 m long, is evident in the central portion of slope No. 10NE-B/C262 to the north of the landslide site in 1982 (Section 2.4.3). The extent of instability appears to be confined within the cut slope boundary with no visible signs of distress in slope No. 10NE-B/C254 to the south, as shown by the 1983 aerial photographs.

There are no discernible changes in the appearance of the slope from the remaining photographs, due to the fact that the maturing trees planted over the cut slopes in 1982 have obscured surface details. However, between November 1997 and August 1998, it can be seen that the crest channel above slope No. 10NE-B/C254 has had maintenance works carried out along it, but no signs of slope distress are evident from these photographs. Based on site observations made as part of this study, these works comprised the application of a narrow strip of shotcrete above and below the crest channel.

2.4.3 Previous Landslides

A landslide, GEO Incident No. NT 1982/8/15, was recorded at the lower northern portion of slope No. 10NE-B/C254 on 16 August 1982 (Figure 1). No details of the size or nature of the failure were recorded on the GEO incident report, but the location shown on the incident report coincides with where an erosion scar was observed by the HyD in July 1998 (Section 2.4.7).

A major landslide occurred in the central portion of the adjacent cut slope No. 10NE-B/C262 in 1982 (Figure 7 and Plate 6), which involved a large-scale failure of an area about 70 m wide by 40 m high. In accordance with correspondence from the GEO file records, the landslide had an estimated volume of 10 000 m^3 to 15 000 m^3. The exact time and date of the failure was not known, but was believed to have been triggered by intense rainfall between 28 May and 2 June 1982. The full extent of failure was first recorded during an inspection on 30 June 1982 by representatives from the Geotechnical Control Office (GCO, renamed GEO in 1991), the HyD and the Tsuen Wan New Town Development Office (TWNTDO). At this time, it was noted that the landslide mass had moved downslope by approximately 1 m and, although the displaced mass did not detach from the slope, it posed a significant hazard to the road users. Tension cracking and severe disruption to the surface drainage channels across the disturbed mass were noted. Details of the distress are provided in Hencher (1983). A set of oblique aerial photographs taken in 1983 (Table 1) gives an excellent view of the landslide.

Based on information recorded in the GEO files, the failure did not affect slope No. 10NE-B/C254 to the south. A photograph of the cascade channel at the boundary between the two slopes shows no signs of distress (Plate 7).

Following the failure, SWKP carried out an investigation into the landslide for the purpose of determining the probable causes of the failure and designing remedial works. Ground investigation (GI) works, comprising drillholes and trial pits, were originally proposed as part of SWKP's landslide investigation, but following renewed instability during a rainstorm on 16 August 1982, the trial pits were deleted because of the potential risk involved. The renewed instability comprised further movement with localised sliding and major washout below the southern portion of the main scarp, resulting in material being deposited on the southbound carriageway of Tsing Yi Road. Heavy seepage was observed in the central portion of the failed mass following the rainstorm. In view of the further slope movement, the southbound carriageway of the road was closed to traffic in the vicinity of the failed slope and a steel barrier was constructed.

The revised GI works comprised 7 drillholes, which were undertaken between August and November 1982. Continuous Mazier samples were retrieved in 5 drillholes in an attempt to locate the slip surface(s). The locations of drillholes are shown in Figure 7.

The geotechnical report prepared by SWKP following the 1982 landslide identified that the underlying geology of slope No. 10NE-B/C262 comprised granite that was intersected at about the slope's mid-point by a near-vertical feldspar porphyry dyke trending in an east-west direction. No adverse joint planes dipping out of the slope were identified. The depth to rockhead, defined as Grade III material or better, varied considerably across the failed slope, ranging from about 5 m to 10 m in the upper portion of the landslide to a maximum of about 20 m at the toe of the landslide. This was significantly deeper than that assumed in the original slope design, which assumed a maximum of 7 m below the finished cut slope profile.

Near the dyke, bands of completely to highly decomposed granite (C-HDG) or completely to highly decomposed feldspar porphyry (C-HDFP) alternate with less decomposed materials, forming alternating bands of loose and dense soil that are inferred as being sub-parallel to the slope surface. These alternating layers were not found in areas further away from the dyke. Above the failed section of the slope the depth to rockhead was shallower, being between 1 m and 5 m below ground level. A cross-section through the failed slope indicates a sharp drop in the rockhead profile at the toe of the slope (Figure 8).

A piezometer was installed in each of the drillholes as part of the post-failure investigation, which, together with the one piezometer installed previously in drillhole BH275D, gave a total of eight piezometers. Halcrow buckets do not appear to have been installed in the piezometers. Five of the piezometers were located within the failed area, while the remaining three were located higher upslope. The piezometers in two of the drillholes (BH643D and BH647D) were located in the completely decomposed materials overlying bedrock. These drillholes were located towards the toe of the slope where the depth to rockhead was greater. The piezometers for the remaining drillholes were positioned towards the base of each drillhole within bedrock.

Monitoring was carried out between September 1982 and March 1983 for the newly installed piezometers and from July 1979 to March 1983 for the pre-existing piezometer. Records indicated a moderate response to rainfall in the upper portion of the failed slope with the groundwater table typically rising between 2 m and 3 m following 100 mm of rainfall during a 24-hour period on 2 March 1983 (the main groundwater table remained below rockhead). In contrast, a more pronounced response of between a 5 m and 9 m rise in the groundwater table was recorded by the piezometers installed in the middle to lower portion of the failed slope. The 9 m rise in groundwater level was recorded near the boundary of the dyke. This reading represented a very localised increase, with the groundwater table rising to about 14 m above rockhead (approximately 6.5 m below ground level at that location). This behaviour was attributed by SWKP to the influence of the subvertical dyke, with groundwater "flowing directly along this dyke from other sources".

No discussion was presented in SWKP's report about whether a perched groundwater table existed in the failed slope. However, according to Hencher (1983), "the groundwater table that saturated the slope is a perched groundwater table". The reason for perching was not certain as no aquaclude was recongnised in any of the drillholes, but it was concluded that perching probably occurred at the soil/rock interface. A lower permanent groundwater table was identified within the underlying bedrock (Figure 8).

Hencher (1983) also noted that the lower portion of the failed slope had become saturated, casting doubts as to whether the horizontal drains installed as part of the original design (Section 2.4.4) were effective in lowering the groundwater table.

SWKP considered that there were two dominant factors affecting the stability of the slope, namely "a high groundwater table, probably associated with the feldspar porphyry dykes" and "bands of loose soil to depths of up to 20 m, forming planes of weakness".

The precise location of the failure surface(s) within the slipped area could not be identified from SWKP's landslide investigation, but the geotechnical report noted that "the presence of alternating layers of loose and dense soil does suggest that the failure is likely to be associated with one or more of the loose layers". Accordingly, the failed slope was back-analysed using two assumed slip surfaces within the soil mantle overlying bedrock, namely a deep slip surface of about 14 m maximum depth and a shallower slip surface of about 9 m maximum depth (Figure 8). Back-analyses using soil strength parameters derived from laboratory testing (Table 2), showed that a groundwater table daylighting at the slope toe and with a maximum depth of 8 m below original ground level was required to achieve a Factor of Safety (FOS) of unity for the shallower slip. A deeper groundwater table of maximum depth about 12 m was required to achieve a FOS of unity for the deeper slip.

In view of the moderate slope angle (33° to 34°) and the potential for high groundwater conditions, further cutting back the slope was not considered a feasible slope remedial option. Instead, the report proposed to replace the disturbed soil mass with suitable fill material (compacted to a dry density of not less than 95% Maximum Dry Density), with sub-surface drainage provided by a 500 mm thick drainage blanket at the base of the fill. A row of 1.5 m diameter caissons at 3 m centre to centre spacing and socketed at least 3 m into bedrock was proposed to provide additional toe support. Stability analysis, based on assumed shear strength parameters for compacted fill (Table 2) and a design groundwater table at the base of the recompacted layer, typically 6 m to 8 m below the reprofiled slope surface, showed that the proposed remedial works had a minimum FOS of 1.2.

The design report for the remedial works to the slope No. 10NE-B/C262 was submitted to the GCO on 4 May 1983. The report was certified as being checked by the GCO in a memo from the GCO's Mainland West Division to the Project Manager/TWNTDO, dated 15 March 1985.

Based on API, remedial works to slope No. 10NE-B/C262 were completed by September 1985.

2.4.4 Original Slope Design

Slope No. 10NE-B/C254 was formed as part of PWD Contract No. 524/77, which covered cut slopes designed by SWKP along a 1.4 km stretch of 8 m wide road at the southwestern corner of Tsing Yi Island.

The geotechnical design report for the scheme was submitted to the GCO for comment in October 1980, following formation of the cut slopes between 1978 and 1980. In the report, the concerned cut slopes were designed to achieve a minimum FOS of 1.2 for a groundwater table corresponding to a 1 in 10 year return period rainstorm. Soil slopes were generally cut to between 33° and 34°, except those overlain by colluvium, which were cut to 22°.

The geotechnical report identified that "horizontal drains have been installed at the locations where water has been observed seeping from the cut face and at other locations where it was considered necessary to lower the water table for stability reasons". No details were provided in the report regarding the length and spacing of horizontal drains, but a plan indicates that two rows of horizontal drains were installed along the lower northern portion of slope No. 10NE-B/C254 and two rows along the mid-point of slope No. 10NE-B/C262 (Figure 6).

Drillholes in the vicinity of the 1999 landslide identified colluvium to a maximum thickness of about 5 m (before slope cutting) overlying between about 2 m and 20 m of completely decomposed granite (CDG). According to the cross-section nearest to the 1999 landslide located about 50 m to the north, the proposed cutting of maximum depth 15 m was to be in CDG with all the colluvium being removed. Analysis of this section by SWKP using soil strength parameters derived from laboratory testing (Table 2) and a 1 in 10 year groundwater table taken to be 1 m above the highest groundwater level recorded during the monitoring period, which was about 1 m above rockhead, showed that the slope had a FOS less than the required value of 1.2. Analysis of the same section assuming a reduced groundwater table due to the installation of two rows of horizontal drains along the mid-point of the slope showed that the FOS would be 1.24. The design report made reference to the fact that seepage had been observed along the toe of the slope to the south of the analysed section, i.e. towards slope No. 10NE-B/C254.

Binnie and Partners (B&P) reviewed the geotechnical submission on behalf of the GCO in January 1981 and recommended that the stability of the slopes needed to be reviewed. Further study of triaxial test results and more frequent monitoring of piezometers were also recommended to validate the design parameters. In particular, B&P noted that no calculations or data on the performance of the horizontal drains was provided in the report to validate design groundwater assumptions.

Following the failure of slope No. 10NE-B/C262 in June 1982, seven drillholes were undertaken in the failed slope, with a piezometer installed in each drillhole. Monitoring of these piezometers was carried out between September 1982 and March 1983 (Section 2.4.3). Five additional piezometers were installed in new drillholes located along the cut slopes to the north of the 1999 landslide, as proposed by SWKP in April 1983 to satisfy the GEO comments regarding design groundwater table assumptions. Monitoring of these piezometers started in late 1983.

In December 1984, SWKP submitted updated piezometer readings to the end of September 1984, along with monitoring records (which commenced in October 1984) for horizontal drains. SWKP provided revised geological cross-sections and stability analysis based on the new information and concluded that the latest piezometric data showed that the assumptions used in the original design report were in general conservative. A sensitivity check was performed on cross-sections between chainage 2530 m and 3300 m to the north of the 1999 landslide assuming that the horizontal drains were not functioning (the nearest section to the 1999 landslide was about 200 m away). A minimum FOS of 1.16 (rounded up to 1.2 for the purpose of the submission) was obtained under such conditions. In view of this, SWKP considered that they had validated their design assumptions.

After reviewing the latest information, the GCO were still not satisfied with the design groundwater table assumptions and considered that it was "important to continue monitoring the groundwater level".

The cut slope design was eventually approved by the GCO in June 1988, following protracted discussions about the design groundwater table assumed in the stability analysis and the adequacy of this assumption should the horizontal drains become blocked. This issue was resolved following SWKP's recommendation that annual maintenance should be provided to the horizontal drains, including cleaning and flushing out as necessary.

Following approval of the cut slopes design, the HyD were requested to take up the maintenance responsibility of the slopes along this section of Tsing Yi Road taking into consideration the importance of monitoring and maintaining the horizontal drains. HyD agreed to take up the maintenance responsibility of the slopes as from 1 September 1988, as stated in their memo to TWNTDO dated 2 August 1988. However, the HyD noted in the memo that it was "not possible to flush these [the horizontal drains installed along the upper levels of the slopes] using compressed air/water". Also, it was noted that when removing the perforated filter tube for cleaning, the hole had a tendency to collapse. This made it impossible to reinsert the filter tube and the HyD enquired as to whether "your Consultants would enlighten me [Chief Highways Engineer/New Territories] with any effective method to serve the above maintenance purposes". No response to this memo was found in the GEO files.

The HyD were approached as part of this study to establish whether maintenance of the horizontal drains was undertaken, but no records could be found on HyD's files.

2.4.5 Special Project Report No. 9/83

The 1982 landslide in slope No. 10NE-B/C262 was the focus of Special Project Report SPR 9/83 (Hencher, 1983). The report analysed the original slope design in light of the information arising from and following the 1982 landslide. Using drillhole information arising from SWKP's 1982 landslide investigation (Section 2.4.3), it was noted that the assumed depth to bedrock was significantly deeper than that assumed in the original stability analysis for slope No. 10NE-B/C262 (Figure 8). The FOS of the slope based on the deeper bedrock profile and using SWKP's 10-year groundwater table (modified to the deeper bedrock profile) was about 0.9. However, based on post-failure groundwater monitoring and field observations, it was considered that slope conditions were near saturated following the

May 1982 rainstorm. Stability analysis assuming groundwater conditions believed to be representative of the May 1982 rainstorm gave a FOS of 0.84.

The reason for the discrepancy in bedrock profiles was cited as being the termination of drillholes in corestones and "the erroneous interpretation of large corestones at the slope surface as representing pinnacles of bedrock".

The report noted that horizontal drains installed along the toe of an adjacent slope were "seen to be issuing water during site visits in summer 1982 but were clearly not effective in draining the slope as the whole toe of the slope was waterlogged". Based on desk study records, this comment refers to the portion of slope where the 1999 landslide occurred.

2.4.6 Other Studies

In mid-1992, the GEO initiated a consultancy agreement, entitled "Systematic Inspection of Features in the Territory" (SIFT), to search systematically for slopes not included in the 1977/78 Catalogue of Slopes and to update information on previously registered features, by limited site inspections and studying aerial photographs. Slope No. 10NE-B/C254 was categorised as a Class C1 features under the SIFT project, i.e. "Assumed formed pre-1978 or illegally formed". This implies that the feature was not designed and checked to current geotechnical standards, which is not the case as discussed previously.

In 1994, the GEO commenced a consultancy agreement, entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST), to systematically update the 1977/78 Catalogue of Slopes and to prepare the New Slope Catalogue. The GEO's consultants for the SIRST project inspected the slope on 1 May 1997, and reported no signs of seepage or distress. Inferred past instability was recorded as "Minor".

2.4.7 Previous Inspections

A Routine Maintenance Inspection by HyD in July 1998 identified extensive cracking and displacement along the surface drainage channels of slope Nos. 10NE-B/C254 and 10NE-B/C262 and noted that "soil movement was found". The Record of Routine Maintenance Inspection stated that an immediate Engineer Inspection was required. The most notable areas of distress observed are:

(a) the lower northern portion of slope No. 10NE-B/C254 and the cascade channel at the boundary of slope Nos. 10NE-B/C254 and 10NE-B/C262 showed significant signs of distress, with evidence of slope movement and localised areas of washout,

(b) a washout/erosion scar in the middle southern portion of slope No. 10NE-B/C262, and

(c) a localised area of surface erosion at the upper northern portion of slope No. 10NE-B/C262.

The locations of these areas of distress, referenced Areas 1 to 3 respectively, are shown on Figure 9 and photographs showing the distressed areas are presented in Appendix A.

In view of the observed slope distress, HyD requested the GEO to provide geotechnical advice regarding the necessary improvement works for slope Nos. 10NE-B/C254 and 10NE-B/C262. A joint site inspection between the GEO and HyD was undertaken on 23 February 1999. The areas of distress identified during the Routine Maintenance Inspection were examined and the GEO made the following recommendations to HyD in a memo dated 3 March 1999:

(a) repair the broken/cracked channels,

(b) remove loose soil and debris in failure scars and backfill with compacted fill,

(c) provide surface protection to concerned areas, and

(d) provide a baffle wall along the outer curved section of the stepped channel in Area 3.

The GEO also recommended that an Engineer Inspection, including a stability assessment of the slope portions concerned, should be carried out as soon as possible. It was noted that the stability assessment should cover "a review of the original slope design, investigation of the causes of signs of distress and any long term remedial works proposals".

Based on photographic records, the distress in Area 1 comprised cracked and dislocated drainage channels, a washout/erosion scar below the drainage channel above the first batter and broken sections of the cascade channel/concrete staircase (Plates A1 to A5, Appendix A). Distress was particularly noticeable along the drainage channel above the second batter, where downslope ground movements of the order of 1 m were inferred. Movement across the cascade channel/concrete staircase was concentrated in two areas where the drainage channels above the second and third batters joined with the cascade channel (Figure 10). In these areas, the channel/staircase had moved sideways by about 250 mm, causing the sidewall to collapse locally.

Area 2 comprised a narrow scar, 3 m to 5 m wide by 20 m long and 2 m deep, between the fourth and fifth batters at the southern end of slope No. 10NE-B/C262 (Plates A6 to A7, Appendix A). The drainage channel above the fourth batter had been severed and a section of the channel collapsed. Based on the small lateral displacement of the severed channel, it would appear that it had failed due to a lack of foundation support rather than being sheared by a soil mass moving downslope. Accordingly, the scar would appear to have been caused by erosion. This is corroborated by the narrow and steep-sided nature of the scar and the erosion gullies observed below the toe of the scar. No flattening of vegetation or scouring of the surface materials was evident in the slope above the scar, suggesting that the erosion was either caused by surface water overtopping the channel due to blockage or by sub-surface water flow.

Area 3 comprised shallow (typically less than 0.5 m) surface erosion at the northern end of slope No. 10NE-B/C262. Based on the proximity of the erosion scars to the crest channel, it is considered the cause of distress was probably surface water overtopping the crest channel at this location (Plates A8 to A9, Appendix A).

The recommended urgent repair works were carried out by the HyD's maintenance contractor between April and May 1999.

In July 1999, an Engineer Inspection (EI) of slope Nos. 10NE-B/C254 and 10NE-B/C262 was carried out by the HyD's consultant, Binnie Black and Veatch Hong Kong Ltd. (BBV), under Agreement No. CE 89/97. The draft EI reports identified evidence of recent erosion and movement, comprising severe cracking to surface drainage channels and stepped-channels across the features similar to that previously identified. The 300 mm wide toe channel along the northern portion of slope No. 10NE-B/C254 was also noted as having been crushed (Figure 9 and Plate 10, Appendix A). The width of the U-channel had narrowed by up to 250 mm over about a 20 m long section. No records relating to horizontal drains were recorded in the draft EI report and no special measures or monitoring requirements for horizontal drains were recommended in the draft Maintenance Manual for slope No. 10NE-B/C254.

Recommendations for routine maintenance in the EI report included the clearance and repair of all blocked and cracked drainage channels as well as the removal of overhanging vegetation. Recommendations for preventive maintenance works included the provision of a concrete apron along both sides of the drainage channels and to fill undermined areas with no-fines concrete. The EI report classified the overall state of maintenance of the slopes as being "poor" and recommended that emergency repair works and priority stability assessment be carried out due to the observed signs of distress.

It is noteworthy that the inspection record for slope No. 10NE-B/C254 noted an area of undermining along the crest channel to the southeast of the 1999 landslide (Figure 9). In this area, a post-failure inspection undertaken by FMSW as part of this identified a pre-existing tension crack and an area of shotcrete repair (undertaken sometime between December 1997 and August 1998 based on API) to a severed section of the crest channel (Section 3.3). Based on visual inspection of the sheared channel, the inferred vertical displacement across the channel prior to movements associated with the 1999 landslide was of the order of 400 mm (Plate 8). No specific reference was made in the record of inspection prepared by BBV to this sheared crest channel or the presence of pre-existing tension cracks.

A letter from BBV to the HyD dated 12 August 1999, confirming the need for emergency stability assessment regarding slopes Nos. 10NE-B/C254 and 10NE-B/C262, noted that "There is evidence that at least two areas within the boundaries of these features have been subject to lateral and vertical mass movement of up to 300 mm". The locations of the areas of movement referenced in the letter are shown on Figure 9.

Following a request by the GEO in early August 1999, the affected slopes were inspected by FMSW to ascertain the nature and extent of instability. The first inspection on

5 August 1999 noted that repair works, as recommended by the GEO, had been carried out for Areas 2 and 3, and that no distress was observed in these areas. In Area 1, repair works had only been partly completed. The cascade channel had been repaired and an area of shotcrete 10 m wide by 15 m long applied to the erosion scar in the first batter, but the cracked and dislocated surface channels trending laterally across the slope above the second and third batters had not been repaired.

Area 1 formed the focus of FMSW's study. Dense vegetation cover hampered and delayed inspection, with clearance works required to facilitate detailed inspection. Clearance works revealed significant signs of movement along the drainage channel above the second batter, with the complete dislocation of the channel at a number of locations (Figure 10 and Plates B1 to B4, Appendix B). Most notably two major dislocations were observed at a distance of about 13 m and 26 m to the south of the concrete stairway adjoining the cascade channel between slopes Nos. 10NE-B/C254 and 10NE-B/C262. The section of channel between these dislocations had been displaced downslope by a distance of about 1 m and an area of voiding was observed along the upslope side of the displaced channel (Plate B5, Appendix B). A tension crack, 3 m long by about 50 mm wide and up to 150 mm deep, was observed above the most southerly portion of these dislocations (Figure 10 and Plate B6, Appendix B).

Along the drainage channel above the third batter, a 9 m long section of channel had been severely undermined to a depth of about 1 m (Plates B7 and B8, Appendix B). This area of distress was directly above the major dislocations observed along the drainage channel above the second batter (Figure 10), which in conjunction with the 3 m long tension crack suggest that these points may represent the margins of related significant ground movement in the second and third batters. Based on the localised northerly displacement observed in the cascade channel at this elevation, the direction of movement was slightly oblique to the direction of the slope. No signs of significant distress were observed along the drainage channels above the first, fourth and fifth batters.

At the time of the landslide on 24 August 1999, FMSW were still in the process of assessing the nature and extent of instability in Area 1 and inspection of the remaining areas of slope Nos. 10NE-B/C254 and 10NE-B/C262 had not been undertaken. Therefore, the severed crest channel along the upper northern portion of slope No. 10NE-B/C254, which appears to have been present before the 1999 landslide, had not been identified.

Following the 24 August 1999 landslide, FMSW undertook a more widespread inspection of areas adjoining the failed slope, the details of which are discussed in Section 3.3.

3. DESCRIPTION OF THE LANDSLIDE

3.1 Time of the Landslide

In accordance with discussions between the GEO Inspecting Officer and the taxi driver whose car was trapped following the landslide, the failure occurred at about 8:00 a.m. on 24 August 1999.

3.2 Description of the Landslide

The first inspection of the landslide by FMSW was at about 9:00 a.m. on 25 August 1999, approximately 24 hours after the landslide was inspected by the GEO.

The landslide occurred at the northern end of slope No. 10NE-B/C254, in the second and third batters, with the crown encroaching slightly into the base of the fourth batter and the toe daylighting above the drainage channel between the first and second batters (Figure 2). A layer of landslide debris, typically less than 300 mm, covered much of the surface of rupture and the batter below. The majority of landslide debris was deposited onto the road at the toe of the slope (Plates 2 to 4). The landslide scar was between 10 m and 15 m wide by 20 m long, with a maximum depth of about 2 m. The estimated volume of the failure was about 300 m^3.

Inspection of the main scarp revealed that the landslide had failed through colluvium. The depth of the landslide was greatest at the crown and along the upper southern flank, where the surface of rupture was between 1.5 m and 2 m deep and steeply inclined. The depth and inclination of the surface of rupture decreased towards the toe and towards the northern flank of the landslide (Plates 9 to 11).

A series of sub-parallel shear planes were exposed in the main scarp (Plate 12). Sub-surface groundwater flow, with an estimated flow rate of between 10 and 15 litres per minute, was observed from the lowest shear plane exposed in the main scarp at the crown of the landslide during FMSW's inspection on 25 August 1999 (Plate 12).

