INVESTIGATION OF SOME SELECTED LANDSLIDE INCIDENTS IN 1997 (VOLUME 5)

GEO REPORT No. 91

Halcrow Asia Partnership Ltd.

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG

SPECIAL ADMINISTRATIVE REGION

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PREFACE

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

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

R.K.S. Chan
Head, Geotechnical Engineering Office
June 1999

EXPLANATORY NOTE

This GEO Report consists of four Landslide Study Reports on the investigation of selected slope failures that occurred in 1997. The investigations were carried out by Halcrow Asia Partnership Ltd (HAP) for the Geotechnical Engineering Office as part of the 1997 Landslip Investigation Consultancy.

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

(a) establishment of an improved slope assessment methodology,

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

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

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

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

SECTION 1: DETAILED STUDY OF THE LANDSLIDE AT HONG TSUEN ROAD, SAI KUNG ON 3 JULY 1997

Halcrow Asia Partnership Ltd

This report was originally produced in December 1998
as GEO Landslide Study Report No. LSR 21/98

FOREWORD

This report presents the findings of a detailed study of a landslide (GEO Incident No. ME 97/7/6) that occurred in the early hours of 3 July 1997 on a soil cut slope adjacent to Hong Tsuen Road, Sai Kung. Landslide debris blocked a two-lane highway. No fatalities or injuries were reported.

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

The report was prepared as part of the 1997 Landslip Investigation Consultancy (LIC), for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 68/96. This is one of a series of reports produced during the consultancy by Halcrow Asia Partnership Ltd (HAP). The report was written by Dr Mark Swales and reviewed by Dr S Hencher and Dr R Moore. The assistance of the GEO in the preparation of the report is gratefully acknowledged.

G. Daughton
Project Director/Halcrow Asia Partnership Ltd.

CONTENTS

1. INTRODUCTION

In the early hours of 3 July 1997, a landslide (GEO Incident No. ME 97/7/6) occurred on a cut slope (No. 8SW-C/C3) adjacent to Hong Tsuen Road, Sai Kung (Figure 1). The landslide blocked a two-lane highway, but no fatalities or injuries were reported.

Following the landslide, Halcrow Asia Partnership Ltd (the 1997 Landslip Investigation Consultants) carried out a detailed study of the failure for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 68/96. This is one of a series of reports produced during the consultancy by Halcrow Asia Partnership Ltd (HAP).

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

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

(a) a review of relevant documents relating to the history of the site,

(b) analysis of rainfall records,

(c) interviews with witnesses,

(d) detailed observations and measurements of the landslide site, and

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

2. THE SITE

2.1 Site Description

The landslide occurred on a registered soil cut slope on the southeast side of Hong Tsuen Road (Figure 1 and Plate 1). The cut slope is 185 m long, 17 m high in its central part and consists of three batters with two 1.5 m- to 2 m-wide berms. The lower, middle and upper batters are inclined at about 42°, 35° and 24°, respectively. Above its crest, a narrow strip of natural terrain separates the cut slope from a platform occupied by the Sai Kung Outdoor Recreation Centre. The cut slope and surrounding natural terrain are vegetated by shrubs, trees, bamboo, grass and ferns.

The two berms are drained by 250 mm diameter U-channels that discharge into a culvert at the northeastern end of the slope (Figure 2). In the centre of the cut slope, there is a catchpit on each berm. These are connected by a stepped channel leading to a covered drain at the toe of the slope. There is also a U-channel in the natural terrain above the crest of the cut slope.

During inspections by HAP on 3 July 1997 and 23 March 1998, the U-channels in the vicinity of the 1997 landslide were partly blocked with vegetation debris (Plates 2 and 3).

According to the SIMAR (Systematic Identification of Maintenance Responsibility of Slopes in the Territory) consultancy, the Highways Department (HyD) is responsible for the maintenance of the slope.

2.2 Site History

2.2.1 General

The history of the site was determined from a review of aerial photographs and available documentary information. Key events in the history of the site are described below and the pertinent observations are summarised in Figure 3.

2.2.2 History of Development

In 1975, the lower part of the northwest-facing hillside was undeveloped and overlooked a shallow, vegetated, northeast-trending valley leading to Sai Kung Hoi. An army base was present on the upper part of the slope at the current site of the Sai Kung Outdoor Recreation Centre.

Construction of the slope and installation of the drainage system on the cut slope was completed in 1978 (WSA, 1981). Corestones and exposures of rock were evident in the centre and at the southwestern end of the cut slope on the 1978 aerial photographs.

By 1979, Hong Tsuen Road had been completed and construction of a large building was nearing completion above the crest of the cut slope, within the army base. The stepped-channel in the centre of the cut slope had also been installed. Redevelopment of the army base commenced in 1988.

No further significant development is apparent at the site after 1988 and the density of vegetation covering the cut slope increased during this period. By 1990, the entire cut slope was covered by vegetation (Figure 2).

2.2.3 Previous Landslides

An area of surface erosion of about 50 m^2 in the centre of the cut slope is visible on aerial photographs taken in 1982 (Figure 3). The location of this erosion corresponds approximately to the toe on the southwestern side of the main scarp of the recent failure. Observations from aerial photographs also show that further erosion occurred at the same place on the slope in 1989. A smaller area of erosion on the upper batter is apparent in the 1984 photographs.

According to GEO's landslide database, no previous landslide incidents have been reported on the cut slope.

2.3 Previous Studies and Assessments

The cut slope was inspected on 6 April 1978, whilst under construction, by consultants engaged by the Government to prepare the 1977/78 Catalogue of Slopes (Binnie & Partners, 1978). The feature was registered as Slope No. 8SW-C/C3. The inspection noted no signs of seepage or signs of distress.

The cut slope was assessed by Wilbur Smith & Associates (WSA) in the late 1970's (WSA, 1979) as part of their design responsibilities for the Sai Kung Development Phase 3. In early 1979, WSA made a preliminary geotechnical submission, which included cut slope No. 8SW-C/C3. In their calculations, WSA assumed that groundwater would not influence slope stability but Binnie & Partners (B & P) in their review stated that "there is evidence to indicate that groundwater cannot be ignored" (B & P, 1979). Following a ground investigation, WSA (1980) made another submission in December 1980. Details of the stability analyses of Section Q_1-Q_1 taken through cut Slope No. 8SW-C/C3 in the area of the 1997 landslide are given in Appendix A. WSA's Report states that "for the purpose of the stability analysis, 1 metre and 3 metres rise in water levels are assumed for a 1 in 10 year and 1 in 1000 year storm respectively. These water levels have been superimposed on the maximum water level recorded or on the bedrock surface" (WSA, 1980).

On 14 January 1981, WSA, B & P and GCO carried out a joint site inspection of the slopes and a retaining wall on the Sai Kung Development. According to the caption to one of the site record photographs, "there is a perennial discharge into cracked surface water drains" (Plate 4). The precise location of this discharge is not known but it is believed to be close to the 1997 landslide site.

In their comments on the WSA submission, B & P (1981a) stated that "there may be colluvium exposed in the top of the cutting. If a perched water table were to form in this material the factor of safety could fall dangerously." WSA (1981) responded by stating "The possibility of perched water tables occuring on this slope was based on the possibility of the upper layer of material being colluvium. Further inspection does not reveal this to be the case. The upper layer is more highly weathered and has a greater proportion of clay than the lower layer. It would therefore be expected to be less permeable and this is confirmed by the permeability results from the consolidation stage of the triaxial test.... Thus, a perched water level is not possible at the interface of the two layers and additional investigation is considered unnecessary." In reply to this, B & P (1981b) stated "The laboratory test results indicate that the permeability increases with depth through the volcanics which would indicate that a perched water table is unlikely. However, no measurements were made in the top 3 m and the highest permeability was recorded between 3 and 6 m at the top of this layer so it is still possible that water could perch at about 3 m below original ground level." They went on to say "The effect of this perched table on the overall stability is small and local slips involving only the top of the slope have factors of safety close to 1.2 with the shear strength parameters $c' = 0$, $\phi' = 37°$. A very small cohesion intercept of 1 to 2 kPa which could be justified from the test results is all that is necessary to maintain a satisfactory factor of safety." In their final letter on the subject of the perched water table, B & P (1982) stated "A perched water table is therefore still possible and we have considered this in our analyses. A satisfactory factor of safety was obtained allowing a small cohesion intercept of 1 to 2 kPa.... Further site investigation, which is probably not justified in the circumstances, would be required to prove or disprove a perched water table."

Monitoring of piezometers continued and in June 1982 WSA "re-examined the stability of Section Q1-Q1" through Slope No. 8SW-C/C3, following a recorded rise of about 4 m in a piezometer installed in a borehole on Slope No. 8SW-C/C86, about 250 m to the southeast of the 1997 landslide site (Figure 1). They concluded, "there is some doubt as to the stability of the slope". An existing piezometer in borehole S2 had been damaged and WSA recommended to the Junk Bay Development Office (JBDO) that "it would be advisable to install a new piezometer". In September 1982 a piezometer was installed at a depth of 11.2 m (one metre above rockhead) in borehole S8 on the first berm (Figures 3 and 4).

In October 1982, GCO noted the need for WSA to submit piezometer readings and their interpretation of groundwater conditions (GCO, 1982). Piezometer readings covering the periods up to October 1982 and February 1983 were submitted to the GCO in November 1982 and February 1983, respectively. These show that the water level in the new piezometer varied between about 8.3 m and 9.6 m below ground level. In January 1983, JBDO requested GCO to "confirm that you now have sufficient calculations and information to establish the water regimes in the various slopes and that further monitoring ... is not required". They also stated that "if you cannot confirm that further monitoring is not required will you please supply in details what further work is required and also your reasons for requiring this work" (JBDO, 1983). The GCO responded that there was "no further requirement on piezometer monitoring" (GCO, 1983).

