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萬佛寺山泥傾瀉事件報告
REPORT ON THE LANDSLIDE AT TEN THOUSAND BUDDHAS' MONASTERY OF 2 JULY 1997

土力工程處報告系列第 77 號
GEO REPORT No. 77

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
CIVIL ENGINEERING DEPARTMENT
THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION

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PREFACE

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

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

R.K.S. Chan
Principal Government Geotechnical Engineer
November 1998

FOREWORD

This GEO Report presents the forensic investigation of the 2 July 1997 fatal landslide at Ten Thousand Buddhas' Monastery 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.

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

ABSTRACT

On 2 July 1997, a landslide occurred at a slope above Ten Thousand Buddhas' Monastery, Shatin, and resulted in one fatality and one slight injury. The landslide involved the sudden collapse of part of registered cut Slope No. 7SW-B/C116 and part of the hillside above. A comprehensive investigation into the landslide was carried out for the Geotechnical Engineering Office (GEO) during the period July 1997 to February 1998 by GEO's landslide investigation consultants, Halcrow Asia Partnership Ltd. (HAP) with input and assistance from the GEO. This investigation 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 1997 landslide at the Ten Thousand Buddhas' Monastery was probably caused by elevated water pressure along adversely orientated discontinuities in the weathered granite of the slope, following the heavy rainfall that immediately preceded the failure. A lack of design and maintenance of the cut slope prior to the failure had probably resulted in local progressive slope deterioration.

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

CONTENTS

1. INTRODUCTION

In the morning of 2 July 1997, a landslide occurred at a slope within the grounds of Ten Thousand Buddhas' Monastery, Shatin (Figure 1). A building known as Kun Yam Din was damaged and an adjacent annex building was buried by the landslide debris. The landslide resulted in one fatality in the annex building, and one person in Kun Yam Din was slightly injured.

After the landslide, the Geotechnical Engineering Office (GEO) of the Civil Engineering Department (CED) commenced a detailed investigation into the failure. The investigation was undertaken by GEO's landslide investigation consultants, Halcrow Asia Partnership Ltd. (HAP), with geological input from the Hong Kong Geological Survey (HKGS) and assistance from other Divisions within GEO.

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

(a) review of all known relevant documents relating to the development of the site and the sequence of events leading up to the landslide,

(b) analysis of rainfall records,

(c) interviews with witnesses to the landslide,

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

(e) geological mapping,

(f) execution of a programme of ground investigation by drilling, trial pitting, in situ testing and laboratory testing,

(g) theoretical stability analysis of the slope that failed, and

(h) diagnosis of the probable contributory 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

On 2 July 1997, five separate landslides occurred at Ten Thousand Buddhas' Monastery. Four of the landslides affected a cut and fill platform, where Kun Yam Din was located (Figure 1 and Plate 1). According to the District Lands Office, the landslide that resulted in the fatality occurred entirely within the boundaries of private lot Nos. 323 and 324

(Figure 1). Part of the cut slope (No. 7SW-B/C116) behind Kun Yam Din was involved in the landslide, as well as part of the hillside above the cut slope (Figure 2).

The elevation of the platform was about 101 mPD. Before the landslide, the toe of the cut slope behind Kun Yam Din was located about 4 m from the building. The maximum height of the cut slope at this location was about 15 m, with the lower 5 m and the upper 10 m being inclined at about 75° and 56°, respectively. Adjacent unfailed sections of the cut slope, remaining after the landslide, were observed to be covered with highly deteriorated chunam, which was severely cracked and had become detached in many places. Unplanned vegetation, including mature trees, had established on the cut slope.

The natural hillside above the cut slope was densely vegetated. It rose at an average inclination of about 36° from the crest of the cut slope to the top of the hill at about 220 mPD. A major stream course passes the northeast end of the platform (Figure 1). On the 1904 topographic map (Ordnance Survey Office, 1904) five minor stream courses are shown crossing the site where the platform was later constructed, one of which appears to have been near the site of the landslide that caused the fatality. These five streams are not shown on subsequent survey maps, but aerial photographs taken in 1924 indicate three possible stream courses, located within about 20 m of where Kun Yam Din would later be constructed (see Figure A1, Appendix A).

The single storey annex building, measuring about 8 m long and 4 m wide, was joined to the northeast corner of the two storey Kun Yam Din building. There are no records of the buildings at the Buildings Department but, from inspections, they were evidently constructed with brick walls and the latter had a reinforced concrete platform roof.

The remains of an old, infilled, man-made concrete drainage channel was found adjacent to and truncated by the southwest flank of the landslide (Figure 2). The channel was located along a 2 m wide track that had originally been formed as a shallow cutting, but which had subsequently become almost completely obscured over a period of years by soil and vegetation debris. The track and channel were not found on the northeast side of the landslide. No other man-made surface drainage channels were present in the cut slope or hillside behind the platform.

3. DESCRIPTION OF THE LANDSLIDE

A photograph of the landslide that resulted in the fatality, is shown in Plate 2. A plan and a cross-section through the landslide are given in Figures 2 and 3, respectively.

According to the accounts of witnesses, the failure occurred at about 6:15 a.m. on 2 July 1997. The maximum depth of the landslide, measured approximately perpendicular to the pre-failure ground surface, was about 6 m on the southwest flank and about 1 m on the northeast flank. About 1 500 m^3 of debris was released during the landslide, leaving a main scarp about 25 m wide with a rupture surface about 25 m long. The landslide debris buried the annex building. The debris piled up to a depth of about 6 m behind Kun Yam Din, causing structural damage to part of the rear wall. The ground floor of Kun Yam Din was partly filled with wet soil and trees from the landslide. The maximum horizontal travel distance of the debris was about 40 m, as measured from the crest of the landslide to the distal end of the debris. The inclination of the line that joins the distal end of the debris and the crest of the landslide was about 32°, which is a typical value for rain-induced landslides of this scale in soil cut slopes in Hong Kong (Wong & Ho, 1996).

The majority of the landslide debris comprised loose, moist to wet, slightly clayey, gravely coarse sand with highly decomposed granite cobbles and small boulders and some moderately to slightly decomposed granite boulders up to 5 m^3 in volume.

During an inspection after the landslide on 2 July 1997, copious seepage was observed issuing from about 2 m below ground level in the main scarp of the landslide (Section 7.4). Seepage flow and surface runoff water resulted in the formation of erosion gullies along both sides of the debris (Figure 2).

4. HISTORY OF THE SITE

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

The earliest available aerial photographs, taken in 1924, show that the site was situated on a southeast-facing hillside and was undeveloped at the time. The platform was formed between 1924 and 1945 by cutting into the hillside, constructing a retaining wall on the downhill side and placing fill behind the wall. The retaining wall was constructed as a dressed block masonry wall with horizontal beams and weepholes, which was up to about 10 m high and extended for most of the length of the platform. The cut slope and three buildings were present on the platform by 1945, including Kun Yam Din with its annex building. The platform was extended to the southwest by 1954, and three of the other six buildings currently located on the platform were built by 1963.

The GEO has no records of previously reported landslides at the cut slopes above the platform since systematic reporting of landslides in Hong Kong began in the early 1980's. From a review of aerial photographs, however, it was found that five landslides probably occurred on the cut slope between 1924 and 1973 (Figure 4). Two areas of possible shallow erosion might also have occurred previously on the cut slope prior to 1963.

Between 1949 and 1961, a small cutting was made for a track traversing above the crest of the cut slope. This appears to correspond to the position of the old drainage channel (Section 2) that was observed during inspections after the landslide (Figure 2).

The cut slope was not identified in the 1977/78 Catalogue of Slopes. In 1996, the cut slope was registered as No. 7SW-B/C116 in a project initiated by the GEO entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST), which aims to systematically update the 1977/78 Catalogue of Slopes and compile the New Catalogue of Slopes.

5. ANALYSIS OF RAINFALL RECORDS

Rainfall data were obtained from GEO automatic raingauge No. N02, the nearest raingauge to the site, located at Shun Wo House on Wo Che Estate, approximately 950 m to the southeast of the landslide. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the Hong Kong Observatory and the GEO. The daily rainfalls recorded by the raingauge in June and July 1997, together with the hourly rainfalls from 29 June 1997 to 2 July 1997, are shown in Figure 5.

Records from two other nearby automatic raingauges, Nos. N01 and N09, which are approximately 2 km to the southwest and to the northeast of the landslide respectively, were also examined. The pattern of rainfall and the intensities recorded at these rainguages were broadly similar to that recorded at raingauge No. N02. The record from raingauge No. N02 is therefore considered appropriate for analysis purposes.

The rain was heavy in the morning of 2 July 1997 to the time of the landslide and continued for most of the day. The 24-hour and 12-hour rainfalls before the landslide were 179 mm and 150 mm, respectively. The maximum 60-minute rolling rainfall was recorded as 118.5 mm between 5:15 a.m. and 6:15 a.m. on 2 July 1997. Analysis of the return periods of the rainfall intensities for the 2 July rainstorm for different durations based on historical rainfall data from the headquarters of the Hong Kong Observatory (Lam & Leung, 1994), shows that between durations of 5 minutes and 31 days, the 60-minute rainfall was the most severe, with a corresponding return period of about 25 years.

The maximum rolling rainfalls for the rainstorm on 2 July 1997 have been compared with the previous most extreme rainstorms in the period from February 1980, when the raingauge began operation, to July 1997 (Figure 6). The maximum rolling rainfall for the rainstorm prior to 6:15 a.m. on 2 July 1997 exceeds that from previous rainstorms for durations of between 20 minutes and 3 hours.

6. SEQUENCE OF FAILURE

The sequence of failure was re-constructed from accounts given by eye-witnesses, including those of five occupants of the Monastery buildings on the platform, from records of the incident by the Hong Kong Police Force (HKPF) and Hong Kong Fire Services Department (HKFSD).

According to the eye-witnesses, the landslide occurred at about 6:15 a.m., during torrential rain, on 2 July 1997. Immediately before the landslide, one person saw a large tree fall onto the annex building whilst others reported hearing a loud bang and the sounds of falling trees. The accounts of eye-witnesses indicate that the landslide occurred suddenly. The fast-moving debris from the landslide buried the annex building in a very short period of time, and a missing person was presumed trapped in the building. The rear wall of the Kun Yam Din was also partly demolished by the landslide and an occupant in a ground floor bedroom was slightly injured as debris entered the building. After the landslide, several eye-witnesses recalled surface water up to 0.3 m deep within the Kun Yam Din building and on the platform.

Following a brief search, the missing person was not found, so the occupants of the buildings on the platform walked to a Monastery Temple lower down the hillside, from where the event was reported to the police by telephone, in a call logged at 6:33 a.m.

7. SUBSURFACE CONDITIONS AT THE SITE

7.1 General

The subsurface conditions at the site were determined using information from desk and field studies. The desk study comprised a review of existing data, whilst the field study included geological mapping and ground investigation.

The rocks at the site were previously mapped by the Hong Kong Geological Survey (HKGS) as coarse-grained granite, according to the 1:20 000 scale geological map for the area (Addison, 1986 and Geotechnical Control Office, 1986). Two faults are shown in the area, one about 300 m southwest of the site, trending northwest to southeast and the other about 160 m northeast of the site, trending west-northwest to east-southeast. No faults are shown intersecting the site. Geological mapping of the slope was carried out by HAP and HKGS, following the failure on 2 July 1997.

Ground investigation began on 12 September 1997 and was completed on 12 November 1997. The investigation comprised 2 vertical drillholes with standpipe piezometers and Halcrow buckets, 6 trial pits and 24 GCO probe holes. Jet-fill tensiometers were installed in six of the GCO probe holes (Figure 7).

7.2 Geology

A geological plan of the landslide area is shown in Figure 8, and a geological cross-section through the landslide site is presented in Figure 9.

The lithology in the area of the landslide consisted of medium-grained to coarse-grained granite, with occasional areas of very coarse-grained pegmatite granite. Weathered granite rock was present in parts of the exposed failure scar, at the toe of the cut slope and in the hillside beyond both flanks of the landslide. Information from trial pits, GCO Probe results and field observations suggest that part of the landslide rupture surface formed along the interface between moderately decomposed granite rock and highly decomposed and completely to highly decomposed granite soil (Figure 9). Boulders of granite rock were present on the surface of the natural hillside.

Thin, discontinuous layers of residual soil and colluvium were typically present to a depth of 0.5 m below the ground surface.

The deepest part of the failure was typically 4 m to 6 m in the center and on the southwest side of the landslide scar. This coincided with a zone of PW 0/30 partially weathered granite, comprising predominantly highly decomposed material. A section through this zone was exposed on the southwest flank of the landslide, where a layer of heterogeneously weathered granite, comprising corestones set in a weaker highly decomposed granite, was typically present within 1 m to 3 m of the ground surface (Plate 3). With increasing depth, the corestones were no longer present and the granite became more homogeneous.

The landslide of 2 July 1997 was largely controlled by pre-existing discontinuities. The surface of rupture was partly exposed in the landslide scar following a pre-existing, highly persistent, undulating and rough joint, dipping to the south-southeast at about 52° and 43° in the upper and lower parts of the main scarp, respectively (Plate 4). The increasing depth of the failure towards the southwest side of the landslide was attributed to the joint striking slightly obliquely to the strike of the natural hillside. Joint wavelengths of up to 5 m and amplitudes of up to 0.8 m were measured, giving a potential maximum roughness angle of about 16°. The characteristic orientation of the joint in relation to the natural hillside, and its persistence and roughness, indicated that it was likely to be a sheeting joint.

The upper part of the landslide rupture surface was formed both along the sheeting joint and through the intact weathered granite, giving rise to a steep curved surface (Plate 5), whereas the largest central section of the rupture surface was formed entirely along the sheeting joint. The lower part, which was exposed in the erosion gully on the northeast side of the landslide and in four of the trial pits, coincided with a set of infrequent, discontinuous, undulating joints which dipped southeast, directly out of the slope, at about 30° (Plate 6). A thin layer of pale grey, slightly sandy clayey silt, up to 20 mm in thickness, was found to partly coat and infill some of these joints.

A major set of sub-vertical veins and relict joints was observed to strike at right angles to the slope throughout the landslide scar and in adjacent rock outcrops (Figure 8). These discontinuities were closely spaced (60 mm to 200 mm apart) on the southwest side of the failure scar, becoming very widely spaced (up to 4 m apart) towards the northeast flank. The veins were infilled with vertical layers of quartz that were slightly open, which would have allowed the flow of seepage water. The apertures of the majority of the joints were also slightly open in the upper part of the main scarp and became closed with increasing depth. Several of these veins and joints formed the sub-vertical face of the exposed southwest flank of the landslide. Close examination of the highly decomposed granite revealed localised zones of extremely closely spaced micro-fracturing, which were of the same orientation as the major set of veins and joints.

Partly infilled sub-vertical cracks, with the same dip direction as the hillside, were observed in the zone of heterogeneous weathered granite exposed in the southwest flank of the landslide (Figure 8 and Plate 7). Typically, the apertures of the cracks were moderately narrow (20 mm to 60 mm) and partly infilled with dry to moist, slightly clayey, sandy silt debris, which was probably derived from material washed-in from the ground surface of the hillside. Occasional relict joints, with similar orientations to the cracks and persistences generally less than about 1 m, were observed in the main scarp. It is thought that these joints may have opened in the past to form the cracks, although in some cases, the cracks have clearly formed along the vertical boundaries between corestones and the weaker highly decomposed granite.

In the center of the main scarp, a block of relatively intact material of about 5 m^3 volume was found to have been displaced by about 3 m down the sheeting joint from its original position at the crest of the landslide (Figure 8). The release surfaces around the back of the block coincided with two relict joints coated with the same debris material that was observed in the sub-vertical, infilled cracks noted on the southwest flank of the landslide. The presence of these infilling materials and cracks is thought to be indicative of minor local slope movement at sometime in the past.

Immediately above the lower part of the landslide rupture surface, a 0.2 m thick layer of soft, wet, medium-brown, slightly clayey, gravelly, silty sand was observed in trial pit excavations (Plate 8). This layer showed obvious signs of disturbance and represented the end product of intensive shearing at the base of the landslide. It appears that a significant proportion of the debris above this layer moved *en masse* as a 'raft' of material in the landslide.

7.3 Soil and Rock Properties

A comprehensive series of geotechnical laboratory tests was conducted on soil and rock samples retrieved during the ground investigation. The tests included particle size distribution tests, Atterberg limit tests, direct shear tests, triaxial compression tests, and permeability tests.

Particle size distribution and Atterberg limit tests were carried out in accordance with Chen (1994). The average fines (i.e. clay and silt) content of the highly decomposed and completely decomposed granite was found to be 13 % and 17 %, respectively. The plasticity index of the fines in these materials ranged from 26 to 39, and the liquid limit ranged from 61 to 74.

Results of the permeability tests carried out in the laboratory on samples of highly and completely decomposed granite provided an average value of about 2×10^{-5} m/s. Permeability values obtained from the falling head tests in boreholes ranged between 7×10^{-4} m/s and 1×10^{-3} m/s, for relict jointed completely and highly decomposed granite.

The shear strength properties of the weathered granite and the pale grey, slightly sandy, clayey silt were assessed by direct shear tests (BS 1377, 1990a) and consolidated undrained triaxial compression tests with pore pressure measurement (BS 1377, 1990b). The test results and the shear strength parameters of the materials, determined from the line of best-fit by the least squares method, are shown in Figures 10 and 11.

Surface roughness of the sheeting joint was estimated from field measurements in accordance with the method recommended by Richards & Cowland (1982). Surface roughness values of between 8° and 14° were predicted which, when added to an assumed basic friction angle for the sheeting joint of 40° (Papaliangas *et al,* 1995), gave effective shear strength parameters of c' equal to 0 kPa and ϕ' ranging between 48° and 54°.

7.4 Groundwater Conditions

Shortly after the landslide on 2 July 1997, copious water seepage was observed issuing from the main scarp at about 2 m below ground surface (Figure 8). The seepage emanated from erosion pipes formed through the soil (Plate 9), and from the slightly open relict joints and quartz veins. The amount of seepage reduced substantially about one week after the heavy rain stopped on 3 July 1997.

The landslide occurred within 4 hours of the heavy rainfall that commenced in the early morning of 2 July 1997. Surface runoff of the rainfall would have been slightly impeded by vegetation and the undulating bouldery slope. It is inferred that surface water had infiltrated rapidly through the discontinuous thin layers of residual soil and colluvium into the partly open cracks, joints and soil erosion pipes present in the 1 m to 6 m of weathered granite above rockhead. The hydrological conditions were therefore favourable to the development of transient groundwater pressure in the weathered granite at times of heavy rain. A best estimate of the likely pressure acting on the failed portion of slope would be a water table about 1 m to 4 m above inferred rockhead, with rockhead being the interface between moderately decomposed granite and highly decomposed granite (Figure 9).

On the day of the failure, water was observed issuing from the base of the retaining wall below the platform, at an elevation of about 90 mPD (Figure 1). The water flowed into an erosion gully formed by a landslide below the platform. About one week later, water was observed issuing from a lower elevation of about 75 mPD, with the stream course above being dry. This change in level probably coincided with the lowering of the transient groundwater table in the hillside.

The two piezometers, installed as part of the ground investigation after the landslide, have been in operation during the dry season and to date, have not recorded any groundwater.

8. THEORETICAL STABILITY ANALYSES

Theoretical stability analyses were carried out to assist the diagnosis of the mechanism and causes of the landslide. These analyses were aimed to investigate the likely operative range of shear strength parameters along the failure surface for possible, different groundwater levels at the time of failure.

The information used in these analyses was obtained from the post-failure ground investigation, fieldwork, laboratory testing, and site observations and measurement. A representative cross-section of the landslide site and the input parameters adopted in the analyses are shown in Figure 12. A transient water table up to 4 m above the inferred rockhead was used in the analyses to simulate the possible range of groundwater conditions at the time of the failure.

The results of the analyses are summarised in Figure 13, for cases of 0 kPa and 12 kPa cohesion (c') with variable angle of shearing resistance (ϕ') operating along the surface of rupture. Zero cohesion is taken as representative of the sheeting joint, which coincides with most of the surface of rupture. A cohesion value of 12 kPa is used to examine the theoretical stability of the slope in the case of failure through intact decomposed granite, without the influence of a pre-existing discontinuity. The results show that for the likely ranges of shear strength parameters for the slope-forming material (Section 7.3), a rise in the groundwater table above rockhead of about 2.5 m to 4.0 m would have been sufficient for failure to have occurred. This suggests that the theoretical stability analyses, for these assumptions, are credible.

9. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

Based on the information collected from this investigation, it is postulated that the failure of the cut slope at Ten Thousand Buddhas' Monastery resulted from three principal factors :

(a) the development of transient groundwater pressures following intense rainfall,

(b) the presence of adverse discontinuities, and

(c) a lack of slope design and maintenance.

The landslide occurred during a major rainstorm that, for durations of between 20 minutes and 3 hours preceding the failure, produced the highest maximum rolling rainfall recorded at the nearby raingauge No. N02, since its installation in 1980. The maximum 60-minute rolling rainfall occurred in the hour immediately before the landslide and was recorded as 118.5 mm, for which a return period of about 25 years is estimated. This return period, whilst relatively large, is not considered to be exceptional.

Copious seepage was observed flowing from several locations about 2 m below ground level in the main scarp on the morning of 2 July 1997 and this indicates that the water table had responded very quickly to the heavy rainfall prior to the failure. It is likely that the presence of pre-existing open soil erosion pipes and partly open vertical cracks, relict joints and veins, that extended to the ground surface in the vicinity of the cut slope, allowed rapid infiltration of surface runoff water into the slope. With a relatively shallow rockhead located between about 1 m and 6 m below the ground level, water pressures probably built up along the sheeting joint and within the near surface jointed and weathered granite. As a result, the available shear resistance along the potential planes of failure would have been significantly reduced.

The failure mechanism of the landslide was controlled by the presence of adversely orientated geological discontinuities in the slope. The majority of the slip surface formed along a highly persistent sheeting joint, dipping at an average inclination of about 52° to the south-southeast. The dip angle and depth of the sheeting joint were such that, whilst it came close to the ground surface, it did not actually daylight in either the cut or hillside slopes. However, the presence of discontinuous low angle joints, daylighting at about mid-height in the pre-failure cut slope, allowed the establishment of the lower section of the landslide rupture surface. A persistent set of vertical veins and relict joints provided the release surfaces along the southwest flank, on the deepest part of the landslide.

The existence of open, partly infilled, sub-vertical cracks exposed in the landslide scarp is evidence of past movement of the slope. At about 55° to 75°, the angle of the cut slope was significantly higher than other unreinforced soil slopes of this height in weathered granite designed to current geotechnical standards. The cut slope, which was created between 1924 and 1945, was probably not up to modern design standards and its formation might have induced significant distress to the hillside above. Downslope movement of the hillside and cut slope during subsequent rainstorms, would have allowed open, sub-vertical cracks to form and dilation to occur along discontinuities such as the sheeting joint. The extremely poor condition of the chunam covering, unplanned vegetation and the lack of any functioning drainage for the slope, possibly since 1963 when shallow erosion occurred (Section 4), indicates a lack of slope maintenance. The resulting infiltration of rainfall and surface runoff water into the slope would have led to progressive deterioration of the weathered rock mass.

10. CONCLUSIONS

It is concluded that the 1997 landslide at the Ten Thousand Buddhas' Monastery was probably primarily caused by elevated water pressure in adversely orientated discontinuities in the weathered granite of the slopes, following the heavy rainfall that immediately preceded the failure.

A lack of design and maintenance of the cut slope prior to the failure had probably contributed to the local progressive slope deterioration.

11. REFERENCES

Addison, R. (1986). Geology of Sha Tin. Geotechnical Control Office, Hong Kong, 85 p. (Hong Kong Geological Survey Memoir No. 1).

BSI (1990a). BS 1377:1990 Methods of Test for Soils for Civil Engineering Purposes, Part 7 – Shear Strength Testing (Total Stress), British Standards Institution, London, 1990, 48 p.

BSI (1990b). BS 1377:1990 Methods of Test for Soils for Civil Engineering Purposes, Part 8 – Shear Strength Testing (Effective Stress), British Standards Institution, London, 1990, 28 p.

Chen, P.Y.M. (1994). Methods of Tests for Soils in Hong Kong for Civil Engineering Purposes (Phase 1 Tests). Geotechnical Engineering Office, Hong Kong, 91 p. (GEO Report No. 36).

Geotechnical Control Office (1986). Sha Tin: Solid and superficial geology. Hong Kong Geological Survey, Map Series HGM 20, Sheet 7, 1:20 000 scale. Geotechnical Control Office, Hong Kong.

Geotechnical Control Office (1987). Geotechnical Area Studies Programme – Central New Territories. Geotechnical Control Office, Hong Kong, GASP Report no. II, 165 p, plus 4 maps.

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

Ordnance Survey Office (1904). Topographic Map of 1904, Sha Tin, Sheet No. 12. Ordnance Survey Office, Southampton, United Kingdom, 1904.

Papaliangas, T.T., Hencher, S.R. & Lumsden, A.C. 1995. A comprehensive peak shear strength criterion for rock joints. Eighth International Congress on Rock Mechanics, Tokyo, pp 359-366.

Richards, L.R. & Cowland, J.W. (1982). The effect of surface roughness on the field shear strength of sheeting joints in Hong Kong granite. Hong Kong Engineer, vol. 10, no. 10, pp 39-43.

