REPORT ON THE SHEK KIP MEI LANDSLIDE OF 25 AUGUST 1999

GEO REPORT No. 181

Fugro Maunsell Scott Wilson Joint Venture
&
Professor J. B. Burland

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING AND DEVELOPMENT DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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Prepared by:

Geotechnical Engineering Office,
Civil Engineering and Development Department,
Civil Engineering and Development Building,
101 Princess Margaret Road,
Homantin, Kowloon,
Hong Kong.

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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. The GEO Reports can be downloaded from the website of the Civil Engineering and Development Department (http://www.cedd.gov.hk) on the Internet. Printed copies are also available for some GEO Reports. For printed copies, a charge is made to cover the cost of printing.

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R.K.S. Chan
Head, Geotechnical Engineering Office
April 2006

VOLUME 1:
FINDINGS OF THE LANDSLIDE INVESTIGATION

Fugro Maunsell Scott Wilson Joint Venture

**This report was originally produced in March 2000
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1. INTRODUCTION

At about 4:30 p.m. on 25 August 1999, the cut slope (No. 11NW-B/C90) behind Blocks Nos. 36 and 38 of Shek Kip Mei Estate was noted by residents to be suffering from significant signs of distress. Housing blocks Nos. 35, 36 and 38 were temporarily evacuated in the evening of 25 August 1999 at the recommendation of the Geotechnical Engineering Office (GEO) of the Civil Engineering Department (CED), as a precautionary measure. These three housing blocks were subsequently evacuated on a permanent basis on 26 August 1999.

Immediately following the landslide incident, the GEO instigated a comprehensive investigation into the failure. The investigation was carried out by GEO's landslide investigation consultants, Fugro Maunsell Scott Wilson Joint Venture (FMSW), with additional geological input from the Hong Kong Geological Survey (HKGS).

The investigation was carried out during the period August 1999 to February 2000, and comprised the following key tasks:

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

(b) detailed observations and measurements at the landslide site,

(c) interviews with witnessess to the landslide,

(d) establishment of the sequence of events leading up to the landslide,

(e) analysis of the rainfall records,

(f) geological mapping,

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

(h) establishment of geological and hydrogeological models,

(i) theoretical slope stability analyses, and

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

2. DESCRIPTION OF THE SITE

2.1 Site Description

The landslide (GEO Incident No. MW 1999/8/42) occurred on the cut slope to the rear of Blocks Nos. 36 and 38 of Shek Kip Mei Estate and at the south-eastern end of a ridge which extends northwards towards Beacon Hill (Figure 1 and Plates 1 and 2). A site plan

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and a plan of the landslide are shown in Figures 2 and 3 respectively.

The cut slope was registered as No. 11NW-B/C90 by the consultants engaged by Government to prepare the 1977/78 Catalogue of Slopes in 1977. The cut slope has a maximum height of about 50 m. The average gradients at the upper and lower portions of the slope are about 30° and 50° respectively to the horizontal. The northern end of the slope is different from the rest in terms of slope profile and comprises five batters each dipping at about 55° to the horizontal, separated by 1 m to 2 m-wide berms (Figure 3). The slope toe along the southern and central portions is at an elevation of about 17.5 mPD and descends to about 15 mPD at the northern portion. There is a platform at the central portion of the slope toe and between Blocks Nos. 36 and 38. A temple, comprising a light-weight steel frame structure, is located on the platform (Figure 3).

At the time of the landslide, much of the cut slope and adjoining cut slopes Nos. 11NW-B/C68, 11NW-B/C91 and 11NW-B/C585 (see Figure 2) were covered with chunam (and locally with shotcrete). However, no rigid surface cover was present at the unauthorized cultivation area near the hilltop above slope No. 11NW-B/C90 (Figure 2 and Plate 3). About 100 m north of the distressed slope is a heavily vegetated east-facing natural hillside. Further to the north of the natural hillside is slope No. 11NW-B/C582 (Figure 2), the lower portion of which comprises a sub-vertical rock face. A partially lined open channel is located along the toe of this rock face (Figure 2 and Plate 4).

An active site for the construction of a reinforced concrete service reservoir is located at the hilltop about 200 m to the north of slope No. 11NW-B/C90 (Figure 2 and Plate 5). The service reservoir, which forms part of the Shek Kip Mei No. 2 Fresh Water Service Reservoir Project of the Water Supplies Department (WSD), was under construction and had not been filled with water prior to the 25 August 1999 landslide.

There are no water-carrying services within or in the vicinity of slope No. 11NW-B/C90.

2.2 Maintenance Responsibility

Slope No. 11NW-B/C90 is located on "unallocated Government land" outside the Vesting Order (VO) boundary for Shek Kip Mei Estate. The VO boundary was vested in the Hong Kong Housing Authority (HKHA) in June 1985 by the Lands Department.

In 1996, the Lands Department commissioned a project called "Systematic Identification of Maintenance Responsibility of Slopes in the Territory" (SIMAR) to identify the maintenance responsibility of all man-made slopes registered in the Government's New Catalogue of Slopes compiled between 1994 and 1998.

In May 1997, the SIMAR project identified Housing Department (HD) as being responsible for maintaining slope No. 11NW-B/C90, based on one of the principles promulgated in an internal Government circular at the time that a slope on unallocated Government land utilised by or affecting Government Department without allocation would be maintained by the Department utilising the land. As the slope lies outside the boundary of Shek Kip Mei Estate that has been vested in the HKHA, the HD, also as the agent of the HKHA, lodged an appeal in October 1997.

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In 1998, there were exchanges of correspondence among concerned bureaux and departments on the subject of maintenance of slopes located on unallocated Government land adjacent to public housing estates. Consensus view was reached in March 1999 that these slopes should be maintained by HD using Government funds. The funding arrangements were agreed between the HD and the Works Bureau in June 1999.

3. DESCRIPTION OF THE LANDSLIDE

3.1 Observations Made Prior to the Landslide

Perennial seepage from the toe at the central portion of slope No. 11NW-B/C90 was noted by the residents of Shek Kip Mei Estate and personnel of the temple over the past years prior to the landslide (Figure 4).

In the summer of 1997, personnel of the temple noted minor bulging of the slope toe (over an area that measures about 1.5 m by 1.0 m) at the back of the north-western corner of the temple and cracking of the slope cover immediately to the west of the bulged area. Minor cracking of the concrete pavement was also observed on the platform to the north-east of the temple. The cracking and bulging were not reported to the GEO at the time.

3.2 Observations Made on 25 August 1999

The following pertinent observations at slope No. 11NW-B/C90 on the day of the landslide are based on interviews with eye-witnesses:

(a) At about 8:00 a.m., personnel of the temple observed some minor vertical movements (about 5 mm to 15 mm) along an old crack which developed on the concrete pavement of the temple platform in 1997 (see Section 3.1). This movement was not evident the evening before.

(b) At about 10:00 a.m., staff of the District Maintenance Office (DMO) of the HD observed seepage from the toe wall below the northern end of the temple platform during a daily routine inspection of Block No. 36.

(c) At about 11:00 a.m., minor detachment of material from an area of about 1 m across was noted on the slope below the northern end of the temple platform.

(d) At about 4:00 p.m., minor heaving of the concrete pavement and slight tilting of the fence along the toe of the northern portion of slope No. 11NW-B/C90 were noted.

(e) At about 4:30 p.m., the following observations were made:

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(i) the distressed concrete pavement referred to in (d) above had heaved by about 150 mm,

(ii) cracking of the chunam cover occurred on the lower part of the northern portion of the slope, and

(iii) the minor failure referred to in (c) above enlarged to about 2 m by 2 m by 1 m deep.

(f) At about 5:00 p.m., the heaving referred to in (e) above increased to about 300 mm and a few pieces of chunam fell from the northern end of slope No. 11NW-B/C90. Abnormal noises "ging gung, ging gung…" were heard from the slope behind the central part of Block No. 36.

(g) At about 5:10 p.m., collapse of debris, accompanied by a loud noise, was noted at the slope behind the central part of Block No. 36. The debris comprised wet soil and a fallen tree.

(h) At about 6:00 p.m., the slope toe at the northern portion was noted to have moved outwards by about several hundred mm to 1 m. No significant movement was observed at the southern and central portions of the slope toe.

(i) At about 7:15 p.m., the GEO emergency duty staff arrived at site. Shortly after that, the 2 m high steel chain link fence along the northern portion of the slope toe collapsed.