Inspection of the landslide debris showed that it comprised a completely remoulded matrix of slightly gravely, sandy, silty clay/clayey silt, with some cobbles and boulders and sections of severed surface drainage channel. The morphology of the debris trail, comprising a relatively smooth surface profile with no irregular steps, platforms or intact rafts of material, suggests that the landslide involved a single phase of movement. The debris was very wet with surface water flow evident down the central portion of the landslide. The surface water flows came from subsurface groundwater flows emanating from the main scarp at the crown of the landslide. Groundwater flows were also observed from a horizontal drain above the first batter and from what appeared to be a soil pipe daylighting at the base of the slope, both on the southern side of the landslide (Figure 2 and Plates 13 and 14).

A gradual reduction in groundwater flows was noted during subsequent site inspections, with no groundwater being observed at the crown of the landslide and from the horizontal drain (a level difference of about 8 m) on 26 and 27 August 1999 respectively.

The travel angle of the failed debris measured from the crown of the landslide to the distal end of the failure was 26°. This is lower than that for a typical rain-induced landslide of comparable volume, which indicates that the detached debris was fairly mobile (Wong & Ho, 1996).

3.3 Inspection of the Area Surrounding the Landslide Site

The area surrounding the 1999 landslide was inspected in detail by FMSW to ascertain whether the failure was a localised incident or whether it formed part of a larger instability. Details from the inspection are discussed below. The locations of significant distress observed on site are shown in Figure 11 and details of the distress are summarised in Table 3.

The drainage channel along the toe of the slope showed some localised signs of distress (Figures 9 and 11). Along the southern portion of the slope, the distress comprised damage along the outside edge of the channel, which may have been caused by vehicular traffic, as lorries were seen to park along the road from the API. In the northern portion of the slope, the distress comprised the narrowing of the channel as reported by BBV in their Engineer Inspection Report (Section 2.4.4). FMSW observed that the slope profile above this area was slightly steeper and that the kerb line along the road further to the north appeared to be hogging slightly.

Inspection of the surface drainage channel above the first batter identified a number of cracks through the channel. Most cracks were narrow with little separation, but one major crack, 100 mm wide, and three smaller cracks, between 15 mm and 30 mm wide, were observed. No lateral (i.e. horizontal downslope movement) or vertical displacement was noted across the cracks. A 3.5 m long section of the channel was blocked with loose soil debris and voiding was noted over localised areas behind (i.e. upslope) the channel (Figure 11). Horizontal drains were observed at regular intervals (typically 5 m spacing) above the channel. Significant groundwater flow was observed from the drain adjacent to the southern flank of the landslide on the 25 August 1999 (Figure 2 and Plate 14), but the remaining horizontal drains were either dry or only showed very minor groundwater flow. It was noted that most of the horizontal drains were either partially or totally obscured by vegetation indicating inadequate maintenance.

More significant signs of distress and slope movement were observed along the drainage channels above the second and third batters. A number of open cracks up to 70 mm wide, with lateral and vertical displacements of a maximum of 50 mm were recorded. An area of shotcrete about 10 m wide was present in the second batter, and in the third batter above the shotcreted area the vegetation had been flattened and surface erosion was evident over a 4 m wide section (Figure 11).

Considerable signs of distress and ground movement were identified along the crest channel. The distress was concentrated along a 30 m long section of the channel starting at a distance of about 10 m north of the cascade channel descending the mid-portion of the failed slope. In this area, sections of the channel were completely dislocated and certain portions had rotated and moved downslope by about 500 mm, such that a low point existed in the channel, where water was ponding (Plates 15 and 16). There was evidence of surface scouring across the slope below this section of the crest channel, indicating that water had overtopped the channel (Figure 11).

Distress along the crest channel ended abruptly in a major dislocation about 40 m north of the cascade channel (Figure 11). At this point, an old shotcrete repair to the crest channel was identified. The shotcrete repair was cracked, with about 100 mm to 150 mm of vertical displacement and 150 mm to 200 mm of lateral separation (Plate 8). This cracking appeared to be recent and may have occurred during the 24 August 1999 rainstorm. The total vertical displacement across the severed channel was between 500 mm and 600 mm, which implies that about 400 mm of movement took place prior to the application of shotcrete to seal the severed channel. Based on API (Section 2.4.2), shotcreting works took place sometime between December 1997 and August 1998.

Shotcrete had also been applied along a narrow strip above the crest channel. The shotcreted area was badly cracked at the time of FMSW's inspection, exposing a pre-existing tension crack that was open to a maximum depth of 1.2 m. The tension crack could be traced into the slope to the north of the major dislocation in the crest channel as far as the crown of the landslide (Figure 2 and Plate 16). The steeply inclined nature and orientation of the rupture surface in the upper portion of the main scarp suggests the tension crack probably extended into the failed area of the slope prior to the landslide.

Based on the displacement across the severed shotcrete repair, renewed downslope movement of a maximum of about 200 mm had occurred across the tension crack. This renewed movement may have resulted from the 24 August 1999 rainstorm. Displacement across the crack diminished from a maximum at about the mid-point to nothing at the northern and southern extremities of the crack.

Inspection of the southern portion of slope No. 10NE-B/C254 identified cracking up to a maximum of about 20 mm over the initial 10 m to 15 m of the drainage channels to the south of the cascade channel in the central portion of the feature. This surface distress was less severe than that observed in the northern portion of the slope and suggests that little or no movement has occurred along the southern portion of slope No. 10NE-B/C254.

Signs of distress were observed along the cascade channels descending the mid-point and northern end of slope No. 10NE-B/C254 (Figure 10 and Plate 17). Movement across the northern cascade did not appear to have increased significantly from that observed during FMSW's inspection on 5 August 1999, but the areas of recently repaired/reinstated channel had cracked again.

The cascade channel descending the mid-point of the slope also showed signs of distress in the form of cracking, but no discernible sideways movement was observed. Two glass telltales had been installed across a minor crack, less than 5 mm wide, in the sidewall of the cascade channel at an elevation of the drainage channel above the second batter. The telltales had been installed within about 150 mm of each other. One had cracked, but the other remained intact. No details regarding the installation/monitoring of the glass tell-tales was identified during the desk study.

FMSW also inspected slope No. 10NE-B/C262. Cracking was observed along some of the drainage channels, but this was of a minor nature and overall the slope was considered to be in fair condition.

4. SUBSURFACE CONDITIONS

4.1 Geology

With reference to the published geological map (GEO, 1991), the landslide site is mapped as comprising fine- and medium-grained granite, intersected by a series of parallel feldsparphyric rhyolite dykes. A possible fault with a north-northwest to south-southest trend is indicated along the line of a prominent valley about 200 m to the east of the landslide site (Figure 12). The orientation of the dykes is approximately perpendicular to the line of the fault.

4.2 Previous Ground Investigations

Fifteen drillholes had previously been sunk in the vicinity of the landslide site, with eight carried out for the original design of the road and the other seven carried out following the landslide at slope No. 10NE-B/C262 in 1982 (Figure 7). Twelve of the fifteen drillholes were sunk for the investigation of slope No. 10NE-B/C262. The information from the drillholes is in general agreement with that shown on the geological map.

4.3 Current Investigation

4.3.1 General

Ground investigation (GI) works comprising 7 trial pits and vegetation clearance were undertaken to identify the nature and condition of materials involved in the landslide and to investigate the extent of tension cracking and surface distress (Figure 2). These works were supplemented by field mapping, details of which are presented in Figure 7. Trial pit logs prepared by FMSW are presented in Appendix C.

It is noteworthy that during the installation of H-piles along the toe of the slope as part of the urgent repair works following the 1999 landslide, it was noted in a memo from the Mainland West Division of the GEO to the HyD dated 13 September 1999, that “no sound rock was encountered in pile drilling”. In accordance with the desk study records, the piles should have had an embedded length of at least 9 m.

4.3.2 Trial Pitting Works

Seven trial pits (TP1 to TP7) were undertaken as part of the post-failure investigation between September 1999 and October 1999. Four trial pits (TP1 to TP4) were positioned along the major pre-existing tension crack identified above the 1999 landslide. The remaining three trial pits were located within the landslide scar (Figure 2).

The trial pits across the tension crack encountered colluvium to a maximum excavated depth of 4 m. The boulder content of the colluvium varied between the trial pits, but typically the colluvium comprised a sandy gravelly silt/clay with some to many cobbles and some, occasionally many, small to large (up to 3 m wide), sub-rounded to sub-angular boulders of highly to moderately decomposed and moderately to slightly decomposed granite and feldsparphyric rhyolite. An infilled tension crack, typically 100 mm to 200 mm wide, but up to 0.5 m wide, was identified within the colluvium. The crack was infilled with a very loose, and in places voided, silty sand with many roots and occasional gravel (Plate 18). The width of the crack decreased with depth and became increasingly difficult to distinguish, but it was still present to the base of each trial pit, with a maximum depth of 4 m in trial pit TP2.

Trial pits TP5 to TP7 within the landslide scar identified between 100 mm and 500 mm of colluvium overlying completely decomposed and completely to highly decomposed granite (CDG and C-HDG respectively) with corestones of moderately decomposed granite (MDG) to a maximum excavated depth of 2.4 m. The total depth of colluvium allowing for material removed during landsliding, is estimated to be between 1.5 m and 2 m. The CDG comprised a stiff orange and reddish brown, mottled greyish white, slightly gravelly, sandy clayey silt. The C-HDG comprised a very stiff reddish and pinkish brown, slightly sandy, clayey silt, with medium-spaced, smooth planar and smooth undulating relic joints. A silty clay joint infill disseminated with flecks of white kaolin and typically 5 mm thick, was observed in the C-HDG in trial pit TP5 and manganese staining was noted around the MDG corestones. In trial pit TP6, a thicker layer of clayey joint infill, typically 30 mm to 60 mm wide, was observed. It was also noted that in trial pit TP6, small voids giving rise to zones of more porous material were present around the margins of the MDG corestones. In accordance with trial pit information and mapping of the exposed portions of the rupture surface, CDG was not involved in the 1999 detachment.

Insitu testing in the trial pits showed that the dry density of the colluvium varied from a maximum of 1.57 Mg/m^3 to a minimum of 1.33 Mg/m^3. Testing in the CDG showed a typical dry density of 1.4 Mg/m^3.

Following vegetation clearance, it was possible to identify the southerly extent of the main pre-existing tension crack, together with two smaller localised pre-existing tension cracks to the north (Figure 2 and Plate 19). These tension cracks were about 4 m long, up to 100 mm wide and were open to a depth of about 0.6 m. The removal of vegetation also revealed the bouldery nature of the colluvium (Plate 20).

4.3.3 Field Mapping

Detailed field mapping of the area in the vicinity of the landslide site was carried out by FMSW. Two dominant rock types have been identified, namely medium-grained granite and feldsparphyric rhyolite (forming dykes within the granite), but gradational changes exist within both units on a local scale. Bouldery colluvium covered much of the hillside and a moderate convex break of slope associated with the colluvial deposits was observed above the crest channel of slope No. 10NE-B/C254. Pertinent information from the field mapping is presented in Figure 7.

Along the ridgeline to the east of the 1999 landslide, a shallow depression forming an ephemeral drainage line lies between two prominent hilltop features (Figure 7). The ground below the hilltop is characterised by bouldery colluvium and outcrops of feldsparphyric rhyolite. These correspond to areas where rockhead is at or close to the ground surface. Between the hilltop features is a lower lying saddle, which is characterised as an area with a deeper weathering profile that is overlain by colluvium with less boulders.

Above and to the east of the 1999 landslide, a prominent outcrop of slightly decomposed feldsparphyric rhyolite, approximately 7 m in width, has been mapped. Further outcrops were identified along the ridgeline further uphill indicating that the dyke trends east-west through the area of the 1999 landslide within slope 10NE-B/C254 (Figure 6). The southern contact of the dyke with the granite was abrupt (Plate 21). The northern contact was less distinct with what is considered to be a gradational transition between the outcropping rhyolite and decomposed granite forming the lower lying saddle. The superficial deposits exposed within the area of the saddle comprised residual soil overlain further downhill by colluvium with few boulders. Possible shallow relic landslide scars were visible in the upper portion of the saddle.

The boundary between the materials within the saddle and those forming the northernmost hilltop also appears to be gradational. Insitu boulders of feldsparphyric rhyolite were mapped at the hilltop and the boulder content of the surface colluvium increased to the north as the ground profile becomes steeper, but no contact could be identified. These observations indicate the presence of another east-west trending dyke with rockhead at or close to the ground surface.

Based on visual inspection, toppling is kinematically feasible within the prominent outcrop of slightly decomposed feldsparphyric rhyolite forming the rocky bluff above the 1999 landslide. The basal plane is formed by a joint set dipping at an angle of 28° in a west south westerly direction (240°) with a number of different joint sets intersecting the basal plane to form block edges. Details of joint measurements recorded on site are presented in Figure 7. This may explain the deposits of bouldery colluvium identified below the prominent outcrop.

No significant signs of distress were observed across the hillside above the cut slopes, apart from some small shallow failure scars in the vicinity of the saddle, which may be related to disturbed ground associated with the graves in this area (Figure 7).

4.4 Deduced Ground Conditions

The geological setting of the landslide site comprises granite intruded by east-west trending feldspharphyric rhyolite dykes that lie approximately perpendicular to the alignment of Tsing Yi Road, i.e. parallel with the line of the cut slopes (Figure 7). Two dykes have been identified in the vicinity of the landslide site, one of which is considered to pass beneath the location of the 1999 landslide. The dykes form areas of higher ground, with rockhead at or close to the ground surface above an elevation of between 115 mPD and 120 mPD. The area of lower lying terrain between the two dykes forms an ephemeral drainage line, along which possibly exists a locally deeper weathering profile within the granitic materials. The road has been cut through this drainage line and the landslide occurred about 15 m to the south of it.

Drillhole records in slope No. 10NE-B/C262 to the north of the failed slope indicate a gradual increase in the depth to rockhead in a downhill direction, with a local but significant drop in rockhead profile towards the toe of the cut slopes. This drop in rockhead may extend southwards across the landslide site as indicated by the piling records along the toe of the failed slope (Section 4.3.1). The sharp drop in rockhead may indicate an area of preferential weathering along a fault, as two faults with a similar north to south and north-northwest to south-southeast trend are indicated to the east of the landslide site on the geological plan (Figure 12).

Bouldery colluvium covers much of the hillside above the cut slopes. In particular, field observations and trial pit records have identified colluvium overlying CDG in the failed cut slope. The colluvium appears to be thickest at the crest of the cut slope where it is at least 4 m thick. The colluvium thins out in the lower portion of the cut slope towards the toe of the 1999 landslide, where it has been removed by cut slope formation.

4.5 Groundwater Conditions

Post-failure inspections identified subsurface water flows at the crest, mid point and toe of the landslide scar (Section 3.2). Furthermore, an erosion scar about 20 m to the north of the 1999 landslide, as identified by HyD in July 1998 (Section 2.4.7), provides a possible indication of concentrated subsurface groundwater flow in this locality. No other evidence of subsurface flows was identified during site inspections.

Site mapping identified the presence of a feldsparphyric rhyolite dyke, probably of lower permeability than the decomposed granite, passing through the slope at the location of the 1999 landslide. These ground conditions are similar to those for slope No. 10NE-B/C262 to the north, where groundwater monitoring identified a highly variable groundwater table that was strongly influenced by the presence of a dyke (Section 2.4.3). Accordingly, it is considered that the hydrogeological setting of the landslide site is favourable to the rapid development of locally high groundwater pressures due to channelling and concentration of groundwater flow by the dykes.

Based on available drillhole information and records of soldier pile installation as part of urgent repair works after the 1999 landslide, there is a significant drop in rockhead along the toe of the slope. This also indicates the possibility of concentrated subsurface water flows and a high groundwater table.

The presence of significant tension cracks promotes direct infiltration, which would promote the formation of a shallow perched water table in the surface colluvium as well as development of cleft water pressures in the open cracks.

5. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge No. N11 is located at Tsing Yi South Fire Station, 100 Tsing Yi Road, about 800 m to the north of the landslide. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone link to the GEO. These records have been analysed to determine the characteristics of the rainstorm associated with the landslide.

For the purposes of this analysis, the landslide has been assumed to have occurred at 8:00 a.m. on 24 August 1999 based on eye-witness accounts.

The daily rainfalls recorded by the raingauge in July and August 1999, together with the hourly rainfalls from the 22 to 24 August 1999, are shown in Figure 13. The storm was concentrated around the day of 23 August 1999, with a peak rainfall of 86 mm/hour between 5:00 a.m. and 6:00 a.m. The storm intensified again in the morning of 24 August 1999, with a peak rainfall of about 40 mm/hour between 2:00 a.m. and 7:00 a.m.

Daily rainfalls for the May 1982 rainstorm recorded at the Hong Kong Observatory's raingauge No. 105, located at the western end of Tsing Yi Bridge, approximately 3 km from the landslide, were analysed as part of Special Project Report No. 9/83 (Hencher, 1983). No hourly breakdown of rainfall was provided, but a peak daily rainfall intensity of 290 mm was recorded on the 29 May 1982 (Figure 14). This is very similar to that recorded during for the rainstorm prior to the 1999 landslide, with a peak daily rainfall of about 285 mm on 23 August 1999.

Table 4 presents the estimated return periods for the maximum rolling rainfall recorded at raingauge No. N11 for selected durations preceding the landslide based on historical rainfall data recorded at the Hong Kong Observatory (Lam & Leung, 1994). The maximum 48 hour rolling rainfall of 599.5 mm was the most severe, with a return period of 50 years.

A comparison of the patterns of previous heavy storms affecting the landslide site is shown in Figure 15. It is evident from the rolling rainfall profile that the 22 to 24 August 1999 rainstorm was the most severe event experienced by the site since the raingauge came into operation in August 1984. A comparison of the rolling rainfall profile arising from the 22 to 24 August 1999 rainstorm with that prior to the 1982 landslide in slope No. 10NE-B/C262 was not possible as the monitoring records were not directly comparable.

6. DISCUSSION

6.1 Mode and Nature of Instability

The instability observed across slope No. 10NE-B/C254 prior to the 1999 landslide comprised slope movements without full detachment of displaced material, giving rise to surface distress and deterioration, including the formation of a laterally extensive tension crack. Such instability had probably been ongoing for some time based on site observations. The spatial relationship between the laterally persistent tension crack and the cut platforms formed between 1963 and 1968 suggests that the tension cracks may have developed as a result of the formation of the steep cuttings in the hillslide (Section 2.4.2).

The 24 August 1999 landslide represents an apparent change in the mode of instability of the slope with intermittent slope movements locally developing into an uncontrolled failure involving detachment of the near-surface material in a fast-moving manner. The detached debris was comparatively more mobile than typical rain-induced landslides, with a travel angle of 26°. The relatively high mobility of the debris may be due to a number of reasons, including failure under high groundwater pressures.

The landslide serves as an indication that:

(a) pre-failure distress observed in this part of the slope can be viewed as a precursor to the landslide,

(b) slope movements without detachment would result in progressive deterioration of the slope condition through the formation of tension cracks in the near-surface material, and

(c) progressive slope movements may develop into an instability involving the complete detachment of all or part of the previously displaced material from the slope, particularly where slope deterioration involving the development of tension cracks allows direct ingress of surface water into the distressed ground.

Renewed slope movements were also experienced across the pre-existing tension crack above and to the southeast of the 1999 landslide, which reflects the continued deterioration of the northern portion of slope No. 10NE-B/C254.

It is considered that the 1999 landslide represented localised detachment of material from a larger-scale progressive instability, approximately 80 m wide, bounded by the pre-existing tension crack. Trial pitting along the tension crack showed that the larger-scale instability was at least 4 m deep, but it was not possible to identify the base of the tension crack or whether it extended into the underlying CDG under the current investigation. However, the widespread nature of distress observed across the slope and the lateral extent of the pre-existing tension crack suggest that deep-seated instability exists or had been developing progressively in the northern portion of slope No. 10NE-B/C254.

Inspection of the drainage channels along the southern portion of slope No. 10NE-B/C254 and across the whole of slope No. 10NE-B/C262, identified some cracked channels, but these cracks were typically less than 10 mm wide and did not appear to form part of a larger instability. In July 1996, the HyD identified a narrow scar at the southern end of slope No. 10NE-B/C262 (this slope was upgraded following the major landslide in 1982). The geometry and nature of the scar suggest that it was probably caused by erosion, and that the source of erosion may have been subsurface groundwater flow (Section 2.4.7). This indicates the possibility of a locally high groundwater table.

Due to the presence of a significant tension crack at the crest of cut slope No. 10NE-B/C254, there has been a reduction in support to the lobe of bouldery colluvium in the hillside above. There is therefore the possibility of instability retrogressing up the hillside with time.

The ground conditions of the unstable area at the northern end of slope No. 10NE-B/C254 have certain similarities to those identified at the site of the 1982 landslide in slope No. 10NE-B/C262 about 100 m to the north. This suggests that there may be some generic factors contributing to slope instability in this area, such as complex geology and hydrogeology, high groundwater conditions, etc.

6.2 Probable Causes of Failure

The close correlation between rainfall and the time of the landslide suggests that the failure was triggered by heavy rainfall. Information collected during this investigation indicates that other factors probably contributed to the failure, namely:

(a) unfavourable hydrogeological setting of the site giving rise to high groundwater conditions,

(b) deterioration of the slope condition, and

(c) possible reduction in the efficiency of the horizontal drains due to progressive ground movements and lack of maintenance.

In the vicinity of the 1999 landslide, an ephemeral drainage line lies between two dykes. Tsing Yi Road has been cut through the hillside across these features. The dykes form sub-vertical bands of less permeable material that confine sub-surface water flows arising from the ephemeral drainage line. This leads to concentrated groundwater flows downslope. The drop in rockhead near the toe of the slope also provides an indication of possible concentrated subsurface water flow.

Post-failure inspections identified evidence of high groundwater conditions locally within and in the vicinity of the 1999 landslide. In addition, the presence of significant tension cracks across the slope would have facilitated direct infiltration with the possibility of cleft water pressures developing within the cracks. Accordingly, it is considered that seepage forces and transient elevated groundwater pressures probably developed rapidly following the 22 to 24 August 1999 rainstorm. It is possible that the setting of bouldery colluvium overlying CDG and localised areas of fill also promoted infiltration and could have led to the formation of a perched water table in the near-surface materials.

The possibility of locally high groundwater conditions in the failed section of slope is not unexpected given the findings of previous investigations undertaken for cut slopes in the vicinity of the 1999 landslide (Section 2.4.4). This is supported by post-failure observations following the 1999 landslide. The potential for such groundwater conditions was also identified by SWKP during their investigation of the large-scale instability in slope No. 11NE-B/C262 in 1982 (Section 2.4.3).

The 1999 landslide occurred at the lower northern end of a major pre-existing tension crack. While the failure was not a surprise given the prolonged slope distress and deterioration observed in this area, it is not certain as to the cause of the original instability leading to the formation of such distress. API indicates the possibility that the major tension crack may have formed in the mid 1970's due to a lack of toe support some 10 m downhill, following the formation of steep 5 m to 6 m high cut slopes in the late 1960's (Section 2.4.2). Alternatively, the adverse geological and hydrogeological setting of the site (as discussed above) gave rise to the distress following the significant build-up of groundwater pressures during severe rainstorms.

The continued performance of the horizontal drains was crucial to the stability of the slope, as highlighted by stability analysis in SWKP's design submission. No records could be found on the HyD files to indicate that monitoring and maintenance of the horizontal drains had been carried out. Site observations between July 1998 and August 1999 showed the drains to have been obscured by vegetation and slope wash material, indicating lack of maintenance. The presence of horizontal drains was not recorded in BBV's draft EI Report and no special measures or monitoring requirements were recorded in the draft Maintenance Manual.

Although horizontal drains have been installed along the slope toe and surface drainage has been constructed at regular intervals across the failed slope, little attention appears to have been given to the control of subsurface water flows in the area where the slope cuts across the ephemeral drainage line. This may have been significant with regard to the 1999 landslide and also the 1982 landslide in slope No. 10NE-B/C262 to the north, which are separated by the ephemeral drainage line (Figure 5).

7. CONCLUSIONS

It is concluded that the 24 August 1999 landslide was triggered by severe rainfall with an estimated return period of about 50 years. The unfavourable hydrogeological setting of the area giving rise to concentrated groundwater flows towards the landslide site is considered to be a significant contributory factor to the failure. Other significant contributory factors include lack of maintenance of the horizontal drains along the lower portion of the slope and absence of appropriate drainage measures in the upper to mid-portion of the slope to deal with sub-surface water flow arising from the ephemeral drainage line in the vicinity of the landslide site.