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

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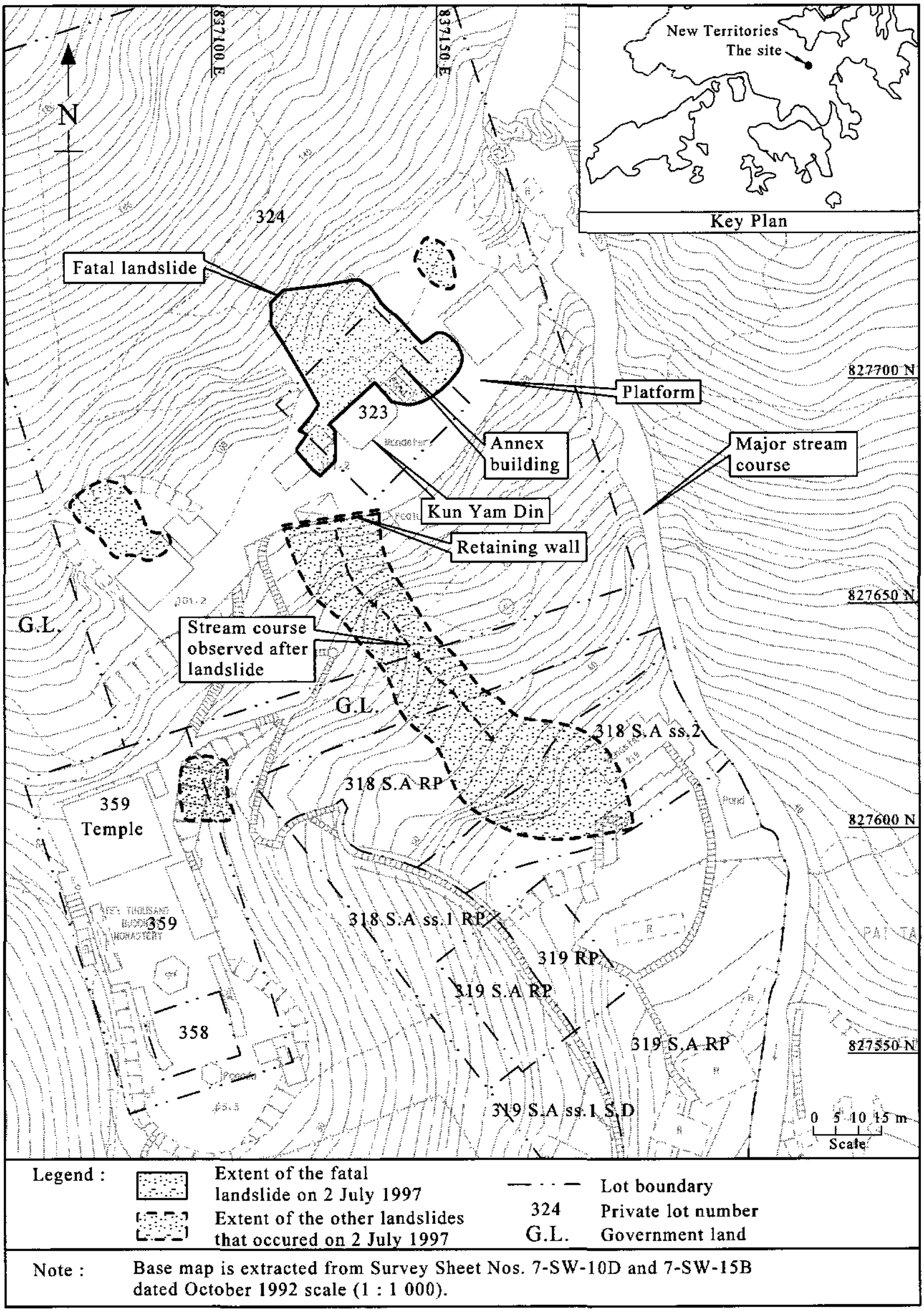
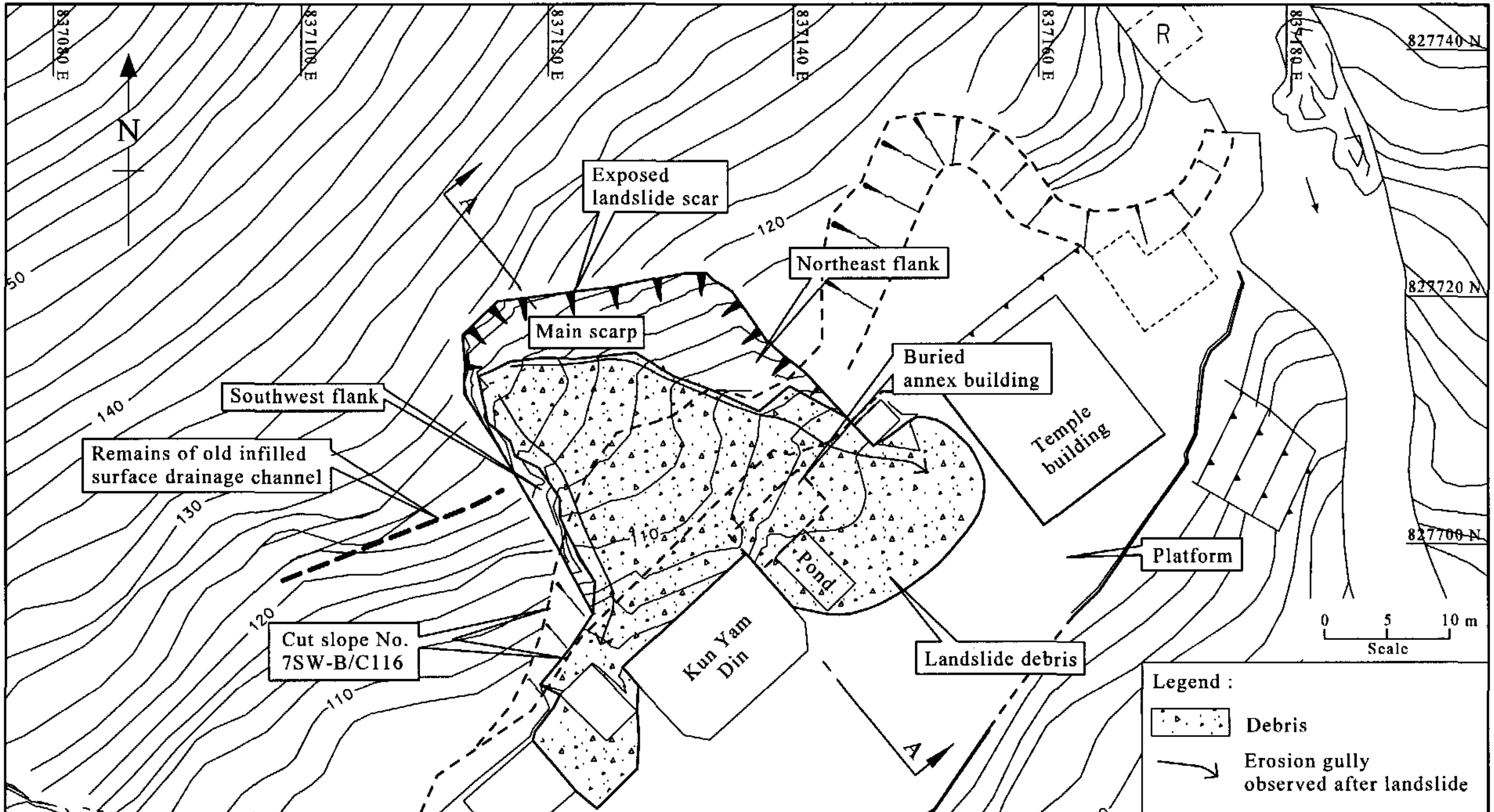


Figure 1 - Site Location Plan



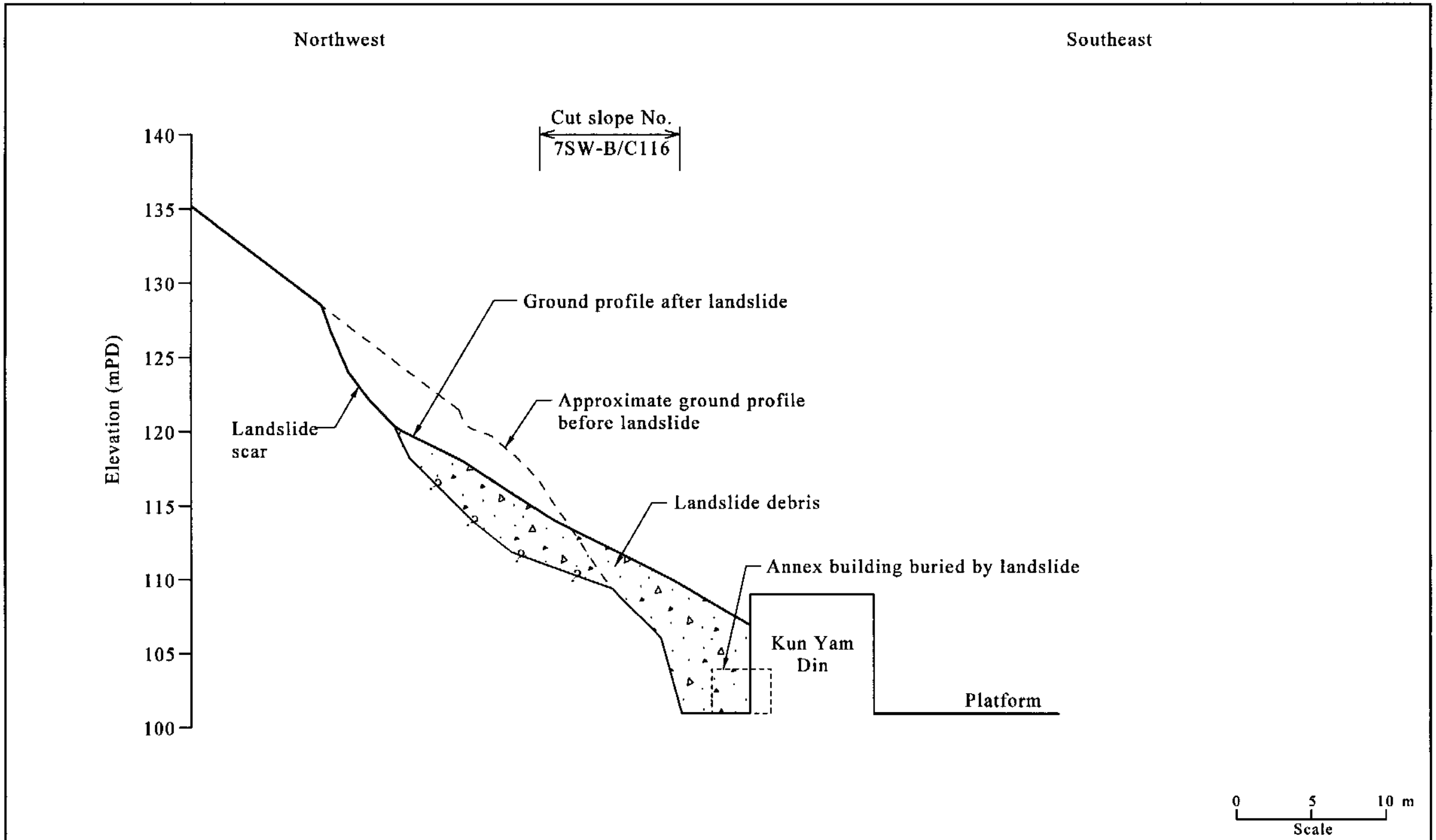
Notes : (1) See Figure 3 for section A-A.

(2) Information shown in this figure is based on topographic survey, geological mapping, field observations and documentary records.

(3) Base map is based on Survey sheet No. 7-SW-10D dated October 1992 (scale 1:1 000) and photogrammetric analysis from 1963 aerial photographs.

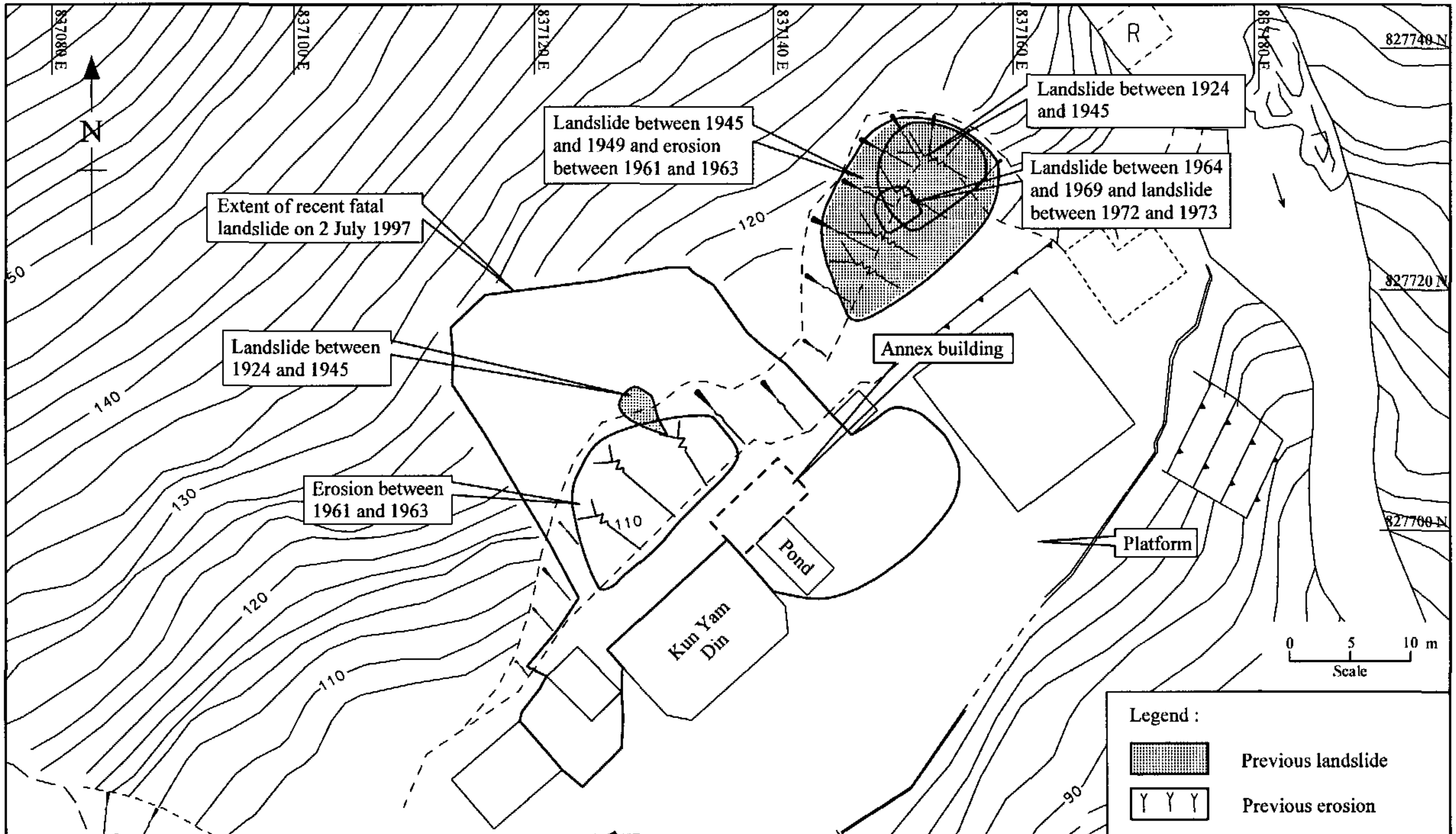
(4) Contours within landslide scar and debris are from topographic surveys carried out by the CED after the landslide.

Figure 2 - Plan of the Landslide



Notes : (1) See Figure 2 for location of section.
(2) Information shown in this figure is based on topographic survey, field observations and documentary records

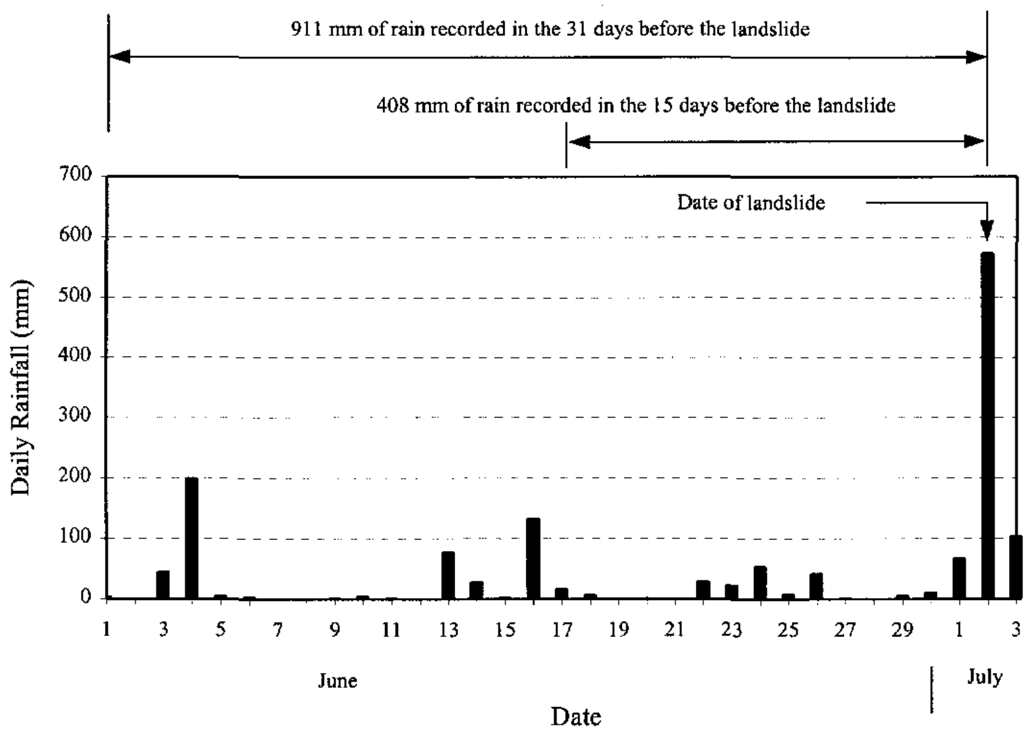
Figure 3 - Section A - A Showing Details of the Landslide Site



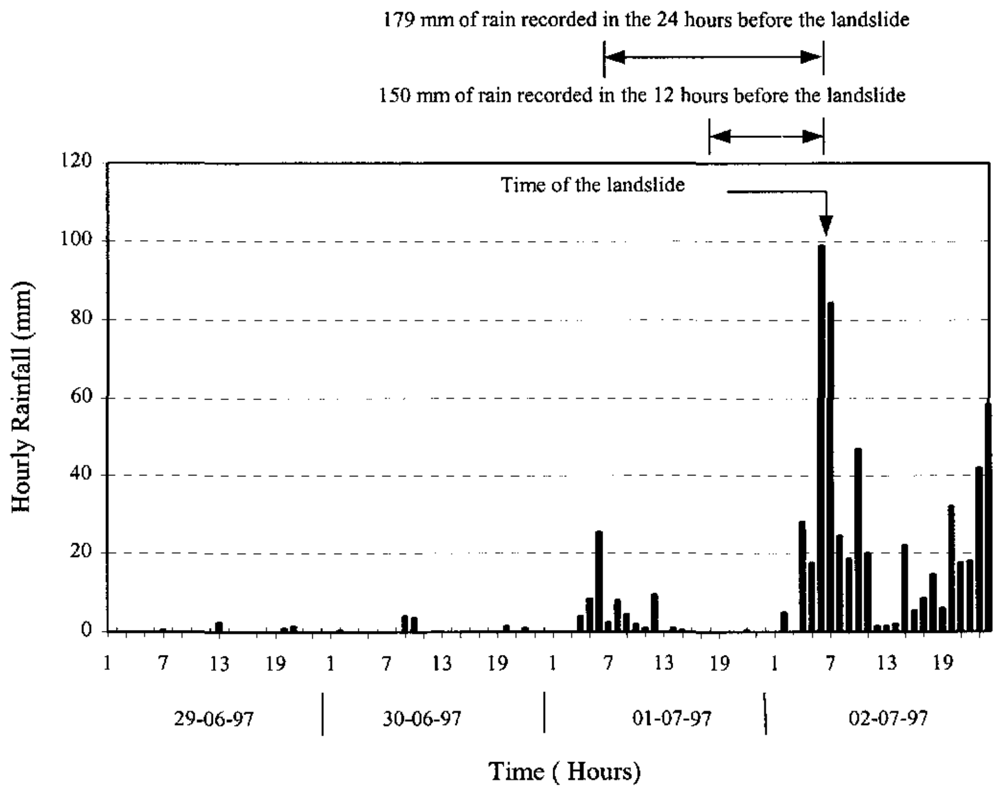
Notes : (1) Information shown in this figure is based on topographic survey, field observations and aerial photographs.

(2) Base map is based on Survey Sheet No. 7-SW-10D dated October 1992 (scale 1:1 000) and photogrammetric analysis from 1963 aerial photographs.

Figure 4 - Location Plan of Previous Landslide and Erosion Events



(a) Daily Rainfall Recorded between 1 June and 3 July 1997



(b) Hourly Rainfall Recorded between 29 June and 2 July 1997

Figure 5 - Rainfall Records of GEO Raingauge No. N02

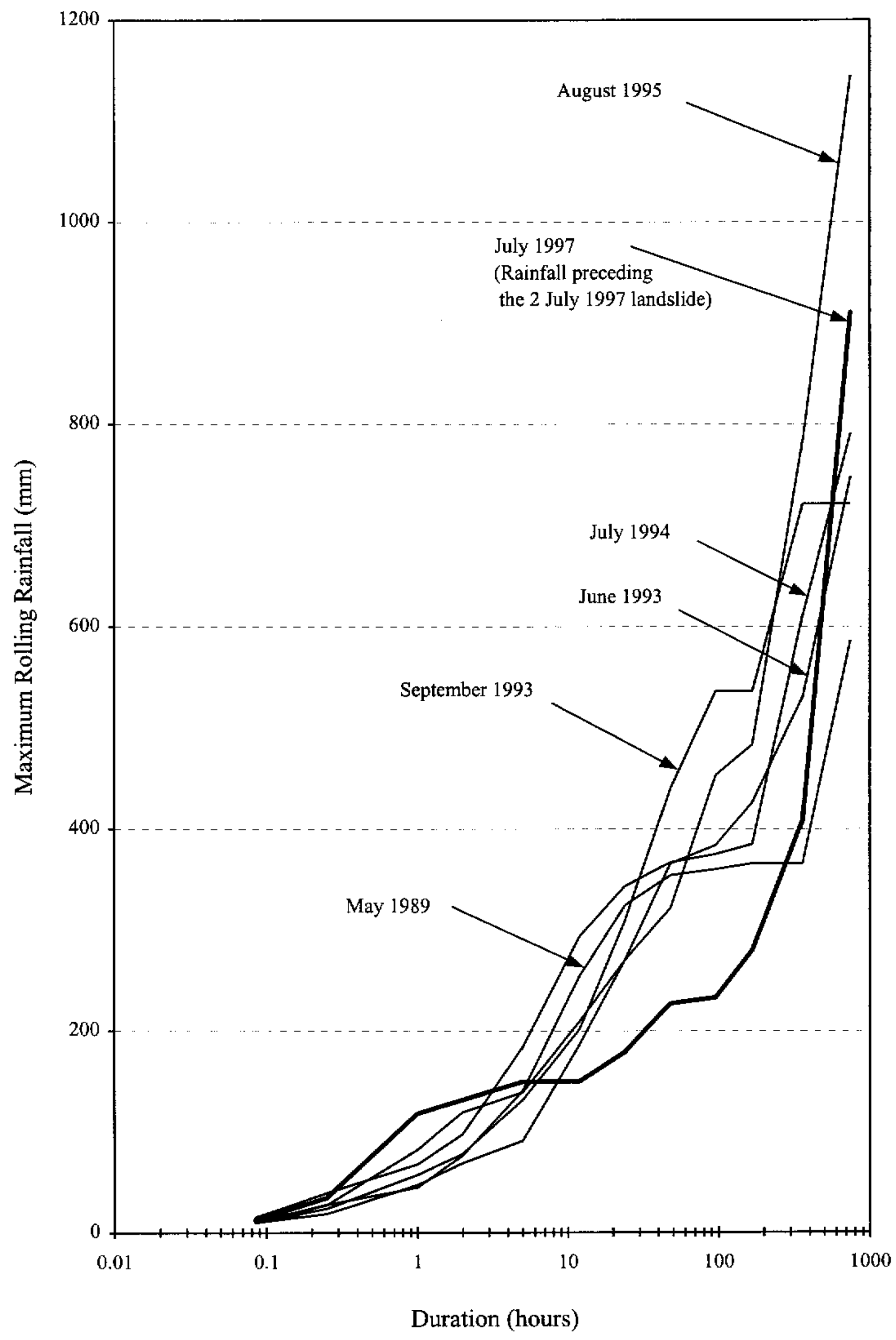
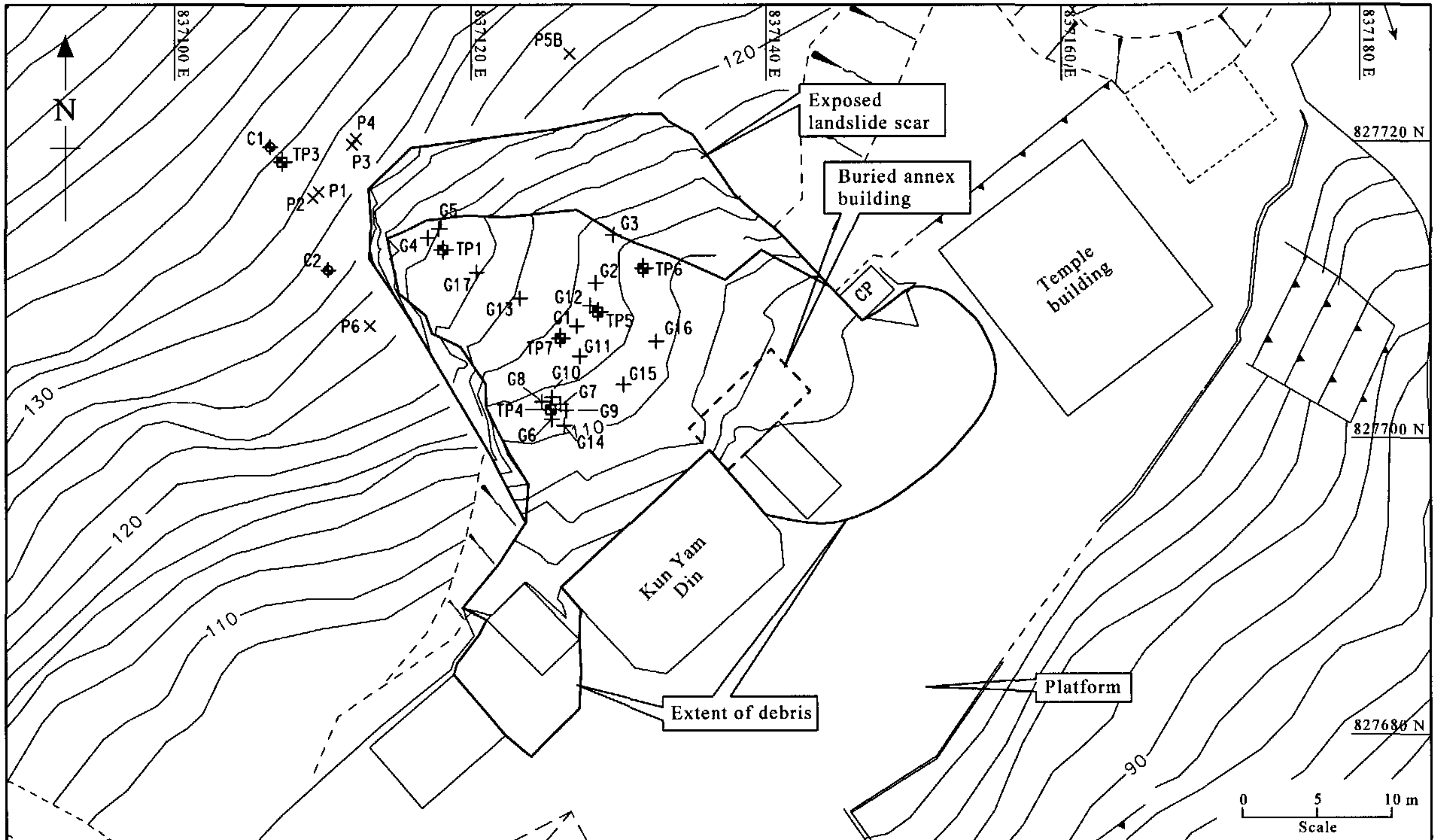