In 1992, the GEO initiated a consultancy agreement entitled "Systematic Inspection of Features in the Territory" (SIFT), which, inter alia, aims to identify features not registered in the 1977/78 Catalogue of Slopes and to update information on registered slopes based on studies of aerial photographs and limited site inspections. In August 1996, SIFT assigned the cut slope (No. 8SW-C/C3) to Class "C1", i.e. a slope "formed or substantially modified before 30.6.78".

In July 1994, the GEO initiated a consultancy agreement entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST) to update the 1977/78 Catalogue of Slopes and to prepare the New Catalogue of Slopes. The SIRST consultants inspected the cut slope on 11 November 1994. No signs of seepage or distress were recorded and no emergency action was considered necessary.

In July 1995, a Stage 1 Study was carried out on the slope by GEO (1995) under SIRST. It was considered probable that works would "be required to bring the feature up to current engineering standards".

In January 1996, the slope was inspected by consultants appointed by HyD to undertake the "Roadside Slope Inventory and Inspections" of their slopes (HyD, 1996). The Engineer Inspection Record states that "routine maintenance (has not been) carried out satisfactorily because surface drainage systems (are) not clear (and) because drainage channels (are) cracked/damaged". The vegetated surface of the slope was found to be in good condition and no seepage was reported. The Inspection Record made recommendations to "clear drainage channels in vicinity of feature, repair cracked/damaged channels (and) improve the protection against infiltration above the crest". HAP has found no information to confirm that these works have subsequently been carried out. The Inspection Record further noted that "geotechnical conditions (are) not known (and) stability assessment (is) considered to be required".

2.4 Subsurface Conditions

According to Map Sheet No. 8 produced by the Hong Kong Geological Survey (GCO, 1989), the site is underlain by coarse-grained, ash crystal tuff of the Tai Mo Shan Formation. This is characterised by a "lack of intraformational boundaries or segregations" (Strange et al, 1990) and typically weathers to "an orange to reddish brown saprolitic soil with large corestones" (Addison, 1986).

The structural geology of Sai Kung is dominated by northeast- and northwest-striking faults. In the vicinity of the site, a northeasterly-striking minor fault is present in the valley floor beyond the cut slope.

There have been four ground investigations in the vicinity of the slope (Gammon, 1979; WSA, 1980; Gammon, 1981 and Gammon, 1987). Four boreholes and one trial pit are of relevance (Figure 3). HAP has used information from these investigations and site observations to prepare a geological section (Figure 4).

In the area of the landslide, borehole information indicates that the slope is composed of PW 0/30 volcanic rock consisting of completely decomposed volcanics (CDV) with highly and moderately decomposed volcanics (HDV/MDV) that are probably corestones. This overlies less weathered rock, predominantly consisting of MDV. The PW 0/30 zone can be differentiated into two layers. The upper layer, interpreted to be up to about 7 m thick, comprises soft to firm sandy silt and clay (i.e. predominantly silt and clay in approximately equal proportions - subsequently referred to as clay-rich CDV), with SPT values of 3 or less. At depth, this becomes stiff to very stiff with SPT values in excess of 20. The lower layer consists of up to about 6 m of medium to very dense silty sand (CDV).

The results of Atterberg limit tests (WSA, 1980) on samples from the upper layer are plotted on the standard plasticity chart given in Geoguide 3 (GCO, 1988) in Figure 5. The failure occurred entirely within this upper layer of clay-rich CDV.

An extensive area of rock, exposed at the southwestern end of the slope, indicates that the rockhead probably rises in elevation towards the southwest. However, small rock exposures in the lower and middle batters in the central part of the cut slope are probably corestones as they are above the rockhead, based on information derived from boreholes.

The closest piezometers to the 1997 landslide were installed in boreholes S2 and S8, which were first monitored on 30 April 1981 and 22 October 1982 respectively (Figure 3). The response zone in piezometer S2 (23.7 mPD to 25.2 mPD) was just below rockhead (25.3 mPD). Piezometer S8 was installed at a depth of 11.2 m, one metre above rockhead (16.9 mPD) and the response zone was not recorded. The maximum water levels recorded in these piezometers were 30.8 mPD (S2) and 20.7 mPD (S8), respectively (Figure 4). Piezometer S2 was monitored between 30 April 1981 and 11 August 1981. Piezometer S8, however, was monitored mainly in the dry season between 22 October 1982 and 9 February 1983.

3. THE LANDSLIDE

3.1 Time of the Failure

According to the Senior Officer at the fire station located immediately across the road from the cut slope, the landslide occurred at 00:23 hours on 3 July 1997.

3.2 Description of the Landslide

A plan of the landslide is shown in Figure 6 and Plate 5 shows a general view of the landslide on the day of the incident.

The landslide occurred in the centre of the middle batter and locally undercut the second berm and its associated U-channel. The toe of the rupture surface, which was exposed following the clearance of landslide debris (Plate 5), was observed to daylight at the first berm. The rupture surface of the landslide was about 25 m wide, 12 m long and was up to 2.4 m below the original surface of the cut slope. The angle of the rupture surface, measured relative to the horizontal, increased from 18° near the base of the landslide to near-vertical towards the crest. The angle between the toe of the landslide rupture surface and the crest, which extended to the back of the second berm, was about 28° to the horizontal.

The volume of landslide debris was estimated to be about 250 m^3, about 75 % of which was deposited on the road. The remaining debris came to rest on the first berm and as a thin cover on the lower batter. At the toe of the slope, the wet debris spread across both lanes of the highway and pedestrian footpaths to a depth of between about 2 m and 0.5 m. Silty and sandy material was washed down the road in a northeasterly direction. The travel angle of the landslide debris, measured from the crest of the main scarp to the distal end of the debris, was about 25°. This indicates greater mobility of debris than is typical for rain-induced landslides of comparable volume in Hong Kong (Wong & Ho, 1996).

The weathering grade of material exposed in the main scarp decreased from northeast to southwest (Figure 6 and Plate 5). In the northeastern part of the main scarp residual soil, comprising mostly homogeneous, soft to firm, moist to wet, orange red, sandy silt to sandy clay, was exposed. This material is also locally exposed to the northeast of the landslide (Plate 6). In the centre of the main scarp, there was CDV of mass weathering grade PW 0/30, comprising completely decomposed, coarse ash crystal tuff described as fissured to intact, firm to stiff, moist to wet, orange yellow, slightly gravelly, sandy silt to sandy clay. Similar material was exposed in a shallow trial trench excavated along the second batter to the southwest of the failure (Figure 6 and Plate 7). The southwestern part of the main scarp revealed a large corestone of pale orange yellow, highly and moderately decomposed, coarse ash crystal tuff, with medium- to closely-spaced discontinuities. This tuff was intersected by both near vertical and sub-horizontal, but not adverse, planar discontinuities. Some discontinuities had apertures up to about 30 mm wide and were stained with iron and manganese oxides (Plates 8 and 9). The width of the discontinuity may have been a result of the landslide.

During an inspection at about 16:00 hours on 3 July 1997 HAP observed water issuing approximately 1.5 m below the ground surface from the large corestone in the southwestern side of the main scarp (Plate 9). When the site was inspected 6 days later this flow of water had ceased. HAP also observed that the U-channels in the vicinity of the main scarp were partially blocked with vegetation (Figure 6).

Following the landslide, HyD carried out urgent repair works comprising removal of debris, shotcreting of the affected area of the slope and re-instatement of the surface drainage system (Plate 10).

4. RAINFALL

Raingauge No. N15, located at Sung Tsun Primary School, Yau Ma Po, some 750 m to the northeast of the landslide, is the nearest GEO automatic raingauge. This raingauge began operation in June 1990. Between 1984 and 1990, the raingauge had been located at the Management Centre of Pak Kong Country Park, about 1 km to the northwest of the landslide.

The daily rainfall recorded in June and early July 1997, together with the hourly rainfall from 29 June to 4 July 1997, is shown in Figure 7. The daily record shows that in the 31 days and 15 days prior to midnight on 2 July 1997 (i.e. shortly before the landslide at 00:23 hours on 3 July 1997), the rainfall totals were 1 193 mm and 683 mm respectively. The rainfall totals in the 24-hour and 12-hour periods before midnight on 2 July were 341 mm and 225 mm respectively.

The 5-minute rainfall record (Figure 8) shows that heavy pulses of intense rainfall occurred on the day before the landslide, with the heaviest being around 06:00 hours on 2 July 1997. The last pulse of heavy rainfall reached peak intensity at 23:35 hours on 2 July 1997, about 50 minutes before the landslide occurred.

Isohyets of rainfall over Hong Kong for the period between 00:00 hours on 2 July to 00:25 hours on 3 July are given in Figure 9.

The estimated return periods of the maximum rolling rainfall for selected durations prior to the landslide, based on historical rainfall data at the Hong Kong Observatory, are given in Table 1. The rolling rainfall totals for durations up to 15 days ranged from 1 to 9 years and were not particularly unusual. However, the 31-day rolling rainfall was more severe, having an estimated return period of over 40 years.

The daily rainfall on 2 July 1997 was the highest since raingauge No. N15 was installed in 1984 (Figure 10).

5. THEORETICAL STABILITY ANALYSIS

Stability analyses were carried out to assess the possible range of shear strengths and groundwater pressures that may have been operational at the time of failure. The cross-section through the landslide, on which stability analyses were carried out using the rigorous method of Morgenstern and Price (1965), is presented in Figure 11. The pre-failure profile was established from topographical survey plans and the geometry of the failure surface was based on site measurements made by HAP (Figure 4).

The presence of clay-rich material would suggest that the shear strength parameters for the upper layer were probably lower than those assumed in design ($c' = 0$ and $\phi' = 37°$). Accordingly, for the purpose of the analyses, a range of shear strengths for clay-rich CDV of $c' = 0$ to 6 kPa and $\phi' = 24°$ to 34° was considered, together with various perched water tables. The development of high groundwater pressure is consistent with the observation of a groundwater issue from the main scarp on the day of the failure.