The influence of the cut slope platforms formed in the late 1960's on the stability of the hillside is not certain, but the spatial relationship between the platforms and the laterally persistent tension crack (50 m long) suggests that there could be a connection between the two.

The 1999 landslide involved local detachment of material in a fast-moving manner from an area where slope distress, comprising prolonged slope movements and progressive deterioration over an extensive area, had been observed since at least July 1998. This highlights that deterioration in slope condition arising from ground movements associated with the development of a progressive failure can, if allowed to continue, develop into a more mobile instability involving detachment of near-surface materials. The apparent change in the failure mechanism is probably linked to the increased rate of infiltration into near-surface materials arising from the development of open cracks brought about by intermittent slope movements and deterioration of slope condition.

8. REFERENCES

Geotechnical Control Office (1991). Silver Mine Bay: Solid and Superficial Geology. Hong Kong Geological Survey, Map Series HGM 20, Sheet 10, 1:20 000 scale. Geotechnical Control Office, Hong Kong.

Hencher (1983). Landslide Studies 1982. Case Study No. 8 Tsing Yi (1). Geotechnical Engineering Office, Hong Kong, 36 p.

Lam, C.C. & Leung, Y.K. (1994). Extreme rainfall statistics and design rainfall profiles at selected locations in Hong Kong. Royal Observatory, Hong Kong, Technical Note No. 86, 89 p.

Wong, H.N. & Ho, K.K.S. (1996). Travel distance of landslide debris. Proceedings of the Seventh International Symposium on Landslides, Trondhiem, Norway, vol. 1, pp 417 – 422.

LIST OF TABLES

Table 1 – Record and Details of Aerial Photographs Studied (Sheet 1 of 4)

Year	Photograph Reference	Altitude (feet)	Description
1924	Y00146	-	See below
06-02-63	Y08262 - 63	3900	The area surrounding the landslide site comprises mountainous terrain. A prominent ridgeline trending in a north-south direction is evident to the east of the landslide site. Large sections of the hillside to the east of the ridgeline are bare of vegetation and rills can be seen indicating active erosion. In contrast, the terrain to the west of the ridgeline, where the 1999 landslide occurred, is covered by vegetation comprising mainly grasses and less erosion is evident. Much of the hillside is covered by bouldery colluvium. In the immediate vicinity of the landslide site, a prominent boulder-filled gully descends from the ridgeline in a west-northwesterly direction towards the coastline. Further north of this gully, a number of drainage lines can be seen descending the slope in a westerly direction towards the coastline. The northernmost drainage line is longer and more distinct and lies about 20 m to the north of the 1999 landslide. A prominent rock bluff lies about 50 m above and to the east of the 1999 landslide, below which an area of bouldery colluvium is evident. There is no evidence of development across the hillside.
1968	Y13985 - 86	-	The area to the southeast of the landslide site is being developed, with construction works for the power station along the coastline ongoing. The alignment of Tsing Yi Road has been formed as far as the southwestern corner of Tsing Yi Island, but no road surfacing appears to have been laid as yet. Quarrying works are ongoing along the western and southwestern corner of Tsing Yi Island and an access track with three small structures has been cut into the hillside. For ease of identification the three structures will be referred to as the northern, the middle and the southern structures based on their relative geographic locations. The most northerly structure lies directly to the south of the prominent drainage line seen in the 1963 photographs. Based on later photographs, it is apparent that the mid and northerly structures lie in the mid to upper-portion of cut slope C252 and in the vicinity of where the 1999 landslide occurred. The northern structure has been cut into the area of bouldery colluvium identified below the rock bluff in the 1963 aerial photographs and the middle structure appears to have been formed below a lobe of colluvium, with a moderately sharp convex break of slope between the two. The cut slopes for the mid and northern structures appear stable, but instability is evident in the cut slopes for the southern structure.

Table 1 – Record and Details of Aerial Photographs Studied (Sheet 2 of 4)

Year	Photograph Reference	Altitude (feet)	Description
1969	Y14822 - 23	-	There is no discernible change in the appearance of the site, but the vegetation cover across the hillside in the vicinity of the 1999 landslide is thinner and the areas of bouldery colluvium are more apparent. The cut slopes along the temporary access track appear quite steep and there is some indication of localised instability. The cuttings for the northern and middle structures, which are both about 20 m wide by 15 m long, have been formed in, and slightly below, areas of bouldery colluvium respectively. The excavated slopes for these cuttings are estimated to be about 5 m to 6 m high and are very steep, but no signs of instability are apparent.
10-12-73	9602 - 03	2000	Construction works for the power station are largely complete and quarrying works appear to have stopped. The structures identified in 1968 are now derelict otherwise there is no discernible change in the area of the landslide site.
09-08-74	9242 - 43	2000	There is no other change apparent in the vicinity of the 1999 landslide site.
16-08-76	14661 - 62	1900	The hillside above the excavation for the middle structure has a stepped/irregular surface profile. Two arcuate lines are in this area. The lower line is about 50 m long and passes about 10 m above the cutting for the middle structure. This line occurs in the location of an old tension crack identified on site following the 1999 landslide. The upper line is about 20 m long and lies about 20 m above the cutting for the middle structure. No signs of surface distress were observed in this area of the hillside during post-failure site inspections. Another arcing line about 25 m long and a localised area of erosion is evident above the northern structure. No signs of surface distress were observed in this area of the hillside during post-failure site inspections.
21-12-77	20325	4000	Site formation works are ongoing along the valley floor to the east of the landslide site and slope Nos. C79 and C278 are being formed. Quarrying activities and/or site formation works for the western section of Tsing Yi Road have commenced at the southwestern corner of Tsing Yi Island. The arcuate lines observed in 1974 are no longer visible.

Table 1 – Record and Details of Aerial Photographs Studied (Sheet 3 of 4)

Year	Photograph Reference	Altitude (feet)	Description
07-12-78	23790 - 91	4000	Works for the western section of Tsing Yi Road are in progress and excavation works for cut slopes C254, C262, C263, C246, C249 and C251 have largely been completed. Surface drainage channels are being constructed along slopes C262 and C254. A stepped channel descends the mid-portion of slope C254. This stepped channel has been identified on site and lies some 80 m to the south of the 1999 landslide. The crest of the road cutting over the northern half of slope C254 daylights towards the toe of the cuttings for the northern and middle structures. Consequently, the upper sections of these cutting can still be seen in the hillside. Above the crest of the road cutting lies a strip of natural ground 10 m to 15 m wide and about 70 m long, above which a drainage channel is in the process of being constructed. The northerly portion of slope C254 has been cut into an area of bouldery colluvium and at the toe of the slope an accumulation of coarse material is evident. This suggests that instability may have been a problem in this area during excavation works.
16-04-80	29758 - 59	4000	Road works along the western section of Tsing Yi Road are largely complete. The areas of cutting for the middle and northern structures in the northern half of slope C254 have been filled. Over the lower northerly portion of slope C254, a number of small regularly-spaced white strips are evident. These have been identified on site as horizontal drains. These are not evident along adjacent sections of slope, which suggests that a high groundwater table was encountered in this section of the slope during construction. What appears to be distress is evident in the adjoining slope (C262) to the north of the landslide site. The fourth and fifth drainage channels above the road appear to be dislocated at the southern end and the section to the north appears bowed in a downslope direction. This slope distress does not appear to extend into Slope C254. Shallow instability is evident along the lower portions of slopes C263 and C246, with debris observed at the toe of the slopes.
11-08-82	43691 - 92	2000	The newly-formed cut slopes along the southwestern portion of Tsing Yi Road have been planted with small trees. A major landslide is evident in slope C262. There is no significant change in the layout of slope C254.

Table 1 – Record and Details of Aerial Photographs Studied (Sheet 4 of 4)

Year	Photograph Reference	Altitude (feet)	Description
04-05-83	48455 - 466	-	This set of very clear oblique aerial photographs clearly show the extent of 1982 landslide within slope No. 10NE-B/C262. The distress appears to be confined within the boundary of the feature with no visible distortion of the cascade channels descending either side of the feature. Based on the displacement of severed surface drainage channels it is estimated that the landslide involved approximately 3 m to 4 m of down slope movement. A 5 m to 10 m wide stripped along the southern flank of the landslide has been severely eroded or has experience secondary washout during and/or after the main landslide event. The main scarp locally exposes large corestones, either side of which lies more decomposed materials. It is noted in photograph No. 48455 that there is evidence of overland flow to the north of slope No. 10NE-B/C262, above the location of the cascade channel. The overland flow starts in the mid to upper portion of the slope. No signs of distress in the form of tension cracks can be seen in the aerial photographs covering slope No. 10NE-B/C254. In particular, no distortion is evident along the crest channel, although vegetation does obscure some portions. Horizontal drains are evident in the first and second batters along the northern portion of the slope No. 10NE-B/C254.
04-09-85	66691 - 92	4000	Newly planted trees are developing along the lower portion of slope C254, obscuring detail. A lighter shaded area is visible in the central portion of the slope, which has been identified on site as a rock outcrop that has been shotcreted.
17-09-86	A05770	4000	As 1985
04-10-87	A10594 - 95	4000	The trees planted along the slope are maturing, thereby obscuring much of slope.
06-10-88	A14748 - 49	4000	As 1987
21-03-90	A20927 - 28	4000	As 1987
01-10-91	A27623 - 24	4000	As 1987
05-12-93	CN5218 - 19	6000	As 1987
15-05-96	CN13564 - 65	4000	As 1987
14-11-97	CN18928 - 29	4000	As 1987
14-08-98	CN20714 - 15	3500	The drainage channel above the crest of the cut slope C254 has had maintenance works carried out along it.
10-11-98	CN21906 - 07	8000	No discernible change in slope C254.

Table 2 – Soil Strength Parameters Used for Slope Design by SWKP

Soil Type	Strength Parameters	
	c′ (kN/m^2)	ϕ' (degrees)
CDG (N<100)	5	35
CDG (N>100)	10	35
Colluvium	0	35
CD Feldspar Porphyry	5	30
CD Granodiorite	5	30
CDG Fill	5	35

Table 3 – Details of Significant Cracks Observed by FMSW

Crack Details			
No.	Description	No.	Description
1	Up slope face of toe channel has moved in wards by up to 250 mm	15	20 mm sep. and 10 mm to 15 mm of lat. movement
2	100 mm sep.	16	Cracked channel with 100 mm of rotational movement
3	20 mm sep.	17	10 mm sep. with 50 mm outward movement and 100 mm vertical displ.
4	30 mm sep.	18	40 mm sep.
5	15 mm sep.	19	70 mm sep. with 210 mm of vertical displacement
6	15 mm sep.	20	20 mm sep.
7	20 mm sep. and 50 mm vertical displ.	21	80 mm sep. with 100 mm outward movement and 50 mm vertical disp.
8	40 mm sep. with 50 mm outward movement	22	30 mm sep.
9	40 mm sep.	23	Severe distress across channel with about a 300 mm drop in ground level creating a low point in the channel with ponding water
10	70 mm sep.	24	
11	20 mm sep.	25	200 mm drop in ground level, with open tension crack 250 mm wide and 1.2 m deep
12	10 mm sep. with 50 mm outward movement	26	40 mm sep. with 50 mm outward movement and 60 mm vertical disp.
13	20 mm sep.	27	Location of old shotcrete repair to severed crest channel and renewed movement
14	50 mm sep. vertical disp. with 50 mm outward movement		
Note: For location of cracks see Figure 11.			

Table 4 - Maximum Rolling Rainfall at GEO Raingauge No. N11 for Selected Durations Preceding 24 August 1999 Landslide and the Estimated Return Periods

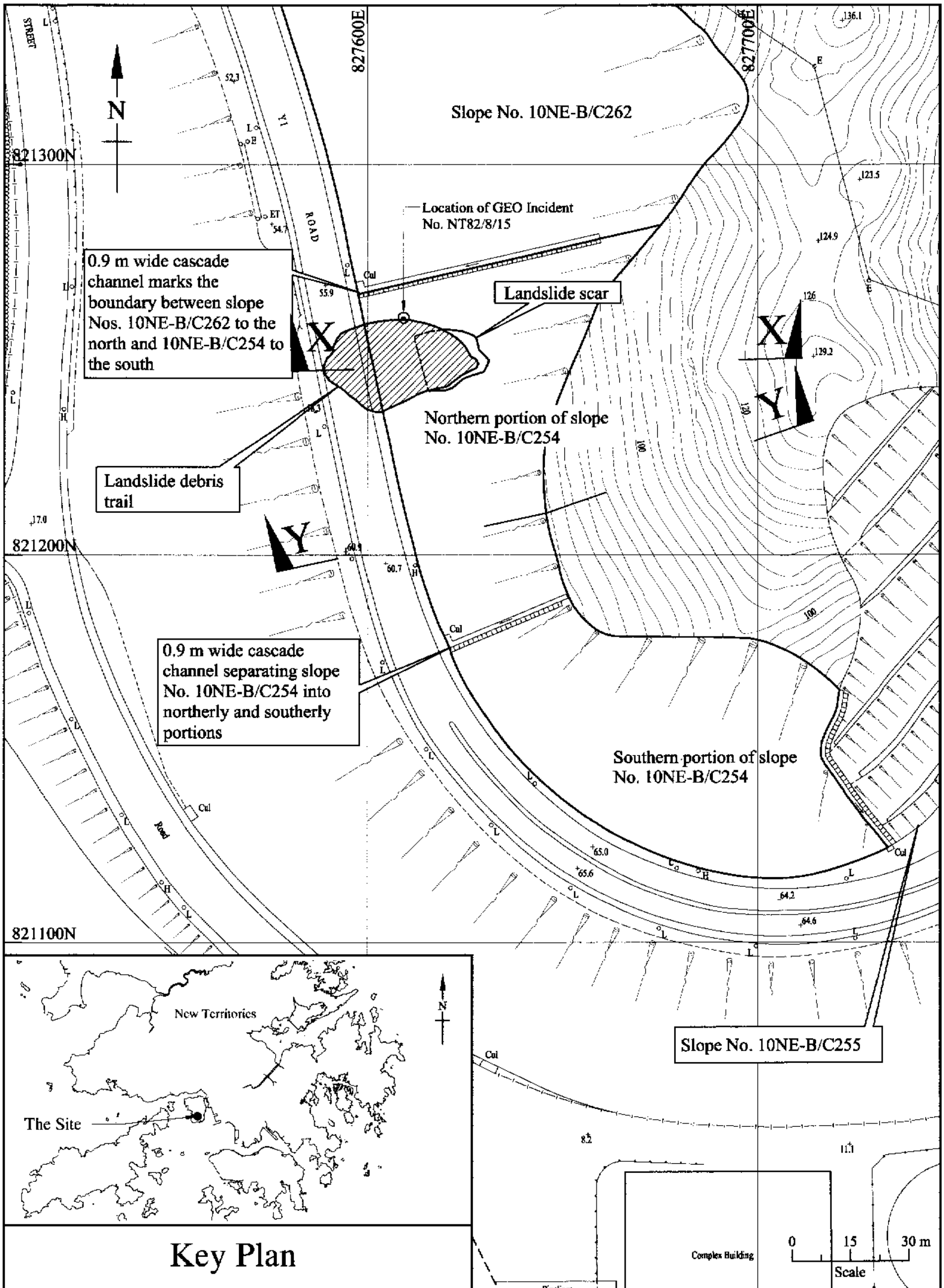
Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 Minutes	10.5	05:35 on 23 August 1999	2
15 Minutes	30.5	05:40 on 23 August 1999	3
1 Hour	90.0	06:10 on 23 August 1999	8
2 Hours	123.0	07:05 on 23 August 1999	5
4 Hours	165.5	08:55 on 23 August 1999	6
12 Hours	247.5	10:10 on 23 August 1999	6
24 Hours	373.5	11:55 on 23 August 1999	10
2 Days	599.5	08:00 on 24 August 1999	50
4 Days	606.0	08:00 on 24 August 1999	20
7 Days	606.0	08:00 on 24 August 1999	12
15 Days	804.5	08:00 on 24 August 1999	15
31 Days	931.5	08:00 on 24 August 1999	9

Notes: (1) Return periods were derived from Table 3, of Lam and Leung (1994).

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

(3) The use of 5-minute data for durations between 4 hours and 31 days results in better data resolution, but may slightly over-estimate the return periods using Lam and Leung (1994)'s data, which are based on hourly rainfall for these durations.

LIST OF FIGURES



Note: Base Map is extracted from Survey Sheet No. 10NE-14A, scale 1 : 1000.