(j) At about 8:00 p.m., the GEO and FMSW landslide investigation staff observed the detachment at the northern portion of the slope as well as significant tension cracking and slope distress at both the northern and southern portions of the slope. Seepage was observed from the slope toe in the northern portion of slope No. 11NW-B/C90 (Figure 4) but no obvious seepage was observed in the southern portion of the slope toe. Based on periodic inspections made between 8:00 p.m. and 11:00 p.m., the slope outward movement in the northern portion of the slope was noted to have increased by a noticeable amount (about several hundred mm).

3.3 <u>Observations Made from 26 to 28 August 1999</u>

The following is a summary of the main observations made by FMSW and GEO staff between 26 and 28 August 1999:

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(a) The observed signs of distress, movement and seepage as of 26 August 1999 are shown in Figure 3. Seepages were noted at locations observed previously on 25 August 1999 as well as at other locations, which were not obvious on 25 August 1999 due to poor visibility. The adjacent slopes and hillside were also inspected in detail and the distressed area was found to be confined to slop No. 11SW-B/C90.

(b) In the morning of 27 August 1999, further cracking in the glass panels and tilting of the window frames of the temple structure were noted, indicating that additional movement of the slope had occurred at the area behind the temple. Seepages at the slope toe were observed to be continuing at previously noted locations.

(c) During the inspection of the WSD service reservoir site in the morning of 27 August 1999, it was noted that the permanent surface drainage provisions of the site were under construction and that a number of areas were ponded with standing water (Plate 5).

(d) Movement monitoring stations were installed on slopes Nos. 11NW-B/C90, 11NW-B/C68 and 11NW-B/C91 by the Survey Division of the CED and monitoring commenced on 26 August 1999. At about 5:00 p.m. on 27 August 1999, outward movements of up to about 10 mm were recorded at about mid-height of the central portion of slope No. 11NW-B/C90. No significant movement was recorded in the other portions of the slope and in the adjacent slopes.

(e) On 28 August 1999, a total outward movement of up to about 70 mm was recorded at the central portion where movement was previously recorded on 27 August 1999. No further slope movement was recorded at the monitoring stations from 29 August 1999 onwards.

3.4 Time of the Landslide

The timing of the landslide incident was assessed based on accounts of witnesses.

In the southern and central portions of the slope (referred to as the southern distressed zone in this report) where the mode of instability involved significant distress, cracking and deformation without complete detachment of the slipped material, slope movement was first observed by personnel of the temple at about 8:00 a.m. on 25 August 1999.

In the northern portion of the slope (referred to as the northern distressed zone in this report) where the mode of instability involved significant distress and outward movement of the slope toe together with local collapses of parts of the slope, slope instability was first

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observed some time about 4:00 p.m. on 25 August 1999. The majority of the movement occurred from 4:00 p.m. to 6:00 p.m. It is not known if slope instability at the northern portion had developed prior to 4:00 p.m.

The available information suggests that the distressed areas probably began to reach a state of instability in the morning of 25 August 1999. However, the expression of the instability probably involved a process in which signs of distress at different parts of the slope took time to develop and be observed by people near-by, as the instability did not involve complete detachment of the slipped material.

3.5 Detailed Field Mapping

3.5.1 General

FMSW staff commenced detailed field mapping of the landslide on the morning of 26 August 1999. The location and extent of slope distress and seepages are shown on Figure 3. The main features of the distressed slope are illustrated in Plates 6 to 16.

Notable signs of distress were generally confined to the middle and lower portions of the slope, along almost the entire length of slope No. 11NW-B/C90, behind Blocks Nos. 36 and 38 of Shek Kip Mei Estate. The distress was characterised by two zones, namely the northern and southern distressed zones, respectively (Figure 3 and Plate 2). The northern distressed zone was characterised by a well-developed main scarp, whereas the southern distressed zone was characterised by extensive surface cracking and bulging.

3.5.2 Northern Distressed Zone

In the northern distressed zone, the landslide scar was about 37 m wide, 31 m long and comprised essentially intact debris, which had been displaced downwards at the crest of the scar by up to about 1.2 m (Plates 6 and 7) and horizontally at the slope toe by about 1 m (Plate 8). The maximum depth of the landslide was estimated to be up to about 8 m, and the estimated volume of the debris was about 2500 m^3. The displaced material was mainly covered with cracked chunam and comprised medium to coarse-grained completely decomposed granite (CDG) with some moderately to slightly decomposed corestones. Apart from localized collapse of debris and corestones, the majority of the displaced material had a limited mobility and remained on the slope.

The movement of the displaced material, which was principally downwards along the face of the slope, caused an outward displacement of the concrete pavement in front of the slope toe. This in turn pushed over a steel wire chain link fence into a sub-horizontal position (Figure 3 and Plate 8). Several trees tilted and toppled at the slope toe as a result of the displacement.

A crack up to about 300 mm wide, 1.5 m deep and 18 m long was noted between the main scarp and the displaced material, and several cracks up to 150 mm wide together with minor scarps were observed on the displaced slope surface (Figure 3). The main scarp extended southwards behind a large corestone (about 2 m across) and into a local depression on the slope surface (Figure 3 and Plate 6).

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Within the area of the local depression, up to 0.5 m thick of loose debris, comprising mainly silty sand, was noted covering vegetation on the slope surface (Plate 9). Several corestones had been displaced from the upper portion of the scar (Figure 3), coming to rest within the dense, unplanned vegetation towards the toe of the slope. There was also evidence of surface erosion, probably related to the lower end of a gully, which had generally been lined with chunam, feeding surface runoff directly into the depression. Release planes along the right flank of the scar included relic joint sets dipping at 75° to 80° towards east southeast (Figure 3 and Plate 9), some of which were coated with kaolin and a black-brown deposit likely to comprise manganese and iron oxides (referred to as manganese oxide in this report). The left flank comprised north northwest stepping (upslope) en echelon release planes dipping at 50° to 75° towards the southeast and south southeast (Figure 3).

Seepages were observed along the toe of the slope and within the lower area of the local depression in the southern portion of the northern distressed zone (Figure 3).

3.5.3 Southern Distressed Zone

In the southern distressed zone, the cut slope comprised a complex series of east-facing slope batters and berms, the configuration of which was controlled mainly by a series of north northeast-south southwest trending coreslabs, i.e. tabular and laterally extensive bodies of slightly to highly decomposed granite as distinct from more equidimensional corestones (Figure 3 and Plate 2). On plan, the area of distress, mainly in the form of cracking, extended in a northeast to southwest orientation for about 60 m, up to about 35 m above the toe of the slope. The maximum depth of the displaced/distressed ground mass is estimated to be about 6 m, involving about 3500 m^3 of material. The displaced material exhibited only limited mobility and remained on the slope. No obvious scarp development was observed. There were also no signs of significant horizontal displacement at the toe of the slope.

The western margin of the southern distressed zone was characterised by a series of northwest stepping (upslope) en echelon tension cracks (Figure 3). The crest of the distressed area was denoted by the uppermost, recently-activated cracks observed on the chunam cover. Here the cracks were up to 50 mm wide and vertical displacement of up to 50 mm was noted, and about 50% of the cracks exhibited a relative downward displacement on the upslope side.

Displacement along the cracks was inferred to be mainly downslope, i.e. towards the east and southeast. In several areas, local relative upward displacement (about 25 mm) of the downslope side of the crack was observed, indicating a toppling mode of movement. Lateral displacement of up to 100 mm was observed across a concrete drainage channel along the right flank of the southern distressed zone (Figure 3 and Plate 12). This displacement, together with the nature of the cracks on the western margin, indicate a more general minor component of north-south orientated downslope movement.

Cracks up to 12 m long and 20 mm wide and generally orientated north-south to northeast-southwest were observed on the shotcreted surface of the slope toe opposite the northern portion of Block No. 38 (Figure 3). Similar cracks were noted in and beneath the chunam cover, to the north of the temple. Here, cracks up to 200 mm wide were noted,

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particularly on the downslope side of the coreslabs and corestones, often within a zone of exfoliation (Plate 10), i.e. along exfoliation planes within the weathered rock.