The results of the stability analyses, which support the idea that the failure was likely to have been the result of a combination of low shear strength material and high groundwater, are summarised in Figure 12.

6. PROBABLE CAUSES OF FAILURE

The correlation between the rainfall and the time of failure indicates that the landslide was probably triggered by rainfall. A number of contributory factors probably combined to cause the landslide, including :

(a) infiltration and subsurface groundwater flow,

(b) the presence of clay-rich CDV in the upper part of the cut slope, and

(c) the development of transient positive water pressure within the clay-rich CDV in the upper part of the cut slope.

It is postulated that the upper layer of CDV was probably heterogeneous with bands of low permeability clay-rich material, which could lead to downslope seepage flow (i.e. non-vertical) and hence development of seepage pressure, which is effectively equivalent to perching within the layer. The most likely sources of water were by direct infiltration and groundwater ingress through more permeable layers and zones within the heterogeneous upper CDV layer.

7. DISCUSSION

The landslide occurred at a soil cut slope that had previously been subjected to a detailed stability assessment based on site-specific ground investigation and laboratory testing.

The landslide is of technical interest in that it occurred on a slope where the gradient over the failed section was only about 28 degrees. The upper layer of CDV in which failure took place is also of interest in that it has a high fines (i.e. silt and clay) content. The composition of this material probably resulted in a lower shear strength than that adopted in the design of the cut slope (viz. $c' = 0$ kPa, $\phi' = 37°$).

Infiltration and the presence of low permeability bands within the upper layer of CDV probably led to the development of adversely high groundwater conditions. It is noteworthy that the possibility of perched water within the slope was recognised by the consultant checking the assessment of the slope in the early 1980s on behalf of GCO. The theoretical stability analyses support the proposition that the failure on such moderately sloping ground probably involved relatively high groundwater conditions coupled with low shear strength material. However, without ground investigation, groundwater monitoring and laboratory testing it is not possible to identify the relative contributions of these to the failure.

There is evidence of inadequate slope maintenance in that the U-channel near the crest of the landslide was observed to be partially blocked after the failure. However, the contribution of this to the landslide was unlikely to be significant given the scale and mode of failure.

Past erosion at the portion of the slope that failed in 1997 was observed in aerial photographs but there is no documentary information on the incidents. It is not possible to establish whether these were maintenance-related problems or if they were indications of slope instability.

8. CONCLUSIONS

It is concluded that the landslide was triggered by rainfall. The failure occurred at a slope that had previously been subjected to a detailed stability assessment based on site-specific ground investigation and laboratory testing.

The landslide is of technical interest in that a large-scale failure occurred at a moderately-inclined soil cut slope and that the landslide debris was relatively mobile.

The failure involved CDV with a relatively high fines content. The low shear strength of this material and the development of high groundwater pressure within the upper clay-rich layer were probably significant contributory factors to the failure.

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Table 1 – Maximum Rolling Rainfall at GEO Raingauge No. N15 for Selected Durations Preceding the 3 July 1997 Landslide and The Corresponding Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	7.5	06:00 hours on 2 July 1997	1
15 minutes	19.5	06:05 hours on 2 July 1997	1
1 hour	52.5	06:45 hours on 2 July 1997	1
2 hours	81	06:00 hours on 2 July 1997	1
4 hours	123.5	07:00 hours on 2 July 1997	2
12 hours	225	06:00 hours on 2 July 1997	4
24 hours	341	00:00 hours on 3 July 1997	7
2 days	429	00:00 hours on 3 July 1997	9
4 days	442	00:00 hours on 3 July 1997	5
7 days	498	00:00 hours on 3 July 1997	5
15 days	683	00:00 hours on 3 July 1997	7
31 days	1193	00:00 hours on 3 July 1997	46

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

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

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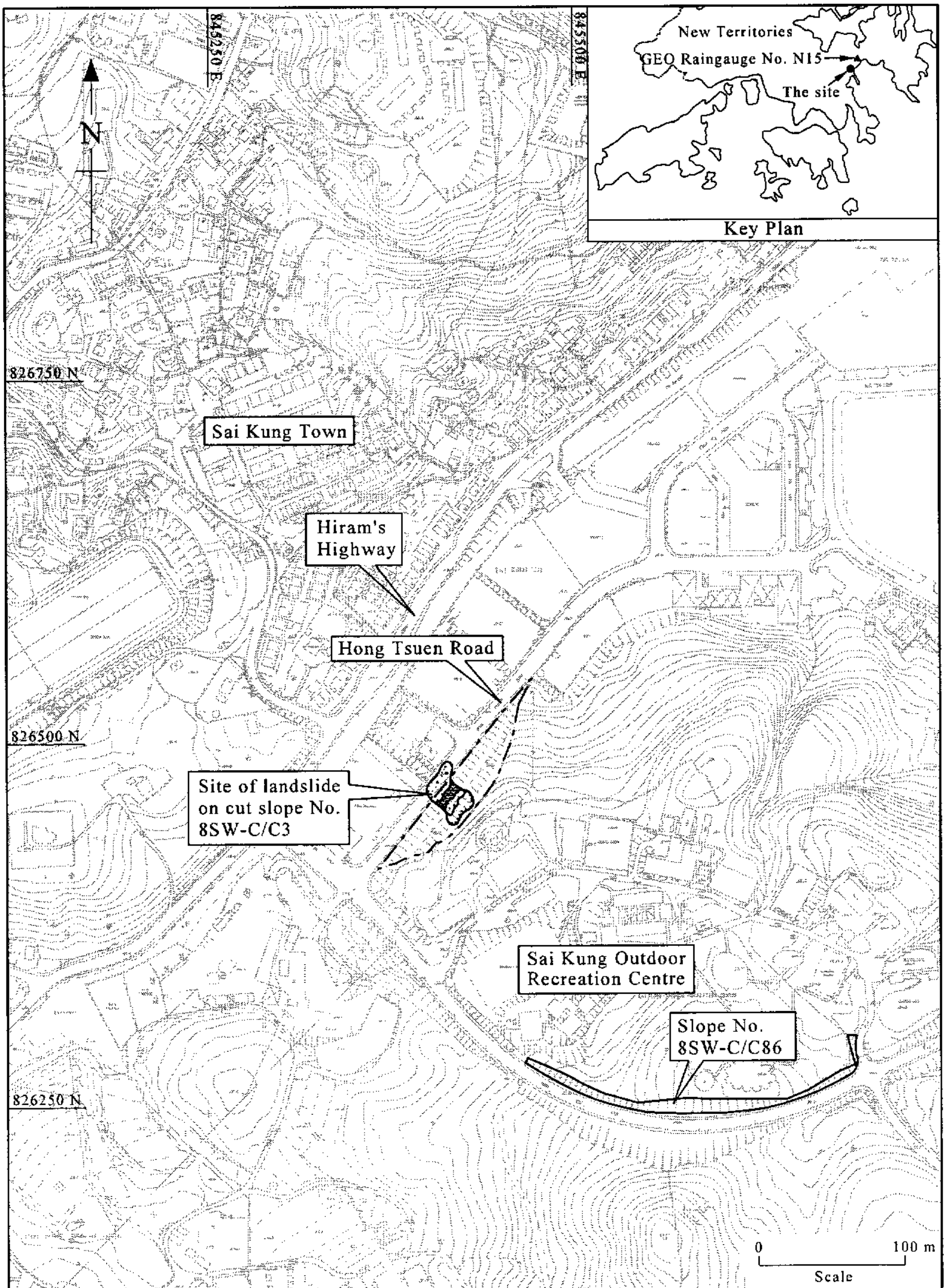


Note : Base map is extracted from Survey Sheet Nos. 8-SE-11C and 8-SW-16A dated October 1992.