Figure 6 - Maximum Rolling Rainfall at GEO Raingauge No. N02 for Major Rainstorms



Note : Information shown in this figure is based on survey map, geological mapping, field observations and documentary records.

Legend : ⊕ Borehole + GCO probehole
⊞ Trial pit × GCO probehole with tensiometer installed

Figure 7 - Location Plan of Ground Investigation Works

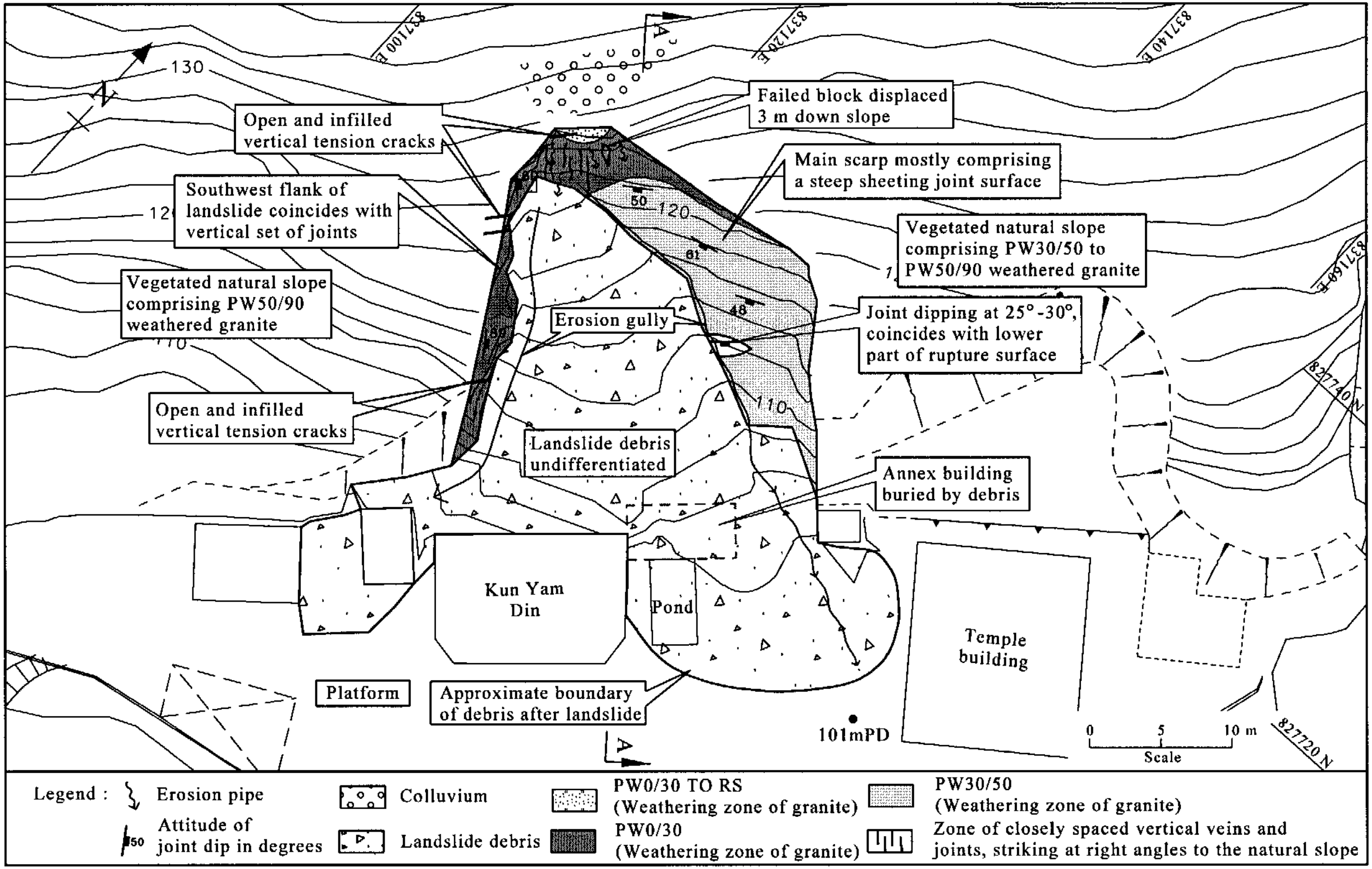


Figure 8 - Geological Plan of the Landslide Area

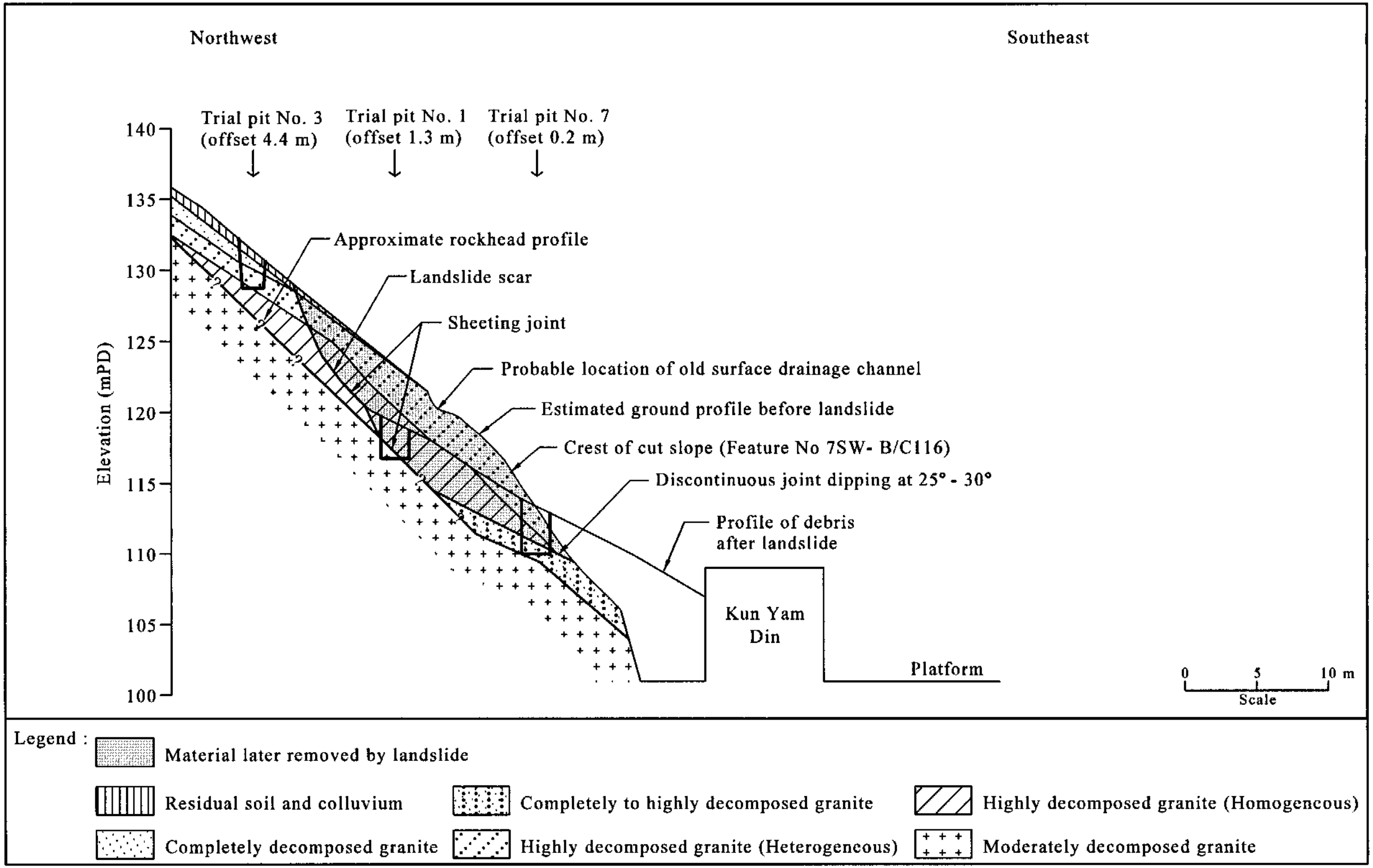
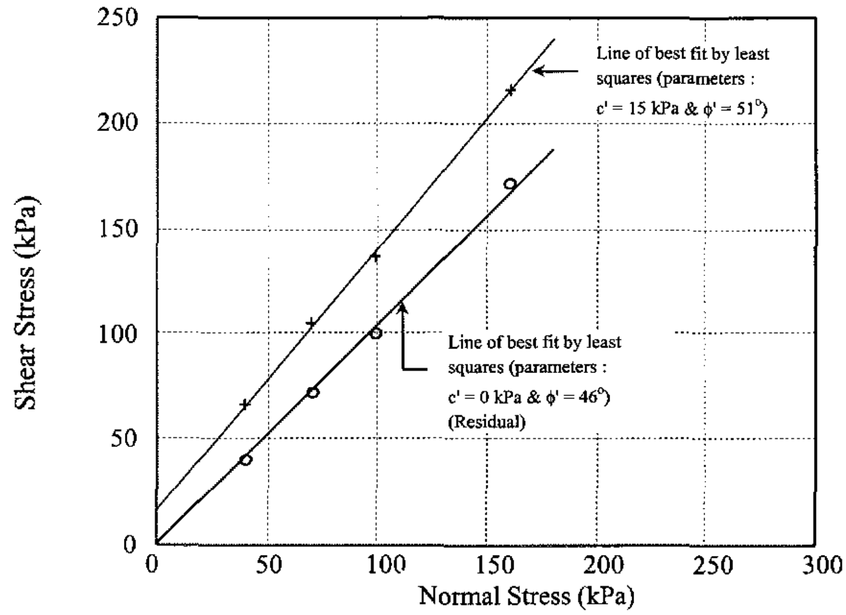
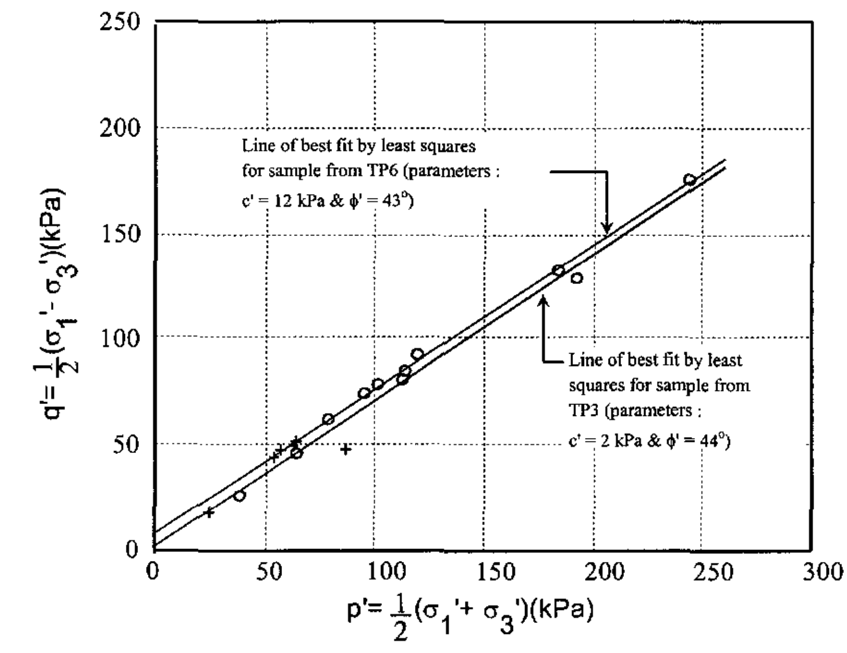


Figure 9 - Section A-A Showing the Typical Stratigraphy Through the Landslide Site



(a) Results of Drained Direct Shear Tests on Highly Decomposed Granite



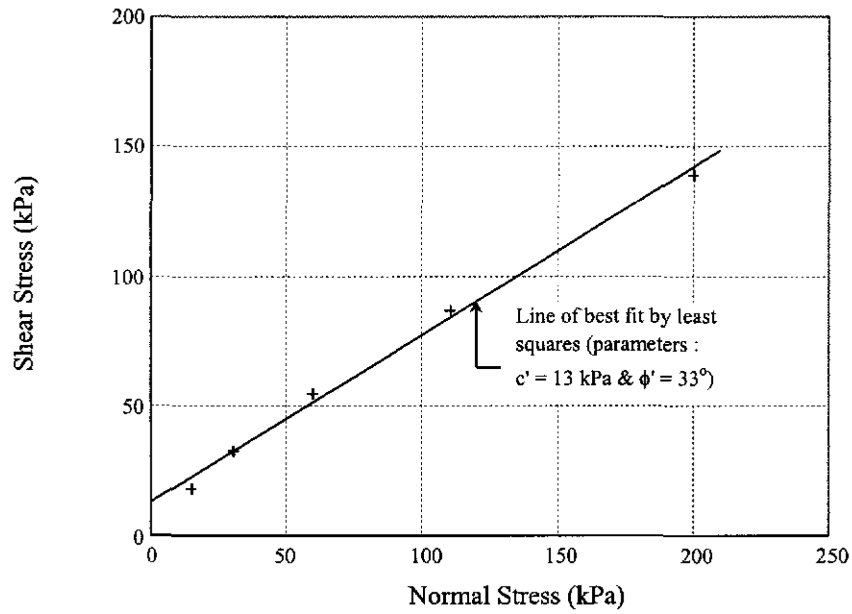
(b) Results of Consolidated Undrained Triaxial Compression Tests with Pore Water Pressure Measurement on Completely Decomposed Granite

Legend :	+ Single-stage test (HDG and CDG)	σ_1' Major principal effective stress	c' Cohesion (effective)
	o Multi-stage test (HDG and CDG)	σ_3' Minor principal effective stress	ϕ' Angle of shearing resistance (effective)

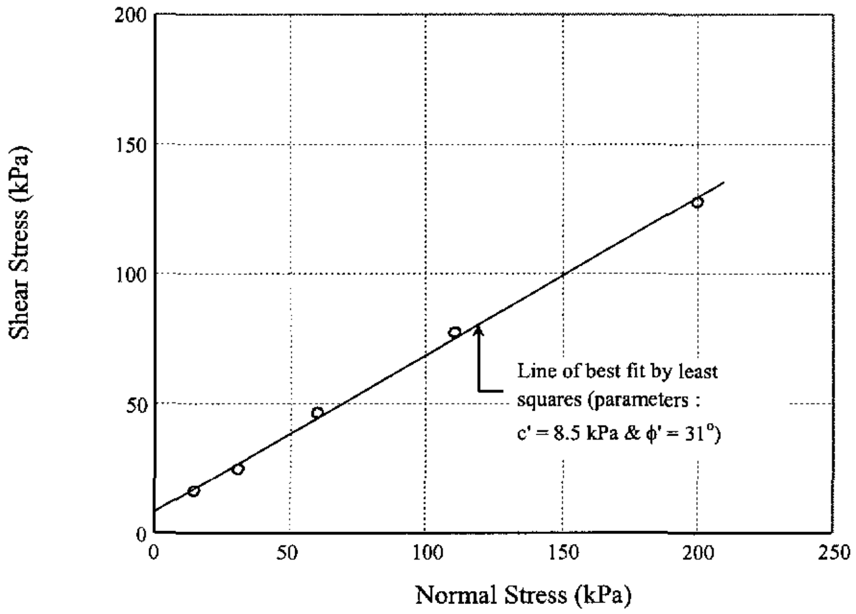
Notes :

1) Direct shear test results shown in this figure correspond to those at the point of peak shear stress (except for the residual strength envelope, which was those at the point of residual shear stress), and triaxial compression tests results correspond to those at the point of maximum stress obliquity, i.e. maximum stress ratio σ_1'/σ_3'.
2) See Figure 7 for location of trial pits TP3 and TP6.

Figure 10 - Direct Shear and Triaxial Compression Test Results for Highly and Completely Decomposed Granite



(a) Results of Peak Shear Strength Tests

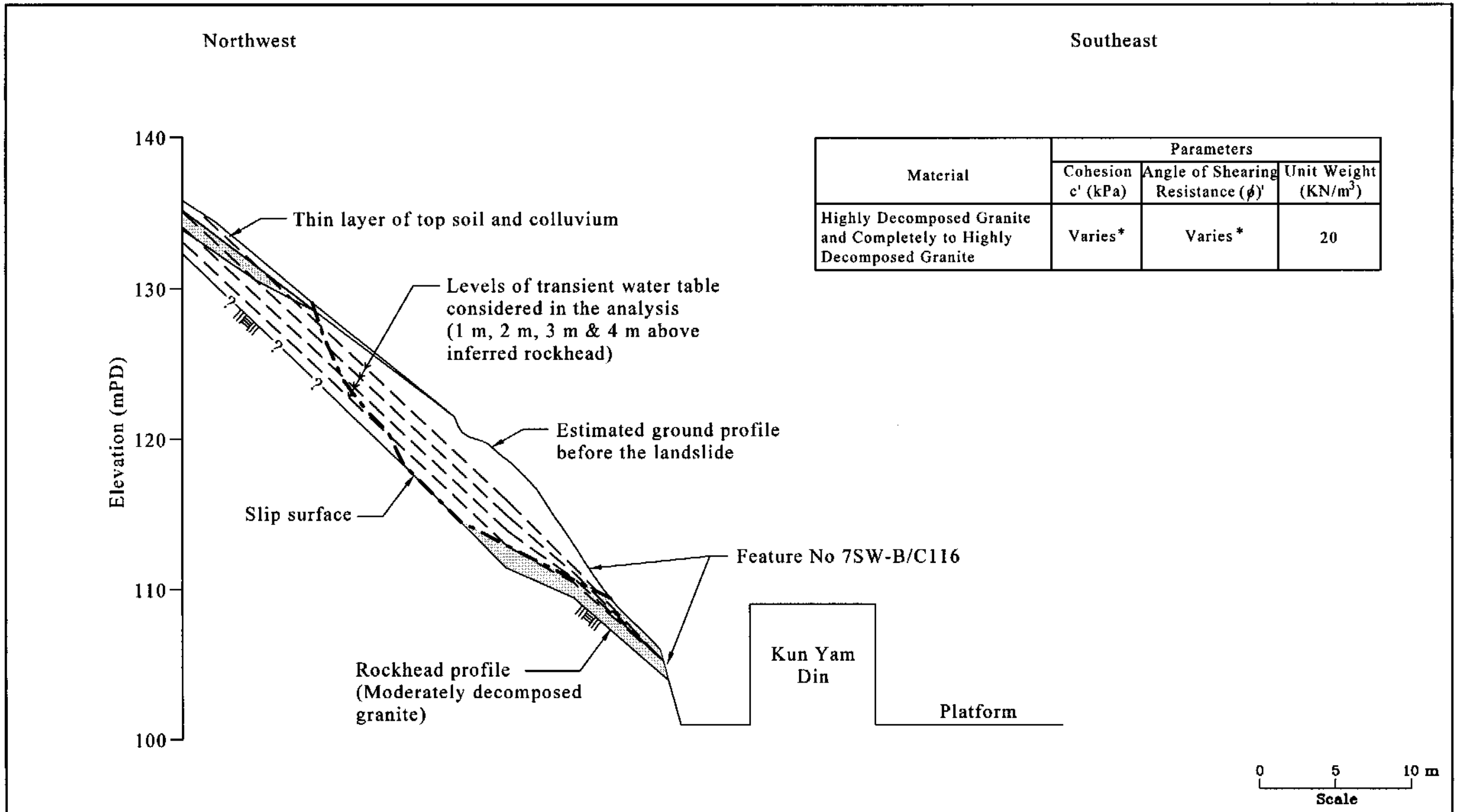


(b) Results of Residual Shear Strength Tests

Legend :	+ Single-stage test	c' Cohesion (effective)
	o Multi-stage test	ϕ' Angle of shearing resistance (effective)

Note : The material used for all tests was remoulded.

Figure 11 - Direct Shear Test Results for Pale Grey Slightly Sandy Clayey Silt



Material	Parameters		
	Cohesion c' (kPa)	Angle of Shearing Resistance (ϕ)'	Unit Weight (KN/m^3)
Highly Decomposed Granite and Completely to Highly Decomposed Granite	Varies*	Varies*	20

Notes : (1) See Figure 2 for location of section.