Cracking and heaving were observed on the concrete platform located to the north-east of the temple (Plate 13). In addition, minor displacement (up to 5 mm) was noted along a horizontal crack across a concrete pier supporting the temple structure (Figure 3 and Plate 14). The temple structure also showed signs of deformation, most notably on the southern side where glass window frames had tilted, the upper parts of the frame moving from northwest to southeast up to 85 mm (Plate 15).

A small landslide scar, characterised by shallow (up to about 200 mm) detachment of chunam and soil, was observed on the slope directly above the temple (Plate 9). Another small landslide scar was observed on the slope between the 1.8 m-high brick toe wall opposite the southern portion of Block No. 36 and the concrete platform to the north-east of the temple (Plate 16). The toe wall itself exhibited only minor cracking.

In many areas where the chunam slope cover was removed during the landslide investigation, widespread tree root systems were observed between the chunam cover and the ground underneath (Plates 17 and 18). This substantial root growth had in many cases, pushed the chunam cover away from the slope creating a void up to 100 mm wide. The presence of such voids beneath the chunam cover probably allowed ingress of water through local defects in the chunam cover to spread to other parts of the slope and infiltrate into the slope over a large area.

Localised seepages were noted, albeit to a lesser extent than that in the northern distressed zone, mainly at two locations, namely at the northeastern part of the temple and to the southwest of the temple about 4 m to 5 m above the slope toe (Figure 3).

4. HISTORY OF THE SITE

4.1 General

The site development history, previous slope assessments and slope works, together with history of past instability, have been determined from a review of the available aerial photographs and relevant documentation. Details are presented in Appendix A and the salient points are given in the following sections.

4.2 Site Development

Blocks Nos. 36 to 41 of the Shek Kip Mei Estate were constructed in 1954. As part of the site formation works undertaken by the Public Works Department (PWD), several cut slopes were formed at the southern toe of the hillside to the northwest of the Estate (Figure 2). Cut slopes Nos. 11NW-B/C68, 11NW-B/C90 and 11NW-B/C91 (Figure 2) were formed and covered with surface protection between 1949 and 1964.

Between 1974 and 1975, temporary structures were erected on the site of the present-day temple. These structures were extended in 1978 and some slope works involving trimming, minor filling and application of surface protection, were completed to the

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northeast of the temple. A concrete platform was subsequently constructed within this area in 1995.

By April 1980, cut slopes Nos. 11NW-B/C583, 11NW-B/C584, 11NW-B/C585 and 11NW-B/C232 had been formed (Figure 2). Slopes Nos. 11NW-B/C583 and 11NW-B/C584 and the uppermost part of slope No. 11NW-B/C232 were covered with grass whilst slope No. 11NW-B/C585 was covered with chunam. Slope No. 11NW-B/C90 was resurfaced, apart from the area at the toe of the slope opposite the southwestern portion of Block No. 36 and to the west of Block No. 38 (Figure A2 in Appendix A). Slope No. 11NW-B/C91 was trimmed back and horizontal drains were installed. The above cut slopes had not been modified since 1980.

Cut slopes Nos. 11NW-B/C583 and 11NW-B/C584 and the uppermost part of slope No. 11NW-B/C232 remained mostly covered with grass and a few bushes until around 1995 when these areas started to be used for cultivation purposes.

Between 1992 and 1999, the unplanned vegetation at slope No. 11NW-B/C90 became progressively denser, particularly within the old drainage lines (see Section 4.3) and along the berms, and more extensive in areal terms (see Figure A4 in Appendix A).

Excavation into the hilltop area for the WSD service reservoir site began in March 1995, and by November 1998 the main reinforced concrete structures had been constructed.

4.3 History of Instability

Prior to the construction of the Shek Kip Mei Estate in 1954, the hillside to the north of the Shek Kip Mei Estate site comprised a rounded, north-south trending ridgeline dissected on either side by many deeply incised gullies. The earliest known aerial photographs, which were taken in 1945, indicate a large relic landslide scar in the form of a concave depression located along the toe of the southeastern portion of this hillside within the present-day slope No. 11NW-B/C90. The relic landslide could possibly have been a coastal slide because this part of the hillside was not far away (within about 150 m) from the old coast-line according to the HKGS. This relic scar coincides with the depression on the present-day cut slope formation to the west of Block No. 36.

Aerial photographs taken in 1949 showed that the western margin of the depression was represented by a steep, linear face trending north northeast-south southwest (Figure A1 in Appendix A). This linear feature corresponds approximately to the main area of coreslabs identified to the northwest of the temple on the present-day slope configuration. The northern and eastern margins of the depression formed a generally south-facing slope below a small spur. The spur was delineated to the north by a drainage line leading to the northern portion of the present-day slope No. 11NW-B/C90 behind Block No. 36. Two drainage lines were also noted above the intersection between the western and northern margins of the landslide scar. One extended upslope towards a smaller landslide scar and a comparatively minor one trended across the coreslabs along the western margin (Figure A1 in Appendix A). Below the margins of the large relic landslide scar the area was terraced for agricultural purposes. The geometry of the scar at this time indicated that the direction of landslide

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debris movement was essentially from north to south.

In October 1995, boulders (about 2 m^3) fell from the central portion of slope No. 11NW-B/C90 (Figure 4). At the recommendation of the GEO, urgent repair works to the damaged slope area were carried out by the Highways Department (HyD).

4.4 Previous Assessments and Slope Works

4.4.1 Previous Assessments

In November 1973, Binnie & Partners (Hong Kong) (B&P) was commissioned by the PWD to report on slope stability in the North Kowloon area covering, inter alia, the present slope No. 11NW-B/C90 and the hillside above. The report stated that "Although the slope appears in good condition, its toe is very close to the buildings and it should be inspected after heavy rainfall. A phase II stability analysis is needed".

In June 1973, the Highways Office (renamed HyD in 1989) engaged Scott Wilson Kirkpatrick & Partners (SWKP) to carry out a geotechnical study of the slopes in the Shek Kip Mei area, which included slope No. 11NW-B/C90. In January 1977, SWKP prepared a geotechnical report on the study, which comprised site inspections and stability analyses on selected slopes. The report did not recommend any specific works to slope No. 11NW-B/C90, but some general recommendations were made concerning improvement of defective slope cover and drainage provisions.

In January 1978, B&P prepared a report, under the Phase IIC Landslide Study commissioned by the Geotechnical Control Office (GCO, renamed GEO in 1991), on the stability of slopes in the Shek Kip Mei area, which included slope No. 11NW-B/C90. No ground investigation was carried out on this slope. The report recommended that "All cracks in the chunam should be repaired", "Weepholes should be installed in the lower third of the slope and all U channels should be cleared out and repaired" and "Major trimming and protection works will be required above slope C90".

In July 1994, the GEO commenced 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 GEO's consultants for the SIRST project inspected slope No. 11NW-B/C90 in September 1996. The SIRST report recorded the presence of fine cracks in the chunam cover at the mid-portion and toe of the slope. The report also recorded signs of seepage and that, stepped channels were cracked.

The HKHA commissioned consultants to carry out Annual Geotechnical Inspections (AGI) in 1980, 1982, 1984 and 1986 on slope No. 11NW-B/C90. After the establishment of the VO boundary for Shek Kip Mei Estate in 1985 by the Lands Department, HKHA's consultants have carried out a Special Area Inspection (SAI) in 1993 and AGI in 1995, 1997 and 1998, on the lower part of slope No. 11NW-B/C90. During these inspections, disrupted chunam cover, disrupted drainage channels and unplanned vegetation were observed. Details of the observations and recommendations made in the inspection reports are summarised in Table A2 in Appendix A and the key observations are summarised in Figure A5 in Appendix A.

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4.4.2 Previous Slope Works

Following the studies by SWKP (1977) and B&P (1978), slope works were carried out by the Highways Office under PWD Contract No. 668 of 1977. The works at slope No. 11NW-B/C90 included partial replacement/repair of the existing chunam cover and surface channels, removal of vegetation and clearing of surface channels. In addition, the hillside above slope No. 11NW-B/C90 was trimmed and subsequently covered with chunam. Details of the works on the adjoining slopes are given in Section A.2 in Appendix A. The works carried out covered all the items of works recommended by SWKP (1977) and B&P (1978).

Based on HKHA's records, general slope maintenance works were carried out on slope No. 11NW-B/C90 in 1981, 1983, 1984, 1985, 1991 and 1992.

5. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge (No. K06), installed in 1982, is located at So Uk Estate, which is about 1 km to the northwest of the landslide site. As slope No. 11NW-B/C90 was constructed around 1959, the rainfall data from the Principal Raingauge (No. R01) of the Hong Kong Observatory (HKO) at Tsim Sha Tsui with records dating back to 1884, which is about 4 km to the southeast of the landslide site, was also examined to assess the likely rainfall pattern at the landslide site between 1959 and 1982.

The rainfall data from the two raingauges were generally consistent and reliable. As raingauge No. K06 is closer to the landslide site, its records should be more representative than that from raingauge No. R01.

The daily rainfall recorded by the raingauges for the period of 23 July 1999 to 31 August 1999 is shown in Figure 12. The hourly rainfall records of raingauges Nos. K06 and R01 between 21 and 25 August 1999 are presented in Figure 13. As discussed in Section 3.4, the slope movement at the southern distressed zone was first observed at about 8:00 a.m. of 25 August 1999 and the movement essentially stopped on 28 August 1999. The instability at the northern distressed zone was first observed at about 4:00 p.m. of 25 August 1999 but it is not known if slope instability had developed prior to that time. For the purpose of rainfall analysis, the landslide was assumed to occur at 8:00 a.m. of 25 August 1999.

Analysis of the return periods of the rainfall intensities of the rainstorm before the landslide for different durations based on historical rainfall data at the HKO shows that the 4-day rainfall was the most severe, with a corresponding return period of about 31 years (Table 1).

A comparison between the patterns of past major rainstorms recorded by raingauge No. K06 between 1982 and 1999 is shown in Figure 14. The rolling rainfall for the period between 24-hour and 14-day durations preceding the landslide was one of the most intense rainfall events experienced by the raingauge No. K06 since its installation in 1982, and was comparable to the severe rainstorm in July 1997.

A comparison between the patterns of past major rainstorms recorded by raingauge No. R01 between 1959 and 1999 is shown in Figure 15. The rolling rainfall indicates that

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the rainstorm preceding the 1999 landslide was also one of the most intense rainstorms recorded by raingauge No. R01 since slope formation around 1959.

6. SUBSURFACE CONDITIONS AT THE SITE

6.1 General

There was no previous ground investigation at slope No. 11NW-B/C90.

The subsurface conditions of the landslide site and the adjacent area were determined using information from desk and field studies. The desk study comprised a review of existing data, including the results of the ground investigation carried out between 1975 and 1998 on cut slopes Nos. 11NW-B/C66 and 11NW-B/C232 and at the service reservoir construction site at the top of the hill (Figure 16). The field studies included post-failure geological mapping and ground investigation supervised by FMSW.

Geological mapping of the landslide site commenced on 26 August 1999. The fieldwork for the post-failure ground investigation commenced on 13 September 1999.

Bachy Soletanche carried out the initial ground investigation works, which comprised four drillholes and three pumping wells near the top of the hill above the distressed slope, together with two trial pits and two surface strippings (Figure 17).

Fraser Construction Co Ltd continued this initial ground investigation within and adjacent to the distressed slope with an additional seven trial pits (Figure 17).

The main phase of the ground investigation within the distressed slope was carried out by Gammon Construction Ltd and commenced on 1 November 1999 after the completion of emergency slope repair works. The ground investigation comprised ten vertical and four horizontal drillholes, eight trial trenches, four trial pits and eleven surface strippings (Figure 17).

Additional ground investigation, comprising six double-ring infiltration tests and 12 percolation tests, together with installation of 11 tensiometers (Figure 17), was carried out by MateriaLab Ltd in November and December 1999.

Pumping tests and dye tracer tests (Figure 17) were carried out by FMSW in December 1999 and January 2000 respectively.

A geophysical survey using the gravity method was carried out by the Institute of Geophysical & Geochemical Exploration of the People's Republic of China between October and December 1999.

6.2 Geology and Geomorphology

An extract of Sheet 11 of the 1:20,000 scale geological map of Kowloon and Hong Kong Island (GEO, 1989) is shown in Figure 18. The site was mapped by the HKGS as medium-grained granite of Jurassic to Cretaceous age. Two photolineaments are indicated

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trending northeast to southwest through the hillside at the site currently occupied by the WSD service reservoir (under construction) approximately to the north of the landslide site. Joints dipping at 50° and 70° towards the northeast are also indicated on the eastern side of the hill. To the north of the WSD service reservoir site, a north northeast-south southwest trending fault is inferred, together with a similarly trending photolineament, both of which can be traced northwards into coarse-grained granite at Beacon Hill (Figure 18). Quaternary alluvium is shown on the low ground underlying Shek Kip Mei Estate but no superficial deposits are shown on the steeper slopes surrounding the landslide site.

As the hillside to the north of Shek Kip Mei Estate lies at the foot of the large catchment area of Beacon Hill, and given the regional orientation of the ridgelines (i.e. north-south trending), it is likely that subsurface water will be directed towards the hillside and its surrounding areas.

A photolineament trending east-west to the south of the present-day WSD service reservoir site can be identified on aerial photographs (Figures 2 and A1 in Appendix A). The photolineament was associated with a low point on the north-south trending ridge. It is not certain as to the effects of this photolineament on the hydrogeology of this part of the hillside. To the north of the low point, the ridge stepped upwards. This area is where the WSD service reservoir is now under construction, involving significant excavation (up to about 30 m) of the original ridge crest.

The geological features observed at the landslide site are shown in Figure 19. The inferred bedrock contour plan, based on results of drillholes, geological mapping and geophysical survey, is shown in Figure 20. Geological sections through the landslide site are shown in Figures 21 to 26.

The geology at the landslide site, as mapped by FMSW (Figure 19), comprises partially weathered, predominantly medium-grained and occasionally porphyritic granite. The partially weathered rock (mainly PW 0/30) consists of light yellowish and reddish brown, spotted black and white, highly to completely decomposed granite (H-CDG), with corestones of pinkish to light grey, speckled black and white, moderately to slightly decomposed granite (M-SDG). Complete or partial kaolinisation of the feldspars was common in the highly to completely decomposed granite.

Areas of PW 50/90 (comprising the coreslabs) and PW 30/50 are located to the west and northwest of the temple (Figure 19 and Plates 2 and 9). The coreslabs form prominent north northeast-south southwest orientated bands of rock. These were also identified as the western margin of a probable relic landslide scar in the earliest known aerial photographs taken in 1945 and 1949 (Figure A1 in Appendix A).

The orientation of the coreslabs is generally defined by a set of persistent, medium to widely (occasionally very closely) spaced, planar, often slickensided and steep (70° to 85°) discontinuities dipping west to west northwest (dip directions 265° to 285°), i.e. into the slope. The slickensides commonly plunge down the dip of the discontinuities. Slickensides also plunge gently to north northeast. The same set of discontinuities was also observed as closely and extremely closely-spaced, dipping at 55° to 85° within the HDG and CDG, especially along broad (generally up to 1.5 m but locally up to 3 m) exfoliation zones around corestones. Infilling of slickensided and polished kaolin (up to 30 mm thick) and manganese

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oxide deposits was typically found along relic discontinuities within the HDG and CDG. The zones of exfoliation and slickensided, moderately to steeply dipping discontinuities were commonly the focus of slope displacement within the distressed zones.

Two sets of persistent and widely-spaced joint sets were mapped within the coreslabs dipping at about 48° and 56° towards the northeast and southwest (dip directions 015° and 220°) respectively (Figure 19). In addition, a sub-horizontal to shallowly inclined, closely to widely-spaced and undulating joint set was mapped dipping variably at 10° to 30° towards the east and southeast (dip direction 100° to 150°). Other significant joint sets were mapped, particularly along the northern portion of the distressed area dipping moderately to steeply (52° to 88°) towards the east southeast to south southeast (dip direction 160°), see Figure 19. These joints locally formed release planes along the flanks of the northern distressed zone. Stereoplots for the measured orientations of discontinuities within the highly to completely decomposed granite are shown in Figure 27.

In general, granite slopes of Hong Kong and Kowloon are characterised by two orthogonal, and sometimes four, sets of sub-vertical joints. Therefore, the dominance of moderately to steeply inclined discontinuities, and particularly the occurrence of a moderately to steeply inclined zone of coreslabs dipping into the slope, are noteworthy features of slope No. 11NW-B/C90.