Figure 1 - Site Location Plan

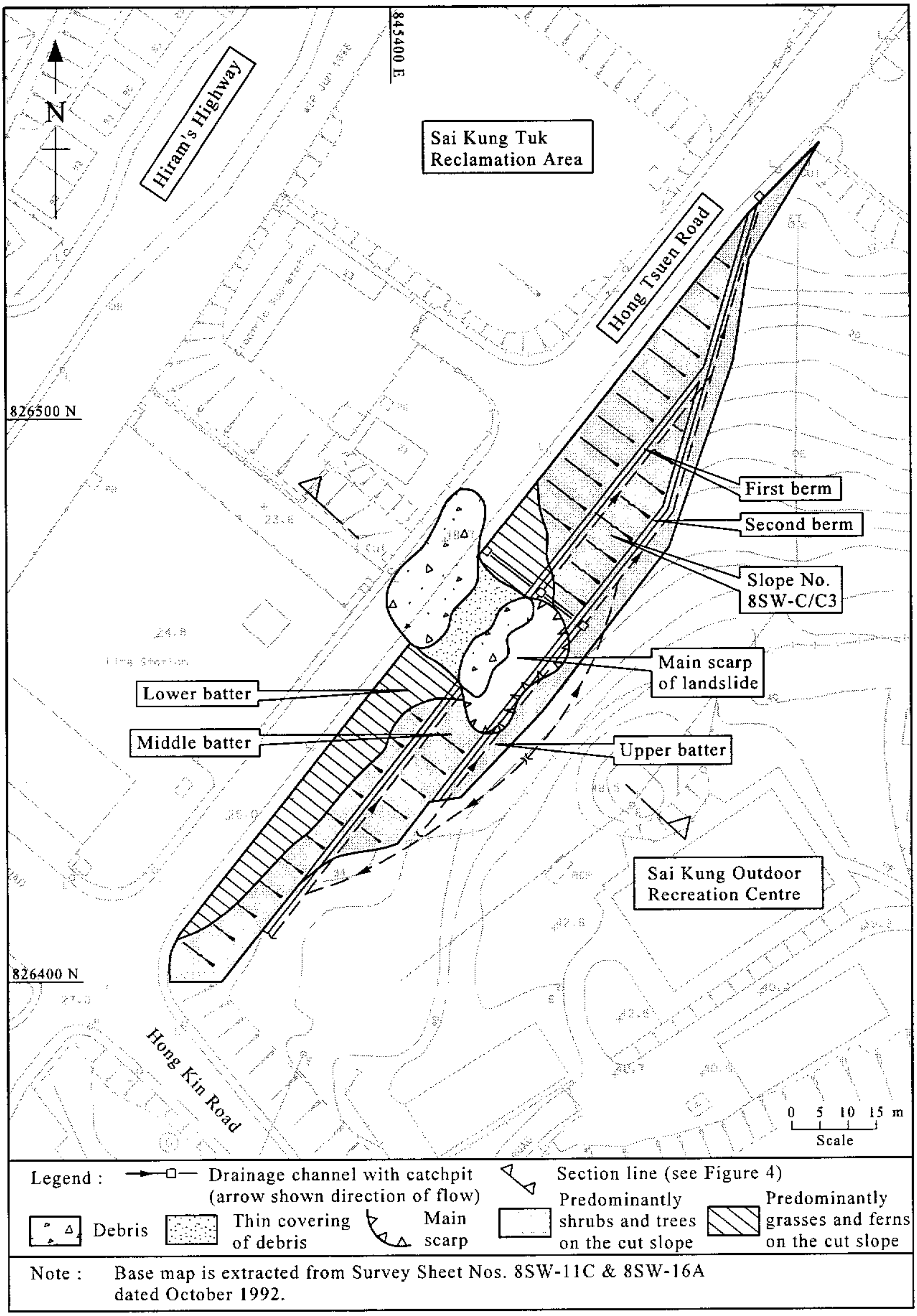
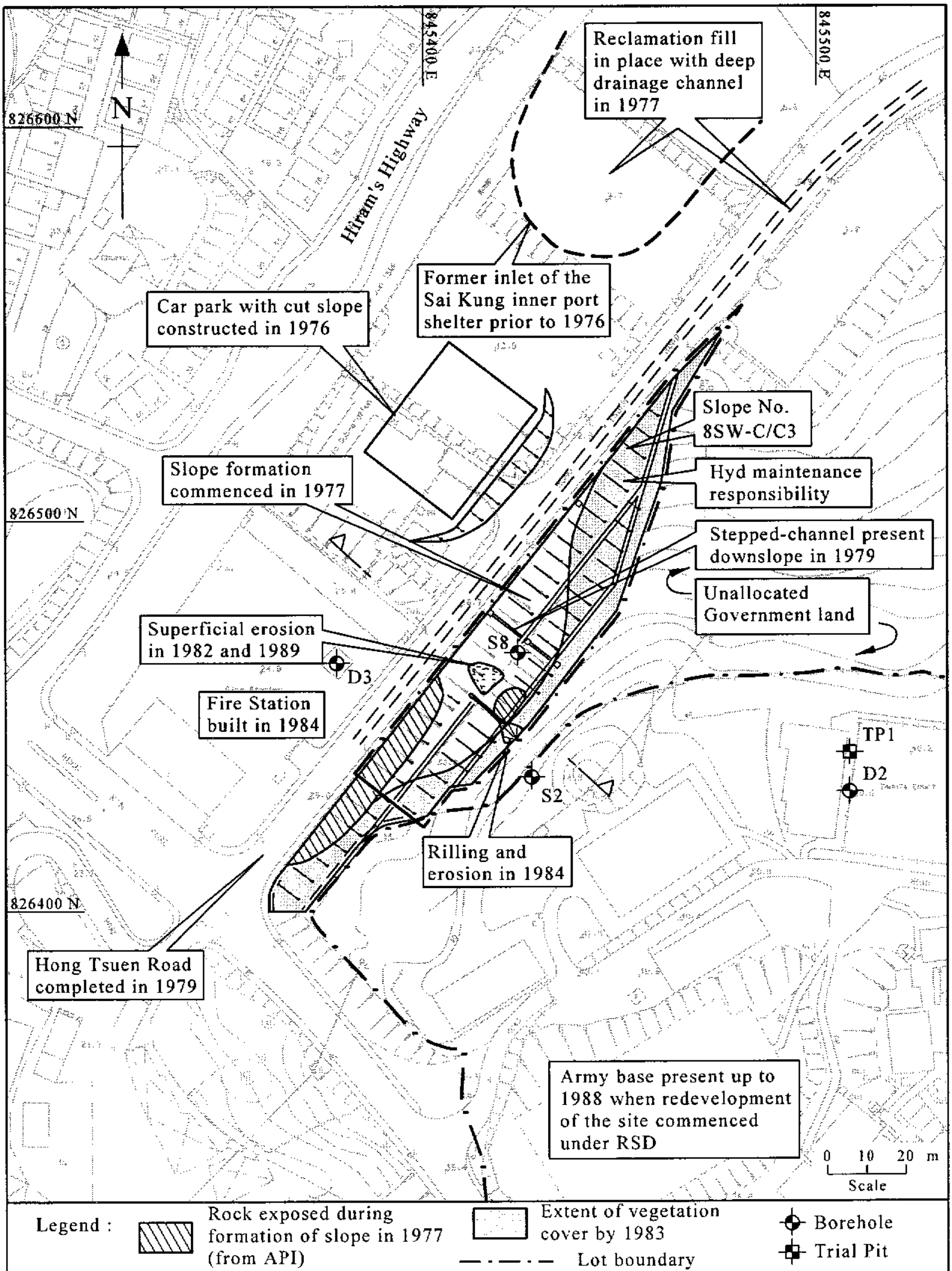
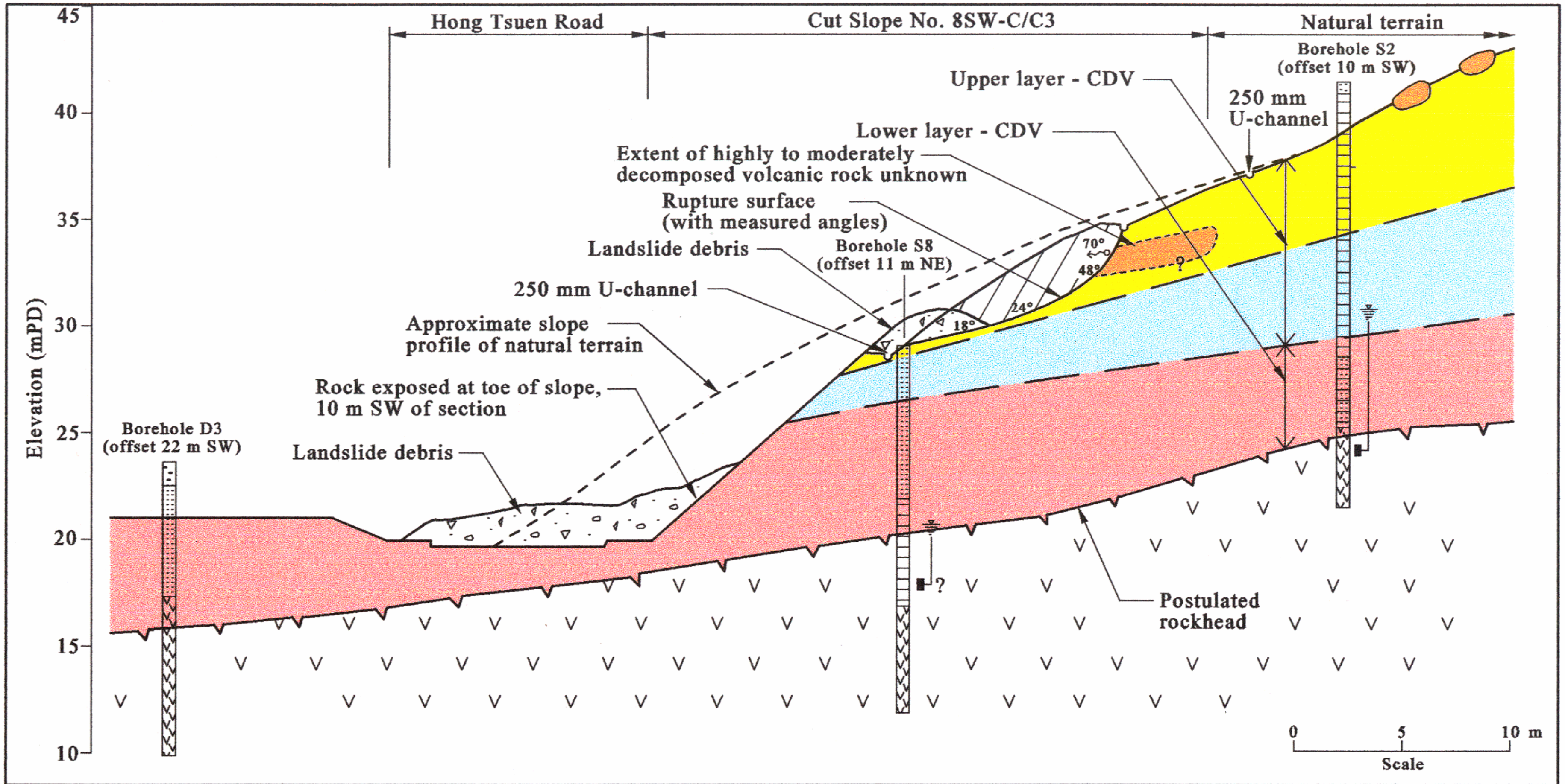


Figure 2 - General Plan of the Site and the Landslide



Notes : (1) Land status is only shown for area on southeast side of Hong Tsuen Road.
(2) Maintenance responsibility according to SIMAR.
(3) Abbreviations : HyD (Highways Department); RSD (Regional Services Department).