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

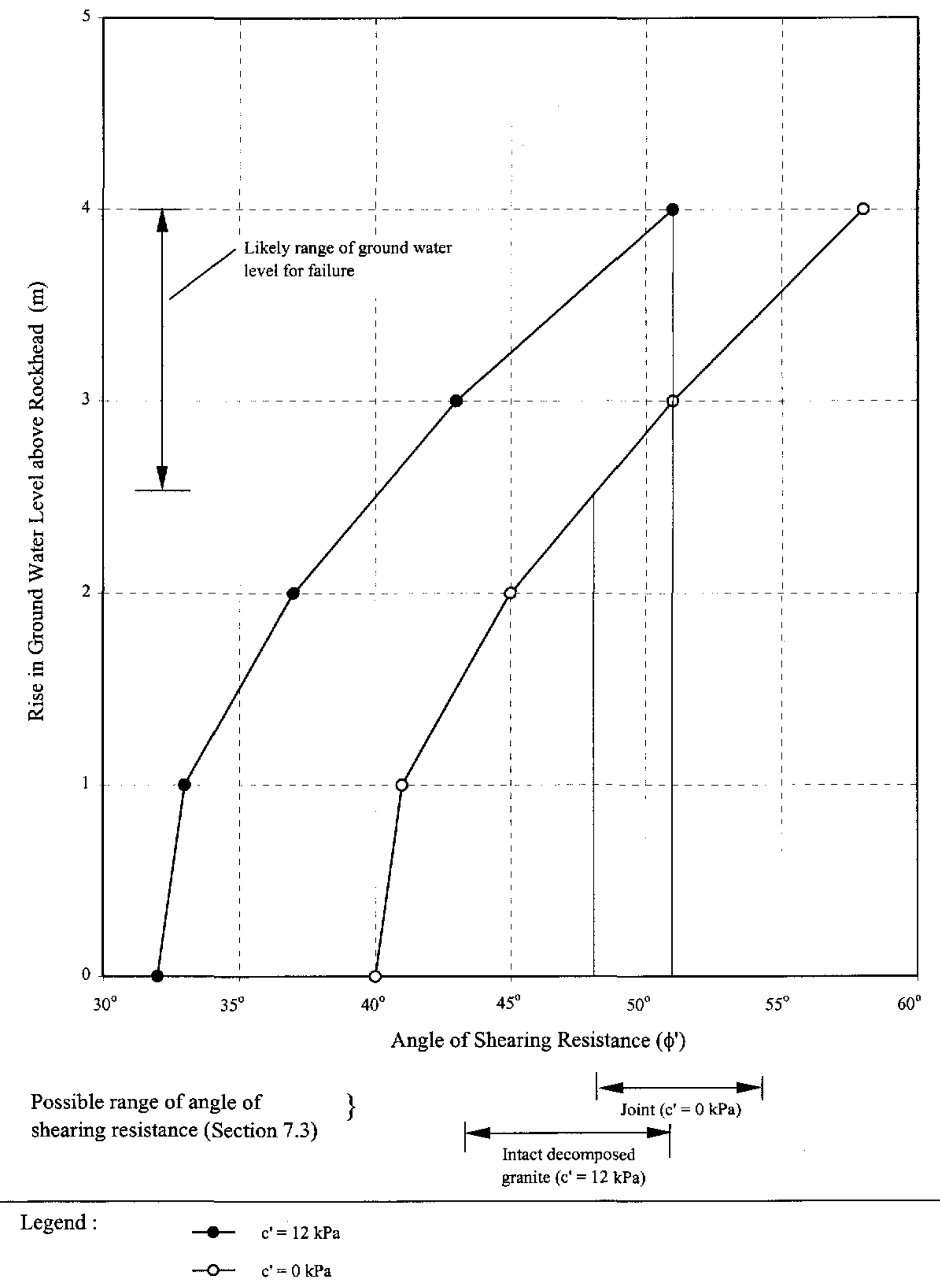
Legend :

* See Figure 13

Completely to highly decomposed granite

Highly decomposed granite

Figure 12 - Representative Cross-section of the Landslide for Theoretical Stability Analysis



Notes : (1) Refer to Figure 12 for analyses details.
(2) Results for a factor of safety = 1.0.

Figure 13 - Results of Theoretical Stability Analyses

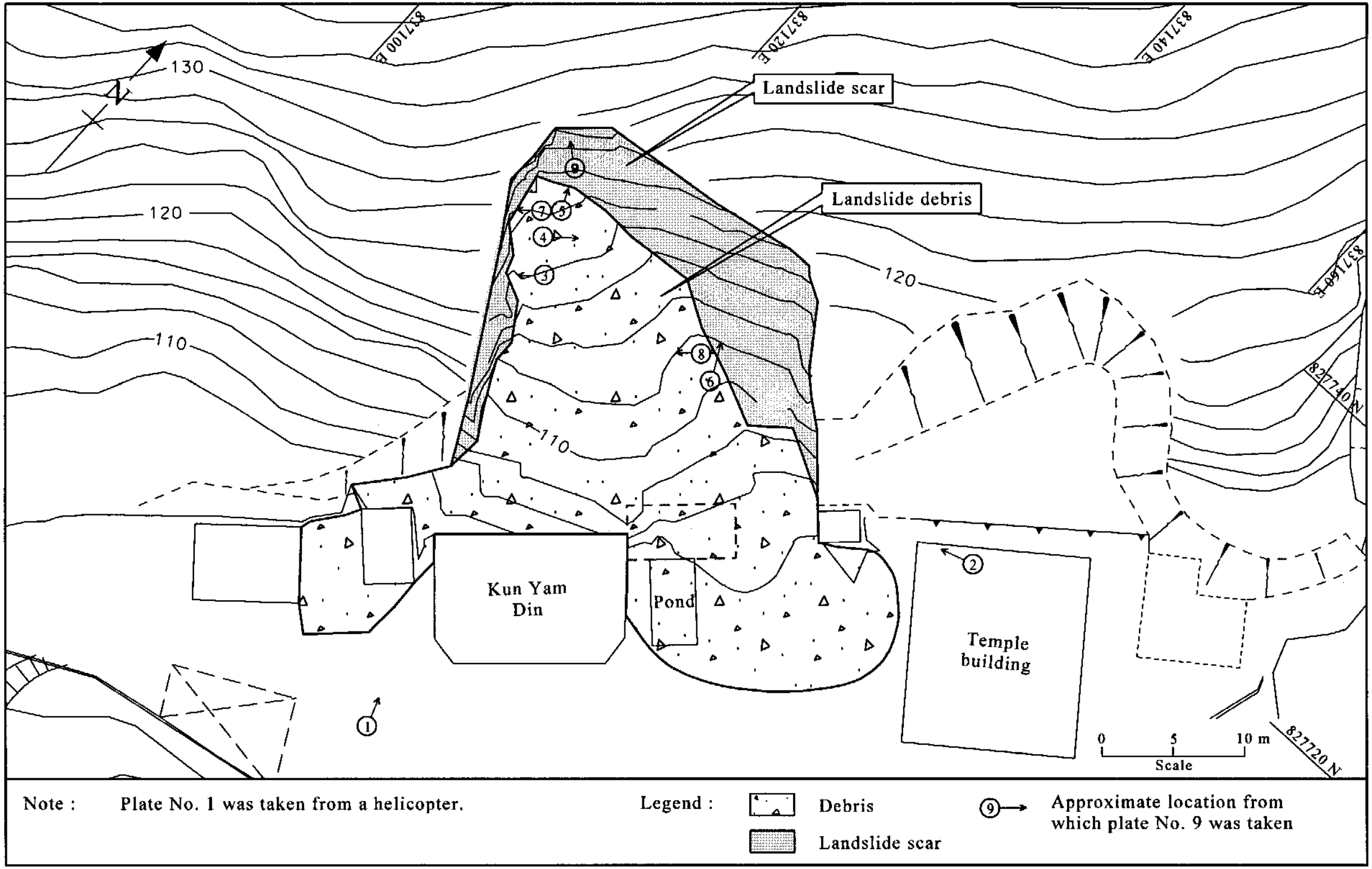


Figure 14 - Location Plan of Photographs Taken

LIST OF PLATES

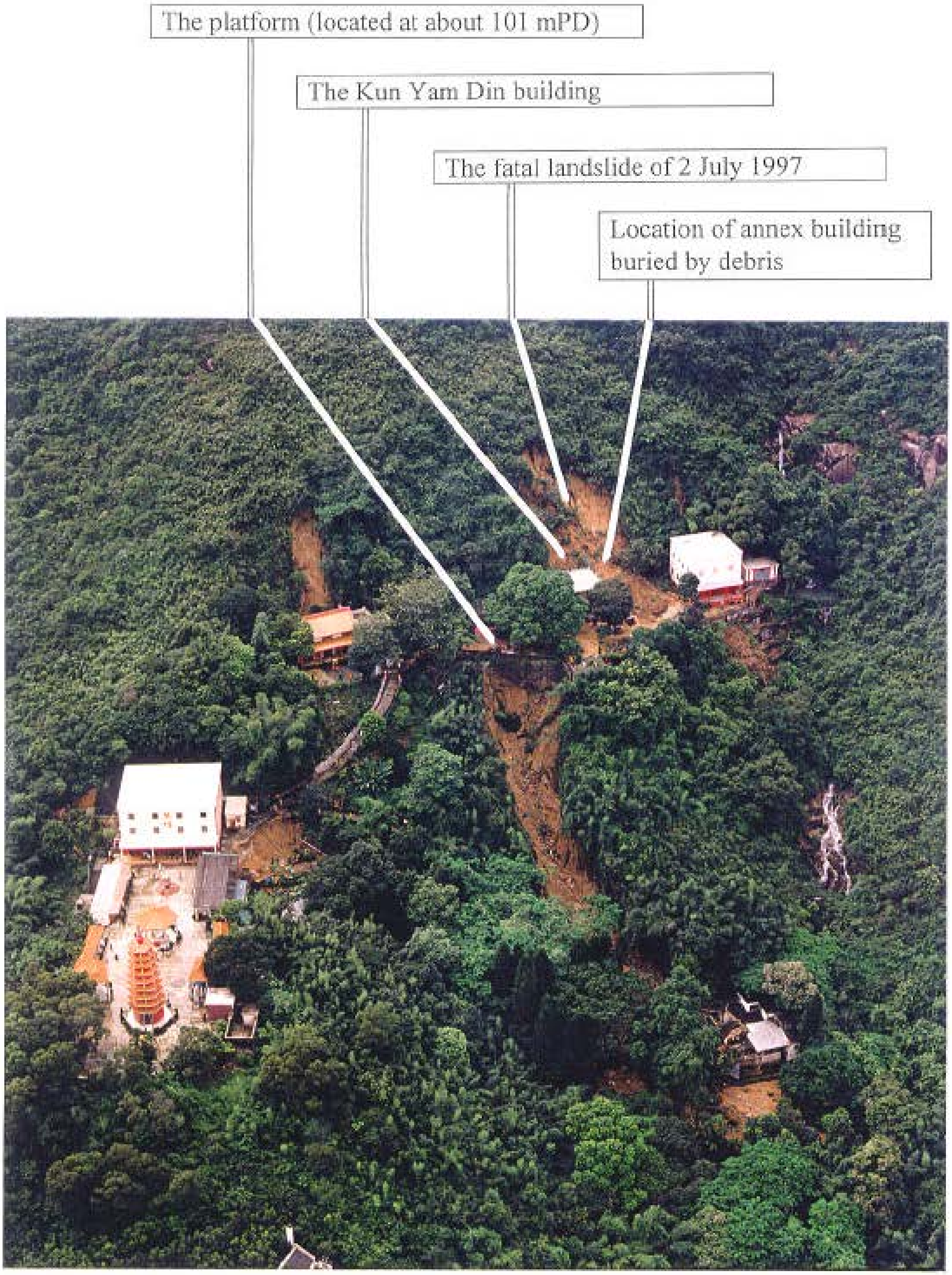


Plate 1 - Oblique Aerial View of the Site of the Landslide (Photograph Taken in the Afternoon of 3 July 1997. See Figure 14 for Location)

Plate 2 - View of the Landslide (Photograph Taken on 8 July 1997. See Figure 14 for Location)

Plate 3 - View of Southwest Flank of the Landslide (Photograph Taken on 7 July 1997. See Figure 14 for Location. Note the Grey Decomposed Granite Boulder on the Ground Surface and the Vertical Planar Discontinuity in the Lower Right Corner of the Photograph Associated with a Dominant Set of Sub-Vertical Veins and Joints)

Plate 4 - View of the Sheeting Joint Surface (Photograph Taken on 7 July 1997. See Figure 14 for Location. Note that the Northeast Flank of the Landslide Merges with the Natural Hillside)

Plate 5 - View of the Upper Part of the Main Scarp Showing the Zone of PW0/30 to RS Weathered Granite and the Sheeting Joint Surface (Photograph Taken on 7 July 1997. Scale is a 1 m Tape. See Figure 14 for Location)

Plate 6 - A Joint Exposed in an Erosion Gully, Dipping at 30°, Forming the Lower Part of the Rupture Surface (Photograph Taken on 4 August 1997. See Figure 14 for the Location)

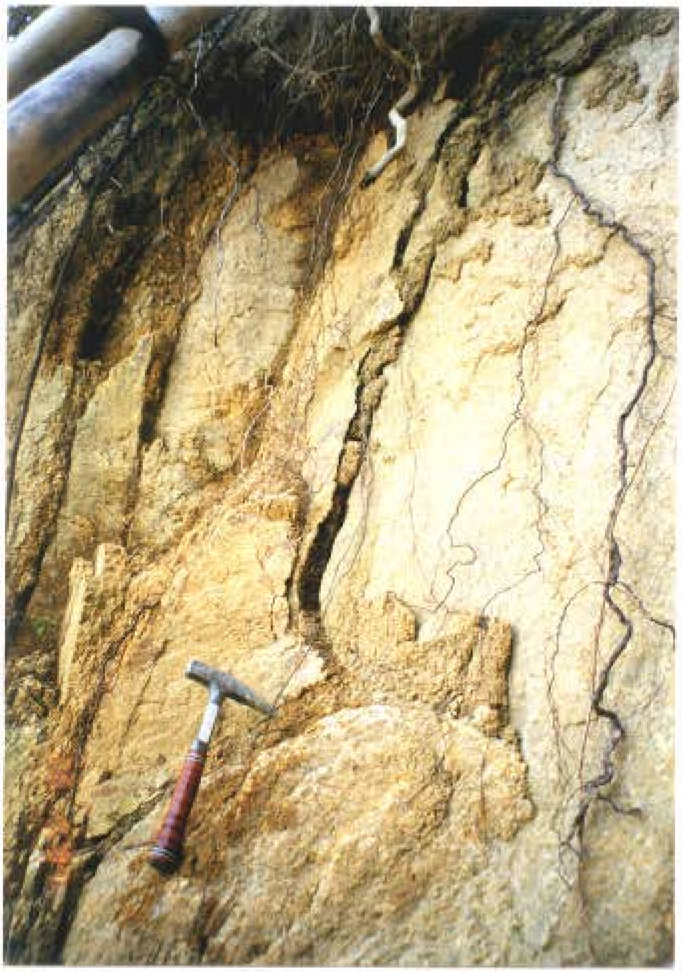


Plate 7 - View of Sub-Vertical Infilled Crack in the Southwest Flank of the Landslide (Photograph Taken on 26 September 1997. See Figure 14 for Location)

Plate 8 - Layer of Sheared Material at the Base of the Landslide (Photograph Taken on 12 November 1997. Note the Layer is about as Thick as the Length of Pencil. See Figure 14 for Location)

Plate 9 - View of an Erosion Pipe in the Northeast Corner of the Main Scarp (Photograph Taken on 23 September 1997. See Figure 14 for Location)

APPENDIX A

SUMMARY OF SITE HISTORY

CONTENTS

A.1 INTRODUCTION

References

The history of the site has been determined from :

(a) old topographic maps,

(b) a sequence of aerial photographs taken between 1924 and 1996 and,

(c) documentary records from GEO's files.

Topographic Map, Sha Tin, Sheet No. 12. Ordnance Survey Office, Southampton 1904.

A plan showing the locations of the various features discussed in this section, including the building reference numbers used and previous landslides, is given in Figure A1.

A.2 HISTORY OF SITE DEVELOPMENT

The earliest available aerial photograph, taken in 1924, shows no development of the site. Between 1924 and 1945, a platform was constructed, comprising a cut into the natural hillside and fill behind a retaining wall. Building No. 4 (Kun Yam Din), the small annex (in which the fatality occurred), the forerunner of building No. 5 and building No. 6 were present by 1945 (Figure A1). The man-made slope at the rear of the platform comprised a chunam covered cut slope behind building Nos. 4 and 5, typically 15 m high with the lower 5 m and the upper 10 m being inclined at about 75° and 56°, respectively.

A new cut and fill platform was constructed in 1954 extending the original platform to the southwest. Building No. 1 was constructed on the new platform in the same year. The cut slope behind building No. 1 was about 23 m high and inclined at about 55°.

The forerunner to Building No. 5 was demolished in 1956. A larger building was then constructed closer to the base of the cut slope. The remaining buildings on the platform, all of which were present prior to the recent fatal landslide, were completed by 1963.

Crown Lands and Survey Office, Hong Kong, Sheet No. C-145-NE-D, 1966 Revision.

In the period 1949 to 1961 a small cutting was made for a track traversing above the crest of the cut slope. This appears to correspond with the position of an old man-made drainage channel, the remains of which were observed in the hillside adjacent to the southwest flank of the recent fatal landslide.

From 1954 onwards, the platform became increasingly vegetated, until 1990 when many of the trees and shrubs were removed from the platform and resurfacing works were carried out.

A.3 PREVIOUS ASSESSMENTS

A.3.1 Slope Registration

The cut slope behind building Nos. 4 and 5 was not registered in the 1977/78 Catalogue of Slopes. It was identified in June 1996 under a project initiated by the GEO entitled "Systematic Identification of Features in the Territory" (SIFT). The project aims to search systematically for sizeable man-made slopes not previously registered in the 1977/78 Catalogue and to update information on existing registered slopes, based on studies of aerial photographs.

Phase II SIFT Study
Map Sheet Report 1:1000
Map Sheet 7SE/10D,
Planning Division, GEO,
1996.
Phase II SIFT Study

Subsequently, the cut slope was registered as No. 7SW-B/C116 under GEO's project entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST), which aims to systematically update the 1977/78 Catalogue of Slopes and compile the New Catalogue of Slopes. The Consultants appointed by the GEO to undertake the SIRST project inspected the cut slope on 4 December 1996. The cut slope face, which was recorded to be 8 m in height and inclined at 40°, and the ground beyond the crest of the cut slope were both assessed to be in fair condition with no signs of seepage or distress. The cut slope was assigned a category of high consequence in the event of failure. A Stage 1 Study Report on the slope was prepared on 13 March 1997 by the Consultants recommending further study.

Binnie & Partners SIRST
Field Sheet for Feature
7SE-B/C116.

The other cut slope, located behind building No.1 and formed as part of the new cut and fill extension to the platform in 1954, was registered as Feature No. 7SW-B/C113 under the SIRST project. An inspection, carried out on 4 December 1996 by the SIRST Consultants, recorded minor signs of distress on the shotcreted cut slope and assigned a category of high consequence in the event of failure. A Stage 1 Study Report on the slope was prepared on 13 March 1997 by the Consultants recommending further study.

A.3.2 Surveys and Studies

On the 1904 topographic map (Ordnance Survey Office, 1904) five minor stream courses are shown crossing the site where the platform was later constructed. One of these streams appears to have been near the site of the landslide that caused the fatality. The five minor streams are not shown on subsequent survey maps.

Aerial photographs taken in 1924 also show the pre-development drainage pattern at the site (Figure A1). Two ephemeral stream channels appear to be present, passing through an old landslide scar behind building No. 3, which then coalesce to form a single stream course. The location of this stream coincides with the scar the landslide that occurred below the retaining wall of the platform on 2 July 1997 (Figure 1). Also from the 1924 photographs, another ephemeral stream course appears to have flowed between building Nos. 4 and 5.

Apart from the inspections by the Consultants engaged by the GEO for the SIRST project, there are no other GEO records of inspections or assessments of the slope before the 1997 failure.

A.4 PREVIOUS LANDSLIDES

According to the GEO's records, no landslide incidents had previously been reported at the site of the 1997 landslide. There are also no past natural terrain failures in the area recorded in the GEO's Natural Terrain Landslide Inventory (Evans *et al*, 1997). From the study of the old aerial photographs carried out as part of this investigation, twelve probable previous slope failures were observed in the vicinity of the landslide, seven of which occurred between 1924 and 1973 with the remainder probably being pre-1924 landslides (Figure A1). The aerial photographs also indicate that there have been three erosion events. One of the failures, about 20 m^3 in volume, and one of the erosion events, affected the cut slope and hillside that failed during the recent fatal landslide.

Natural Terrain Landslide Inventory Map, contained in the Natural Terrain Landslide Study Phases I and II. Geotechnical Engineering Office, Hong Kong. Special Project Report SPR 5/97.

A.5 REFERENCES

Binnie and Partners (1997). SIRST Field Sheet for Feature No. 7SW-B/C-116. Dated 13 March 1997.

Crown Lands and Survey Office (1966). Sheet No. C-145-NE-D, 1966 Revision. Crown Lands and Survey Office, Hong Kong.

Evans, N.C., Huang, S.W. & King, J.P. (1997). The Natural Terrain Landslide Study Phases I and II. Geotechnical Engineering Office, Hong Kong. Special Project Report no. SPR 5/97, 119 p. (Unpublished).

Geotechnical Engineering Office (1996). Phase II SIFT Study. Map sheet report, 1:1000 Sheet number 7SW/10D. Dated June 1996.

Ordnance Survey Office (1904). Topographic Map of 1904, Sha Tin, Sheet No. 12. Ordnance Survey Office, Southampton, United Kingdom, 1904.

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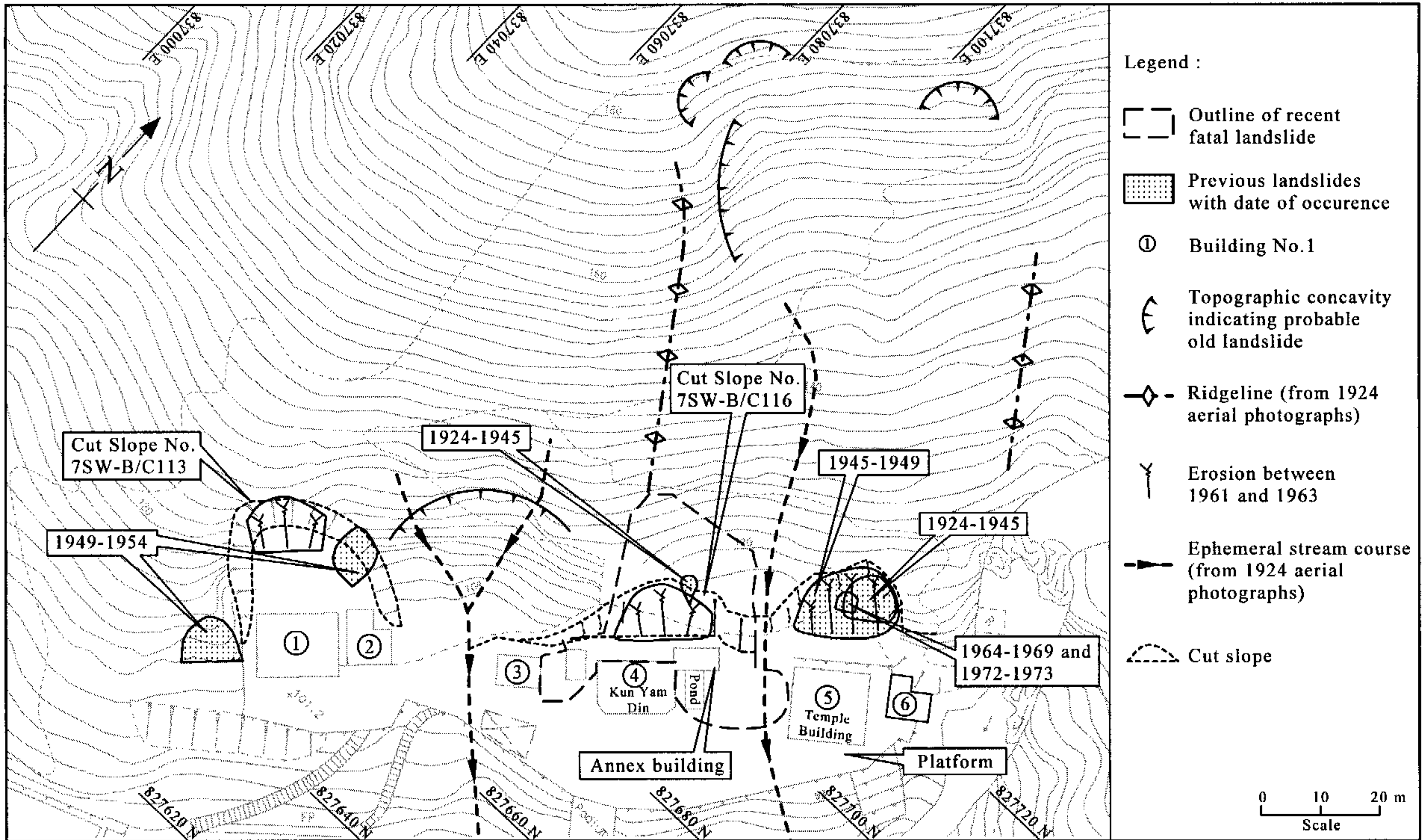


Note : Base map is based on Survey Sheet No. 7-SW-10D dated October 1992 (scale 1:1 000) and photogrammetric analysis from 1963 aerial photographs.

