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呈祥道山泥傾瀉事件報告
REPORT ON THE CHING CHEUNG ROAD LANDSLIDE OF 3 AUGUST 1997

土力工程處報告系列第 78 號
GEO REPORT No. 78

合樂亞洲顧問公司
Halcrow Asia Partnership Ltd.

香港特別行政區政府
土木工程署
土力工程處
GEOTECHNICAL ENGINEERING OFFICE
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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
Principal Government Geotechnical Engineer
November 1998

FOREWORD

This GEO Report presents the forensic investigation of the 3 August 1997 landslide at Ching Cheung Road by Halcrow Asia Partnership for the Geotechnical Engineering Office as part of the 1997 Landslide Investigation (LI) Consultancy.

The LI Consultancy aims to achieve the following objectives through review and studies 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 Landslip Investigation Division of the Geotechnical Engineering Office worked closely with the LI Consultants and provided technical input and assistance to the forensic investigation.

Attention is drawn to the fact that the final report on the 1997 Ching Cheung Road landslide investigation issued in February 1998 has been expanded to include the photogrammatic analyses and some minor editorial amendments made in this GEO Report.

M C Tang
Government Geotechnical Engineer/
Landslip Preventive Measures
August 1998

ABSTRACT

On 3 August 1997, a landslide occurred on cut slope No. 11NW-A/C55 adjacent to Ching Cheung Road. The landslide debris completely blocked a 50 m section of Ching Cheung Road, trapping a private car travelling on the westbound carriageway. The driver of the car was uninjured. The landslide was the final stage of a progressive process of failure and collapse that began on 7 July 1997 in cut slope No. 11NW-A/C55. The landslide was selected for detailed investigation partly because of its size, consequence and history but also because the slope had been subjected to upgrading works between 1990 and 1992 following investigation and design in 1989 under the Landslip Preventive Measures Programme.

A comprehensive investigation into the landslide was carried out for the Geotechnical Engineering Office (GEO) during the period August 1997 to February 1998 by GEO's landslide investigation consultants, Halcrow Asia Partnership Ltd. This detailed study included review of documentary information, analysis of rainfall records, interviews with witnesses to the landslide, site survey, ground investigation, theoretical stability analyses and diagnosis of the causes of failure.

The investigation concluded that the landslide of 3 August 1997 was probably the result of the progressive deterioration of the slope following an initial failure on 7 July 1997. The early movements caused disruption of surface drainage channels and ground deformation thereby allowing surface water infiltration during later intense rainstorms which contributed to the subsequent collapses.

The investigation has identified a long history of instability on the slope following initial large-scale failures caused by quarrying below the site from before 1924 up to 1954. The failed mass has remained largely in place throughout the subsequent history of the slope and indeed is still there.

The previous failures at the site may have contributed to the 1997 landslide, which is shown from photogrammetric analysis to be largely coincident in plan area with failures seen in 1954 and 1963 aerial photographs. It is concluded, however, that the main cause of the 1997 failure was the adversity of groundwater conditions which developed following extremely severe rainfall.

A high, transient perched water table probably developed prior to the failure, particularly beneath a natural drainage line at the head of the failed slope. The natural subsurface groundwater pathway seems to be largely controlled by a system of natural pipes, associated partly with decomposed basalt dykes within the local granite, but the pipes may also be exploiting zones of previous disturbance. A system of raking drains, which was installed to reduce groundwater pressures following a failure at the same location in 1972, was unable to prevent the development of critical water pressures, which caused the 1997 landslide.

Details of the investigation and its findings are given in this report on the landslide.

CONTENTS

1. INTRODUCTION

At 11:15 a.m. on Sunday 3 August 1997, a landslide occurred on a roadside cut slope above Ching Cheung Road, east of Butterfly Valley, Kowloon (Figure 1). The landslide debris completely blocked a 50 m section of Ching Cheung Road (Plate 1), trapping a private car travelling on the westbound carriageway. The driver of the car freed himself and was uninjured. The road was subsequently closed to enable clearance of debris and the implementation of urgent repair works to the slope.

Following the landslide, the Geotechnical Engineering Office (GEO) of the Civil Engineering Department instigated a comprehensive investigation into the failure. The investigation was carried out by GEO's 1997 landslide investigation consultants, Halcrow Asia Partnership Ltd (HAP).

The investigation was carried out during the period August 1997 to February 1998, and comprised the following tasks :

(a) review of all known documents and aerial and terrestrial photographs relating to the development history of the site,

(b) detailed observation and measurement at the landslide site,

(c) interviews with witnesses to the landslide,

(d) establishment of the sequence and details of landsliding events,

(e) analysis of rainfall records at the time of failure and comparison with the historical records,

(f) geological mapping,

(g) detailed ground investigation to determine the subsurface conditions at the site by drilling, trial pitting, in situ testing, sampling and logging,

(h) establishment of slope hydrogeological and geological models,

(i) review of engineering properties of materials at the site,

(j) review and back-analysis of previous instability on the failed slope and within the vicinity,

(k) analysis of theoretical stability and groundwater conditions of the failed slope under various conditions, and

(l) conclusions on the probable causes of the failure.

This report presents the findings of the investigation. Full details of the investigation work undertaken and the results obtained are contained in a set of documents, which is placed in the Civil Engineering Library on the First Lower Ground Floor of the Civil Engineering Building.

2. DESCRIPTION OF THE SITE

The landslide occurred on the central portion of registered cut slope No. 11NW-A/C55 located above Ching Cheung Road, to the east of Butterfly Valley, Cheung Sha Wan, Kowloon (Figure 1).

The cut slope was originally formed as part of the construction works of Ching Cheung Road, which was completed by 1967. The slope was cut into predominantly highly weathered granite with localised stronger rock on the lower western batter slope. The central portion of the slope is located beneath a natural drainage line (Figure 2), where there is a localised concavity in the slope's plan profile formed during regrading works following a failure in 1972. In the east and west, the slope is cut into topographical spurs.

The recent failures were located within the cut slope beneath the drainage line and at the failed section involved the full cut slope height of about 35 m with a maximum width of about 52 m.

Prior to the recent failures, the slope geometry comprised two to six batter slopes with a total vertical height ranging from 12 m to 47 m. The slope inclination to the horizontal varied, but was typically 45° to 50°. The slope covering included vegetation with localised shotcreting. A photograph taken of the slope on 15 September 1993 shows the condition of the central portion of the slope (Plate 2).

The natural hillside above the cut slope is densely vegetated with trees and shrubs, rising at an average inclination of about 26° from an elevation of about 80 mPD at the crest of the cut slope, to 150 mPD at the top of the hill. In the east, the lower batter of the cut slope adjoins a larger registered cut slope No. 11NW-A/C54 with an incised densely vegetated natural drainage line separating the two slopes (Figure 1). To the west and also adjoining, is registered cut slope No. 11NW-A/C56.

The surface drainage system of cut slope No. 11NW-A/C55 comprised drainage channels located along berms connected by downslope stepped channels feeding into the roadside drainage system. The natural drainage line truncated above the central portion of the cut slope was intercepted by a headwall and catchpit and connected into the slope drainage. A subsurface raking drain system was installed in the lower two batters of the cut slope in 1972, following a slope failure at the site as discussed in Section 4.

Ching Cheung Road lies along the toe of the slope at an elevation of about 33 mPD. At the time of the recent landslide, the section of road fronting the slope was closed as part of the construction works for the 'Lung Cheung Road and Ching Cheung Road Improvements Project' for the Highways Department (HyD). At this location, all traffic was diverted onto the recently completed elevated 4-lane carriageway, which runs parallel and along the southern edge of the closed road (Figure 2).

3. DESCRIPTION OF THE LANDSLIDE

The recent landslides at Ching Cheung Road involved a sequence of three successively larger progressive failures from cut slope No. 11NW-A/C55. The first occurred on 7 July 1997 followed by a further failure on 17 July 1997 and culminated in the major failure on 3 August 1997. The description of the recent landslides has been compiled from eye-witness accounts and documentary records.

The first failure occurred at about 9:00 a.m. on 7 July 1997 and caused the collapse of an estimated volume of 500 m^3 of soil from the lower part of the central eastern portion of the cut slope (Figure 2 and Plate 3). The debris just encroached onto the closed section of Ching Cheung Road. The observed failure scar was confined to the lower two batters, and affected a 52 m length of slope that extended up to 15 m above Ching Cheung Road. The upper portion of the cut slope was heavily vegetated, which made inspection of the slope above the observed failure difficult. The exposed failure surface was apparently shallow but variable in depth, typically less than 2 m. The majority of the displaced material originated from the western portion of the failure within the concave section of the slope, with generally only minor surficial failure in the east. The eastern limit of the failure was defined by an erosion gulley which developed beneath the location where several surface channels met at a catchpit.

The second failure occurred at about 3:00 p.m. on 17 July 1997 and involved an estimated volume of 700 m^3 of soil from the concave section of slope immediately above the western part of the earlier failure (Figure 2 and Plate 4). The failure debris run-out was again confined to the closed section of Ching Cheung Road. The failure reached up to 30 m above the road and affected about a 30 m length of slope, comprising two distinct failure scars. The larger scar formed a bowl-shaped depression, some 15 m wide, lying beneath the line of the truncated natural drainage line. The maximum vertical depth of the depression was estimated at 5 m. A similar shaped but smaller scar lay to the west. Located at the head of the larger scar was a severed outflow channel from a catchpit where several surface channels converged, including the channel connected to the truncated natural drainage line. Notable gully erosion in the floor of the scar indicated water flow. A severed surface channel was also present at the head of the smaller scar.

On 29 July 1997, upon clearance of vegetation from the slope, an area of extensive cracking and displacement was identified above the earlier failures (Figure 2 and Plate 5). The area of distress encompassed the earlier failures, with its lateral extent on the lower batter slopes coincident with the extent of the 7 July failure. The distressed ground involved a plan area of about 1800 m^2 that extended slightly above the full height of the cut slope below the natural drainage line and up to 52 m wide.

A major perimeter tension crack encompassed the area of distressed ground (Figure 2). Measurement of ground displacement at points along the tension crack on 30 July 1997 indicated a maximum vertical downward movement of up to 2.5 m and a horizontal downslope movement of up to 1 m at the upslope margin of the tension crack (Plate 5). Along the western and northern boundaries the tension crack appeared fresh. However, observations recorded on 30 July 1997 possibly indicated an earlier phase of movement. Locally, a loose infill of topsoil was observed within a splay of the tension crack, and along the eastern section of the tension crack the surface was stained, indicating that it may have been exposed for some time. Furthermore, within the upper western section a small area of shotcrete surfacing, from previous remedial works, covered a displaced berm section, above which was a small subdued scarp feature, again suggesting previous movement. As discussed later in this report, other indications of previous movement were found during the ground investigation following the landslide.

The third failure resulted in the complete blockage of Ching Cheung Road. Based on an eye-witness account, the landslide occurred at 11:15 a.m. on 3 August 1997. A photograph taken during the clear-up operations, a few hours after the failure, is shown in Plate 6. The estimated failed volume of mobilised soil material was 2000 m^3.

The third failure originated from the upper western part of the distressed area, its extent being limited by the previously identified perimeter tension crack. A depression was formed up to 7 m deep and 20 m wide extending 30 m above Ching Cheung Road and the earlier landslide scars were deepened and eroded. A typical cross-section through the landslide scar is shown in Figure 3. As shown in Plate 7 the landslide debris covered the closed section of Ching Cheung Road and the adjacent newly completed elevated 4-lane carriageway reaching the southern parapet fence. The maximum width of the debris on the road was estimated as less than 50 m with a maximum thickness of about 3 m (Plate 6). The maximum horizontal travel distance of the landslide debris, as measured from the crest of the failure scar, was 78 m.

A private car travelling along the west-bound elevated carriageway of Ching Cheung Road was caught in the debris and pushed onto the southern parapet fence (Figure 2). The driver was uninjured and released himself from the vehicle. Ching Cheung Road remained closed for the clear-up operations and during emergency repair works to the failed slope. During the works further movement within the head of the failure occurred with some minor slippage of material. The road was fully re-opened on 24 August 1997.

The landslide debris comprised predominantly brown to light brown and reddish brown gravely silty clayey sand with occasional cobbles and rare boulders of moderately decomposed granite, together with some man-made materials. The latter included construction material, such as concrete blocks, pipes and scaffolding stockpiled in the closed section of Ching Cheung Road.

The mobility of the landslide was assessed by measuring the inclination of the line joining the distal end of the debris with the crest of the landslide scar (Figure 3). The smaller the angle the greater the debris mobility. According to Wong & Ho (1996) this angle is generally greater than 30° for typical rain-induced landslides on cut slopes. The angle measured on site for the 3 August 1997 failure at Ching Cheung Road was 20°, indicating greater debris mobility than commonly observed. The greater mobility might be attributable in part to the ponding of water on the enclosed section of Ching Cheung Road.

4. HISTORY OF THE SITE

The site history, summarised in Appendix A, was determined from interpretation of available aerial photographs of the site, together with a review of other available documentary information.

The earliest available aerial photographs, which were taken in 1924, show the site of the 1997 failure being used as a borrow area. The floor of the borrow area was located at approximately 10 mPD, some 23 m below the current level of Ching Cheung Road. The next available photographs, taken in 1945, show an enlarged borrow area still being worked, with a clearly defined failure scarp above the workings, with slipped material beneath the scarp and within the head of the borrow area. Photographs taken in 1949 show the scarp apparently unchanged with no significant erosion or further failure evident. By 1954, the slope had failed again behind the previous scarp. Undermining of the natural slope by continued borrowing activity possibly caused the enlargement of the failure. As discussed later, the failure scarp identified in the 1954 photography corresponds closely, in part, with the perimeter tension crack of the 1997 landslide. No further extension of the landslide is evident in the 1963 photographs. By that time, buildings had been constructed at the foot of the slope and surface drains constructed across the failed area. A cut slope was formed at the location of the subsequent 1997 failure between 1963 and 1967, when the Public Works Department constructed Ching Cheung Road. It is apparent that the road crossed the old landslide and that the cut slope was formed partly within the retrogressed landslide mass which was observable on the photographs from 1945 to 1963.

A major landslide occurred in the cut slope in 1972 during the widening works of Ching Cheung Road (Plate 8). The failure volume was reported to be about 7,500 m^3 (Maunsell, 1973a). Remedial works involved cutting back the slope at the same gradient as in the original cutting, but during this process, further movements occurred (Plate 9). Subsequent monitoring recorded horizontal displacement of up to 21 mm, concentrated within the lowest batter at a location corresponding to the central section of the 1997 failure. The slope was eventually stabilised by the installation of 151 raking drains up to 16 m long in the lower two batters. It was reported by Maunsell (1973b) that 70% to 80% of the drains intercepted groundwater, in that they drained water continuously following installation, particularly those at the centre of the failure area. The maximum recorded rate of outflow for any one drain-hole was about 38 litres per hour. The slope apparently remained stable for the following ten years.

In August 1982, following a severe rainstorm, a landslide involving more than 1,500 m^3 of debris occurred in a nearby slope No. 11NW-A/C56. There was no major failure in slope No. 11NW-A/C55 as a result of that storm, although a very shallow failure occurred on the upper central cut face, above the top berm of the slope, west of the natural drainage line (Plate 10). Photographs taken following the 1982 rainstorm show water issuing from most of the raking drains installed in 1972 in the lower two batters, in the central portion of the slope. A comparison between photographs taken in August 1982 with those taken a month later shows a small reduction in water flow from the drains, which confirms the transient response of the site to heavy rainfall.

Recommendations made by the Geotechnical Control Office (GCO) (renamed GEO in 1991), for remedial works following the minor 1982 failure in slope No. 11 NW-A/C55, included trimming back the entire slope by 3.8 m and then checking the horizontal drain system installed in 1972 and, if necessary, clearing and flushing or installing new drains. Aerial photographs indicate that, two additional batters were cut at the top of the slope but that the lower two batters were not regraded. No record has been found to confirm whether the raking drains were upgraded, nor whether they were subsequently maintained.

The slope was selected by the Landslip Preventive Measures Committee (LPMC) in August 1987 for inclusion in the 1988/89 Landslip Preventive Measures (LPM) Programme together with adjoining slope No. 11NW-A/C56. Following an engineering geology study by GCO (1989a), which included ground investigation boreholes, trial pitting and surface stripping, a stability assessment and the design of upgrading works were carried out by GCO (1989b). Analysis indicated that sections of slope, to the east and west of the recent failure in slope No. 11NW-A/C55, had inadequate factors of safety. No analysis of stability of the section of slope involved in the 1997 failure is given in the design report (GCO, 1989b).

The upgrading works to slope No. 11NW-A/C55 included cutting back the western portion of slope, removal of colluvial deposits at the lower part of the eastern portion and the formation of new batters by minor trimming works at the top of the slope. These works were carried out between 1990 and 1992 under the LPM Programme. The central portion of the slope, including the eastern and central parts of the 1997 landslide area, was not regraded. Existing hard surface protection and raking drain channels on this portion of the slope were stripped off for inspection of the surface material and it was specified that the entire slope should be hydro-seeded. Photographs taken during the LPM works show that the lower part of the lowest batter was shotcreted in response to surface erosion observed during the works. A full-height section of the lowest batter, close to the centre of the recent failures, was also surfaced with shotcrete (Plate 2) rather than hydroseeded as originally specified.

The LPM works were certified complete on 5 March 1992, although site records indicate that various site works on the portion of slope that failed in 1997 continued until 15 May 1992. Seepages were observed on 19 May 1992 at two locations within the section of slope which failed in 1997 (Plate 11). The need to install raking drains was considered, but analysis of the slope, assuming discrete, thin, perched water tables associated with each seepage point, indicated a factor of safety greater than 1.2. It was decided that no improvement works were required.

Following completion of the LPM works, a minor failure occurred in 1993 on the third batter of the slope within the area which failed in 1997. This failure was inspected by the GEO who recommended shotcreting of the scar. No major modification of the slope has been made since 1993. Inspections of the slope in September 1993 by GEO and in December 1995 by consultants engaged by HyD reported "partly blocked" and "moderate cracking" of surface channels. It was also reported that "weepholes" were blocked with vegetation. Some minor routine maintenance works including "clear drainage channels", "repair cracked/damaged channels" and "unblock weepholes" were recommended. "Engineer Inspections carried out every 3 year(s)" were also recommended by the consultants. The maintenance works were completed by HyD in January 1997. Based on the 1995 inspection findings, HyD recommended the slope to LPMC in May 1996, along with 1009 other slopes, for inclusion in the LPM Programme. The slope was not selected because works had been carried out previously under the LPM Programme.

Construction works to form an access road, on the north side of the hill above slope No. 11NW-A/C55, for the construction of the Butterfly Valley Primary Service Reservoir (Figure 1), commenced in late June 1997. Cutting and stripping of vegetation, also on the north side of the hill, commenced in early July.

The recent development from 1988 to 1997 of the cut slopes along Ching Cheung Road is illustrated in Figures 4 to 6.

5. ANALYSIS OF RAINFALL RECORDS

Rainfall data were obtained from GEO automatic raingauge Nos. K06 and N04, the nearest raingauges to the site. Raingauge Nos. K06 and N04 are located at Carnation House, So Uk Estate approximately 1.2 km to the east and at Kai Kwong Lau, Cho Yiu Estate approximately 1.7 km to the northwest of the landslide respectively. The raingauges record and transmit rainfall data at 5-minute intervals via a telephone line to the Hong Kong Observatory and the GEO.

Both raingauges show similar rainfall patterns and intensity prior to the landslides. Rainfall data from raingauge No. K06 have been used for the analysis as slightly more rainfall was recorded in that raingauge.

The hourly rainfall data recorded at raingauge No. K06 between 1 July and 5 August 1997 and the reported times of each landslide event are presented in Figure 7. Heavy rainfall was recorded for the first four days of July with further periods of heavy rain from 15 to 19 July and from 1 to 3 August, the latter being associated with the passage of Typhoon Victor. Periods of rain, some heavy, preceded each landslide event.

Between 24:00 hours on 30 June and 09:00 hours on 7 July, when the first landslide is reported to have occurred, 621 mm of rain was recorded. A significant proportion of this rain (582 mm) fell between 1 July and 12:00 hours on 4 July.

The daily (calendar) rainfall recorded on 2 July (420.5 mm) was the highest recorded since the establishment of raingauge No. K06 in March 1983.

For several days immediately preceding the first failure, however, rainfall was relatively slight. In this respect the first failure can be considered as a 'delayed' failure relative to a major rainstorm event. Previous major failures on adjacent slopes on Ching Cheung Road, in 1972 and 1982 were also noted to have been 'delayed' by several days following major rainstorms (Appendix A).

The second and third landslide events were also preceded by heavy rainfall. Between 15 July and the time of the second event at about 15:00 hours on 17 July, 219 mm of rainfall was recorded. A further 278.5 mm fell between 1 August and the time of the third event at about 11:15 hours on 3 August. These two events were not 'delayed', but occurred shortly after or during heavy rainfall (Figure 7).

A comparison between patterns of previous heavy rainstorms affecting the landslide site and rainstorms prior to the 1997 events is shown in Figure 8. The highest 31-day rainfall recorded prior to the 7 July failure at raingauge No. K06 was for the 31 days preceding 4 July 1997 (1557.5 mm), and exceeded the highest recorded monthly rainfall at the Hong Kong Observatory since records began in 1884.

The rainfall for the month preceding the major, 3 August 1997 event was not particularly severe compared to historical rainfall at the site (Figure 8).

The maximum rainfall intensities for different durations and their corresponding return periods assessed from historical rainfall records are shown in Table 1. It can be seen that the 31-day rainfall prior to 4 July was the most severe, with an estimated return period of about 500 years based on historical data from the Hong Kong Observatory. The return period of such an event for Hong Kong as a whole cannot be estimated from available data but would be somewhat less. Nevertheless it can be concluded that the rainfall which preceded the 7 July 1997 landslide was unusually severe.

6. SUBSURFACE CONDITIONS AT THE SITE

6.1 General

The subsurface conditions at the site were determined using information from desk and field studies. The desk study included a review of various relevant existing reports, whilst the field study included geological mapping and ground investigation designed specifically for this study.

Geological mapping of the slope was carried out by the Hong Kong Geological Survey during several site inspections from 31 July 1997 to 13 August 1997 (GEO, 1997).

Ground investigation for this study commenced in September 1997 and comprised 12 vertical boreholes, 4 inclined boreholes, 12 trial pits and 3 chunam strips (Figure 9). Boreholes were sunk using foam flush with Mazier sampling wherever possible in order to gain maximum recovery for description.

The locations of ground investigation works undertaken prior to the landslide of 1997 are shown in Figure 10.

6.2 Geology

A geological plan of the landslide area is shown in Figure 11 and geological cross-sections through the landslide site are presented in Figures 12 and 13.

The dominant material in the failed slope is extremely to very weak, reddish brown mottled yellowish brown spotted with white and black, highly and completely decomposed, medium- to fine-grained granite, with medium to closely spaced joints. The granite has been locally hydrothermally altered (manifested by advanced chemical weathering), associated sometimes with the intrusion of basalt dykes up to 1.2 m in thickness. Basalt dykes were encountered in several boreholes, occasionally weathered to clayey silt. The basalt dykes appear to be dipping sub-parallel to the natural ground surface, i.e. obliquely out of the slope, south to southwest at 5° to 65°, with an average of 35°. The orientation and encountered depths of some dykes suggest that they may be persistent over tens of metres. The permeability of the dykes would be notably lower than the surrounding granite and therefore the dykes probably locally act as aquitards, inhibiting the downward movement of groundwater.

Materials interpreted as infill to natural erosion pipes, up to 250 mm in height, were identified in boreholes, both above the dykes, and elsewhere throughout the weathered rock profile (Figures 12 and 13). Pipes were typically encountered at about 6 m intervals but occurred at closer spacings to the northeast of the landslide scar. The infill typically comprised quartz-rich fine silts and sands at the top of the pipes, grading to fine gravels at the base. Occasionally, the infilling material comprised well-graded sands and gravels with layers of silts and clays. These well-sorted sediments were probably deposited within a complex, very extensive and changing underground stream system, composed of basins or "sinks" connected by arrays of pipes.

Zones of material were encountered in many boreholes, which indicate possible disturbance (Figures 12 and 13). The material in these disturbed zones typically consisted of wet, pale to medium brown, soft gravelly (fine) sandy silty clay (not sorted) and occasionally containing thin (1-5 mm) roots. The material was observed to have no structure and exhibited a uniform colour throughout. Some of these zones may have been disturbed by the recent landslide but others occur at depths below and outside the probable limits of the failure.

Up to 2.5 m thickness of colluvium was mapped in the failure scarp, along the drainage line in the upper centre of the slope and was found to thin to both east and west (GEO, 1997). The material comprises brown to dark brown gravelly, silty and clayey sand with large boulders up to 4 m in diameter of slightly to moderately decomposed, coarse-grained granite. Mapping of the face carried out in 1988 (GCO, 1989a) indicated a discordant tongue of colluvium extending down to the second batter of the cut slope at a location coincident with the recent failure and this can be seen in photographs taken during LPM works (Plate 11). The field relationships indicate a steep, narrow valley, infilled with bouldery soil. This feature may have been an incised gully and associated colluvium lobe in its natural state but the only available photographs are post-1945, by which time the geomorphological setting had been disturbed by quarrying activities. It is likely that this feature would have acted as a stream line (perhaps with associated sub-surface flow).

A representative outline stratigraphical cross-section through the landslide site is presented as Figure 14.

6.3 Structural Geology

Persistent joints, generally dipping steeply into the slope at about 50° to 70° in a northeast direction (approximately 050° to 080°) were measured in the failure scarp. The measured joint orientations are similar to these measured by surface mapping during the investigation for LPM design (GCO, 1989a) (Figure 15). These joints are commonly coated with a dark brown limonite or manganiferous deposit and locally infilled with white to pale buff coloured kaolin, generally less than 3 mm thick (GEO, 1997). The orientations measured largely confirm measurements made during LPM studies (GCO, 1989a), as illustrated in Figure 15a.

During the recent ground investigation, impression packer tests were performed in boreholes to identify the orientation of discontinuities throughout the zone of less weathered rock. In addition, an acoustic borehole televiewer was used in boreholes DH1 and DH6 to identify the orientations of the rock discontinuities for comparison with the impression packer test results. The televiewer results show joints predominantly dipping at about 40° to the west northwest and south southwest (Figure 15b). This joint set orientation which indicates the potential for instability by sliding, is quite different to those measured during field mapping. The differences in measured joint orientations probably result from measurement bias (Terzhagi, 1967). It is concluded that all measured joint sets exist in the weathered rock mass, i.e. both near-vertical (release) joints and sets of adverse joints which dip out of the slope.

Several, generally fine-grained granitic dykes, varying from 20 mm up to several metres thick, were identified from field mapping (GEO, 1997) (Figure 11). The dominant orientation of dyke contacts was reported to be oblique into the hill, dipping at about 30° to 45° in a northwest direction (approximately 320° to 345°). However, in the lowest batter, on the east side of the slope, a major dyke contact associated with sub-parallel fractures and joints, some infilled with kaolin clay, was mapped dipping in a southeast direction (about 150° to 170°), obliquely out of the slope. As noted in Section 6.2, basalt dykes were encountered in several boreholes and appear also to be dipping out of the slope face, but none were observed in the exposed face following the failure.

6.4 Soil and Rock Properties

A comprehensive series of geotechnical laboratory tests was conducted on soil and rock samples retrieved from nearby ground investigations, carried out following the 1982 landslide on slope No. 11NW-A/C56 and under the 1988/89 LPM programme on slope Nos. 11NW-A/C55 and C56. The tests included particle size distribution, density determination, Atterberg limits, direct shear and multi-staged triaxial compression. Samples tested included materials described as "residual/transition", "completely decomposed granite", "highly decomposed granite", "completely decomposed aplite" and "completely decomposed dolerite". The soil strength parameters from the two investigations (GCO, 1982a and GCO, 1989b) are summarised in Table 2. Despite the two sets of laboratory tests being conducted at different times with samples taken from various locations, strengths measured were generally of the same order and are considered sufficiently indicative of the strength of intact materials at the site, for the purposes of this study. Any inaccuracies and lack of representativeness will be outweighed by the uncertainties in assessing mass strength which will depend on the closeness and degree of adversity (and persistence) of the relict joint network and the part contribution from previously slipped surfaces, as discussed below in Section 7.

6.5 Groundwater Conditions

The groundwater conditions at the site were evaluated from a review of the available groundwater records and seepage observations. These include the following:

(a) observed water seepage from slope faces and especially from raking drains (Maunsell, 1972; GCO, 1982b; GCO, 1989a),

(b) groundwater monitoring data for 14 vertical boreholes (DH1 to DH14, Figure 10) under the 1988/89 LPM programme over a period between June 1988 and July 1989,

(c) pre- and post-landslide groundwater monitoring data for 23 vertical boreholes for the Butterfly Valley Primary Service Reservoir project over periods from March 1996 to February 1997 and from July 1997 to November 1997,

(d) observations of water seepage from several locations in the main scarp of the landslide between 30 July 1997 and 5 August 1997 and from old raking drains exposed in the failure,

(e) the performance of additional raking drains installed as part of the emergency remedial works following the 1997 landslide, and

(f) post-failure ground investigation in 1997 including insitu permeability tests.

There is conflicting information regarding groundwater conditions in the slope. Certainly, evidence of locally high groundwater conditions has been noted from time to time, prior to the 1997 landslide as reported in Section 4. The general lack of observed seepage during LPM works (1990-1992) may relate to the fact that the annual rainfalls for the eight years prior to 1992 were below average.

Boreholes in the vicinity have indicated generally low groundwater levels with broad response to rainfall events. The piezometer in borehole DH12, for example, installed for the LPM investigations at a location close to the top of the subsequent failure scar, showed a gradual rise in water level in September 1988, following a wet period and continued to rise for two months, despite dry weather (GCO, 1989a).

Water levels in the Butterfly Valley Primary Service Reservoir piezometers, with their tips close to rockhead level, show a rise in water level, generally of about 8 m, in response to the recent series of rainstorms in June and July 1997.

Confirmation of adverse groundwater conditions during the recent landslide comes from many observations. An inspection on 30 July 1997 showed a line of the raking drains which had been installed in 1972, hanging out of the landslide backscar and flowing with water (GEO, 1997). Whether they were flowing before they were disturbed by the failure of 7 July is unknown.

Following the 3 August failure, many water seepage points were observed in the backscar, from about 10 m below the crest of the rear scarp and at lower levels (Figure 11). In addition, three raking drains installed at mid-height of the landslide backscar as part of the remedial works have continued to flow since installation (Plate 12).

An important observation following the 1997 failure was that, "even following heavy rain, the natural drainage line in the centre of the site was dry, suggesting that surface water is being intercepted at some point upslope" (GEO, 1997). It is considered likely that rain falling on the catchment above the slope infiltrated rapidly into the pipe system within the hill and then migrated towards the slope at a rate controlled by the hydraulic conductivity of the system. In the event, it took several days for the effect of the major rainstorm of early July to be translated into a rise in groundwater pressure in slope 11NW-A/C55 and for the failure to be triggered.

7. THEORETICAL STABILITY ANALYSES

7.1 General

Theoretical stability analyses were carried out to assist the diagnosis of the mechanism and causes of the landslide. These analyses were used to determine probable operative mass shear strength parameters, given the range of possible groundwater conditions at the time of the landslide. For this purpose it was estimated that the top of the transient water system was probably in the range of 5 m to 7 m below the existing ground surface (Figures 12 and 13). Water levels are given relative to a postulated continuous basalt dyke shown in Figure 16 for ease of reference.

7.2 Analysis of 1997 Failure on Cut Slope No. 11NW-A/C55

Possible geotechnical models have been compiled on the basis of information from desk study, site observation and post–failure ground investigation as detailed elsewhere in this report. Representative cross-sections of the landslide site used for the analyses are shown in Figures 12 and 13. The basic geological model differs little from that used in the original design of the slope in GCO (1989b), the main difference being in the assumed groundwater levels. Sensitivity analyses for the 1997 failure were carried out using a range of possible strength parameters based on site-specific laboratory tests and experience. Two possible slip surfaces were used in the analyses (Figures 12 and 13), namely;

(a) the best estimated slip surface geometry for the 1997 failure, and

(b) a postulated deeper-seated slip surface.

The stability of the slope has been analysed using the Janbu Rigorous method (Janbu, 1954). The results of the analyses are summarised in Figure 16.

The lowest shear strength parameters used for the trial analyses were $c' = 0$ and $ø' = 35°$. Such low mass strength might only be generally applicable if much of the 1997 landslide geometry replicated that of a previous failure, in which case remoulded shear strengths might be appropriate. It can be seen that, given such low strength, little water pressure would be required to cause failure. Observed water seepage indicated water levels higher than this.

Conversely, if the weathered mass had the strength of intact highly decomposed granite as measured during LPM investigations (GEO, 1988), i.e. with parameters $c' = 15$ kPa and $ø' = 39°$, then much higher water pressures would be necessary to cause failure, namely 20.6 m above the reference dyke for the likely 1997 landslide geometry, and 24 m above for the deeper-seated slip surface.

The most likely piezometric head above the postulated dyke, based on field observations, was of the order of 17 to 19 m. That being so, the effective shear strength would then be defined with parameters of c' = 5 to 9 kPa and ø' between 36° and 39°. This strength would be a function of the presence of material of variable intact strength and the influence of relict joints and zones of previously sheared material along the rupture surface.

8. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

8.1 Factors for Consideration

The various factors that need to be considered in assessing the causes of the 1997 failures include:

(a) the history of landsliding at the site,

(b) groundwater and the role of the raking drains used to stabilise the 1972 failure, and

(c) the apparently progressive nature of the failure.

8.2 History of Landsliding

The Most significant finding about this site is its long history of instability. Figure 17 shows the approximate extent of the 1954/1963 failure as indicated on 1972 as-built drawings for the Ching Cheung Road project superimposed on a recent site plan. Results from photogrammetric analysis of the 1963 photographs confirm that the recent failure scarp largely coincides with the 1963 failure scarp in plan. Figures 18 and 19 show sections with slope profiles through the landslide area based on the photogrammetric analysis of the 1963 aerial photographs and surveying before and after the recent landslides. The geometry of the 1954 (1963 photographs) postulated failure surface has been estimated based on the backscarp observed from the aerial photographs, the known position of the original slope toe in the borrow area as well as the depth of occurrence of material found on site from the recent ground investigation works which was identified as "disturbed". The position of the postulated deep-seated failure surface for the 1997 landslide was based on the location of the tension crack observed on site, the reported lack of movement at the slope toe as well as the disturbed material found during ground investigations. It is clear that the slope has been progressively cut back or has failed (1972, 1997) towards the 1954 failure geometry but that displaced material has remained and still remains in cut slope 11NW-A/C55.

Previous landsliding will have caused local weakening of the weathered rock mass. Failed zones may have contained loose disturbed material, as encountered in the recent investigation (Figure 12). Despite the known presence of an old landslide, however, there is no clear evidence other than partly coincidental geometry, in plan, that the 1997 failure is a reactivation of earlier failures. Reported lack of disturbance at the toe of the slope limits the possible geometry of the recent failure to above road level. That being so, it is postulated that the geometry of the recent failure is constrained by the insitu stress conditions imposed by slope geometry, interaction with structures at the toe of the slope and groundwater conditions and that much of the failure surface is through material which has not been sheared previously. Locally, however, failed sections were almost certainly reactivated, thereby contributing to a lower overall mass strength. Analysis shows that failure is possible in such circumstances (Section 7).

The history of landsliding is also considered important in that the disturbance has probably contributed to the development of the natural pipe system which is thought to control the hydrogeology of the area.

8.3 Groundwater and the Role of the Raking Drains

The analyses confirm that, in the absence of severe water pressure, the slope should be stable. For the likely groundwater conditions, based on field observations following the failure, however, analysis shows that the slope would be unstable either as a relatively small failure in the lower two batters or through the complete height of the slope, as per the July and August failures.

The raking drains installed in 1972 were unable to prevent the critical water pressure developing within the slope. This was almost certainly largely because the rainfall prior to the failure was extremely severe (an estimated 1 in 500 year return period for the 31-day rainfall prior to 4 July) and much higher than previous rainfall survived by the slope since the drains were installed (Figure 8). The inability of the drains to lower the groundwater adequately may also have been affected by possible deterioration and/or blockage of the drains prior to the failure.

8.4 Progressive Nature of the Failure

The apparent progressive nature of the landslide (7 July, 17 July and 3 August) is considered potentially misleading with respect to the cause of the failure. Analysis shows that the lower two batters which collapsed on 7 July would be unstable under severe groundwater conditions, as considered reasonable following the extreme rainfall of early July, but similarly analysis shows that the whole slope could have been destabilised under those conditions. Analysis also indicates minimal reduction in the FOS of the whole slope (full extent of the 1997 landslide) from removal of a comparatively small toe weight.

The delayed nature in response to the rainstorm event of the first incident is considered indicative of a deep-seated failure, four days being a reasonable delay for water to transfer from the catchment to the failure zone, based on measured field hydraulic conductivities. Similarly, the fact that the width of the first incident was much greater than the exposed height of distress and that the width remained constant as the upper parts of the slope subsequently collapsed, is taken as indicative that the main surface of rupture formed as a result of the early July groundwater conditions. The incidents of 17 July and 3 August were probably collapses of already moved material, mobilised by (and during) heavy rainstorms.

9. CONCLUSIONS

It is concluded that the failures on 17 July and 3 August 1997 represented the progressive collapse of a large-scale landslide which occurred on 7 July 1997.

The principal cause of failure was adverse transient groundwater conditions, which developed following severe rainfall in early July 1997 and the previous month. A system of raking drains, installed in the slope in 1972, was unable to prevent the critical water pressures developing.

The plan shape of the failure was largely coincident with that of an earlier landslide and previously sheared surfaces may have been reactivated in part. A large volume of failed material from the 1954/1963 failure remains in place on the slope and beneath Ching Cheung Road.

10. REFERENCES

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Table 1 - Maximum Rolling Rainfall at GEO Raingauge No. K06 for Selected Durations Preceding the Recent Landslide Events and Corresponding Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
(a) Selected durations preceding the 7 July 1997 Landslide			
5 minutes	13	05:45 hours on 2 July 1997	2
15 minutes	36	06:10 hours on 2 July 1997	7
1 hour	114.5	06:15 hours on 2 July 1997	19
2 hours	162.5	06:30 hours on 2 July 1997	18
4 hours	192	07:00 hours on 2 July 1997	10
12 hours	265	15:00 hours on 2 July 1997	7
24 hours	431.5	02:00 hours on 3 July 1997	20
2 days	534.5	02:00 hours on 4 July 1997	27
4 days	628	09:00 hours on 4 July 1997	25
7 days	673	15:00 hours on 6 July 1997	21
15 days	827.5	15:00 hours on 6 July 1997	19
31 days	1557.5	02:00 hours on 4 July 1997	498
(b) Selected durations preceding the 17 July 1997 Landslide			
5 minutes	13	05:45 hours on 2 July 1997	2
15 minutes	36	06:10 hours on 2 July 1997	7
1 hour	119	06:25 hours on 2 July 1997	24
2 hours	162.5	06:30 hours on 2 July 1997	18
4 hours	192	07:00 hours on 2 July 1997	10
12 hours	265	15:00 hours on 2 July 1997	7
24 hours	431.5	02:00 hours on 3 July 1997	20
2 days	534.5	02:00 hours on 4 July 1997	27
4 days	628	09:00 hours on 4 July 1997	25
7 days	673	15:00 hours on 6 July 1997	21
15 days	870	02:00 hours on 16 July 1997	25
31 days	1212.5	15:00 hours on 17 July 1997	52
(c) Selected durations preceding the 3 August 1997 Landslide			
5 minutes	9	04:00 hours on 19 July 1997	1
15 minutes	23	07:00 hours on 17 July 1997	1
1 hour	52.5	07:20 hours on 17 July 1997	1
2 hours	68.5	07:55 hours on 17 July 1997	1
4 hours	75	10:00 hours on 17 July 1997	1
12 hours	133	21:00 hours on 2 August 1997	1
24 hours	199	09:00 hours on 3 August 1997	2
2 days	267	10:00 hours on 3 August 1997	2
4 days	307.5	15:00 hours on 19 July 1997	2
7 days	341	20:00 hours on 19 July 1997	2
15 days	484.5	20:00 hours on 19 July 1997	2
31 days	851.5	11:00 hours on 3 August 1997	5

Notes: (1) Return periods were derived from the Gumbel equation and data published in Table 3 of Lam and Leung (1994).