Figure 1 - Site Location Plan

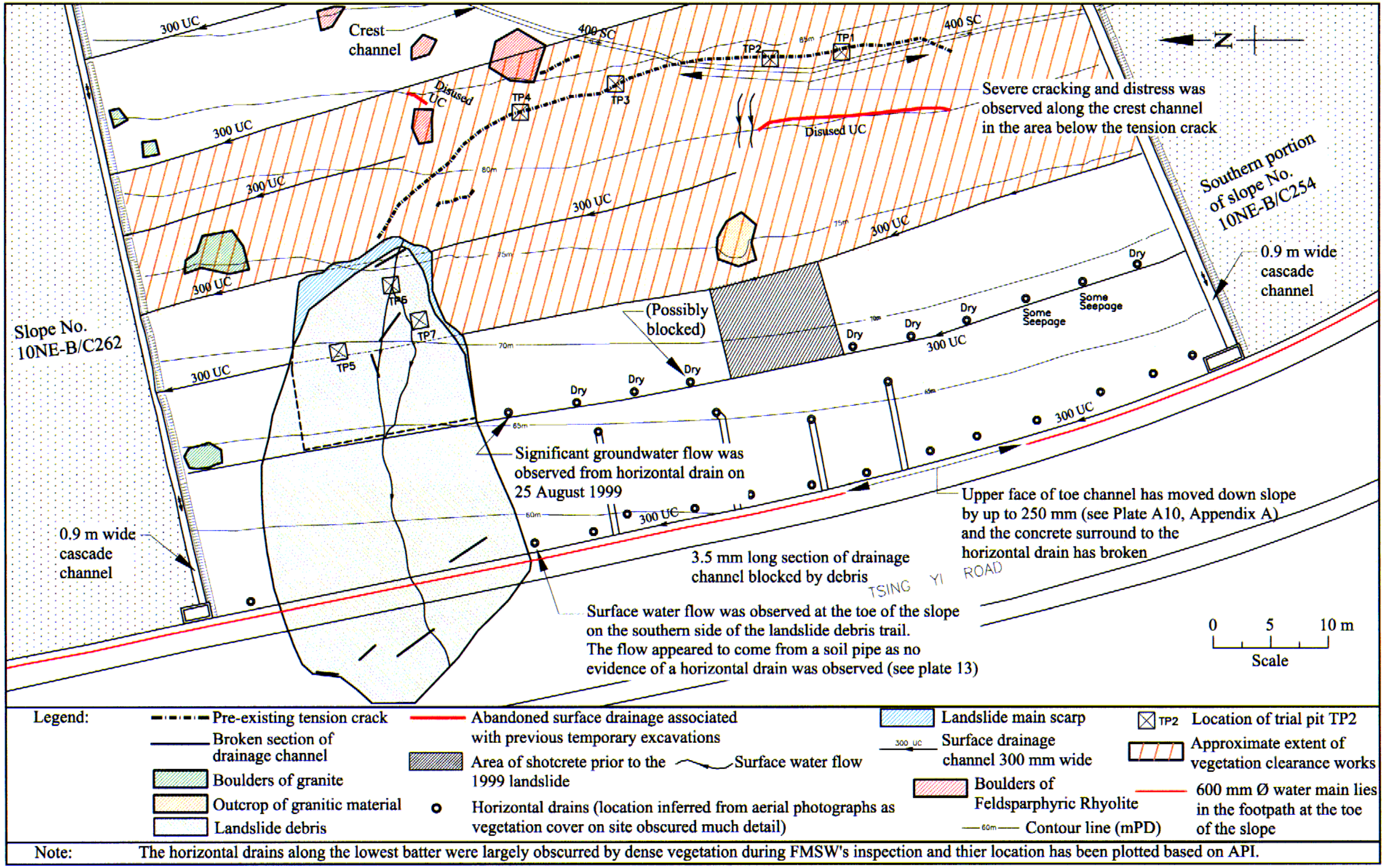


Figure 2 - Plan of the Landslide Site

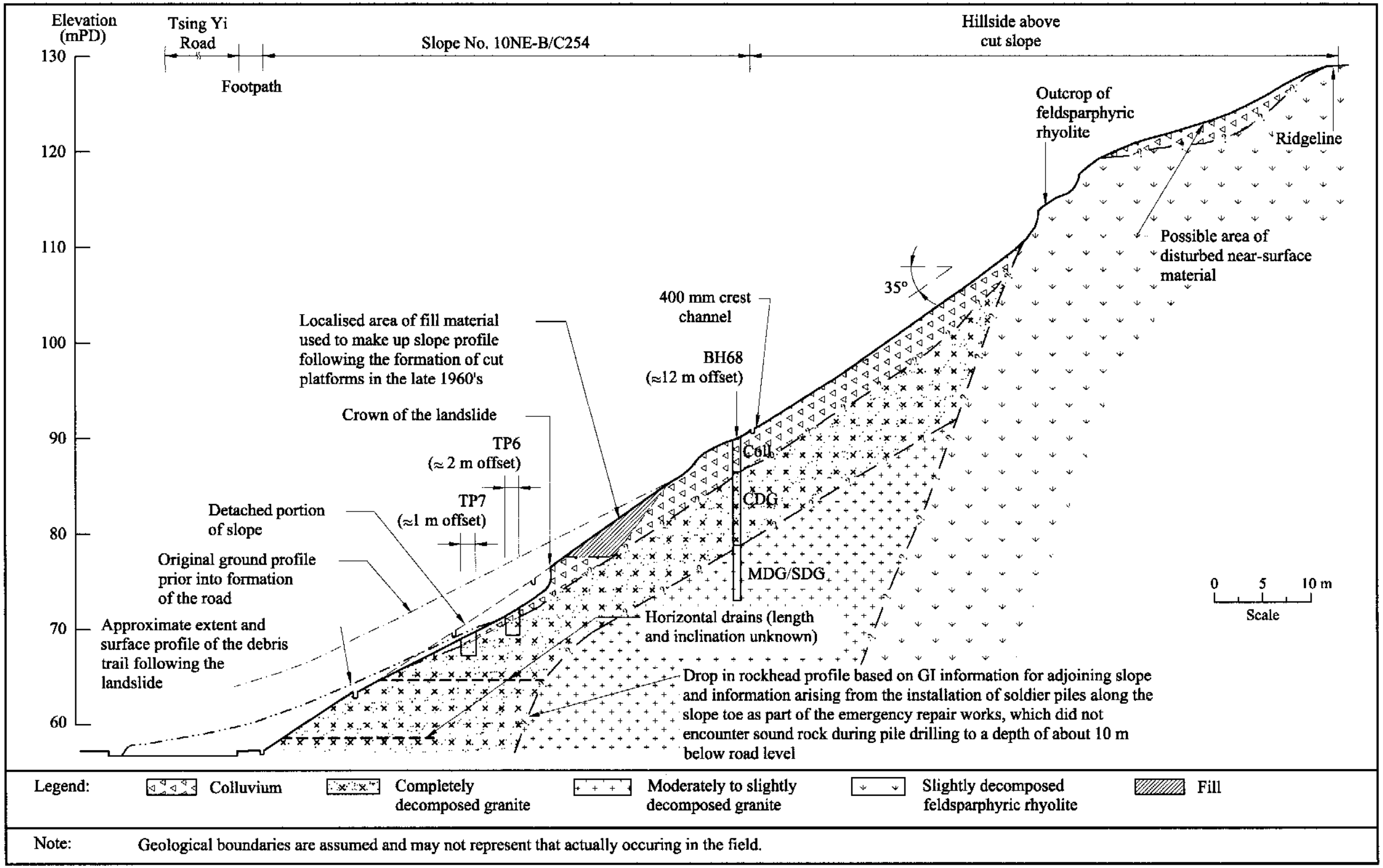


Figure 3 - Cross-section X-X through the Landslide Site

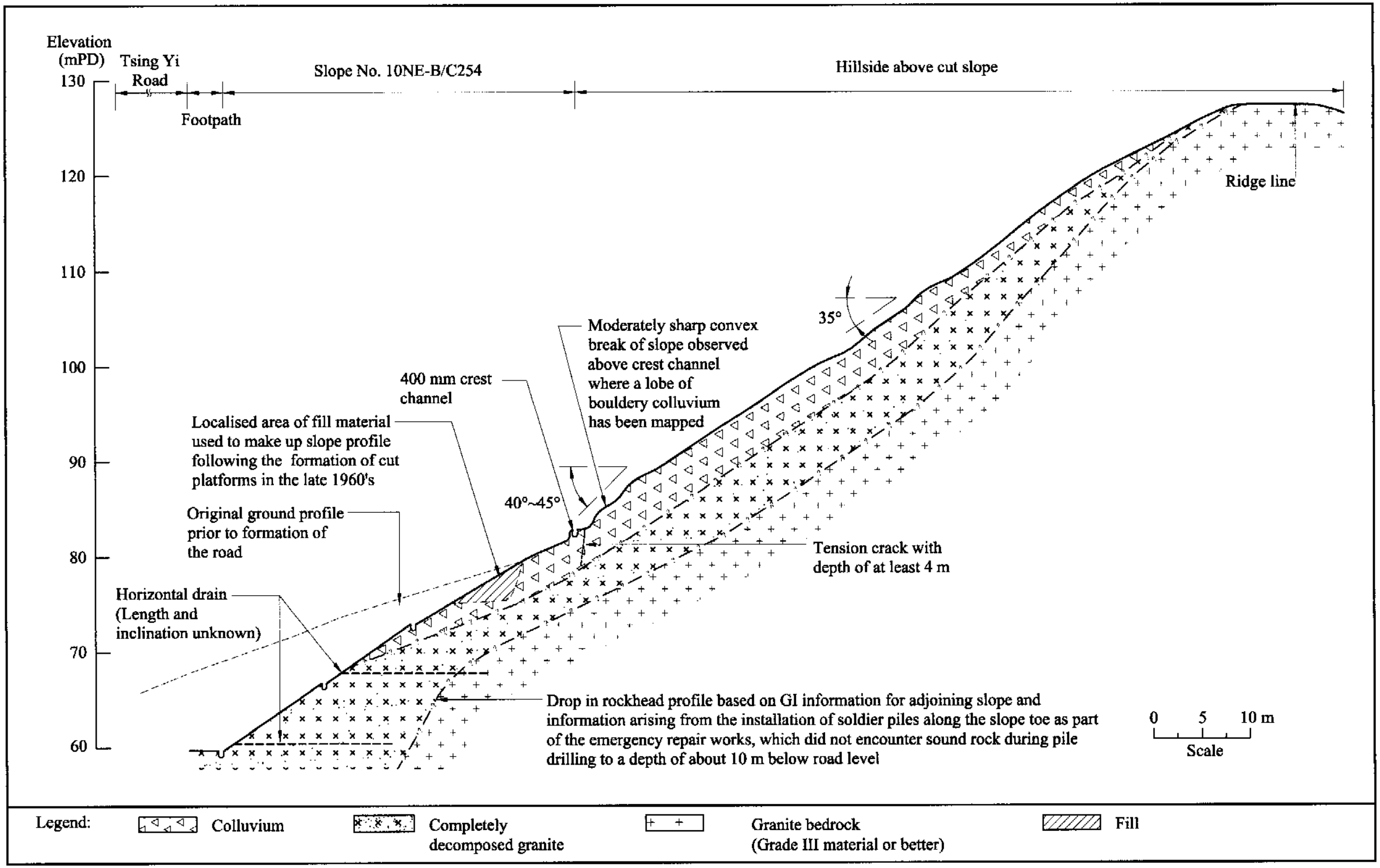


Figure 4 - Cross-section Y-Y through the Landslide Site

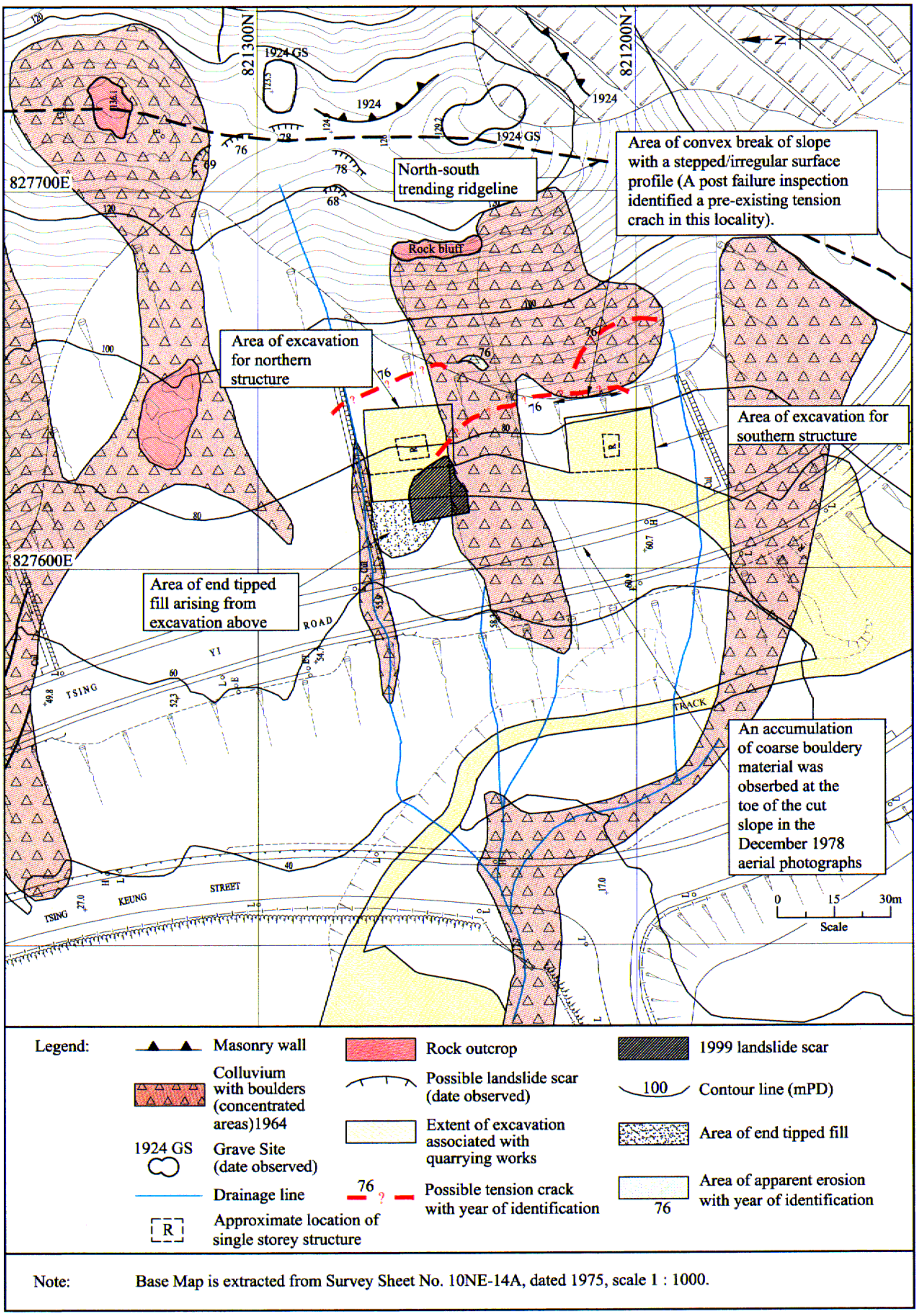


Figure 5 - Features Observed from API (Sheet 1 of 2)

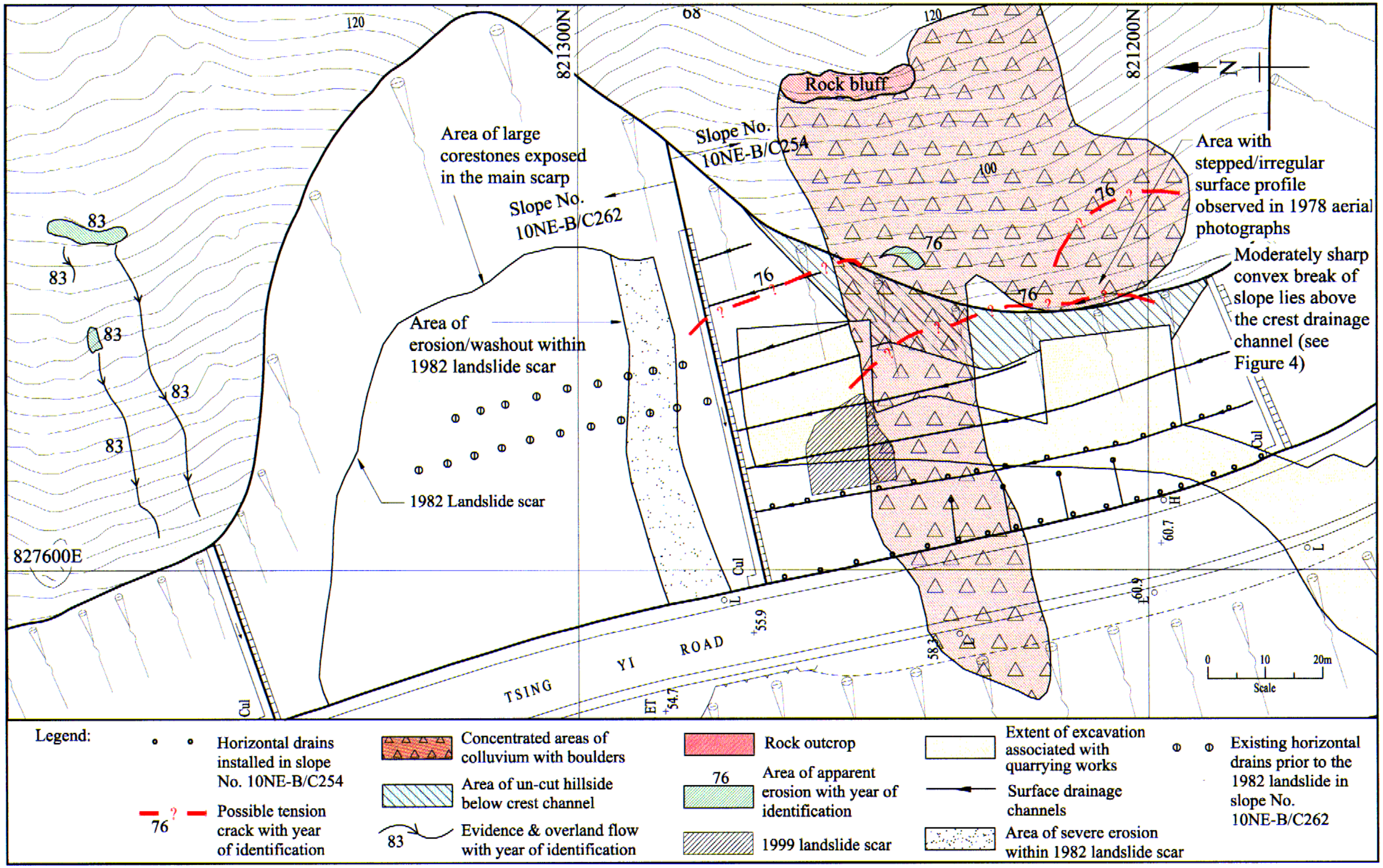


Figure 6 - Features Observed from API (Sheet 2 of 2)

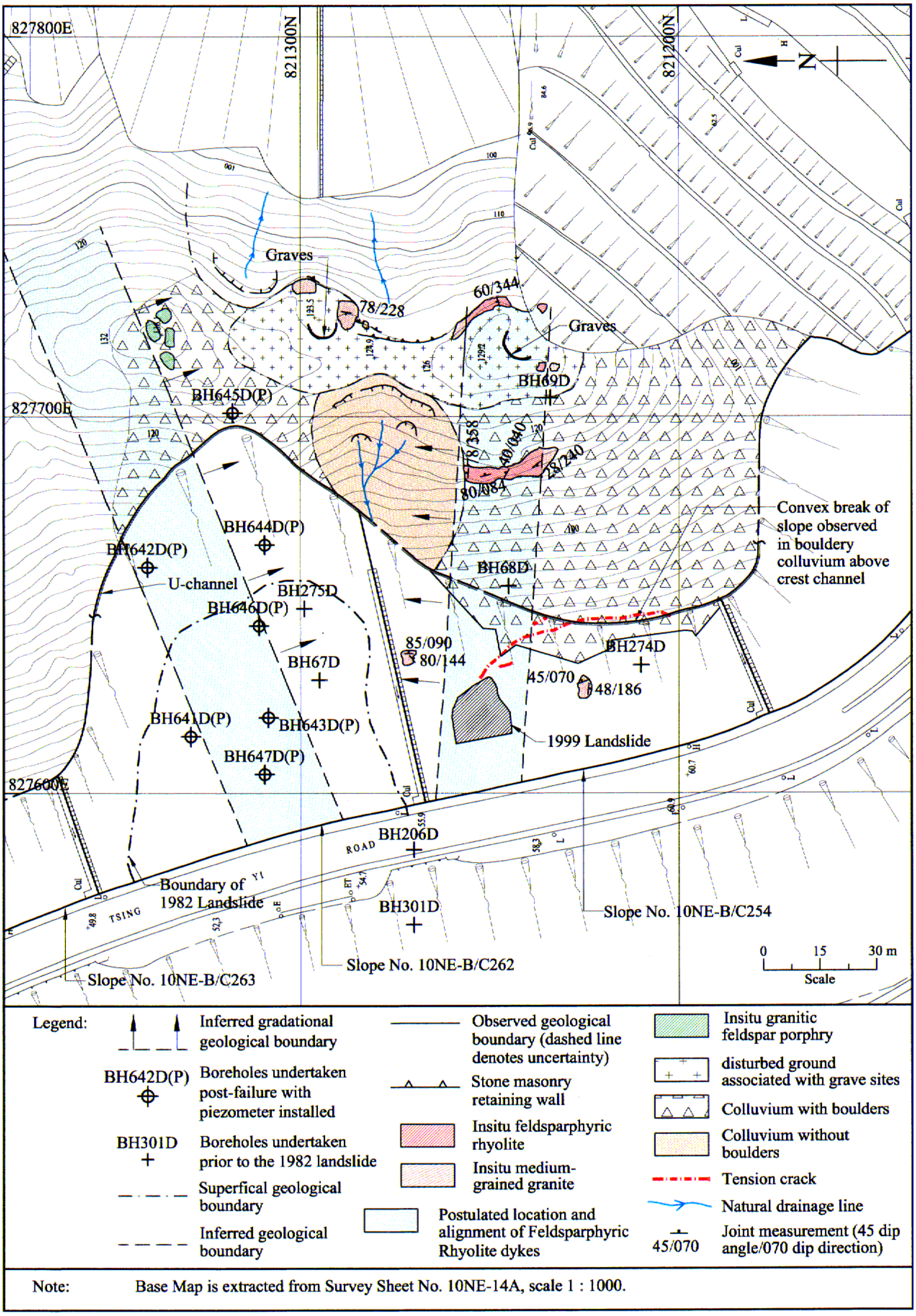
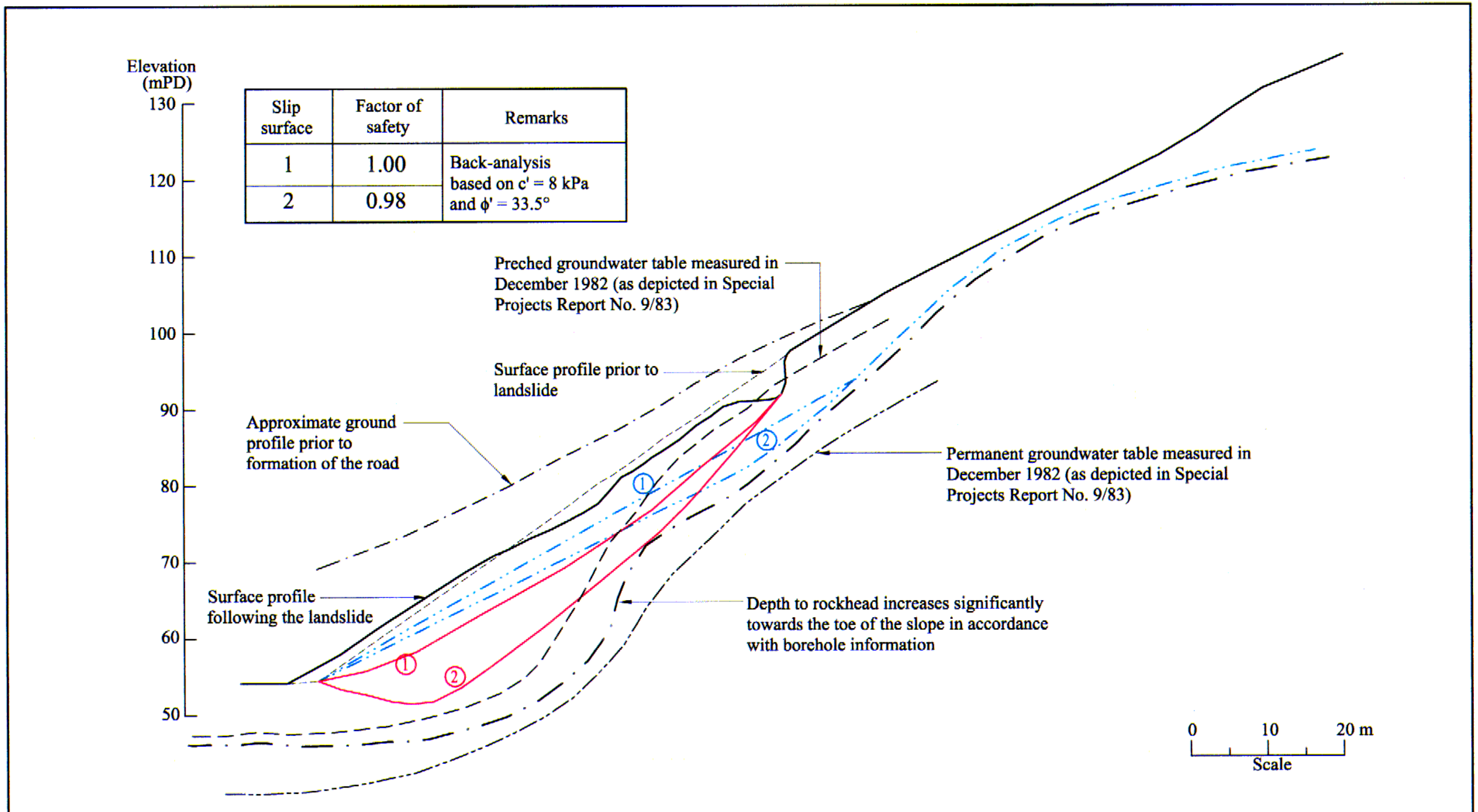


Figure 7 - Geology of the Landslide Site Based on Field Mapping

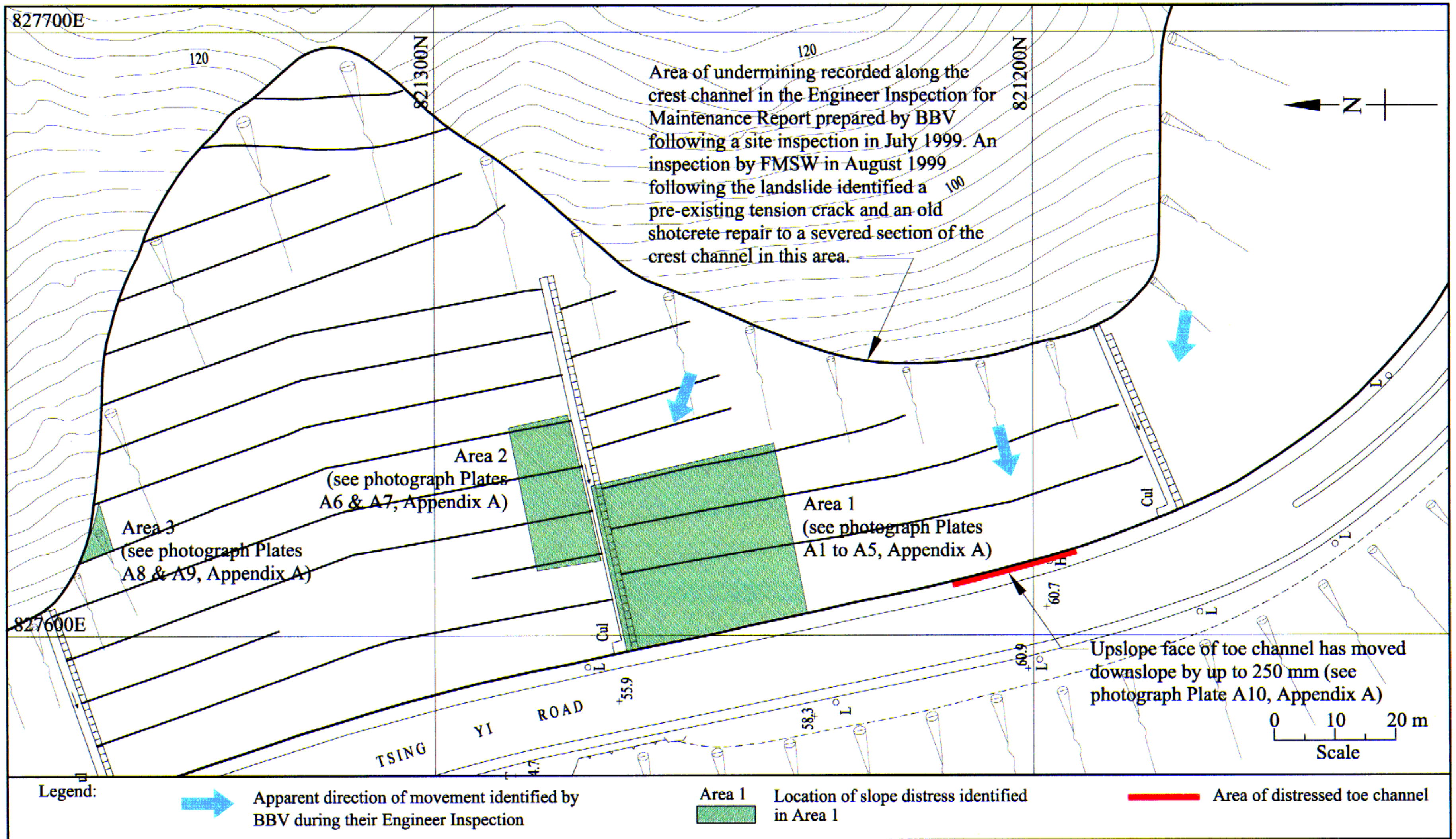


Slip surface	Factor of safety	Remarks
1	1.00	Back-analysis based on c' = 8 kPa and ϕ' = 33.5°
2	0.98	

Legend: ① Water Table associated with slip surface No. 1 ① Slip surface No. 1 Rockhead profile (Grade III material or better)

Note: Cross-section extracted from Scott Wilson's Landslide Investigation report "Geotechnical Assessment of Slope Above Road TYI South-west at Chainages 2340 m to 2440 m -April 1983".

Figure 8 - Cross-section through the 1982 Landslide at Slope No. 10NE-B/C262



Notes:
(1) Photographs of the distress observed in Areas 1 to 3 are presented in Appendix A, and a more detailed plan of the distress in Areas 1 and 2 is presented in Figure 10.
(2) Observed distress is based on site inspection records by HyD, GEO and FMSW between July 1998 and August 1999.