Figure A1 - Location Plan

一九九七年七月二日
萬佛寺山泥傾瀉事件報告

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

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)就一九九七年七月二日引致人命傷亡的萬佛寺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，並對萬佛寺山泥傾瀉事件之詳細調查提供技術支援和協助。

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

一九九七年七月二日，沙田萬佛寺後山上的斜坡發生山泥傾瀉，導致一人死亡，一人受輕傷。 編號 7SW-B/C116 削土坡的一部份，以及其削土坡頂部的部份山坡於這次事件中突然崩塌。

合樂亞洲顧問公司(HAP)，作為土力工程處(GEO)的山泥傾瀉顧問，在土力工程處的協助和配合下，於一九九七年七月至一九九八年二月期間就這次山泥傾瀉展開了全面的調查。 詳細研究工作包括翻查資料文件、分析雨量紀錄、訪問山泥傾瀉目擊人士、進行現場調查、場地勘探、以理論方法進行穩定性分析以及診斷事件的成因。

調查所得結論是：一九九七年萬佛寺的山泥傾瀉可能是由於崩塌前下了極大的雨，使斜坡內沿風化花崗岩層內不利穩定的，沿著不連續面走向的地下水壓上升所致。 削土坡在崩塌發生前缺乏足夠的設計及維修，亦可能導致斜坡局部逐漸惡化。

山泥傾瀉調查的詳情及所得的結果載於本報告內。

目 錄

1. 引言

一九九七年七月二日上午，沙田萬佛寺範圍內的一幅斜坡發生山泥傾瀉（圖1）。山泥傾瀉的泥石毀壞了一個名為觀音殿的建築物，並掩埋了其毗鄰的附屬建築物。山泥傾瀉導致附屬建築物內一人喪生，觀音殿內一人受輕傷。

土木工程署(CED)轄下的土力工程處(GEO)於事發後就著手對這次山泥傾瀉事件進行詳細的調查。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，在香港地質測量組（HKGS）及土力工程處的其它部門協助下，具體承擔了這次調查工作。山泥傾瀉場地的地形測量由土木工程署轄下的測量部承擔進行。

調查工作於一九九七年七月至一九九八年二月進行，主要項目如下：

(a) 就事發地點的發展歷史及山泥傾瀉前發生的事，翻查所有已知的有關記錄；

(b) 分析雨量記錄；

(c) 訪問目擊山泥傾瀉的人士；

(d) 於事發地點進行地形測量、詳細的觀察和量度；

(e) 進行地質勘察；

(f) 採用鑽探、探井、現場試驗及室內試驗進行全面的場地勘探；

(g) 以理論方法分析崩塌時斜坡的穩定性；以及

(h) 診斷導致崩塌發生的可能原因。

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列載於一套文件中，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。

2. 事發地點的描述

一九九七年七月二日，萬佛寺發生了五處山泥傾瀉，其中四處均影響到觀音殿所處的填挖平台（圖 1 及照片 1）。 根據沙田地政署的資料，導致一人死亡的山泥傾瀉發生於私人地段編號 323 及 324 的界線內(圖 1)。 觀音殿後面削土坡的一部份(編號 7SW-B/C116)及其頂部的部份天然山坡涉及於這次事件中（圖 2)。

平台的高程約為海拔 101 米。 山泥傾瀉發生前，位於觀音殿後面的削土坡，其坡腳距離觀音殿約 4 米。 此部份削土坡最高約為 15 米，下部 5 米和上部 10 米的傾角分別約為 75 度及 56 度。 在山泥傾瀉後從鄰接削土坡而未塌下的部份，觀察到有相當大部份的坡面為高度變壞的灰泥並且有嚴重的破裂和分離。 削土坡坡面被野生植物包括大樹所覆蓋。

削土坡頂部的天然山坡植被茂盛。 自削土坡頂部至山頂的山坡平均仰角約為 36 度，山頂的高程約為海拔 220 米。 有一條主山溪流過平台的東北端(圖 1)。從一九零四年的地形圖上（Ordnance Survey Office，1904）可看到有五條小溪流穿越如今的平台所在地。 其中一條小溪流似乎剛好穿越導致一人死亡的山泥傾瀉地點。 在後來的測量圖上則沒有顯示到這五條小溪流，但從一九二四年航測照片指出在後來建成觀音殿後的地點的 20 米範圍內有可能有三條山溪(見圖 A1，附錄 A)。

只有一層高的附屬建築物約 8 米長、4 米闊，與兩層高的觀音殿的東北角相鄰接。 這兩幢建築物在屋宇署都沒有任何結構記錄，但從視察所得明顯地所有建築均為磚結構，而觀音殿有一個鋼筋混凝土的平面屋頂。

在山泥傾瀉地點的西南翼，發現了一條已被充填的舊人造混凝土排水渠的殘留段，它被山泥傾瀉的西南翼截斷並與之相鄰接（圖 2)。 排水渠沿一條 2 米闊的小徑展佈；該小徑最初為一淺層開挖，但經過多年後被泥土及植物的雜物幾乎完全阻塞。 在山泥傾瀉的東北方沒有找到小徑和排水渠。 在削土坡或在平台後的山坡上並不存在任何人造的表面排水渠。

3. 山泥傾瀉的描述

導致一人死亡的山泥傾瀉照片載於照片 2，其平面圖及橫貫山泥傾瀉地點的剖面圖則分別載於圖 2 和圖 3。

根據目擊人士所作的描述，山泥傾瀉發生於一九九七年七月二日上午六時十五分。 以大致垂直於山泥傾瀉發生前的地面進行量測，山泥傾瀉的最大深度在西南側約為 6 米，在東北側約為 1 米。山泥傾瀉瀉下的泥石大約有 1500 立方米，產生的主崩塌殘痕約 25 米闊，破裂面約為 25 米長。 泥石掩埋了附屬建築物，堆積於觀音殿後面達 6 米深，並導致觀音殿的部份後牆產生了結構性的破壞。觀音殿的地下層部份被山泥傾瀉產生的濕土壤及樹木所充填。 從山泥傾瀉的頂部至泥石末梢所量得的泥石移動的平面距離最大約為 40 米。 自泥石末梢至崩塌殘痕頂部的仰角約為 32 度，這符合本港削土坡的一般情況，即由豪雨造成的山泥傾瀉的典型值（Wong & Ho，一九九六）。

山泥傾瀉的泥石主要為鬆散、潮濕至濕、含少量黏土質、礫質的粗砂，並含有高度風化的花崗岩質卵石和漂石及少量輕度風化的花崗岩漂石（最大的體積可達 5 立方米）。

一九九七年七月二日，於山泥傾瀉發生後進行的現場調查發現，在山泥傾瀉的主崩塌殘痕地面以下約兩米處有大量的地下水滲流（第 7.4 節）。地下水的滲流及地表徑流使泥石堆的兩側均產生了侵蝕沖溝（圖 2）。

4. 事發地點的場地歷史

一個概括的事發地點的場地歷史介紹載於附錄 A。 這個文件是透過研究該處的航空照片和翻查現有資料文獻而得。

最早可得到的航空照片拍攝於一九二四年，照片顯示事發地點位於一幅面向東南的山坡上，在當時尚未開發。 平台於一九二四年至一九四五年期間建成，是通過削掉坡腳並且在山坡下部修建擋土牆並在牆後填土而成。 這個擋土牆是用大塊的岩石建成，大約高 10 米，延伸至整個平台的長度，還安裝了水平梁和排水孔。 在一九四五年，平台上只有三個建築物，其中包括觀音殿及其附屬建築。到了一九五四年，平台已向西南方向延伸了；而到一九六三年時，現在平台上的另外六個建築物中的三個也已建成。

自從八十年代初本港對山泥傾瀉事件進行系統地記錄以來，土力工程處沒有位於平台上面削土坡上的先前山泥傾瀉的記錄。 然而，根據對航空照片的判釋，發現削土坡於一九二四年至一九七三年期間可能發生了五次山泥傾瀉事件(圖 4)。 另外，一九六三年之前，削土坡的兩個區域可能發生過淺層的侵蝕現象。

從一九四九年至一九六一年期間，為修建一條橫穿削土坡頂部的小徑而建成了一幅小削土坡。 其平面位置似乎與在山泥傾瀉發生後進行的現場調查中所發現的舊排水渠（第 2 節）的位置一致（圖 2)。

在一九七七至一九七八年的斜坡登記過程中，這個削土坡並沒有被確認。一九九六年在土力工程處推行“有系統的鑒定和登記全港的斜坡計劃”(SIRST)下，這個削土坡被登記為編號 7SW-B/C116。 這個計劃的目的是系統地更新 一九七七/七八年的斜坡登記並且編輯新的斜坡目錄。

5. 雨量計錄分析

雨量數據來自土力工程處在山泥傾瀉現場附近設置的自動雨量計 N02，編號 N02 雨量計座落在禾輋村順和樓；離事發地點東南約 950 米。 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。 該雨量計在一九九七年六月及七月所錄得的每日雨量，以及一九九七年六月二十九日至七月二日所錄得的每小時雨量，均載於圖 5。

對設在山泥傾瀉現場附近的兩個自動雨量計 N01 及 N09 的雨量數據也進行了分析。 編號 N01 及 N09 雨量計分別離現場西南及東北約 2 公里外，該兩個雨量計所錄得的雨量變化及強度與編號 N02 雨量計所錄得的大致相同，故編號 N02 雨量計的雨量紀錄被認為適用作雨量分析。

由一九九七年七月二日清晨至山泥傾瀉發生時，雨勢均頗大，並且幾乎持續了一整天。 山泥傾瀉前的 24 小時及 12 小時期間所錄得的雨量，分別為 179 毫米和 150 毫米。 而六十分鐘的最高滾存雨量則為 118.5 毫米，於一九九七年七月二日上午五時十五分至六時十五分所錄得。 根據香港天文台以往的雨量計錄數據（Lam & Leung，1994)，按不同的降雨時段分析七月二日豪雨的雨量強度的重現期，發現在五分鐘至三十一日時段間，六十分鐘時段所錄得的雨量為最大，相應重現期為二十五年一次。

對一九九七年七月二日豪雨的最大滾存降雨量與雨量計 N02 自一九八零年裝置以來至一九九七年七月期間所錄得的過往嚴重豪雨的最大滾存降雨量進行了比較，結果載於圖 6。 以介於二十分鐘至三小時之間的降雨時段計算，一九九七年七月二日上午六時十五分之前的豪雨，其最大滾存降雨量超出了過往所有嚴重豪雨事件的最大滾存降雨量。

6. 山泥傾瀉的經過

根據包括五名萬佛寺居民在內的目擊人士的陳述、香港警務處和消防處的事件報告，山泥傾瀉的經過可重組如下：

根據幾位目擊人士的陳述，山泥傾瀉發生於一九九七年七月二日上午六時十五分，此時雨勢猛烈。 緊靠山泥傾瀉發生之前，有一位目擊人士看見有棵大樹倒向觀音殿的附屬建築物；其它的目擊人士稱聽到巨大的轟隆聲及大樹倒下的聲音。 目擊人士的陳述表明，山泥傾瀉發生得很突然。 快速移動的山泥傾瀉泥石在很短的時間內掩埋了觀音殿的附屬建築物。 一名失蹤人士假設被圍困在附屬建築物內。 山泥傾瀉毀壞了觀音殿後牆的一部份；瀉下的泥石沖入建築物內，導致地下臥室內的一名居民受輕傷。 山泥傾瀉發生後，幾名目擊人士回憶到，平台及觀音殿內的地面水流達 0.3 米深。

經過簡單搜尋之後，沒找到失蹤者。 平台建築物內的居民於是走到山坡下面的萬佛寺大殿內，透過電話向警方報告了此次山泥傾瀉事件，根據警務處的記錄，報案的時間是上午六時三十三分。

7. 事發地點的地下情況

7.1 概述

事發地點的地下情況是根據文件和實地研究所得的資料而確定。 文件研究工作包括翻查現有的岩土資料，而實地研究工作則包括地質勘察及場地勘探。

按照該區 1:20 000 比例地質圖上（Addison ，一九八六及土力工程處，一九八六），香港地質測量組測繪事發地點的岩石屬於粗粒花崗岩。 該區域有兩個斷層，一個位於事發地點西南面約 300 米處，由西北傾向東南；另一斷層位於東北面約 160 米處，由西到西北傾向東到東南。 沒有斷層穿過事發地點。 一九九七年七月二日崩塌發生後，合樂顧問公司及香港地質測量組對斜坡進行了地質勘察。

場地勘探工作於一九九七年九月十二日展開，於一九九七年十一月十二日結束。 場地勘探工作包括兩個垂直鑽孔並安裝了開敞式測壓計和合樂水桶、六個探井和二十四個 GCO 輕型動力觸探計，其中六個隨後安裝了充填式張力計(圖 7)。

7.2 地質

事發地點的地質圖載於圖 8 ，而橫貫山泥傾瀉地點的地層剖面圖則載於圖 9 。

事發地點的岩性組成為中粒至粗粒花崗岩，個別地方為完全粗粒偉晶花崗岩。 風化花崗岩在部份外露的滑坡殘痕、削土坡腳及山泥傾瀉部位兩側之外的山坡上出露。 根據探井資料、GCO 輕型動力觸探結果及現場觀察建議，部份山泥傾瀉破裂面沿中度風化花崗岩與高度風化和完全至高度風化花崗岩土的交界面形成 (圖 9)。 自然山坡上有花崗岩漂石。

薄的殘積土和坡積物不連續層典型地出現在地表下 0.5 米處。

崩塌部位最深的部份是在中部和滑坡殘痕的西南側，其典型深度為四米至六米。 這種情況同該部份是由 PW0/30 部份風化花崗岩帶組成是相一致的。 經由此風化帶的部份在山泥傾瀉部位西南翼露出，在那裏，一非均質的風化花崗岩層，由岩塊和較軟的高度風化的花崗岩組成，典型地出現在地表下一米至三米處 (照片 3)。 隨著深度的增加，岩塊消失，而花崗岩則變得更均勻。

一九九七年七月二日的山泥傾瀉大體由事前已存在的不連續面所控制。 破裂面在滑坡殘痕部份地露出，沿業已存在的、持續的、起伏不平及粗糙的節理，分別在主斷層崖的上部和下部傾向南到東南面約 52 度和 43 度 (照片 4)。 順著山泥傾瀉西南側，滑坡深度增加是由於節理的走向稍微傾向於自然山坡的走向。 測得的波狀節理的波長達 5 米，變化幅度達 0.8 米，節理面上最大的粗糙角度約為 16 度。 節理與自然山坡有關的方位特徵及它的持續和起伏特徵，表明其可能為一面狀節理。

山泥傾瀉破裂面的上半部份沿面狀節理及穿過完整風化花崗岩而形成，導致形成一陡曲的表面 (照片 5)，而最大的破裂面的中心部份完全沿著面狀節理形成。 破裂面的下半部份，在山泥傾瀉部位東北側的侵蝕谷裏及四個探井中出露，與一組不易見到的、間斷的起伏不平並傾向東南，以 30 度角穿出斜坡的節理是一致的 (照片 6)。 一淡灰色、微黏土質粉土薄層，達 20 毫米厚，局部地蓋在或充填在一些節理上。

觀察到一組主要的近於垂直的岩脈和殘餘節理，其走向與斜坡成直角，穿過山泥傾瀉殘痕和鄰近的岩石露頭 (圖 8)。 這些不連續面在滑坡殘痕的西南側間距較密(60 毫米到 200 毫米)，而沿著東北側則變得較疏至非常疏(間距達到 4

米)。 岩脈為垂直石英層充填，稍有裂開，這將容許滲水流過。 在主滑坡崖的上部，大多數節理的裂縫，稍有張開，隨著深度的增加變成閉合。 幾個這樣的岩脈和節理形成了在山泥傾瀉西南側出露的近於垂直的面。 對高度風化花崗岩近距離觀察發現極緊密的微破裂富集帶，方位與大多數岩脈和節理相同。

在山泥傾瀉部位的西南側出露的非均勻的風化花崗岩帶，觀察到與山坡具有相同傾向的，部份被充填的近於垂直的裂縫（圖 8 與照片 7)。 典型地，裂縫中等程度地狹窄(20 毫米到 60 毫米)，並且部份地充填有干硬至潮濕、微黏土質、含砂粉土殘留物，其可能是來自山坡面上沖刷進去的物質。 在主滑坡崖，偶然觀察到殘餘節理，具有與裂縫相似的走向，延續一般小於一米。 由此認為，這些節理可能在過去業已張開而形成裂縫，盡管在一些情況下，裂縫已經明顯地沿著"岩石塊體"與較弱的高度風化花崗岩的垂直的邊界面形成。

在主滑坡崖中部，發現一塊體積約 5 立方米相對完整的物質，從山泥傾瀉部位頂部原來位置被移下約 3 米至面狀節理處（圖 8)。 在塊體背部周圍的釋放面同兩個殘餘節理是一致的，其包有與在山泥傾瀉部位西南側發現的充填在近於垂直的裂縫物質相同的殘留物。 這些充填物和裂縫的出現表明，在山泥傾瀉之前，斜坡移動較小。

緊鄰破裂面下部之上的部位，在探井中發現有一層 0.2 米厚，軟、潮濕、中棕色、微黏土質、含碎石的粉土質砂（照片 8)。 這層物質表明了明顯的擾動信號及代表了山泥傾瀉底層強烈剪切的最終產物。 其表明該層之上的大部份的泥石在土體中有如一整體的，漂浮的物質移動。

7.3 <u>土壤和岩石的性質</u>

本處在場地勘探時取得岩土試樣，進行一系列全面的實驗室試驗，包括粒徑分佈試驗，阿太堡界限試驗，直接剪切試驗、三軸壓縮試驗和滲透試驗。

本處按照 Chen(一九九四)的建議，進行了粒徑分佈及阿太堡界限試驗。 該處物質為高度風化的花崗岩和完全風化的花崗岩，細粒土（即黏土和粉砂）的平均含量分別為13%與 17%。物質中的細粒土的塑性指數平均在 26% 至39% 之間，液限則在 61%至 74%之間。

從實驗室測試得到的高度風化的花崗岩和完全風化的花崗岩的平均滲透性的值是大約 2×10^{-5} 米/秒。 至於有殘餘節理的，高度風化和完全風化的花崗

岩，從鑽孔水頭降低測試，得到的滲透性的值介於 7×10^{-4} 米/秒和 1×10^{-3} 米/秒之間。

本處按照 BS1377（一九九零 a ）進行直接剪切試驗和 BS1377（一九九零 b ）進行固結不排水三軸壓縮試驗評估風化花崗岩和灰色黏土質粉土的抗剪強度性質。 試驗結果和物質抗剪強度參數，以最小平方法求得的最佳擬合線確定，載於圖 10 和圖 11。

本處按照 Richards 和 Cowland （一九八二）推薦的方法，從實地度量結果估計面狀節理的表面粗糙度。 預測的表面粗糙度值在 8 度至 14 度之間，加上假定的面狀節理基本摩擦角 40 度(Papaliangas 等，一九九五)，則有效剪切強度參數是 0 kPa 的內聚力值和 48 度到 54 度的抗剪角。

7.4 地下水情況

一九九七年七月二日山泥傾瀉發生後在很短時間內，觀察到大量的滲水從低於地表約 2 米的主滑坡崖上流出（圖 8 ）。 滲水源自貫穿土體的侵蝕管（照片 9 ）和微張開的殘餘節理及石英岩脈。 滲水量大體在一九九七年七月三日豪雨停頓約一周後大幅減少了。

山泥傾瀉於一九九七年七月二日清晨豪雨開始後四小時內發生。 降雨的地表流量輕微的被植被和起伏不平的斜坡所妨礙。 因此，推斷地表水通過不連續薄層殘積土和坡積物急速滲透到局部張開的裂縫、節理和土壤侵蝕管出現在基岩界面之上一米至六米的風化花崗岩土層中。 水文地質條件有利於豪雨時在風化花崗岩中產生瞬態地下水壓。 最好的估計為在山泥傾瀉發生時，作用於滑動部份斜坡上的可能水位是位於基岩界面之上約一米至四米之間，基岩面為中度風化花崗岩與高度風化花崗岩的交界面(圖 9)。

崩塌當日，觀察到水從高程約 90 米的平台下的擋土牆底部流出（圖 1 ）。水流入平台下由山泥傾瀉形成的侵蝕沖溝內。 大約一周後，觀察到水從高程約 75 米的較低處流出，同時，上面的山溪變乾。 這種在高程上的變化與山坡內瞬態地下水位的變化是一致。

在山泥傾瀉後進行的場地勘探工作所安裝的兩個測壓計，從乾旱季節直至目前為止沒有到有任何地下水的記錄。

8. 理論穩定性分析

本處曾以理論方法進行穩定性分析，以協助判斷這次山泥傾瀉的機制和成因。 這些分析旨在查明在不同水位的情況下，崩塌時沿滑動面可能的抗剪強度範圍值。

這些分析時採用的資料來自發生崩塌後所進行的場地勘探工作、野外作業、室內試驗、現場觀察和實地量度。 具代表性的崩塌地點剖面，以及這些分析所採用的輸入參數，載於圖 12。 為模擬一九九七年七月二日降雨後在斜坡內的水位，分析中採用的瞬態地下水位在假設的基岩界面之上達四米。

沿著斷裂面的剪切強度參數是: 黏聚力分別為 0 kPa 和 12 kPa，同時抗剪角是變化的，這個分析所得的結果摘要載於圖 13。 為了代表性的面狀節理，零的黏聚力是被考慮，並且是一致的與大多數斷裂的表面。 當採用完整的風化的花崗岩，不考慮業以存在的非連續面的影響來確定理論上的斜坡的穩定性，採用了 12 kPa 的黏聚力進行分析。 為了類似的斜坡材料性質的剪切強度參數(第 7.3 節)，理論的穩定性分析表明，當地下水位上升到基岩之上約 2.5 米至 4.0 米時，是導致斷裂的主要原因。 這個表明，在上述的假設條件下，這個理論的穩定性分析是可信的。

9. 山泥傾瀉成因的診斷

根據此次調查所得的資料，推測到萬佛寺削土坡的山泥傾瀉主要是由下述三個主要的因素造成的:

(a) 豪雨後瞬態地下水壓上升，

(b) 不利於穩定的不連續面的存在，及

(c) 斜坡缺乏足夠的設計和維修。

山泥傾瀉發生在暴雨期間，該暴雨在山泥傾瀉前二十分鐘至三小時之間的降雨時段內錄得的最高滾存雨量是事發地點附近編號 N02 雨量計自一九八零年安裝以來，所錄得的最高雨量記錄。 發生山泥傾瀉之前的一小時，錄得最大六十分鐘最高滾存雨量為 118.5 毫米，估計其相應重現期約為二十五年。 即使這個重現期相對較大，但並不認為是例外的。

一九九七年七月二日早上，觀察到大量滲水從位於主滑坡崖、低於地表約 2 米的多處地方流出，這個表明在崩塌前，地下水位對豪雨的反應非常快。 業已存在的張開的土體侵蝕管和局部張開的垂直裂縫、殘餘節理和岩脈，延伸到削土坡範圍內的地表，容許地表徑流快速滲透進入到斜坡內。 在地表下約一米至六米之間，相對淺的基岩界面，水壓可能沿面狀節理和接近地表內的節理及風化花崗岩層形成。 結果，沿著潛在滑動面的剪切抵抗力被減小很多。

山泥傾瀉的崩塌機制是由斜坡內業已存在的，不利穩定的，沿著地質的不連續面走向所控制。 主滑動面沿著高度延續的、以平均約 52 度角傾向南到東南面的面狀節理形成的。 面狀節理的傾角和深度是如此，因而當它接近地表時，實際上既不能出露在削土坡也不能出露在斜坡面上。 但是，小傾角節理的存在，出露在崩塌前削土坡的中部，形成山泥傾瀉低部的破裂面。 在山泥傾瀉最深部份，一組連續的垂直岩脈和殘餘節理沿著西南側為崩塌提供了釋放面。

出露在滑坡崖，張開的、局部充填的、近於垂直裂縫的存在是斜坡過去移動的證據。 這個形成於 1924 年到 1945 年之間的削土坡，以大約 55 度至 75 度的削土坡坡角，在相當大的程度上高過當前設計標準下，在風化花崗岩中，超過了未加固削土坡可接受的坡角，達到了削土坡的形狀可能在削土坡之上的斜坡內產生斷裂的程度。 在這個應力體系下，隨後的暴雨期間，山坡和削土坡的移動使得張開的、近於垂直裂縫形成並沿著如面狀節理的不連續面擴大，因此，山泥傾瀉前風化岩體上的裂縫可能比在削土坡形成時更加大了。 自一九六三年當淺層侵蝕發生時（第四節），斜坡灰泥護面狀況極差及缺乏任何排水設施，表明斜坡缺乏維護。 這些原因導致雨水的滲透和地表徑流進入斜坡內造成風化岩體的進一步變形。

10. 結論

所得的結論：一九九七年發生於萬佛寺的山泥傾瀉，可能主要是由於事發前的豪雨，使不利於穩定的沿著不連續面走向內的水壓上升所引致的。

事發前削土坡缺乏足夠的設計和維修，導致斜坡局部逐漸惡化。

11. 參考書目

Addison, R.(一九八六) Geology of Sha Tin. 香港土力工程處，85 頁。 (Hong Kong Geological Survey Memoir No.1)。

BSI(一九九零 a) Methods of Test for Soils for Civil Engineering Purposes. (BS1377:1990) Part 7: Shear Strength Testing (Total Stress). British Standards Institution，London，48 頁。

BSI(一九九零 b) Methods of Test for Soils for Civil Engineering Purposes. Part 8: Shear Strength Testing (Effective Stress). British Standards Institution，London，28 頁。

Chen，P.Y.M(一九九四) Methods of Tests for Soils in Hong Kong for Civil Engineering Purposes (Phase 1 Tests). 香港土力工程處，91 頁。(土力工程報告第 36 號)。

土力工程處(一九八六) 沙田:solid and superficial geology.， Hong Kong Geological Survey. Map Series HGM 20，第 7 張， 比例 1:20000。香港土力工程處。

土力工程處(一九八七) Geotechnical Area Studies Programme – Central New Territories. 香港土力工程處，第 II 號 GASP 報告，165 頁加 4 張圖。

Lam, C.C.&Leung, Y. K.(一九九四) Extreme Rainfall statistics and design rainstorm profiles at selected location in Hong Kong. 香港天文台, Technical Note no.86，89 頁。