To the northeast of the temple, three areas of rock outcrop (PW 90/100) were mapped (Figure 19), the upper boundary of which dipped gently (20°) to the northeast along the toe of slope No. 11NW-B/C90. The lower limit of slope distress was observed about 0.5 m above this boundary.

A fault/fracture zone was mapped on the rock slope at the toe of slope No. 11NW-B/C91 (Figure 19 and Plate 19). The zone was characterised by persistent, widely to very closely-spaced, tight to narrow, often slickensided (plunging gently to the north northeast), planar, and steep (82° to 90°) discontinuities dipping east to east southeast (dip direction 100° to 110°) and west to west northwest (dip direction 270° to 290°), see Figure 19. The dominant discontinuity set defining the boundary of the coreslabs may reflect the same tectonic regime as this zone. The presence of the fault/fracture zone, with persistent discontinuities trending towards slope No. 11NW-B/C90, may have a significant influence on the hydrogeology at the landslide site. This is further discussed in Section 8.3.2.

Persistent sub-horizontal discontinuities were also identified during the post-failure ground investigation (see Sections 6.3.2 and 6.3.3).

6.3 Ground Investigation

6.3.1 Northern Distressed Zone

The majority of the landslide debris was composed of CDG with corestones of S-MDG. Open and infilled cracks were noted, extending to a depth of up to about 1.5 m and 200 mm wide along discontinuities, around corestones and within the CDG. The cracks often contained mature root systems, indicating that extensive cracking had occurred some significant time before the recent instability.

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At the southern and central parts of the slope toe, a distinct slip surface (i.e. surface of rupture) was observed at about 150 mm above S-MDG within PW 0/30 mass weathering zone and dipping gently downwards (about 6° in the southern portion and about 20° in the central portion) towards the southeast. The slip surface was typically planar and within a relatively uniform layer (about 10 mm thick) of moist, soft, dark grey silty clay (Figure 21 and Plate 20).

A layer (about 100 mm thick) of slightly disturbed, moist and loose CDG with some localised shear surfaces was generally observed above the dark grey silty clay. Immediately below, and occasionally above, the dark grey silty clay layer, kaolin and manganese oxide deposits were noted overlying a thin (5 mm to 50 mm) layer of sheared H-CDG. Seepages were observed directly above the dark grey silty clay layer following the landslide event. The origin of the dark grey silty clay layer may be related to previous infilling along a pre-existing discontinuity, which is close to the main boundary between weathering grades V/IV and III/II.

At the crest of the scar, the slip surface was partially located along a soil-infilled crack (up to about 1.5 m deep and 300 mm wide) which passed into CDG (Figures 21 and 22, and Plate 21). Within the CDG, the slip surface generally comprised a 60 mm thick layer of loose remoulded CDG comprising fine to coarse sand and fine gravel.

Between the crest and toe of the landslide scar, the postulated slip surface was characterised by disturbed, remoulded CDG based on detailed logging of the full-length core from drillhole. However, it is possible that the slip surface also locally exploited discontinuities with weak infill material. Immediately above the crest of the landslide scar the slope locally comprised up to 0.5 m of fill above CDG, with medium to closely-spaced soil-infilled tension cracks, typically 1.5 m deep and 100 mm to 200 mm wide (tapering with depth), and dipping very steeply towards the southeast (Figure 28). Above many of the soil-infilled tension cracks, distress was observed on the chunam cover (some of which was previously re-sealed).

Within the local depression in the landslide scar, drillhole No. DH6 revealed 8 m of CDG above SDG. A sheared layer (up to 20 mm thick) of clay/silt, encountered at 7.6 m depth, indicated a possible slip surface at this horizon.

At the northern end of the slope toe, a dark grey silty clay layer was noted to be about 300 mm below the slip surface associated with the August 1999 instability, and extended below and beyond the left flank of the scar, suggesting that at least locally in this area, the slip surface was through the CDG and did not exploit this pre-existing, clay-infilled discontinuity. A similar dark grey silty clay layer, which also did not appear to have been exploited by the slip surface, was observed within the CDG at 3.8 m depth in drillhole DH1 (Figure 21).

6.3.2 Southern Distressed Zone

Towards the toe of the slope, a laterally-persistent (over 60 m long) discontinuity was mapped, dipping very shallowly (less than 10°) towards the east and northeast (Figures 19, 23 and 25). Towards the toe of the western margin of the distressed zone, the discontinuity split locally into several discontinuities, within a zone up to about 1 m thick, and were dipping variably between 10° and 30° towards the southwest and south to southeast. These

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discontinuities were generally infilled with polished, slickensided kaolin and manganese oxide deposits, but the kaolin was less well developed than along the principal discontinuity further north. The principal discontinuity was slightly undulating and predominantly infilled with polished, slickensided kaolin (up to 15 mm thick) and manganese oxide deposits (Plates 24 and 25). A planar slip surface was observed passing through the kaolin and manganese oxide infilling. This laterally-persistent discontinuity is interpreted as the basal slip surface of the 25 August 1999 instability, as the extent of slope distress was generally observed close to or above this discontinuity. To the south of the temple, displaced (about 1.5 m to the southeast) sub-vertical (dipping steeply to the west) discontinuities infilled with kaolin and manganese oxide deposits were observed immediately above this laterally-persistent discontinuity (Figure 25).

The slickensides along the laterally-persistent discontinuity were measured as plunging gently (about 5°) in a north-south to north northwest-south southeast orientation. A similar orientation of slickensides, plunging at about 10° to 30°, was observed on other discontinuities dipping more steeply into the slope. These observations of previous movements in a general north-south orientation provide further evidence consistent with relic landsliding as noted in the earliest available aerial photographs (1945).

Slickensides noted along the recent slip surface, comprising thin, grey silty clay within the kaolin and manganese oxide infilling, were generally orientated to the southeast and east southeast (i.e. out of the slope) and in the same direction as the slope movement associated with the August 1999 slope instability. This suggests that the recent slope movement had exploited the pre-existing, laterally-persistent discontinuity within the PW 0/30 mass weathering zone. The discontinuity was located above a zone of PW 90/100 at the toe of the slope to the northeast of Block No. 38, and could be traced to, and about 30 m to the southeast of, the southern side of the temple. The discontinuity is similar in nature to those noted on the slope beyond the distressed zones and this is discussed further in Section 6.3.3.

To the north of the temple, dislocated sub-vertical discontinuities infilled with kaolin and manganese oxide deposits, indicating probable previous displacement of up to about 0.5 m, were located in trial pit No. AP6 (Figures 23 and 24 and Plate 22). On the southern face of AP6, a relatively intact raft of CDG appeared to have been locally forced over an area of fill, which indicates a similar amount of displacement, suggesting that slope deformation had occurred in this area some time previously following placement of fill (Plate 23).

Closely-spaced, sub-vertical tension cracks (up to 250 mm wide and more than 3 m deep), infilled with loose to dense brown slightly clayey sandy gravelly silt were noted on a spur-type feature within the PW 0/30 at the crest of the southern distressed zone (Figure 28 and Plate 26). The sub-vertical cracks were generally dipping towards the east and west. Cracks in the chunam cover (indicating up to 50 mm vertical displacement) were observed directly above some of the infilled tension cracks within the CDG. Similar features were noted beyond the distressed zones and these are discussed further in Section 6.3.3.

Seepages were observed above the laterally-persistent discontinuity to the north of the temple after the instability and throughout the ground investigation period. Seepages were also noted from weepholes on the cut slope to the south of the temple, at a similar elevation as the inferred basal slip surface, though it was dry when exposed in surface strip No. SS9.

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6.3.3 Areas Beyond the Distressed Zones

Laterally-persistent discontinuities of similar orientation to those seen near the toe of the southern distressed zone, were also traced for over 25 m within the CDG along slope No. 11NW-B/C68 (Figure 19). In general, the discontinuities dip very gently (less than 10°) to moderately (30°) to the east southeast, southeast and south southeast (dip direction 100° to 160°) and typically comprise a polished and slickensided slightly undulating plane (plunging towards the southeast), infilled with kaolin and manganese oxide deposits (up to 4 mm thick). It is likely that this discontinuity is the same set as that located near the toe of the slope within the southern distressed zone and the set of low-angle discontinuities mapped across the site dipping towards the southeast (Section 6.2). The slickensides suggest that relic movement along these discontinuities is generally in a southeast direction. Roots and rootlets were commonly observed along the laterally-persistent discontinuities as well as within other steeply inclined discontinuities.