Figure 9 - Plan Showing Areas of Slope Distress Observed before the 1999 Landslide

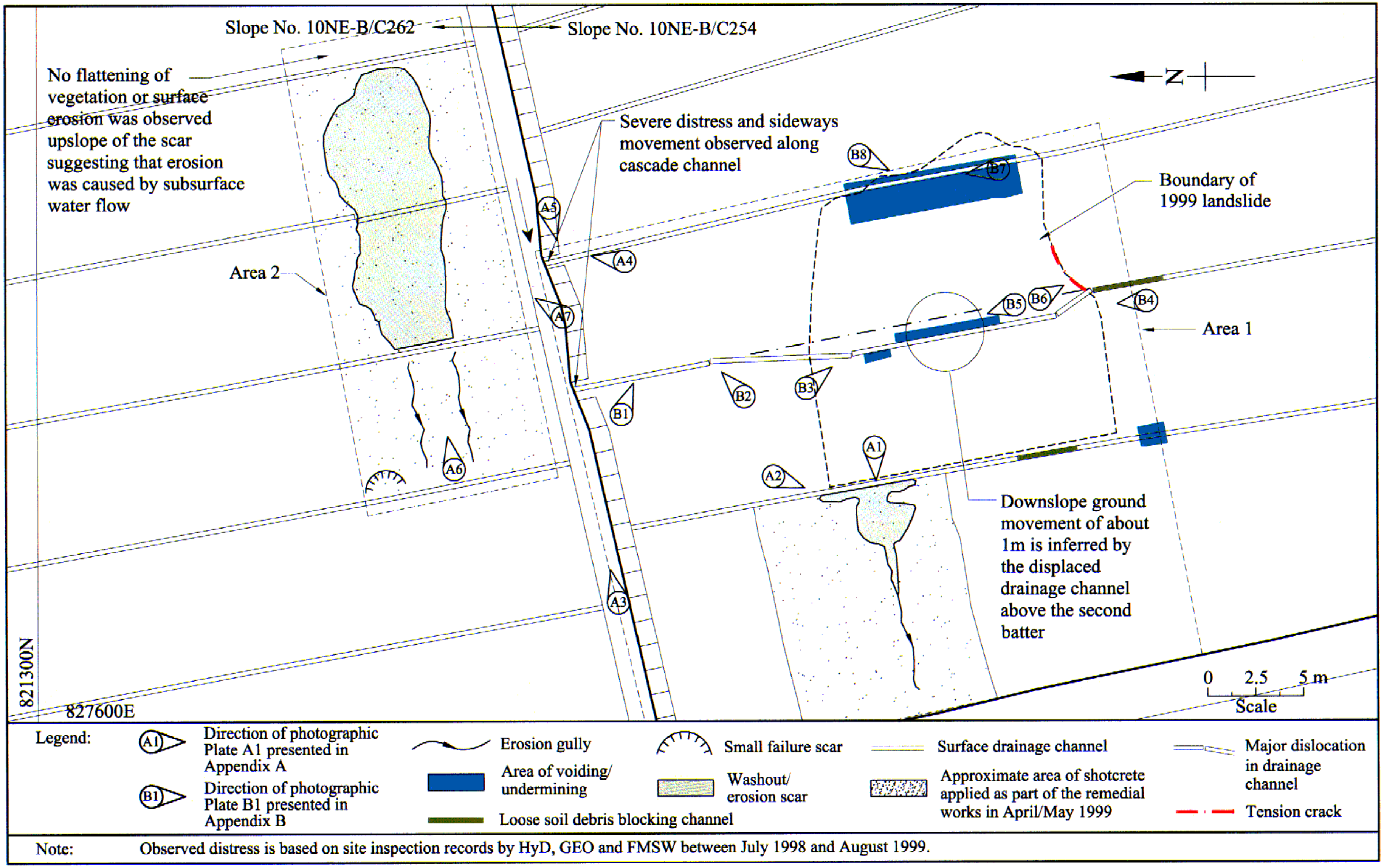


Figure 10 - Plan Showing Slope Distress Observed in Area 1 and Area 2 before the 1999 Landslide

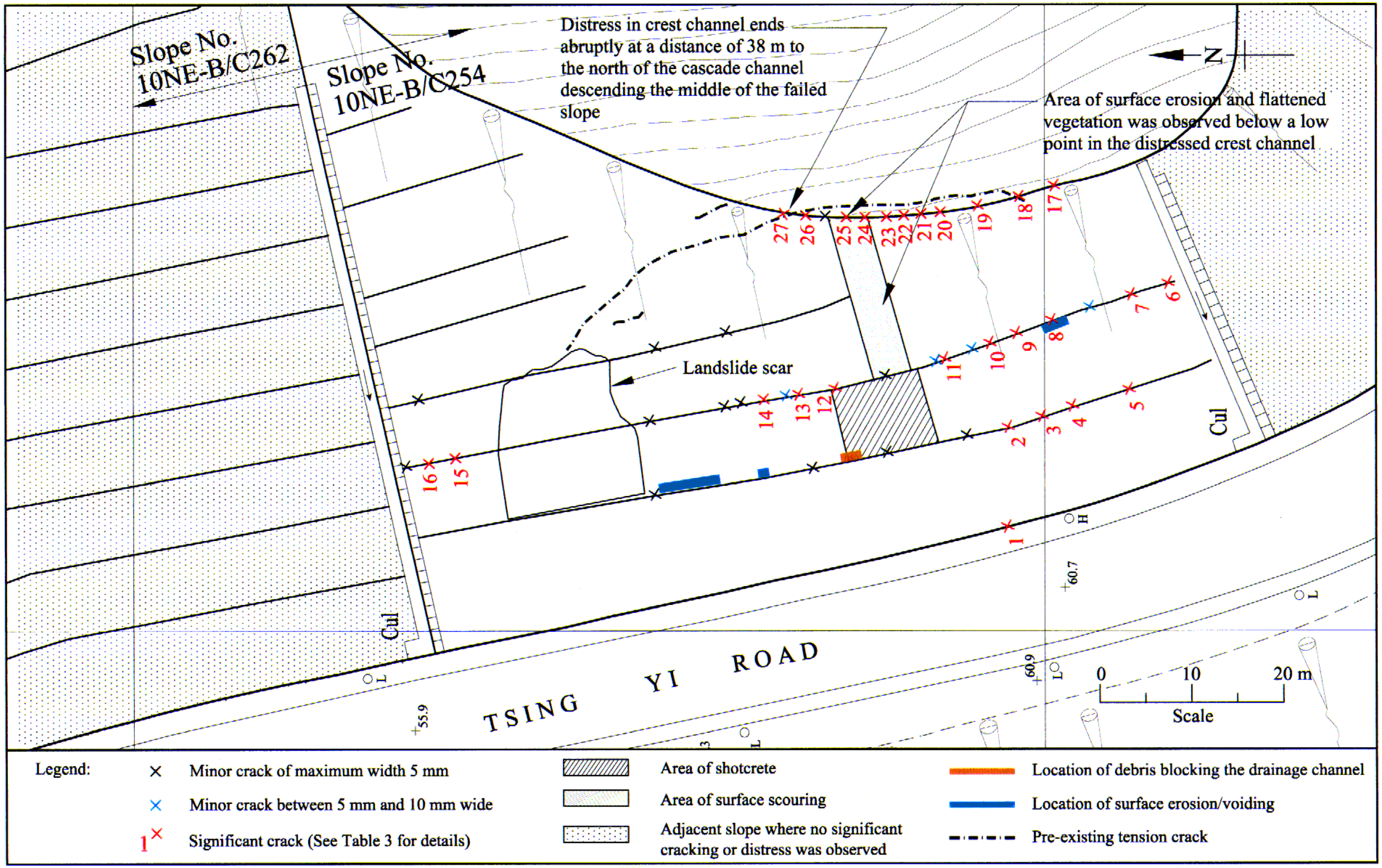
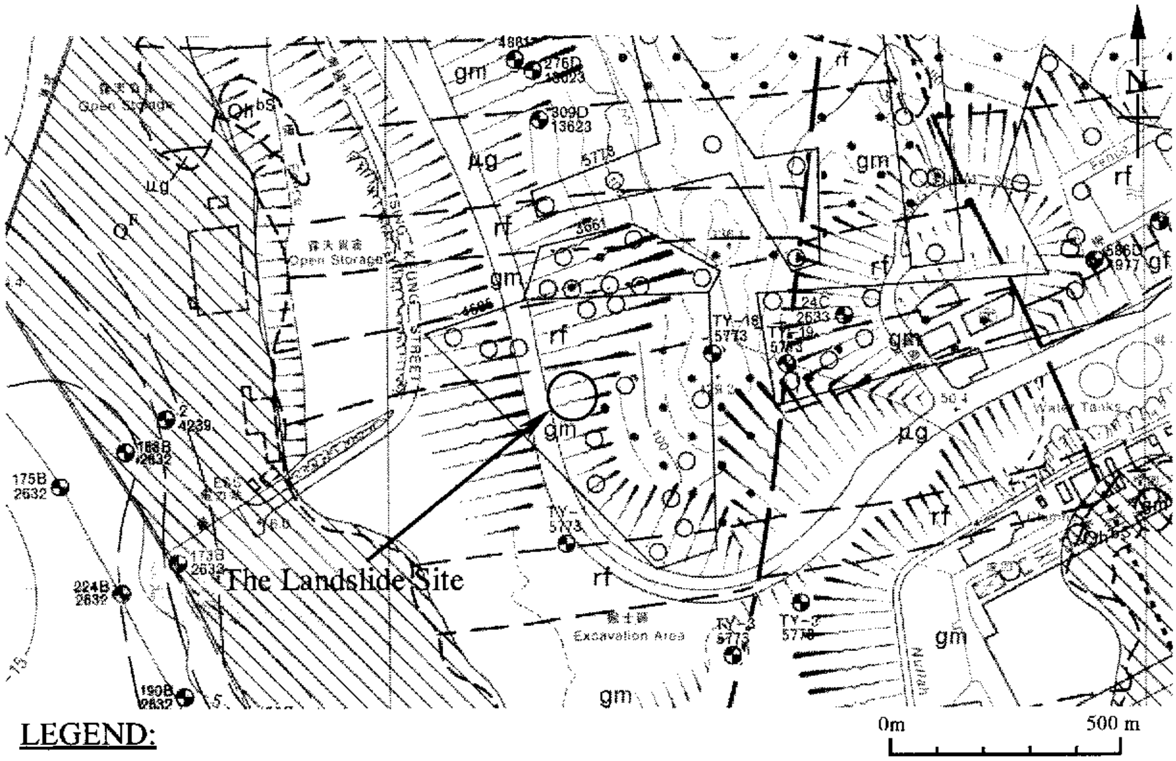


Figure 11 - Plan Showing Slope Distress Observed Following the 1999 Landslide



LEGEND:

SUPERFICIAL DEPOSITS

- Q^F — Natural earth and waste
- Qh^bS — Sand
- Qh^mS — Sand; some gravel and mud
- Q^sD — Sand, gravel, cobbles and boulders in silt matrix

SOLID GEOLOGY

- gm — **Medium-grained granite**
- gm (megacrystic) — Megacrystic Medium-grained granite
- rf — Feldsparphyric rhyolite
- μg — Porphyritic microgranite

GEOLOGICAL LINES

- Geological boundary, superficial deposits
- Fill boundary
- Geological boundary, solid rock
- Fault
- Concealed fault
- Mineral vein
- Photogeological lineament (mapped by HKGS)
- Isopach of Hang Hau Formation, 5 m interval
- Limit of acoustic turbidiry

(* Broken lines on map face denote uncertainty)

STRUCTURAL SYMBOLS

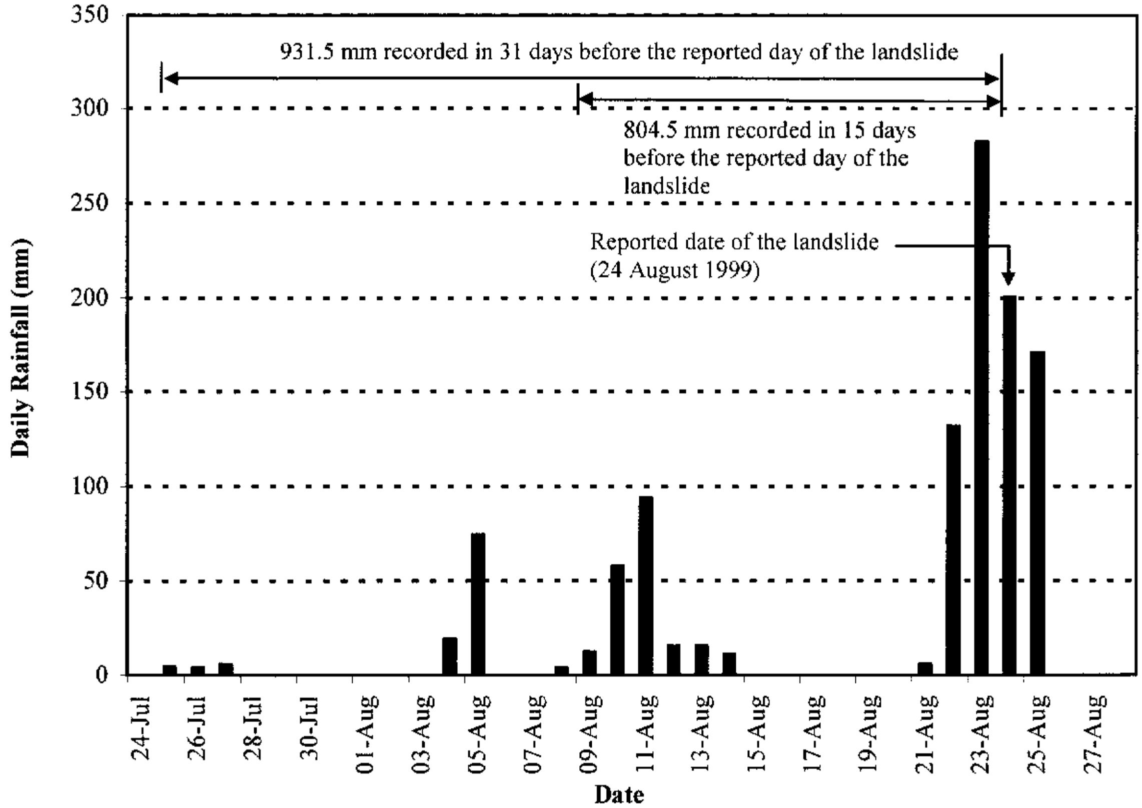
- 20 Inclined bedding
- 20 Inclined jointing

BOREHOLE SYMBOLS

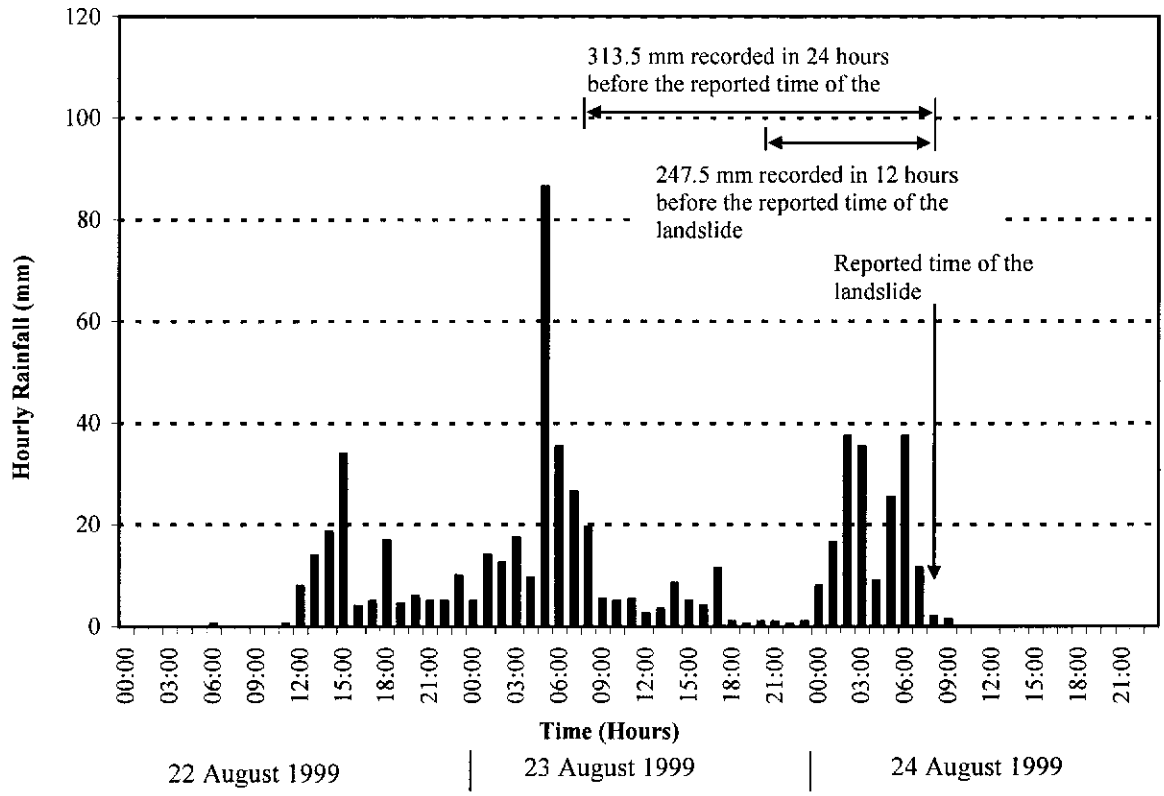
- Selected boreholes are shown with the borehole number above the GIU number of the relevant.
- Where the borehole density is too great to show individual borehole numbers at this scale, the site investigation areas are outlined and only show borehole locations with the relevant GIU report number.

Notes: (1) Geology relevant to study area shown bold.
(2) Extracted from Sheet 10-NE-B, D of the Hong Kong Geological Survey 1 : 5,000 Scale Map Series HGP5 (GEO, 1995).

Figure 12 – Solid and Superficial Geology of the Site

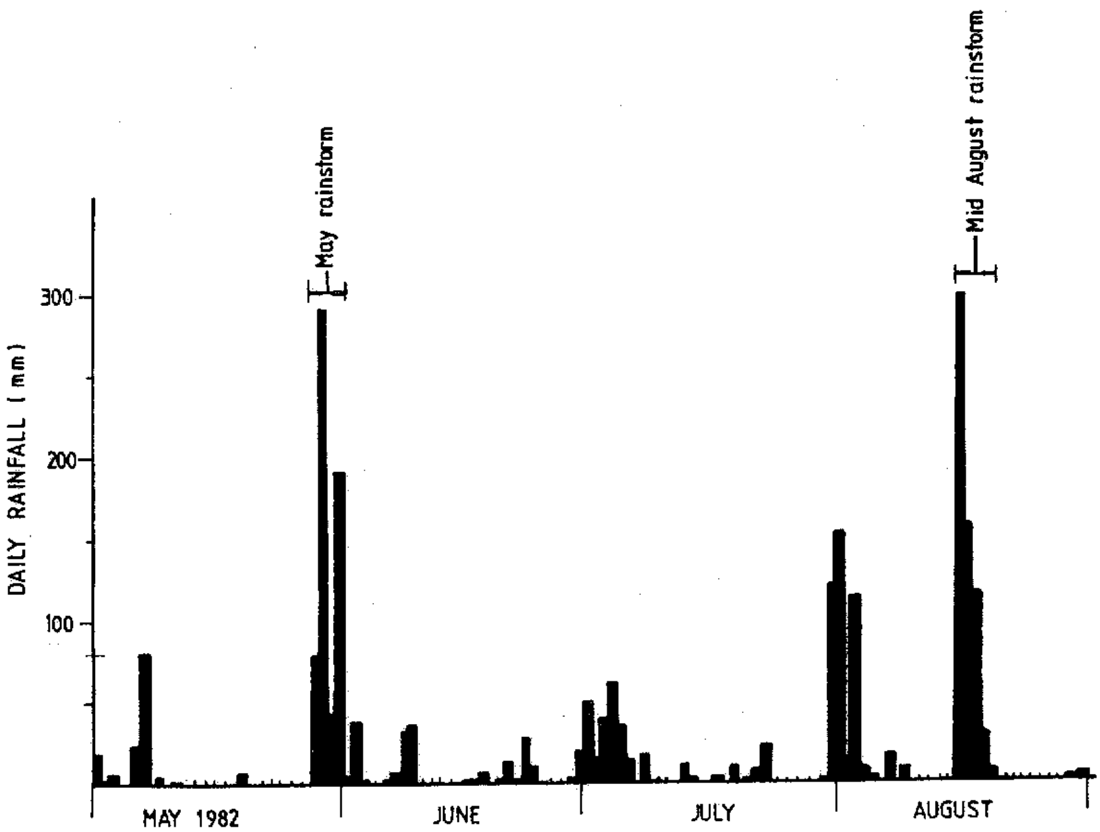


(a) Daily Rainfall Recorded at GEO Raingauge No. N11 from 24 July to 27 August 1999



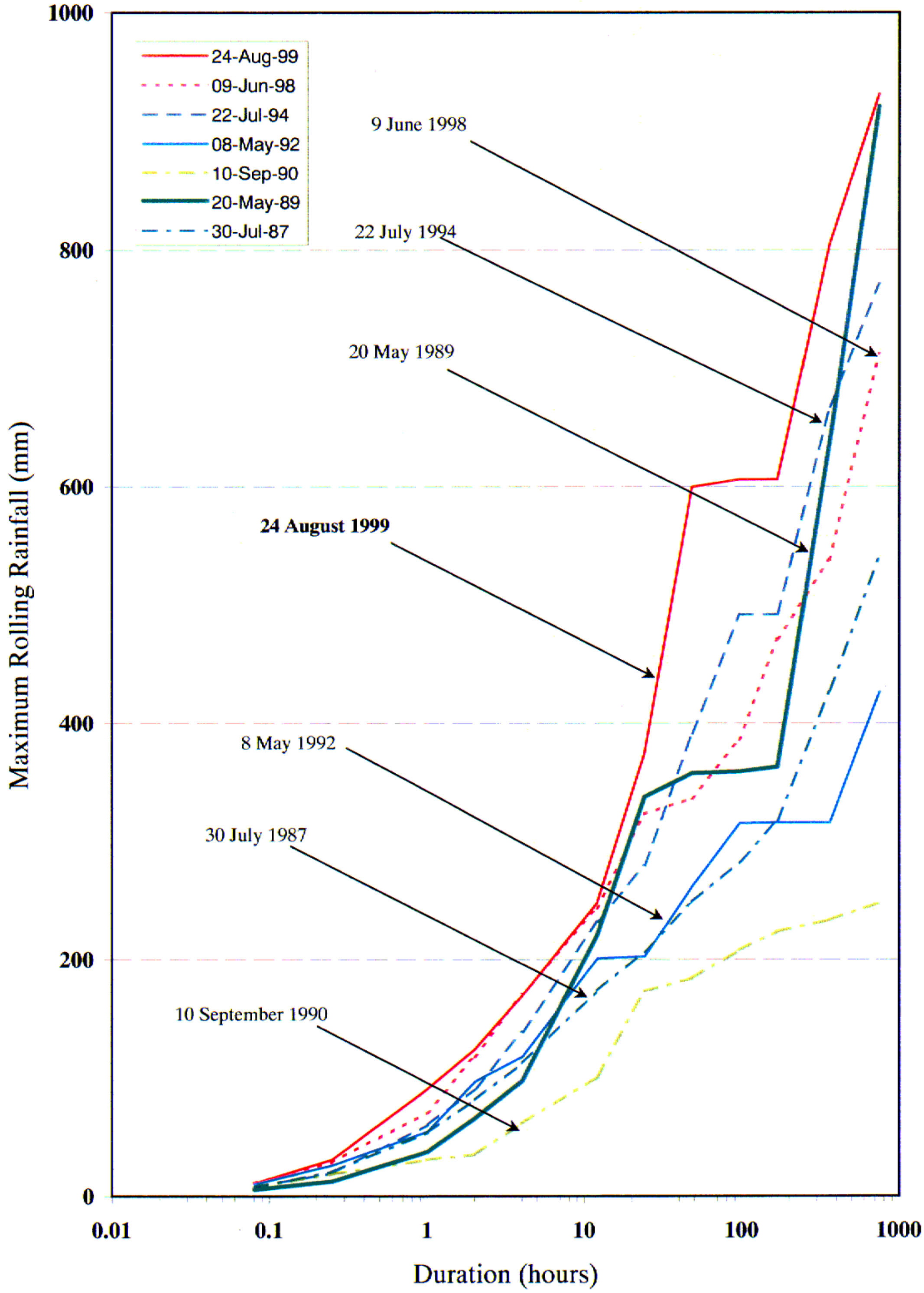
(b) Hourly Rainfall Recorded at GEO Raingauge No. N11 from 22 August to 24 August 1999

Figure 13 – Daily and Hourly Rainfall Recorded at GEO Raingauge No. N11



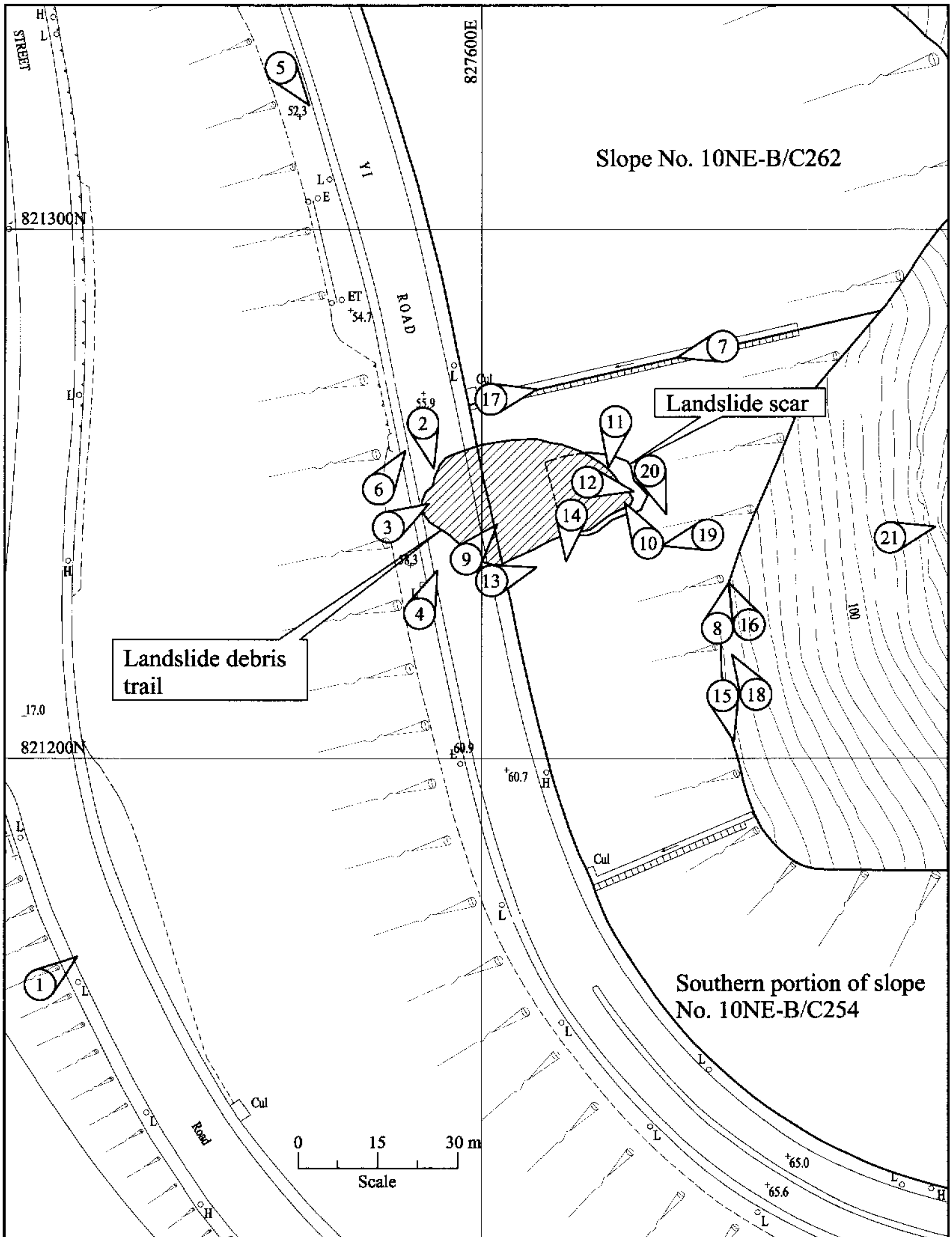
Note: Information extracted from Special Project Report No. 9/83.