(2) Maximum rolling rainfall was calculated from 5-minute data for durations up to two hours and from hourly data for longer durations.

Table 2 - Summary of Soil Strength Laboratory Test Results from Previous Investigations

Soil Type	Soil Strength Parameters			
	GEO (1982)		GEO (1988)	
	c′ (kPa)	ø′ (degrees)	c′ (kPa)	ø ′ (degrees)
Residual / transition soil	-	-	2	37
CDG	-	-	9	39
HDG	12	37.5	15	39
CD Aplite	-	-	7.2	40
CD Dolerite (flooded)	20	28	10	40

Notes: (1) Materials described in 1982 as microfractured HDG on the basis of index properties (Schmidt Hammer Value and Slakeability) have been described by others as CDG on the basis that they can be crumbled by hand.

(2) The terms dolerite and basalt (used in text) are essentially synonymous.

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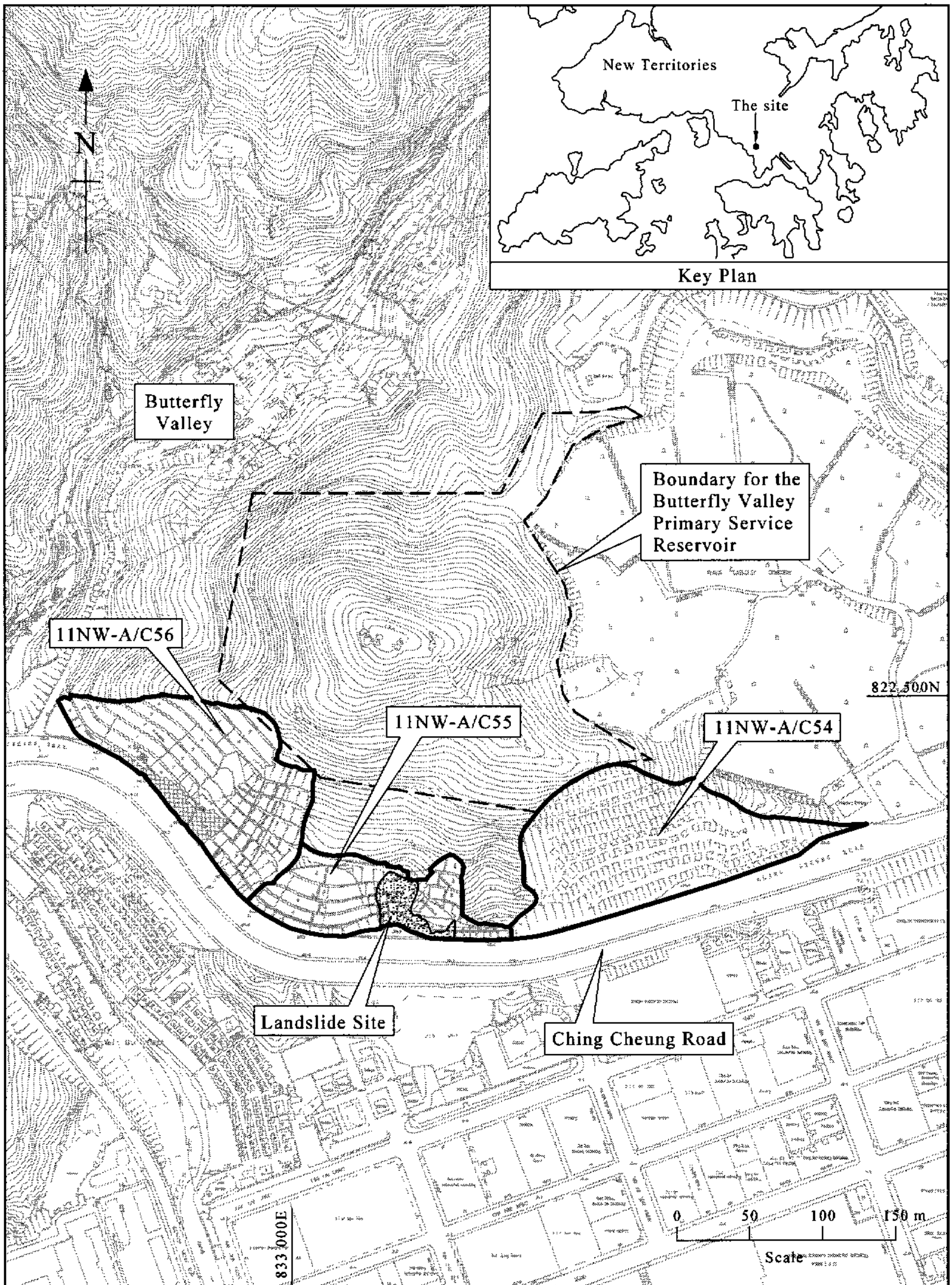
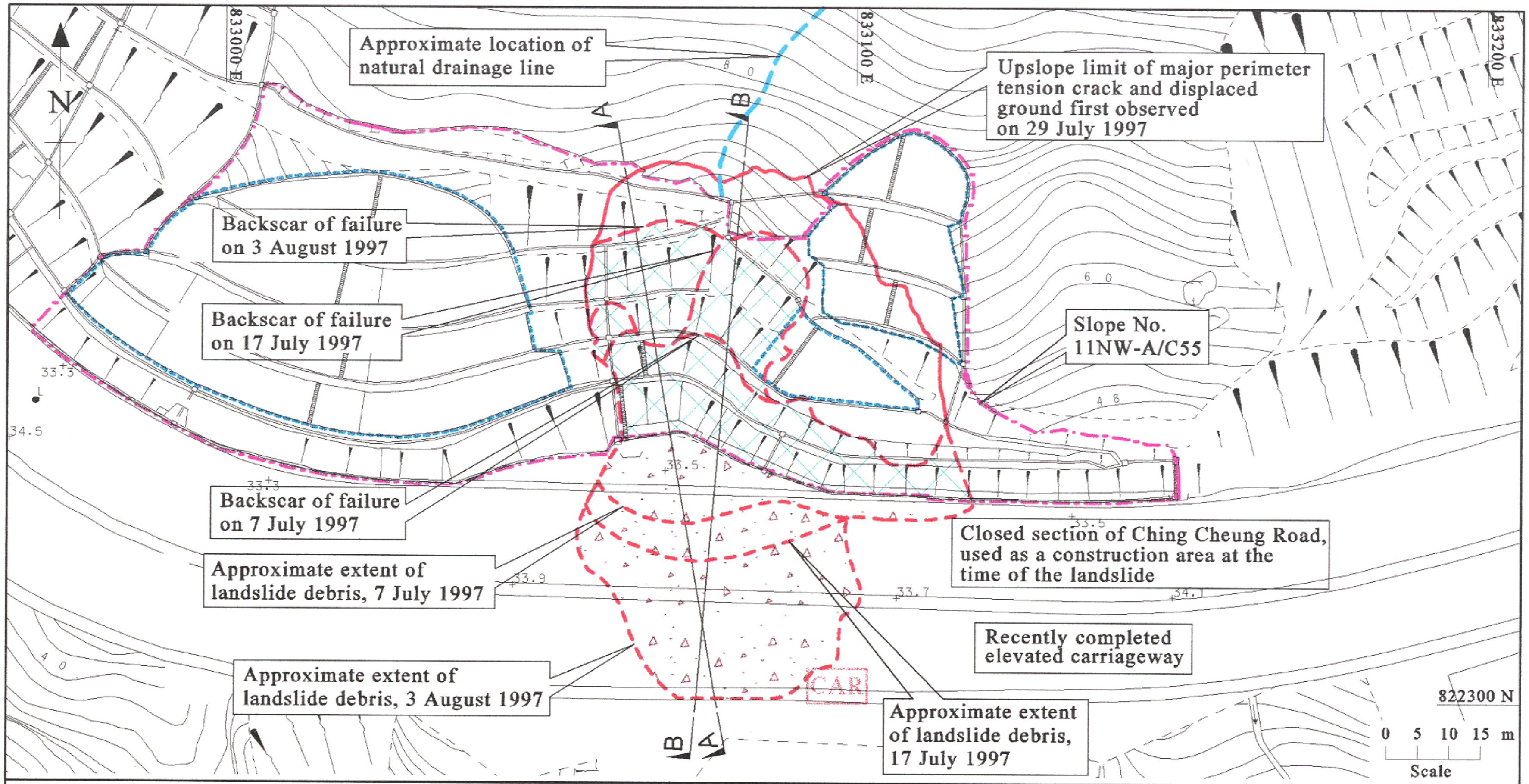


Note : Base map is extracted from Survey Sheet Nos. 11NW-2D, 11NW-3C, 11NW-7B, 11NW-7D, 11NW-8A and 11NW-8C dated October 1992 (Scale 1:1 000).

Figure 1 - Site Location Plan



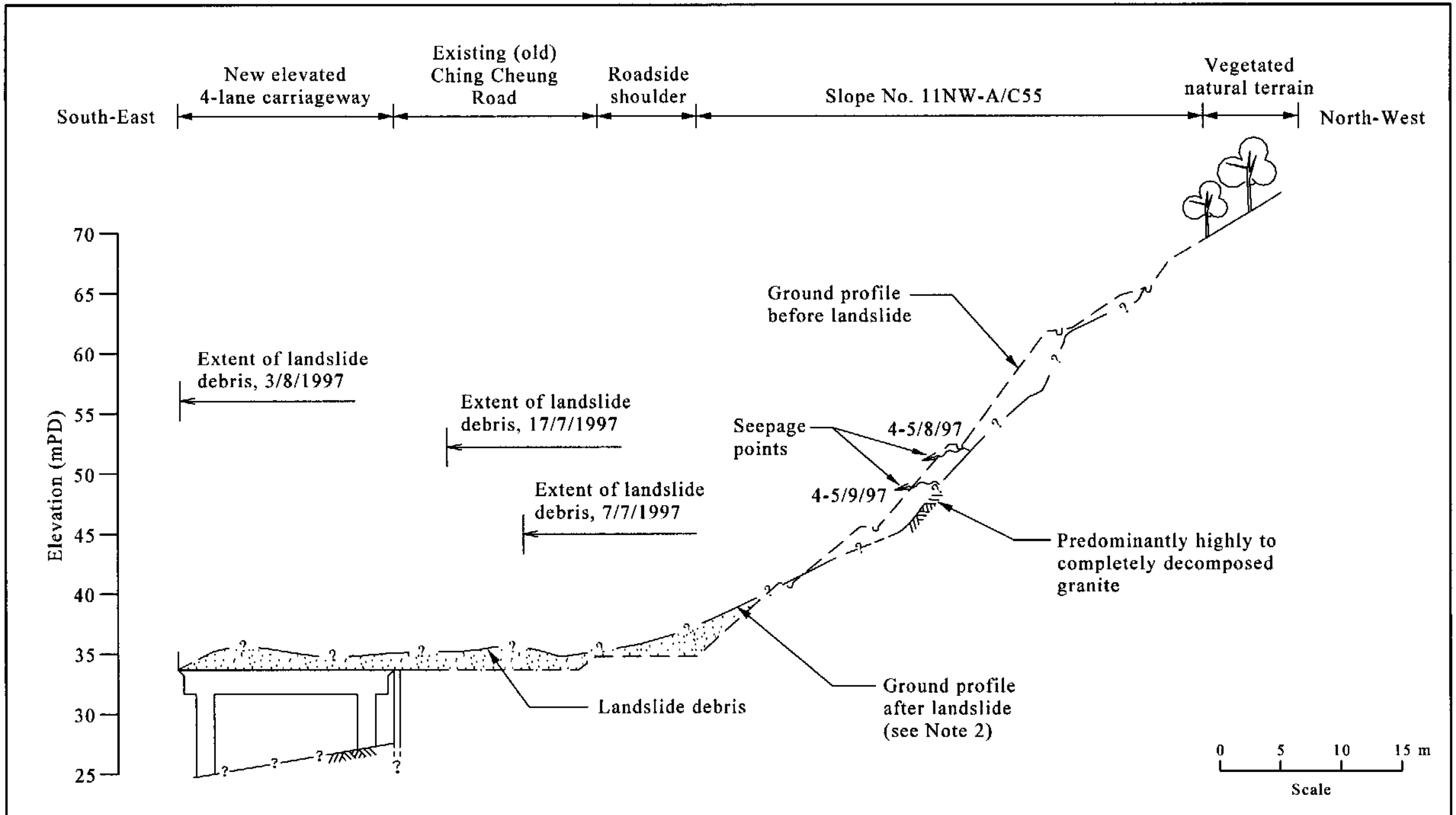
Legend : Slope constructed/reprofiled during LPM Works 1990-92; Landslide scar; Landslide debris; Approximate slope boundary

Notes :

(1) Information shown in this figure is based on topographic survey, 1 : 1 000 survey maps, geological mapping, site observations and documentary records.

(2) Section A-A and B-B are given in Figures 12 & 13 respectively.

Figure 2 - Plan of the Landslide



Notes :

(1) Information shown in this figure is based on topographic survey, 1:1 000 survey map, geological mapping, site observations and documentary records.

(2) The ground and debris profile after landslide is estimated from documentary records.

Figure 3 - Typical Cross-section Through the Landslide

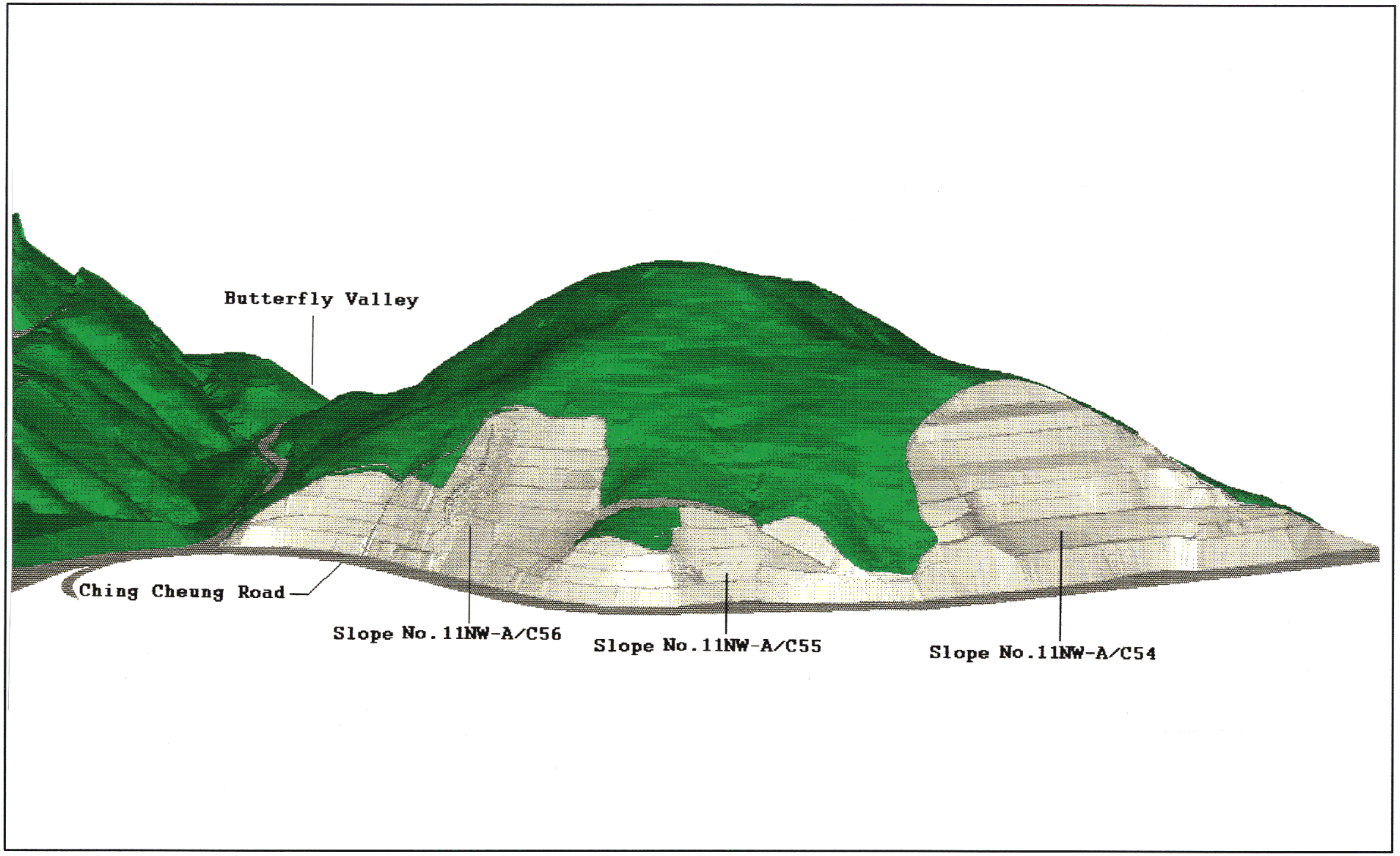


Figure 4 - Computer Generated Three-dimensional View of Cut Slope Topography in 1988 (Before LPM Works)

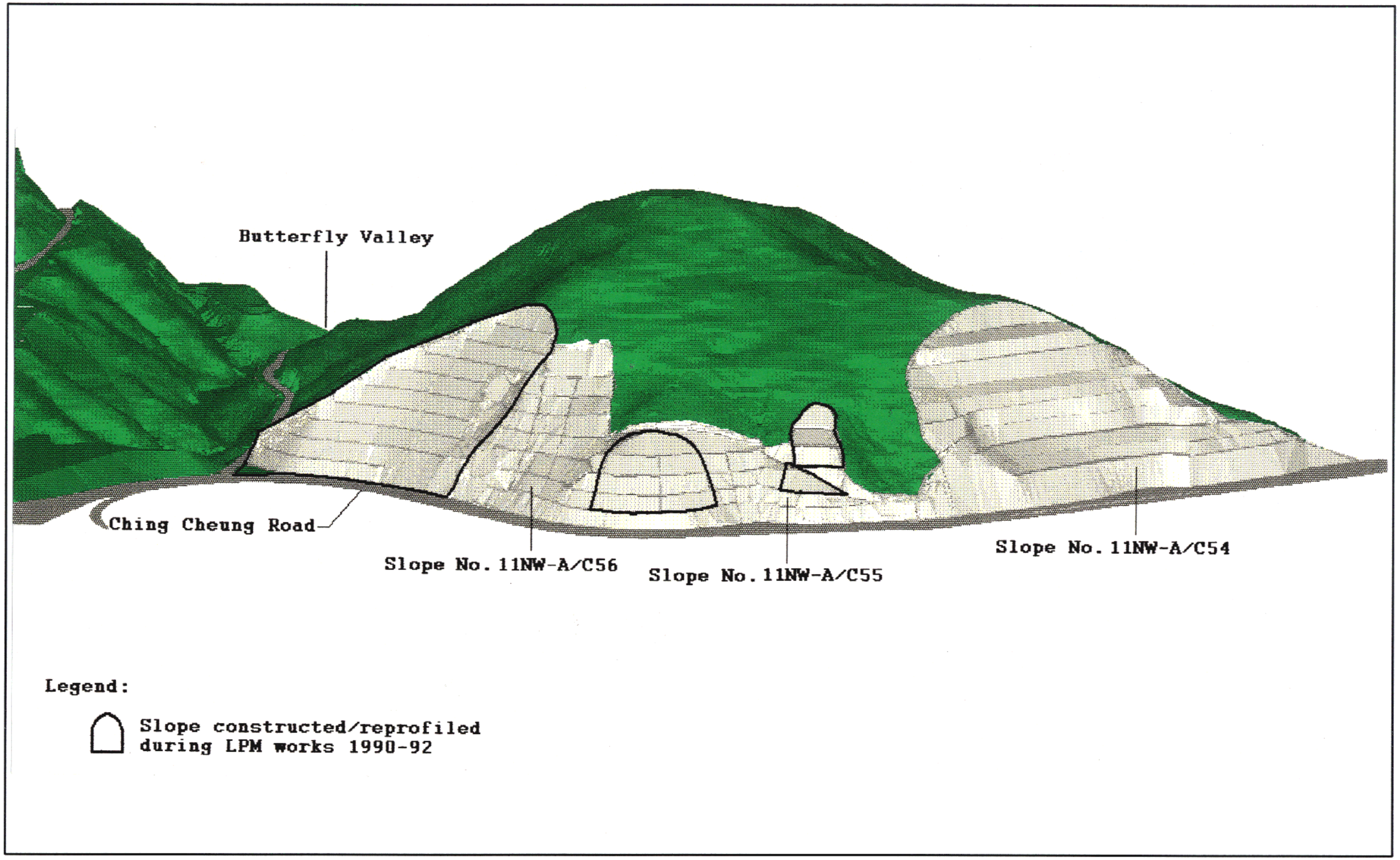


Figure 5 - Computer Generated Three-dimensional View of Cut Slope Topography in 1992 (After LPM Works)

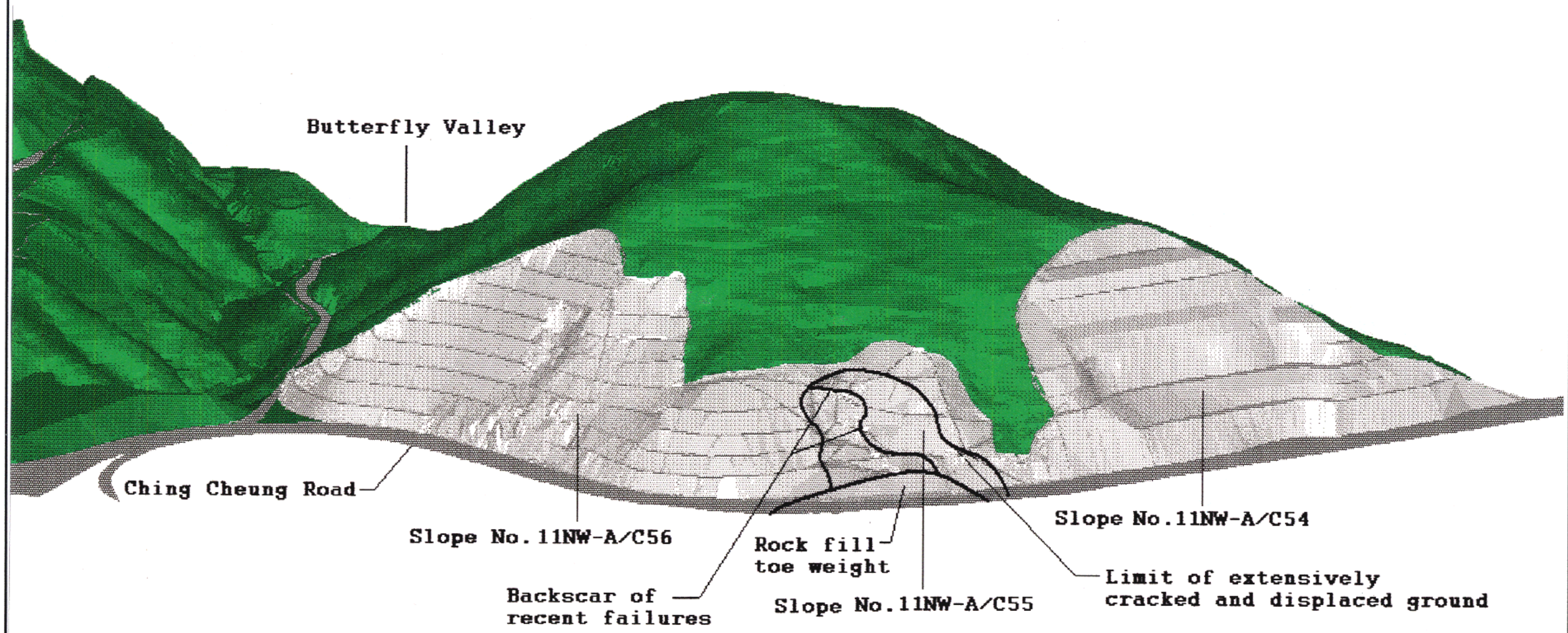
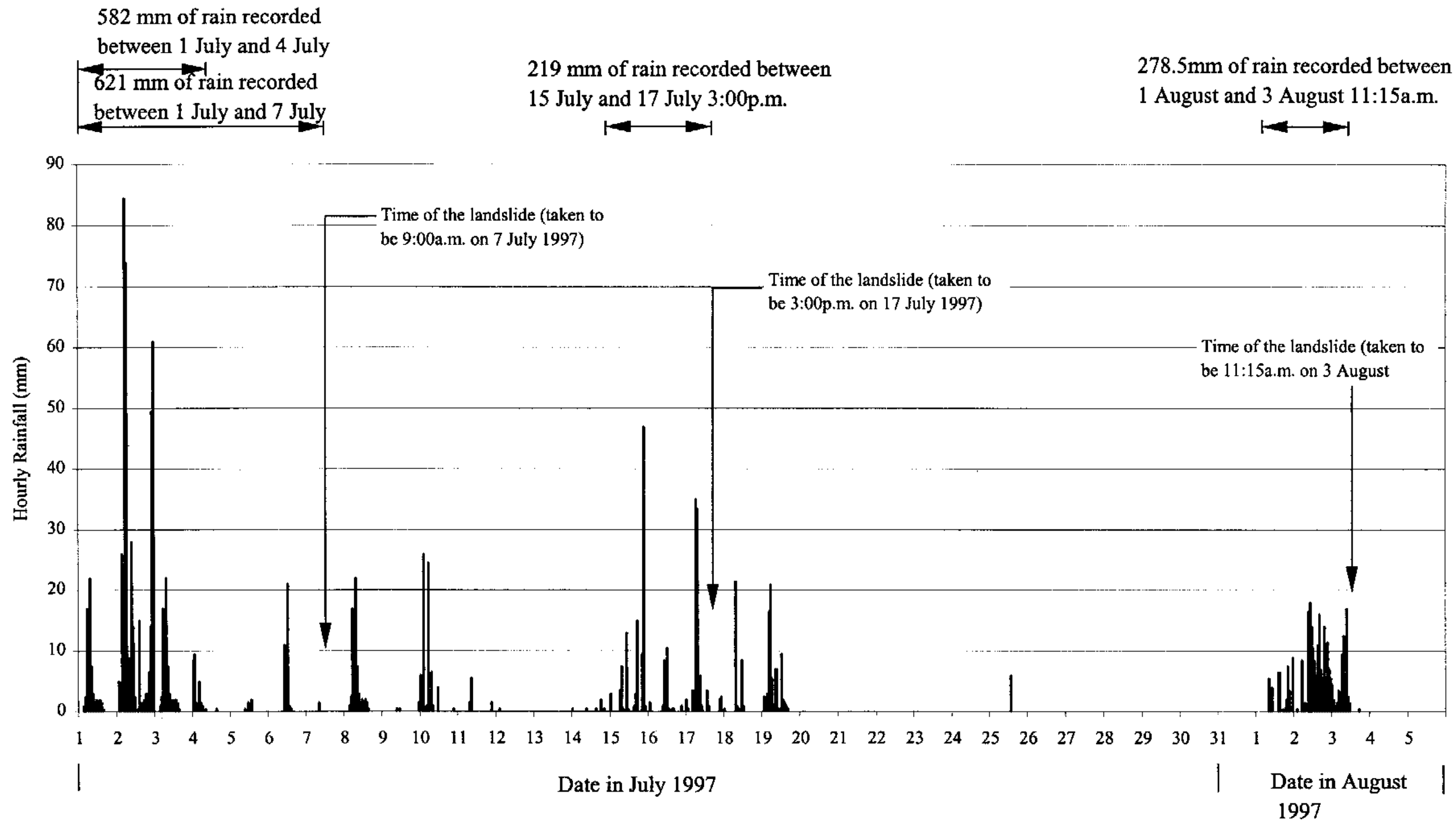
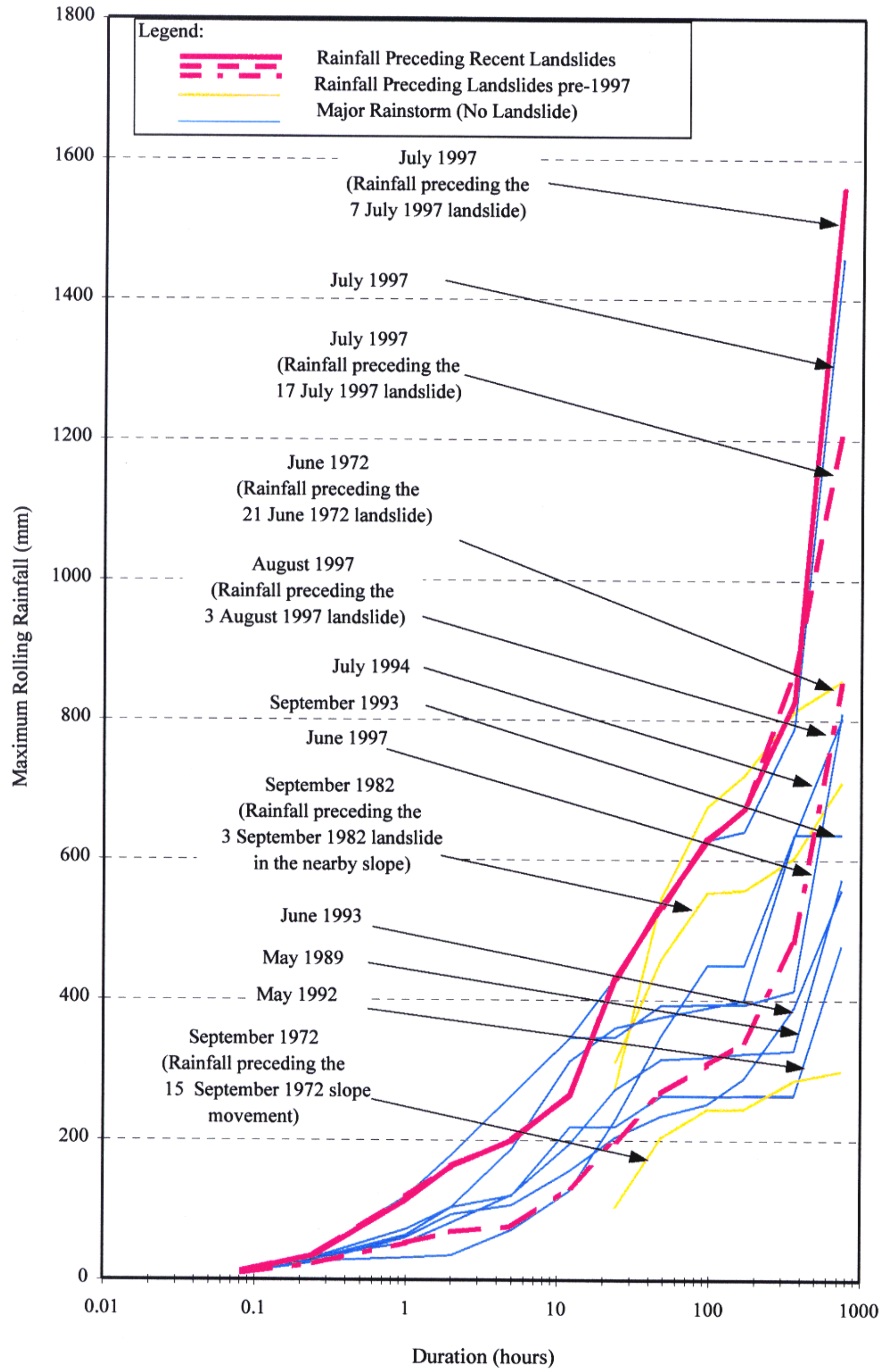


Figure 6 - Computer Generated Three-dimensional View of Cut Slope Topography in 1997 (After Recent Failures)



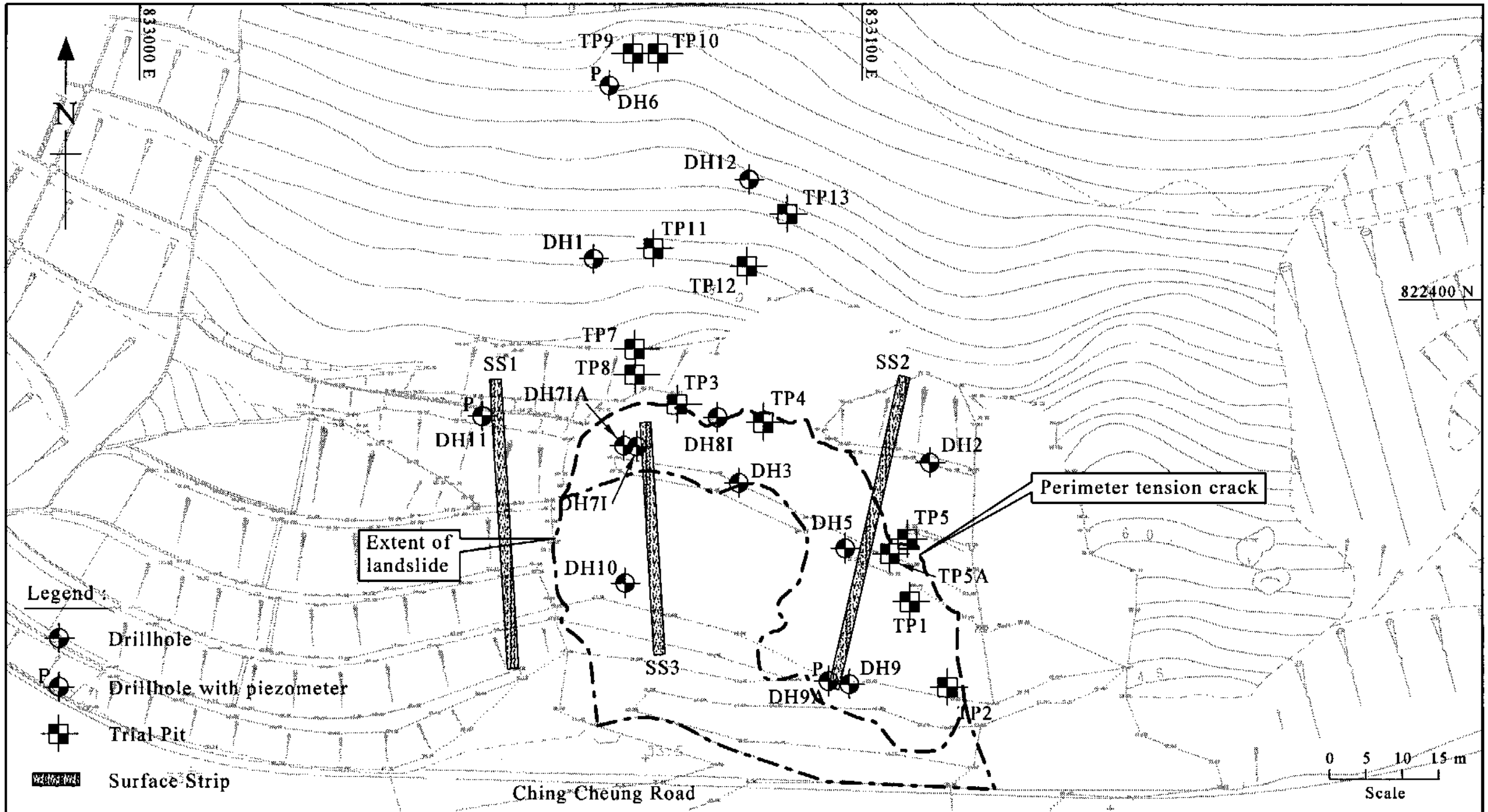
Note: The times of failure are based on interviews with eye witnesses.

Figure 7 - Hourly Rainfall Recorded at GEO Raingauge No. K06



Note:	Rainfall data are from GEO raingauge No. K06, except for: 21 June 1972 and 15 September 1972 from Kowloon Byewash Reservoir raingauge, and 3 September 1982 from GEO raingauge No. N04.

Figure 8 - Maximum Rolling Rainfall at GEO Raingauges for Major Rainstorms



Notes : (1) Extent of landslide is based on documentary records.

(2) Slope has been modified during recent emergency repair work; the extent of which is not shown.

(3) Ground investigation works shown are those scheduled by HAP for the landslide investigation.