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Although the origin of the shallowly inclined discontinuities (including that located at the slope toe) is not fully understood, they may represent stress relief sheeting joints. These discontinuities have been the focus of slope movement in the past. According to the HKGS, shallowly inclined discontinuities of this type are not uncommon but the degree of lateral extent observed at the landslide site is not commonly recognised in Hong Kong.

Pre-existing infilled tension cracks were noted along surface strips below apparently intact chunam cover on slopes Nos. 11NW-B/C90 and 11NW-B/C585 above the distressed areas (Figure 28). Overall, the fact that many infilled tension cracks were noted in the surface strips reflects the widespread existence of tension cracks in both the distressed zones and the area above for some time before the 1999 failure. These cracks varied between very widely and very closely spaced and between 5 mm and 200 mm wide, and were infilled with brown slightly clayey sandy gravelly silt. The cracks were typically striking parallel to the slope face and dipping steeply to the northwest or southeast. However, some were orientated obliquely to the slope and may have been formed by preferential tension cracking along relic joints. These features exhibit very similar characteristics as those noted at and above the crests of the northern and southern distressed zones, and indicate that slope movements occurred before the slope was covered with chunam. However, the age of the tension cracks is uncertain.

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A sub-vertical crack (up to 700 mm wide and at least 3 m deep) dipping steeply towards the southwest was observed to the northeast of a relic landslide scar and deep gully (Figures 28 and A1 in Appendix A). The crack was infilled with silty gravelly sand, except for the upper 0.5 m which comprised fill/placed material (including bricks). The silty gravelly sand exhibited an undisturbed layered structure suggesting that it was predominantly deposited by water and that no reactivation of the crack had occurred. The location and orientation of the crack suggest that it could have been associated with the relic landslide scar to the south (see Figures 28 and A1 in Appendix A), and hence it was probably a local feature.

6.4 Soil Properties

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Laboratory tests were conducted on soil samples retrieved from the ground investigation carried out following the August 1999 landslide. The soil tests included particle size distribution, Atterberg limits tests, direct shear box tests and consolidated undrained triaxial compression tests with pore water pressure measurement.

The results of particle size distribution and Atterberg limits tests on the CDG samples are summarised in Table 2. The fines (i.e. clay and silt) contents range from 6% to 54% with the majority of the results ranging from 10% to 20%. The sand and gravel contents range from 34% to 61% and 12% to 47%, respectively. The plasticity index ranges from 15% to 23% and the liquid limits range from 43% to 56%. These test results indicate that the CDG is a silty gravelly sand of intermediate plasticity.

Results of triaxial compression tests on CDG samples are summarised in Table 3 and presented in Figure 29. The shear strength parameters of CDG derived from the best linear fit of test results are $c' = 8$ kPa and $\phi' = 38°$, which are within the typical range for CDG in Hong Kong (GEO, 1993). The lower bound values of the test results correspond to $c' = 2$ kPa and $\phi' = 36°$.

The shear strength of discontinuities infilled with kaolin and manganese oxide deposits was assessed by direct shear box tests on specimens prepared from block samples. The setting of the direct shear box test was modified by using gypsum to mount the specimen in the shear box in order to confine the shearing essentially along the discontinuity. The specimens were soaked in water, consolidated and then sheared as in a standard test. For tests on slickensided discontinuities, specimens were prepared in such a way that shearing was to take place along the slickensided direction. The results of the direct shear box tests (summarised in Table 4 and Figure 30) indicate that the shear strength parameters are $c' = 0$ kPa and $\phi' = 20°$.

Double-ring infiltration tests and percolation tests in trial pits were carried out in November and December 1999 in different areas of the hillside to determine the infiltration rates of the ground mass. The test locations are shown in Figure 17. The results of field infiltration and percolation tests, which are summarised in Tables 5 and 6 respectively, indicate that the infiltration capacity of the soil surface at the WSD service reservoir site is about one order of magnitude lower (i.e. 10 times less) than the results obtained within the old drainage lines of slope No. 11NW-B/C90, within the cultivation area and on the natural hillside north of slope No. 11NW-B/C91.

6.5 Groundwater Conditions

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

(a) Observations of perennial seepage at the toe of the central portion of slope No. 11NW-B/C90 prior to the landslide (Section 3.1).

(b) Observations from 1995 site record photographs of seepages from weepholes located above the slope toe to the southwest of the temple (Figure A5 in Appendix A).

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(c) Post-landslide observations of seepages at the toe of slope No. 11NW-B/C90 in the northern distressed zone. Water was observed seeping out of material immediately above the slip surface exposed in the trial pits and trial trenches at the toe of the northern distressed zone. This was observed up to 2 months following the landslide.

(d) Post-landslide observations in the southern distressed zone of localized seepages to the northeast of the temple and above the slope toe to the southwest of the temple, i.e. at the same location as described in (b) above (Figure 3). However, the slope to the southwest of the temple was generally dry during the subsequent ground investigation in December 1999. Localised seepages were also observed immediately above the laterally-persistent, shallowly-dipping infilled discontinuity in the trial pits and trial trenches at the platform to the northeast of the temple, as well as in a trial pit and a local landslide scar at the northern portion of the slope toe within the southern distressed zone.

(e) Post-landslide observations of seepages from some of the horizontal drains in slope No. 11NW-B/C91, and at the interface between rock outcrop and the chunam-covered soil portion towards the toe of this slope, and also along the staircase on slope No. 11NW-B/C68 (Figure 3, Plates 27 and 28). Seepage was also noted from horizontal drillhole No. H2 which was drilled at the southern portion of slope No. 11NW-B/C91, i.e. close to the landslide site.

(f) Post-landslide groundwater monitoring data for piezometers with Halcrow buckets from December 1999 to January 2000 (Figure 31).

(g) Post-landslide monitoring data for tensiometers from November 1999 to January 2000. The results indicate that the suctions prevailing in the matrix material of the ground mass range from about 25 kPa to about 80 kPa between 1.5 m and 5.5 m below ground surface. The suctions generally decreased following rainfall.

(h) Pumping tests carried out from 17 to 23 December 1999. The results of the pumping test indicate that groundwater extraction at the well locations within testing levels between 48 mPD and 18 mPD had no obvious effect on the groundwater regimes of the hillside to the north of the distressed slope.

(i) Field infiltration and percolation tests carried out in November and December 1999 over various parts of the

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hillside (Figure 17) indicate that the average infiltration capacity of the exposed soil surface at the WSD's service reservoir site was significantly lower than those along the old drainage line of slope No. 11NW-B/C90, within the cultivation area and on the natural hillside north of slope No. 11NW-B/C91. It would appear that a region of high permeability ground exists beneath the old drainage lines and this would increase any infiltration resulting from damaged chunam in the drainage lines. Also, the cultivation activities probably involved disturbance to the near-surface material resulting in a higher permeability.

(j) Post-landslide observations of water ponding on parts of the exposed surface of the WSD's reservoir site, between 27 August and 30 August 1999, during which time there was no significant reduction in water level within the ponded area.

(k) Pre- and post-landslide observations that various parts of slope No. 11NW-B/C90 had unplanned trees and grass, especially along old drainage lines and within concrete drainage channels (Plates 17 and 29) and weepholes. Voids were noted between the cracked chunam cover and the soil beneath (Plate 30), most notably where tree roots had penetrated, forcing the chunam cover away from the slope (Plate 18).

Based on the above information, it is postulated that there are two groundwater regimes, namely one at depth within or close to the boundary between weathering grades V/IV and III/II (the lower groundwater regime), and the other within the soil profile above this boundary (the upper groundwater regime).

The lower groundwater regime is controlled primarily by the regional groundwater regime within or close to the bedrock (corresponding to the regime extending from Beacon Hill), but it can also be affected by surface infiltration, particularly where high permeability zones (e.g. old drainage lines, pre-existing tension cracks, etc) are present in the soil profile. The upper groundwater regime is controlled primarily by direct infiltration and recharge through subsurface seepage following direct infiltration into the uphill areas. However, the hydrogeological setting of the site, with extensive tension cracks, displaced ground mass, old drainage lines and persistent infilled discontinuities, is such that groundwater flow in the soil profile will be highly complex. Groundwater flow is probably very preferential, being concentrated along conduits or zones of relatively high permeability. The ground mass is also conducive to transient development of cleft water pressures in tension cracks, local build-up of water pressure at infilled discontinuities and seepage pressure associated with downslope groundwater flow, in response to water ingress.