Figure 14 – Daily Rainfall Recorded at RO Raingauge No. 105 for the May 1982 Rainstorm



Note: Rainfall data are 5-minute statistics from GEO raingauge No. N11.

Figure 15 - Maximum Rolling Rainfall at GEO Raingauge No. N11 for Major Rainstorms



Legend:	Location and direction of Photographic Plate No.1
Note:	Base Map is extracted from Survey Sheet No. 10NE-14A, scale 1 : 1000.

Figure 16 - Plan of the Landslide Site Showing the Location and Direction of Photographic Plates

LIST OF PLATES

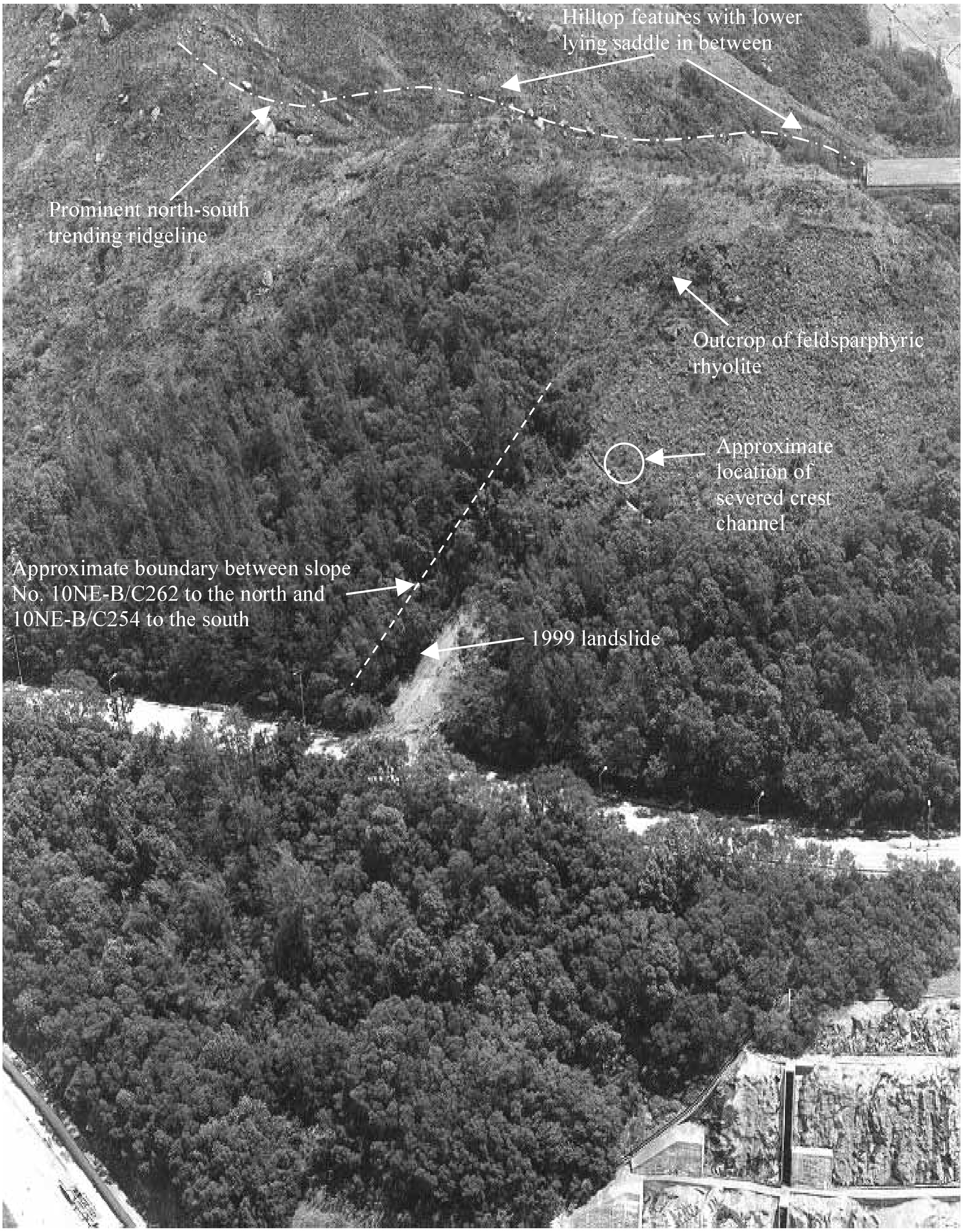


Plate 1 – Oblique Aerial Photograph (GEO Photograph Reference No.PS1000-7) (Photograph Taken on 26 August 1999)

Note: See Figure 16 for Location.

Plate 2 – View of the Taxi and Lorry Trapped in the Landslide Debris at the Toe of the Slope (Photograph Taken on 24 August 1999)

Note: See Figure 16 for Location.

Plate 3 – General View of the Landslide (Photographs Taken on 25 August 1999)

Note: See Figure 16 for Location.

Plate 4 – View of Landslide Debris Covering Tsing Yi Road at the Toe of the Slope
(Photographs Taken on 25 August 1999)

Note: See Figure 16 for Location.

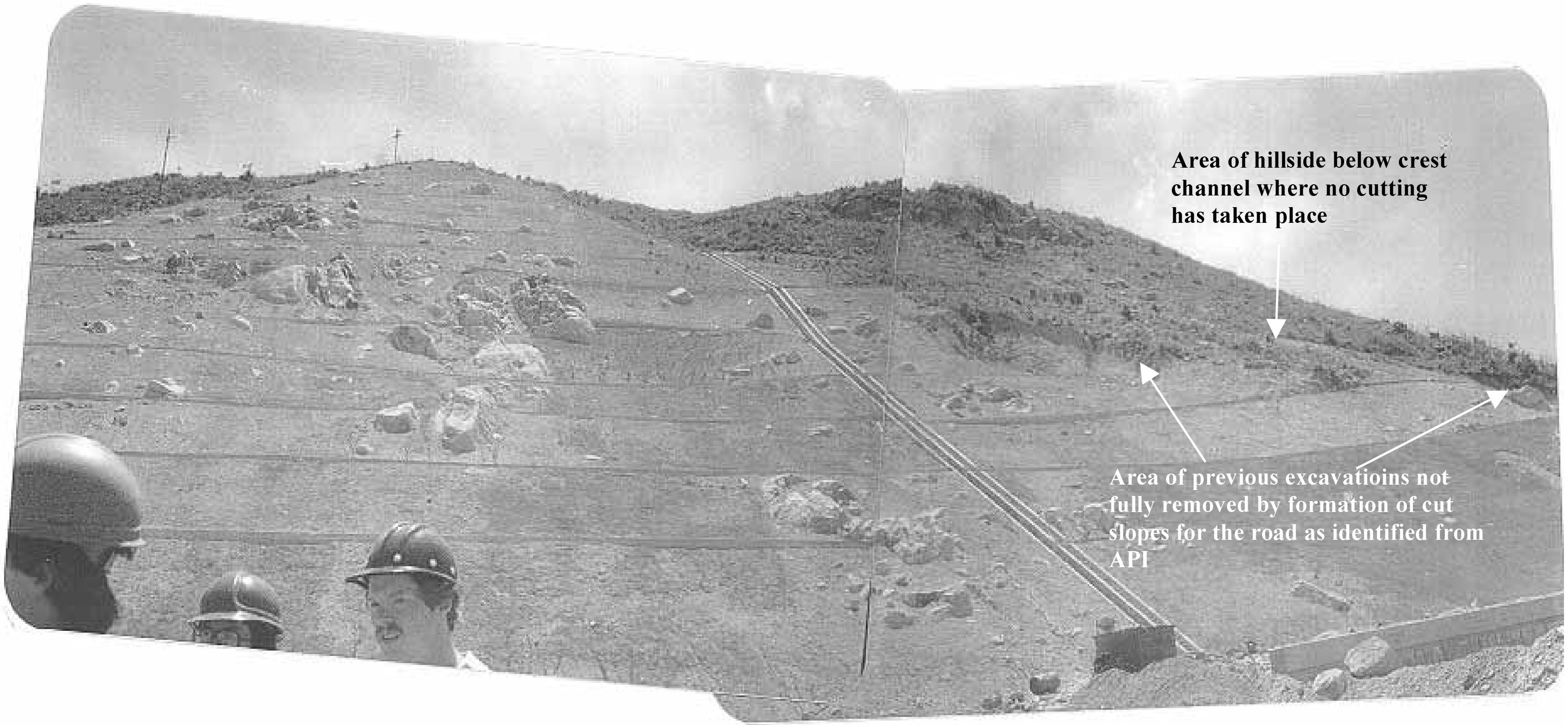


Plate 5 – View of Slope No. 10NE-B/C254 and 10NE-B/C262 During Formation in July 1979
(Photographs Taken on 26 July 1979)

Note: See Figure 16 for Location.

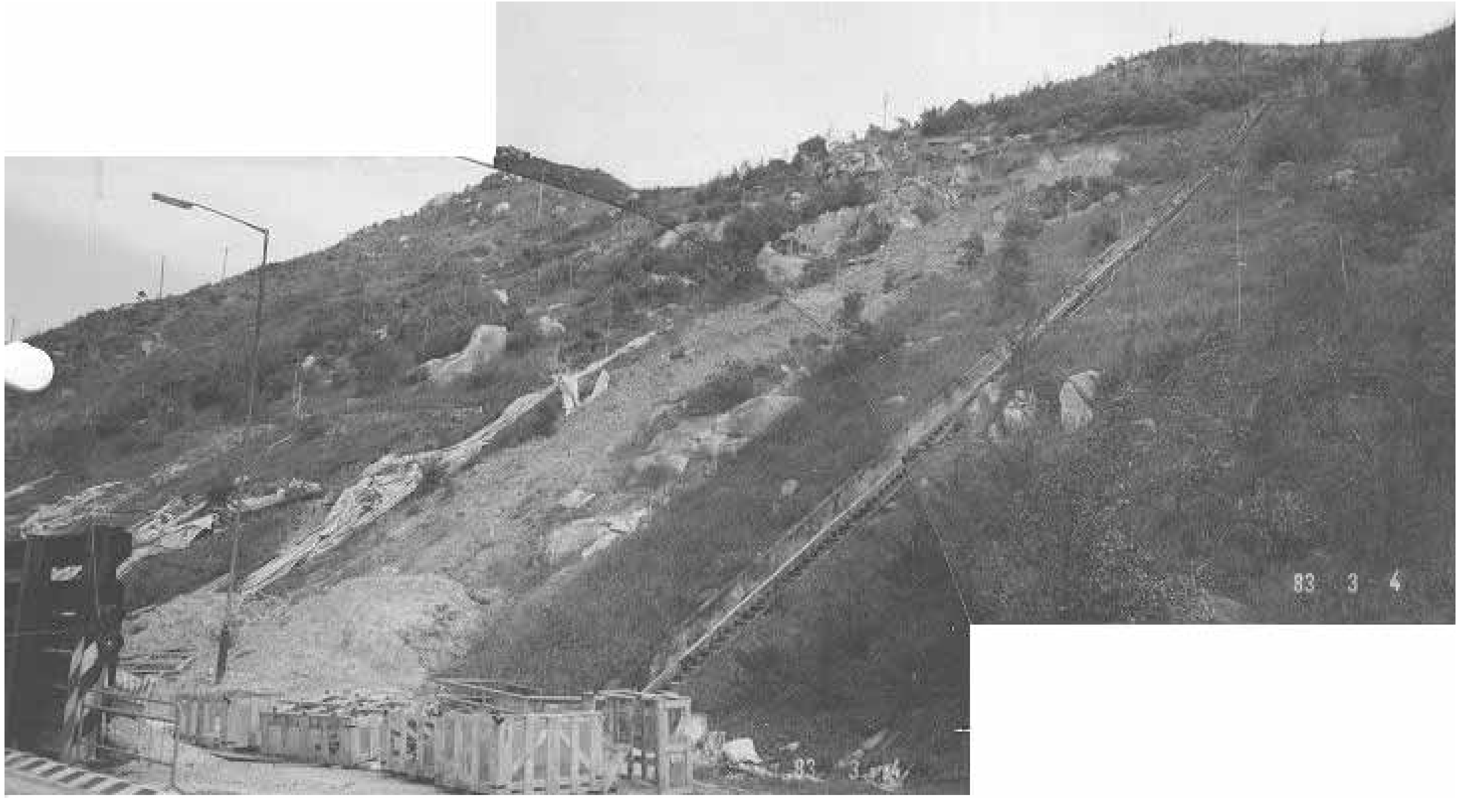


Plate 6 – View of 1982 Landslide Located in Slope No. 10NE-B/C262
(Photographs Taken on 4 March 1983)

Note: See Figure 16 for Location.

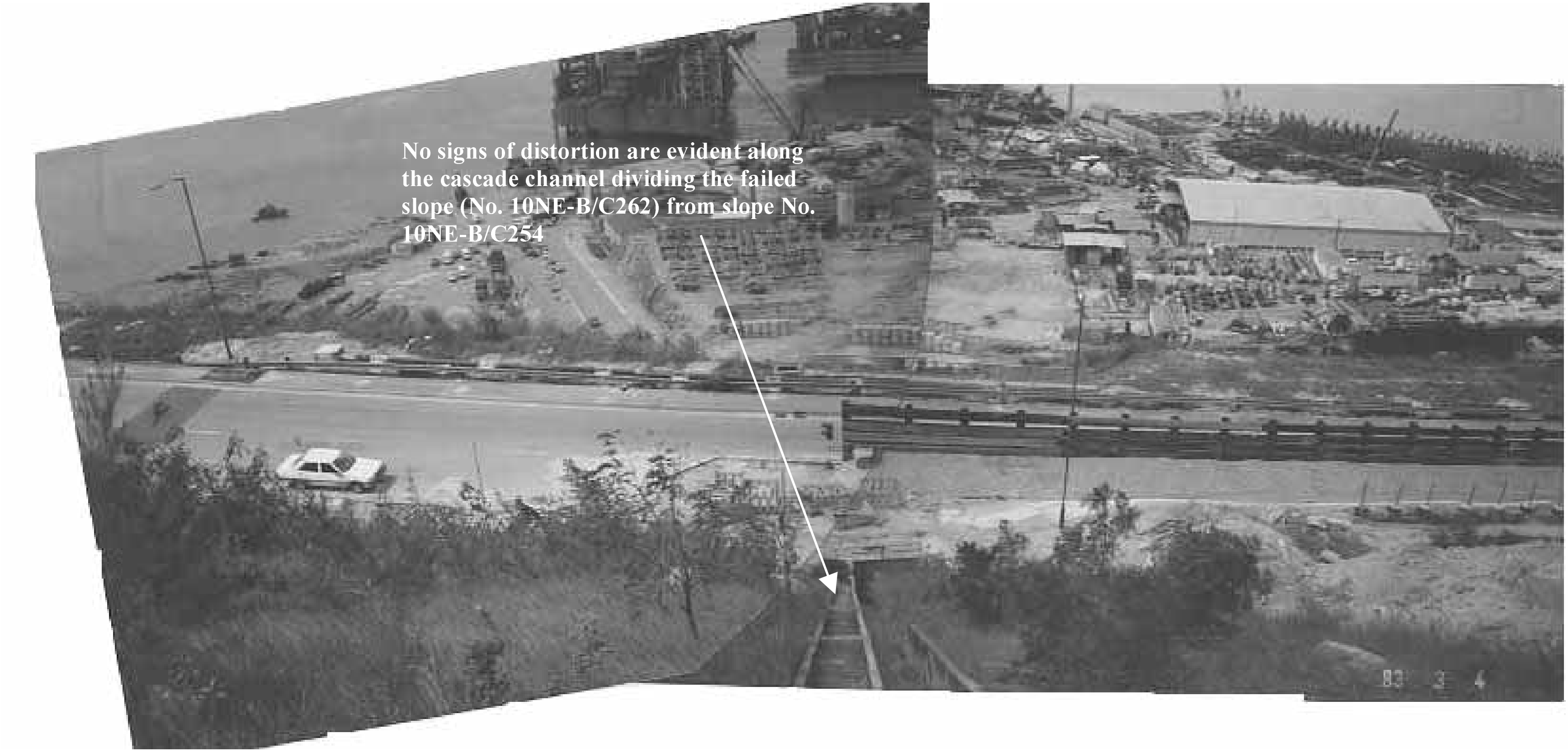


Plate 7 – View of Cascade Channel between Slope No. 10NE-B/C254 and 10NE-B/C262
(Photographs Taken on 4 March 1983)

Note: See Figure 16 for Location.

Old shotcrete repair to severed crest channel indicates that about 400 mm of vertical displacement had occurred prior to movements associated with the 1999 landslide

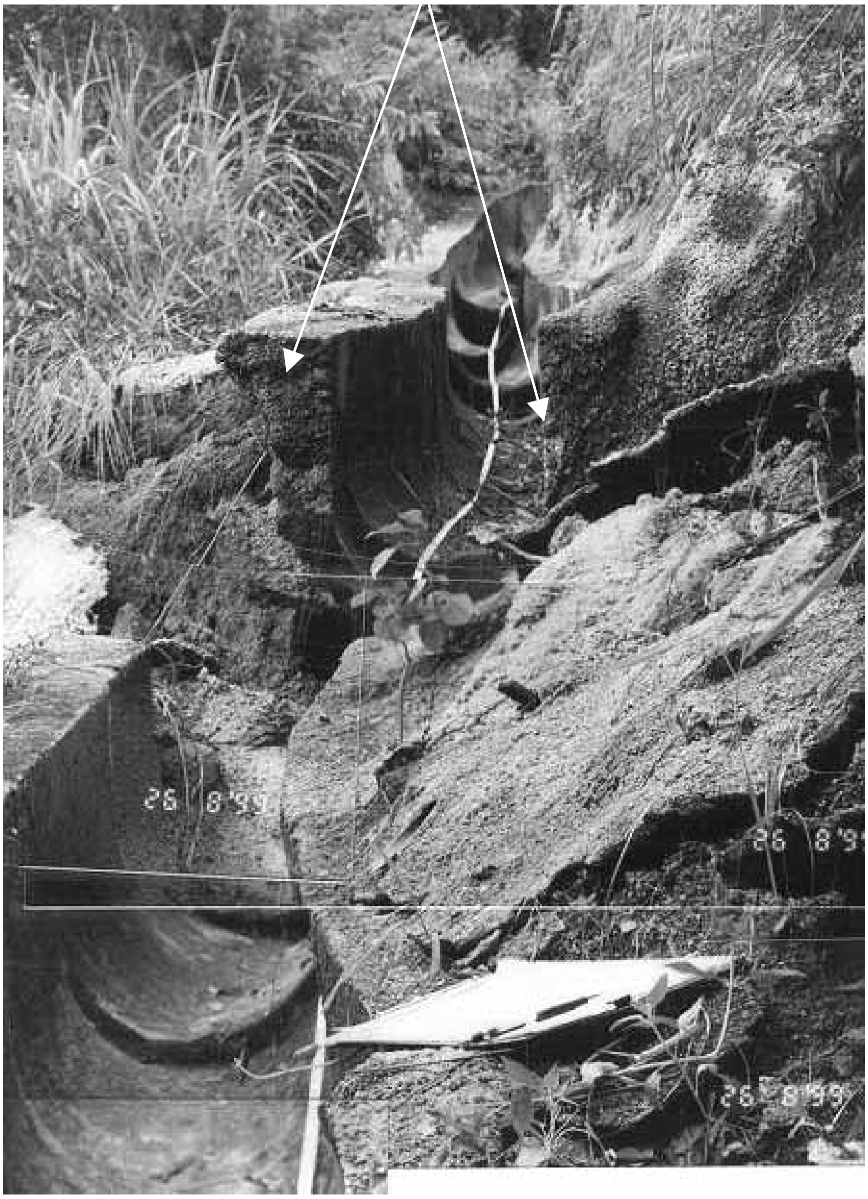


Plate 8 – View of Old Shotcrete Repair to Severed Crest Channel (Photographs Taken on 26 August 1999)

Note: See Figure 16 for Location.

Plate 9 – General View of the Landslide Scar Following Removal of Failed Material
(Photographs Taken on 27 August 1999)

Note: See Figure 16 for Location.