Figure 9 - Ground Investigation Location Plan

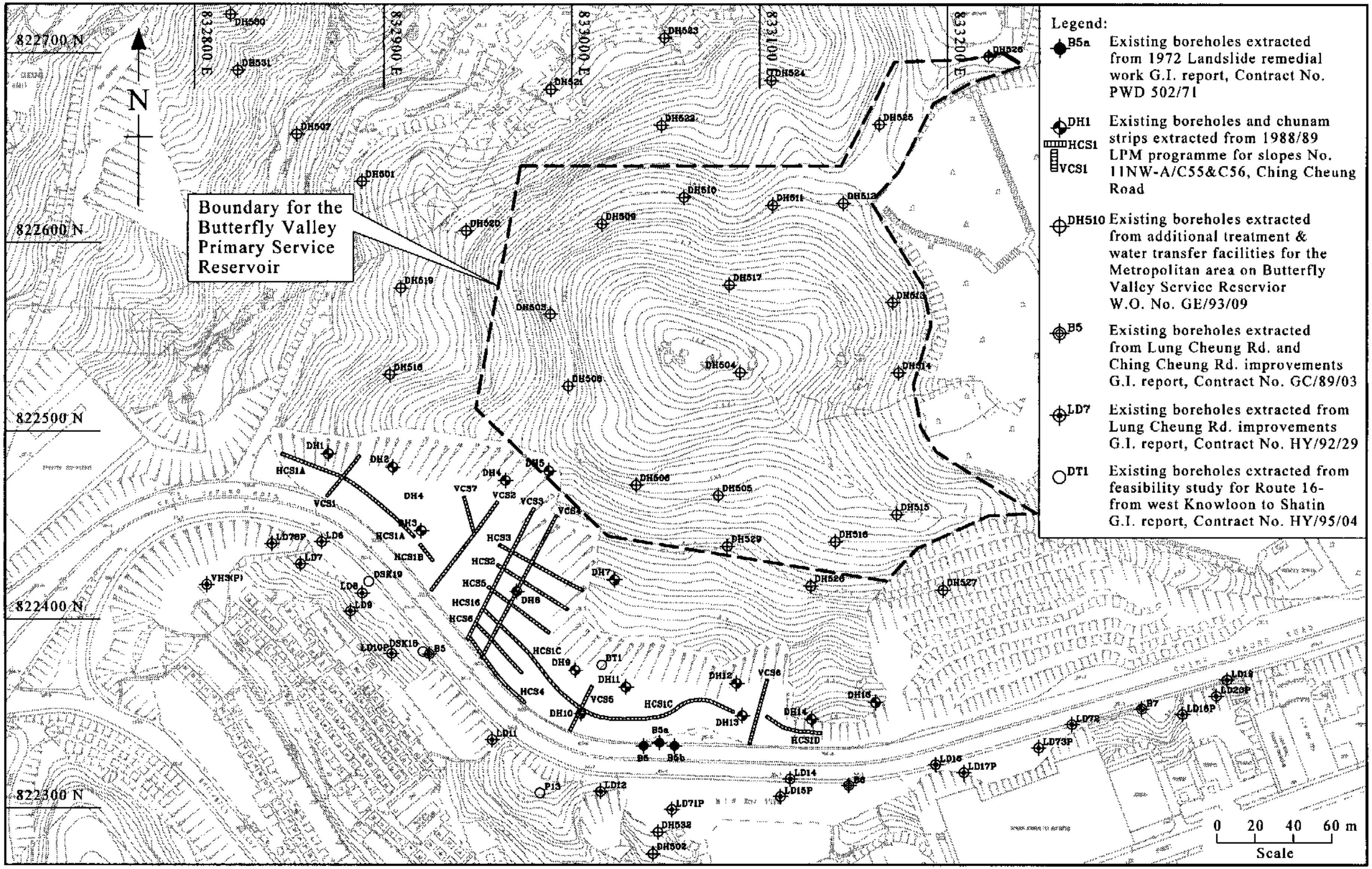
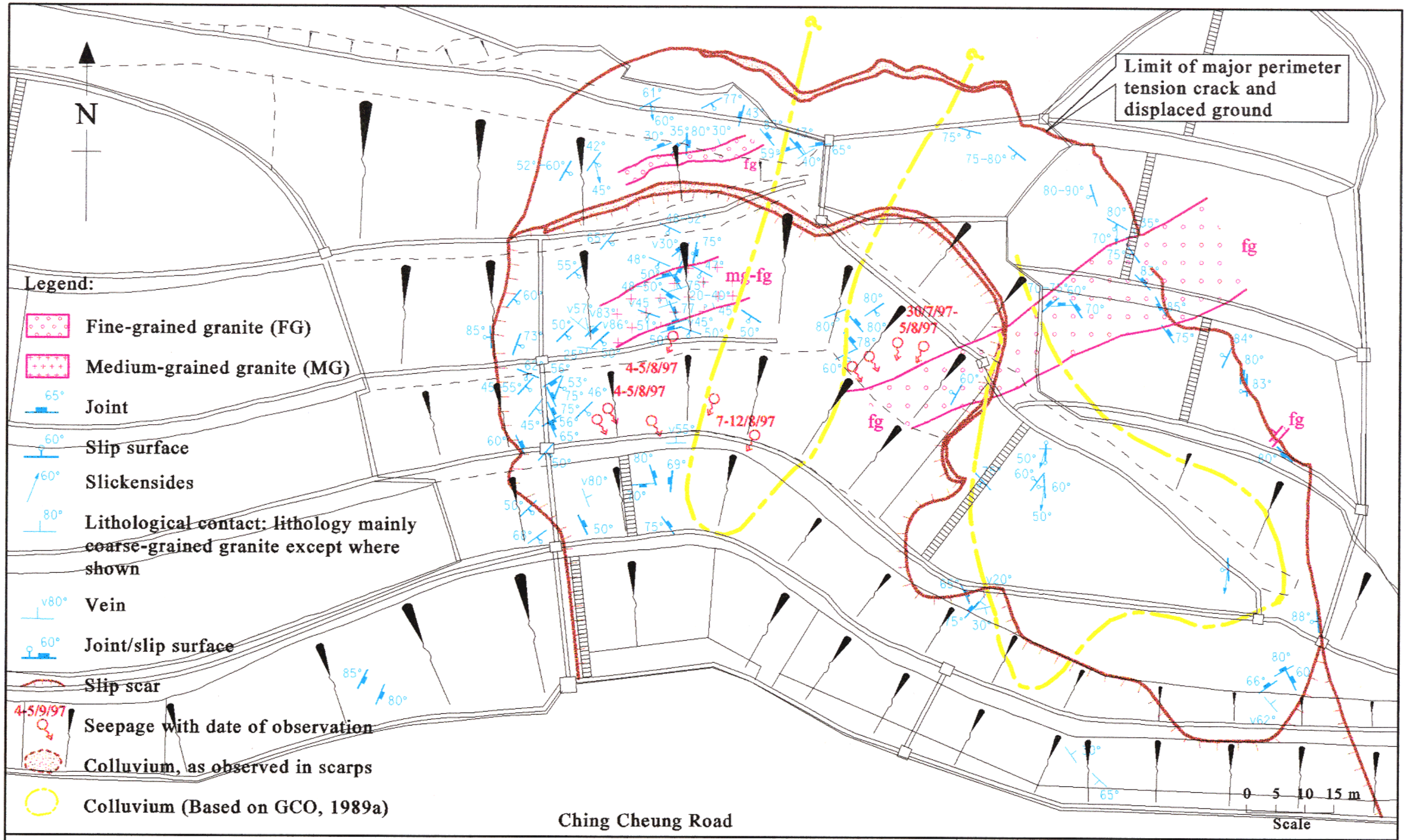


Figure 10 - Pre 1997 Ground Investigation Plan



Note: The information presented is based on field mapping by the Hong Kong Geological Survey.

Figure 11 - Geological Plan of Landslide Area

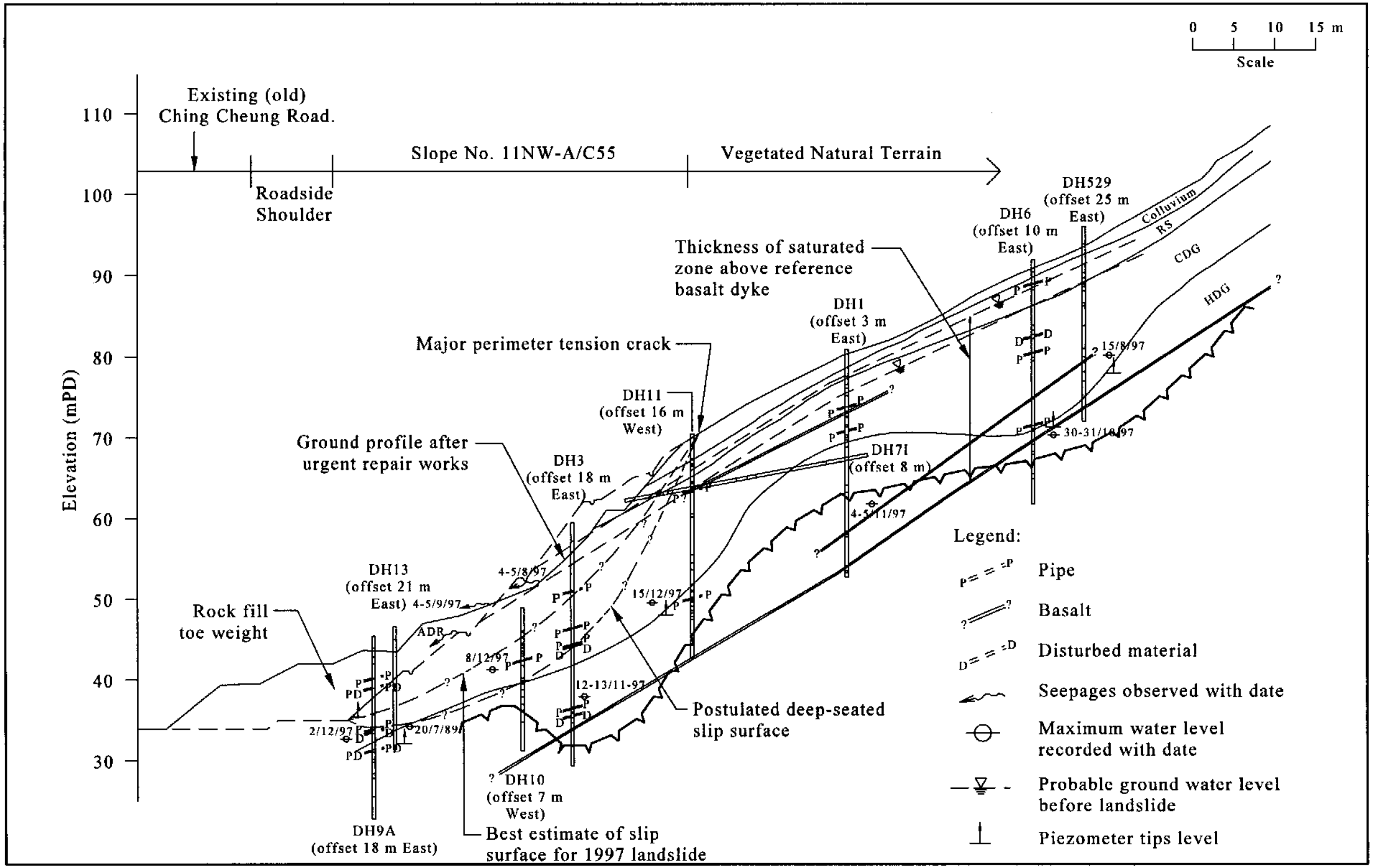


Figure 12 - Geological Cross-section A-A

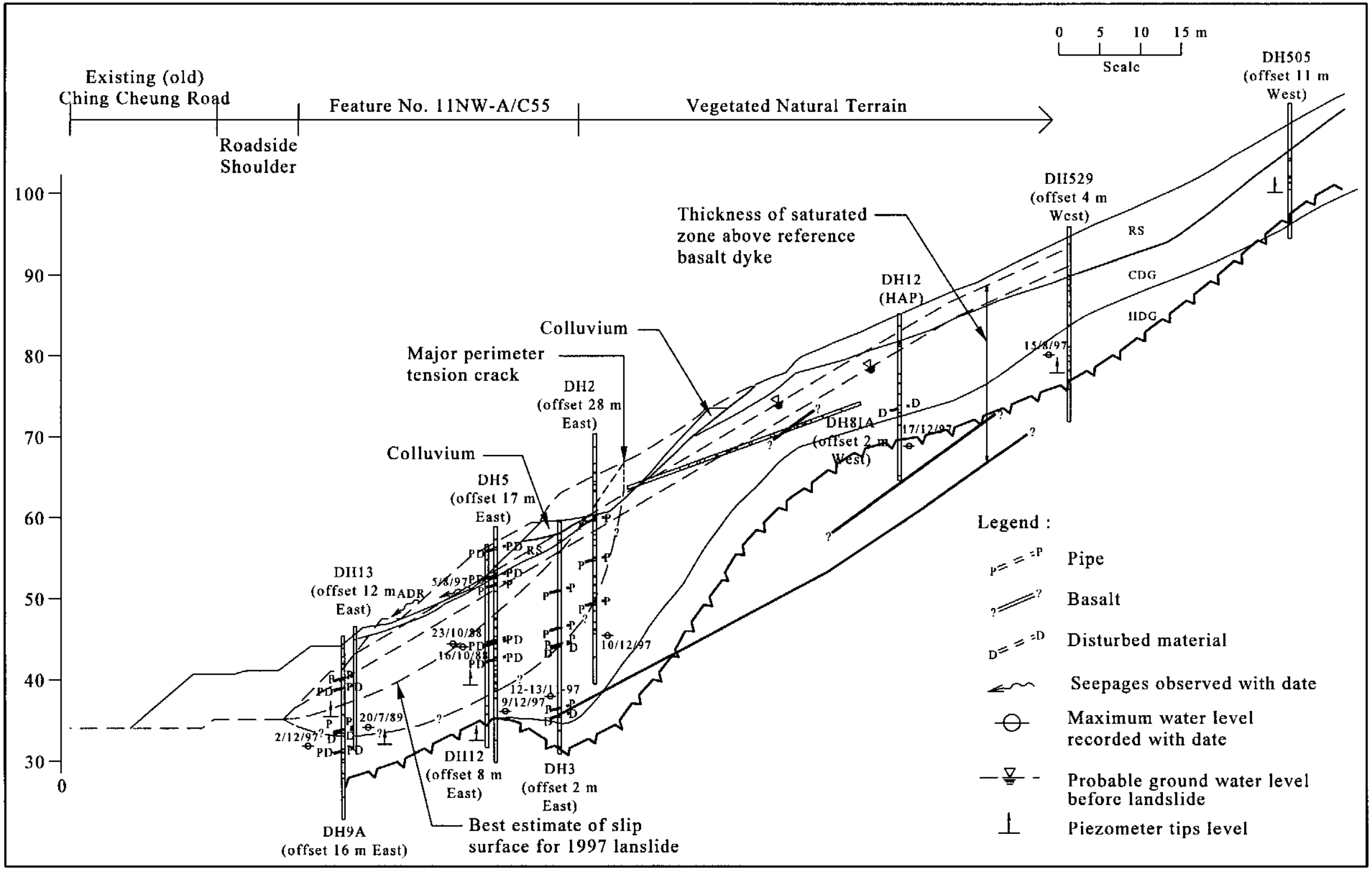


Figure 13 - Geological Cross-section B-B

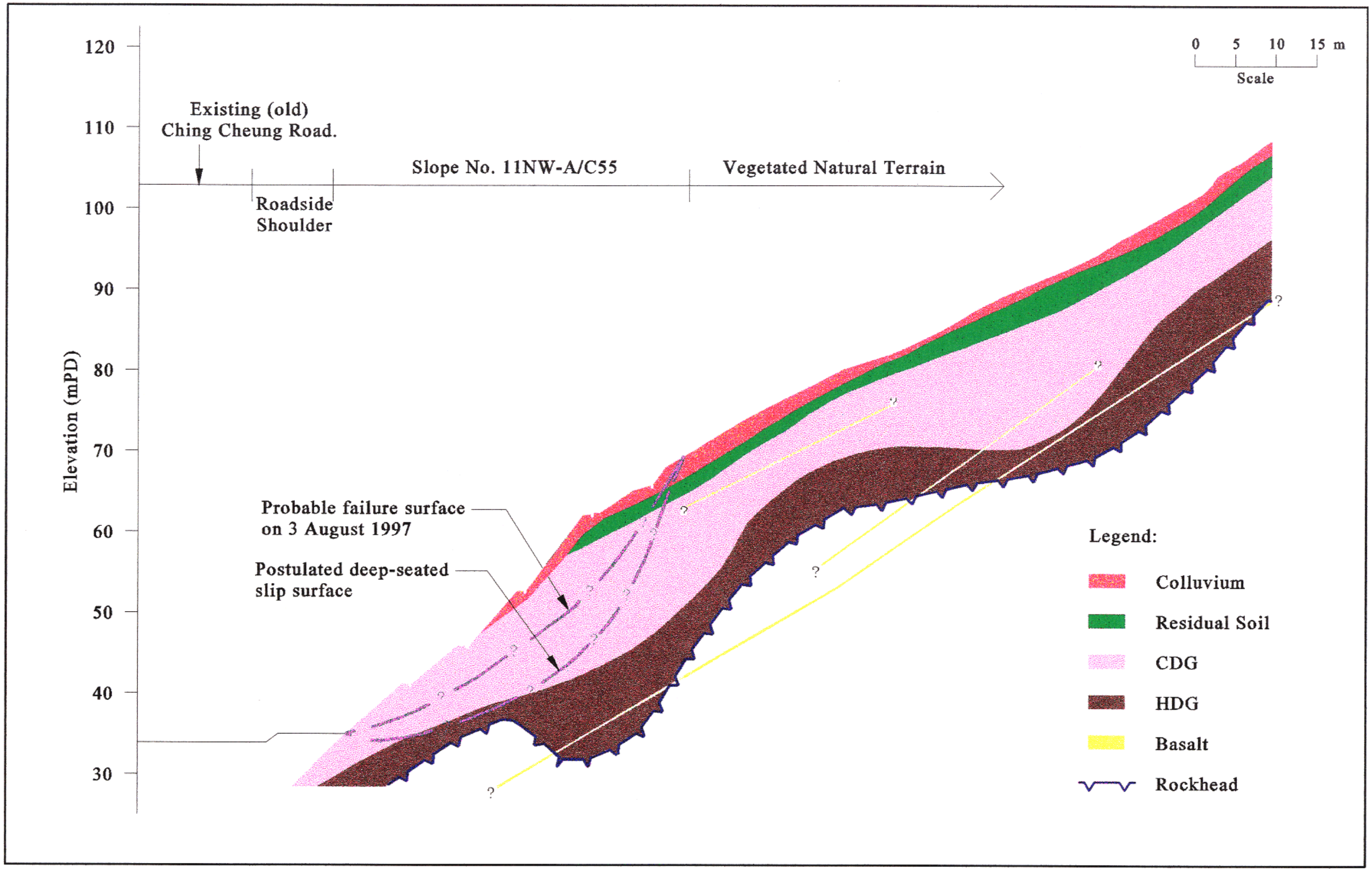
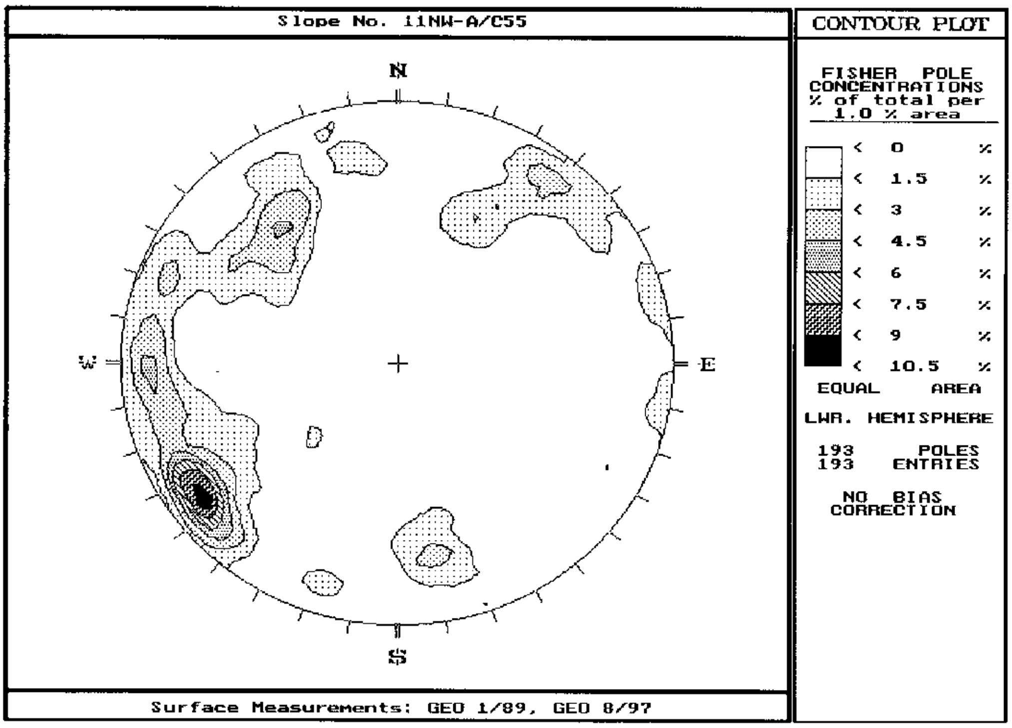
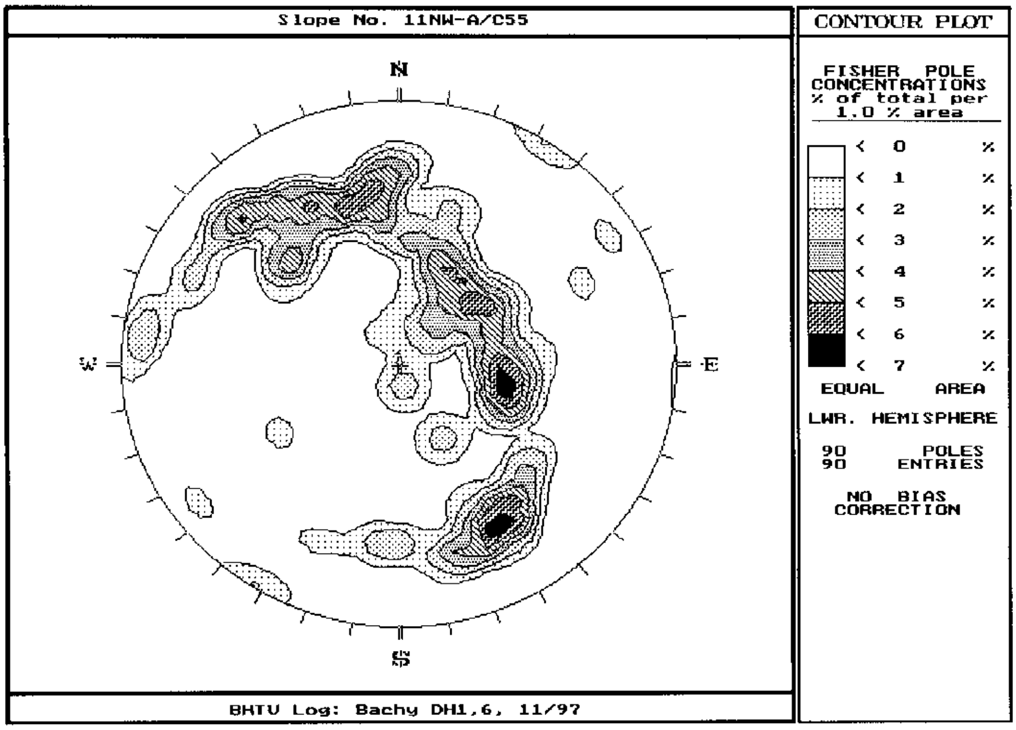


Figure 14 - Outline Stratigraphical Cross-section A-A

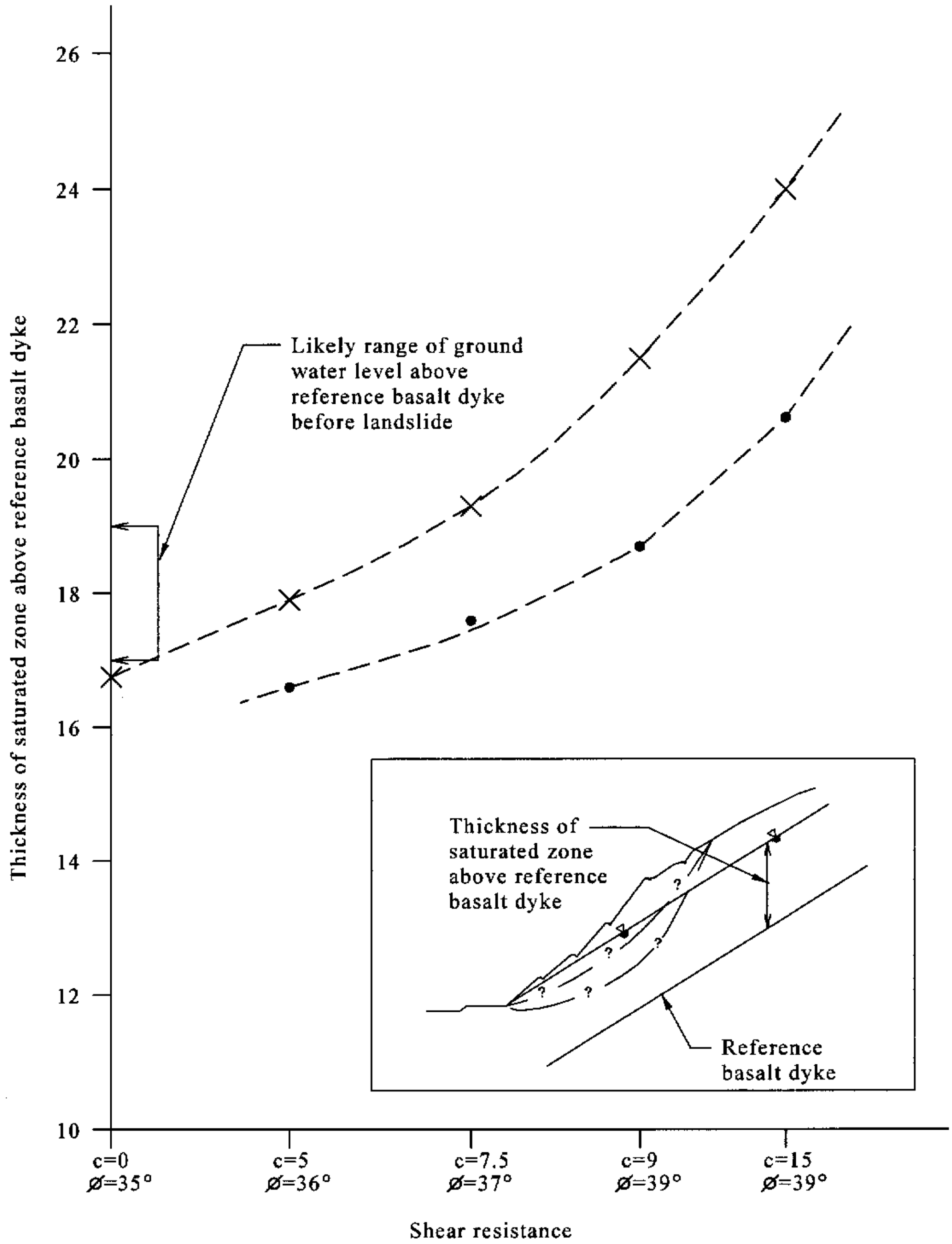


a) Surface Mapping



b) Borehole Data

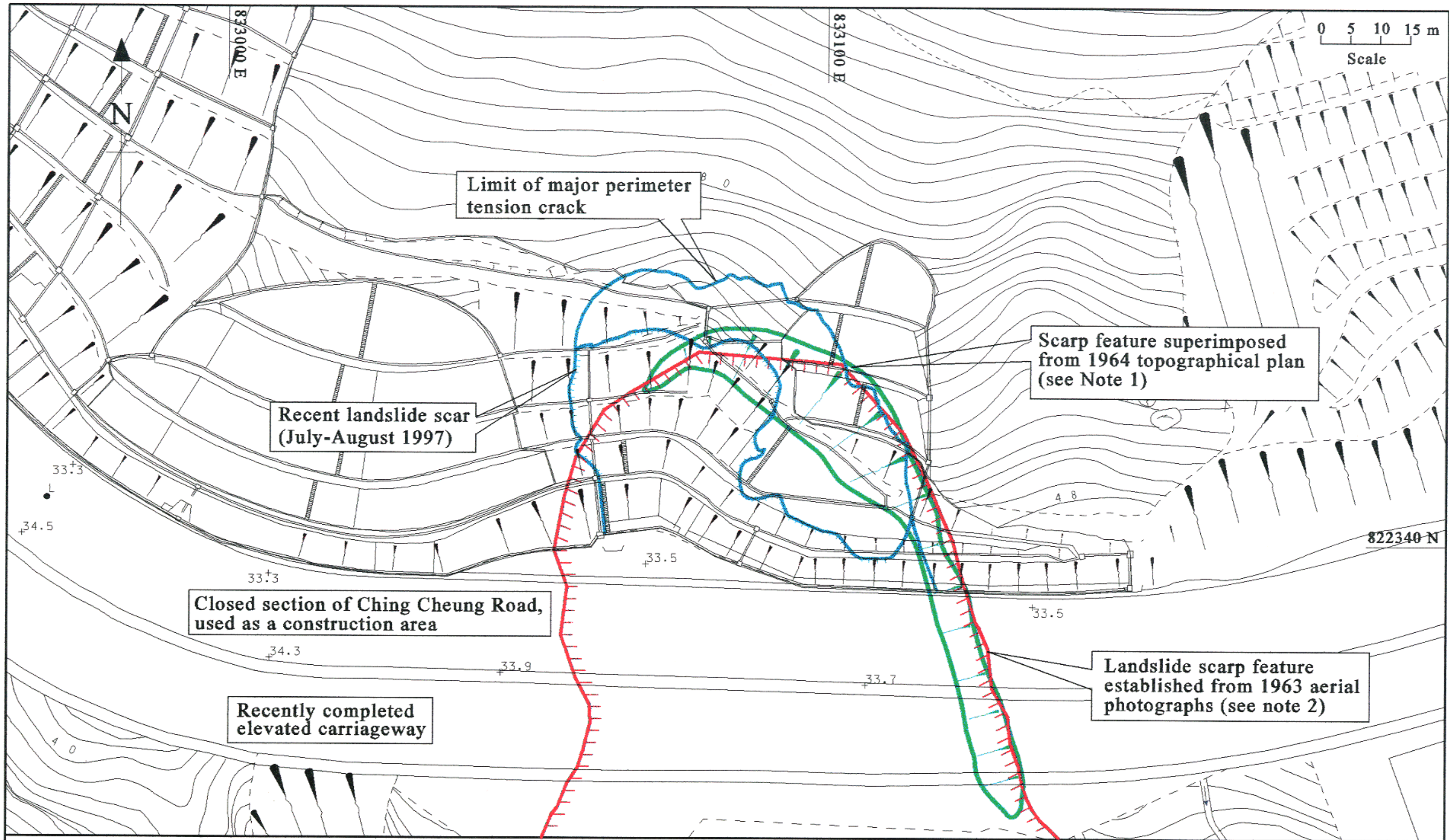
Figure 15 - Stereoplots for the Measured Orientations of Joints for Slope No. 11NW-A/C55



Legend : × For deeper-seated slip surface • For shallow slip surface

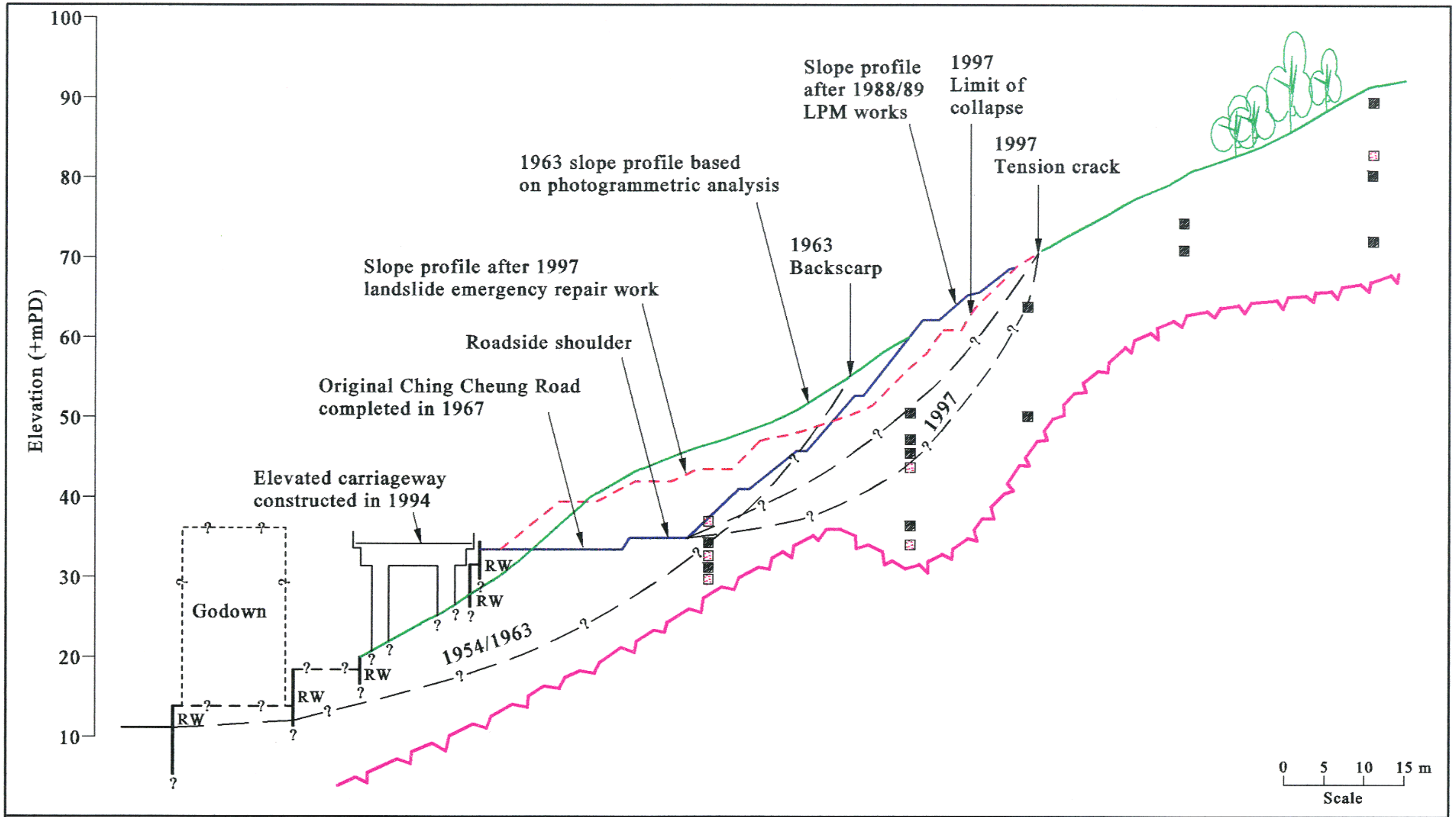
Note : The thickness of the saturated zones obtained from the stability analyses correspond to F.O.S.=1.0

Figure 16 - Results of the Sensitivity Stability Analysis for Cross-section A - A



Notes : (1) Scarp feature from Public Works Department Hong Kong, Drawing No. RK1410/1C, Tai Po Road - Castle Peak Road Link, 1964.
(2) Scarp feature established by photogrammetric analyses.

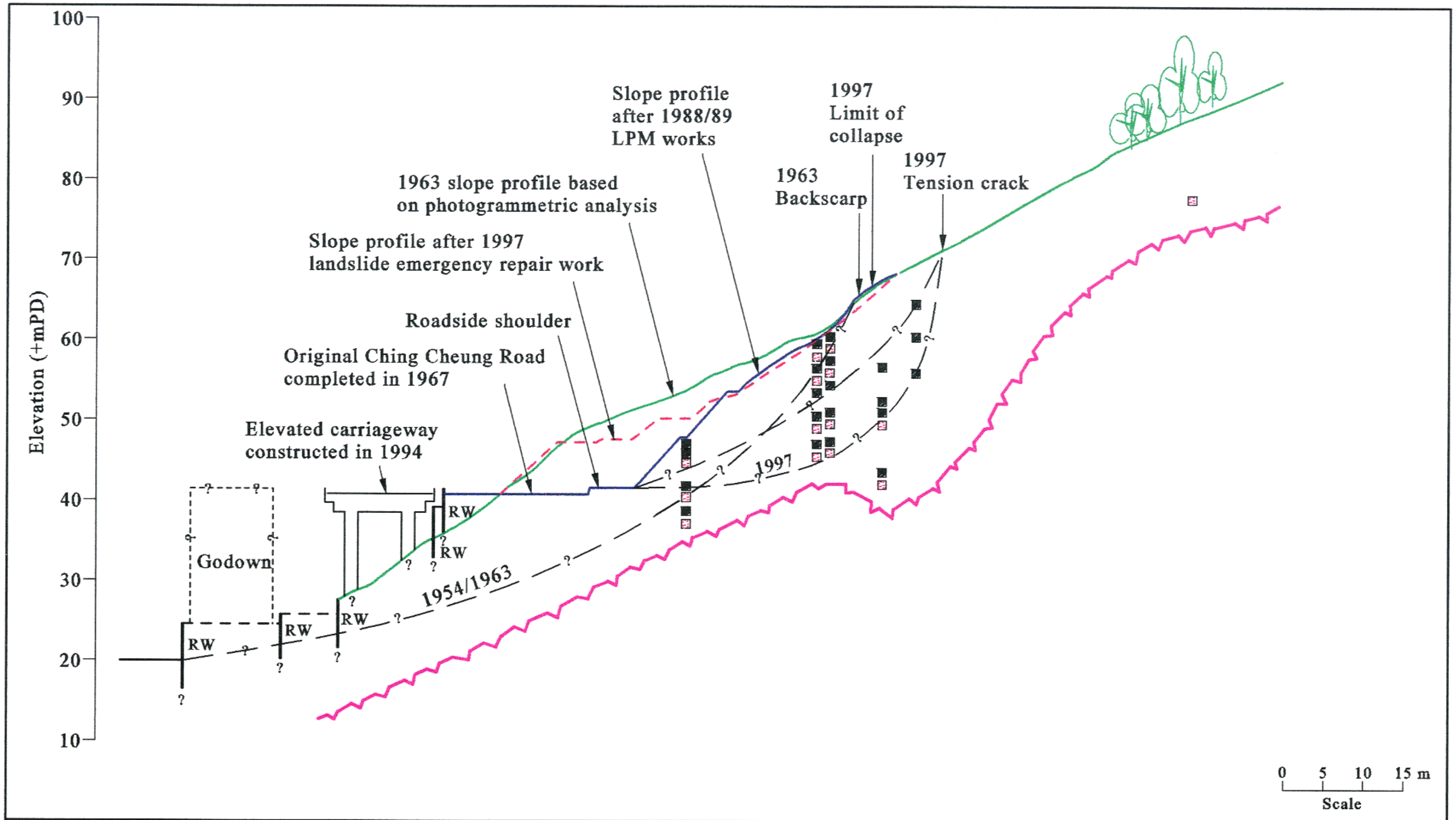
Figure 17 - Plan Showing 1954/63 Failure Scarp Superimposed on Recent Failure



Legend : ■ Pipe ▣ Disturbed material Rockhead RW Retaining wall

Notes : (1) 1988/89 slope profile extracted from 1988/89 LPM drawing.
(2) 1963 slope profile derived from photogrammetric analysis.

Figure 18 - Ground Profiles and Postulated Failure Surfaces Through Cross-section A-A for 1954 and 1997 Landslides



Legend : ■ Pipe ▨ Disturbed material Rockhead RW Retaining wall

Notes : (1) 1988/89 slope profile extracted from 1988/89 LPM drawing.
(2) 1963 slope profile derived from photogrammetric analysis.