In the northern distressed zone, the lower groundwater regime corresponds to a base groundwater table generally within or close to rockhead towards the toe of the slope.

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In the southern distressed zone, the lower groundwater regime is much below the displaced ground mass. The upper groundwater regime following rainfall may correspond to the transient development of groundwater pressure perching above the persistent, shallowly-dipping discontinuity infilled with kaolin and manganese oxide deposits, which is consistent with the observation of isolated seepages after the landslide.

7. THEORETICAL STABILITY ANALYSES

7.1 General

Theoretical stability analyses were carried out as an aid to the diagnosis of the probable mechanism and causes of the landslide. Information obtained from the post-failure ground investigation, laboratory testing, and site observations and measurements was used in the analyses. It must be stressed that the mechanisms adopted in the analyses are, of necessity, highly idealised. The main objective of the work was to carry out simple sensitivity studies.

As discussed in Section 3.5, the northern distressed zone involved a distinct failure whereas the southern distressed zone involved complex slope movements but not a fully developed failure. The detailed investigation has shown the complexity of the ground conditions, particularly for the southern part of the slope where the coreslabs are located. The slip surface for the northern distressed zone is fairly well defined, with the development of a main scarp and significant movement at the slope toe. However, the southern distressed zone did not show a fully developed slip surface, the overall outward displacement was not very significant and there was no main scarp, except for the open tension crack near the crest of the distressed zone and the persistent, shallowly-dipping discontinuity with weak infill of kaolin and manganese oxide deposits as revealed in trial trenches near the slope toe (Section 6.3).

Due to the presence of the laterally continuous coreslab, the ground conditions in the southern distressed zone are complex and locally extreme variations exist in the weathering profile. Consequently a degree of uncertainty exists with respect to the geological model. It is likely that the slip surface was significantly affected by the degree of connection of the coreslabs and the unweathered rock mass in three-dimensional space.

7.2 Northern Distressed Zone

The geological profile for the northern distressed zone is shown in Figure 21. A representative cross-section of the northern distressed zone and input parameters adopted in the conventional limit equilibrium analysis are shown in Figure 32.

Given the uncertainties regarding the groundwater conditions prevailing at the time of failure, sensitivity analyses were carried out using different assumptions on groundwater conditions and suction values assuming the shear strength parameters defined by laboratory testing on undisturbed CDG samples.

The results of the analyses are summarised in Figure 32. For a factor of safety of unity (i.e. at failure), the analyses suggest that failure could have been brought about by

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wetting up of the soil mass following surface infiltration, together with a rise in the base groundwater table. Given the presence of drainage lines, tension cracks and damage to the chunam, it is likely that infiltration played a major role in terms of water ingress into the slope and the average operational soil suction value along the slip surface is unlikely to be very high.

A significant rise in the base groundwater table above the slope toe is not consistent with site observations made on the day of the landslide. The scenario involving failure being caused solely or primarily by a large increase in the base groundwater table with significant suction being maintained in the soil mass above Grade III or better rock is therefore considered unlikely in view of the observations and site setting, where the slope is prone to surface infiltration, wetting up and development of cleft water pressure.

In the case of complete wetting up of the soil mass above Grade III or better rock and assuming average shear strength parameters as determined from laboratory testing, results of analyses suggest that failure would not occur if there was no rise in the base groundwater table. The actual mass shear strength of the ground may be somewhat lower than that determined in the laboratory and this will make the slope more vulnerable to the effects of water ingress.

Overall, it is credible that failure was brought about by the combined effect of wetting up of the soil mass and transient elevation in the base groundwater table.

7.3 Southern Distressed Zone

As explained in Section 7.1, the geological profile within the southern distressed zone (Figure 23) is less certain. If the coreslabs are taken to correspond to essentially isolated, sizeable corestones, then a potential failure surface could be postulated through the weathering profile. The basal plane of the postulated slip surface is fairly well defined by a slickensided, planar slip surface within the persistent infilled discontinuity. The downslope direction of the slickensiding (Section 6.3.2) is consistent with the distress at the temple structure caused by the slope movement. The uppermost margin of the distressed zone is defined by a tension crack along a steeply dipping discontinuity.

For the purposes of examining the likely contribution of structural control of the failure by the weak, persistent infilled discontinuity, analyses assuming a simplified geological model shown in Figure 33 have been carried out. Site observations on the night of the failure suggest that there was much less groundwater involved at the southern distressed zone compared to the northern distressed zone. Also, it should be noted that slope failure had not fully developed and the overall factor of safety could well be a little above unity.

The shear strength of the persistent, infilled discontinuity has been assessed by direct shear box testing. In the analyses, assumptions are made regarding the degree of residual soil suction, strengths of the CDG (as defined by the average and lower bound values of triaxial test results to allow for possible influence of previous landsliding), together with a best estimate of the local build-up of positive water pressure above the persistent, sub-horizontal discontinuity (corresponding to the levels of observed seepages). The results of the analyses are summarised in Figure 33.

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The analyses predict failure, or conditions close to failure, for the postulated failure mechanism and geological model for average suction values in the range of 5 to 15 kPa along the postulated slip surface. Such a range of suction is in line with site measurements and observations. The actual operational shear strength could be different from the shear strength assessed in the laboratory based on small soil samples. This could be due to factors such as the degree of cracking or local weak relic joints within the soil mass and the degree of possible connections of the coreslabs with the rock mass, which have opposing effects on the mass shear strength. The actual groundwater regime would also have been complicated by the possible development of local cleft water pressure within cracks. Overall, the analyses suggest that the postulated simplified geological model corresponds to a potentially credible failure mechanism and that the persistent infilled discontinuity, which formed the basal slip surface, probably played a key role in the failure.

A close examination of the pattern and characteristics of cracks during site mapping identified evidence of local toppling movements (Section 3.5.3). These could have been the result of internal distortion of the jointed ground mass associated with sliding movement along the postulated slip surface, or instability following some form of slope movement (Figure 23). If the coreslabs were connected to the rock mass to a significant degree (Figure 25), it may be possible that a toppling movement mode could develop within the coreslab complex, in association with cleft water pressure and possible reduction in shearing resistance along the persistent infilled discontinuity following water ingress. However, given the significant lateral extent of the southern distressed zone and the uncertainty about the prevalence of sub-surface ground conditions that would tend to favour such toppling instability as the primary failure mode, this may only be a local phenomenon.

The precise mechanism of movement, which will depend heavily on the local ground conditions, cannot be confidently identified based on the available information due to the complexity of the subsurface conditions within this part of the slope. However, the analyses suggest that given the presence of a pre-existing, weak and persistent discontinuity, the movement was probably brought about by water ingress. The analyses further suggest that the instability can be explained without significant build-up in water pressure, which is consistent with the observations of only isolated seepages shortly after the landslide.

8. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

8.1 Trigger of the Landslide

The correlation between the timing of the landslide and the severe rainstorm, together with the absence of a credible alternative source of water ingress (e.g. absence of water-carrying services), suggest that the landslide was triggered by the heavy rainfall preceding the failure.

The 4-day duration rolling rainfall preceding the landslide was one of the highest recorded at the nearest automatic raingauge since its installation in 1982, being comparable to the previous severe rainstorm of July 1997.

Rain ceased for about 12 hours after the main period of the rainstorm that occurred between about mid-day of 22 August and mid-day of 24 August 1999. Rain started to pick

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up again in the early morning of 25 August 1999. Slope distress was first observed at about 8 a.m. on 25 August 1999, although it could have started to develop some time earlier.

8.2 Mode of the Landslide

The northern distressed zone comprised a rotational (sliding) failure with a well-defined main scarp, en echelon tension cracking along the margins of the distressed area, together with significant forward movement of the slope toe and localised collapses of the near-surface slope-forming material. The failure was of limited mobility in that the displaced material did not detach completely from the slope in a fast-moving, uncontrolled manner.

The mechanisms of behaviour of the southern distressed zone were more complex and less readily characterised. Relatively small displacements took place and there was no prominent main scarp. Fairly well-developed en echelon tension cracking occurred along the margins of the distressed zone, together with complex internal distortion and shearing of the displaced ground mass.