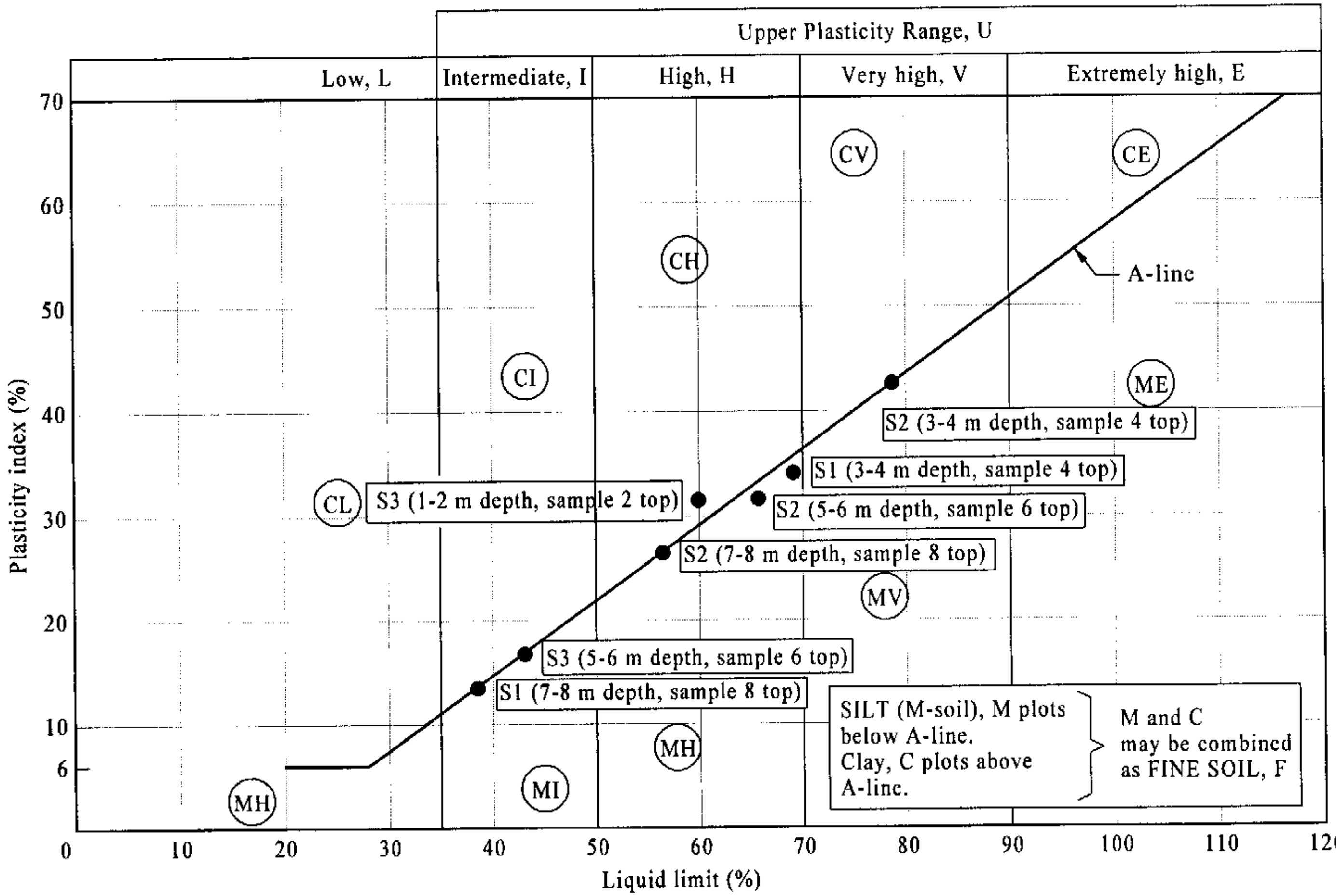
Figure 3 - Land Status, Site History and Previous Ground Investigations



Legend :		Very soft to soft sandy Silt to sandy Clay (CDV) (PW 0/30)		Stiff to very stiff sandy Silt to sandy Clay (CDV) (PW 0/30)		Piezometer tip and maximum water level recorded	Seepage
		Medium to very dense silty Sand (CDV) (PW 0/30)		Highly to moderately decomposed volcanic rock		Moderately to slightly decomposed volcanic rock	

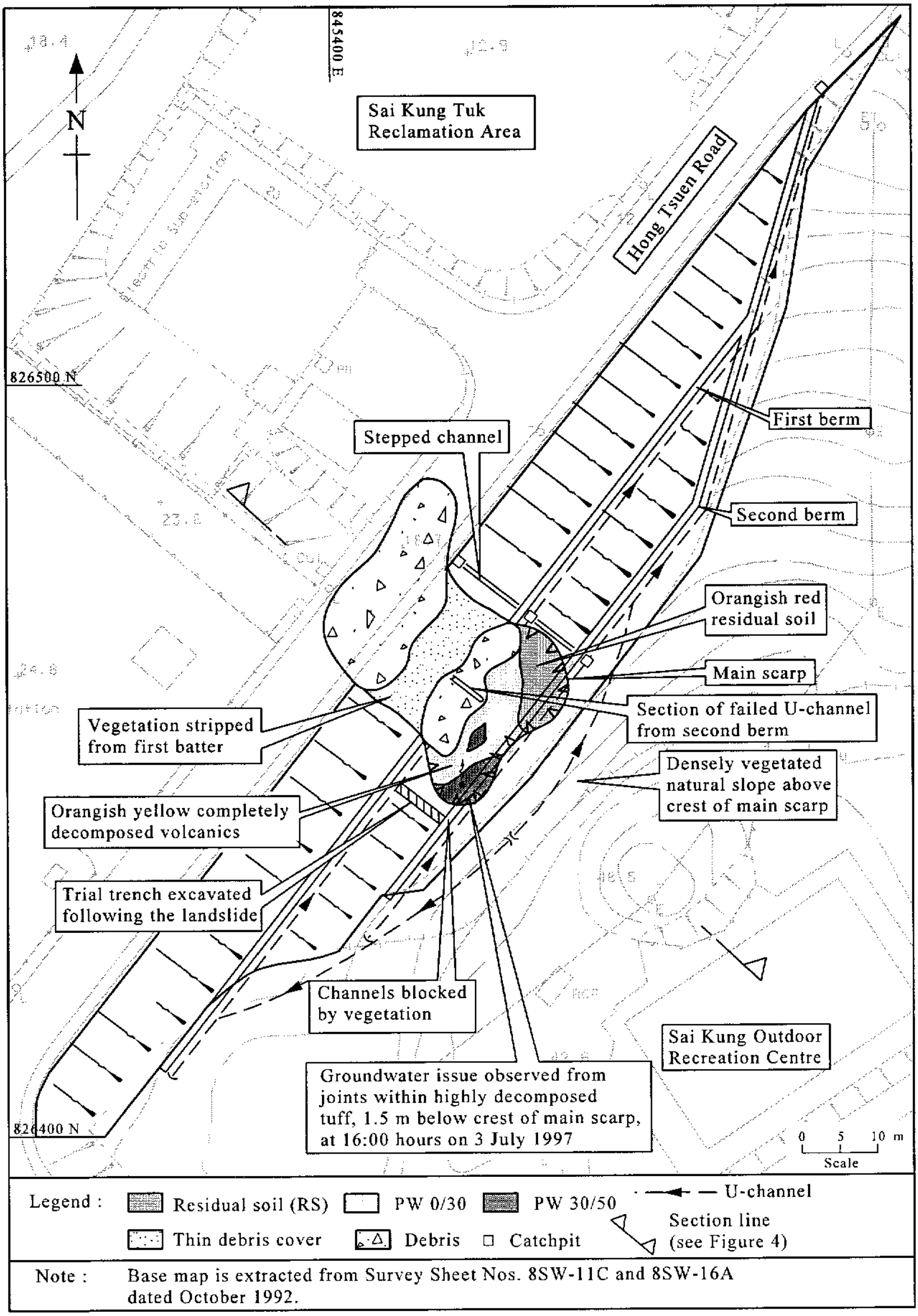
Notes :
(1) Depths of geological boundaries are schematic and do not necessarily correspond precisely with the borehole logs.
(2) Geological descriptions in the legend are based on site mapping and data from borehole logs and laboratory testing.