Figure 19 - Ground Profiles and Postulated Failure Surfaces Through Cross-section B-B for 1954 and 1997 Landslides

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Plate 1 - View of 3 August 1997 Landslide (Photograph Taken on 8 August 1997)

Plate 2 - Photograph of Cut Slope No. 11NW-A/C55 (Photograph Extracted from GCD2/A1/11NW-A/C55 And Taken on 15 September 1993)

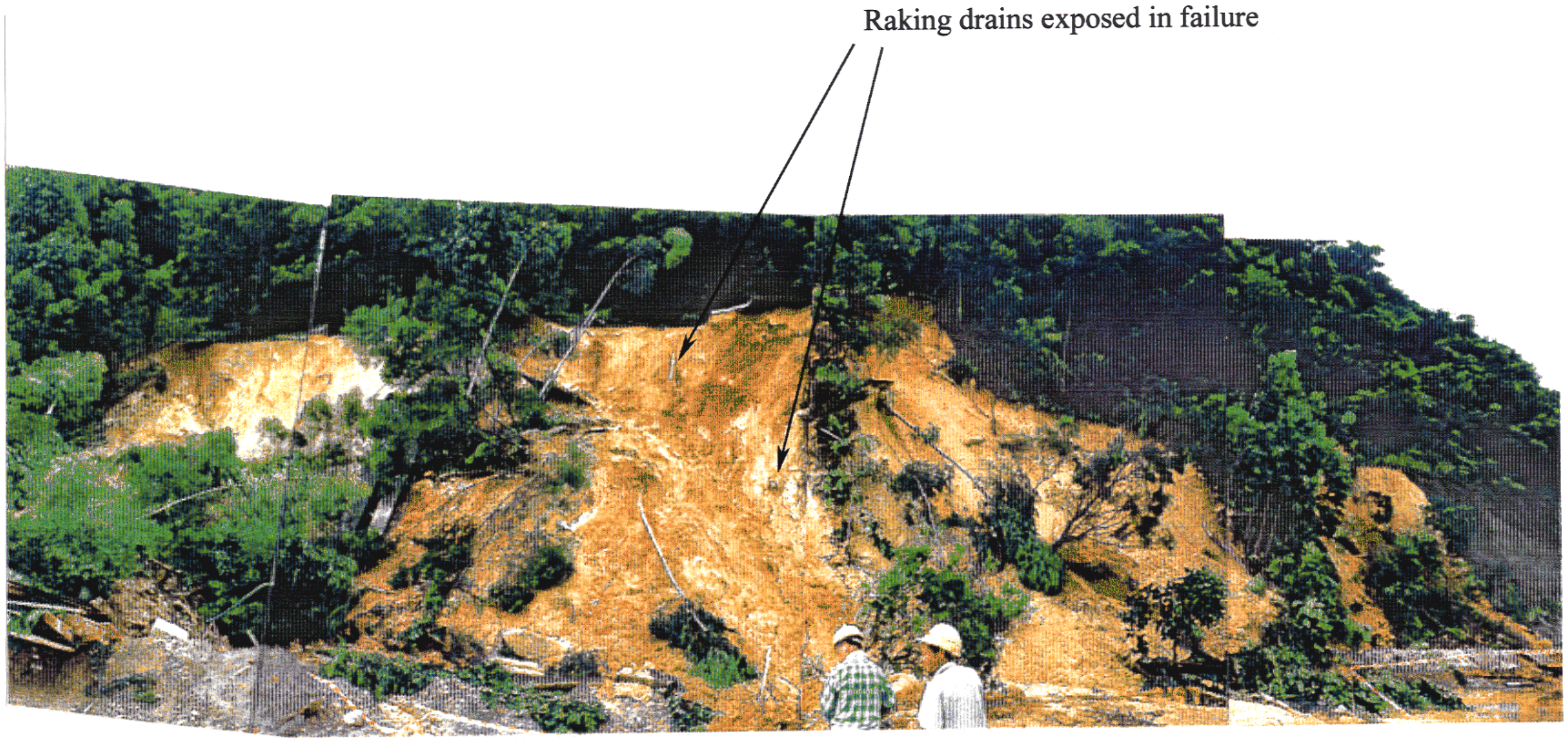


Plate 3 - Photograph of 7 July 1997 Failure (Photograph Taken on 9 July 1997)

Plate 4 - Photograph of 17 July 1997 Failure (Photograph Taken on 17 July 1997)

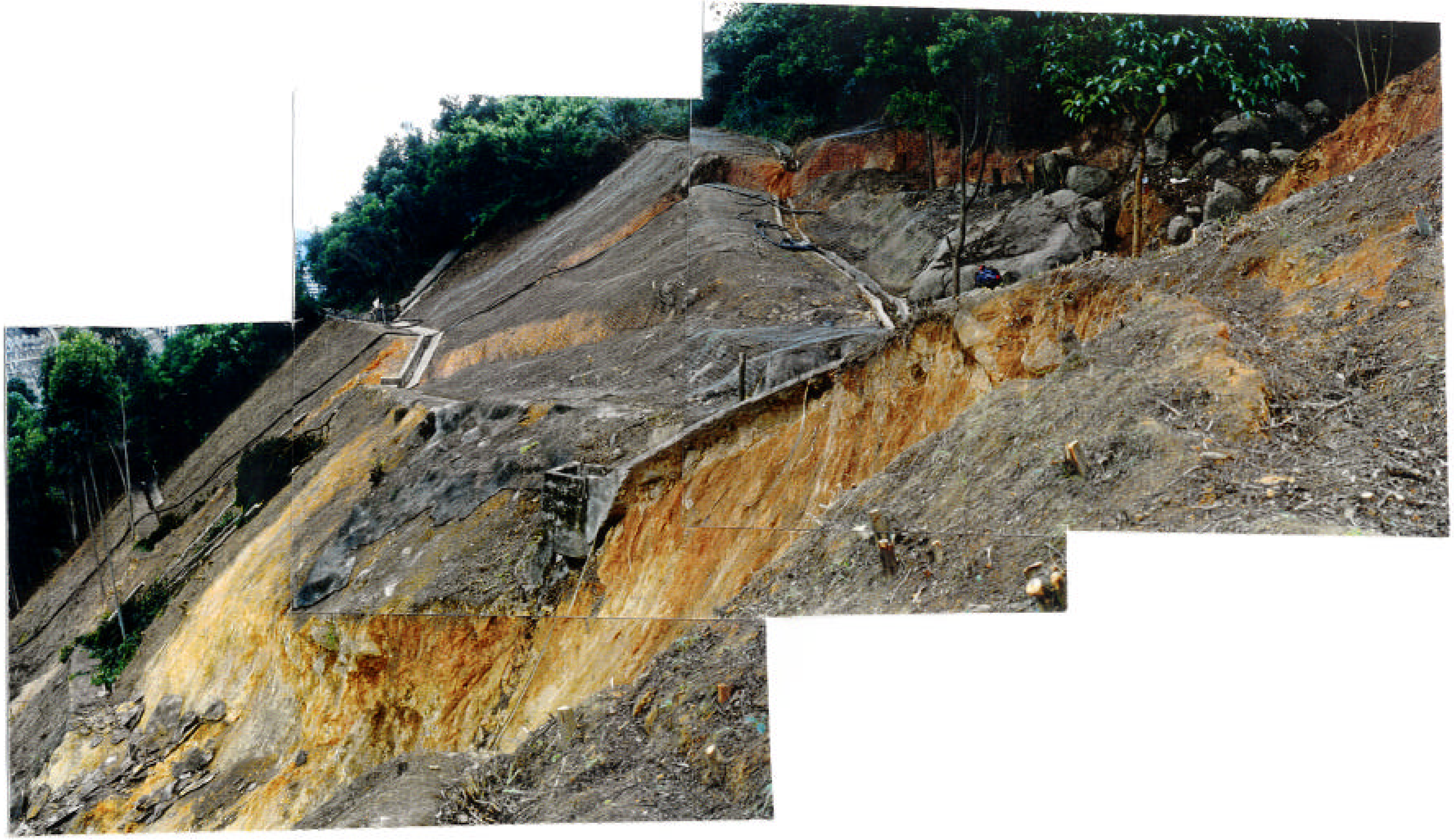


Plate 5 - View of Extensively Cracked and Displaced Ground (Photograph Taken on 30 July 1997)

Plate 6 - View of 3 August 1997 Landslide from Ching Cheung Road (Photograph Taken on 3 August 1997)

Plate 7 - View of 3 August 1997 Landslide from Cut Slope No. 11NW-A/C55 (Photograph Taken on 3 August 1997)

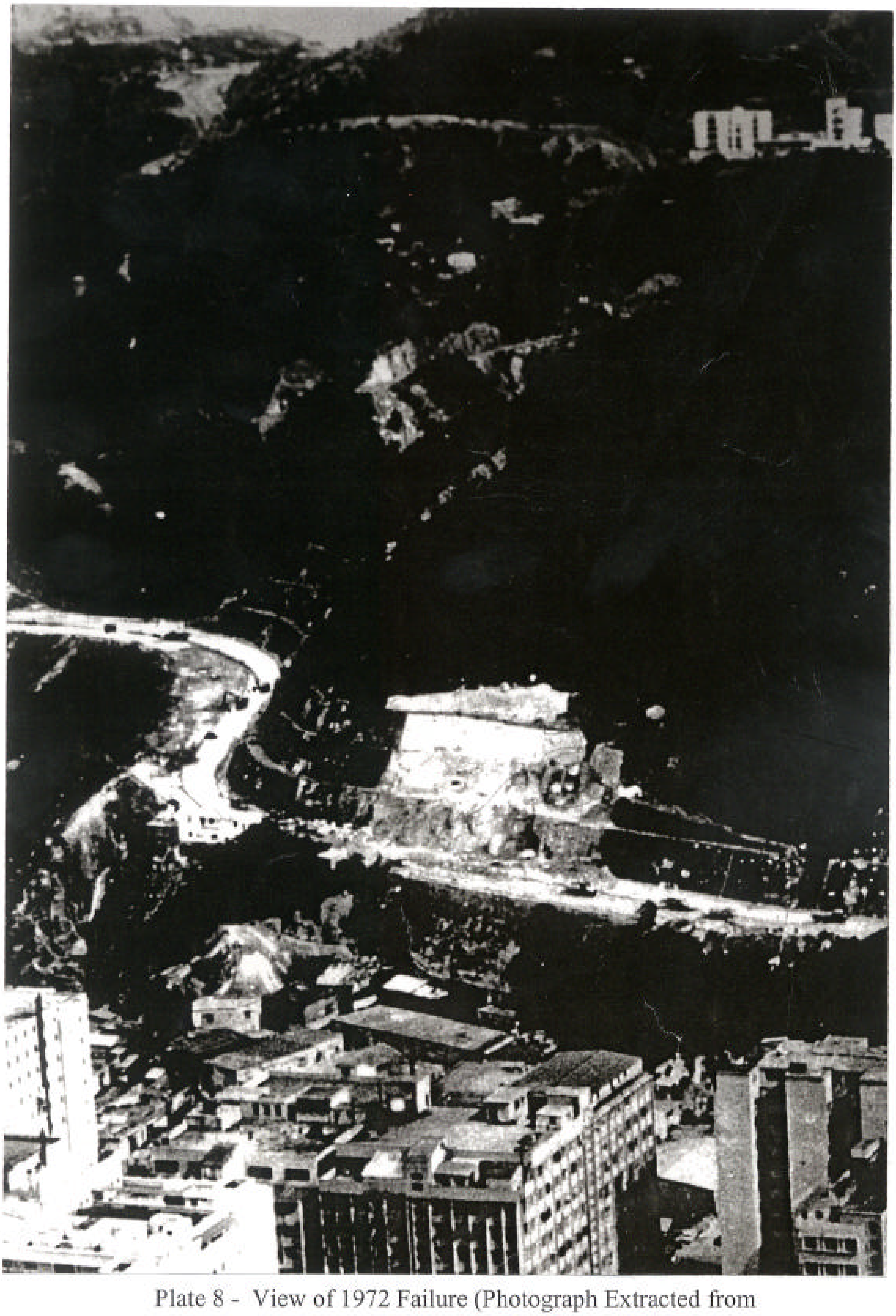


Plate 8 - View of 1972 Failure (Photograph Extracted from Figure 1 of Maunsell, 1972)

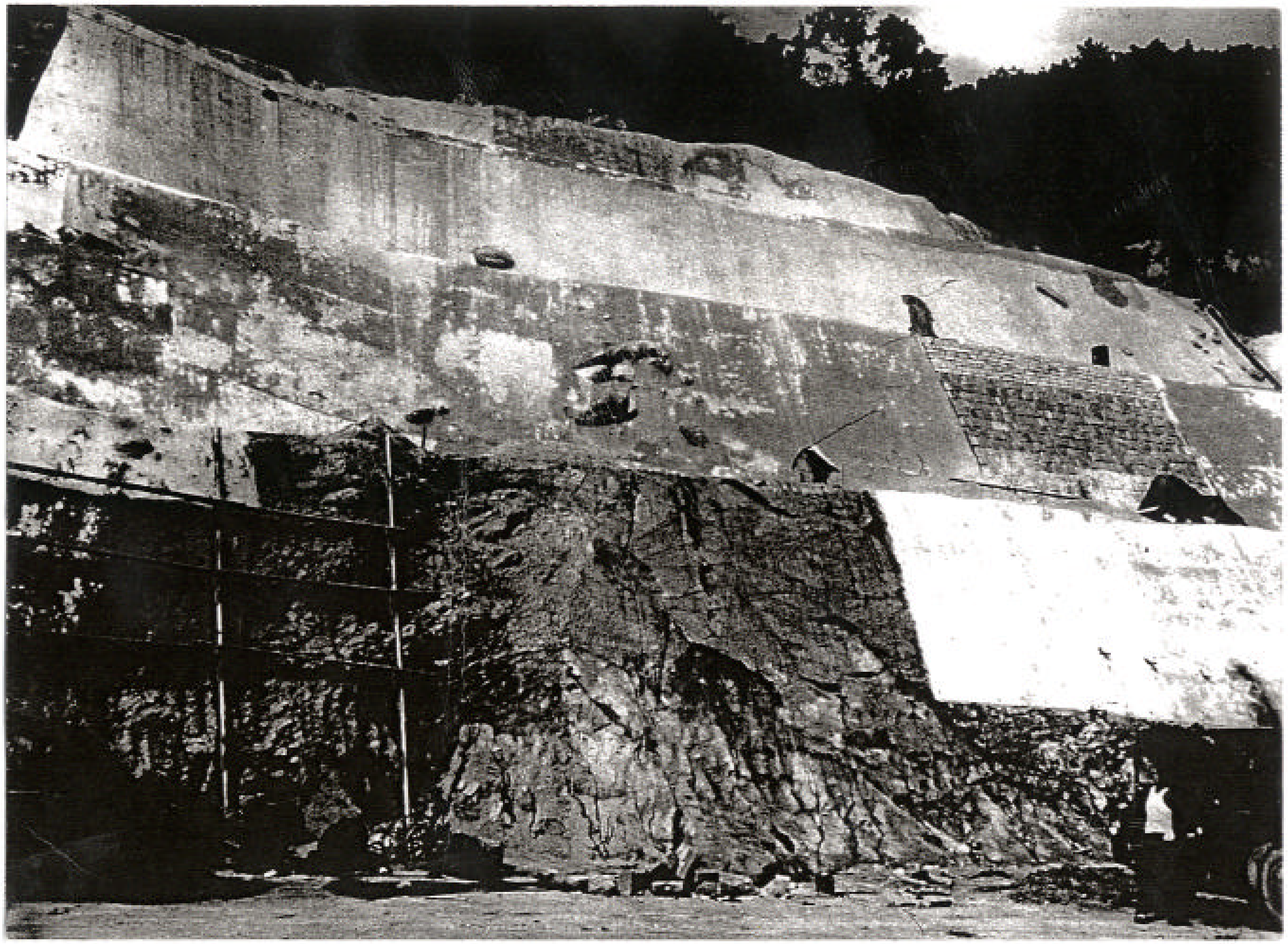


Plate 9 - View of Remedial Works Following 1972 Failure
(Photographs Extracted from Figures 2 - 4 of Maunsell, 1972)

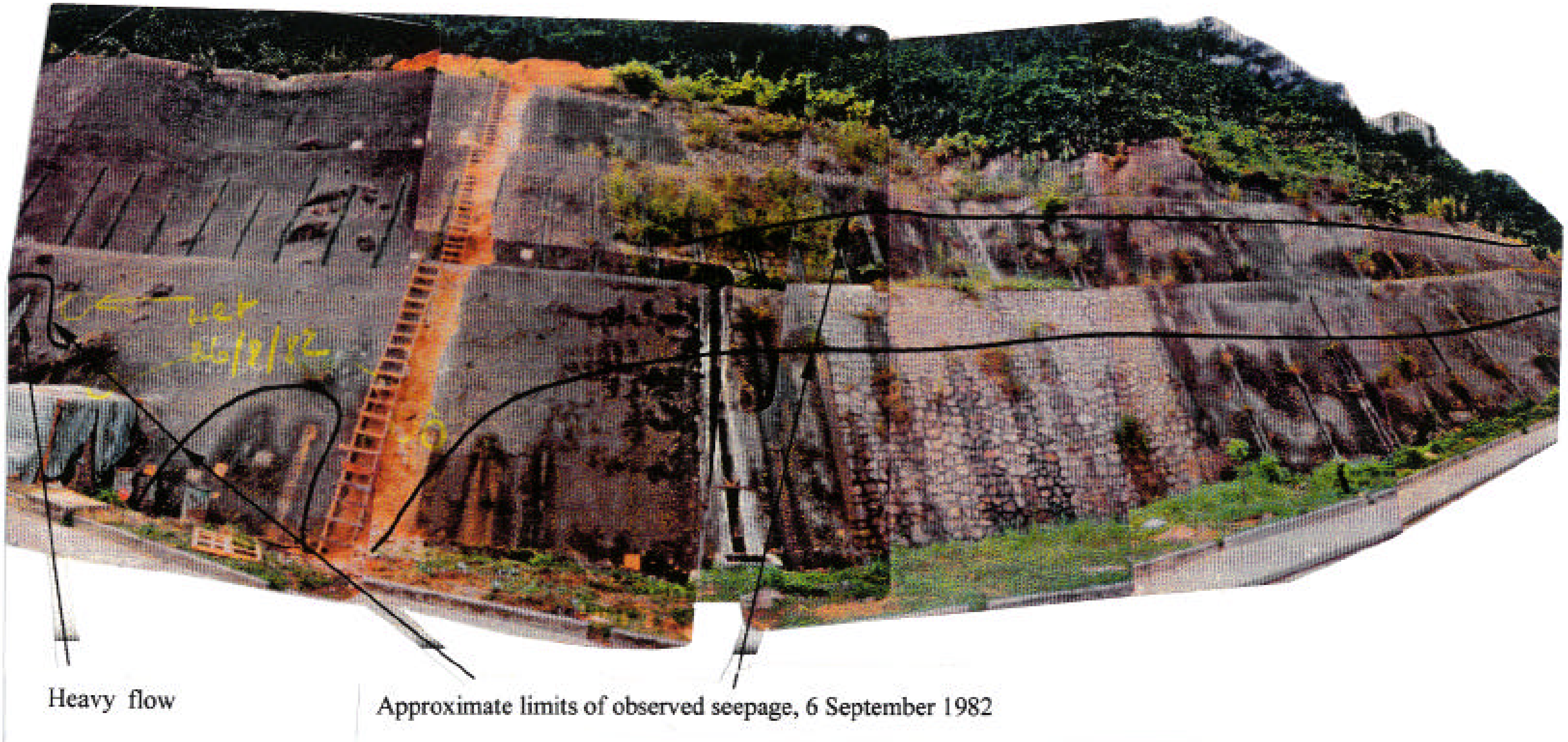


Plate 10 - View of 1982 Failure (Photograph Extracted from CHASE File, GCO, 1982b)

Plate 11 - Photograph of Central Section of Slope No. 11NW-A/C55 During LPM Works
(Photograph Taken on 4 October 1991)

Plate 12 - Photograph Showing Water Flowing from Raking Drains Installed at Mid-height of the Landslide Backscar (Photograph Taken on 8 January 1997)

APPENDIX A

SUMMARY OF SITE HISTORY

CONTENTS

A.1 AIR PHOTOGRAPH INTERPRETATION OF SITE DEVELOPMENT

References

Locations, areas and specific slopes referred to in the following discussions are shown in Figure A1.

The earliest known available aerial photographs of the site of the 1997 failure were taken in 1924. At that time, the base of the foothill into which slope Nos. 11NW-A/C55 and 11NW-A/C56 have been cut, was apparently being used as borrow areas (areas C & D).

Aerial Photo No. 5
Y00103

The next available aerial photographs taken in 1945 show that the foot of the slope Nos. 11NW-A/C55 and 11NW-A/C56 were in active excavation, the area of excavation having expanded since 1924. A large gully can be seen in area B within an area exhibiting deep rilling erosion. A major failure scarp can be seen above the borrow area D (the region of the recent failure) possibly caused by the slope toe removal.

Aerial Photo Nos.
Y00603-4 (20000')

By 1949, further erosion of the gully in area B had taken place. Excavation had continued in the borrow areas at the foot of areas C & D. However, the failure scarp identified above slope D appears unchanged from that seen in the 1945 photographs. The vertical downward movement on the scarp is estimated at approximately less than 3 m. Another scarp (partial) is seen to the east of area D.

Aerial Photo Nos.
Y01731-2 (8000')

The 1954 aerial photographs show a larger failure scarp above slope D compared to that observed in the earlier photographs. The scarp originally seen in 1945 and 1949 can be seen inside the larger failure. Surface drains had been built along the edge of the original failure scarp, and a herringbone drainage system constructed on the lower section of the slope. Buildings had been constructed at the base of the slope. Colluvial deposits can be seen in a valley above the slope.

Aerial Photo Nos.
Y02683-4

No significant changes can be seen in the 1959 aerial photographs other than an increase in vegetation cover.

Aerial Photo Nos.
Y04617-18 (40000')

The 1961 aerial photographs show that further excavation had taken place at area C and minor erosion to the northern and upslope margins of the gully at area B.

Aerial Photo Nos.
Y04905-06 (30000')

By 1963, Butterfly Valley village had been built below the major gully at B. The main gully at area B had extended upslope. Excavation appears to have ceased at area C. Finer vegetation can be seen on the western portion of area D. A very steep section at the base of the slope indicates either a minor retaining wall feature or steep cutting of the toe. The arcuate failure scarp, visible in the 1945 and later photos is still evident. There appears to have been no further

Aerial Photo Nos.
Y08751-2 (3900')
Y08101-2 (2700')

major landslide nor erosion of the feature.

The Public Works Department constructed Ching Cheung Road between 1963 and 1967 which required cutting back slopes in area A to D. The 1967 aerial photographs show cut slopes on the upper side of the road truncating the lower part of the gully at B and the landslide at D. A small cut platform was formed adjacent to the road, with no modifications on the previously failed upper section. The eastern section of the failure scarp is still visible above the road, indicating part of the previously failed mass was left in place.

Aerial Photo Nos.
Y08063-4
Y13415-6

The 1969 aerial photographs show the cut platform running along the base of the heavily vegetated upper scarp. The overall configuration of the scarp can be observed in these photos.

Aerial Photo Nos.
Y14877-8

Two sets of aerial photographs taken in 1972 show that slope No. 11NW-A/C55 had been cut back and reshaped.

Aerial Photo Nos.
2282-3 (13000')
273-4

The 1973 aerial photographs show moderately severe rilling erosion downhill below Ching Cheung Road in front of slope No. 11NW-A/C55. The photographs also show that the remedial works following a failure in 1972 had not involved removal of all of the failed mass (1945, 1954 and 1972) from the subject slope.

Aerial Photo Nos.
6888-9 (3000')

No significant changes to the subject slope or adjacent features can be seen in photographs between 1974 and 1981.

Aerial Photo Nos.
14670-1 (2000')
24127-8 (4000')
30144-5 (4000')
36762-3 (25000')
36601-2 (5500')

The 1982 aerial photographs show minor slope failure above the top berm of the slope No. 11NW-A/C55 and a larger failure in slope No. 11NW-A/C56. Two small failures are also visible at the top of the gully, at area B.

Aerial Photo Nos.
44534-5 (10000')

In the 1983 aerial photographs, remedial works can be seen to have been completed, including benching back slope No. 11NW-A/C56 to a gentler gradient, removal of the failed upper portion of slope No. 11NW-A/C55 and trimming back the upper part of the slope with the formation of two additional batters.

Aerial Photo Nos.
51667-8 (20000')

No significant changes to the subject slope or adjacent features can be seen in photographs between 1974 and 1981.

Aerial Photo Nos.
54013-4 (4000')
A06287-8(4000')
A04472-3(4000')
A095399–40 (4000')

The 1988 aerial photographs show the subject slope to be generally in a poor state of maintenance. Dark staining can be seen on the eastern section of slope (below 1st berm), and dense vegetation on

Aerial Photo Nos.
A14738-9 (4000')

the berm.

The 1990 aerial photographs show that slope works were in progress. Boulders up to 3 m and previously seen in 1949 photographs were exposed above the slope following vegetation clearance. The eastern section of the slope (between the 1st and 2nd berms) is densely vegetated.

Aerial Photo Nos.
A23578-9 (4000')
A23638-9 (4000')

The 1991 aerial photographs show the slope being substantially modified. A fence had been erected across the base of slope and there were cranes on site. The previously cleared upper eastern section of the slope is revegetated.

Aerial Photo Nos.
A27504-5 (4000')

By 1992, a surface drainage system had been constructed. The upper eastern section of slope (where boulders were previously exposed) had been regraded to a gentler angle with the removal of all the exposed boulders. The cut platform above the second berm on the eastern section had been widened. A section of the central part of the slope below the first berm had been resurfaced – adjacent slopes are all lightly vegetated. The area of slope adjacent to an access stairway on the eastern part of slope, from the third berm down to the cut platform directly above the second berm, shows evidence of some minor erosion possibly caused by water flow.

Aerial Photo Nos.
CN3085-6 (3000')
A32728-9 (4000')

The 1993 aerial photographs show the central section of slope (between 2nd and 4th berms) had been resurfaced with adjacent sections remaining lightly vegetated. This resurfacing could have been in response to a 1993 landslide failure. A section below the 1st berm (resurfaced in 1992) appears to have been freshly resurfaced in 1993. Configuration of resurfacing (strip) suggests that the slope had been recently subjected to further investigations. The base of the slope adjacent to the access stairway (between the 2nd and 3rd berms above the cut platform) shows further evidence of erosion, possibly due to increased water flow discharging down the access stairway. The base of the easternmost section of slope (below 1st berm) had been resurfaced. The overall appearance of the central section of slope is that of an area subjected to flows of water greater than its drainage network can cope with.

Aerial Photo Nos.
A36119-20 (4000')
CN4778-9 (5000')

The 1996 aerial photographs show local vegetation clearance on the upper portion of the hill above the subject slope. A new area of slope protection is observed in the lower two berms of subject slope.

Aerial Photo Nos.
CN15581-2 (5000')
CN13552-3 (4000')

A.2 PREVIOUS ASSESSMENTS

A.2.1 Slope Registration

In June 1977, the cut slope in which the 1997 landslide occurred was registered as Slope No. 11NW-A/C55 by the consultants, Binnie & Partners (Hong Kong) Consulting Engineers (B&P) engaged by Government to prepare a catalogue of cut slopes, fill slopes and retaining walls (now commonly known as the '1977/78 Catalogue of Slopes')

Binnie & Partners Field Sheet for Slope No. 11NW-A/C55

A.2.2 Landslip Preventive Measures (LPM) Programme

In August 1987, the inter-departmental Landslip Preventive Measures Committee included slope Nos. 11NW-A/C55 and 11NW-A/C56 in the 1988/89 LPM programme under the category of 'high economic risk'.

Memo M1, GEO file No. GCD 2/A1/11NW-A/C56

In 1989, the GCO completed an engineering geological study to establish the engineering geological model for the stability analysis and preventive works design, and give a preliminary assessment of the stability of the slopes with respect to discontinuity controlled failures both in saprolitic soil and the less weathered rocks.

Para. No. 4, Section 1.1 (GCO, 1989b)

A total of fourteen investigation holes with twenty-seven piezometers installed, seven vertical and six horizontal chunam strips were carried out as part of the Stage 3 Study in the period January to April 1988. (locations are shown in Figure 10 of this report).

Para. No. 1, Section 3.2.1 (GCO, 1989a)

The Engineering Geology study undertaken from January to June 1988 established an engineering model for the slope from which the following recommendations were made with respect to design of up-grading works:

Section 6 (GCO, 1989a)

(a) detailed stability analysis to take into account possible instability from relict discontinuities in the saprolitic soil portions of the slope,

(b) systematic inspection of the slopes during re-construction to identify unfavourable oriented discontinuities,

(c) the effect of basalt dykes in terms of weaker shear strengths and structural control taken into account in stability analysis, and

(d) further monitoring of piezometers to determine an accurate groundwater model for stability analysis and design. Particularly, rainstorms events to determine if intermittent perching occurred.

The study recognised that the groundwater regime was not fully understood, with descrepancies and inconsistencies in the available piezometer monitoring data and previous documented instability. For example, the report stated that "monitoring records for piezometers installed in drillholes within slope Area D show some discrepancies and, most importantly, are inconsistent in their response to rainfall events. For example, they were reported dry immediately after the relatively intense rainfall (over 154 mm) of 19 – 20 July (1988), while showing a progressive rise of water level throughout the reasonably dry months of September and October. However, there would appear to be some correlation between the readings for both piezometers in drillholes DH12 and observations of previous seepage on the slope face from both site inspections and the old photographic records." Para. No. 5, Section 4.2.3 (GCO, 1989a)

Furthermore, the study, in assessing previous instability on the slope, noted the inconsistency between the description of a "dry" failure and the adopted stabilisation measures. The report states, "the fact that the horizontal drains were effective in stabilising the slope would appear to be inconsistent with the reported "dry" nature of the June failure." Para. No. 2, Section 5.2 (GCO, 1989a)

The study identified numerous relict joints in the saprolitic zones. Results of a preliminary kinematic stability assessment of the slopes with respect to discontinuity controlled simple types of failure showed the potential for such failures was small, except in areas A2 and B, apart from toppling phenomenon. Wedge type failures however were shown to be kinematically possible in slope area B. Para. No. 5, Section 6 (GCO, 1989a)

In September 1989, the GCO prepared a Stage 3 Study Report (S3R 11/89) on cut slopes Nos. 11NW-A/C55 and C56 under the 1988/89 Landslip Preventive Measures Programme. The study comprised field inspection, engineering geological mapping of the slope faces, ground investigation, stability analyses of the slopes and design of up-grading works. (GCO, 1989b)

The Stage 3 Study report used the following strength parameters in the stability analyses: - Figures 3k and 9 (GCO, 1989b)

(a) Residual soil/Transition soil - c'=2kPa, ø' = 37°,
(b) CDAp - c'=7.2kPa, ø' = 40°,
(c) CDB - c'=10kPa, ø' = 36°,
(d) CDG - c'=9kPa, ø' = 39°, and
(e) HDG – c'=15kPa, ø' = 39°.

To establish groundwater conditions, piezometers were monitored from June 1988 to July 1989. The report states, "the Para. No. 1, Section 3.3 (GCO, 1989b) piezometric readings from all the piezometers were dry except for DH10, DH12 – DH15 in the eastern end of the slope". DH12 is located within the head of the recent failure and recorded a maximum height of groundwater of about 46 mPD, some 11 metres below ground level. For design, the maximum groundwater recorded (piezometer DH12) was adopted.

The critical, central section of the recent failure with high water level was not analysed. Section 6 – 6 did pass through the eastern side of the 1997 failed area but local ground conditions were different and less severe than of the critical section. For section 6-6 the Stage 3 Study report found that the minimum existing factor of safety was 1.14. The proposed regraded slope profile increased the factor of safety to 1.20.

Para. No. 2, Section 3.4 (GCO, 1989b)

The Stage 3 Study Report made the following recommendations to improve the stability of the existing slope to current standards:

Para. No. 1, Section 4 (GCO, 1989b)

(a) further monitoring of the groundwater levels to confirm the adopted groundwater model,

(b) cutting back of the soil portion and provision of surface drainage and hydroseeding,

(c) rock slope stabilisation works including scaling of rock blocks, installation of rock dowels, construction of concrete buttress and provision of sprayed concrete surface protection, and

(d) detailed inspection of the slope materials and reassessment of the geological model with particular reference to the possibility of potential discontinuity controlled failures.

In December 1995, consultants engaged by HyD inspected slope No. 11NW-A/C55 and reported "partly blocked" and "moderate cracking" of surface channels. It was also reported that "weepholes" were blocked with vegetation. Some minor routine maintenance works including "clear drainage channels", "repair cracked/damaged channels" and "unblock weepholes" were recommended. "Engineer Inspections carried out every 3 year(s)" were also recommended by the consultants. In the Engineer Inspection of Slope No. 11NW-A/C55, HyD's consultants did not "determine whether a Stability Assessment has previously been carried out" and did not assess "is there any change that has taken place which could have reduced the stability of the slope since Stability Assessment" as recommended in Geoguide 5 (GEO, 1995). This may be due to the fact that the terms of the Consultancy for inspections were defined before Geoguide 5 was finalised and promulgated.

Slope/Retaining Wall Record Sheets (5 sheets) and Engineer Inspection Record Sheets (9 sheets), by FMR Consultants dated 7/12/1995

In the 1996/97 LPM selection exercise, HyD nominated 1010 roadside slopes (including slope No. 11NW-A/C55 at Ching Cheung Road which failed in 1997) for consideration by the interdepartmental Landslip Preventive Measures Committee (LPMC) for inclusion in the 5-year Accelerated LPM Project. The slope No. 11NW-A/C55 was not selected by the LPMC because previous LPM actions had been undertaken on the slope.

HyD's memo to GEO dated 20/5/1996; LPMC meeting held on 8/1/1997

A.2.3 Previous Landslides Affecting Slope No. 11NW-A/C55

As noted elsewhere in this report, the slope had been the location of landslides before the construction of Ching Cheung Road. Following construction of the road, three additional landslides occurred in 1972, 1982, and 1993. The approximate locations of these landslides are shown in Figure A1.

The 1972 landslide occurred two days after the end of a major rainstorm and involved failure of about 7500 m^3 of soil on to the existing road and blocked one lane of Ching Cheung Road.

(Maunsell, 1973a)

Following chunaming of the newly cut face of the slope, further movements were observed and measured. Movements appeared to have ceased after installation of horizontal drains into the lowest two batters of the entire re-constructed slope. The maximum recorded rate of outflow from any one hole was about 10 gallons per hour.

During works following a landslide in the adjacent cut slope (11NW-A/C56) in 1982, tension cracks were observed at the crest of the cut slope No. 11NW-A/C55 and the condition of the slope was observed to be deteriorating.

GCO's memo to HyD dated 3/9/1982

Recommendations were made for the slope to be cut back by 3.8 m, for the entire slope to be resurfaced and for the existing raking drains to be maintained or replaced, as necessary.

GCO's memo to HyD dated 30/9/1982

A landslide occurred in July 1993 on the third batter of the slope and was inspected by GEO . The failure volume was estimated at less than 9 m^3. The landslide debris came to rest partly on the slope berms and in the open space in front of slope. Few other data were recorded. The incident report stated that "infiltration is the possible cause of failure" and it was recommended to trim the failure surface and provide surface protection (with weepholes).

GEO incident report dated 27/7/93

A.3 REFERENCES

Geotechnical Control Office (1989a). Engineering Geology Study of Slopes 11NW-A/C55 & C56, Ching Cheung Road. Geotechnical Control Office, Hong Kong, Advisory Report no. ADR 1/89, 199 p. plus 9 drgs. (Unpublished).

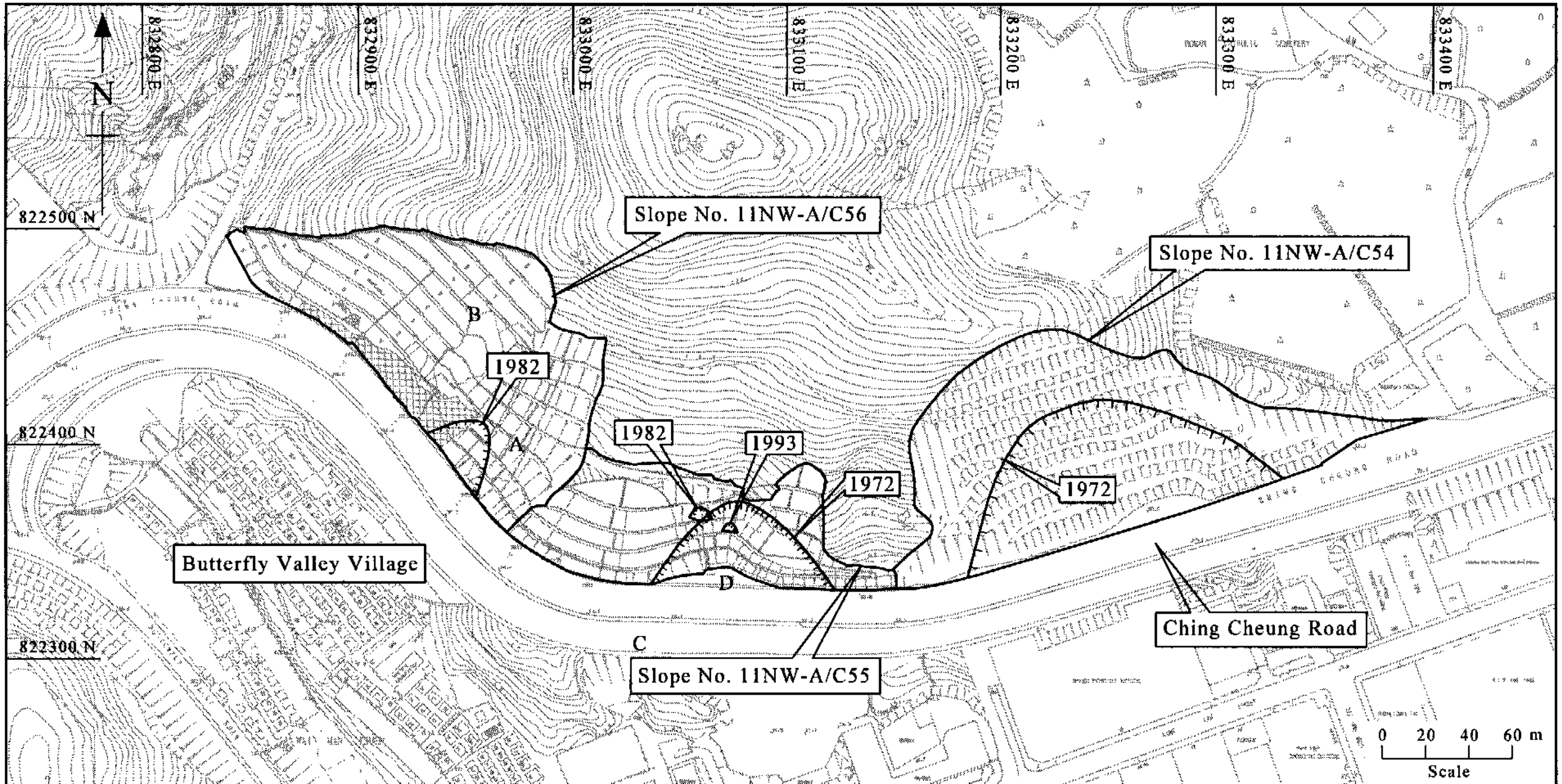
Geotechnical Control Office (1989b). Cut Slopes 11NW-A/C55 & C56, Ching Cheung Road. Geotechnical Control Office, Hong Kong, Stage 3 Study Report no. S3R 11/89, 80 p. plus 8 drgs. (Unpublished).

Geotechnical Engineering Office (1995). Guide to Slope Maintenance (Geoguide 5). Geotechnical Engineering Office, Hong Kong, 92 p.

Maunsell Consultants Asia (1972). Interim Report on Remedial Works for Landslides on Ching Cheung Road. Report to Public Works Department, Hong Kong, 8 p. plus 3 drgs. (Unpublished).

Maunsell Consultants Asia (1973a). Report on Landslides on Ching Cheung Road. Report to Public Works Department, Hong Kong Government, 34 p. plus 4 Appendices and 19 drgs. (Unpublished).

LIST OF FIGURES



Legend : Landslide scar (with year)

Notes : (1) The location of landslide scars are approximate. The 1972 landslide scars are indicative only.

(2) Locations of landslides pre-dating road construction are omitted.

(3) Location A, B, C, and D are referred to in Aerial Photograph Interpretation of Site Development, Appendix A.