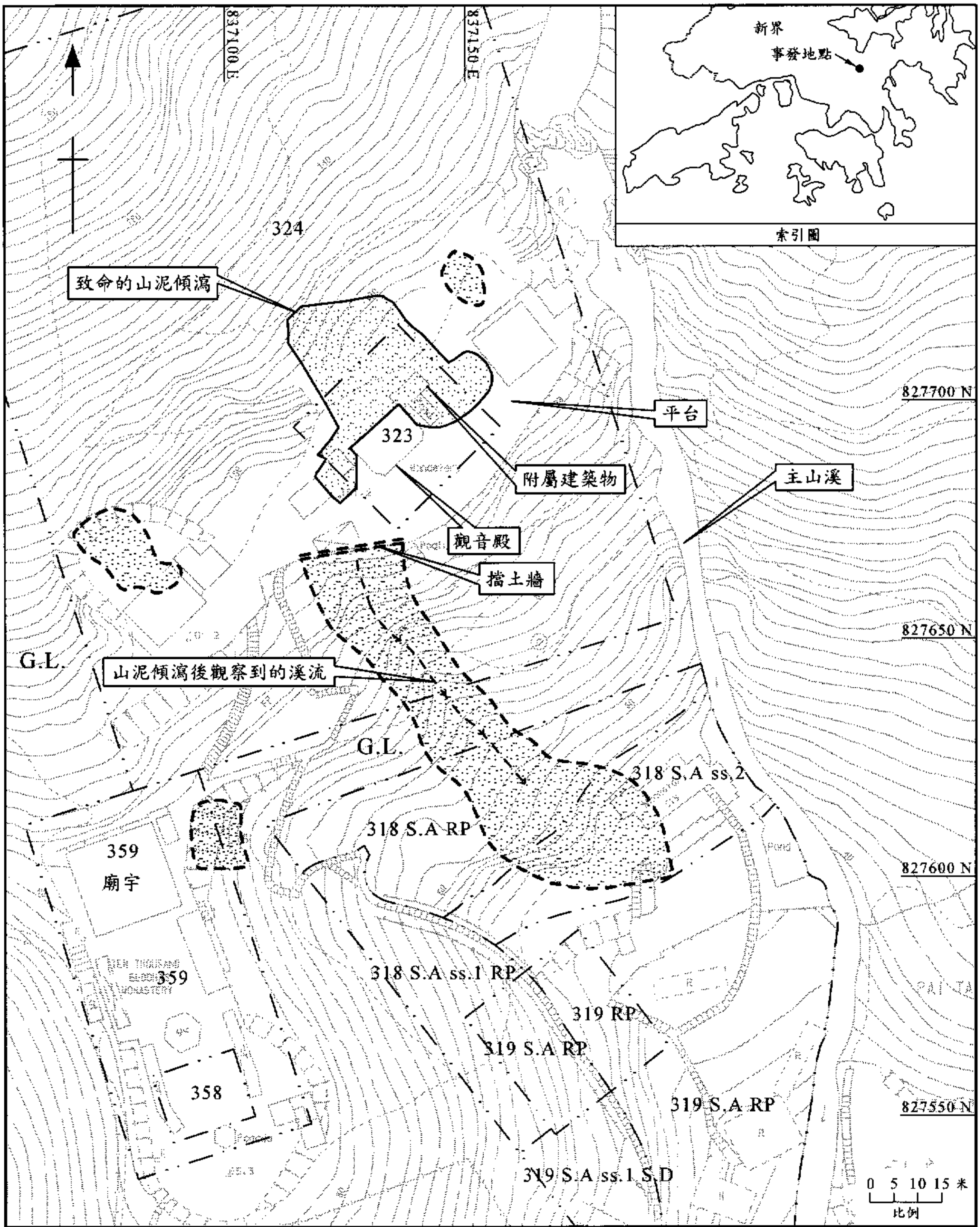
Ordance Survey Office (一九零四) Topographic Map of 1904，Sha Tin，Sheet No. 12. Ordnance Survey Office, Southampton, United Kingdom, 一九零四。

Papaliangas, T.T, Hencher, S.R. & Lumsden, A.C. (一九九五) A comprehensive peak shear strength criterion for rock joints. 8th International Congress on Rock Mechanics， Tokyo，第 359 – 366 頁。

Richards, L.R.&Cowland, J.W. (一九八二) The effect of surface roughness on the field shear strength of sheeting joints in Hong Kong granite. Hong Kong Engineer， vol. 10，no.10，第 39 – 43 頁。

Wong, H.N.&Ho, K.K.S. (一九九六) Travel distance of landslide debris. Preceedings of the Seventh International Symposium on Landslides， Trondheim，Norway，vol. 1，第 417 – 422 頁。

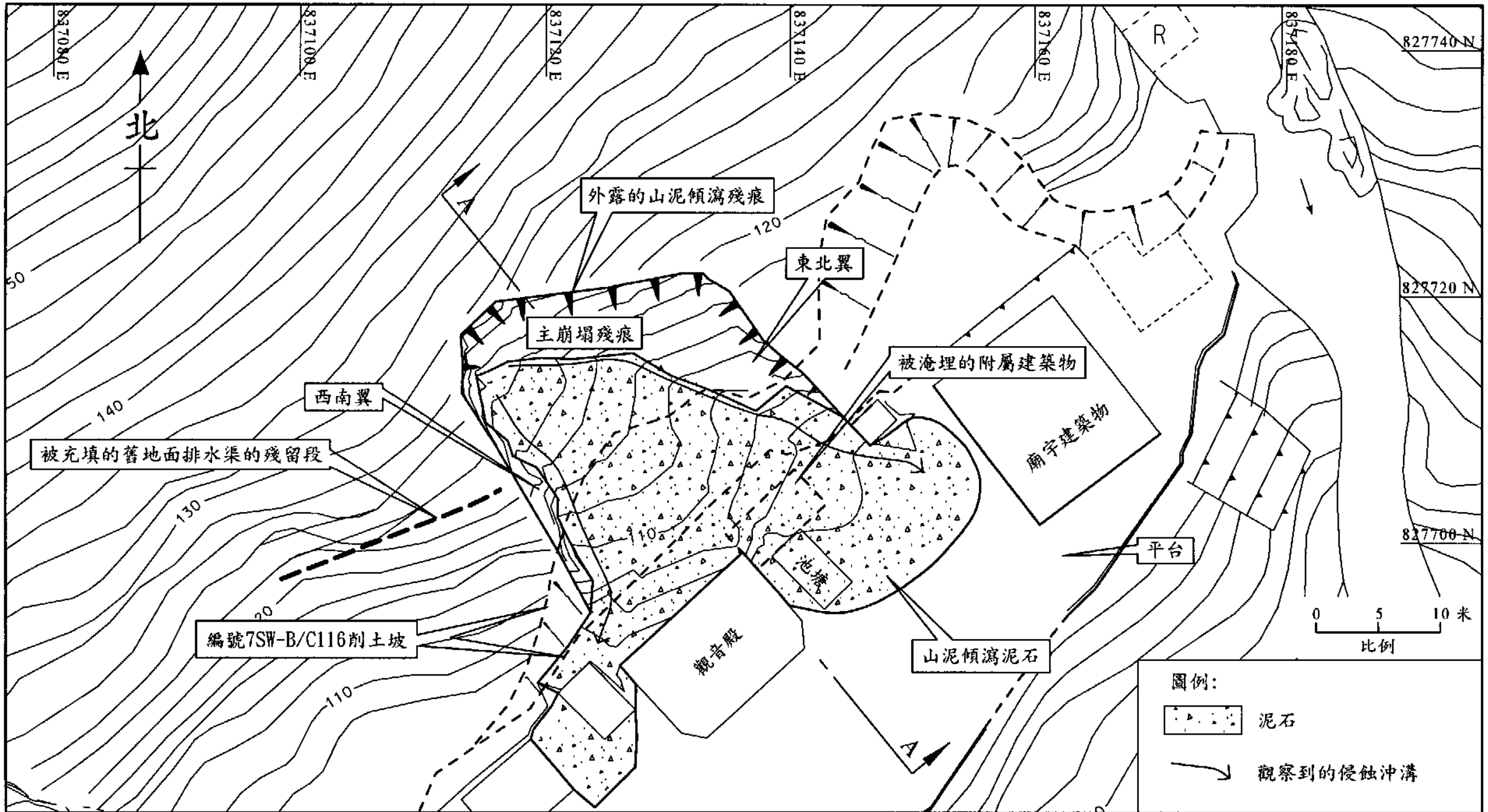
圖



圖例:				
	▨	一九九七年七月二日致命的山泥傾瀉的範圍	— · · —	地段界線
	▨	一九九七年七月二日其他山泥傾瀉的範圍	324	私人地段編號
			G.L.	政府用地

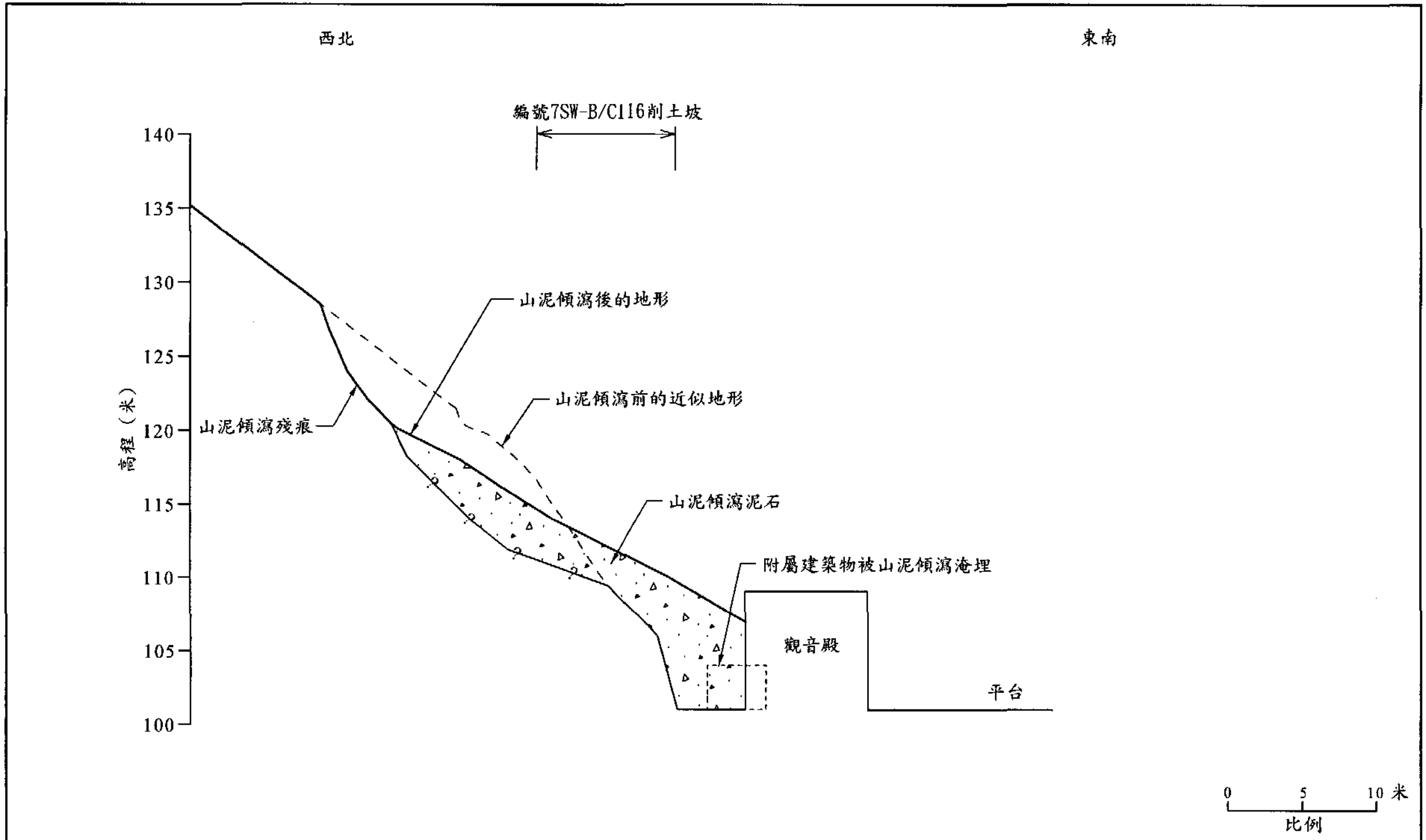
註: 取自一九九二年十月編號 7-SW-10D 及 7-SW-15B 測量圖 (比例 1:1 000).

圖1 - 山泥傾瀉位置圖



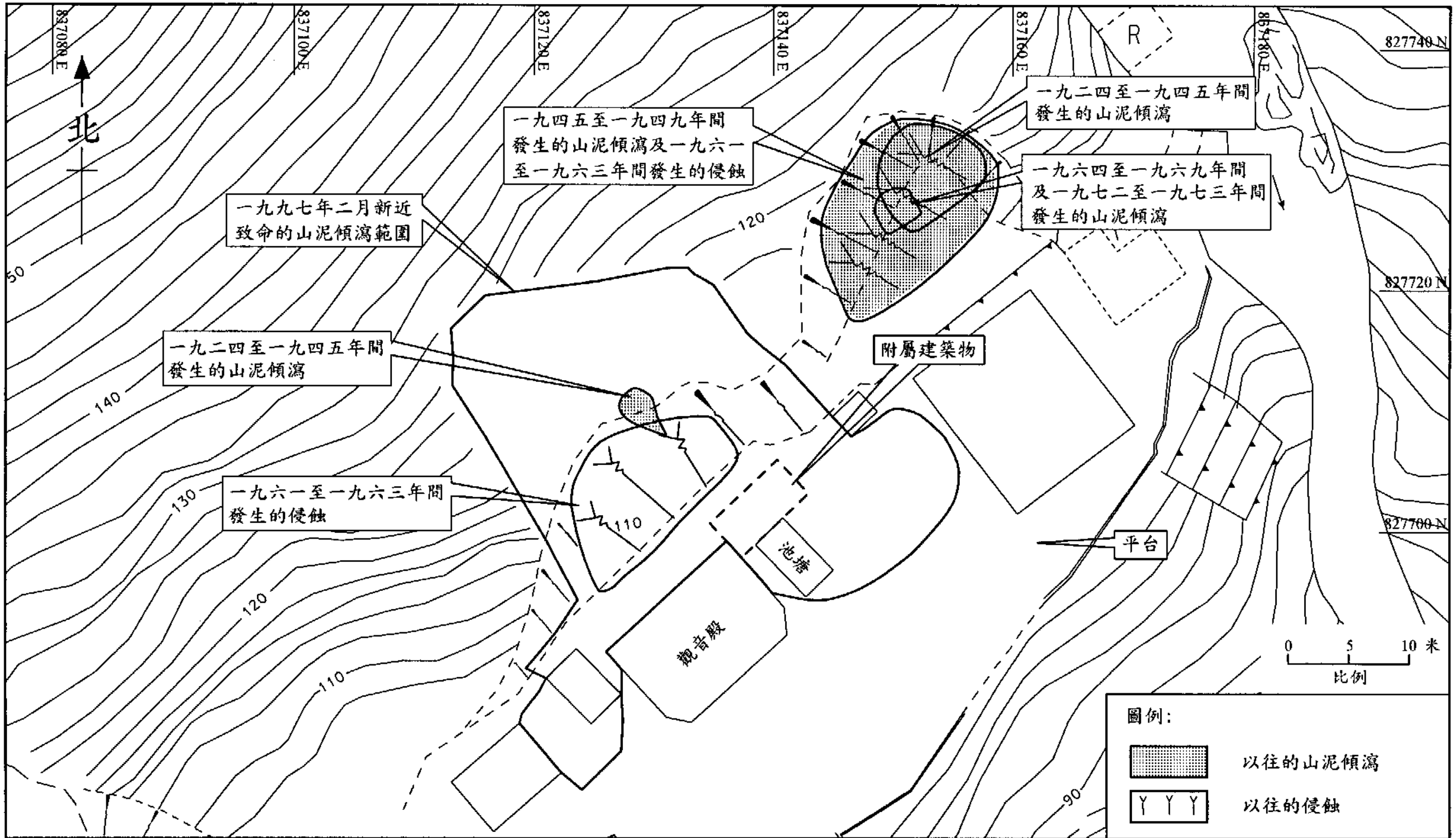
註: (1) 剖面 A-A 見圖3.
(2) 圖中所示資料以地形測量圖, 地質勘察, 現場觀察及文件記錄為根據.
(3) 底圖是根據一九九二年十月編號7-SW-10D測量圖(比例1:1 000)及利用攝影測量分析一九六三年的航測照片.
(4) 山泥傾瀉殘痕及泥石範圍內的等高線是從土木工程署在山泥傾瀉發生後進行的地形測量所得.

圖2 - 山泥傾瀉平面圖



註： (1) 剖面的位置見圖2.
(2) 本圖中所示資料以地形測量, 現場觀察及文件記錄為根據.

圖3 － 剖面 A-A 顯示山泥傾瀉地點的詳情



註　(1) 圖中所示資料以地形測量圖, 現場觀察及航空照片為根據.

(2) 底圖是根據一九九二年十月編號7-SW-10D測量圖(比例1:1 000)及利用攝影測量分析一九六三年的航測照片.

圖4 － 以往幾次山泥傾瀉及侵蝕事件的平面位置圖

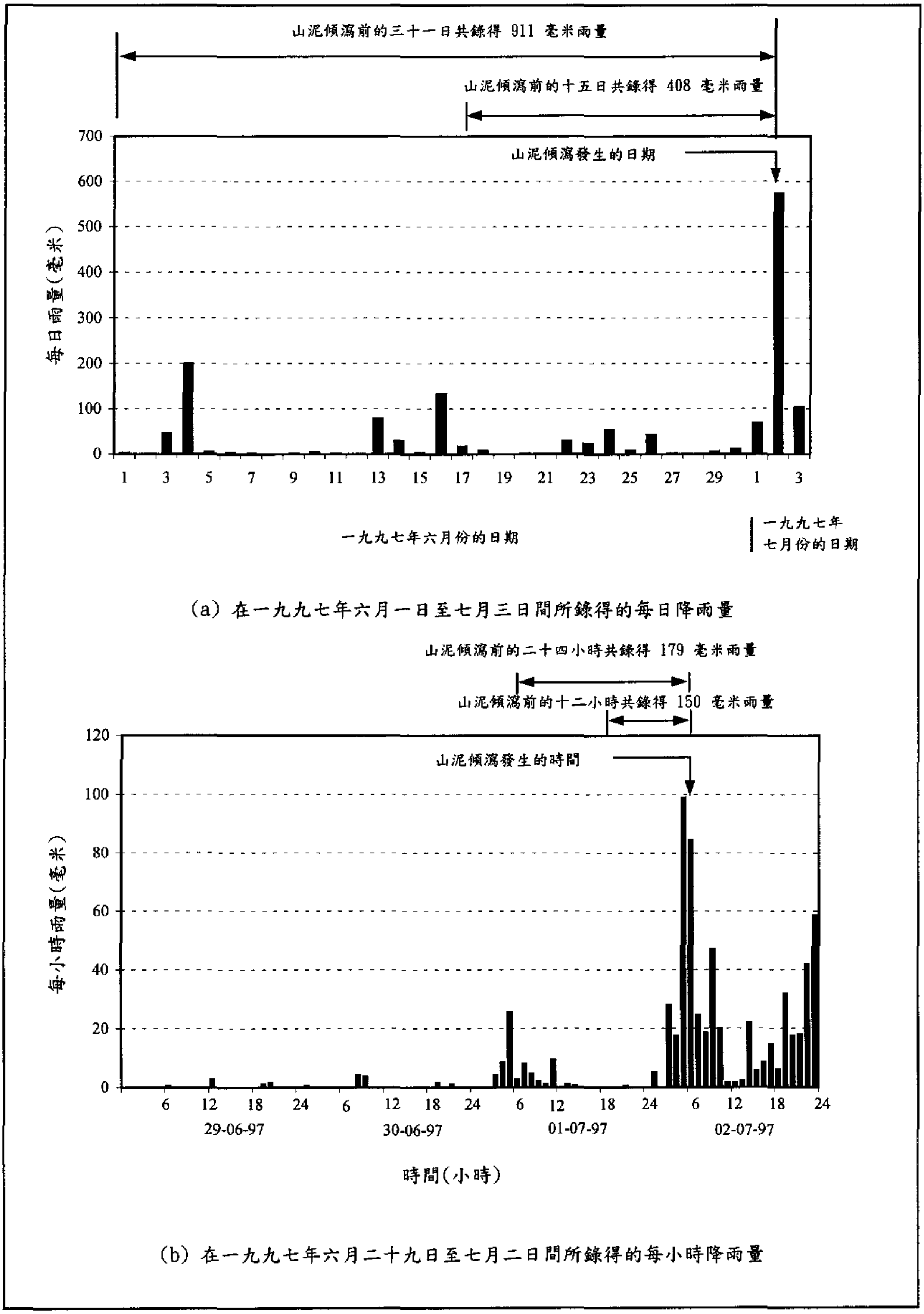


圖5 - 土力工程處編號 N02 雨量計的雨量記錄

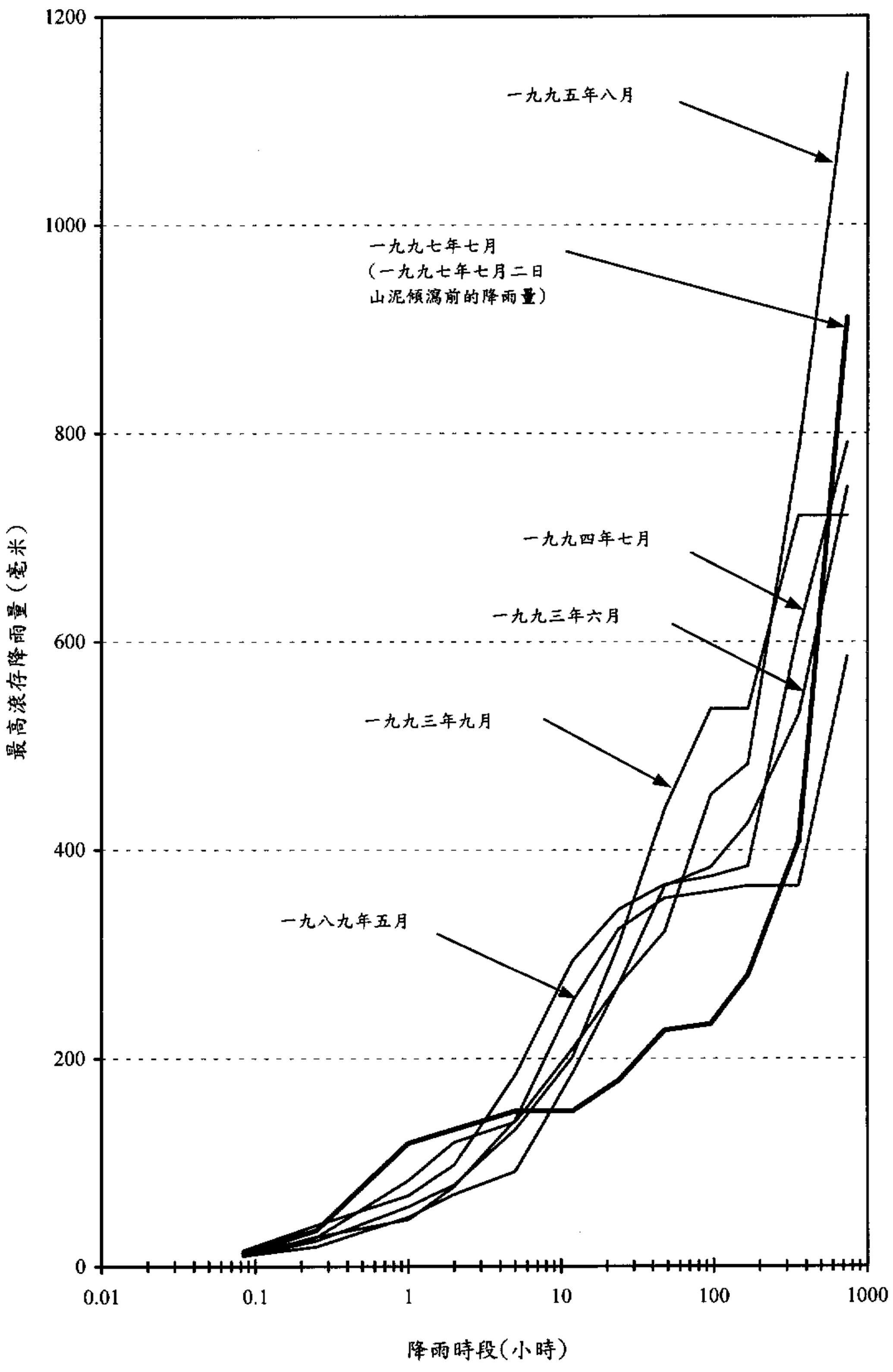
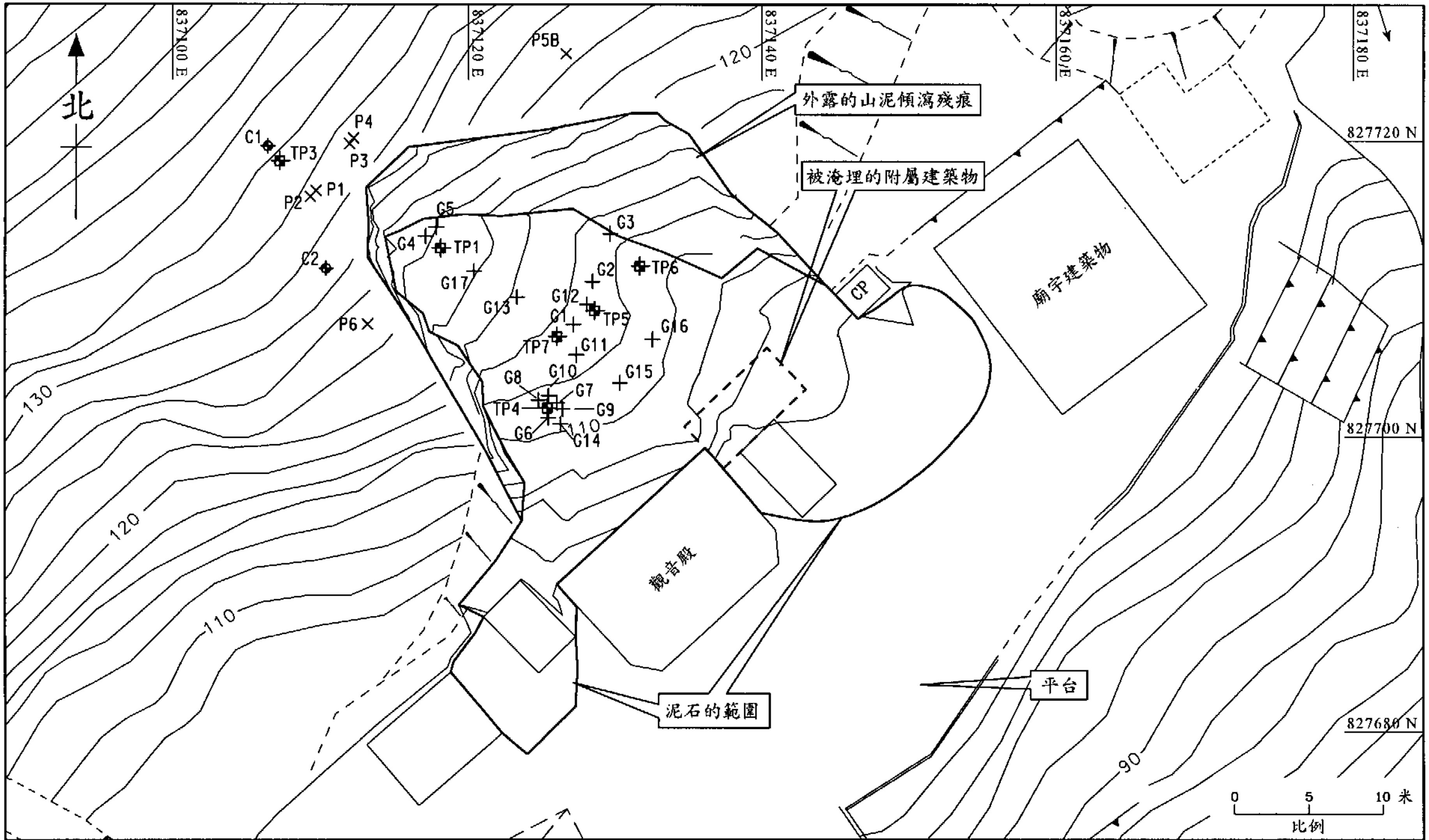


圖6 － 土力工程處編號 N02 雨量計於歷次豪雨中錄得的最高滾存降雨量



註: 圖中所示資料以地形測量圖, 地質勘察, 現場觀察及文件記錄為根據.