Figure 4 - Geological Cross-section through the Landslide



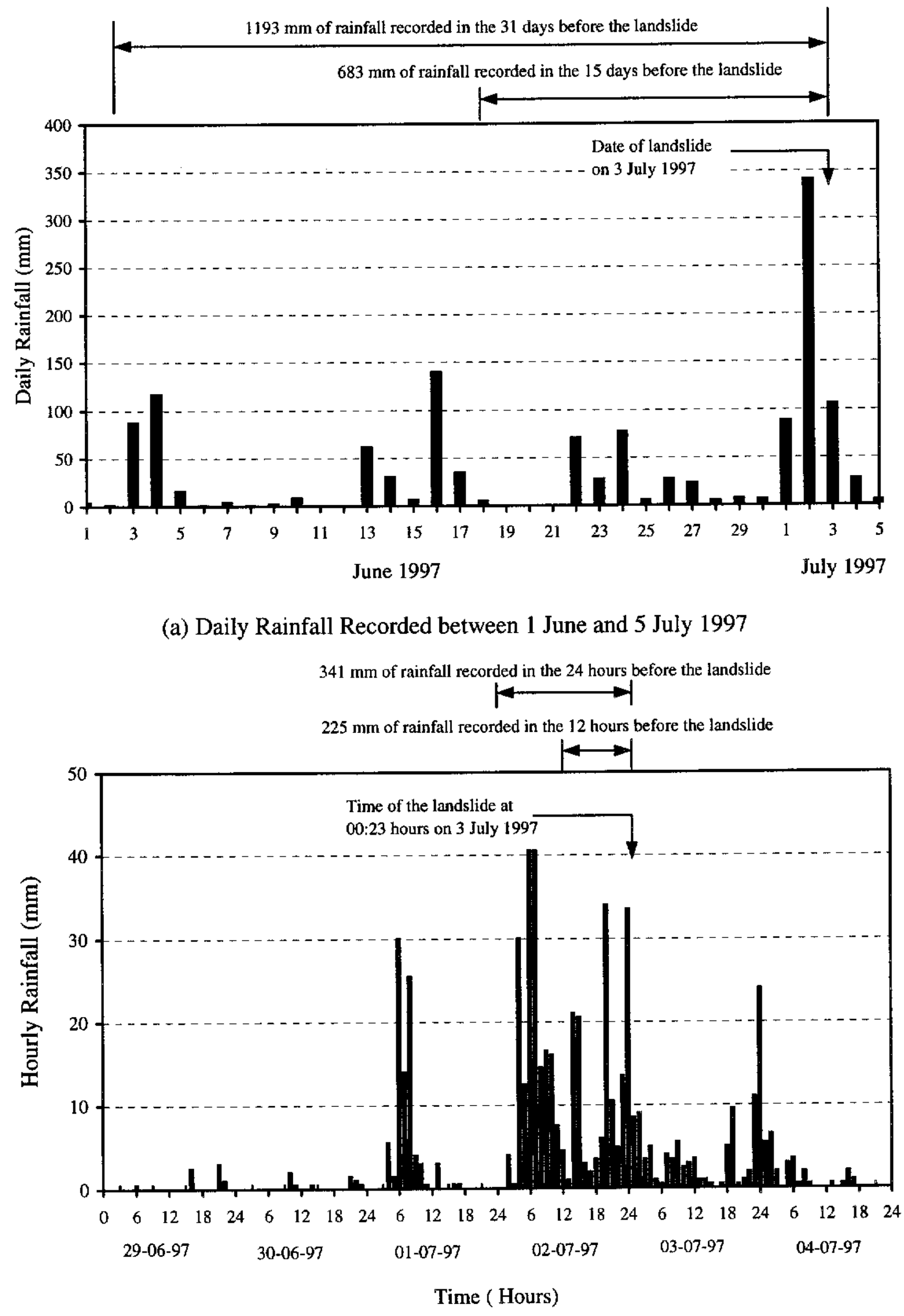
Notes :
(1) The letter O is added to the symbol of any material containing a significant proportion of organic matter e.g. MHO
(2) Plasticity measurements are made on material passing the 425 μm BS sieve.
(3) Results shown are from boreholes S1, S2 and S3 (WSA, 1980). Borehole S2 is located about 5m upslope from the landslide site whilst boreholes S1 and S3 are located 170 m northeast and 250 m southeast of the landslide site respectively.

Figure 5 - Results of Soil Classification Tests in Accordance with the Standard Plasticity Chart of Geoguide 3 (GCO, 1988)



Note : Base map is extracted from Survey Sheet Nos. 8SW-11C and 8SW-16A dated October 1992.

Figure 6 - Site Plan and Details of the Landslide



(a) Daily Rainfall Recorded between 1 June and 5 July 1997

(b) Hourly Rainfall Recorded between 29 June and 4 July 1997

Figure 7 - Rainfall Records at GEO Raingauge No. N15

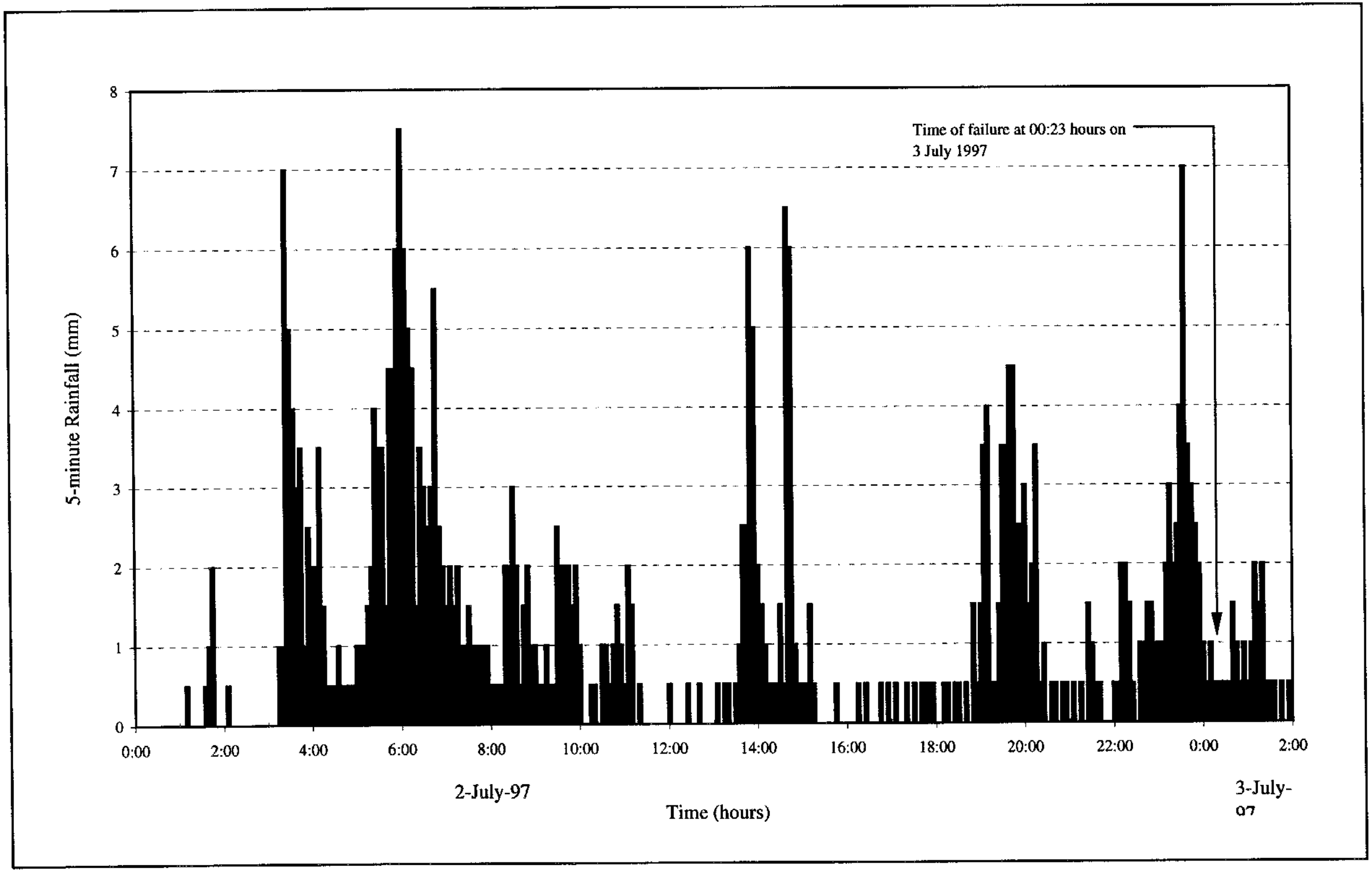
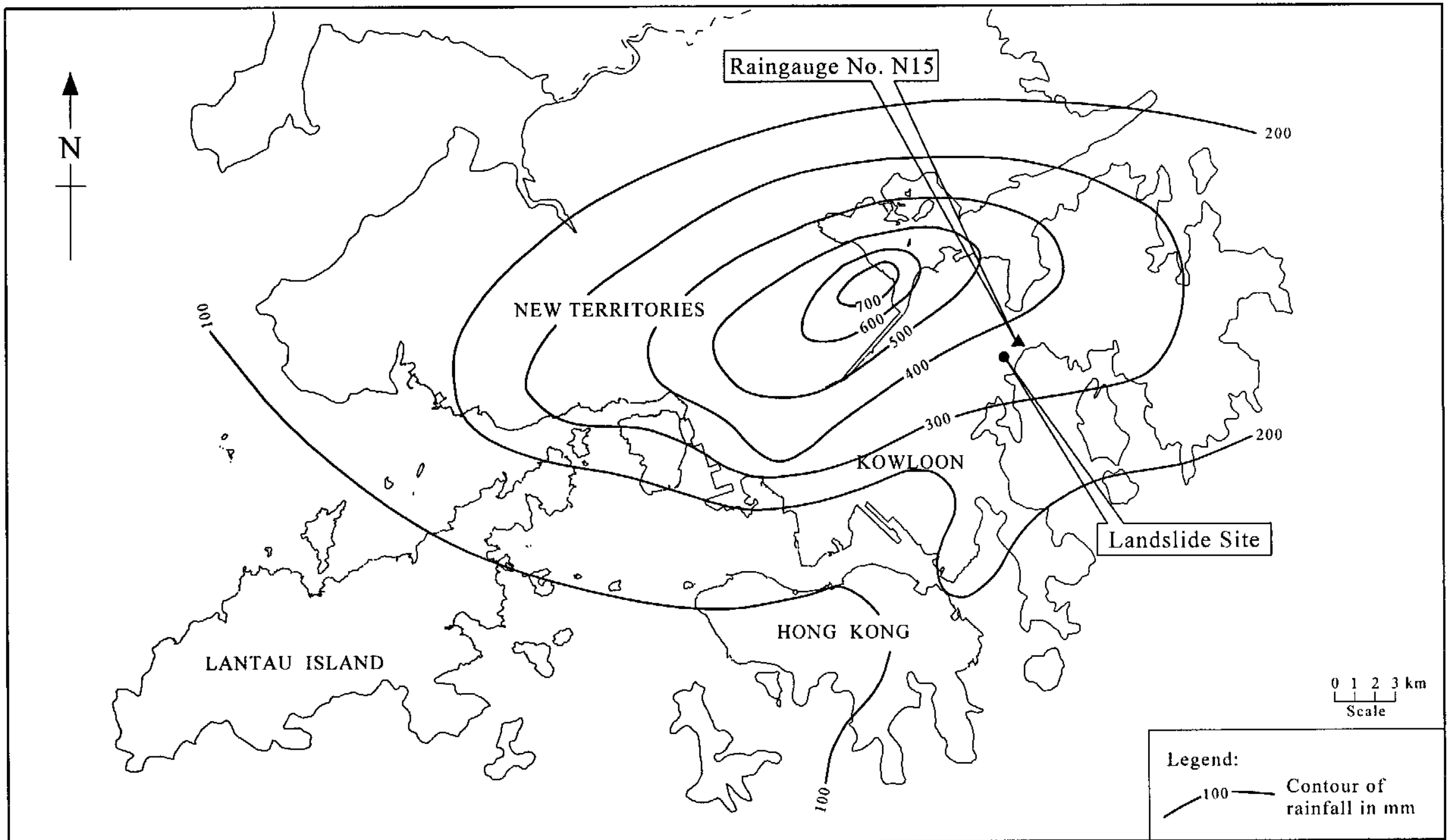


Figure 8 - Rainfall Records at GEO Raingauge No. N15 at 5-minute Intervals



Note : Isohyets prepared by the Hong Kong Observatory.