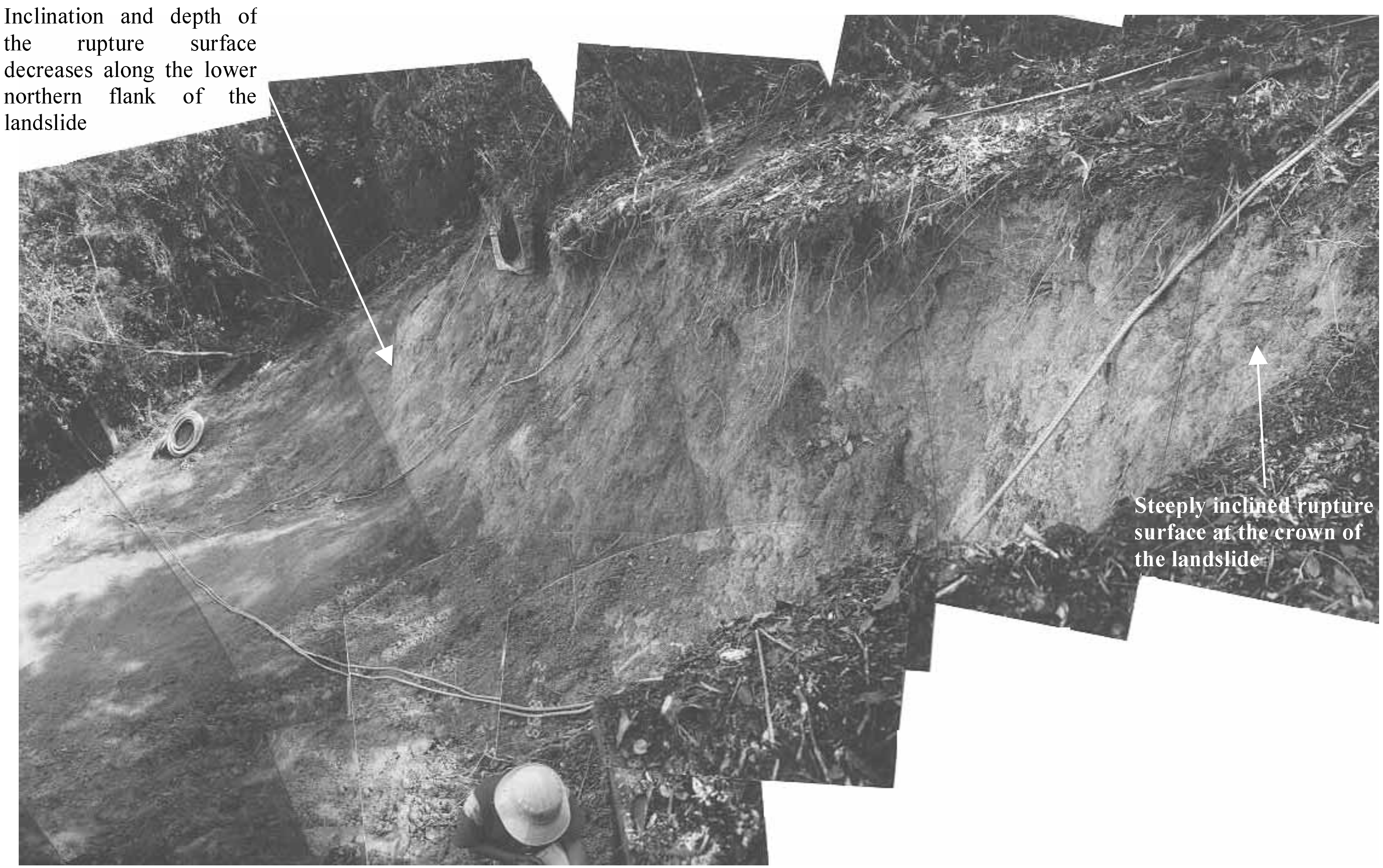


Plate 10 – View of the Northern Portion of the Main Scarp (Photographs Taken on 27 August 1999)

Note: See Figure 16 for Location.

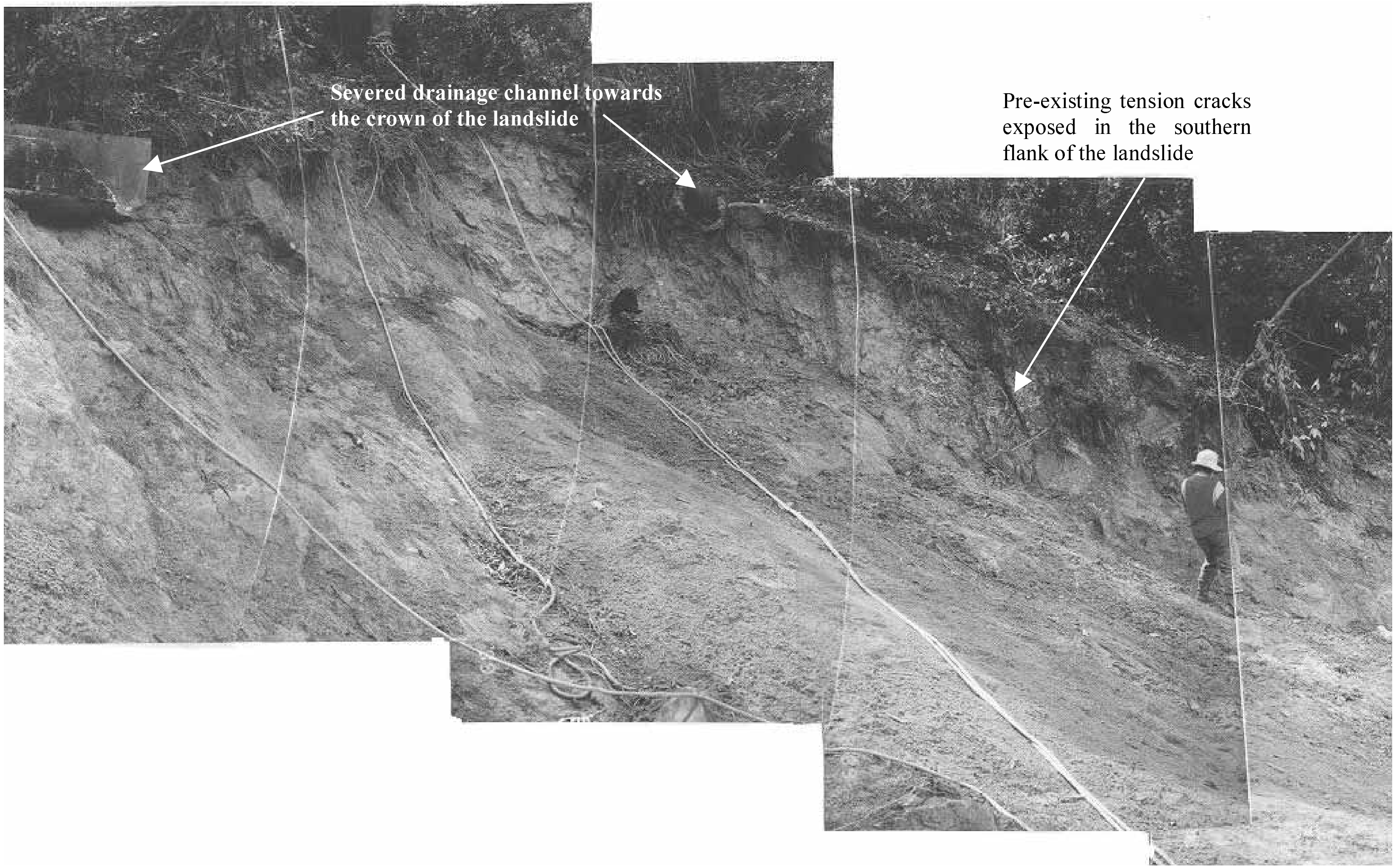
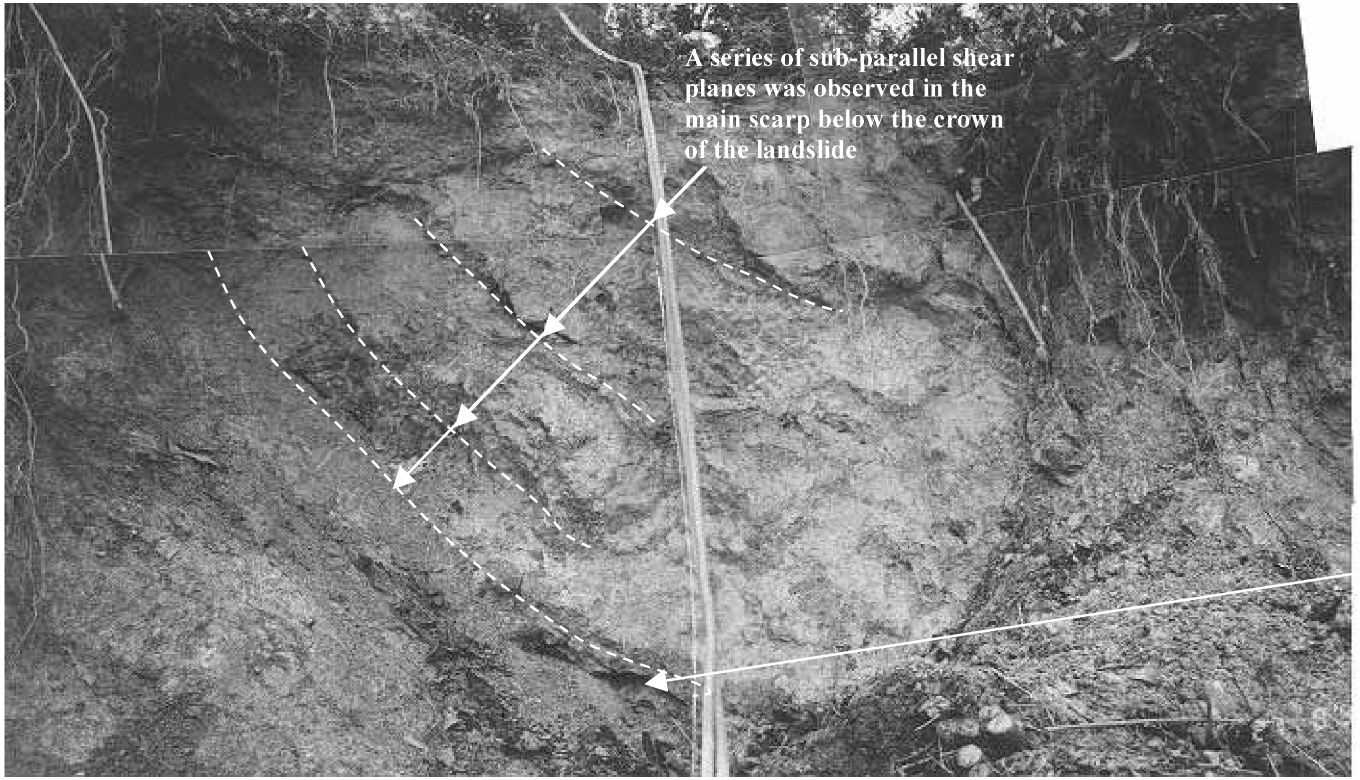


Plate 11 – View of the Southern Portion of the Main Scarp
(Photographs Taken on 27 August 1999)

Note: See Figure 16 for Location.



Ground water was seen flowing from the shear plane exposed at the base of the main scarp at the crown of the landslide on 25 August 1999

Plate 12 – View of the Main Scarp at the Crown of the Landslide
(Photographs Taken on 27 August 1999)

Note: See Figure 16 for Location.

Plate 13 – View of Groundwater Flows from a Possible Soil Pipe at the Base of the Failed Slope (Photographs Taken on 27 August 1999)

Note: See Figure 16 for Location.

Plate 14 – View of Strong Groundwater Flows from the Horizontal Drain to the South of the 1999 Landslide (Photographs Taken on 25 August 1999)

Note: See Figure 16 for Location.

Plate 15 – View of Sheared and Dislocated Crest Channel
(Photographs Taken on 26 August 1999)

Note: See Figure 16 for Location.

Plate 16 – View of Cracking and Distress along the Crest Channel
(Photograph Taken on 26 August 1999)

Note: See Figure 16 for Location.

Sideways movement of about 250 mm is evident in the lower portion of the cascade channel

Recently repaired cascade channel had cracked again during the 1999 landslide

Plate 17 – View of Displaced Cascade Channel (Photographs Taken on 15 March 1999)

Note: See Figure 16 for Location.

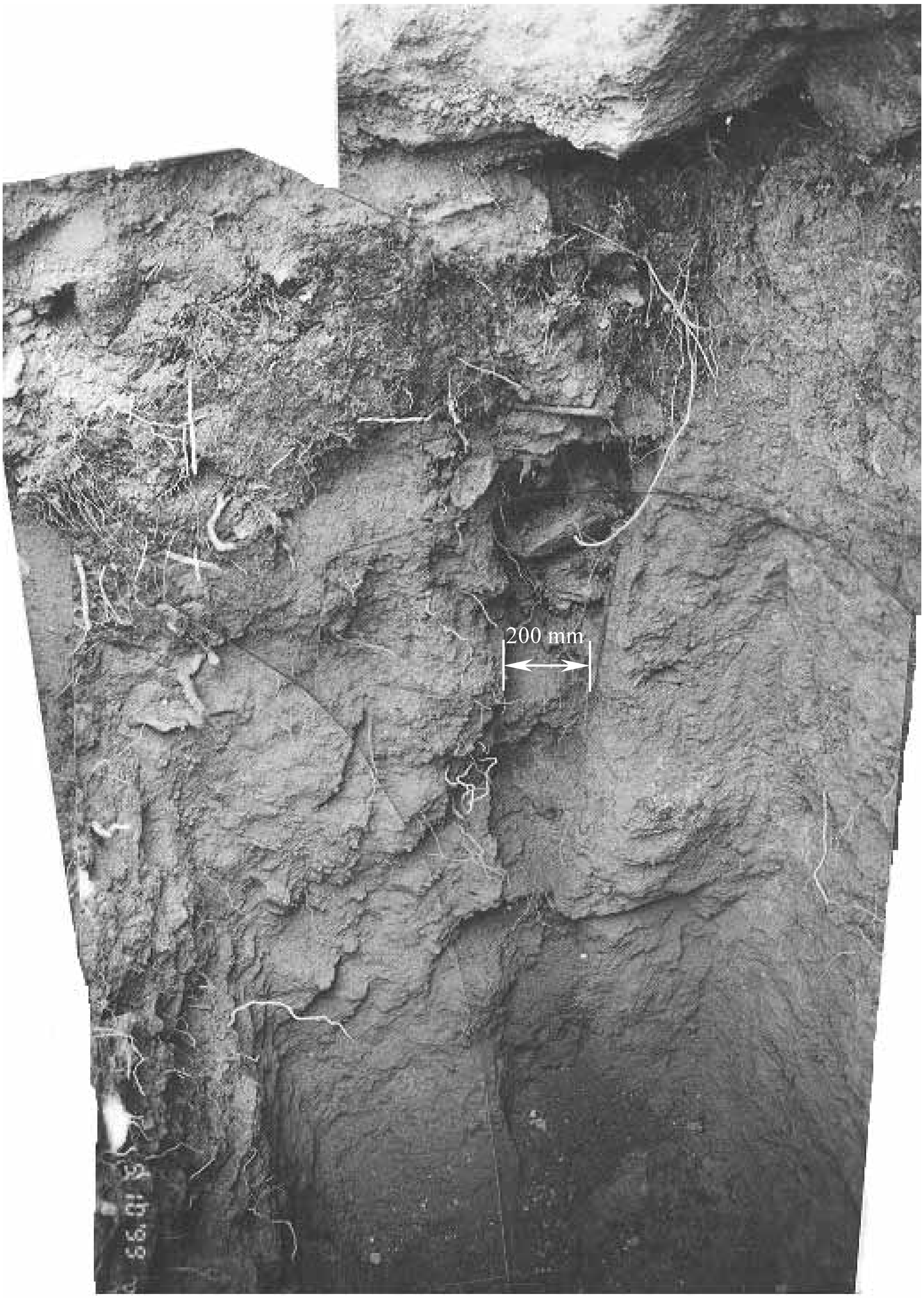


Plate 18 – View of Infilled Tension Crack Identified in Trial Pit TP2
(Photographs Taken on 9 October 1999)

Note: See Figure 16 for Location.

Plate 19 – View of Smaller Tension Crack Identified Following Vegetation Clearance Works
(Photographs Taken on 30 November 1999)

Note: See Figure 16 for Location.

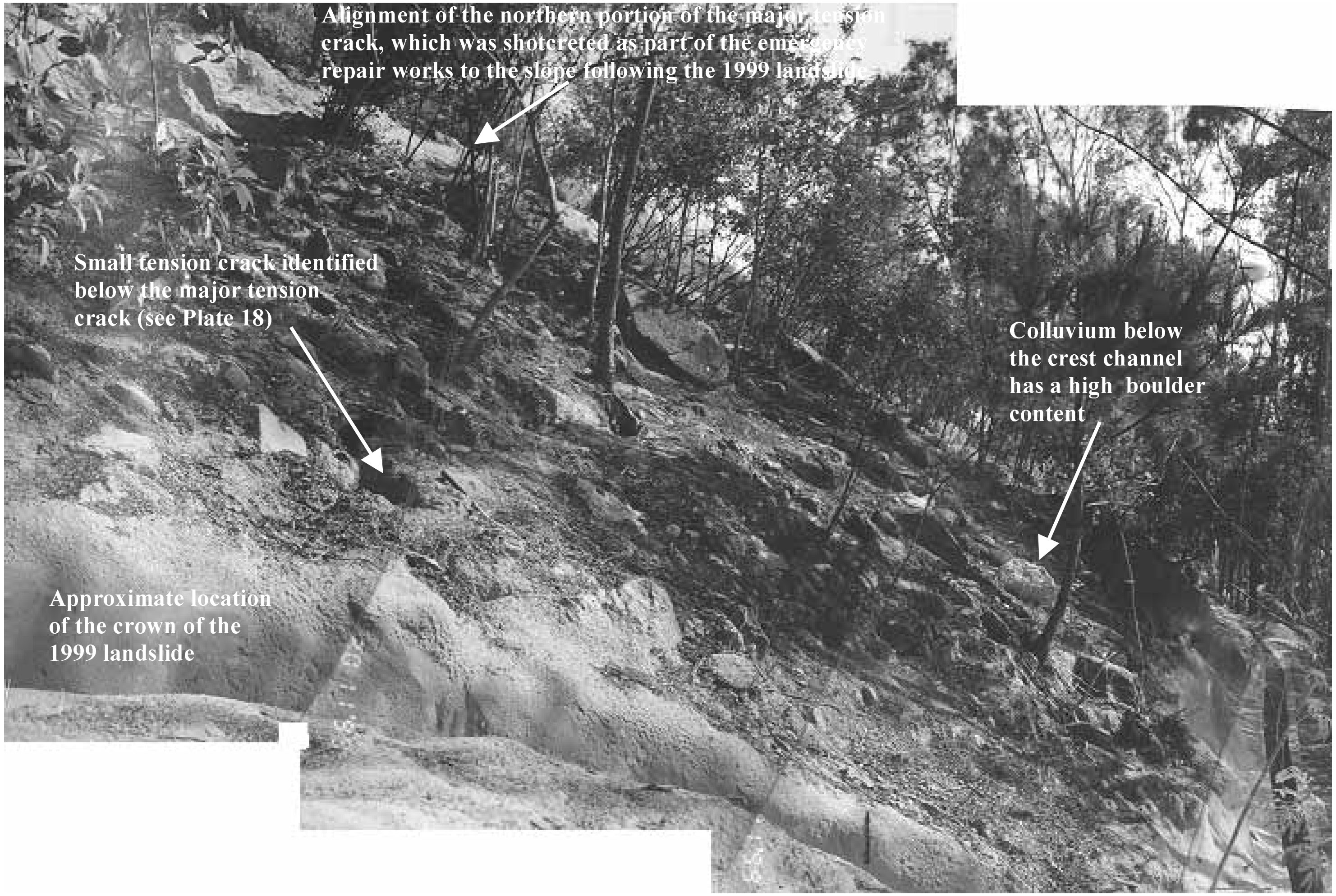


Plate 20 - General View of Slope Above and to the Southeast of the 1999 Landslide Showing Bouldery Nature of Colluvium (Photographs Taken on 30 November 1999)

Note: See Figure 16 for Location.

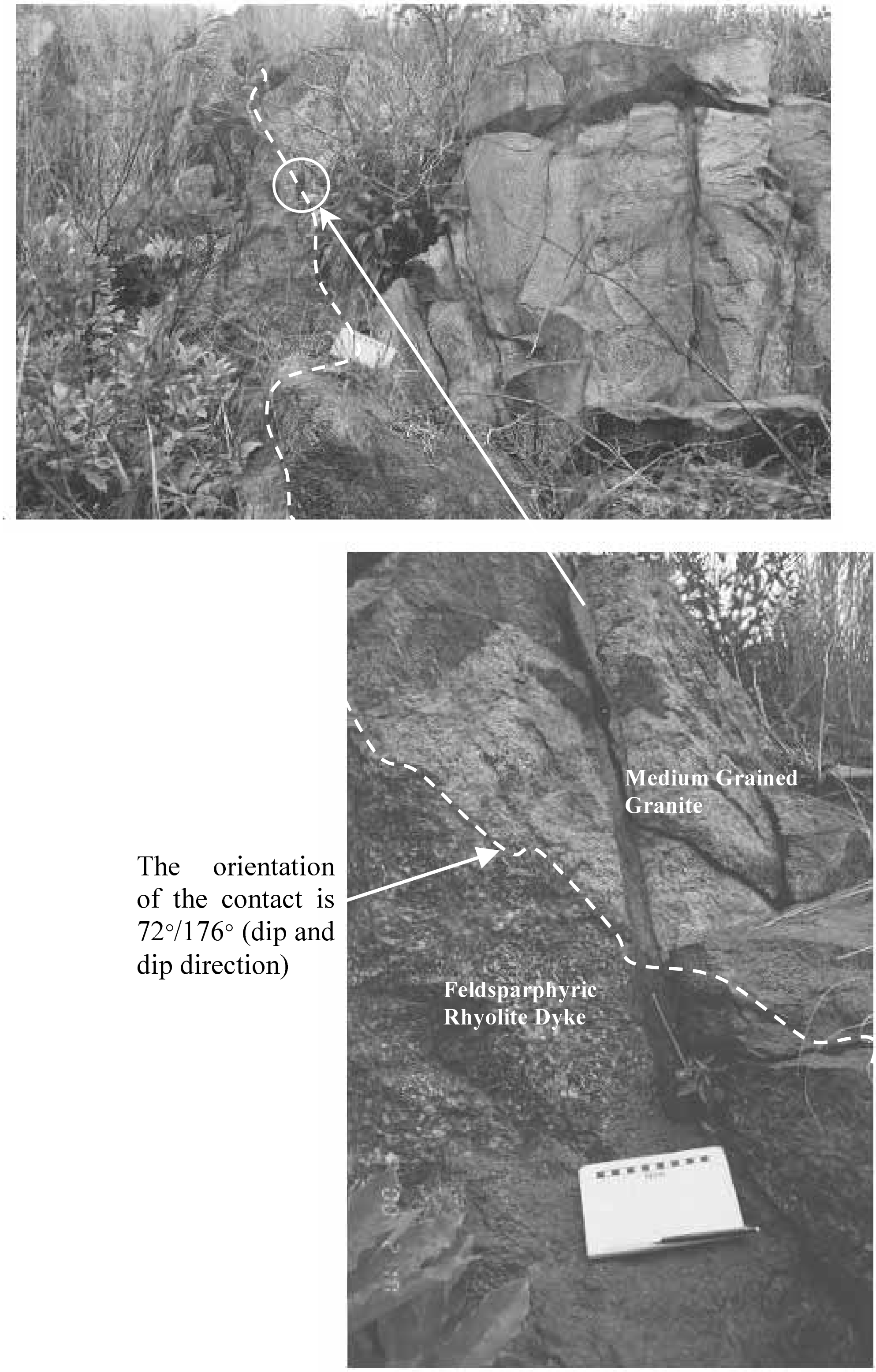


Plate 21 – View of Sharp Contact between Feldsparphyric Rhyolite Dyke and Medium Grained Granite (Photographs Taken on 10 February 2000)

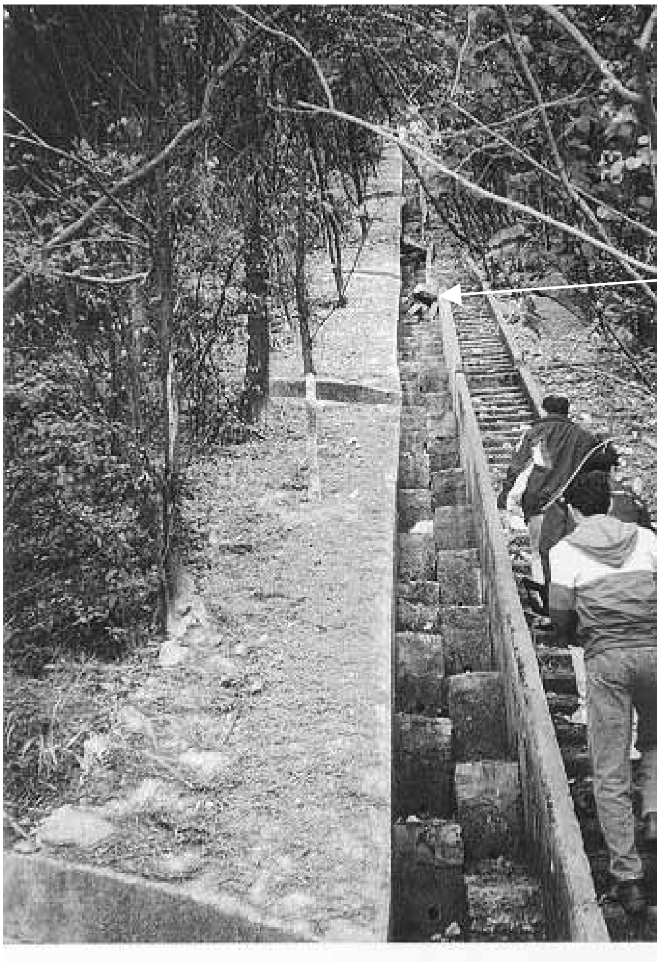
Note: See Figure 16 for Location.