Figure A1 - Location of Past Landslides on Slope No. 11NW-A/C55 and Adjacent Cut Slopes

一九九七年八月三日
呈祥道山泥傾瀉事件報告

合樂亞洲顧問公司

序言

為配合本處公開大眾及岩土工程業人士感興趣的資料的政策，我們把部份內部報告公開予公眾參考，並訂名為土力工程處報告系列。報告的售價將用作補助印製成本。

土力工程處又出版指引文件，成為土力工程處刊物系列。政府新聞處負責銷售這些刊物和土力工程處報告系列，購買詳情載於本報告的末頁。

陳健碩　陳健碩

土力工程處處長
陳健碩
一九九八年十一月

前 言

這份土力工程處報告為一九九七年山泥傾瀉顧問調查的一部份，當中載述了土力工程處委託的合樂亞洲顧問公司(HAP)就一九九七年八月三日的呈祥道山泥傾瀉事件而進行的詳細調查。

上述的山泥傾瀉顧問調查，旨在透過對山泥傾瀉事件的檢討和研究，達至以下的目的：

(a) 確立更佳的斜坡評估方法；

(b) 鑑別需予採取跟進行動的斜坡；及

(c) 就政府的斜坡安全系統以及本港現行的岩土工程作業提出改善建議。

土力工程處轄下的山泥傾瀉勘測部與山泥傾瀉調查顧問公司緊密合作，並對呈祥道傾瀉事件之詳細調查提供技術支援和協助。

此外，本報告又對一九九八年二月出版的一九九七年呈祥道山泥傾瀉調查報告作出了若干校訂，並增錄了航空測量分析結果。

鄧滿祥

土力工程處副處長/防止山泥傾瀉
鄧滿祥
一九九八年八月

撮 要

一九九七年八月三日，鄰接呈祥道的削土坡 11NW-A/C55 發生山泥傾瀉。 瀉下的泥石完全掩埋了呈祥道一段 50 米長的路面，並困住了一輛沿呈祥道西行的私家車。 私家車司機沒有受傷。 此次山泥傾瀉是編號為 11NW-A/C55 的削土坡，自一九九七年七月七日起開始的漸進破壞和崩塌過程的最後階段。 選擇本次山泥傾瀉進行詳細調查，部份是由於它的規模、後果和過去的不穩定記錄，同時也因為此削土坡已於一九八九年的防止山泥傾瀉計劃按照現行工程標準進行過勘察、設計及加固工程。

於一九九七年八月至一九九八年二月期間合樂亞洲顧問公司（HAP）作為土力工程處的山泥傾瀉勘察顧問，就這次山泥傾瀉展開了全面的調查。 詳細調查工作包括翻查資料文件、分析雨量記錄、訪問山泥傾瀉目擊人士、在事發現場進行現場調查、場地勘探、以理論方法進行穩定性分析以及判斷崩塌的成因。

調查所得結論是：一九九七年八月三日的山泥傾瀉可能是由於削土坡最初於一九九七年七月七日發生崩塌後，斜坡狀態不斷惡化所致。 那次崩塌破壞了地面排水渠及導致地面移動，使得日後連場豪雨期間的地面水滲入削土坡內，並導致了後來的幾次崩塌事件。

調查確認，自一九二四年以前直到一九五四年，位於事發地點下面斜坡上的採石活動引起了最初的大規模崩塌；此後，斜坡曾有一段很長時間的不穩定歷史。 在斜坡後來的發展變化過程中崩塌體一直大面積的保留在原地，而且至今仍實際存在。

一九九七年山泥傾瀉的發生地點同從一九五四年及一九六三年的航空照片上看到的崩塌在平面位置上大體一致，故先前崩塌可能促進了此次山泥傾瀉的發生。 然而，調查結論認為，引發這次山泥傾瀉主要原因是由於崩塌前持續下了極其大量的豪雨而產生的不利地下水條件所致。

在崩塌前，斜坡內，特別是崩塌斜坡頂部恰位於一條天然排水溪下面，可能形成了一個瞬態的高上層滯水位。 地下水的天然流徑似乎主要受一套天然管道所控制；這些天然管道部份受位於花崗岩內的風化玄武質岩脈所控制，但也許沿先前崩塌擾動帶而形成。 一九七二年崩塌後裝設用以減低地下水壓力的排水導管，此時無法防止臨界地下水壓的產生，因而導致一九九七年山泥傾瀉的發生。

山泥傾瀉調查的詳情及所得的結果載列於本報告內。

目 錄

1. 引言

一九九七年八月三日星期天上午十一時十五分，九龍蝴蝶谷東側呈祥道路邊削土坡發生山泥傾瀉（圖1）。 瀉下的泥石完全掩埋了呈祥道一段 50 米長的路面（照片1），並困住一輛沿呈祥道西行的私家車。 私家車司機自行逃脫，沒有受傷。 呈祥道隨後封閉，進行清理泥石及斜坡的臨時加固工程。

土木工程署轄下的土力工程處於事發後，組織策劃了對這次山泥傾瀉事件的詳細調查。 作為土力工程處的山泥傾瀉勘察顧問，合樂亞洲顧問公司(HAP）具體承擔了這次調查工作。

調查工作於一九九七年八月至一九九八年二月進行，主要項目如下：

(a) 就事發地點翻查所有已知的資料文件及航空和地面照片;

(b) 於事發地點進行詳細的觀察及量度;

(c) 訪問目擊山泥傾瀉的人士;

(d) 收取山泥傾瀉事件的詳細資料，重建其發生的經過;

(e) 分析事件發生時的雨量記錄並與歷史記錄進行比較;

(f) 進行地質勘察;

(g) 採用鑽探、試井、現場試驗、取樣本及記錄進行全面的場地勘探以確定事發地點的地下情況;

(h) 建立出事斜坡的水文地質及地質模型;

(i) 評估事發地點土壤的工程性質;

(j) 對事發斜坡和附近地區的過去不穩定斜坡進行評估及反饋計算;

(k) 以理論方法分析崩塌斜坡在不同條件下的理論穩定性和地下水情況; 以及

(1) 總結導致山泥傾瀉發生的可能原因。

本報告載列這次調查的結果。 調查工作的詳情及所得的結果則載於一套文件中，該套文件存放於土木工程署大樓地下一樓土木工程圖書館內。

2. 事發地點的描述

山泥傾瀉發生的地點位於九龍長沙灣蝴蝶谷東側呈祥道上路邊登記編號為11NW-A/C55 的一幅削土坡的中部（圖1）。

削土坡作為呈祥道建築工程的一部分，最初於一九六七年建成。 削土坡主要由強風化花崗岩組成，西側下部局部露出較堅硬岩石。 削土坡中部處於一條天然山體排水溪下面（圖2）：一九七二年崩塌發生後進行的斜坡加固工程，在這部分坡面上形成了一處局部的地面凹陷。 削土坡的東西部分削製至天然山咀。

近期的崩塌均發生於天然山體排水溪下面的削土坡內，崩塌所涉及的削土坡全高約 35 米，最闊達 52 米。

近期的崩塌發生前，削土坡由二至六個傾斜面組成，總垂直高度從 12 米至 47 米不等。 削土坡仰角各處不同，變化範圍一般為 45 度至 50 度。 削土坡表面長有大量植物，並有局部的噴射混凝土護坡。 一九九三年九月十五日拍攝的照片顯示了削土坡中部的情況（照片2）。

削土坡上面的天然山坡上長有茂密的樹木和灌木，其仰角約為 26 度，高程由削土坡頂部的 80 mPD 一直增至到山頂的 150 mPD。 在東側，削土坡的底部傾斜面與另一幅較大削土坡（登記編號為11NW-A/C54）鄰接。 兩個削土坡被一條深切長滿植物的天然排水溪分隔開(圖1）。 在西側，削土坡與編號為11NW-A/C56 的削土坡鄰接。

編號 11NW-A/C55 削土坡的地面排水系統由位於坡台上的排水渠和坡面上的梯級渠構成。 斜坡地面水匯流入呈祥道路邊的排水渠。 在天然山體排水溪及中部削土坡的交界處，山體排水由截水牆及排水井與斜坡排水系統相連接而成。在一九七二年的山泥傾瀉之後，在斜坡的下部兩個傾斜面中安裝了一套地下排水導管（詳見本報告第四章）。

呈祥道沿斜坡坡腳延伸，高程約為 33 mPD。 在近期的山泥傾瀉發生期間，因路政署（HyD）的"呈祥道和龍祥道改善工程"正在施工中，呈祥道位於斜

坡前面的部分也因此被封閉。 所有車輛沿呈祥道南側平行延伸的新近建成的四車道架空道路改行（圖2）。

3. 山泥傾瀉的描述

發生於呈祥道路邊編號 11NW-A/C55 削土坡上的山泥傾瀉，涉及三次規模不斷增大的漸進崩塌。 第一次崩塌發生於一九九七年七月七日，接著於七月十七日發生了規模較大的第二次崩塌，主要崩塌於一九九七年八月三日發生。 對這三次崩塌的描述是根據目擊人士的陳述和資料文件作出。

第一次崩塌發生於一九九七年七月七日上午九時，塌下的泥石（土）體積約為 500 立方米，崩塌的地點為削土坡中偏東部份的下部（圖2）。 塌下的泥石覆蓋了呈祥道的封閉段路面（照片3）。 觀察到的崩塌殘痕只出現在斜坡最下部兩個傾斜面內，崩塌影響範圍為呈祥道上面一幅 52 米長、 15 米高的斜坡。 但是，由於當時削土坡的上半部份的茂密植被，這些植被令到崩塌上部的斜坡難於視察。 暴露的破裂面很淺，且深度不等，但一般不超過 2 米。 塌下的泥石主要來自位於削土坡凹陷部分的崩塌西半部，崩塌東部一般僅為小型的地面破裂。 崩塌的東部邊界為一條侵蝕沖溝，此沖溝位於一個有多條排水渠匯流的排水井下面。

第二次崩塌發生於一九九七年七月十七日大約下午三時，塌下的泥石估計為 700 立方米，崩塌的地點為緊接第一次崩塌地點上部的斜坡凹陷部分（圖2和照片4）。 塌下的泥石再一次瀉入呈祥道封閉段路面。 崩塌產生了兩個相互分離的崩塌殘痕，高度達呈祥道以上 30 米並影響到一幅約 30 米闊的坡面。在天然山體排水溪的下面，那較大的崩塌殘痕形成了一個約 15 米闊的碗型凹地。估計凹地的最大豎向深度為 5 米。 在其西側有一個形狀相似而較小碗型凹地。在較大崩塌殘痕的頂部有一條排水渠被切斷，此排水渠是其上面一個排水井的出水口；有多條地面排水渠都匯流於此排水井，與其中一條同被截斷的天然排水溪相連。 崩塌殘痕底面的較大侵蝕沖溝，顯示曾有地面水流。 在較小的崩塌殘痕頂部也有一條排水渠因崩塌而切斷。

一九九七年七月二十九日，清除了斜坡上的植被之後，在位於先前崩塌位置上面的斜坡上，發現了一個已產生大範圍裂隙和位移的變形區域（圖2 和照片5）。 這區域包含了前期的崩塌殘痕，在下部傾斜面上的橫向伸展範圍與七月七日崩塌的影響範圍一致。 有變形跡象的地面僅限於此削土坡範圍內，面積約為 1800 平方米，包括天然排水溪以下寬度達 52 米的整幅削土坡。

沿變形區域的周界，有一條大型的張裂隙(圖2)。 一九九七年七月三十日沿此張裂進行的地面位移量測顯示，向下的最大豎向位移量達 2.5 米，向坡下

的最大水平位移量達 1.0 米。 最大位移量發生於張裂隙的上段邊緣(照片5)。沿張裂隙的西段和北段邊緣，張裂隙面看來為新鮮。 然而，一九九七年七月三十日的發現可能預示了崩塌的一個較早期階段位移。 張裂隙內局部充填鬆散表土，東段的暴露張裂面上有銹跡；這些情況顯示此張裂隙可能已存在了一段時期。 另外，在張裂隙的西上段範圍內，有一處小面積的噴射混凝土護坡，它覆蓋著一處位於一個小的平緩陡坡上面的變形坡台。 這再一次顯示此張裂隙可能已存在了一段時期。本報告的後部將會討論到於山泥傾瀉後進行的場地勘探，及發現此斜坡其他先前位移的証據。

第三次崩塌導致呈祥道被完全堵塞。根據山泥傾瀉目擊人士的描述，山泥傾瀉發生於一九九七年八月三日上午十一時十五分。照片6 是在山泥傾瀉發生後幾小時，那時正在進行現場清理時拍攝的。 這次塌下的泥石體積估計有 2000 立方米。

第三次崩塌始自變形區域的西上部份，其範圍僅限於前述周界張裂隙內。崩塌形成的凹地達 7 米深 20 米闊，延伸到呈祥道以上 30 米高的削土坡，並將前期形成的崩塌殘痕加深和侵蝕。橫貫崩塌殘痕的典型剖面載於圖3。 如照片 7 所顯示，塌下的泥石覆蓋了呈祥道封閉段路面和與其鄰接的新近建成的四車道架空道路，並瀉入南側護欄。 路面上的泥石堆最闊估計不超過 50 米，最厚約為 3 米(照片6)。 由崩塌殘痕的頂部開始量度，泥石所移動的平面距離最遠約為 78 米。

一輛沿呈祥道西行的私家車，被瀉下的泥石圍困並被推至南側護欄(圖2)。 司機自行逃離私家車，並沒有受傷。 在清理泥石和進行緊急加固工程期間，呈祥道一直被封閉。在此期間，崩塌頂部產生了進一步的位移，並有少量泥石落下。 呈祥道於一九九七年八月二十四日重新開放。

塌下的泥石主要為褐色至淺褐色和紅褐色的礫質粉質黏質砂，並有少量卵石和極少量中等風化的花崗岩漂石。 當中並混有人造物料，包括建築材料，如混凝土塊、水管、腳手架，堆積在呈祥道封閉段路面。

量度泥石末梢至崩塌殘痕頂部的仰角，可反映塌下泥石的流動性(見圖3)。 角度越小，泥石流動性越大。 根據 Wong & Ho(一九九六年)，對典型由豪雨造成的削土坡山泥傾瀉而言，角度一般為 30 度以上。但在呈祥道一九九七年八月三日的山泥傾瀉事件中，現場量測的角度為 20 度，顯示泥石的流動性較一般削土坡崩塌為高。 流動性較大的原因可能與呈祥道封閉路段的路面積水有關。

4. 事發地點的場地歷史

撮述於附錄A的場地歷史，是透過研究該處的航空照片及翻查現有資料文件而得。

有關事發地點的現有最早航空照片，拍攝於一九二四年。 照片顯示一九九七年山泥傾瀉地點為一取土區。取土區底部高程約為 10 mPD，比現有呈祥道高程約低 23 米。 接下來所搜集到的航空照片，拍攝於一九四五年。 照片顯示取土區範圍已擴大，並且仍在使用。 從照片上可清楚地看出位於取土作業面上面的一個崩塌崖，滑下的泥石則堆積於崩塌崖下面的取土區頂部。一九四九年拍攝的照片顯示，崩塌崖沒有任何改變，既無重大的侵蝕痕跡，也看不到曾發生過新位移的証據。

到了一九五四年，斜坡再一次發生崩塌。崩塌的擴大可能是由於取土活動對天然斜坡的過度開挖引起。 本報告後部將討論從一九五四年的航空照片上確認的崩塌崖，在平面上部份與一九九七年山泥傾瀉所形成的周界張裂隙很一致。一九六三年的航空照片上看不出有任何進一步滑坡的跡象。 那時坡腳處已建起了房屋，橫貫崩塌面也已建成地面排水渠。 一九六三年至一九六七年期間，公務署修建呈祥道時，在一九九七年山泥傾瀉地點建成了這幅削土坡。很明顯地，呈祥道穿越此古滑坡體，部分削土坡剛好處於牽引滑坡體內，一九四五年至一九六三年拍攝的航空照片上均可看到此滑坡體。

一九七二年呈祥道拓寬工程期間，削土坡發生了一次大型山泥傾瀉（照片8）。 崩塌的體積大約為 7500 平方米(茂盛，一九七三a)。 修葺工程包括削制斜坡至其原坡度，然而在此期間斜坡又產生了進一步的位移（照片9）。 其後的監測錄得平面位移量達 21 毫米，主要產生位於一九九七年崩塌中部的最下一個傾斜面上。 在最下部兩個傾斜面安裝了 151 根 16 米長的導水管之後，斜坡最終穩定下來。 據茂盛顧問公司報導(一九七三b)約 70% 至 80% 多的導管在安裝後就持續流水，崩塌區中部的導管尤其明顯；顯示導管可有效截斷地下水。 測得的最大單管流量為每小時38升。其後十年，斜坡無明顯不穩現象。

一九八二年八月，一場豪雨之後，鄰接編號 11NW-A/C56 斜坡發生了一次大規模山泥傾斜，這次崩塌涉及多於 1500 立方米的泥石。 編號 11NW－A/C55 削土坡沒有發生大的崩塌，只在天然排水溪西側頂部坡台以上的削土坡中上部坡面上有一處很小的表面崩塌（照片10）。 從一九八二年山泥傾斜發生後所拍攝的照片中，發現大部份在一九七二年安裝在削土坡中部的排水導管中有水流出。將一九八二年八月所攝照片同其後一個月的照片比較，顯示出從排水導管流出的水有所減少，這証明削土坡在大雨後有瞬變反應。

土力工程處對編號 11NW-A/C55 削土坡在發生小規模崩塌後建議的善後工程包括，將整個斜坡後削 3.8 米，然後檢查在一九七二年安裝的排水導管的有效性，必要時，清洗、沖洗或安裝新的導管。 航空照片顯示削土坡工程後斜坡上部新建有兩個傾斜面，但削土坡下部沒有重整。 沒有任何記錄顯示排水導管有被改善或進行過任何維修。

防止山泥傾瀉委員會（LPMC）在一九八七年八月將此斜坡及鄰接編號 11NW-A/C56 的斜坡列入一九八八年/一九八九年度防止山泥傾瀉計劃（LPM）內。 土力工程處對斜坡進行了工程地質研究（GCO，1989a），包括場地勘探鑽孔、探井和坡面剝除；在此基礎上，進行了穩定性評估以及斜坡改善工程的設計(GCO，1989b）。 對位於近期崩塌地點西側和東側的削土坡部份進行了穩定性分析，得出的結論是編號 11NW-A/C55 削土坡沒有足夠的安全系數。 在設計報告中(GCO，1989b)沒有對一九九七年山泥傾瀉地點所在的削土坡部分進行過穩定性分析。

編號 11NW-A/C55 削土坡的改善工程包括向後削制斜坡的西半部分，清除東半部份下部坡面上的崩積物，並透過輕微削制工程在坡頂新建傾斜面。 這些改善工程在一九九零年至一九九二年進行，削土坡中段包括一九九七年崩塌地點的東部及中部沒有進行任何改善工程。 為了檢查坡面下的土體性質，剝除了此段斜坡上原有硬質護面和排水導管表面；並指明應將整幅斜坡噴植草。 防止山泥傾瀉工程期間拍攝的照片顯示，在最低傾斜面的下部坡面上有噴射混凝土護坡；這是對工程進行期間發現的坡面侵蝕現象而採取的防護措施。 在最低傾斜面與第一級坡台間的坡面上也有噴射混凝土護坡(照片2)。 而不是原先規定的噴植草。

在一九九二年三月五日防止山泥傾瀉（LPM）工程雖已驗証完工，但地盤檔案記載在這次一九九七年崩塌範圍內仍有數項工程進行而直至一九九二年五月十五日才完工，一九九二年五月十九日發現兩處地下水滲流，其位置在一九九七年崩塌範圍內的削土坡上(照片11)。 隨後對削土坡進行了穩定性分析。 分析時假定在每個滲流處的上面有一薄層上層滯水，斜坡的安全系數大過 1.2，因此決定不需任何改善工程。

在防止山泥傾瀉工程完成後，斜坡的第三級傾斜面上，在一九九三年發生了一次小型崩塌，其位置剛好在一九九七年崩塌的範圍內。 土力工程處調查了這次崩塌，並建議以噴射混凝土防護崩塌殘痕。 從一九九三年至現在該斜坡沒有大的變動。 土力工程處於一九九三年九月以及路政署的顧問工程師為路政署於一九九五年十二月對斜坡進行調查時，發現疏水孔被植物部份堵塞，地面排水渠中也產生了中等程度的裂隙和堵塞。 路政署的顧問工程師建議一些常規的維修包括清理排水渠，修理破裂/破壞的排水渠及清除淤塞的疏水孔及對斜坡每三年進行一次檢查。 路政署在一九九七年一月完成了常規維修工作。 根據一九九

五年調查中的發現，路政署在一九九六年五月向防止山泥傾瀉委員會建議將此斜坡與其它 1009個斜坡一起列入一九九六年度防止山泥傾瀉計劃中。 然而，因為此斜坡剛於近期完成了防止山泥傾瀉工程，所以並沒有被選中。

在編號 11NW-A/C55 削土坡北面建造通往蝴蝶谷主配水庫的通道在一九九七年六月底開始(圖1)，在山坡北面的清除植物工程在一九九七年七月初開始。

圖4至圖6展示了從一九八八年至一九九七年期間，呈祥道路邊削土坡的發展變化情況。

5. 雨量記錄分析

雨量數據來自土力工程處在山泥傾瀉現場附近設有的兩個自動雨量計 K06 及 N04，編號 K06 雨量計位於蘇屋村石竹樓，離現場以東約 1.2 公里，編號 N04 雨量計位於祖堯村啟光樓，離現場以西北約 1.7 公里。雨量計每五分鐘記錄一次雨量數據，同時將數據通過一條電話線傳送給香港天文台和土力工程處。

該兩個雨量計在山泥傾瀉前所錄得的雨量變化和強度，大致相若。 因編號 K06 雨量計所錄得的雨量稍大，因此用作雨量分析。

雨量計 K06 於一九九七年七月一日至八月五日期間錄得的每小時雨量數據及每次崩塌的發生時間均載於圖7。 七月的前四天錄得的雨量頗大，其後的七月十五日至十九日期間和八月一日至三日期間，則有階段性大雨。 八月一日至三日的大雨是由台風維克托掠過本港引起的。每次崩塌發生前都伴有階段性降雨，雨勢有時頗大。

自一九九七年六月三十日午夜十二時至七月七日上午九時，當第一次崩塌發生時，該雨量計共錄得 621 毫米雨量。其中絕大部分雨量（582 毫米）是在七月一日至七月四日中午十二時期間錄得。

七月二日所錄得的每日雨量（420.5 毫米）是雨量計 K06 自一九八三年三月裝置以來所錄得的最高每日雨量。

然而在第一次崩塌發生前的幾天裡，降雨量相對較小。 據此，相對於嚴重豪雨事件，第一次崩塌可看成〝滯後〞崩塌。 一九七二年和一九八二年發生於此削土坡鄰接斜坡上的主要山泥傾瀉，也比嚴重豪雨的發生日期遲後幾天，因此也被看成〝滯後〞崩塌（參閱附錄A）。

第二次和第三次崩塌發生前，同樣也下了大雨。 自七月十五日至七月十七日大約下午三時，當第二次崩塌發生時，該雨量計錄得 219 毫米雨量；從八月一日至八月三日大約上午十一時十五分，當第三次崩塌發生時，錄得 278.5 毫米雨量。 這兩次崩塌沒有"滯後"現象，但均發生於大雨過後不久（圖7）。

圖8顯示了一九九七年山泥傾瀉之前的雨量變化及事發地點過往較嚴重豪雨的雨量變化進行的比較。 一九九七年七月四日之前的三十一天期間，該雨量計K06 錄得的總雨量（1557.5 毫米）為有史以來錄得的最高記錄，並超出了天文台的雨量計自一八八四年以來所錄得的最高月雨量。

一九九七年八月三日崩塌發生前，該雨量計所錄得的月雨量與事發地點過去的月雨量記錄相比，並不是特別嚴重（圖8）。

根據天文台以往的雨量記錄，不同降雨時段的最大雨量強度以及它們的相應重現期的估算結果，列於表1。 從表中可見，在七月四日前的三十一日這時段內所錄得的雨量最為罕有，根據天文台以往的雨量數據，其相應重現期約為五佰年。 對香港地區總體而言，此三十一日所錄得的雨量的相應重現期，不能以現存資料估計到, 但其數值可能較小。然而，可以得出的結論是在山泥傾瀉發生前的雨量是極為罕有的。

6. 事發地點的地下情況

6.1 概述

事發地點的地下情況，是根據文件及實地調查所得的資料而確定。 文件研究包括翻查現有岩土質料。 實地研究則包括地質勘察和特為此調查設計所作的場地勘探。

從一九九七年七月三十一日至八月十三日期間，香港地質測繪處對斜坡進行了多次地質勘察和作圖，場地勘探於一九九七年九月開始。 場地勘探共鑽 12 個垂直鑽孔，4 個傾斜鑽孔，挖 12 個探井和 3 個斜坡表面剝除(圖9)。 為了取得最多岩芯，鑽孔方法是使用泡沫沖洗。

在一九九七年的山泥傾瀉之前，已有的場地勘探工程分佈情況載於圖10。

6.2 地質

事發地點的地質圖載於圖11，地質剖面圖載於圖12 和圖13。

在發生山泥傾瀉的斜坡上，主要物質是極弱至非常弱，淡紅棕色帶淡黃棕色斑及黑白斑點高風化及全風化的中粒至細粒花崗岩，節理間距中至緊密。全風化花崗岩局部被熱液交換(表現為深化學風化)，有時，相應的玄武質岩脈侵入厚達 1.2 米。 在某些鑽孔中探測到玄武質岩脈，部份風化至黏土質粉土。玄武質岩脈與天然地面近於平行，即岩脈斜交於坡面，傾向南至西南面 5 度 至 65 度，平均 35 度。 從岩脈產狀和所測到的深度來看，其延伸可達數十米。 玄武質岩脈的滲水性明顯低於周圍的花崗岩，因此，岩脈可能充當阻水層，抑制地下水向下運動。

在位於山泥傾瀉部位及相鄰斜坡的鑽孔裏，岩脈之上風化土層裏，發現高達 250 毫米的天然侵蝕管(圖12 和圖13)。 這些侵蝕管的典型間距大約為 6 米，而在與滑坡殘痕東北側相鄰外間距則更小。 觀察到的侵蝕管一般是被填充，通常為富含石英的沉積物，依次從充填於侵蝕管頂部的細粉土至充填於侵蝕管底部的小礫石。 偶然地，充填材料由分級好的含粉土及黏土的砂和小礫石組成。這些沉澱物可能是在相當復雜的，較大範圍的和不斷變化的地下水活動系統中沉積下來的。 此系統包括有地下水坑和"豎井"，通過一系列侵蝕管而相連。

在最近勘探期間的鑽孔中，遇到了其他地質帶，其物質顯示了受擾動的可能性（圖12 和圖13）。 擾動帶中的物質典型地為潮濕、淡至中棕色、含礫砂(細砂)、粉土的軟黏土，偶然含有細樹根(1 至5 毫米)，此物質沒有結構並具完全均一的顏色。 這個帶中的部份物質可能已被最近的崩塌擾動，但其餘部份出現在遠離新近破裂面界限的深部。

在位於滑坡崖，於斜坡上中部沿著水流線，觀察到厚度達 2.5 米的崩積土，向東西兩方向逐漸變薄(土力工程處，1997)。 崩積土物料組成有棕色至暗棕色含礫石，粉土，黏土的砂，並有大漂石，漂石直徑大到4米，成份為輕微至中等風化的粗粒花崗岩。 一九八八年(土力工程處，1989a) 進行的地面勘察表明，另一層不同的崩積土層延伸到削土坡的第二個傾斜面，這個位置與最近的斷裂是一致的，並為LPM的照片證實(照片11)。 地層地貌關係表明，一陡而窄的峽谷堆積有含漂石的土層。 在天然狀態下，此處曾經可能是切割峽谷及相應的崩積層土包，但唯一可查到的航空照片所反映的現場是一九四五年之後的，地貌的情況被採石活動所擾動而非天然狀態。 此處曾經可能是山體表面排水溪(也許與相應的地下水流有關)。

一個具代表性橫貫山泥傾瀉地點的地質剖面圖載於圖14。

6.3 構造地質

在滑坡崖測到的連續的節理，一般以大約 50 度至 70 度傾入斜坡，傾向東北面(大約 050 度至 080 度)。 這些節理的傾向與防止山泥傾瀉計劃的地質測量的結果一致(土力工程處，1989a)，(圖15)。 這些節理一般充填有暗棕色褐鐵礦或鐵錳結核，並局部充填有白至淡淺黃色高嶺土，一般少於 3 毫米厚(土力工程處，1997)。 這些節理的傾向也証實了防止山泥傾瀉計劃的調查結果(土力工程處，1989a)，見圖15a。

在最近的場地勘探期間，在鑽孔中利用壓印器測試，測定未風化岩帶中不連續面的方位。 此外，在鑽孔 DH1 和 DH6 ，利用鑽孔聲學遙測器測定岩石的不連續面的方位，並與壓印器測定的結果比較。 結果表明，節理大都傾向西北偏西面和西南偏南面約 40 度 (圖15b)。 這些不利穩定的節理傾斜的方位，不同於在事發地點的測量結果。這個差別，可能是測量偏差的結果(太沙基，1967)。 分析結論是不同測量方法所得節理方位均在風化岩體中存在(垂直和平行於斜坡方位)。

在現場地面勘察中，發現了幾個從 20 毫米到幾米厚的細粒花崗岩岩脈(土力工程處，1997)，(圖11)。 岩脈多傾入山体，傾向西北面(大致 320 度至 345 度方位)，傾角約 30 度 至 45 度，但是，在削土坡東側最低的傾斜面上，一主岩脈之產狀同高岭土裂隙及節理充填物相一致，傾向東南面(大約 150 度至 170 度)，傾出斜坡。 在 6.2 節中提到的玄武質岩脈，在幾個鑽孔中均遇到並且也可能傾出斜坡，但是，在出露的山泥傾瀉區沒有發現。

6.4 土壤及岩石的性質

在一九八八年/一九八九年防止山泥傾瀉計劃及在一九八二年發生於編號 11NW-A/C55 至 56 斜坡上的山泥傾瀉進行的現場勘探中，對收集的大量土和岩石樣品進行了一系列的岩土室內試驗。 試驗包括粒徑分佈，密度測定，阿太堡界限，直剪和多階段三軸壓縮。 試驗的樣品包括殘積/漸變土，全風化花崗岩，強風化花崗岩，全風化細晶岩，全風化粗粒玄武質岩。 從過往勘探中試驗所得到物質強度系數(土力工程處，1982a 和土力工程處，1989b)載於表2。 盡管兩組岩土室內試驗在不同時間進行，樣品取自不同的地點，但測定強度的程序相同並認為充分代表事發地點物質的強度，滿足本研究的目的要求。 任何不精確和代表性不足，同土體中殘餘節理及以前滑動面對土樣的影響相比，要小得多，第七節將就此討論。

6.5 地下水情況

事發地點地下水的情況評估是通過對所有地下水記錄及地面滲水觀察資料而進行的，所研究的資料包括以下各項：

(a) 斜坡面特別是導水管的滲水現察記錄(茂盛，1972; 土力工程處，1982b 和土力工程處，1989a);

(b) 於一九八八年六月至一九八九年七月期間，在一九八八年/一九八九年防止山泥傾瀉計劃工程中，在 14 個垂直鑽孔(圖10編號 DH1 至 DH14)內，收集到的地下水監察資料;

(c) 事發前後，分別於一九九六年三月至一九九七年二 月及一九九七年七月至一九九七年十一月期間，在蝴蝶谷水庫項目中，其中 23 個垂直鑽孔內，收集到的地下水監察資料;

(d) 事發後，於一九九七年七月三十日至一九九七年八月五日期間，滑坡殘痕幾個部位及從一九七二年安裝的導水管中，觀察到有水滲出;

(e) 一九九七年滑坡後，作為緊急修補工作的一部份對新安裝的導水管的效應進行的觀察; 以及

(f) 一九九七年，事發後的場地勘探包括原位滲透試驗。

對於斜坡中地下水位的資料有互相矛盾的情況存在。 如在報告第四節中所述，在一九九七年的滑坡以前，局部的高地下水位情況被多次發現。 在防止山泥傾瀉計劃工程(一九九零年至一九九二年)期間，可能是由於從一九九二年之前的八年的年降雨量低於平均年降雨量的原因，所以滲水現象很少觀察到。

在場地周圍的鑽孔裏，測得的水位一般為低，並且隨著降雨量的變化而改變。 例如，在一九八八年防止山泥傾瀉計劃工程中，安裝在附近的鑽孔 DH12 中的測壓計，滑坡殘痕的頂部。 經過雨季後，在一九八八年九月測到逐漸上升的地下水位，然後測到連續兩個月的地下水位上升，盡管其後的兩個月是旱季(土力工程處，1989a)。

在一九九七年六月和七月份的連場暴雨之後安裝在蝴蝶谷水庫測壓計，測得水位逐漸上升了 8 米。 而測壓計的末端安裝接近在基岩界面。

從不同觀察資料證實，在最近的山泥傾瀉期間，斜坡內存在高地下水位。在一九九七年七月三十日的調查中發現，還掛在滑坡殘痕面上的一些導水管；還在排水，這些導水管是在一九七二年安裝的(土力工程處，1997)。 這些導水管在一九九七年七月七日的滑坡以前是否仍在運作直至現在還不能判斷。

在八月三日的山泥傾瀉以後，在滑坡殘痕，邊坡山脊以下 10 米和邊坡的較低部位，觀察到許多滲水點(圖11)。 此外，在一九九七年滑坡之後新安裝在滑坡殘痕中部的三條導水管，一直有水流出(照片12)。

在一九九七年山泥傾瀉發生後，一個重要觀察到的現象是"甚至在暴雨後，此斜坡場地自然排泄路徑是乾的，說明在邊坡上部某處，地面水是被阻斷(滲入地下)"(土力工程處，1997年)。 這個假設是十分可能接近真正的現象，即:雨水被阻擋在邊坡上部某處，然後急速地滲入到散佈在山坡內的天然侵蝕網路，並且逐漸流向邊坡。 此地下水運動是受整個天然侵蝕管網路系統的滲透性而控制。在這次滑坡過程中，受七月初暴雨的影響，過了幾天才引起編號 11NW-A/C55 削土坡內水壓力的增加，觸發崩塌。

7. 理論穩定性分析

7.1 概述

調查工作的一部份是通過理論穩定性分析，來協助判定引致山泥傾瀉的機制和原因。 分析中，對一系列在崩塌時可能具有的地下水情況，通過反算來確定斜坡內土體的抗剪強度參數。 分析結果顯示在崩塌發生時，最高地下水瞬變水位，估計可能在地面下 5 米至 7 米處(圖12 和13)。 為了容易參考，水位的高度在圖16中是以相對於一假設的玄武質岩脈來表示的。

7.2 一九九七年發生於編號 11NW-A/C55 削土坡上山泥傾瀉的分析

山泥傾瀉的土力模型是根據報告中別處己詳述的資料研究，現場觀察，及事發後的現場勘探結果而建立的。 用於分析具代表性的山泥傾瀉現場剖面圖載於圖12 和13。 基本的地質模型與土力工程處的設計報告(GCO，1989b)中使用的削土坡原設計模型有少許不同; 主要不同在於假定的地下水位。 基於原有的室內試驗結果，並結合有關經驗，採用了一定的抗剪強度參數範圍值，對發生在一

九九七年山泥傾瀉進行了敏感性分析。 分析中採用了兩個可能滑動面（圖12 和 13)，即：

(a) 一九九七年山泥傾瀉時的最可能滑動面; 及

(b) 假設的較深層的滑動面。

分析計算應用了Janbu 的精解法（Janbu，1954）。 分析結果載於圖16。

用在分析試算中的最低抗剪強度參數為 c' = 0 kPa 及 Ø' = 35 度，此低抗剪強度參數只可能適用於當大量先前被擾動土體仍遺留在一九九七年山泥傾瀉的地點上的情況，即此時土體內為重塑抗剪強度。 在此狀況下，斜坡不用什麼水壓力在理論上亦變得不穩定。 但事實上觀察到的滲水位顯示此假設水位偏低。

相反，如應用在防止山泥傾瀉 (LPM) 調查(土力工程處，1988) 所測得的高度風化花崗岩抗剪強度參數，亦即是c' = 15 kPa 及 Ø' = 39 度時，需很多高的水位才能引發滑坡。 即水位大約需位於參考玄武質岩脈上 20.6 米才能導致一九九七年的淺層山泥傾瀉，或需位於玄武質岩脈上約 24 米才會導致深層滑動。

按現場觀察在山泥傾瀉發生前，最有可能的水位，估計在離玄武質岩脈 17 米至 19 米間。 在此水位間，全風化花崗岩的有效抗剪強度參數按計算應為 c' 值在 5 kPa 至 9 kPa 之間和 Ø' 值在 36 度至 39 度之間。 此強度參數的具體數值將取決於斜坡土體中完整材料強度變化和土體原生節理及先前破壞面對強度的影響。

8. 山泥傾瀉成因的診斷

8.1 考慮的因素

在評估一九九七年山泥傾瀉成因時，需要考慮不同的因素，它們包括:

(a) 事發地點發生山泥傾瀉的歷史;

(b) 地下水及用於穩定 1972 年山泥傾瀉斜坡的導水管起的作用; 以及

(c) 山泥傾瀉明顯的漸進崩塌特徵。

8.2 過去的山泥傾瀉紀錄

有關事發地點最重要的發現是其長期不穩定性。 圖17 顯示了按一九七二年呈祥道建造圖紙所示的一九五四年/一九六三年山泥傾瀉的大致範圍在最近的現場平面圖上的位置。 根據對一九六三年的航空照片分析，已經確定這個新近的滑坡崖(一九九七)在平面上大致與一九六三年的滑坡崖範圍是相同的。 圖18 和圖19 是這個斜坡的剖面圖，他們清楚地顯示了一九六三年的滑坡範圍和一九九七年的滑坡之前和之後的範圍，一九五四年的滑坡範圍是從一九六三年的航測分析中估計的，因為最近的地層調查已給發現最初的斜坡腳和地層的深度已經變化，一九九七年的滑動面的位置是建立在現場觀察到的張裂隙，坡腳的運動和分析擾動的地層。 一九七二年和一九九七年的滑坡清楚地顯示這個斜坡的滑坡崖接近到一九五四年的滑坡崖，並且傾瀉的岩土仍然遺留在編號 11NW-A/C55 的削土坡上。

以前山泥傾瀉可假定為由於風化岩体局部弱化引起的。 滑坡帶包含有鬆散被擾動的物質，如在最近勘探中遇到的(圖12) 。 盡管知道有舊滑坡存在，但除了其局部幾何形態相一致外，沒有其它清楚的證據說明一九九七年的山泥傾瀉是舊滑坡的復發。 據報告，斜坡坡腳未被擾動，按此証據推斷，新近崩塌範圍應於路面高程之上。 如此則可推斷，最近的山泥傾瀉的形態是受斜坡幾何形態產生的原位應力條件及其與斜坡坡腳建築物、地下水條件影響;且大部份滑動面為過往未被剪切之土體之內。 但是某些局部崩塌幾乎肯定是舊滑坡的復發，並由此引起斜坡整體強度的降低。 分析表明，此山泥傾瀉可能屬於這種情況(第七章)。

此場地的山泥傾瀉的發展歷史，被認為對斜坡中，天然侵蝕管系統的形成極為重要，此系列侵蝕被認為支配了山泥傾瀉地點的水文地理。

8.3 地下水和導水管的作用

經分析證實在沒有水的情況下，斜坡應是穩定的，但是按崩塌後所觀察到的可能的地下水情況來看，分析証實，斜坡會變得不穩定，此結果不論是以沿斜坡下部兩個傾斜面崩塌，還是沿八月份整個斜坡高度的破壞面崩塌均如此。

研究認為在一九九七年山泥傾瀉前，一九七二年安裝的導水管未能防止在斜坡內產生臨界地下水壓力。其原因可以肯定大致上是由於山泥傾瀉前，降雨量極為罕有（估計在七月四日前的 31天雨量相應重現期約為五佰年）。 此降雨量要比安裝導水管後此斜坡曾經經受過的任何降雨量要大得多（圖8）。 導水管未能充分降低水位，亦可能是由於在崩塌發生前，已可能遭受損壞或淤塞。

8.4 崩塌的漸進破壞性

調查結果認為此次山泥傾瀉明顯的漸進性崩塌特徵（七月七日，七月十七日，八月三日）可能會誤導關於崩塌的引因。 分析証明，隨著七月初的極端降雨，七月七日崩塌的較低兩個斜坡傾斜面在極不利的地下水條件下變得不穩定是合理的，而類似的分析亦証明，在這些條件下，整個斜坡亦會變得不穩定。 計算分析亦顯示，當小部份坡腳的重量被移走後，只會使整個削土坡的安全系數略有下降。

調查結果認為，第一個突發事件的延緩特性是較深崩塌的表徵，根據現場測得的水力傳導率，雨水從匯水區流到崩塌帶，四天是一個合適的延緩期。 類似地，第一個崩塌的寬度比露出變形區的高度要大得多以及當隨後斜坡上部崩塌發生時寬度保持不變的事實，表明主破裂面的形成是七月初地下水作用的結果。七月十七日和八月三日的事件可能是已被大雨（和在大雨期間）擾動物質的崩塌。

9. 結論

此調查所得結論是，一九九七年七月十七日及八月三日的崩塌是繼一九九七年七月七日大規模山泥傾瀉的漸進性崩塌。

引致山泥傾瀉的主要原因是一九九七年七月初的豪雨後及在此之前一個月內的大量降雨而引起的不利的瞬變地下水條件，一九七二年安裝在削土坡的導水管系統不能有效防止臨界水位的產生。