Figure 9 - Isohyets of Rainfall between 00:00 Hours on 2 July 1997 and 00:25 Hours on 3 July1997

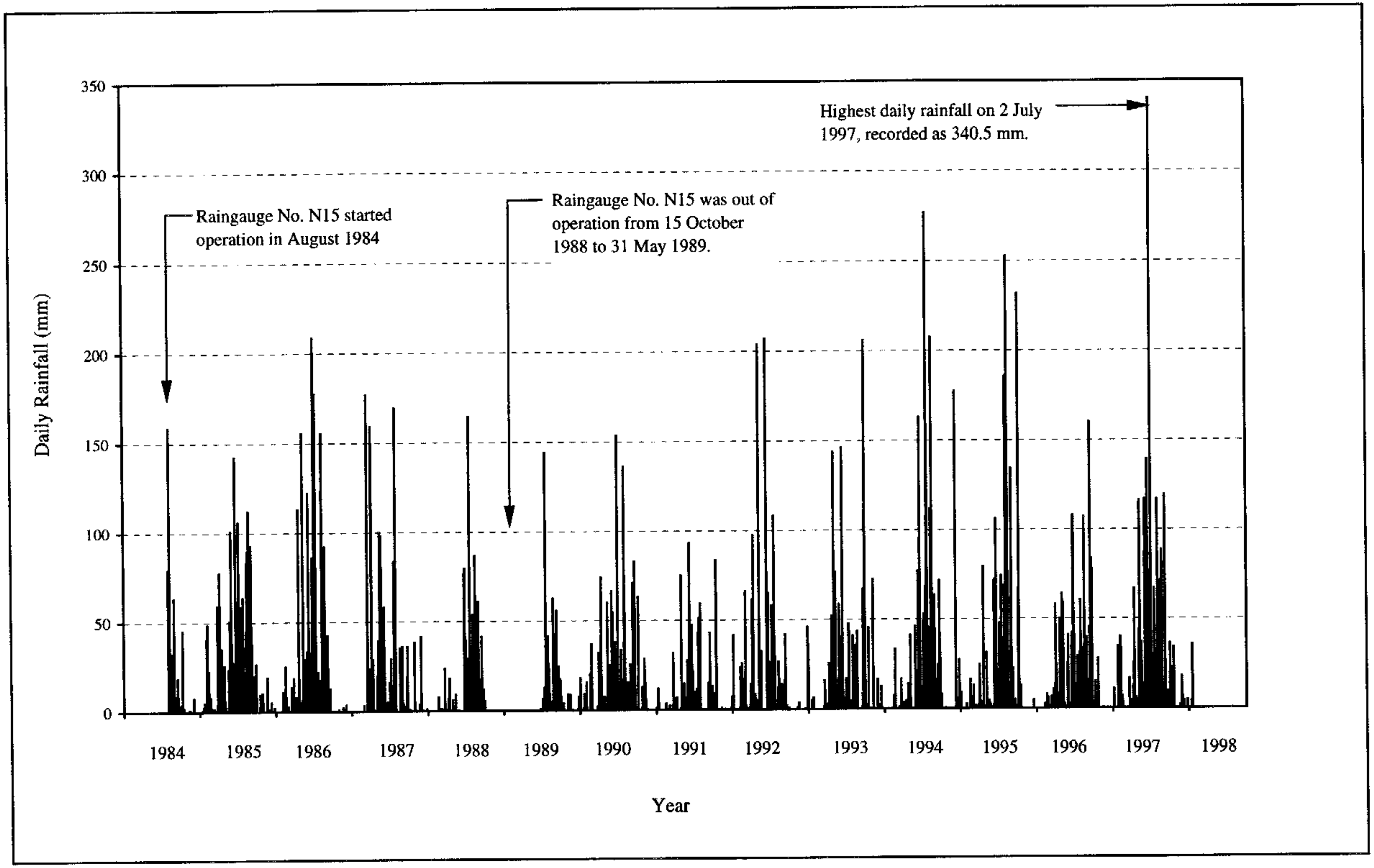
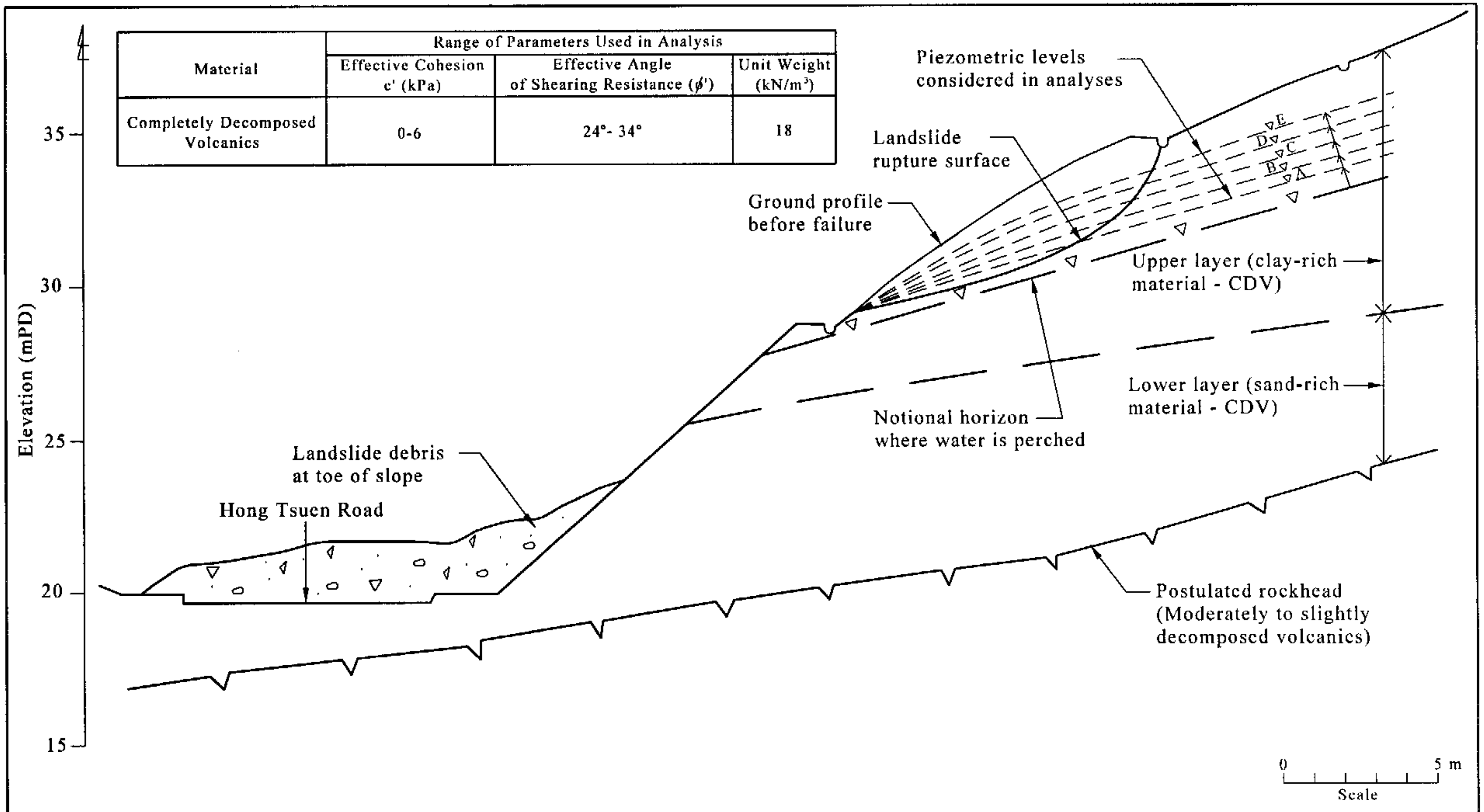