APPENDIX A

PHOTOGRAPHS OF PRE-FAILURE SLOPE DISTRESS TAKEN BY HYD AND GEO BETWEEN JULY 1998 AND FEBRUARY 1999

Plate A1 – View Looking Downslope of Erosion Scar in First Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate A2 – Side View (Looking South) of Erosion Scar in First Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)



Side Wall to cascade channel has collapsed at the same elevation as the drainage channels above the second and third batter

Plate A3 – General View Looking Westwards Up the Cascade Channel Separating Slope Nos. 10NE-B/C262 and 10NE-B/C254 (Area 1)

Plate A4 – Side View (Looking Northwards) of Cracking and Movement of Drainage Channel, Concrete Steps and Cascade Channel at an Elevation of the Drainage Channel above the Third Batter (Area 1)

Plate A5 – View Downslope of Collapsed Side-wall to Cascade Channel at an Elevation of the Drainage Channel above the Third Batter (Area 1)

Plate A6 – View Looking Upslope of the Scar in the Fourth and Fifth Batter at the Southern End of Slope No. 10NE-B/C262 (Area 2)

Plate A7 – Side View (Looking Northwards) of the Scar in the Fourth and Fifth Batter at the Southern End of Slope No. 10NE-B/C262 (Area 2)

Plate A8 – View of Looking Upslope of the Erosion below the Crest Channel at the Northern End of Slope No. 10NE-B/C262 (Area 3)

Plate A9 – View Looking Downslope of the Erosion below the Crest Channel at the Northern End of Slope No. 10NE-B/C262 (Area 3)

Plate A10 – View of Distressed Toe Channel Showing Significant Narrowing of the 300 mm Wide Channel

APPENDIX B

PHOTOGRAPHS OF PRE-FAILURE SLOPE DISTRESS OBSERVED BY FMSW IN AUGUST 1999

Plate B1 – View of Major Crack in Drainage Channel above Second Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate B2 - View of Major Dislocation in the Channel above the Second Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate B3 - View of Major Dislocation in the Channel above the Second Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Tension crack was identified in the third batter above one of the major dislocations in the drainage channel

Plate B4 – View of Major Dislocation in the Channel above the Second Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate B5 – View of Voided Area along the Upslope Side of the Drainage Channel above the Second Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate B6 – View of Tension Crack in the Lower Portion of the Third Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Undermining / Slumping was identified to a depth of about 1 m below the drainage channel above the third batter

Plate B7 – View of Undermined Section of Drainage Channel above the Third Batter at the Northern End of Slope No. 10NE-B/C254 (Area 1)

Plate B8 – View of Voiding along the Upslope Side of the Drainage Channel above the Third Batter at the Northern End of Slope No. 10NE-B/C 254 (Area 1)

APPENDIX C

TRIAL PIT LOGS

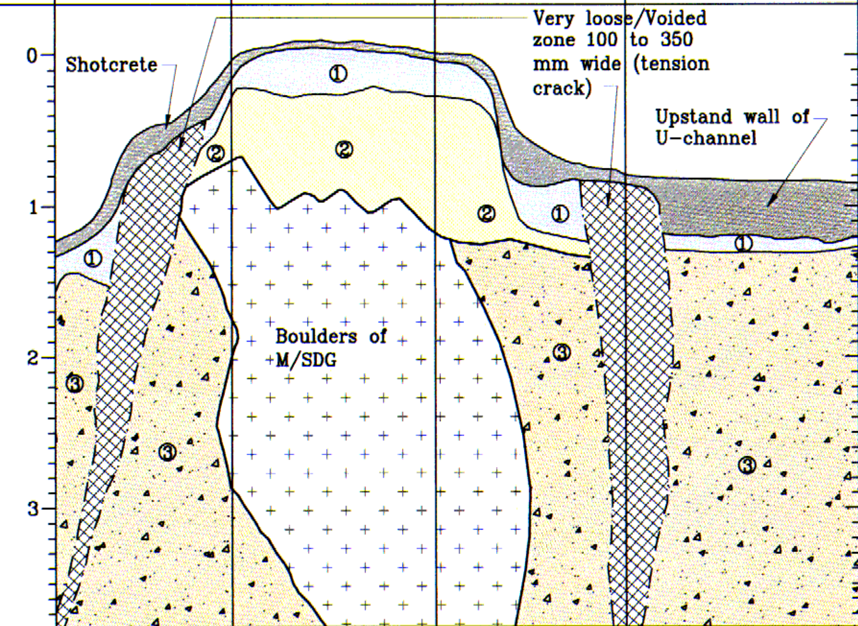
Fugro Maunsell Scott Wilson Joint Venture
Agreement No. CE 101/98
1999 Landslide Investigation Consultancy
Detailed Study No. 6
FMSW No. 113, Tsing Yi Road

TRIAL PIT RECORD

TRIAL PIT No. 1

CO-ORDINATES	827646.80 E	821211.80 N	
GROUND LEVEL	83.66 mPD		
EXCAVATION DATE:	from 28-09-99		
BACKFILL DATE:	09-10-99		
LOGGED BY:	P.H. Chan	DATE:	06-10-99
CHECKED BY:	M. Hughes	DATE:	08-10-99

Samples & tests	Depth (m)	FACE D: 1.40 m	FACE A: 1.60 m	FACE B: 1.50 m	FACE C: 1.90 m	Legend	Grade	Description



Description

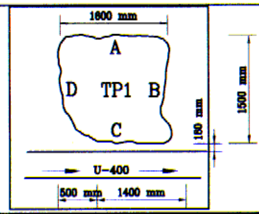
① Desiccated topsoil layer with many rootlets

② Colluvium – Frim, yellowish brown, mottled white and reddish brown slightly gravelly, sandy, clayey SILT with some subrounded cobbles of highly and moderately decomposed granite.

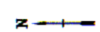
③ Colluvium – Medium dense greyish brown clayey very silty gravelly SAND with some to many subangular to subrounded cobbles and boulders of highly and moderately decomposed granite.
Material in Face D and downslope of tension crack was loose to medium dense and appeared disturbed.

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

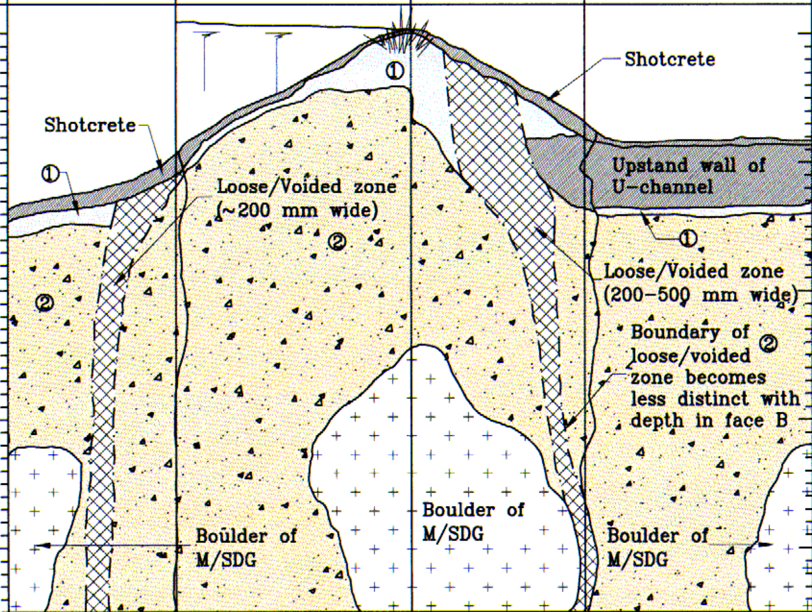


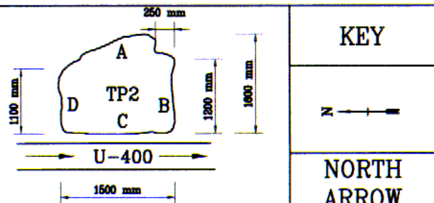
NORTH ARROW

REMARKS

— No seepage was observed in the sides of the trial pit during excavation

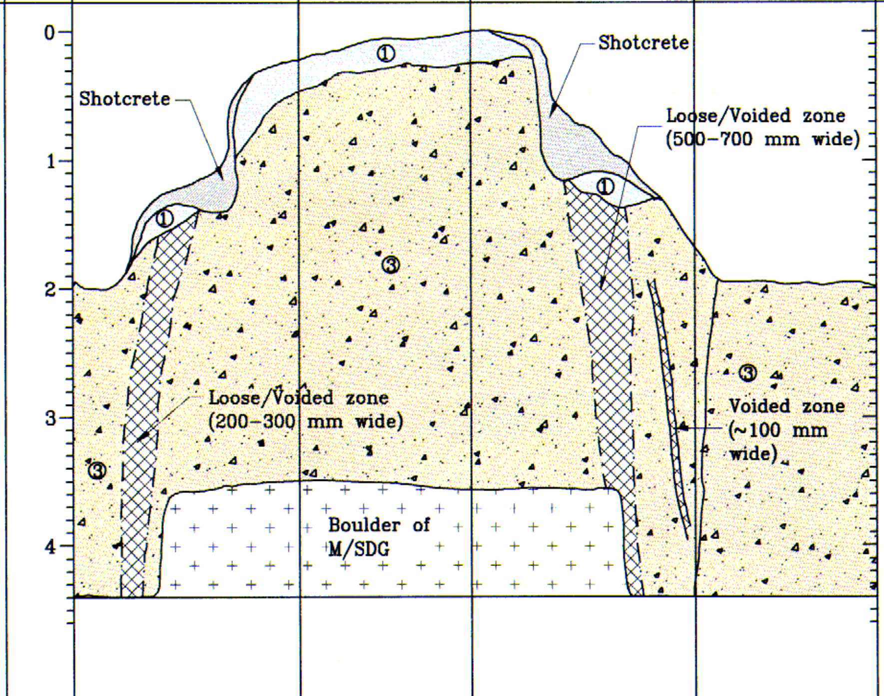
Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 6 FMSW No. 113, Tsing Yi Road	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. 2	EXCAVATION DATE: from 28-09-99 BACKFILL DATE: 15-10-99	
	CO-ORDINATES 827646.10 E 821218.10 N GROUND LEVEL 83.14 mPD	LOGGED BY: P.H. Chan DATE: 11-10-99 CHECKED BY: M. Hughes DATE: 12-10-99	

Samples & tests	Depth (m)	FACE D: 1.10 m	FACE A: 1.50 m	FACE B: 1.20 m	FACE C: 1.50 m	Legend	Grade	Description
	0 1 2 3 4							① Desiccated topsoil layer with many rootlets ② Colluvium – Medium dense greyish brown clayey very silty gravelly SAND with some to many subangular to subrounded cobbles and boulders of highly and moderately decomposed granite. Material in Face D and downslope of tension crack was loose to medium dense and appeared disturbed.

SYMBOL	SAMPLES/TESTS/WATER	PLAN (NOT TO SCALE)	KEY	REMARKS
•	SMALL DISTURBED SAMPLE		KEY	— No seepage was observed in the sides of the trial pit during excavation
↕	BULK DISTURBED SAMPLE			
▬	UNDISTURBED SAMPLE HORI. (U100/U76)			
▯	UNDISTURBED SAMPLE VERT. (U100/U76)			
⊠	BLOCK SAMPLE			
⊔	IN-SITU DENSITY TEST			
△	WATER SAMPLE			
∿	SEEPAGE		NORTH ARROW	

Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 6 FMSW No. 113, Tsing Yi Road	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. 3	EXCAVATION DATE: from 27-09-99 BACKFILL DATE: 09-10-99	
	CO-ORDINATES 827644.00 E 821231.60 N GROUND LEVEL 83.71 mPD	LOGGED BY: P.H. Chan DATE: 06-10-99 CHECKED BY: M. Hughes DATE: 08-10-99	

Samples & tests	Depth (m)	FACE D: 2.10 m	FACE A: 1.60 m	FACE B: 2.10 m	FACE C: 1.70 m	Legend	Grade	Description

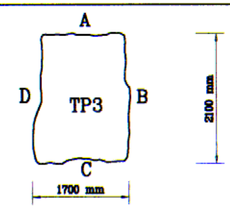


① Desiccated topsoil layer with many rootlets

② Colluvium – Medium dense greyish brown and brown, slightly gravelly to gravelly, sandy to very sandy clayey SILT with some to many subangular to subrounded cobbles and boulders of moderately to slightly decomposed granite and feldsparphyric rhyolite. Small voids/soil pipes observed within the upper 1.5m.
Material in Face D and downslope of tension crack was loose to medium dense and appeared disturbed.

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
☒	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



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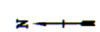


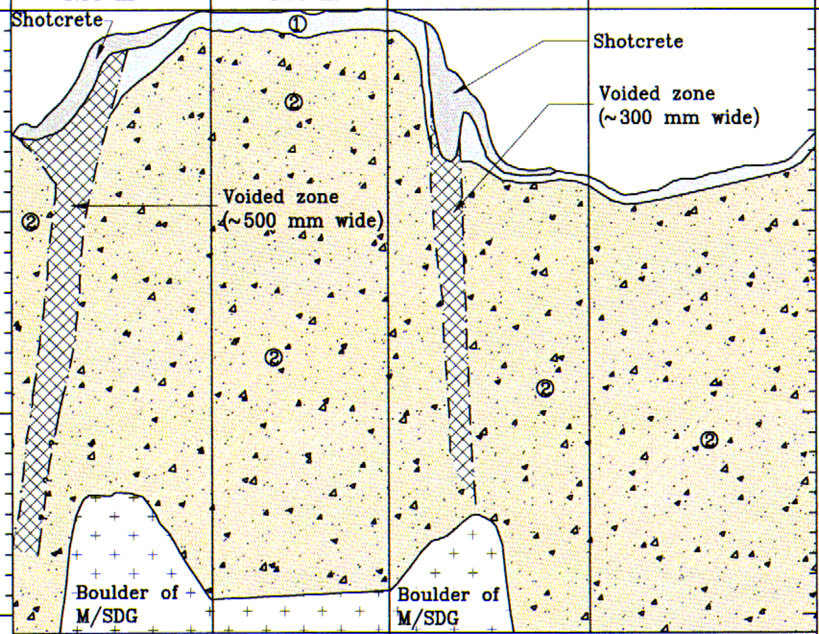
NORTH ARROW

REMARKS

— No seepage was observed in the sides of the trial pit during excavation

Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 6 FMSW No. 113, Tsing Yi Road	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. 4	EXCAVATION DATE: from 27-09-99 BACKFILL DATE: 09-10-99	
	CO-ORDINATES 827641.50 E 821240.00 N GROUND LEVEL 83.12 mPD	LOGGED BY: P.H. Chan DATE: 06-10-99 CHECKED BY: M. Hughes DATE: 08-10-99	

Samples & tests	Depth (m)	FACE D: 1.90 m	FACE A: 1.70 m	FACE B: 1.90 m	FACE C: 2.20 m	Legend	Grade	Description

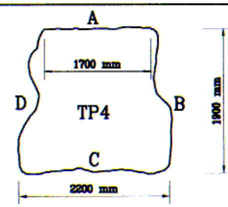


① Desiccated topsoil layer with many rootlets

② Colluvium – Medium dense greyish brown and brown, slightly gravelly to many subangular to subrounded cobbles and boulders of moderately to slightly decomposed granite and feldsparhyric rhyolite. Small voids/soil pipes observed within the upper 1.5m.
Material in Face D and downslope of tension crack was loose to medium dense and appeared disturbed.

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

N

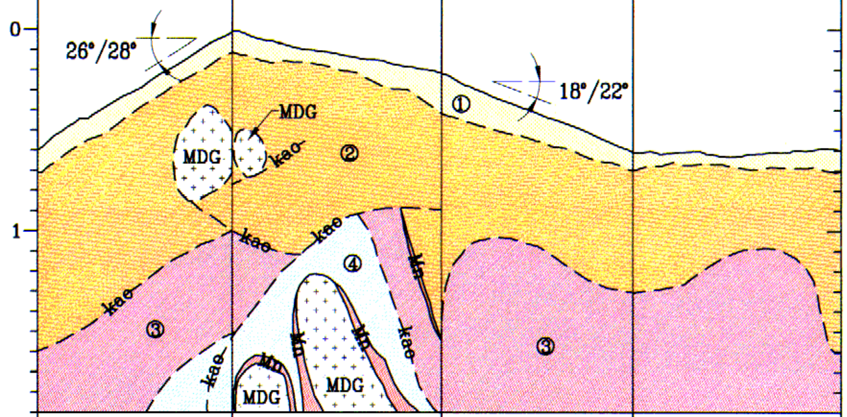
NORTH ARROW

REMARKS

— No seepage was observed in the sides of the trial pit during excavation

Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 6 FMSW No. 113, Tsing Yi Road	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. 5	EXCAVATION DATE: from 10-09-99 BACKFILL DATE: 14-10-99	
	CO-ORDINATES 827622.70 E 821256.40 N GROUND LEVEL 69.00 mPD	LOGGED BY: P.H. Chan DATE: 13-09-99 CHECKED BY: M. Hughes DATE: 08-10-99	

Samples & tests	Depth (m)	FACE D: 1.50 m	FACE A: 1.60 m	FACE B: 1.50 m	FACE C: 1.60 m	Legend	Grade	Description



① Colluvium – Soft to firm, yellowish brown, mottled white and reddish brown, slightly gravelly, sandy, clayey SLIT with some rootlets.

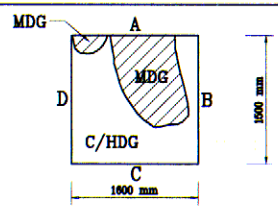
② Completely decomposed granite – Stiff, orangish and reddish borwn, mottled with greyish white, slightly gravelly, clayey and very sandy silt.

③ Completely to highly decomposed granite with non-persistent, planar, occasionally undulating, manganese stained relict joints – Very stiff, reddish and pinkish brown, sandy, fine gravelly, clayey silt.

④ Zone of highly decomposed granite surrounding corestones of moderately decomposed granite (MDG)

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

NORTH ARROW

REMARKS

— No seepage was observed in the sides of the trial pit during excavation

— — kao — — Silty clay joint infill disseminated with fleks of white kaolin and with manganese staining

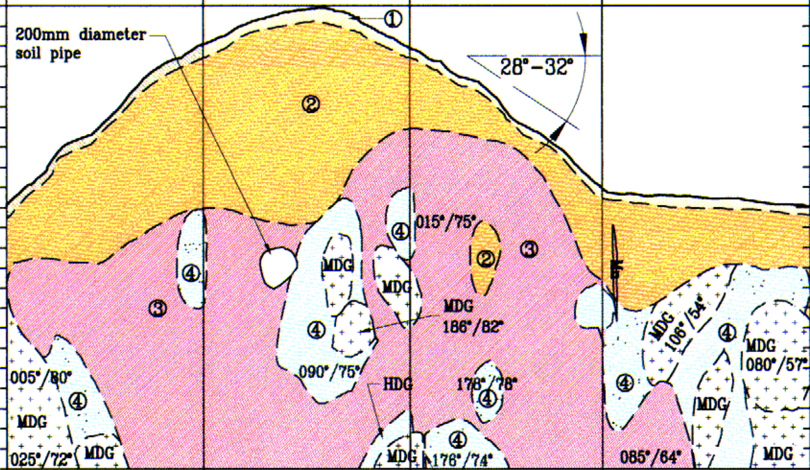
Mn Manganese stained surface

MDG Corestone of moderately decomposed granite

Fugro Maunsell Scott Wilson Joint Venture
Agreement No. CE 101/98
1999 Landslide Investigation Consultancy
Detailed Study No. 6
FMSW No. 113, Tsing Yi Road

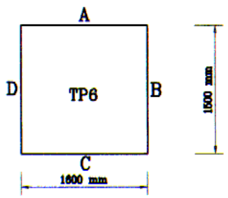
TRIAL PIT RECORD		Sheet 1 of 1
TRIAL PIT No. 6	EXCAVATION DATE: from 10-09-99 BACKFILL DATE: 15-10-99	
CO-ORDINATES 827625.80 E 821254.40 N GROUND LEVEL 70.30 mPD	LOGGED BY: P.H. Chan DATE: 13-09-99 CHECKED BY: M. Hughes DATE: 08-10-99	

Samples & tests	Depth (m)	FACE D: 1.50 m	FACE A: 1.60 m	FACE B: 1.50 m	FACE C: 1.60 m	Legend	Grade	Description
	1 2 3							① Colluvium – Soft to firm, yellowish brown, mottled white and reddish brown, slightly gravelly, sandy, clayey SLIT with some rootlets. ② Completely decomposed granite – Stiff, orangish and reddish borwn, mottled with greyish white, slightly gravelly, clayey and very sandy silt. ③ Completely to highly decomposed granite with non-persistent, planar, occasionally undulating, manganese stained relict joints – Very stiff, reddish and pinkish brown, sandy, fine gravelly, clayey silt. ④ Zone of highly decomposed granite surrounding corestones of moderately decomposed granite (MDG)



SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
△	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

NORTH ARROW

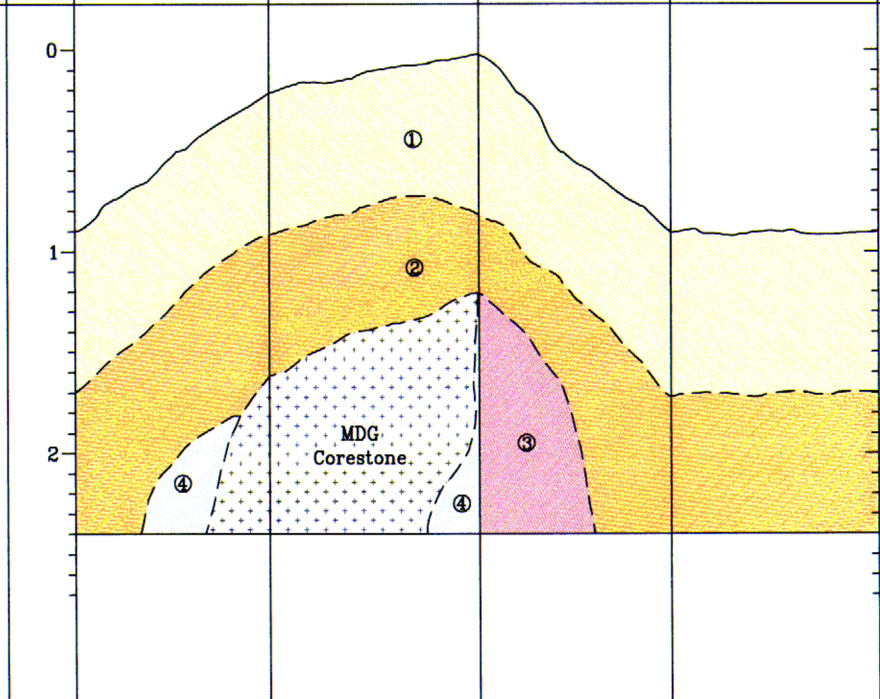
REMARKS

— No seepage was observed in the sides of the trial pit during excavation

MDG Corestone of moderately decomposed granite

Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 6 FMSW No. 113, Tsing Yi Road	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. 7	EXCAVATION DATE: from 09-09-99 BACKFILL DATE: 14-10-99	
	CO-ORDINATES 827622.50 E 821247.80 N GROUND LEVEL 68.10 mPD	LOGGED BY: P.H. Chan DATE: 13-09-99 CHECKED BY: M. Hughes DATE: 08-10-99	

Samples & tests	Depth (m)	FACE D: 1.55 m	FACE A: 1.60 m	FACE B: 1.55 m	FACE C: 1.60 m	Legend	Grade	Description



① Colluvium – Soft to firm, yellowish brown, mottled white and reddish brown, slightly gravelly, sandy, clayey SLIT with some rootlets.

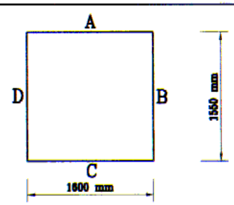
② Completely decomposed granite – Stiff, orangish and reddish borwn, mottled with greyish white, slightly gravelly, clayey and very sandy silt.

③ Completely to highly decomposed granite with non-persistent, planar, occasionally undulating, manganese stained relict joints – Very stiff, reddish and pinkish brown, sandy, fine gravelly, clayey silt.

④ Zone of highly decomposed granite surrounding corestones of moderately decomposed granite (MDG)

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
☒	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

NORTH ARROW

REMARKS

— No seepage was observed in the sides of the trial pit during excavation

MDG. Corestone of moderately decomposed granite