崩塌的平面形態與一較早時的崩塌大致相符，及過往的剪切面可能局部被復發。 大量崩塌下的物質(從一九五四年/一九六三年崩塌產生)遺留在斜坡上及呈祥道下某處。

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表

表1 — 新近發生山泥傾瀉事故之前，土力工程處編號 K06 雨量計在不同時段所錄得的最高滾存降雨量及其估計的重現期

降雨時段	最高滾存降雨量(毫米)	降雨時段結束時間	估計的重現期(年)
(a) 一九九七年七月七日發生山泥傾瀉前的不同降雨時段			
5分鐘	13	一九九七年七月二日凌晨五時四十五分	2
15分鐘	36	一九九七年七月二日早上六時十分	7
1小時	114.5	一九九七年七月二日早上六時十五分	19
2小時	162.5	一九九七年七月二日早上六時三十分	18
4小時	192	一九九七年七月二日早上七時正	10
12小時	265	一九九七年七月二日下午三時正	7
24小時	431.5	一九九七年七月三日凌晨二時正	20
2天	534.5	一九九七年七月四日凌晨二時正	27
4天	628.0	一九九七年七月四日早上九時正	25
7天	673.0	一九九七年七月六日下午三時正	21
15天	827.5	一九九七年七月六日下午三時正	19
31天	1557.5	一九九七年七月四日凌晨二時正	498
(b) 一九九七年七月十七日發生山泥傾瀉前的不同降雨時段			
5分鐘	13	一九九七年七月二日凌晨五時四十五分	2
15分鐘	36	一九九七年七月二日早上六時十分	7
1小時	119	一九九七年七月二日早上六時二十五分	24
2小時	162.5	一九九七年七月二日早上六時三十分	18
4小時	192	一九九七年七月二日早上七時正	10
12小時	265	一九九七年七月二日下午三時正	7
24小時	431.5	一九九七年七月三日凌晨二時正	20
2天	534.5	一九九七年七月四日凌晨二時正	27
4天	628	一九九七年七月四日早上九時正	25
7天	673	一九九七年七月六日下午三時正	21
15天	870.0	一九九七年七月十六日凌晨二時正	25
31天	1212.5	一九九七年七月十七日下午三時正	52
(c) 一九九七年八月三日發生山泥傾瀉前的不同降雨時段			
5分鐘	9	一九九七年七月十九日早上四時正	1
15分鐘	23	一九九七年七月十七日早上七時正	1
1小時	52.5	一九九七年七月十七日早上七時二十分	1
2小時	68.5	一九九七年七月十七日早上七時五十五分	1
4小時	75	一九九七年七月十七日早上十時正	1
12小時	133	一九九七年八月二日晚上九時正	1
24小時	199	一九九七年八月三日早上九時正	2
2天	267	一九九七年八月三日早上十時正	2
4天	307.5	一九九七年七月十九日下午三時正	2
7天	341	一九九七年七月十九日晚上八時正	2
15天	484.5	一九九七年七月十九日晚上八時正	2
31天	851.5	一九九七年八月三日早上十一時正	5

注： (1) 重現期從 Lam and Leung 之表3 的數據以及岡貝爾(Gumbel)公式 (1994) 推算獲得。

(2) 最高滾存降雨量是用每5分鐘的雨量數據計算出二小時以下的降雨時段及用每小時的雨量數據計算出更長的降雨時段。

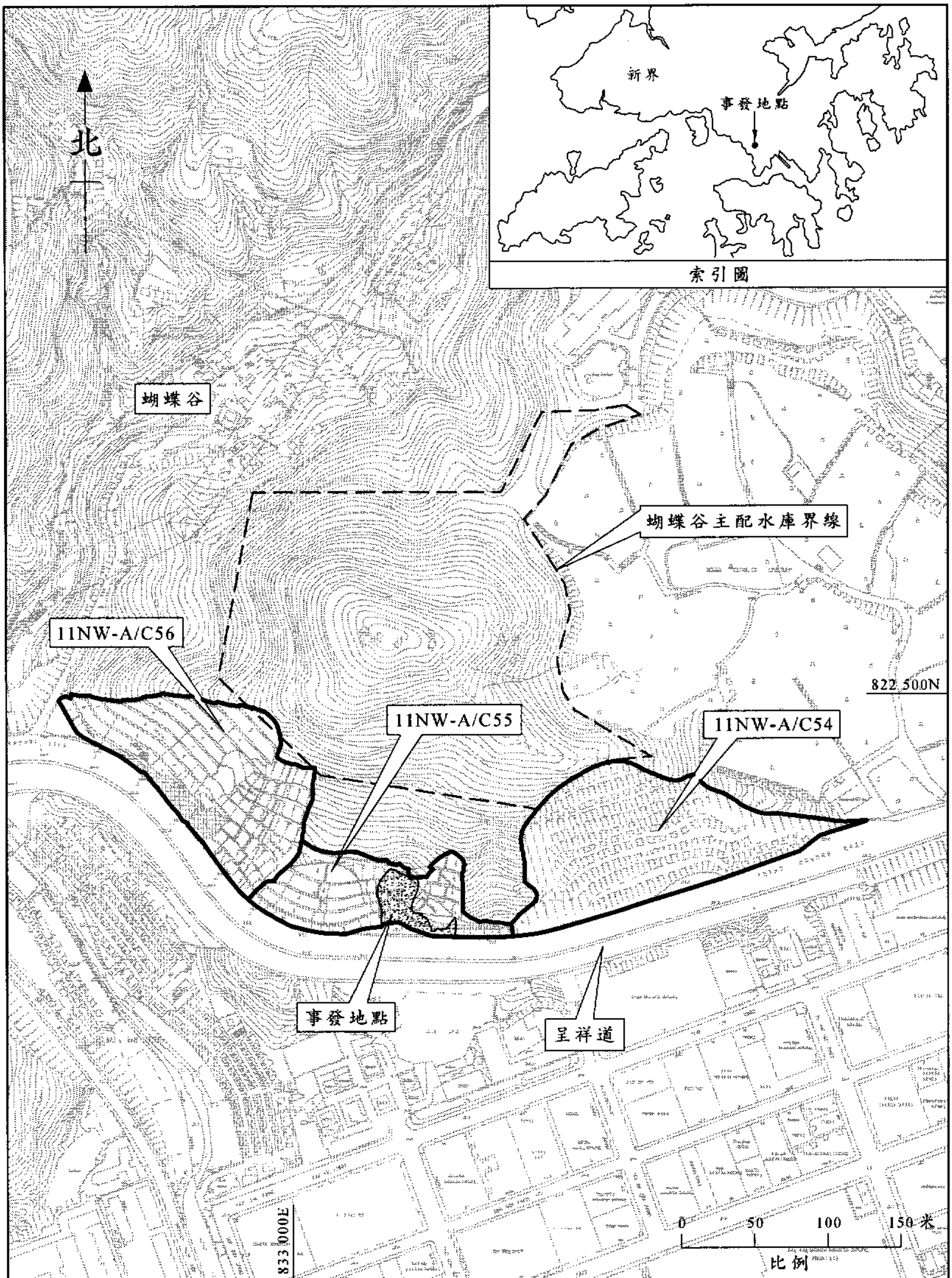
表2 — 土強度試驗結果摘要（錄自過往的勘探）

土種類	土強度系數			
	土力工程處(1982)		土力工程處(1988)	
	c′(kPa)	ø’(度)	c′(kPa)	ø’(度)
殘積/漸變土	–	–	2	37
全風化花崗岩	–	–	9	39
強風化花崗岩	12	37.5	15	39
全風化細晶岩	–	–	7.2	40
全風化粒玄岩	20	28	10	40

註：（1） 物質在一九八二年被描述為微裂隙的強風化花崗岩是根據特性指標(施米特錘回彈數值和抗崩能力)而定，與描述為可被手崩潰的全風化花崗岩同為一物。

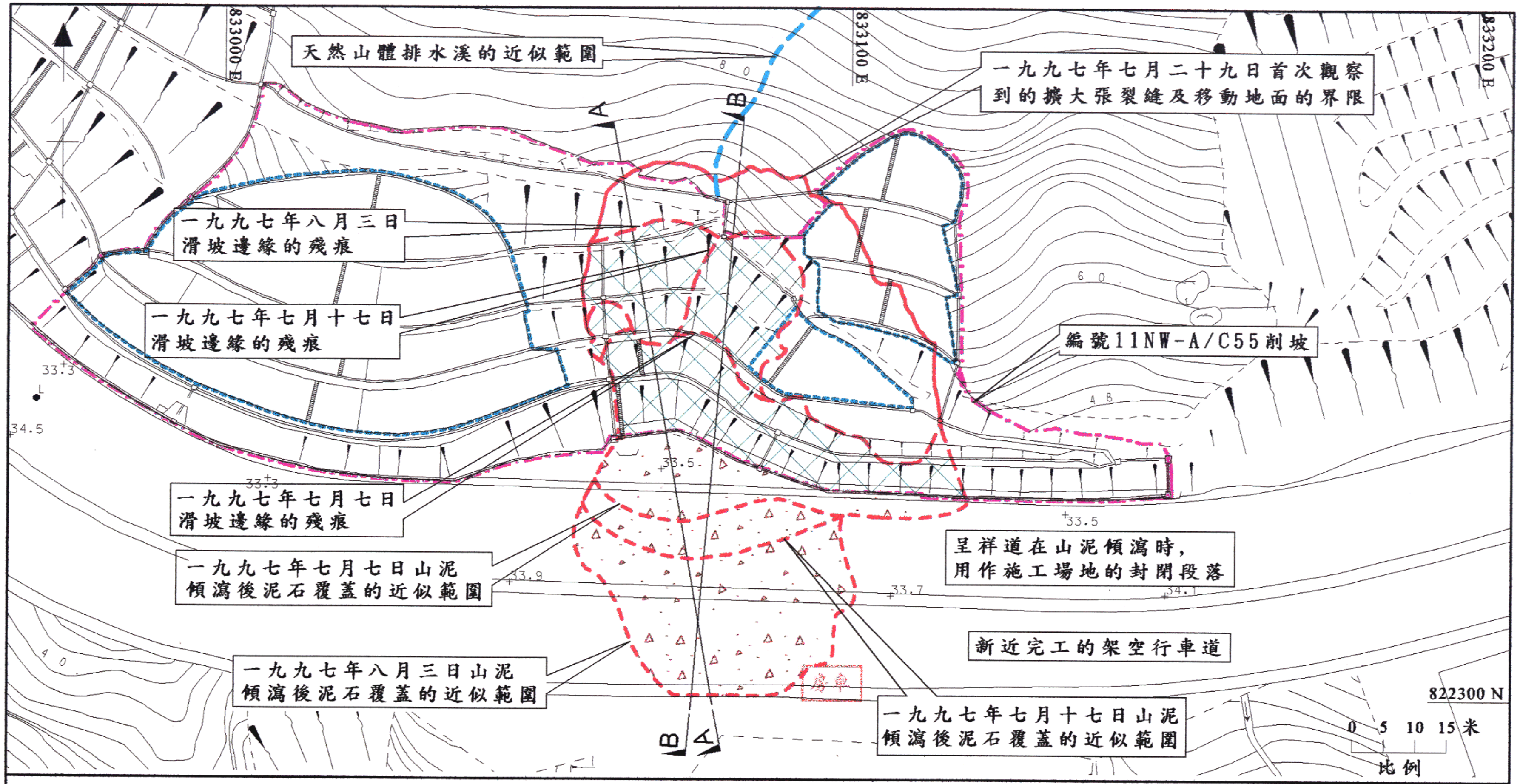
（2） 粒玄岩及玄武質岩(在本報告中提及過)為同物異名。

圖



註： 取自編號11-NW2D, 3C, 7B, 7D, 8A及8C測量圖(比例1:1 000).

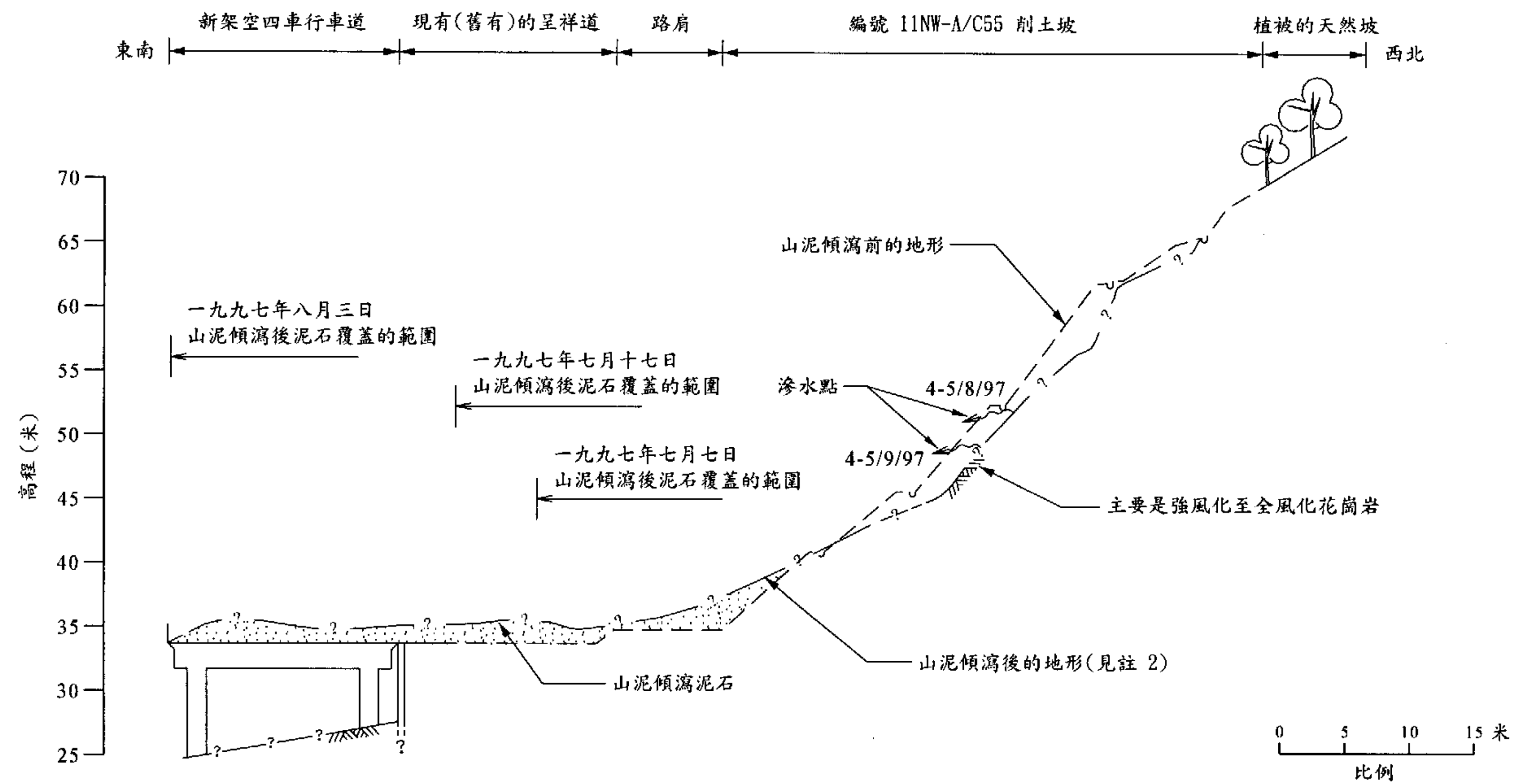
圖1 － 山泥傾瀉位置圖



圖例：在一九九零年至九二年山泥傾瀉防治計劃工程中加固過的斜坡　山泥傾瀉痕跡　山泥傾瀉泥石　近似的斜坡界線

註：（1）圖中所示資料以地形測量，1:1 000地形測量圖，地質勘察，現場觀察及文件記錄為根據。
（2）剖面A-A及B-B見圖12及圖13.

圖2 － 山泥傾瀉平面圖



註: (1) 圖中所示資料以地形測量, 1:1 000地形測量圖, 地質勘察, 現場觀察及文件記錄為根據.

(2) 按文件記錄推斷在山泥傾瀉後的地形及泥石地形.

圖3 - 山泥傾瀉地點的典型剖面

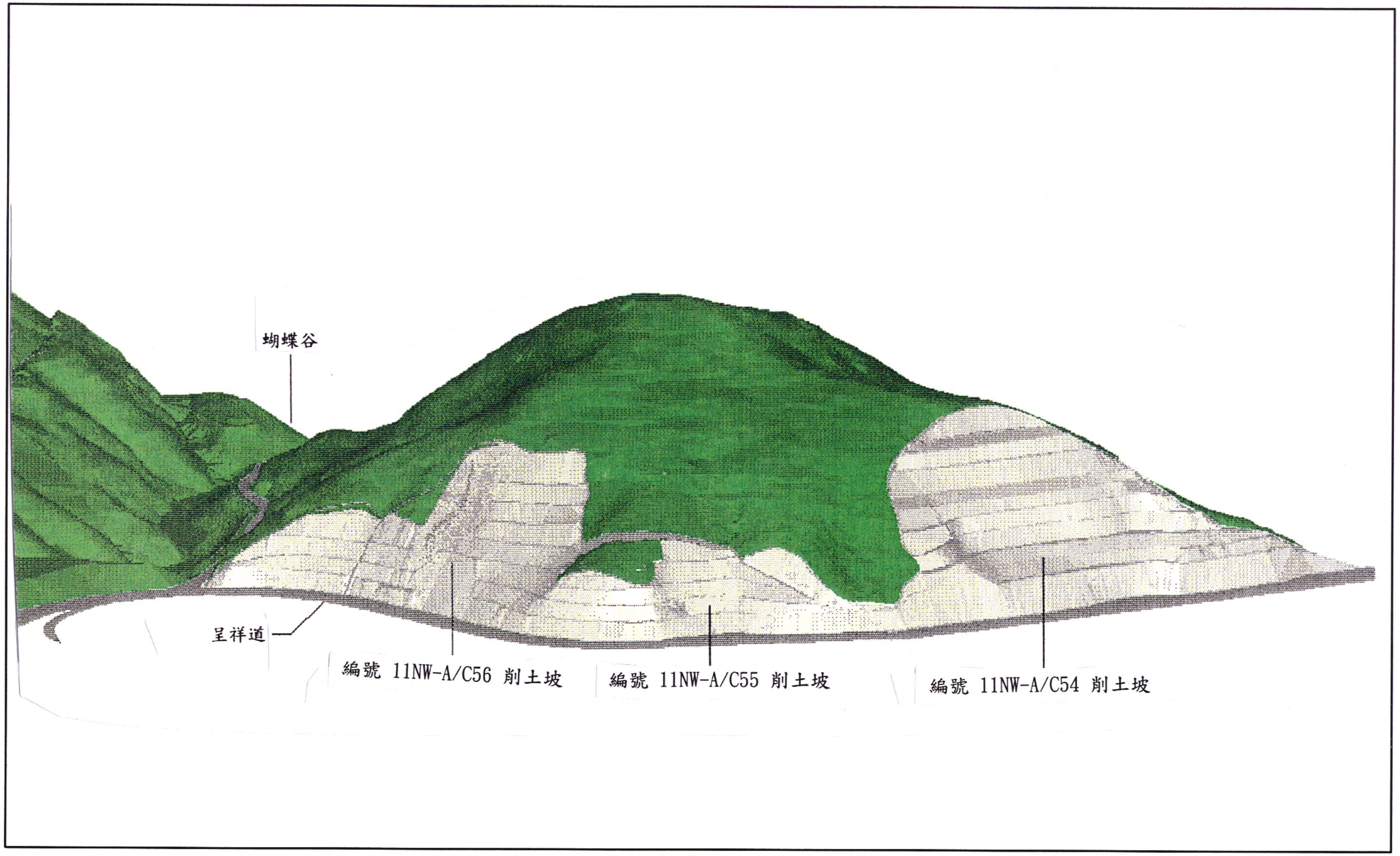


圖4 – 電腦編制的削土坡地形在一九八八年的三維景象
（在山泥傾瀉防治計劃之前）

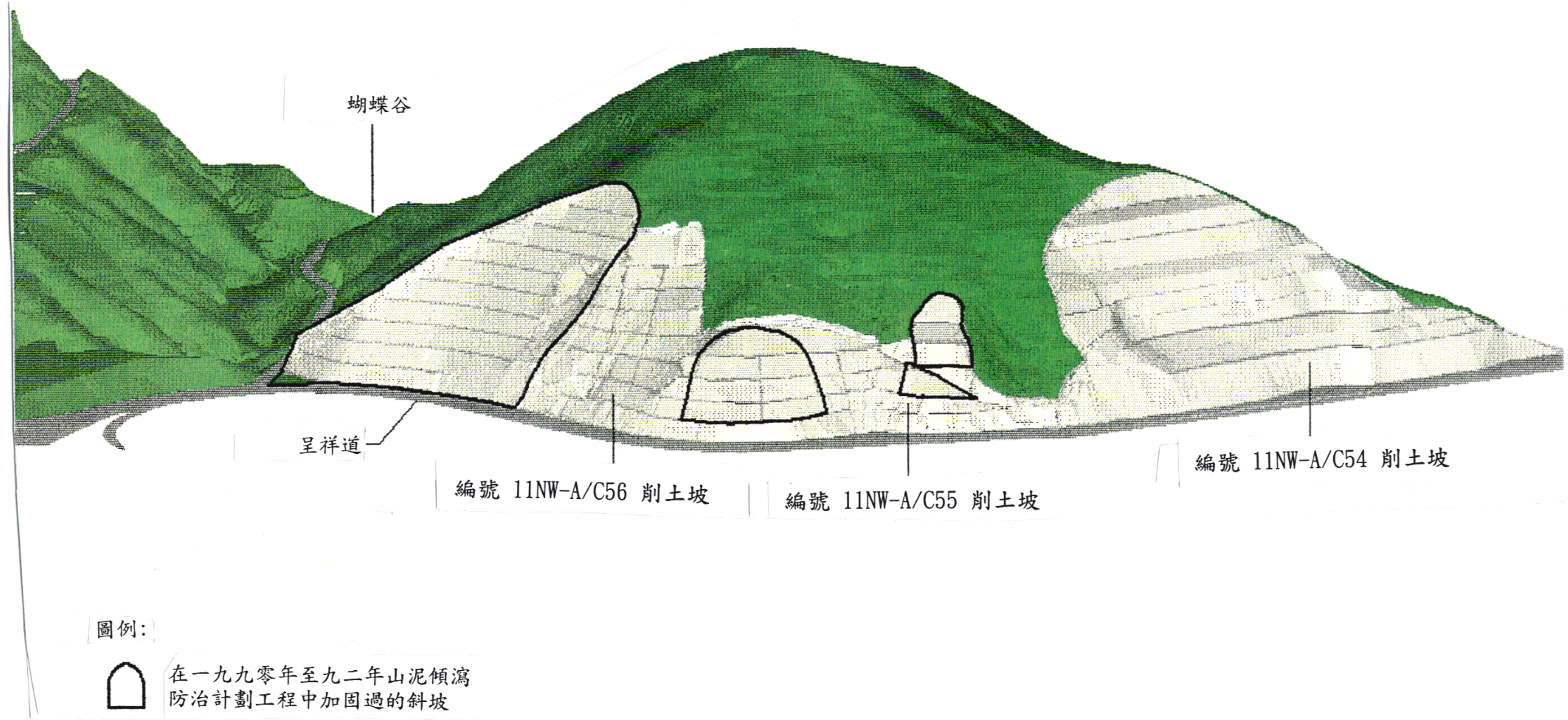


圖5 － 電腦編制的削土坡地形在一九九二年的三維景象
（在山泥傾瀉防治計劃工程之後）

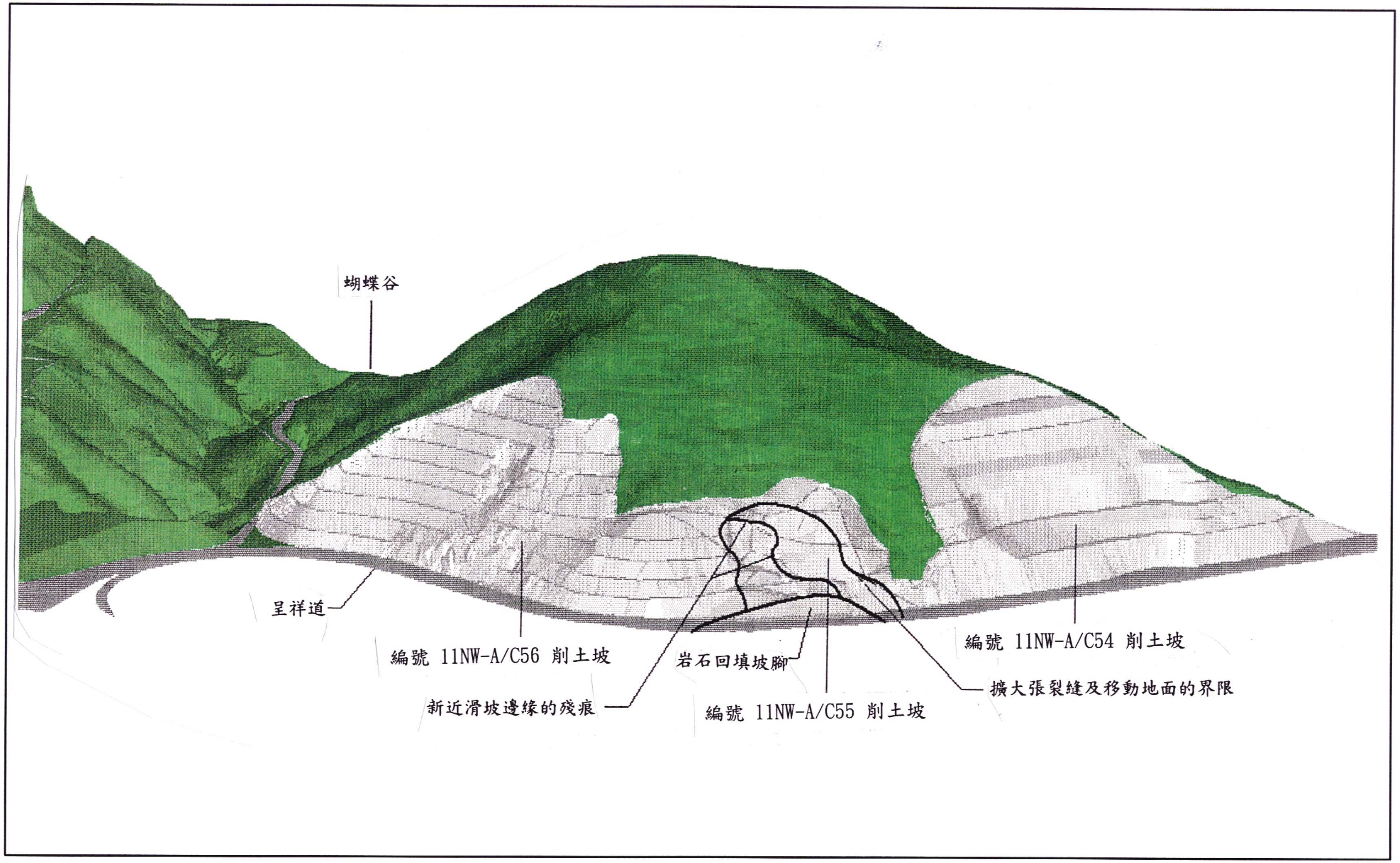
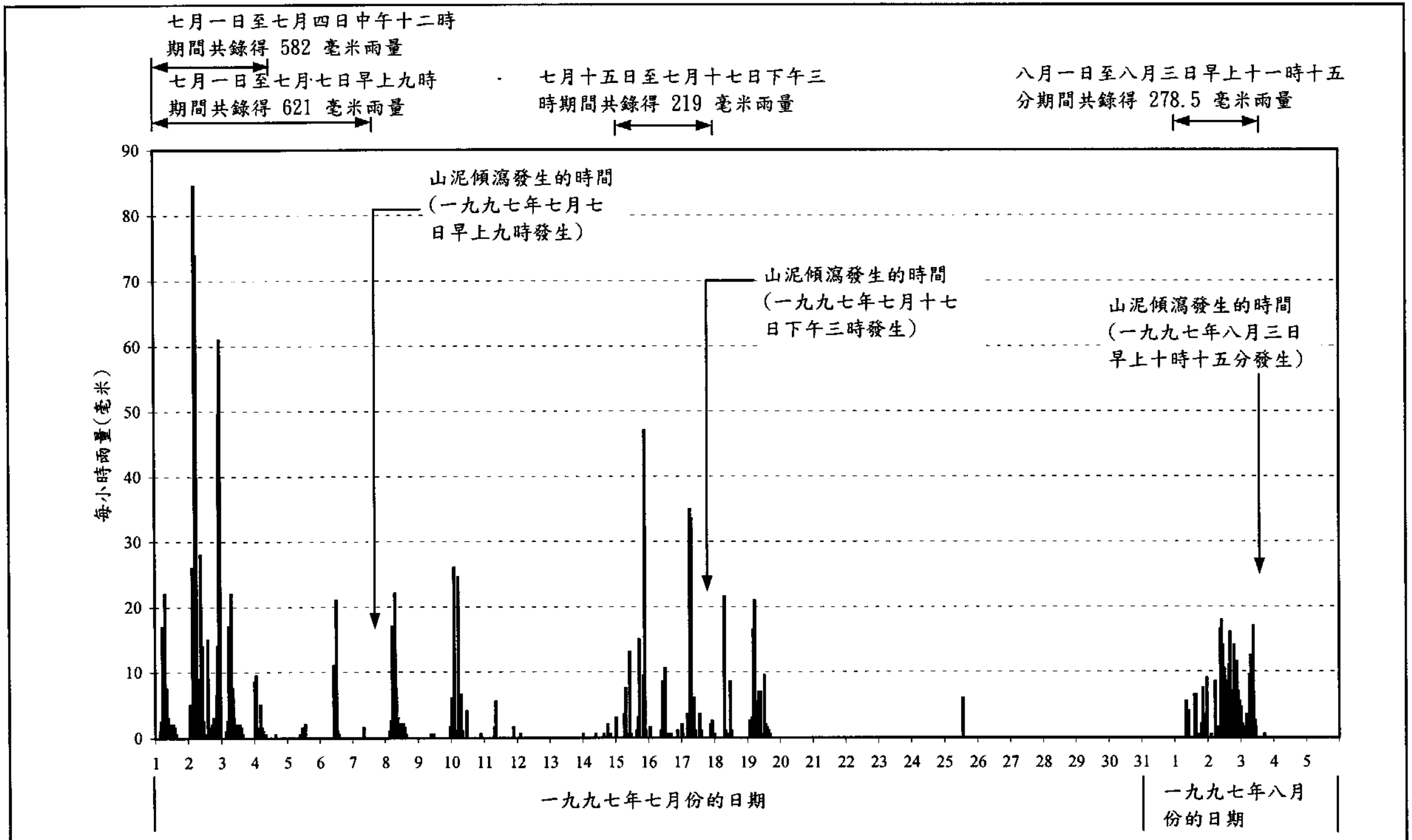
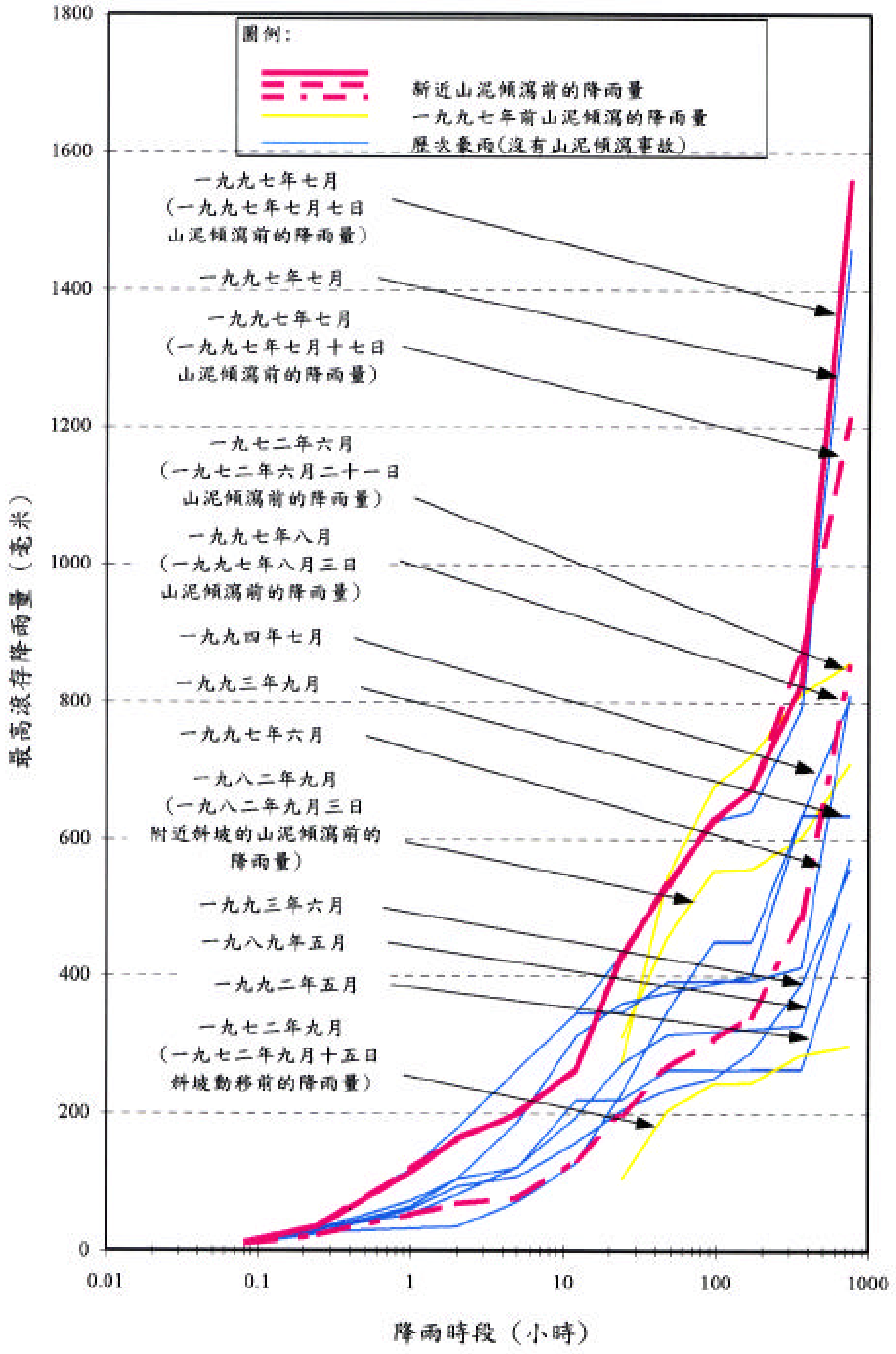


圖6 – 電腦編制的削土坡地形在一九九七年的三維景象
（新近滑坡後）



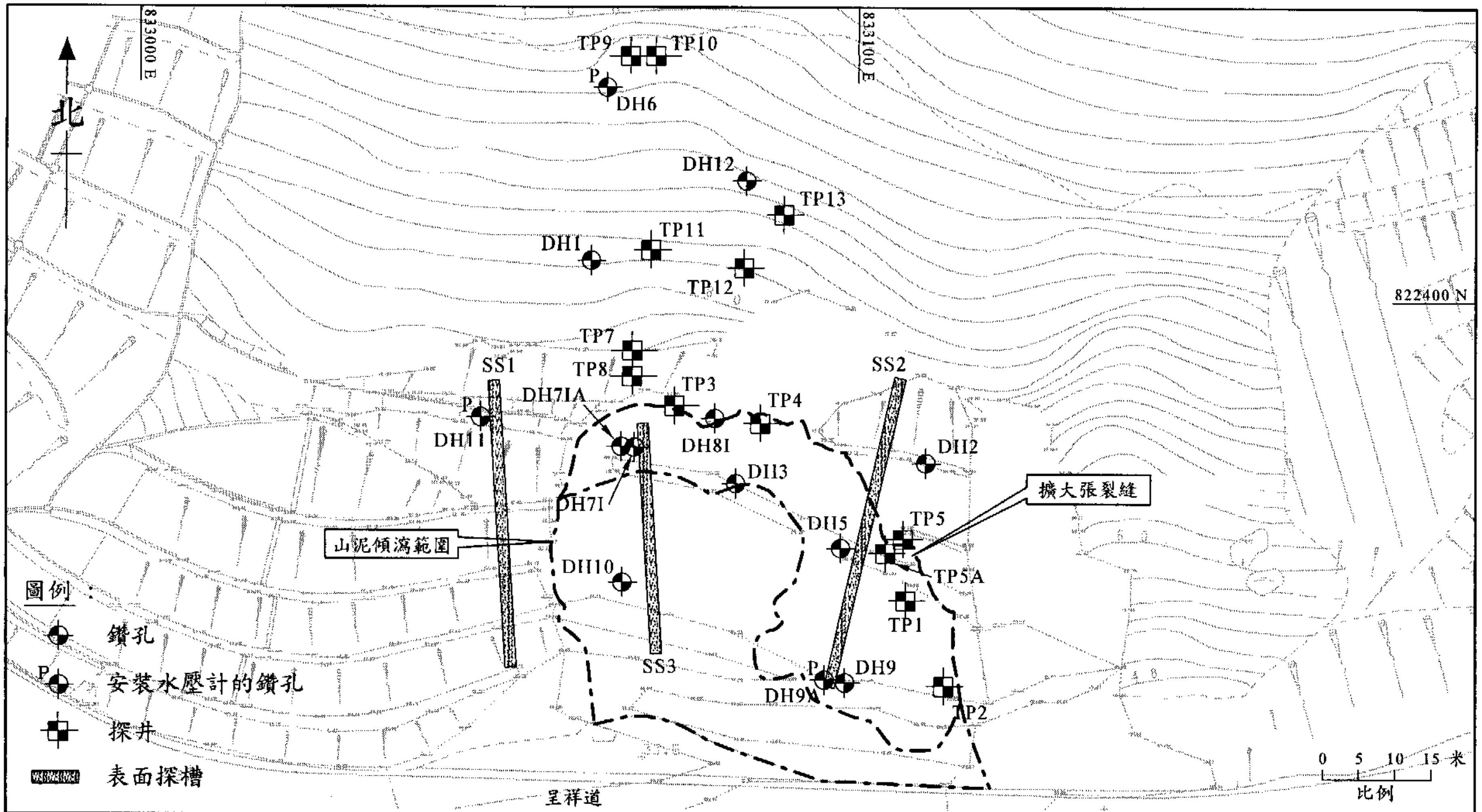
註：滑坡發生的時間源自訪問目擊者

圖7 － 土力工程處編號 K06 雨量計的每小時雨量記錄



註：　圖中的降雨量錄得自土力工程處編號 K06 雨量計，但不包括九龍水塘雨量計在一九七二年六月二十一日及一九七二年九月十五日以及土力工程處編號 N04 雨量計在一九八二年九月三日錄得的降雨量。

圖 8 – 以往山泥傾瀉前(從一九七二年六月)及歷次豪雨中錄得的最高滾存降雨量



註：

(1) 圖中所示的山泥傾瀉範圍以文件記錄為根據.

(2) 斜坡在最近的緊急搶修工程期間已進行過補救，其範圍沒有標示在圖中.

(3) 圖中所示是合樂為此次山泥傾瀉調查所進行的場地勘探工程.