圖例:
- 鑽孔
- 探井
- + GCO 輕型動力觸探孔
- × 安裝有張力計的 GCO 輕型動力觸探孔

圖7 – 場地勘探工程的位置圖

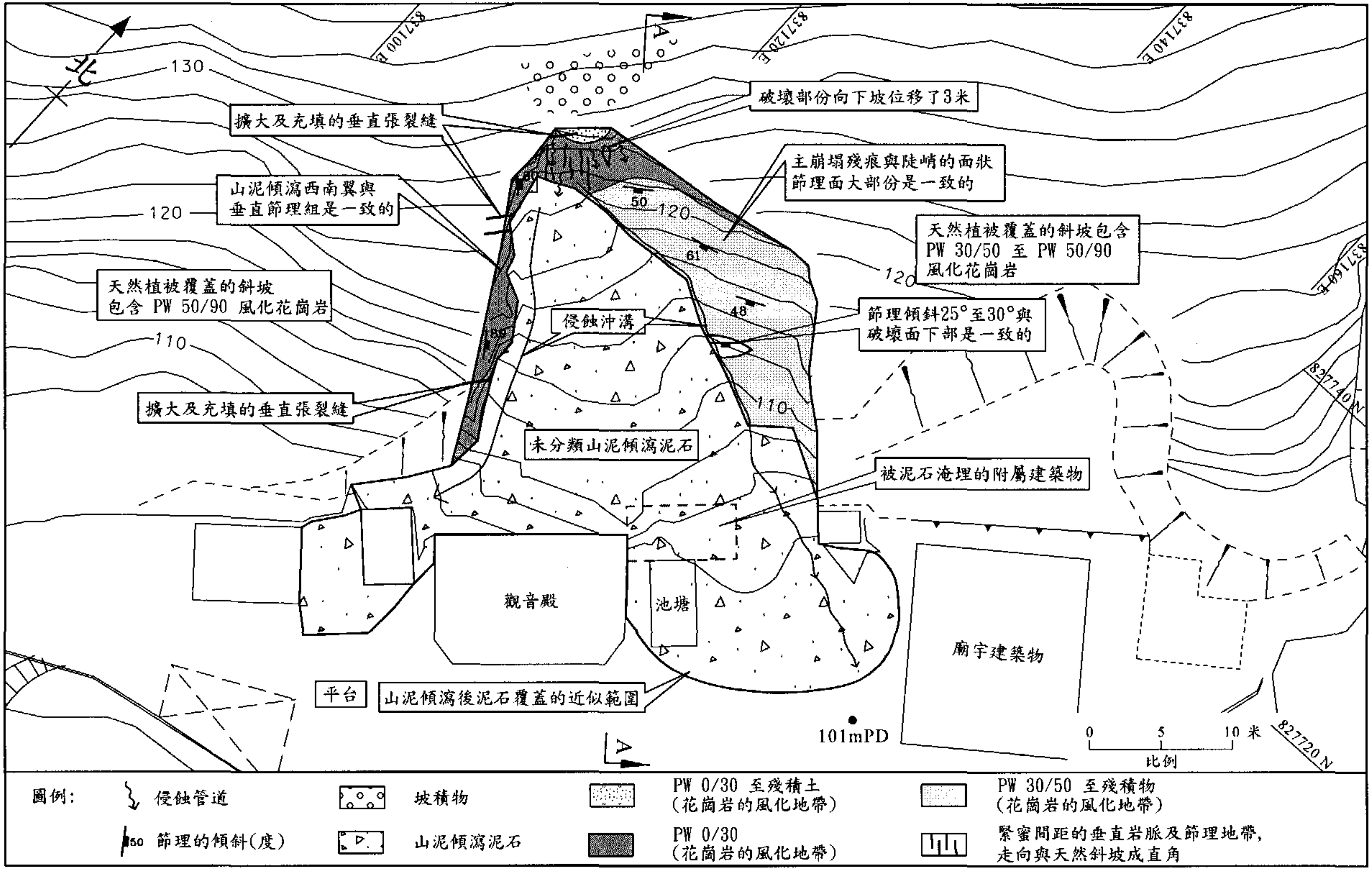


圖8 － 山泥傾瀉地點的地質圖

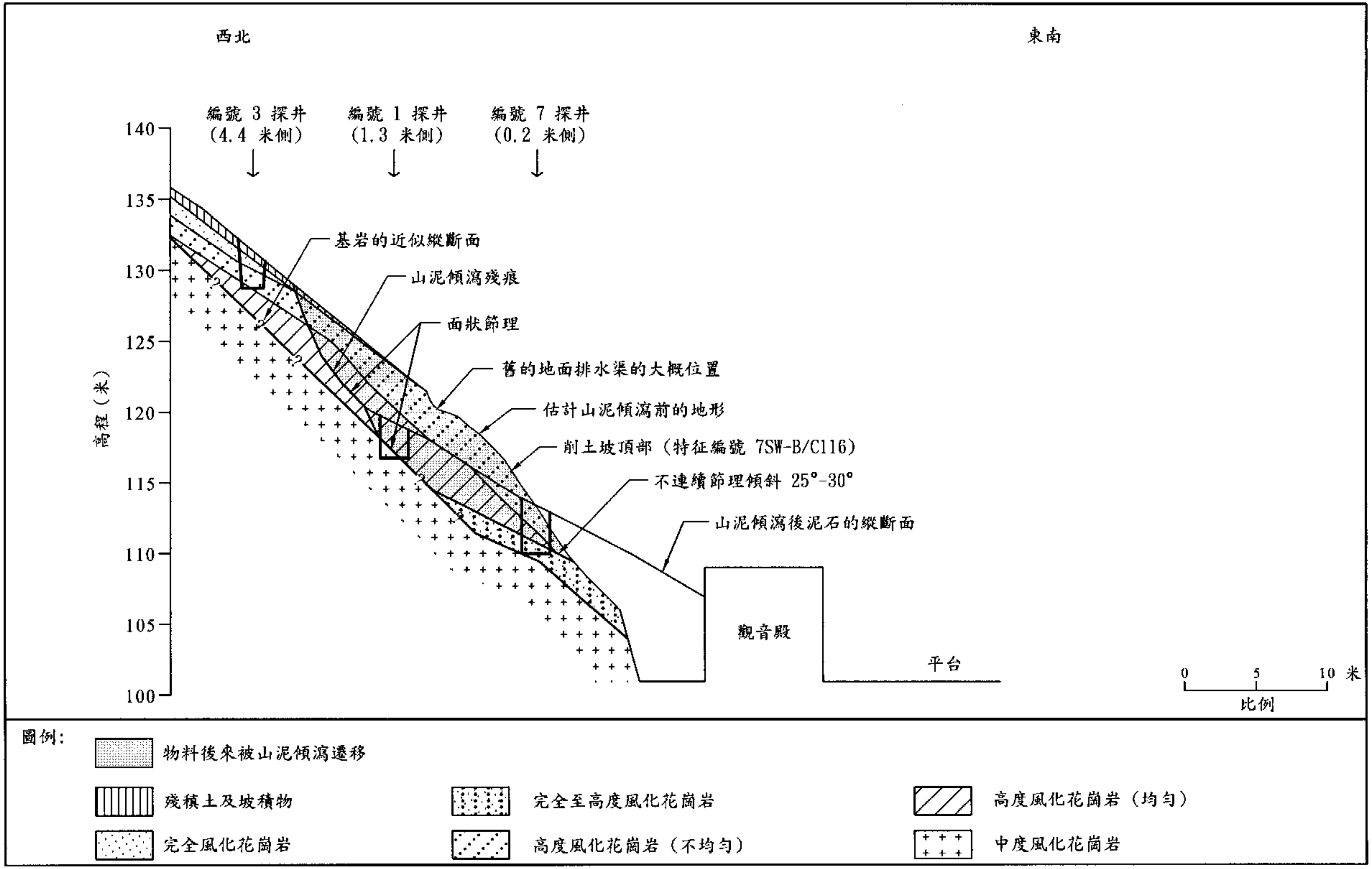
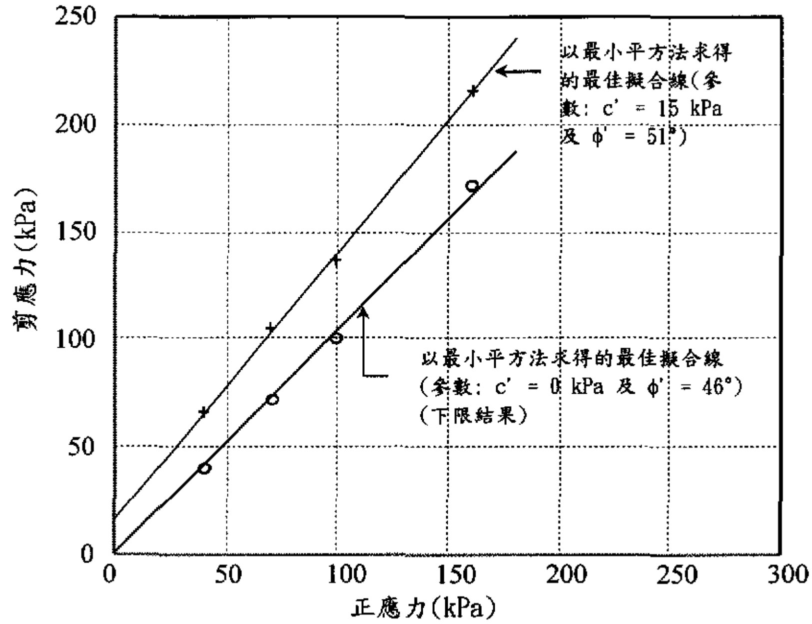
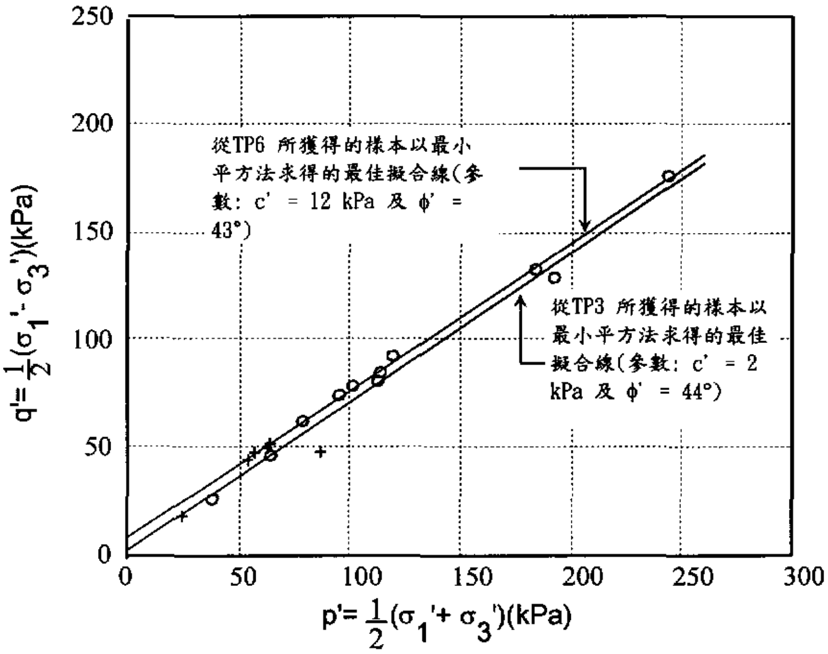


圖9 - 山泥傾瀉地點的典型地層剖面 A-A



(a) 高度風化花崗岩的直接排水剪切試驗所得的結果



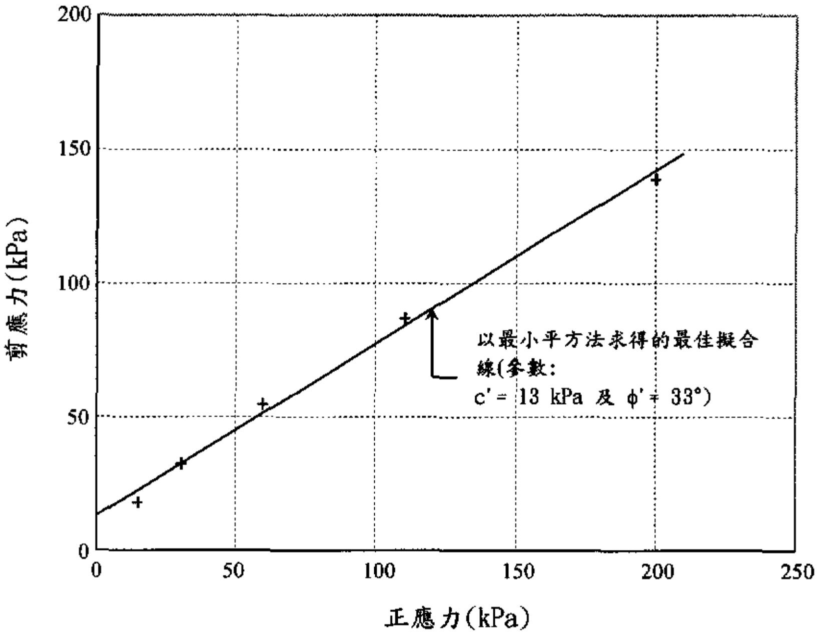
(b) 全風化花崗岩進行包括量度孔隙水壓的固結不排水三軸壓縮試驗結果

圖例:	+	單階段試驗 (HDG及CDG)	σ_1'	最大有效主應力	c'	黏聚力(有效)
	o	多階段試驗 (HDG及CDG)	σ_3'	最小有效主應力	φ'	抗剪角(有效)

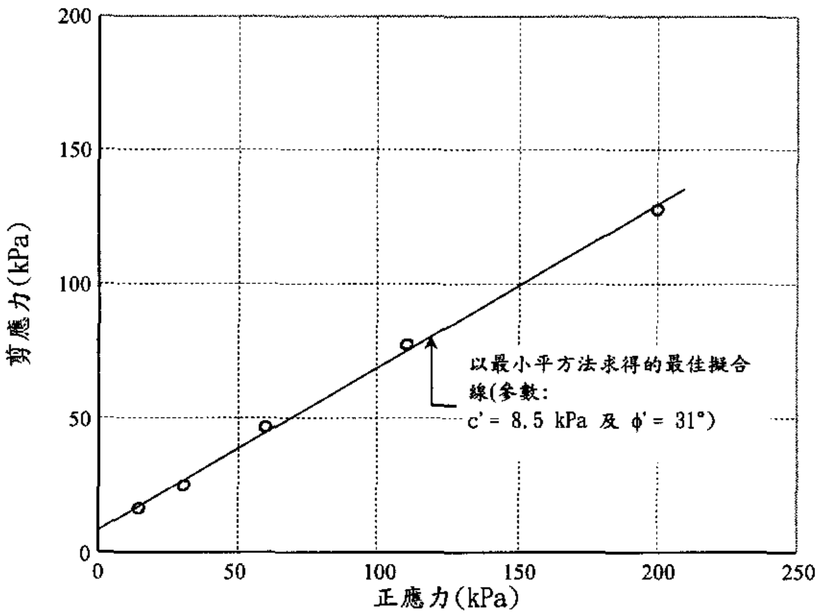
註: (1) 本圖所示的直接抗剪試驗結果，是相應於峰值抗剪應力的結果，除下限結果亦即是第三階段試驗在不變正應力下的最終結果. 三軸壓縮試驗的結果則相應於最大應力比率的結果，即最大 σ_1'/σ_3' 比率.

(2) TP3 及 TP6 探井的位置見圖 7.

圖10 - 高度及全風化花崗岩的直接剪切和三軸壓縮試驗結果



(a) 峰值剪切強度試驗結果

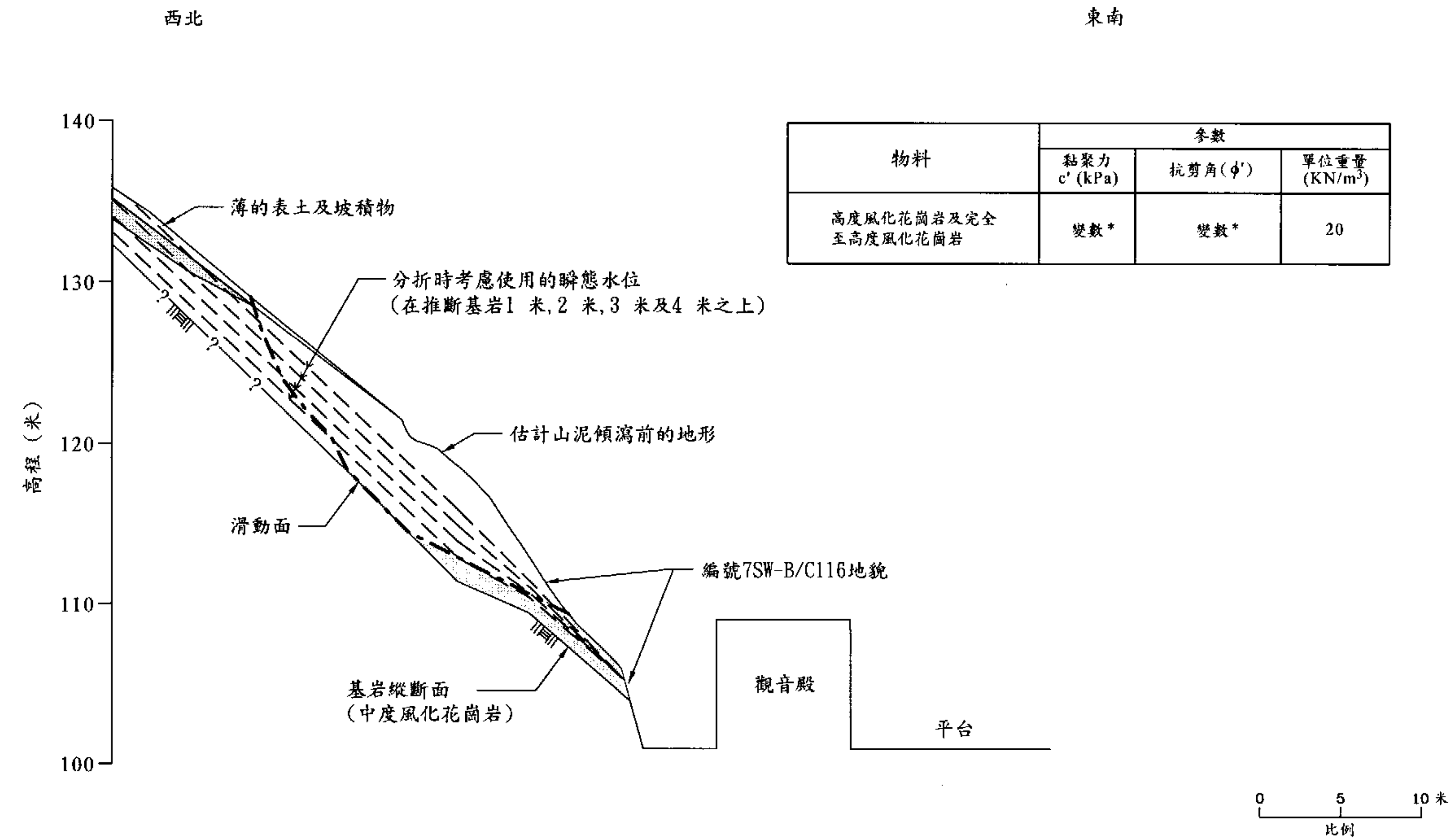


(b) 殘餘剪切強度試驗結果

圖例:				
	+	單階段試驗	c'	黏聚力(有效)
	o	多階段試驗	φ'	抗剪角(有效)

註: 試驗用物質是經過重造的

圖11 - 淺灰色含少量砂質黏土的粉土直接剪切試驗結果

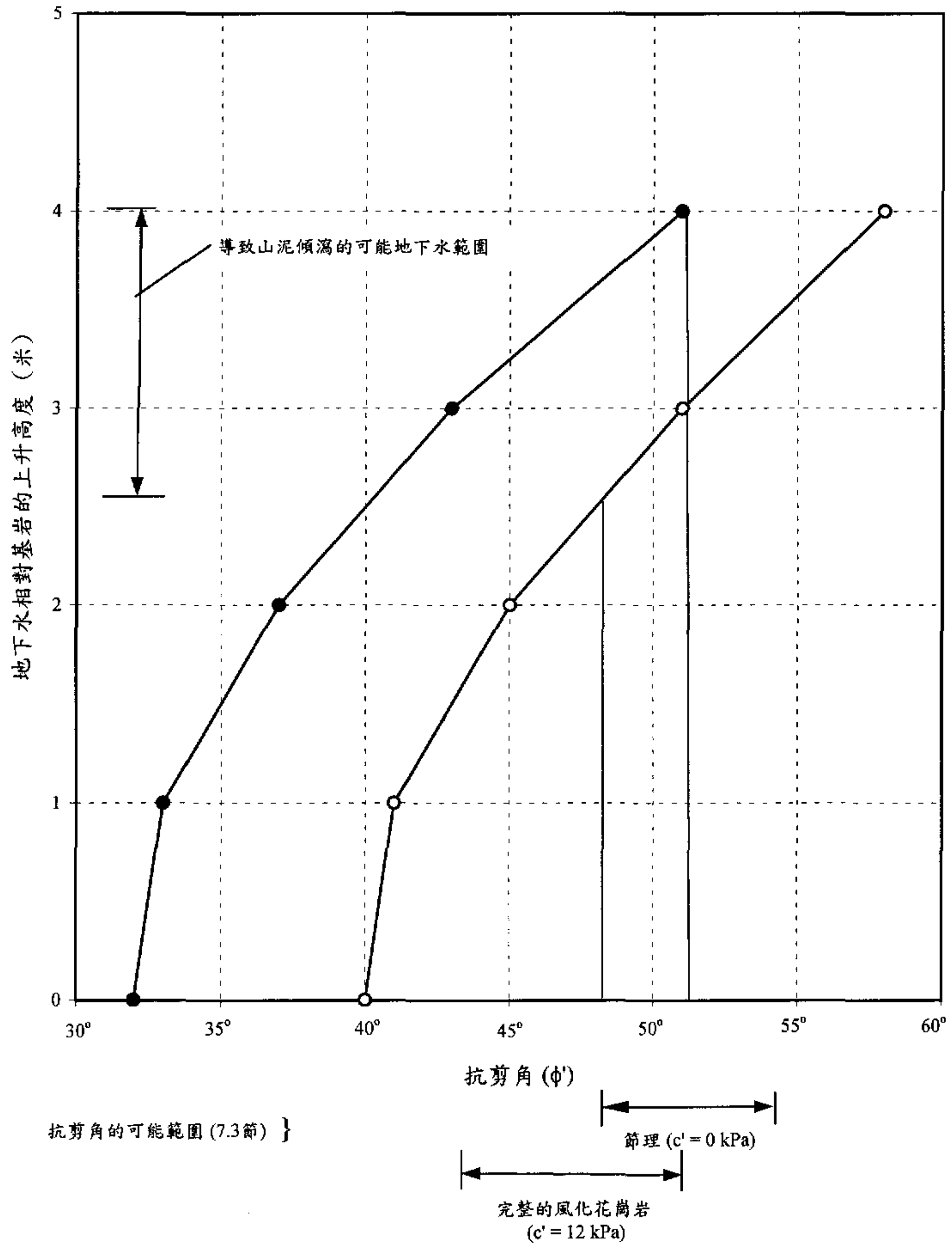


物料	參數		
	黏聚力 c' (kPa)	抗剪角(φ')	單位重量 (KN/m^3)
高度風化花崗岩及完全至高度風化花崗岩	變數*	變數*	20

註： (1) 剖面的位置見圖2.
(2) 圖中所示資料以地形測量，1:1 000測量圖，攝影測量圖，地質勘察，現場觀察及文件記錄為根據.