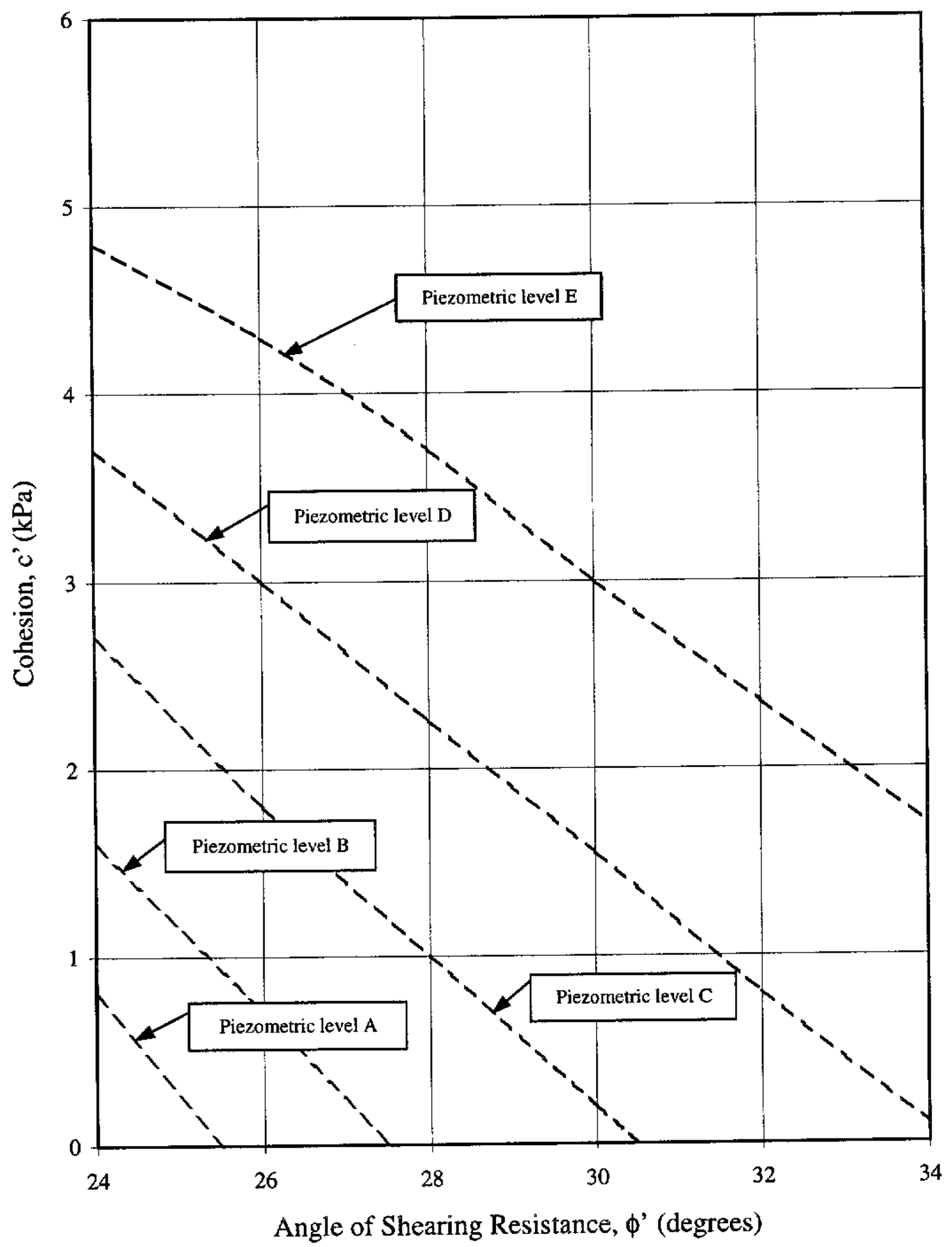
Figure 10 - Historical Daily Rainfall Recorded at GEO Raingauge No. N15 between August 1984 and January 1998



Material	Range of Parameters Used in Analysis		
	Effective Cohesion c' (kPa)	Effective Angle of Shearing Resistance (ϕ')	Unit Weight (kN/m^3)
Completely Decomposed Volcanics	0-6	24°- 34°	18

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.
(3) Geological boundary is approximate only.
(4) Levels A to E refer to the assumed piezometric levels, used in the stability analyses (see Figure 12).

Figure 11 - Cross-section of the Landslide Used for Theoretical Stability Analyses



Legend : – – – – – – Factor of safety equal to unity (i.e. limiting equilibrium).

Figure 12 - Summary of Theoretical Stability Analyses

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Plate 1 - View of Hong Tsuen Road and Adjacent Cut Slope No. 8SW-C/C3 After Urgent Repair Work was Completed (Photograph Taken on 23 March 1998. Note the Photograph is Looking Southwest and the Landslide Occurred Where the Slope is Shotcreted, Near the Lamp Post)

Plate 2 - Partly Blocked Drainage U-channel on the Second Bench Adjacent to the Southwest Flank of the Landslide (Photograph Taken on 23 March 1998)

Plate 3 - Drainage U-channel Partially Blocked by Vegetation at a Catchpit (Photograph Taken on 23 March 1998. Note the Blockage Occurred after Repair Works were Completed)

Plate 4 - Site Record Photograph Showing "Lush Growth Where There is a Perennial Discharge into Cracked Surface Water Drains" (Photograph Taken on 14 January 1981)

Plate 5 - View of the Landslide (Photograph Taken on 3 July 1997. Note the Colour Change Across the Site from Orangish Red Residual Soil to Orangish Yellow Completely Decomposed Volcanics)

Plate 6 - Residual Soil Exposed by Erosion on the Slope Adjacent to the Landslide (Photograph Taken on 23 March 1998)

Plate 7 - Completely Decomposed Volcanics Exposed Near Ground Level in a Trial Trench Excavated to the Southwest of the Main Scarp of the Landslide (Photograph Taken on 9 July 1997)

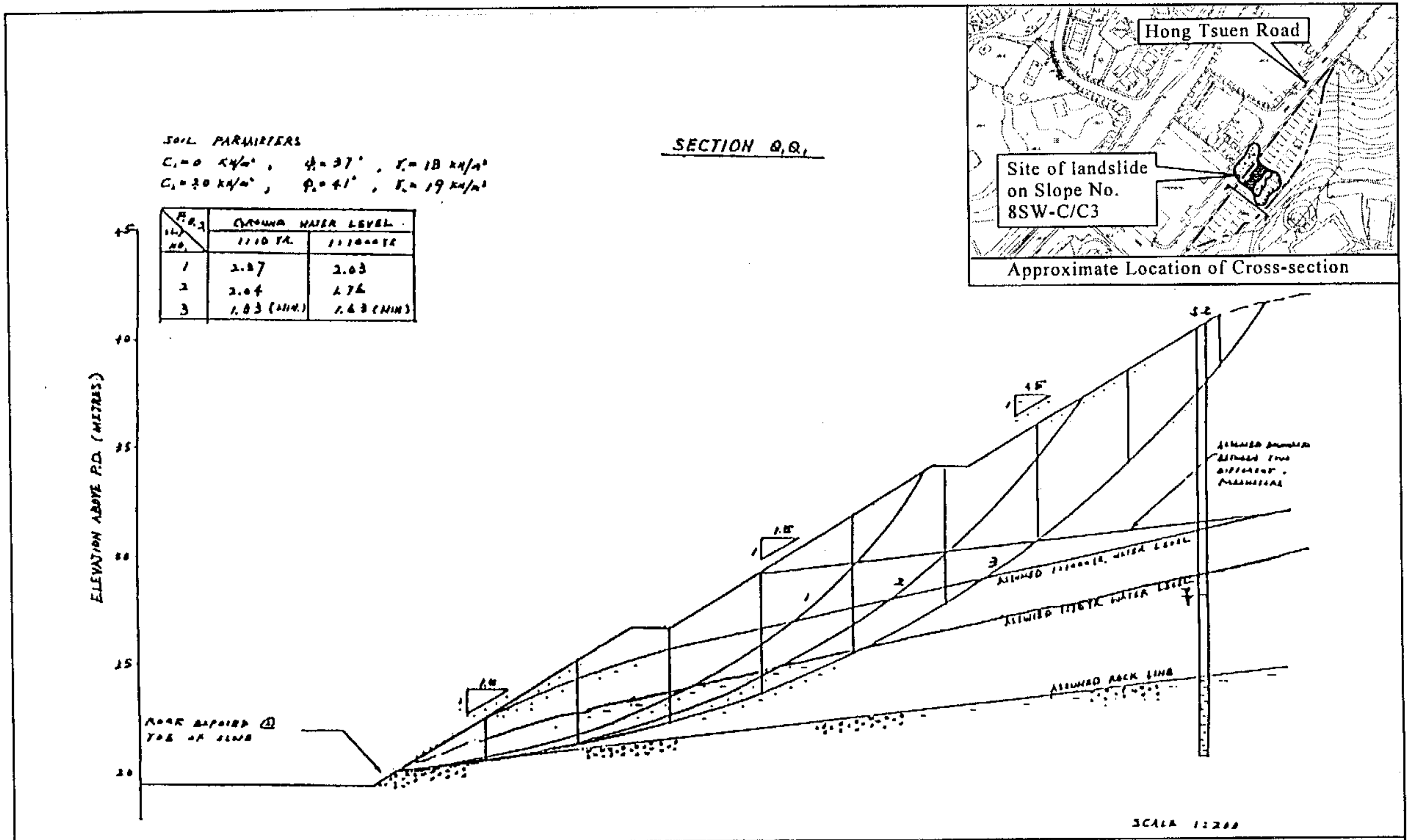
Plate 8 - Close-up View of the Main Scarp (Photograph Taken on 3 July 1997. Note the Blocks of Rock in the Debris Derived from the Southwest Corner of the Main Scarp)

Plate 9 - View of the Main Scarp Looking North (Photograph Taken on 3 July 1997. Note the Exposed Corestone Comprising Jointed Moderately Decomposed Volcanics in the Main Scarp and the Issue of Groundwater)

Plate 10 - View of Slope No. 8SW-C/C3 after Urgent Repair Work was Completed (Photograph Taken on 23 March 1998. Note the Landslide Scar Visible on the Upper Part of the Slope and Daylighting at the First Bench)

APPENDIX A

SECTION Q_1–Q_1 (BASED ON WSA, 1980)



Note: For location of section, see insert

Figure A1 - Stability Analysis for 1970's Slope Assessment (Section Q-Q_1)