圖9 － 場地勘探工程位置圖

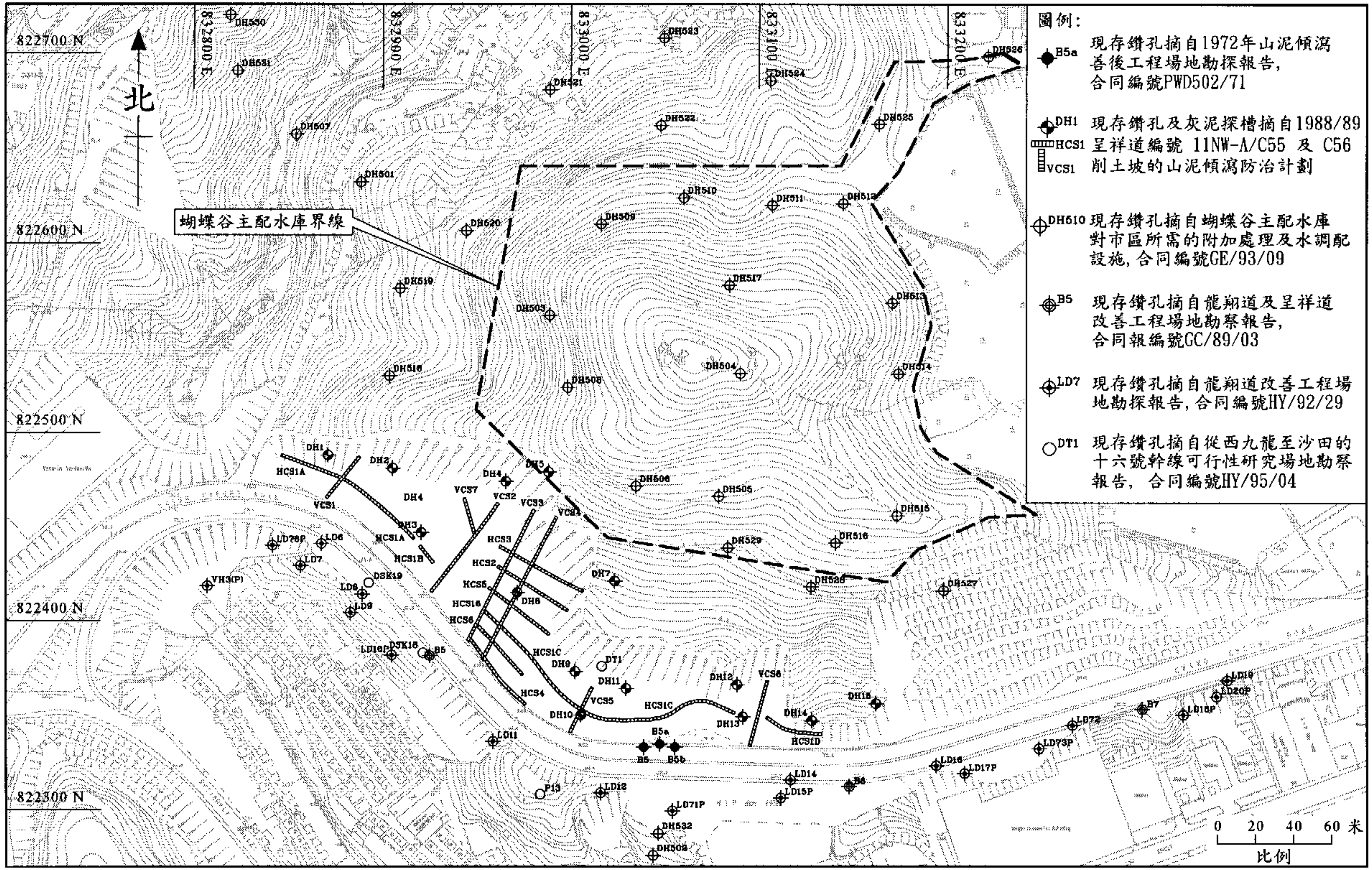
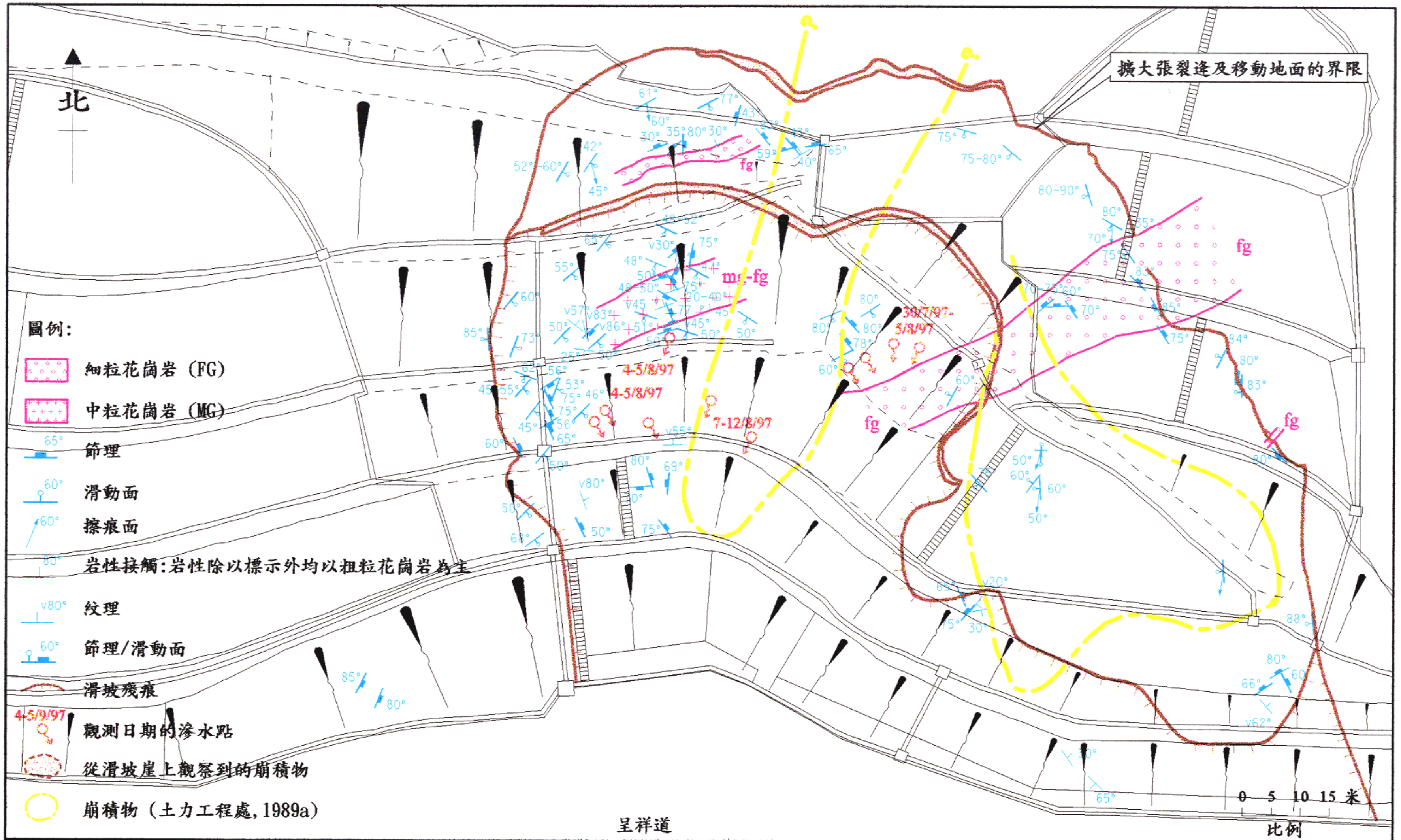


圖10 - 一九九七年以前的場地勘探工程平面圖



註: 圖中所示資料以土力工程處地質測量組之現場勘察為根據.

圖11 - 山泥傾瀉範圍地質平面圖

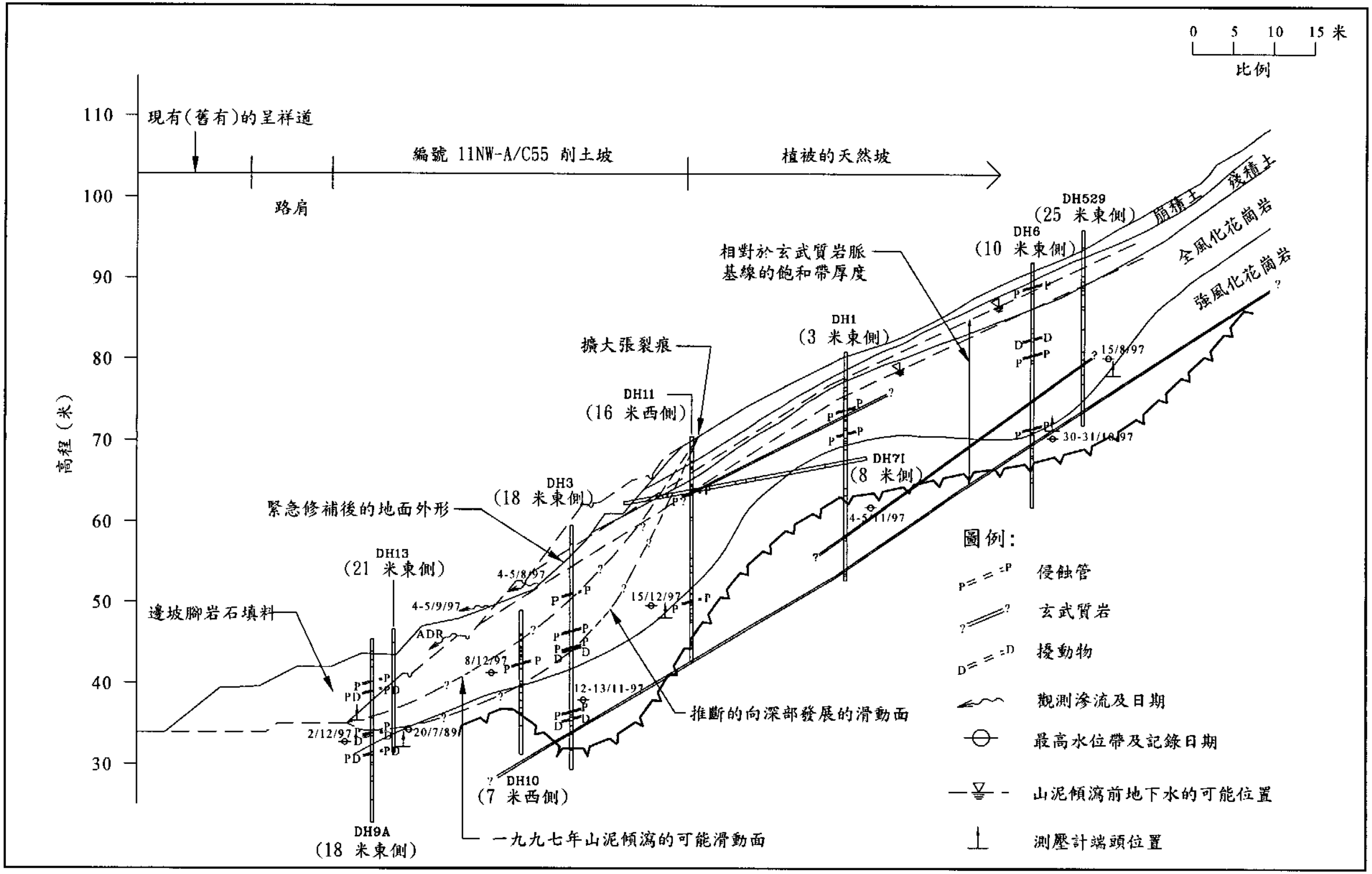

0 5 10 15 米
比例
高程(米)
110
100
90
80
70
60
50
40
30
現有(舊有)的呈祥道
路肩
編號 11NW-A/C55 削土坡
植被的天然坡
崩積土
殘積土
全風化花崗岩
強風化花崗岩
相對於玄武質岩脈基線的飽和帶厚度
DH529 (25 米東側)
DH6 (10 米東側)
DH1 (3 米東側)
擴大張裂痕
DH11 (16 米西側)
DH7I (8 米側)
DH3 (18 米東側)
緊急修補後的地面外形
DH13 (21 米東側)
邊坡腳岩石填料
DH10 (7 米西側)
DH9A (18 米東側)
推斷的向深部發展的滑動面
一九九七年山泥傾瀉的可能滑動面
15/8/97
30-31/10/97
4-5/11/97
15/12/97
4-5/8/97
4-5/9/97
ADR
8/12/97
12-13/11-97
2/12/97
20/7/89
圖例:
侵蝕管
玄武質岩
擾動物
觀測滲流及日期
最高水位帶及記錄日期
山泥傾瀉前地下水的可能位置
測壓計端頭位置

圖12 － 地質剖面圖 A － A

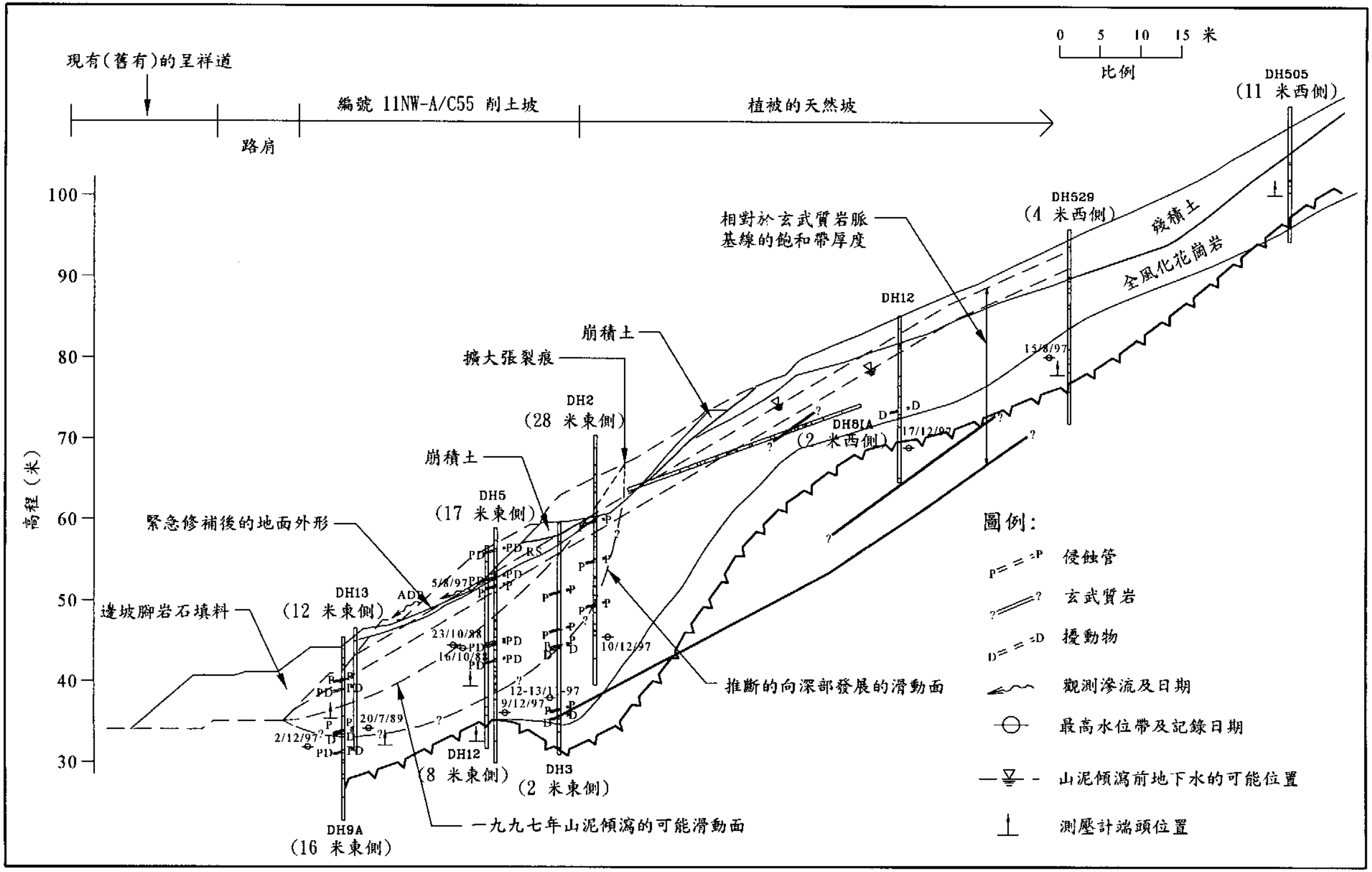

現有(舊有)的呈祥道
路肩
編號 11NW-A/C55 削土坡
植被的天然坡
0 5 10 15 米
比例
DH505
(11 米西側)
DH529
(4 米西側)
殘積土
全風化花崗岩
相對於玄武質岩脈
基線的飽和帶厚度
DH12
崩積土
擴大張裂痕
DH2
(28 米東側)
DH8IA
(2 米西側)
崩積土
DH5
(17 米東側)
緊急修補後的地面外形
DH13
(12 米東側)
邊坡腳岩石填料
高程(米)
100
90
80
70
60
50
40
30
15/8/97
17/12/97
10/12/97
5/8/97
23/10/88
16/10/88
12-13/1/97
9/12/97
20/7/89
2/12/97
推斷的向深部發展的滑動面
一九九七年山泥傾瀉的可能滑動面
DH9A
(16 米東側)
DH12
(8 米東側)
DH3
(2 米東側)
圖例:
侵蝕管
玄武質岩
擾動物
觀測滲流及日期
最高水位帶及記錄日期
山泥傾瀉前地下水的可能位置
測壓計端頭位置

圖13 – 地質剖面圖 B – B

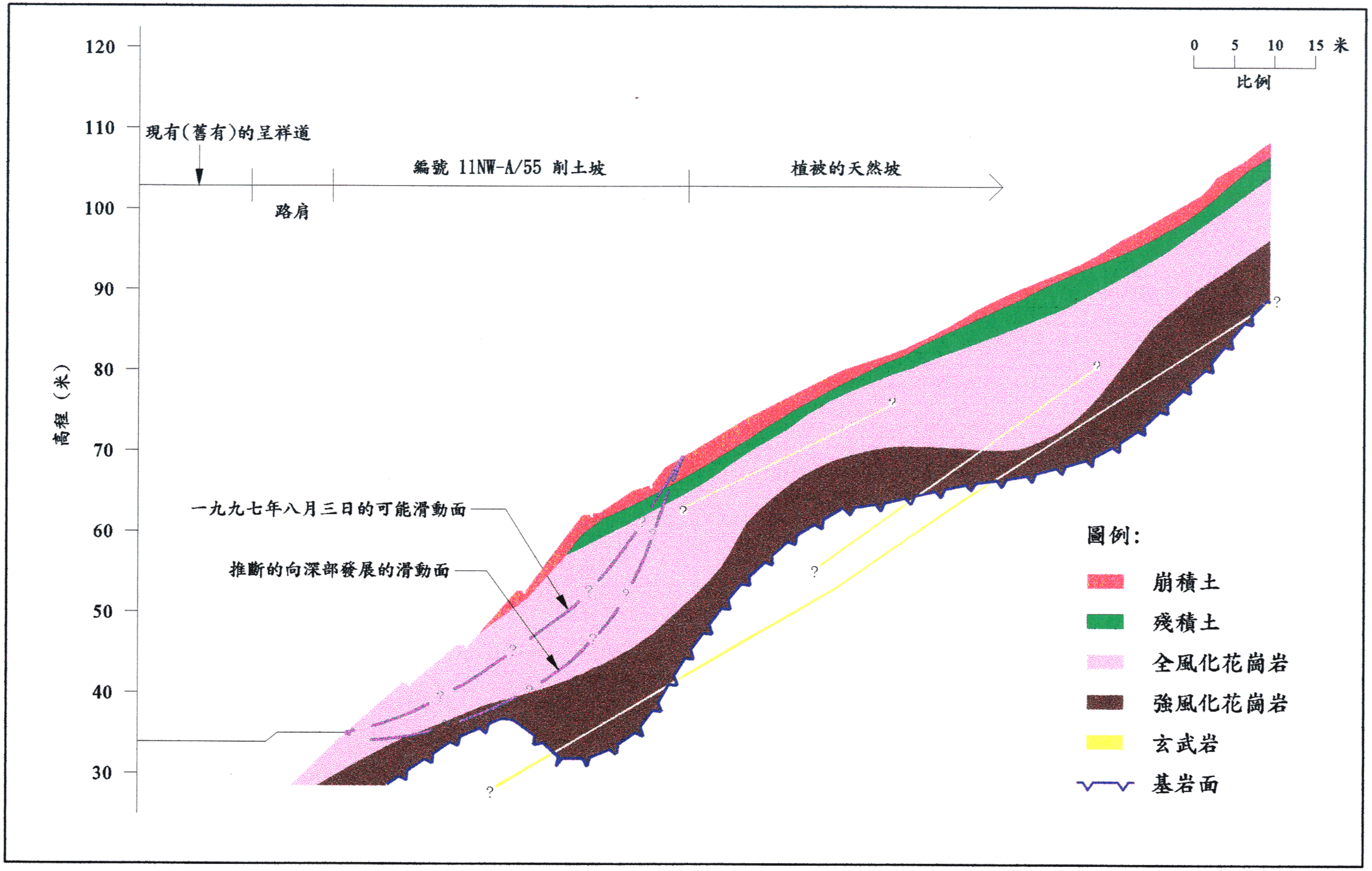
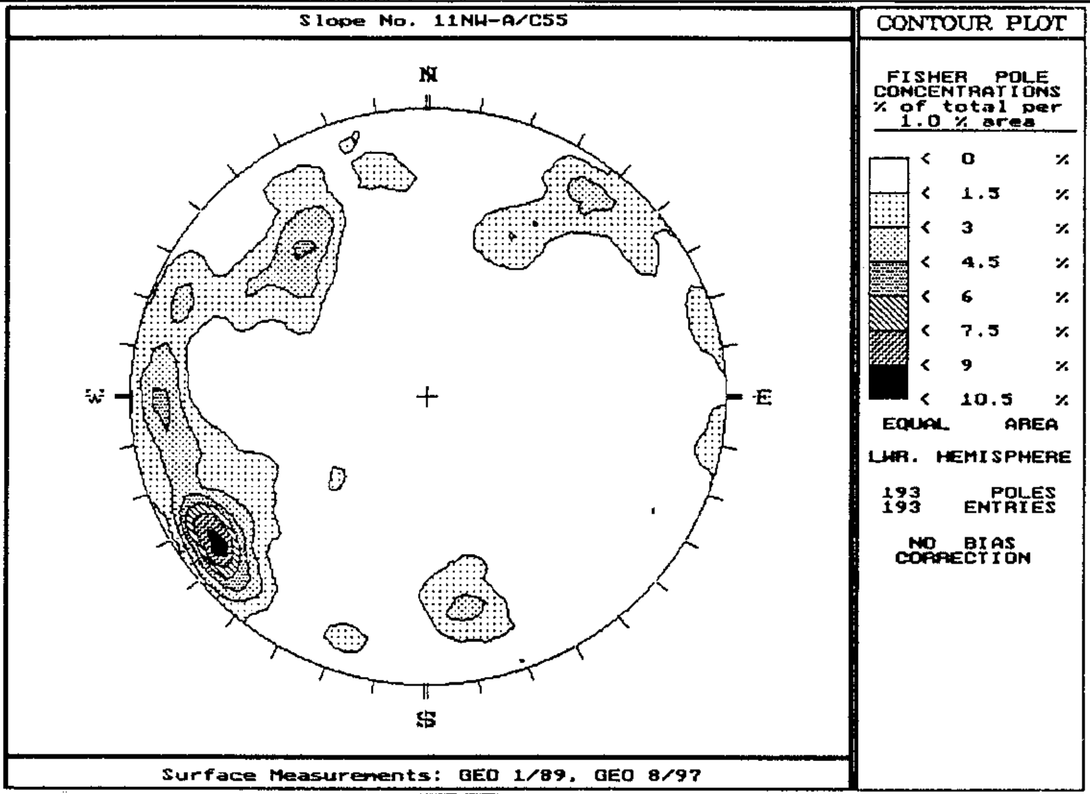
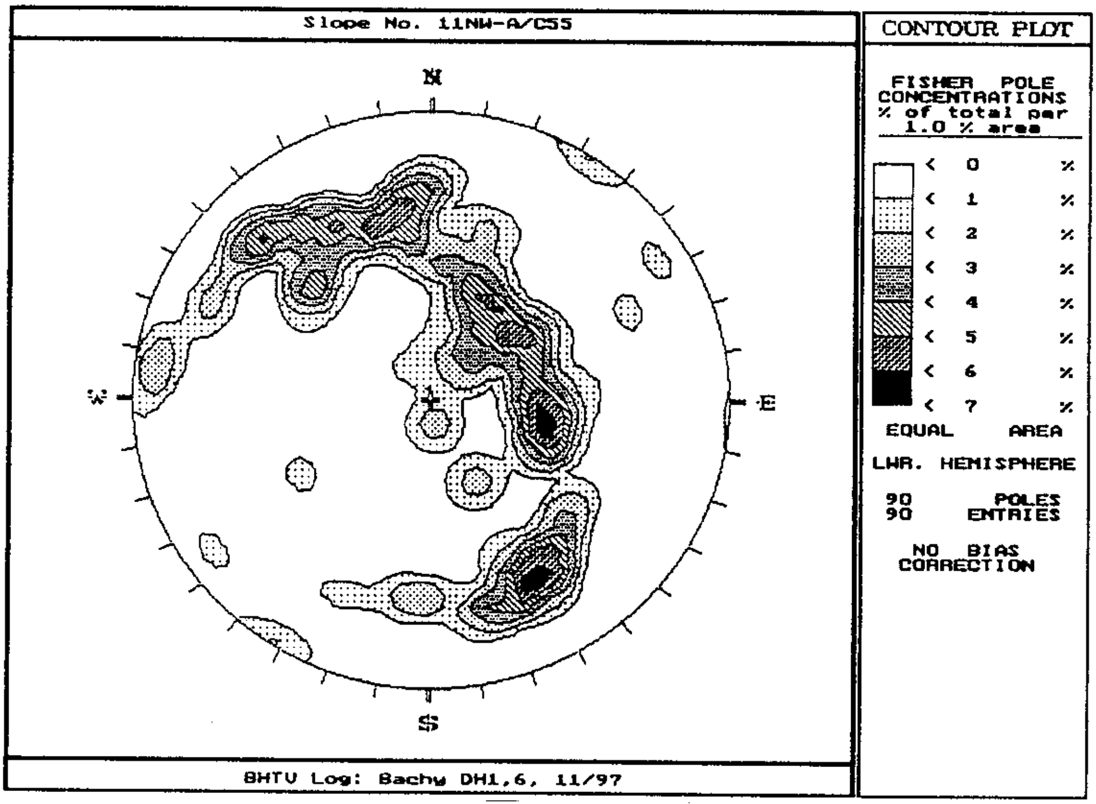


圖14 – 剖面 A – A 的地層分界面

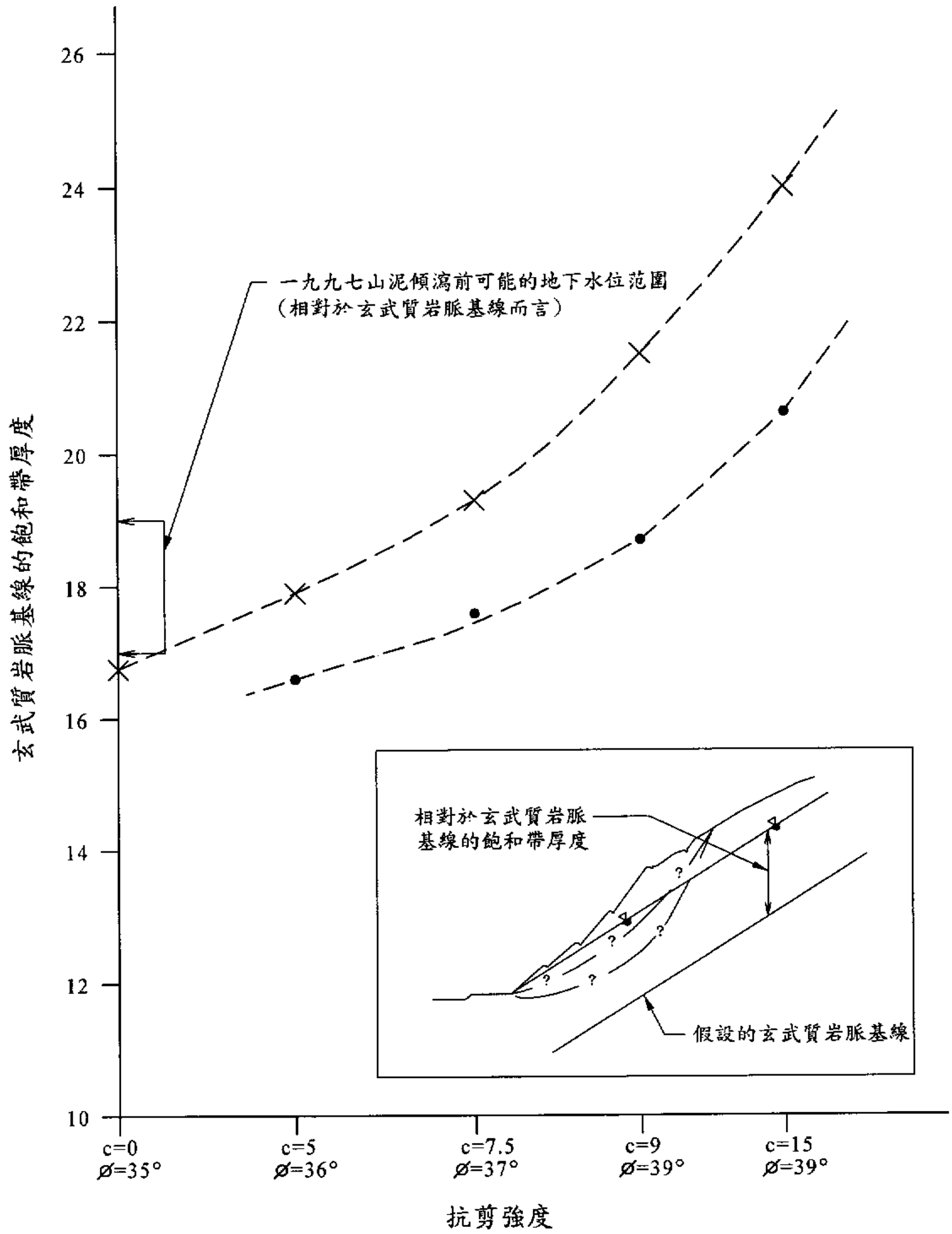


a) 地表節理測量



b) 鑽孔節理測量

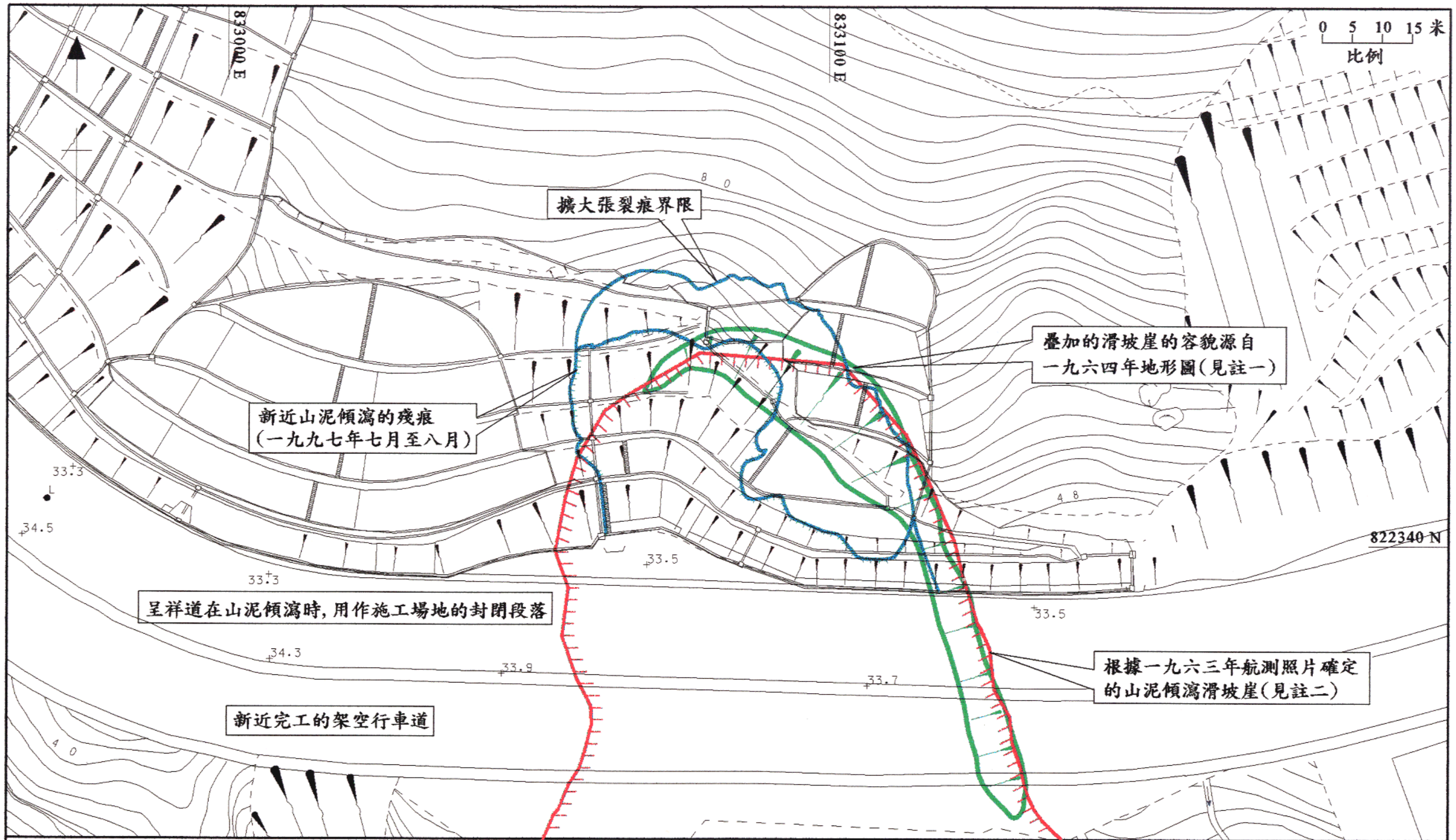
圖15 - 在編號 11 NW-A/C55 削土坡測到的節理傾向的極點統計圖



圖例: × 表示深層的滑動面 • 表示較淺層的滑動面

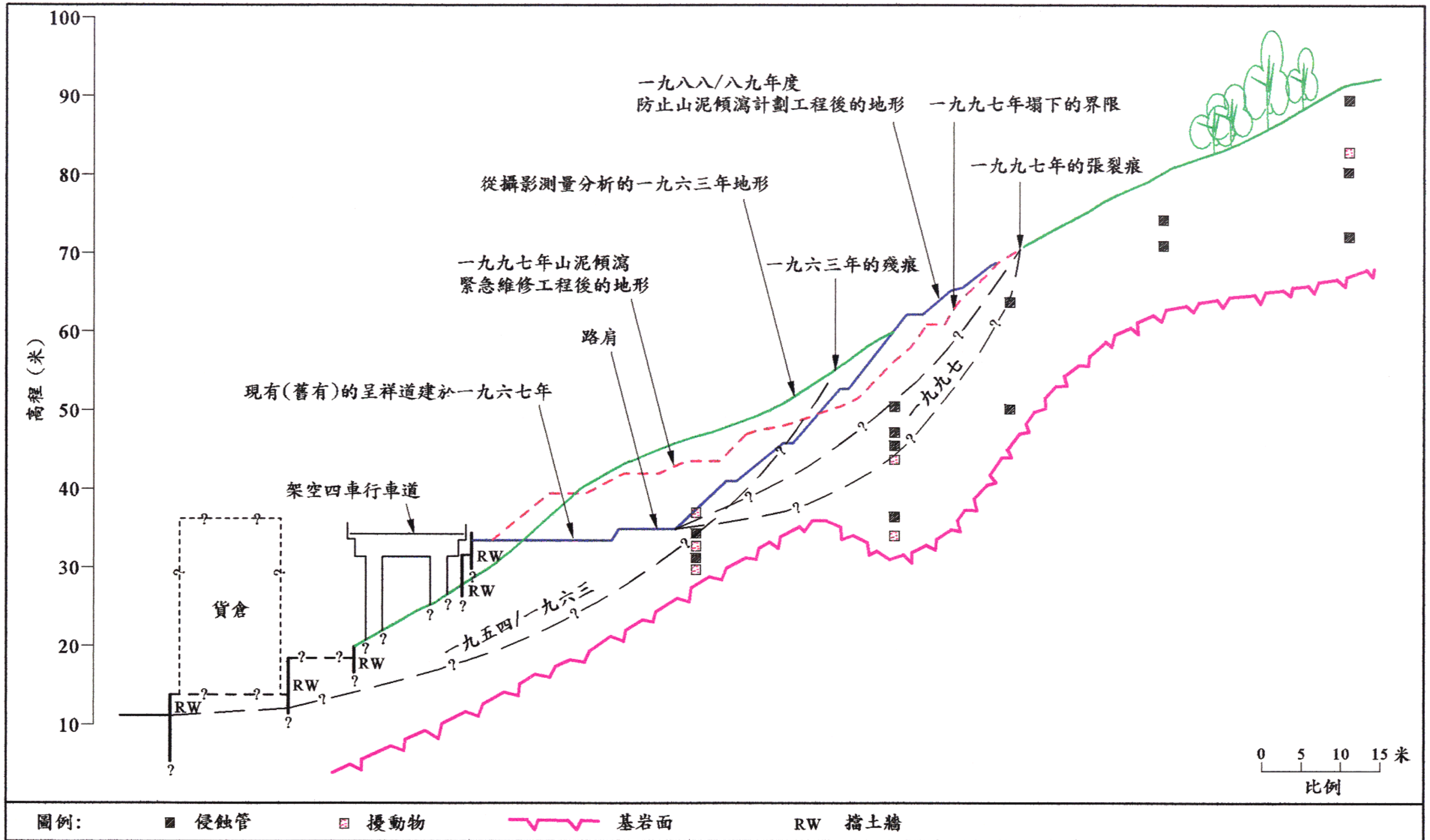
註: 飽和帶的厚度是根據穩定性分析在安全系數=1.0時所得的結果.

圖16 - 對剖面 A - A 進行斜坡穩定敏感
計算結果



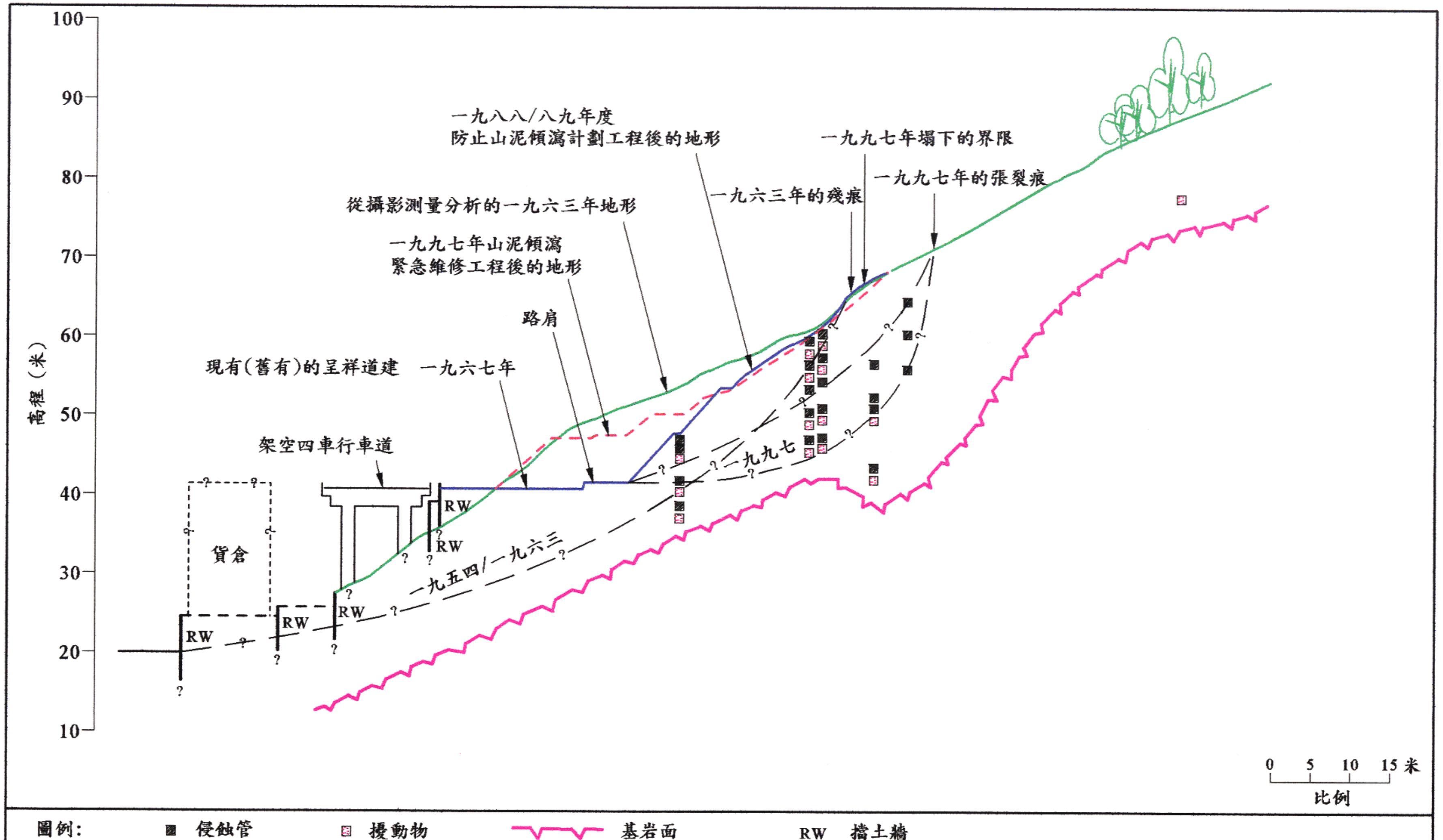
註: (1) 滑坡崖的容貌源自香港公共工程署圖號RK1410/1C,大埔道至青山公路段.
(2) 從攝影測量分析所確定的滑坡崖容貌.