圖例：
* 見圖13
完全至高度風化花崗岩
高度風化花崗岩

圖12 － 理論穩定性分析所採用具代表性的山泥傾瀉剖面



圖例:
- ● c' = 12 kPa
- ○ c' = 0 kPa

註 : 1. 分析詳情參考圖12
2. 在安全系數 = 1.0 時所得的結果

圖13 - 理論穩定性分析的結果

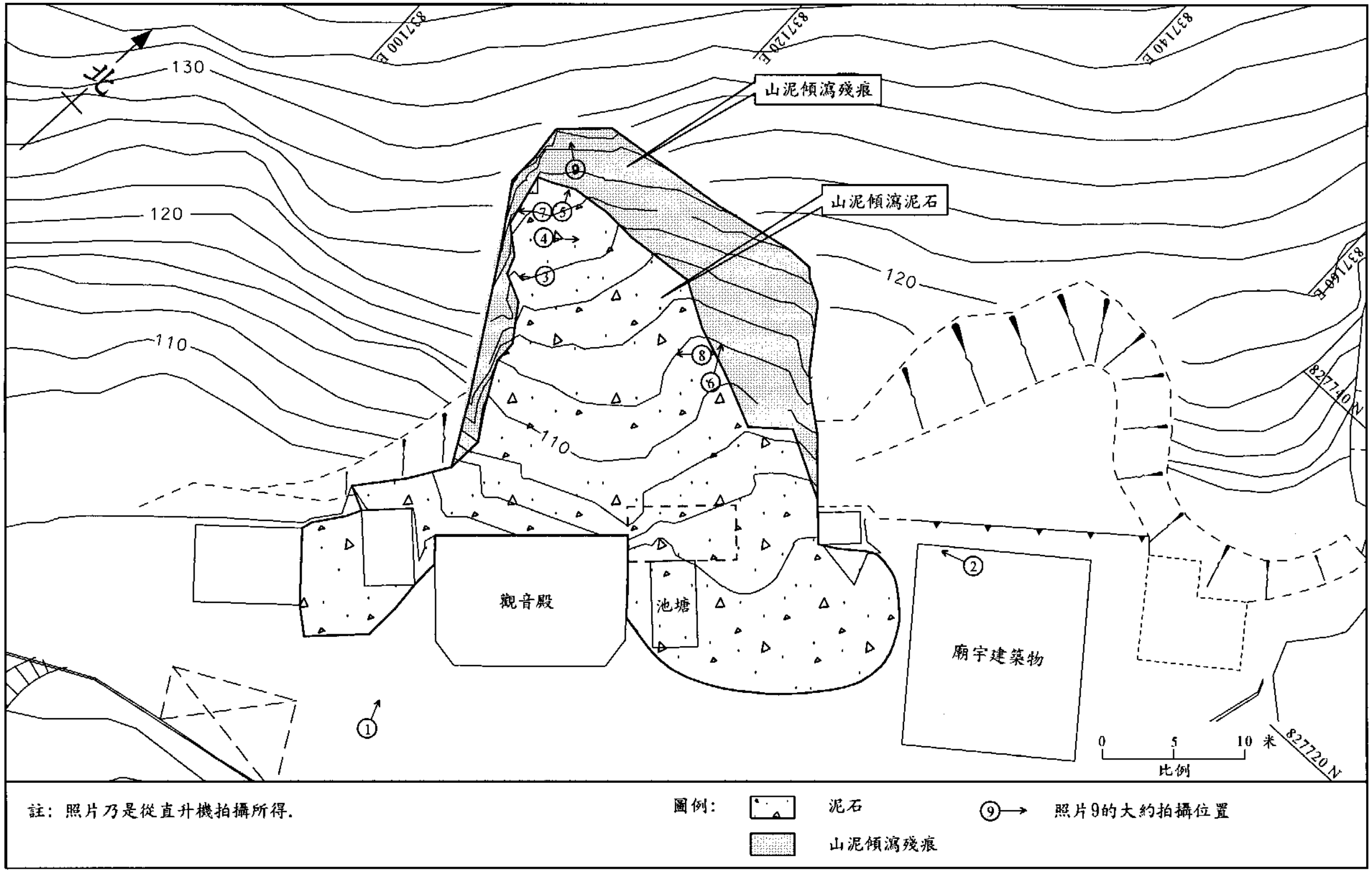
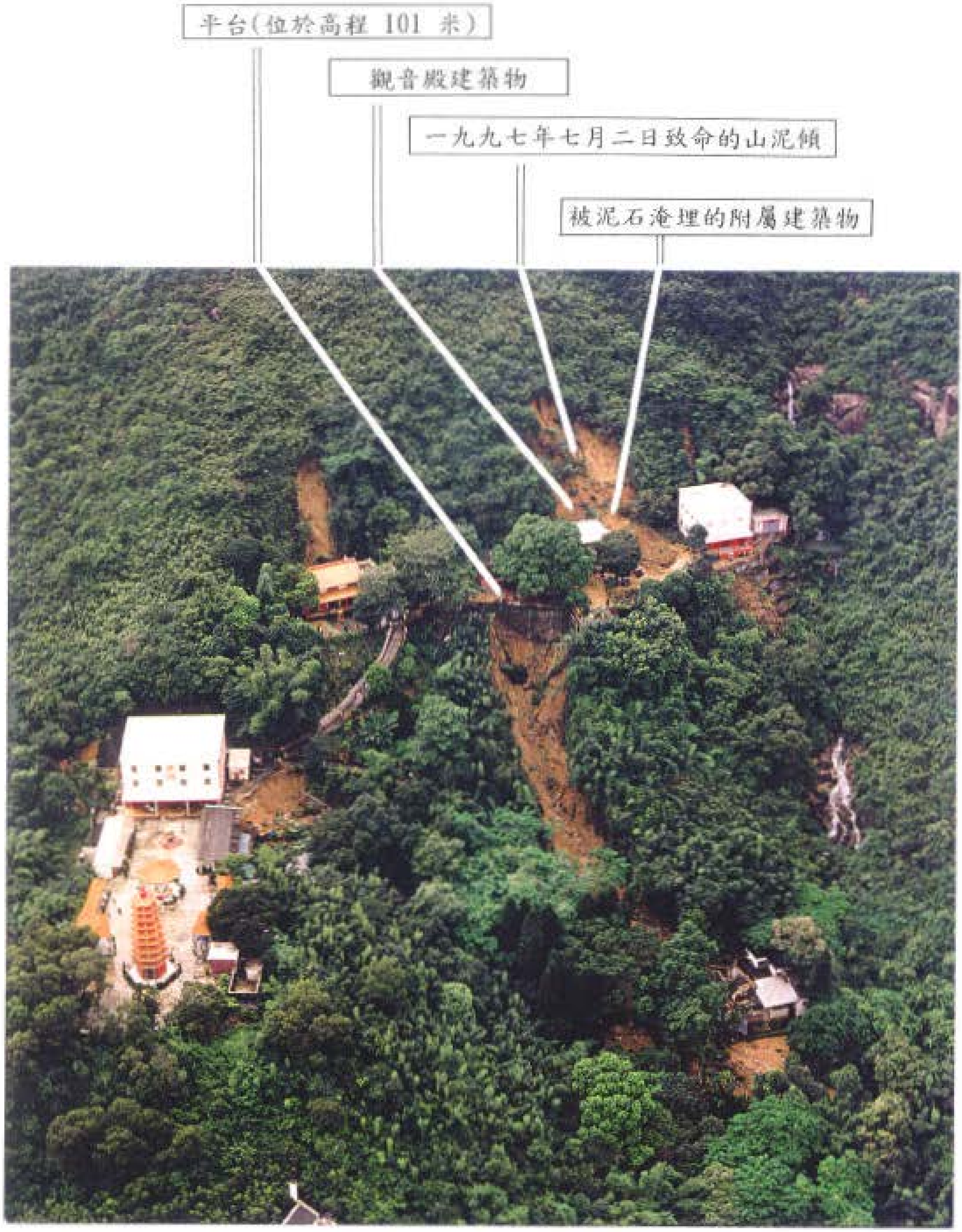

北
130
120
110
837100 E
837120 E
837140 E
837160 E
827740 N
827720 N
山泥傾瀉殘痕
山泥傾瀉泥石
觀音殿
池塘
廟宇建築物
0 5 10 米
比例
註: 照片乃是從直升機拍攝所得.
圖例:
泥石
山泥傾瀉殘痕
照片9的大約拍攝位置

圖14 - 照片位置圖

照片



照片 1 － 山泥傾瀉地點的傾斜航空景象(一九九七年七月三日下午拍攝，其位置見圖 14)

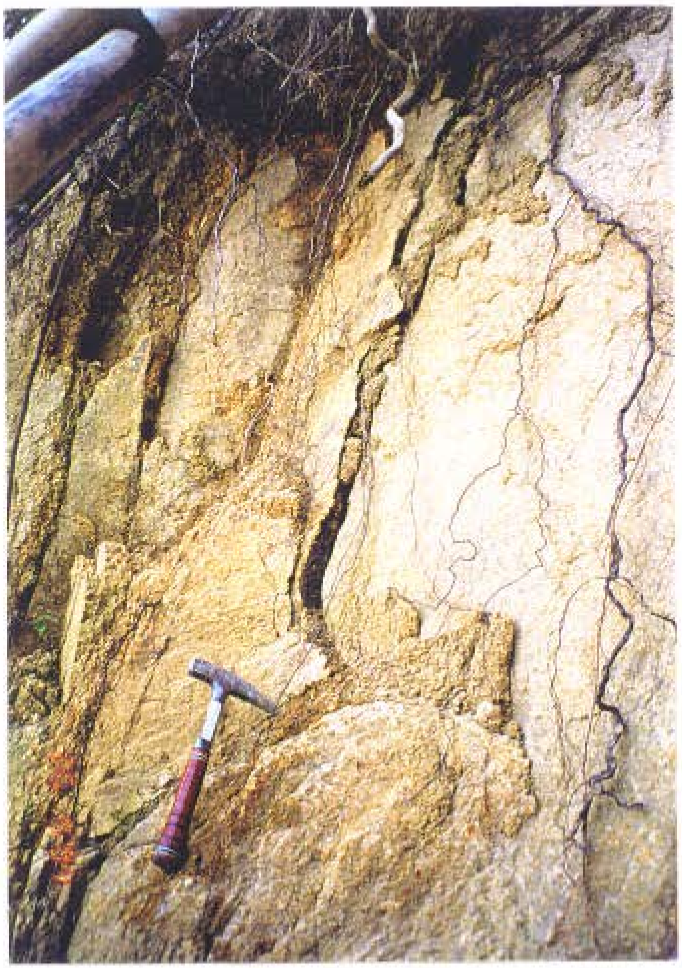
照片2 － 山泥傾瀉的現場景象
（一九九七年七月八日拍攝，
其位置見圖14）

照片3 － 山泥傾瀉西南翼的景象(一九九七年七月七日拍攝， 其位置見圖14。 注意地表的灰色的風化花崗岩塊體及在照片右下角的垂直面狀的不連續面，與一組主要的近於垂直的紋理和節理相互聯系)

照片4 － 面狀節理面的景象(一九九七年七月七日拍攝，其位置見圖14， 注意
山泥傾瀉的東北翼與天然斜坡結合成一體)

照片5 - 主滑坡崖上部的景象，顯示部分風化花崗岩帶
(0-30%岩石至殘積土)及面狀節理面
(一九九七年七月七日拍攝，比例為1米的
卷尺，其位置見圖14)

照片6 － 在侵蝕沖溝裏外露的節理，傾角為30度，形成破裂面的下部
（一九九七年八月四日拍攝，其位置見圖14 ）



照片7 － 位於山泥傾瀉的西南翼近於垂直有充填的裂縫的景象（一九九七年九月二十六日拍攝，其位置見圖14 ）

照片8 － 沿著山泥傾瀉破裂面的下部的棕色軟黏土質砂
(一九九七年十一月十二日拍攝，注意棕色的粘
土層的厚度與鉛筆長度相同，其位置見圖14)

照片9 － 在主崩塌崖東北角的侵蝕管的景象(一九九七年
九月二十三日拍攝，其位置見圖14)

附錄 A

事發地點歷史摘要

目錄

A.1 引言

參考目錄

事發地點的歷史是獲取自：

(a) 舊有的地形圖，

(b) 一九二四年至一九九六年期間的一系列航空照片，及

(c) 土力工程處的文件記錄。

地形圖， 沙田，圖號 12. Ordinance Survey Office，Southampton. 1904

本段所討論到的各斜坡的位置，包括建築物用的參考編號及過往山泥傾瀉的地點，顯示在圖 A1 中。

A.2 事發地點的歷史發展

現存最早的航空照片，拍攝於一九二四年，顯示事發地點並沒有開發。 在一九二四年至一九四五年期間，修築了一個平台，包括對天然山坡的開挖及在擋土牆之後的填土。 編號 4 的建築物（觀音殿），小的附屬建築物（導致喪生的地點）以及編號 5 和 6 的建築物的前身在一九四五年時已經存在（圖 A1）。 平台後部的人造斜坡，包括由編號 4 和 5 建築物後面的有灰泥護面的削土坡所構成，斜坡典型高度為 15 米，下部高 5 米和上部高 10 米，傾斜角分別約為 75 度和 56 度。

在一九五四年，一個新的開挖及填土平台，通過對原有的平台向西南方向擴展修建而成。 編號 1 的建築物在同一年修建在新平台上。編號 1 建築物後面的削土坡高約 23 米，傾斜角度約 55 度。

編號 5 的建築物的前身在一九五六年被拆除。 其後，一個較大型的建築物在較接近削土坡坡腳之處修建。 在這次導致喪生的山泥傾瀉之前仍然存在於平台上的其它建築物，是在一九六三年前修建而成的。

Crown Lands and Survey Office，HongKong. Sheet No. C-145-NE-D，1966 年修訂

在一九四九年至一九六一年期間，進行了小型的開挖，貫穿削土坡頂部的上方留下了一條小徑。 這恰好和舊有的人工排水渠的位置是一致的，其殘餘部分在毗鄰新近導致喪生的山泥傾瀉的西南翼的山坡上被觀察到。

自一九五四年以來，平台上佈滿了濃密的植被，直到一九九零年，大量的樹木和灌木從平台上被鏟除，並且進行了表面工程的整修。

A.3 過往的評估

A.3.1 斜坡的登記

在編號 4 和 5 的建築物後面的削土坡沒有登記在一九七七/一九七八年度斜坡記錄冊中。 然而，在一九九六年六月，通過土力工程處執行一項名為"有系統勘察調查全港的斜坡及擋土牆"計劃 (SIFT)的項目，該削土坡被鑒定出來。該項目的目的是，利用航空照片有系統地勘察出過去沒有記錄在一九七七年/七八斜坡記錄冊中的，較大的人造斜坡，補充最新資料在已有的記錄冊中。

SIFT 第二階段研究圖則報告，圖則編號 7SE-10D，策劃部，土力工程處 一九九六年 SIFT 第二階段研究

隨後，在另一項由土力工程處執行的名為"有系統鑒定和登記全港的斜坡及擋土牆"計劃(SIRST)的項目裏，該削土坡被登記為編號 7SW－B/C116 的削土坡。 該計劃是有系統地對一九七七/七八斜坡記錄冊進行補充最新資料，編輯新的斜坡記錄冊。 土力工程處指定執行"有系統鑒定和登記全港斜坡及擋土牆"項目 的顧問工程師於一九九六年十二月四日對該削土坡進行了勘查。 該削土坡根據記錄有 8 米高及 40 度的傾角，並且，該削土坡頂部的山坡被評定為一般狀況，沒有滲流或斷裂的跡象。 但是該削土坡在崩塌事件後果目錄裏被評定為嚴重的後果。 該顧問工程師於一九九七年三月十三日提交的第一階段調查報告中，建議對該削土坡進行進一步的研究。

賓尼 SIRST 編號 7SE-B/C116 削土坡的現場記錄表

在一九五四年，在編號 1 的建築物後面的削土坡，通過一個新的開挖及填土來擴大平台而形成的，在"有系統鑒定和登記全港的斜坡及擋土牆"計劃中，該削土坡被登記為編號 7SW－B / C113 的削土坡。 土力工程處指定執行"有系統鑒定和登記全港斜坡及擋土牆"項目的顧問工程師於一九九六年十二月四日對該削土坡進行了勘查，發現了在噴漿護面的削土坡坡面上有較小的斷裂的跡象，並且在崩塌事件後果目錄裏被評定為嚴重的後果。該顧問工程師於一九九七年三月十三日提交的第一階段調查報告草稿中，建議對該削土坡進行進一步的研究。

A.3.2 調查及研究

在一九零四年的地形圖上(Ordnace Survey Office，1904)，顯示五條較小的溪流，穿過了後來新形成的平台地點。其中一條溪流看來穿過了導致喪生的山泥傾瀉的地點。 這五條溪流在隨後的測量圖上沒有顯示出來。

一九二四年拍攝的航空照片也顯示出事發地點在未開發前的排水型式(圖 A1)。 兩條短期存在的溪流河道顯示出來，通過了編號 3 的建築物後面的古老滑坡崖，然後匯合成一條溪流河道。 這條溪流的位置與，一九九七年七月二日發生山泥傾瀉後，在平台擋土牆之下的滑坡崖的位置是一致的。從一九二四年的航空照片，顯示出另一條溪流河道通過編號 4 和 5 的建築物。

除了土力工程處指定執行"有系統鑒定和登記全港斜坡及擋土牆"計劃的顧問工程師所進行的勘察外，在一九九七年發生崩塌前沒有任何對該削土坡的勘察或評估的記錄。

A.4 過往的山泥傾瀉

根據土力工程處的紀錄，一九九七年山泥傾瀉事發地點過往沒有山泥傾瀉發生報告。 在土力工程處的天然滑坡詳細目錄裡，也沒有過往的天然地形滑坡發生記錄(Evan et al，一九九七)。 作為本次調查的一部分，從對老的航空照片的研究發現，在山泥傾瀉事發地點附近，觀察到可能有十二次崩塌發生在這次山泥傾瀉地區的附近，其中七個發生在一九二四年至一九七三年之間，其餘的可能是一九二四年以前發生的山泥傾瀉(圖 A1)。 航空照片也表明曾有三次侵蝕事件，其中的一個坍塌體積約 20 立方米，其中的一次侵蝕事件在新近導致喪生的山泥傾瀉事件中，影響了該削土坡和山坡。

包括在天然滑坡研究第一和第二階段內的天然滑坡登記圖。 土力工程處，香港，特別項目報告 SPR5/97。

A.5 参考書目

賓尼(一九九七) SIRST Field Sheet for Feature No.7SW-B/C116. 一九九七年三月十三日。

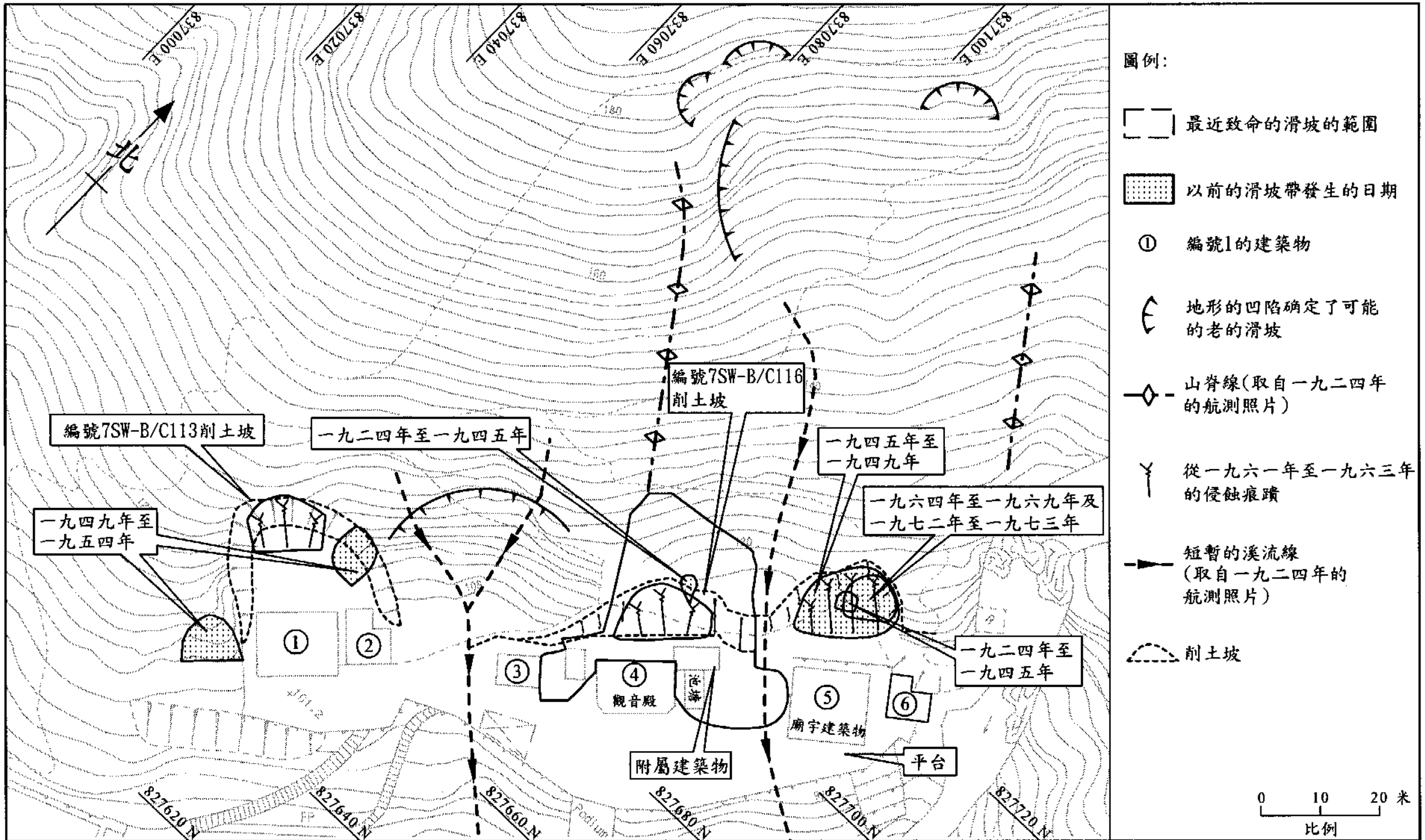
Crown Lands and Survey Office (一九六六) Sheet No. C-145-NE-D，1996 Revision. Crown Land and Survey Office, Hong Kong。

Evans, N.C., Huang, S.W. & King, J.P. (一九九七) The Natural Terrain Landslide Study Phases I and II. 香港土力工程處，119 頁。 特別項目報告編號 SPR5/97 (未出版)。

土力工程處(一九九六) PHASE II SIFT Study. 圖則報告，比例 1:1000 圖則編號 7SW-10D. 一九九六年六月。

Ordnance Survey Office (一九零四年) Topographic Map of 1904，Sha Tin，Sheet No. 12. Ordnance Survey Office，Southampton，United Kingdon 一九零四年。

圖



註: 底圖是根據一九九二年十月編號7-SW-10D的測量圖(比例1:1 000)及利用攝影測量分析一九六三年的航測照片而繪成.

圖A1 - 位置圖