圖17 - 一九五四/六三年的滑坡崖疊加在新近滑坡的平面圖上



註: (1) 一九八八/八九年的地形取自一九八八/八九年度防止山泥傾瀉計劃的圖則.
(2) 一九六三年的地形獲取自攝影測量分析.

圖18 - 剖面 A-A 的一九五四年及一九九七年山泥傾瀉的地形及假設崩塌面



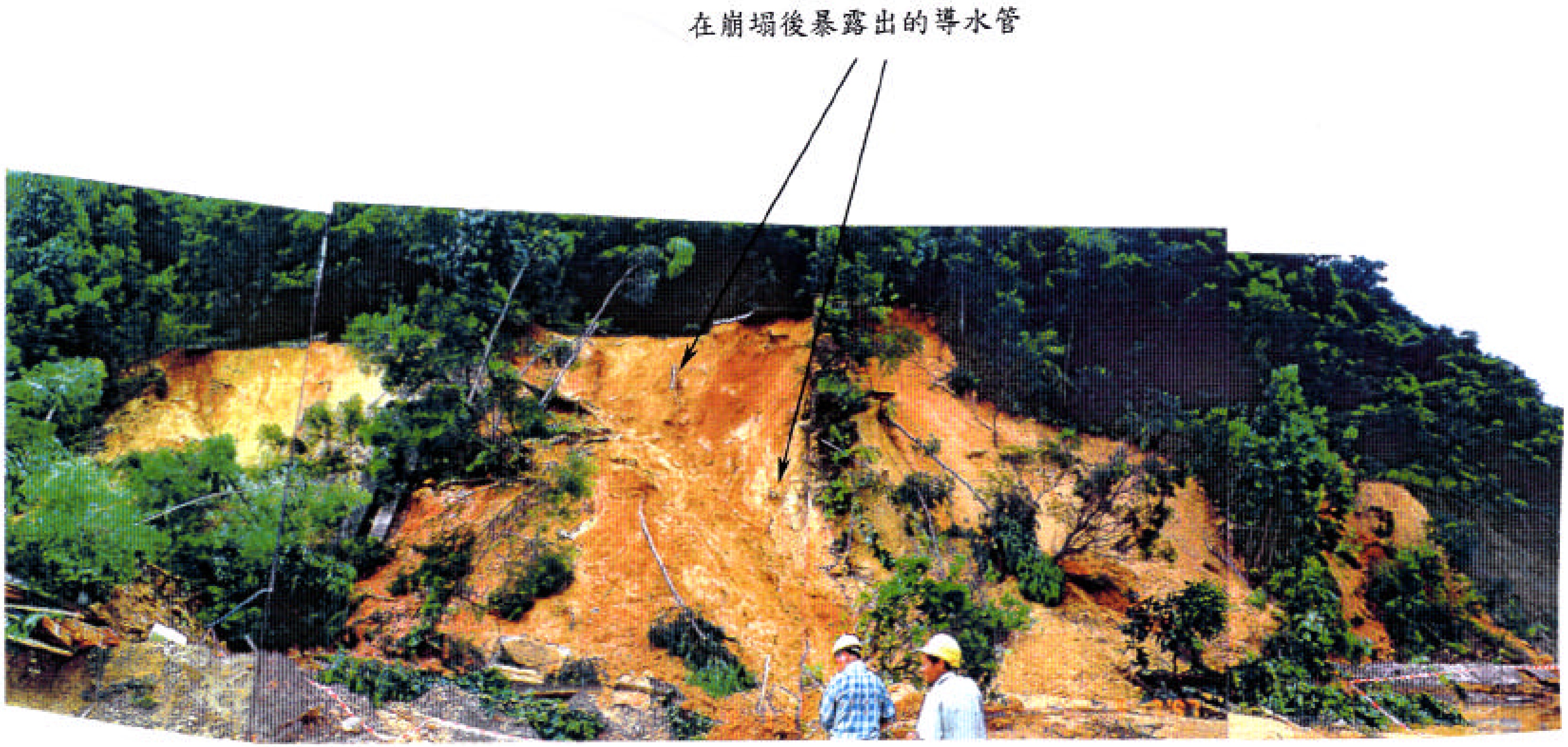
(1) 一九八八/八九年的地形取自一九八八/八九年度防止山泥傾瀉計劃的圖則.
(2) 一九六三年的地形獲取自攝影測量分析.

圖19 － 剖面 B-B 的一九五四年及一九九七年山泥傾瀉的地形及假設崩塌面

照片

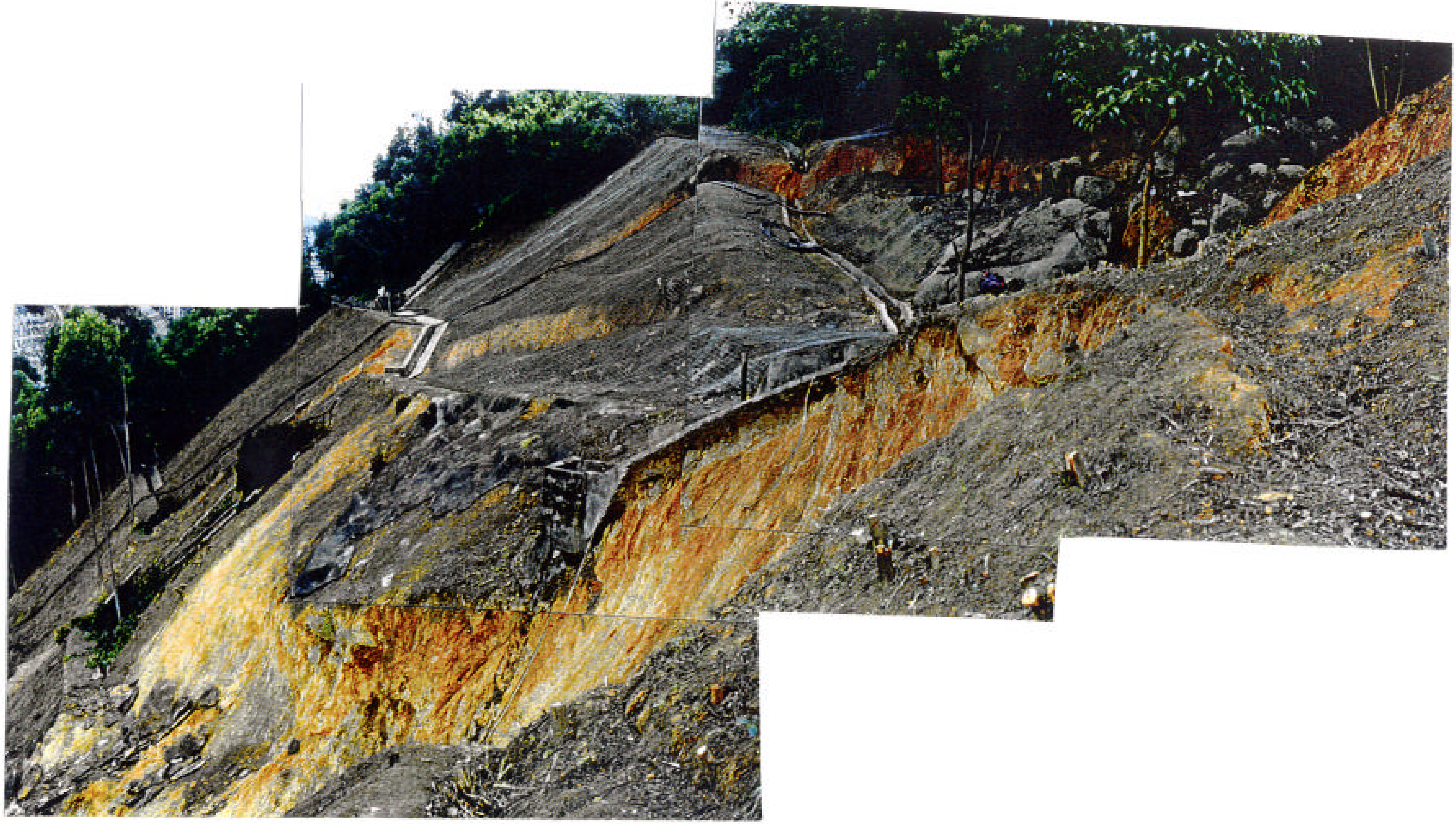
照片1 - 一九九七年八月三日山泥傾瀉的現場景象
（一九九七年八月八日拍攝）

照片2 － 一九九三年九月十五日攝得的編號 11NW-A/C55 削土坡的照片(照片摘錄自土力工程處文件編號 GCD2/A1/11NW-A/C55)



照片3 － 一九九七年七月七日山泥傾瀉的現場景象(一九九七年七月九日拍攝)

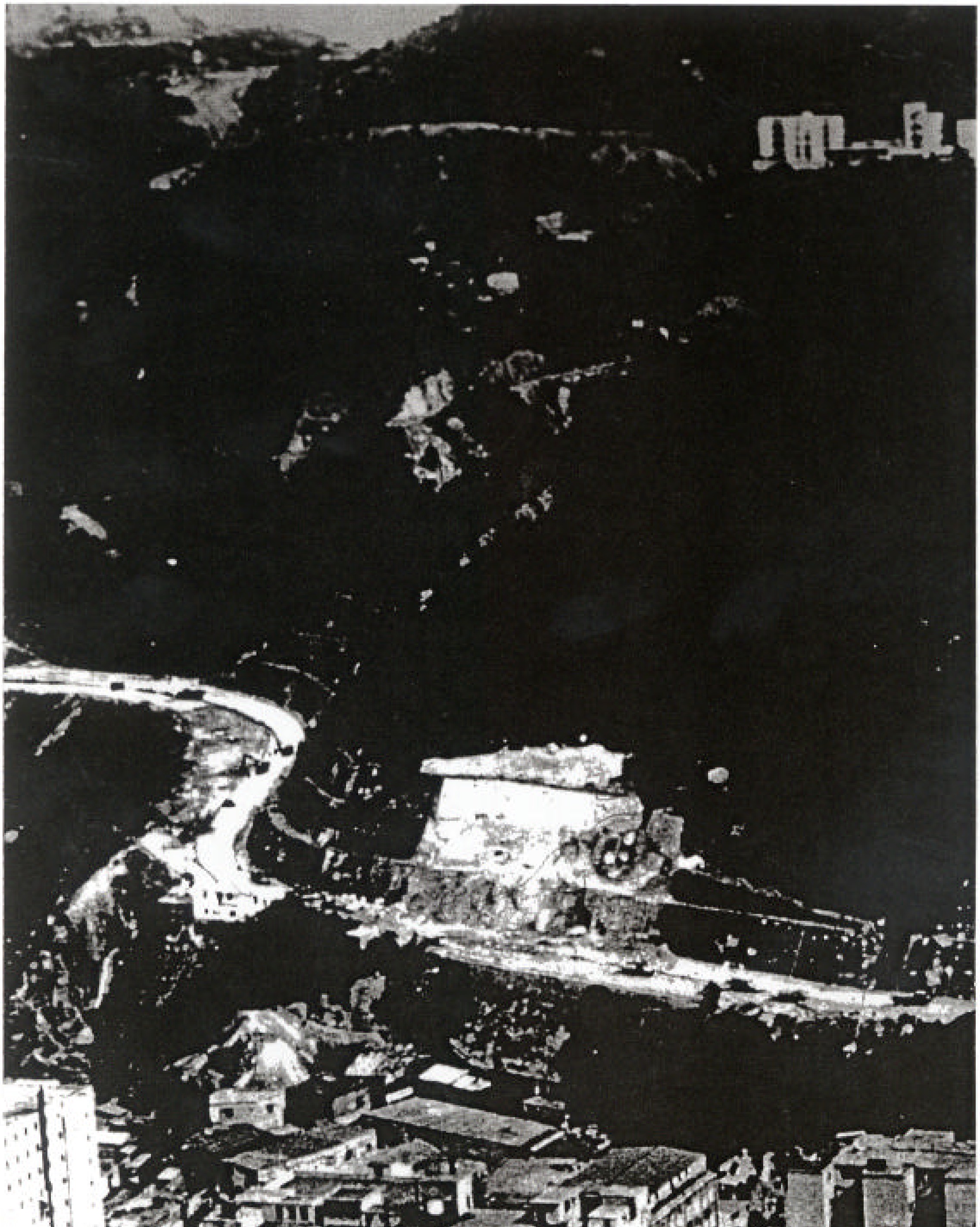
照片4 － 一九九七年七月十七日山泥傾瀉的現場景象(一九九七年七月十七日拍攝)



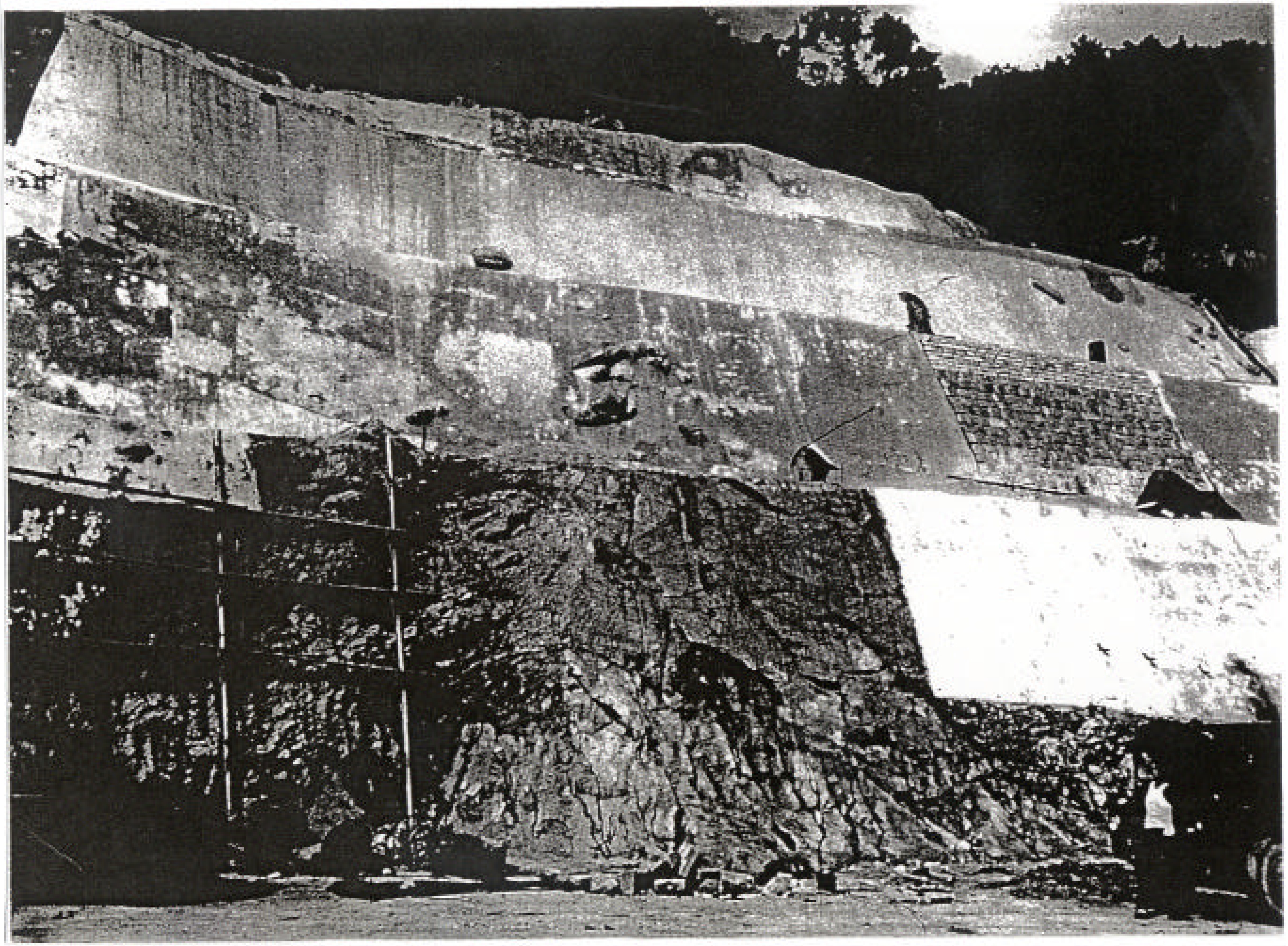
照片5 － 坡體裂隙和移動擴展後的現場景象(一九九七年七月三十日拍攝)

照片6 － 從呈祥道拍攝到的一九九七年八月三日山泥傾瀉後的現場景象(一九九七年八月三日拍攝)

照片7 － 從編號 11NW-A/C55 削土坡上拍攝到的一九九七年八月三日山泥傾瀉後的現場景象
（一九九七年八月三日拍攝）



照片8 － 一九七二年山泥傾瀉後的現場景象(照片摘錄自茂盛一九七二年的呈祥道初步報告的圖一)



照片9 – 一九七二年山泥傾瀉後的善後工程的景象(照片摘錄自茂盛一九七二年的呈祥道初步報告的圖二至四)

地下水大量流出

在一九八二年九月六日觀察到滲水的大致範圍

照片10 – 一九八二年山泥傾瀉後的現場景象(照片摘錄自土力工程處一九八二年的追蹤報告)

照片11 － 照片顯示編號 11NW-A/C55 削土坡中部在山泥傾瀉防止工程進行中的現場景象
（一九九一年十月四日拍攝）

照片12 - 照片顯示水不斷從安裝在山泥傾瀉殘痕中部的導水管流出(一九九七年一月八日拍攝)

附錄 A

事發地點歷史摘要

目 錄

A.1 航空照片判釋的事發地點的發展

下面討論到具體的斜坡地點、地區顯示在圖A1。

現存最早的有關一九九七年山泥傾瀉事發地點的航空照片，攝於一九二四年。 當時，山腳地點亦即在編號 11NW-A/C55 及 1NW-A/C56 斜坡的所在地已經很明顯地正被開挖而作為取土區使用。

航空照片 編號5
Y00103

其次的早期的航空照片是一九四五年拍攝的，顯示這些在編號 11NW-A/C55 及 11NW-A/C56 斜坡所在地的坡腳位置正在被開挖，開挖的地方自一九二四年以來一直在擴大。一條大且深的沖溝在B 區發現，被面狀分布侵蝕的深溝槽包圍著。 一個大的滑坡崖在D 區發現（新近山泥傾瀉事發的地點 ），可能是由斜坡坡腳的遷移所造成的。

航空照片 編號
Y00603-4（20000'）

到一九四九年，進一步的沖溝侵蝕在B 區發生。 在C 區及D 區的坡腳，有証據顯示開挖在繼續。但是，很明顯，D 區斜坡上的滑坡崖與在一九四五年航空照片上看到的沒有變化。滑坡崖上的垂直向下的移動估計少於 3 米。 另一個滑坡崖(部分的）在D 區的東部發現。

航空照片 編號
Y01731-2（8000'）

一九五四年的航空照片顯示，相比早期的航空照片有一個更大的滑坡崖在D 區斜坡上形成。 原來在一九四五年及一九四九年的航空照片上觀察到的滑坡崖的範圍，在這更大的滑坡崖的範圍內仍然可以觀察到。 地面排水溝被安置在沿著原始滑坡崖的邊緣，斜坡的下部安裝了人字型的排水系統。 斜坡的底部可以看到建築物。 崩積物能夠在地形窪地的上部觀察到。

航空照片 編號
Y02683-4

在一九五九年的航空照片上，事發地點，除了植被覆蓋增加了，沒有明顯的變化。

航空照片 編號
Y04617-18（40000'）

一九六一年的航空照片顯示，在C 區有進一步的開挖，以及在B 區的沖溝邊緣的上部及北側有較輕微的侵蝕。

航空照片 編號
Y04905-06（30000'）

到一九六三年，蝴蝶谷村在B 區的大沖溝的底部建造完成。 B 區的大沖溝擴展到上坡。 C 區的開挖已停止了。 D 區的西部有細小植物在生長。 可以見到在斜坡底部很斜的斷面証明有一個小的擋土牆或者坡腳非常斜的削土坡。 拱型

航空照片 編號
Y08751-2（3900'）
Y08101-2（2700'）

的滑坡崖，在一九四五年及後來的航空照片上看到的，仍然明顯存在。 表示沒有進一步的山泥傾瀉，也沒有斜坡受到侵蝕。

公共工程署在一九六三年和一九六七年期間修建呈祥道，要求在 A 區至 D 區上挖方修建斜坡。 一九六七年的航空照片顯示，呈祥道上方的削土坡截去 B 區沖溝的下部和 D 區的滑坡地點。臨近呈祥道開挖形成了一個小的平台，上部過往發生滑坡的段落沒有發生變化。 滑坡崖的東段在呈祥道的上部仍然可以看到，顯示過往的滑坡體仍然保留在原地。

航空照片 編號
Y08063-4
Y13415-6

一九六九年的航空照片顯示平台沿著覆蓋著濃密植被的崩塌崖的底部展布。 在這些航空照片上可以看到崖坎的總體的形狀。

航空照片 編號
Y14877-8

一九七二年拍攝的二套航空照片顯示編號 11NW－A/C55 斜坡被往後開挖過及改變了斜坡的形狀。

航空照片 編號
2282-3 (13000')
273-4

一九七三年的航空照片顯示在呈祥道的下方編號 11NW－A/C55 斜坡的前部有中等嚴重的侵蝕溝槽。 這些航空照片顯示一九七二年善後工程並沒有從有關斜坡上移走過往的滑坡體（一九四五年，一九五四年，一九七二年）。

航空照片 編號
6888-9 (3000')

一九七四年和一九八一年的航空照片上，觀察到有關斜坡及臨近的斜坡並沒有明顯的變化。

航空照片 編號
14670-1 (2000')
24127-8 (4000')
30144-5 (4000')
36762-3 (25000')
36601-2 (5500')

一九八二年的航空照片顯示，編號 11NW－A/C55 斜坡頂部的坡台的上方發生了滑坡和編號 11NW－A/C56 的削土坡發生了一個較大的滑坡。 在 B 區的沖溝的頂部也可以看到兩個小的滑坡。

航空照片 編號
44534-5 (10000')

在一九八三年的航空照片上看到，善後工程已經完成。包括將編號 11NW－A/C56 的斜坡修葺成較緩的坡度，移走編號 11NW－A/C55 斜坡上部的滑動部分，同時將斜坡上部削製，在斜坡上部新建了兩個傾斜面。

航空照片 編號
51667-8 (20000')

一九八四年和一九八七年的航空照片上，觀察到有關斜坡及臨近的斜坡並沒有明顯的變化。

航空照片 編號
54013-4 (4000')

A06287-8 (4000')
A04472-3 (4000')
A095399-40 (4000')

一九八八年的航空照片顯示有關的斜坡普遍缺乏維修，暗色的銹斑分布在斜坡的東段（第一級坡台下面）和坡台上有濃密的植被。

航空照片 編號
A14738-9 (4000')

一九九零年的航空照片顯示，改善斜坡工程正在進行中。 斜坡向上延伸到的地點通過清除植被有 3 米直徑的孤石暴露。 這些孤石在一九四九年的航空照片上看得到的。 濃密的植被仍然保留在斜坡的東段坡面上（第一級坡台和第二級坡台之間）。

航空照片 編號
A23578-9 (4000')
A23638-9 (4000')

一九九一年的航空照片顯示，斜坡最近正在進行重要的整修。 斜坡底部的圍欄和起重機的出現，表示工程仍然在進行中。 斜坡的東段的上部過往清除過植被的地方又長出了植被。

航空照片 編號
A27504-5 (4000')

到一九九二年，整個斜坡安裝了表皮排水系統。斜坡東段的上部（過往孤石暴露的地點），移走了所有的暴露的孤石後，斜坡的坡度變得緩和了。 在第一級坡台土方的平台被加寬。 斜坡中段在第一級坡台的下方完成了坡面重新修整－鄰近的斜坡布滿了較輕的植被。 斜坡鄰近通行階梯的地區（斜坡的東部），在第三級坡台往下到第二級坡台上方的平台之間，顯示出可能是由於水沿著階梯流動而造成的輕微侵蝕的跡象。

航空照片 編號
CN3085-6 (3000')
A32728-9 (4000')

一九九三年的航空照片顯示，斜坡的中段（第二級坡台至第四級坡台之間）完成了坡面的重新整修，而鄰近的段落仍保留著較輕的植被，這次斜坡坡面的重新整修可能是對一九九三年山泥傾瀉的反應。 第一級坡台下面的某段落（在一九九二年重修過坡面），在一九九三年完成了新的坡面修整。 斜面重修的形狀（坡皮剝除）表明近期剛完成對這個斜坡的進一步的勘察。 斜坡底部鄰近通行階梯的地方(平台上方第二級坡台和第三級坡台之間）顯示進一步的侵蝕的証據，可能是由於增加了水流沿著階梯向下流動的緣故。 斜坡的最東部的底部（第一級坡台下面）完成了坡面的重修。 斜坡的中段的總外貌是，這個區的水流比排水網能提供的流量大。

航空照片 編號
A36119-20 (4000')
CN4778-9 (5000')

一九九六年的航空照片顯示，有關的斜坡上方的山丘的上部清除了植被。 新的斜坡保護，在有關斜坡的下面的兩級坡台上可以觀察。

航空照片 編號
CN15581-2 (5000')
CN13552-3 (4000')

A.2 過往的評估

A.2.1 斜坡登記

政府委聘以製備削土坡、填土坡及擋土牆目錄（現通稱為斜坡記錄冊）的顧問：賓尼組合（香港）顧問工程師（賓尼組合）於一九七七年六月登記削土坡為編號 11NW－A/C55。

賓尼 編號
11NW-A/C55
斜坡的現場紀錄表

A.2.2 防止山泥傾瀉計劃（LPM）

一九八七年八月，跨部門防止山泥傾瀉委員會將削土坡 11NW-A/C55 和 11NW-A/C56 包括在一九八八/八九年度防止山泥傾瀉計劃內的"高經濟危險"類別。

土力工程處文件
GCD2/A1/11NW-A/C56內的備忘錄M1

一九八九年，土力工程處的完成對斜坡進行工程地質研究，建立斜坡的工程地質模型，用以對斜坡進行穩定性分析以及加固工程的設計。 同時，針對腐泥土和輕度風化岩石中受不連續面控製的破裂對斜坡穩定性的影響，初步評估斜坡的穩定性。

土力工程處報告
(GCO，1989b)的第1.1部份，第4段

為此而進行的場地勘察包括十四個鑽孔，並在孔內安裝了二十七個測壓計；七處豎向和六處水平向灰土坡面剝除作為第三階段(Stage 3)的研究，場地勘探工程在一九八八年一月至四月間進行(場地勘探工程分佈情況載於圖10)。

土力工程處報告
(GCO，1989b)的第3.2.1部份，第1段

於一九八八年一月至六月期間完成的工程地質研究報告，建立了斜坡的工程地質模型，並就斜坡加固工程的設計提出了如下建議：

土力工程處報告
(GCO，1989b)的第6部份

(a) 對斜坡進行的詳細穩定性分析需考慮斜坡腐泥土內殘餘的不連續面可能引發的不穩定；

(b) 進行斜坡的重建工程前，需對斜坡進行系統地調查，以確定不良的定向不連續面；

(c) 穩定性分析時應考慮玄武質岩脈的低抗剪強度及其構造控製作用對斜坡穩定性的影響；以及

(d) 對鑽孔測壓計進行更長時間的監測，以確定精確的地下水模型。 特別要注意監測豪雨期間的測壓計讀數，以確定斜坡中是否存在間斷性的上層滯水位。

報告認為，對於地下水的狀態並不完全了解，而且在已有測壓計監測數據和先期的斜坡不穩定記錄中存有矛盾和不一致性。 例如，報告指出〝斜坡 D 區內的鑽孔測壓計測得的地下水位數據相互矛盾，並且最重要的是與降雨事件不一致。 例如:一九八八年七月十九日至二十日，一場較大降雨之後（超過 154 毫米），監測到鑽孔內地下水乾涸；而在相對乾燥的九月和十月，監測到的地下水位卻逐漸上升。 然而，編號 DH12 鑽孔內的測壓計所測得的地下水數據與歷次現場調查和從過往照片中所發現的斜坡滲水可能相互關聯。〞

土力工程處報告(GCO，1989a)中第4.2.3部份第5段

另外，報告在評估斜坡的過往不穩定記錄時，注意到對〝乾燥〞崩塌的描述與所採用的加固措施並不一致。 報告指出〝排水斜管在穩固斜坡方面的實際作用可能與所報道的一九七二年的‘乾燥六月’不一致〞。

土力工程處報告(GCO，1989a)第5.2部份第2段

報告確認了腐土帶內含有大量的殘餘節理。 對斜坡進行的運動穩定性評估顯示，除A2 區和B 區外，斜坡發生倒塌破壞以外受不連續面控製的簡單破壞類型破壞的可能性較小。 但在斜坡B 區，則顯示楔形破壞具動態可能性。

土力工程處報告(GCO，1989a)第6部份第5段

一九八九年九月，土力工程處完成了在一九八八/八九年度防止山泥傾瀉計劃下，評估斜坡 11NW-A/C55 和 11NW-A/C56 穩定性的第三階段報告(S3R 11/89)。 研究工作包括現場調查、坡面工程地質測繪、場地勘察、斜坡穩定性分析以及斜坡改善工程的設計。

土力工程處報告(GCO，1989b)

土力工程處報告(GCO，1989b)中圖3K和9

第三階段報告採用了下列強度參數進行斜坡穩定性分析:

(a) 殘積土/漸變土 － C’= 2 kPa，Ø’= 37 度；

(b) CDAp － C’= 7.2 kPa，Ø’= 40 度；

(c) CDB － C’= 10 kPa，Ø’ = 36 度；

(d) CDG － C’= 9 kPa，Ø’ = 39 度； 以及

(e) HDG － C’= 15 kPa，Ø’ = 39 度。

為建立地下水模型，於一九八八年六月至一九八九年七月期間，對鑽孔內的測壓計進行了監測。 報告指出〝除位於斜坡東部的鑽孔 DH10， DH12－DH15 內的測壓計以外，其它測壓計讀數均顯示鑽孔乾涸。〞 鑽孔 DH12 位於近期破裂地點的頂部，測壓計錄得的最高地下水位為 46 mPD，大約位於地面以下 11 米。 設計斜坡加固工程時採用監測到的最高(DH12 測壓計)的地下水位。

土力工程處報告(GCO，1989b)中第3.3部份第1段

在近期崩塌地點的高水位危機部份沒有進行過穩定性分析。6－6 剖面亦橫過這次一九九七年崩塌地點的東側，但相對於這危機部份有不同的局部地下情況及較少的嚴重。 第三階段報告發現 6－6 剖面的原有安全系數為 1.14，改善工程計劃將斜坡的安全系數提高至 1.20。

土力工程處報告(GCO，1989b)中第3.4部份第2段

為使現有斜坡的穩定性符合現行工程標準，第三階段報告（Stage 3 Study Report）建議：

土力工程處報告(GCO，1989b)中第4部份第1段

(a) 進一步監測地下水位，以檢驗所採用的地下水模型的正確性；

(b) 削製土質斜坡部份，修建地面排水渠並在坡面噴植草；

(c) 加固岩質斜坡，包括修整岩塊、安裝岩石銷釘、修建混凝土扶牆、以噴射混凝土防護坡面；以及

(d) 針對可能發生的由不連續面控製的破裂，詳細檢查斜坡材料，並重新評估斜坡的地質模型。

路政署的顧問工程師為路政署一九九五年十二月對斜坡編號 11 NW-A/C55 進行調查時，發現疏水孔被植物部份堵塞，地面排水渠中也產生了中等程度的裂隙和部份堵塞。 路政署的顧問工程師建議一些常規的維修包括清理排水渠，修理破裂/破壞的排水渠及清除淤塞的疏水孔及對斜坡每三年進行一次檢查。 但在對斜坡編號 11NW-A/C55 的工程師檢查，路政署的顧問工程師沒有根據岩土指南五中的建議而〝証實該斜坡之前有否做過穩定性評估〞及沒有確定穩定性評估後的變動會否影響該斜坡的穩定性。

斜坡/擋土牆記錄表(5張)及工程師檢查記錄表(9張)，輝固、芾碩、高鋒組合，一九九五年十二月七日

路政署根據一九九五年調查中的發現，路政署向防止山泥傾瀉委員會建議將此斜坡與其它 1010 個斜坡(包括編號 11NW－A/C55 斜坡)一起列入五年加速防止山泥傾瀉計劃中。 然而，因為編號 11NW－A/C55 斜坡剛於近期完成了防止山泥傾瀉工程，所以並沒有被選中。

路政署發給土力工程處的備忘錄，一九九六年五月二十日;在一九九七年二月八日舉行的防止山泥傾瀉委員會會議

A.2.3 <u>過往的山泥傾瀉對削土坡 11NW-A/C55 的影響</u>

正如在本報告別處提到的，修建呈祥道之前，斜坡是山泥傾瀉的事發現場，呈祥道修成之後，削土坡分別於 1972 年， 1982 年，和 1993 年三次發生山泥傾瀉，這些山泥傾瀉的大致位置載於圖A1。

1972 年的山泥傾瀉是在重大的暴雨結後兩天發生的，使得大約 7500 立方米的泥石堆積在路上，並堵塞了呈祥道一個行車道。

(茂盛，1973a)

在斜坡的新削面上灰泥之後，當局觀察並測到斜坡在移位，在整個重修的削土坡的第一與第二個傾斜面上安裝水平排水孔後，移位停止，任一排水孔錄得最大的水流流出速率大約是每小時 10 加侖。

1982 年，當相鄰的削土坡 (11NW-A/C56) 發生山泥傾瀉之後的搶修工程正在進行時，在削土坡 11NW-A/C55 的頂部發現張裂隙, 並在惡化中。

土力工程處發給路致署的備忘錄，一九八二年九月三日

削土坡最上面的傾斜面被削後 3.8 米並重舖或維修引水導以預防進一步的惡化。

土力工程處發給路政署的備忘錄，一九八二年九月三十日

1993年7月，土力工程處接到山泥傾瀉報告並勘察了現場, 山泥傾瀉發生在第三個傾斜面且規模小，崩塌的泥石體積不超過 9 立方米，山泥傾瀉的泥石部份堆在坡台上和削土坡前的空地上。 該突發事件報告稱〝滲透可能是山泥傾瀉的起因″，並且推薦修整滑坡面及利用排水孔提供表面保護。

土力工程處事件報告，一九九三年七月二十七日

A.3 参考書目

土力工程處 (一九八九a). Engineering Geology Study of Slopes 11NW-A/C55 & C56，Ching Cheung Road. 香港土力工程處，199頁加9幅圖，Advisory - Report no. ADR 1/89 (沒有公開發行)。

土力工程處 (一九八九b). Cut Slopes 11NW-A/C55 & C56, Ching Cheung Road. 香港土力工程處，80頁加8幅圖，Stage 3 Study Report no. S3R 11/89 (沒有公開發行)。

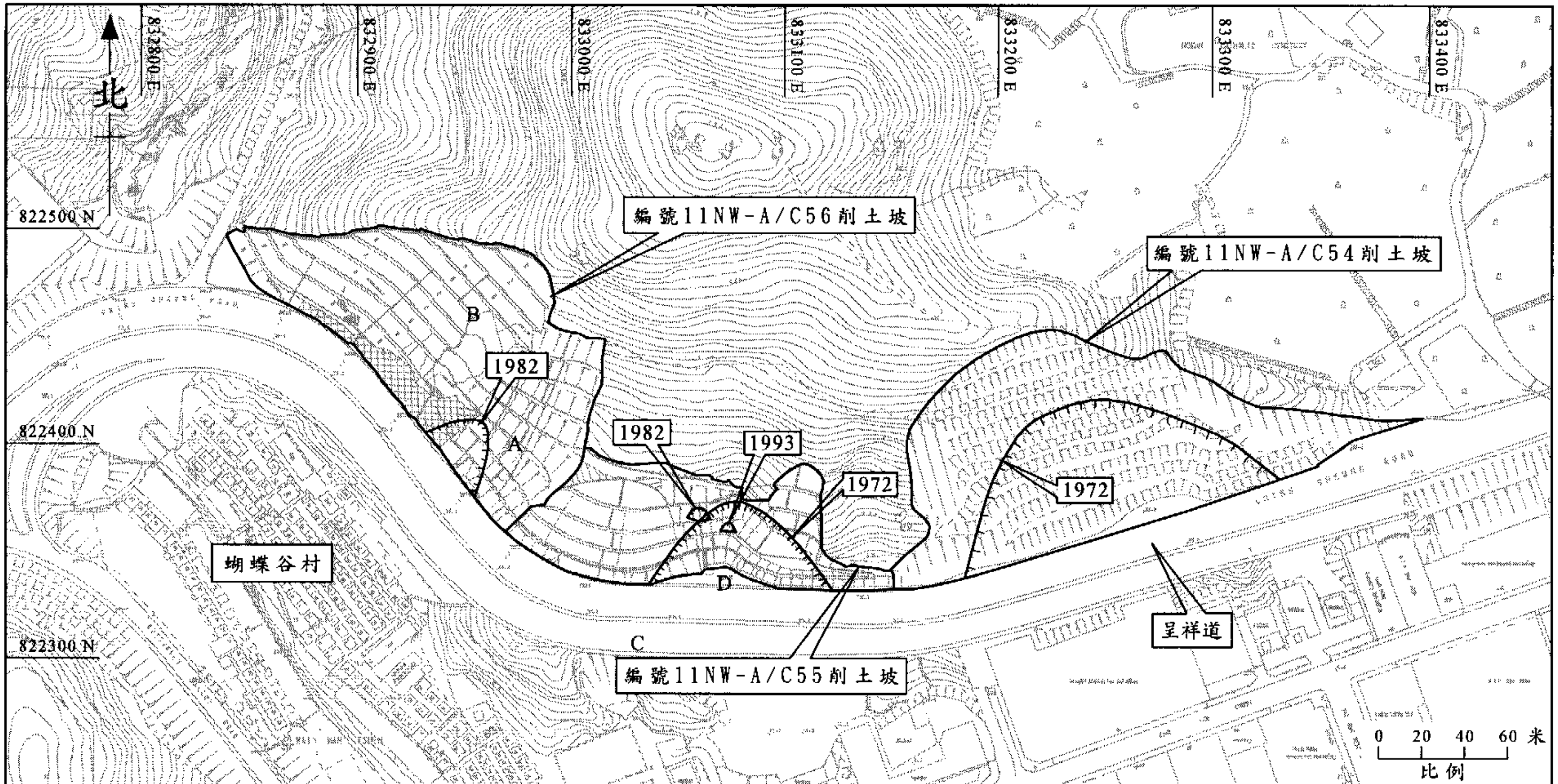
土力工程處 (一九九五). Guide to Slope Maintenance (Geoguide 5). 香港土力工程處，92頁。

茂盛亞洲顧問公司(一九七二). Interim Report on Remedial Works for Landslides on Ching Cheung Road. Report to Public Works Department， 香港， 8頁加 3幅圖(沒有公開發行)。

茂盛亞洲顧問公司(一九七三a). Report on Landslides on Ching Cheung Road. Report to Public Works Department， 香港政府， 34頁加4附錄及19幅圖(沒有公開發行)。

圖

圖號		頁數



圖例： 山泥傾瀉殘痕(帶年)

註： (1) 山泥傾瀉殘痕的大概位置，一九九二年山泥傾瀉殘痕只是像征性的.
(2) 在呈祥道修建以前，曾經發生過山泥傾瀉的位置沒有標示在圖中.
(3) A, B, C及D的位置在附錄A的航空照片判釋事發地點的發展處提及過.

圖A1 - 過往幾次的發生在編號 11NW-A/C55 削土坡上
及周圍削土坡上的山泥傾瀉的地點