The degree of interaction between the northern and southern distressed zones in slope No. 11NW-B/C90 is not certain but is thought to be small in that there is no obvious evidence of a subsequent failure being triggered by an initial significant instability in a retrogressive manner. The instability affected a substantial part of a large cut slope. Different parts of the slope body probably manifested the instability in a different manner, as governed by the local hydrogeological conditions, with relative contributions of the key factors affecting the instability being somewhat different at differing parts of the slope.

8.3 Key Factors Affecting the Landslide

The key factors that need to be considered in assessing the landslide include:

(a) the nature of material involved in the failure and influence of previous landsliding,

(b) sources of water ingress,

(c) slope deterioration, and

(d) safety margin of the slope prior to the August 1999 rainstorm.

The above factors are discussed in the following sections.

8.3.1 Nature of Material Involved in the Failure and Influence of Previous Landsliding

The nature of material involved in a landslide could be influenced by relic landsliding in that the past failure could result in weakening of the ground mass through the development

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of tension cracks and shearing of weak infilled discontinuities, thus adversely affecting mass shear strength.

Extensive pre-existing tension cracks and slickensided surfaces were identified during the investigation, suggesting the possible influence of past instability. Some of the pre-existing tension cracks near the crest of the distressed zones were locally exploited by the slipped mass. Apart from this, the hydrogeology was probably significantly affected by the pre-existing tension cracks because the slope would have become more susceptible to ingress of water, wetting and development of cleft water pressure.

The southern distressed zone incorporated part of a large relic landslide scar which existed prior to cut slope formation, as identified in the earliest available (year 1945) aerial photographs. The current instability exploited a persistent, infilled discontinuity in the CDG which can be associated with this large relic landslide. There is an element of structural control in this part of the landslide in that a significant portion of the slip surface exploited the previously sheared, weak persistent discontinuity.

In the northern distressed zone, the investigation indicates that the slip surface probably exploited discontinuities on a relatively local scale. Also, aerial photographs indicate that this portion of the slope was largely outside the above large relic landslide scar. Tension cracks were identified in the ground mass, but they generally appear to be confined to fairly shallow depths and may not necessarily have a significant effect on the mass shear strength for the relatively deep-seated failure.

The southern part of the slope has a more extreme variation in weathering depth compared to the northern part of the slope. The northern part of the slope has been weathered to greater depths compared to the southern part of the slope where there is an extensive series of coreslabs. The slip surface associated with the northern distressed zone was primarily through the CDG, whereas in the southern distressed zone, any potential slip surface would have been significantly influenced locally by the presence of rock mass.

8.3.2 Sources of Water Ingress

The investigation revealed that there are likely to be two groundwater regimes in slope No. 11NW-B/C90 viz, the upper and lower groundwater regimes (Sections 6.5). The possible sources of water ingress, either separately or in combination, that have been identified are:

(a) direct infiltration into slope No. 11NW-B/C90, particularly through the old drainage lines and the pre-existing tension cracks,

(b) water ingress through uncovered areas, used for unauthorized cultivation, directly above slope No. 11NW-B/C90, followed by subsurface seepage (probably along preferential flowpaths, such as the old drainage lines) towards the cut slope,

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(c) water ingress through the vegetated natural hillside to the north of slope No. 11NW-B/C91 and possibly through the far-field catchment (viz. Beacon Hill), together with water flow in the partially lined open channel (Plate 4) further to the north infiltrating into rock outcrops of slope No. 11NW-B/C582, followed by subsurface seepage (possibly through the fault/fracture zone at the toe of slope No. 11NW-B/C91) towards slope No. 11NW-B/C90 (as evidenced by persistent groundwater seepages from the horizontal drains and isolated joints in the lower rock portion of slope No. 11NW-B/C91), and

(d) water ingress through the active service reservoir construction site at the far side of the hilltop, followed by subsurface seepage.

The groundwater regimes could also have been affected by possible deterioration of the performance of the horizontal drains which were installed in slope No. 11NW-B/C91 in the late 1970's. However, the observation of continued seepages from a fair number of horizontal drains after the 25 August 1999 landslide suggests that the horizontal drains probably have not suffered significant deterioration. Field inspections also confirmed that the horizontal drains were not blocked.

The findings of this detailed investigation have enabled the establishment of the likely relative importance of the various sources of water ingress in causing the failure.

The evidence suggests that water ingress source (d) is unlikely to have been significant. This diagnosis is based on the findings of a comparatively low infiltration rate at the exposed ground surface in the reservoir site (being about 10 times less than other tested areas) and that the pumping test did not identify any positive flowpaths between the reservoir site and the area further to the south, which was corroborated by a very low inflow rate (about 35 l/hr) during the pumping test. The above is also consistent with the absence of any significant subsurface erosion pipes during bulk excavation at the reservoir site.

The extensive voiding beneath the chunam cover is probably a result of periodic slope movement, displacement of the chunam cover due to growth of tree roots, or erosion caused by water flow. These voids have a significant effect on water ingress in that water passing through cracks in the chunam, or through areas of unplanned vegetation, would probably have resulted in direct infiltration over a correspondingly much more extensive area beneath the chunam cover. This postulation is supported by the observation of extensive tree root growth in the soil under much of the chunam cover (Plate 17), which reflects the general ineffectiveness of the protective surface cover at this slope. Overall, the site setting was vulnerable to surface infiltration during rainfall. Therefore, direct infiltration resulting in wetting up of the ground mass is likely to have played a key role in affecting the groundwater regimes.

For the upper groundwater regime, source (a) is likely to have been very significant, source (b) could have some contribution, whilst the contribution of source (c) is uncertain.

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As for the lower groundwater regime for the northern distressed zone, source (c) is likely to have been very significant, source (a) is likely to have some contribution, whilst source (b) is unlikely to have any contribution.

8.3.3 Slope Deterioration

There is evidence of progressive slope deterioration since about 1992 (see Appendix A). Slope deterioration was manifested by the proliferation of unplanned vegetation (particularly along or close to the old drainage lines and along the berms), disrupted chunam cover, disrupted drainage channels, etc.

The presence of extensive pre-existing tension cracks in the slope-forming material beneath the surface cover rendered the slope vulnerable to loss of soil strength arising from infiltration via local defects in the surface protection (e.g. cracks, vegetation growth and voids beneath the cover). Progressive deterioration as a result of lack of maintenance probably rendered the slope more vulnerable to failure.

The August 1999 rainstorm was comparable to the severe rainstorm in July 1997 (Figure 14), when no major instability similar to that of 25 August 1999 was noted at that time. This also suggests that the slope may have been subjected to deterioration such that the slope became more vulnerable to the effects of rainfall.

8.3.4 Safety Margin of the Slope Prior to the August 1999 Rainstorm

The current investigation has established that the stability of the slope prior to the severe rainstorm in August 1999 was marginal at times of heavy rainfall, as corroborated by the pre-existing tension cracks in the slope and observations of slope distress during previous inspections (Sections 4.3 and 4.4). Given its marginal stability, the slope was sensitive to progressive deterioration.

Slope No. 11NW-B/C90 was previously assessed under area geotechnical studies. Only very limited slope works (mainly local replacement of the surface cover and provision of weepholes) were subsequently carried out and the basic form of the cut slope had not been substantially modified since its original formation. It can be inferred from the past assessments that the slope was considered adequately safe on the premise that pore water suction could be sustained in the ground mass. Given the presence of voids beneath the chunam cover, pre-existing tension cracks and old drainage lines in the ground, the site setting is not favourable in so far as reliance on suction for long-term slope stability is concerned.

For the southern distressed zone, the presence of persistent sub-horizontal discontinuities with weak infill was not appreciated in the previous slope assessments. These past assessments were subjected to the limitation that no site-specific ground investigations were carried out at the concerned slope. The above geological features, which are not commonly encountered in the region with the degree of lateral continuity observed here, contributed to lowering the safety margin of the slope as they provided weak planes in the ground mass as well as an unfavourable hydrogeological setting.

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8.4 Causes and Mechanism of the Landslide

The principal cause of the landslide is attributed to the wetting up of the ground mass and elevation in groundwater pressure following severe rainfall, which is consistent with the supporting theoretical slope stability analyses (Section 7).

In the northern distressed zone, site observations of seepage levels, together with the supporting sensitivity analyses, suggest that failure was probably brought about by the combined effect of wetting up of the soil profile above rockhead (primarily through direct infiltration, resulting in reduction in suction and build-up of local water pressure within the soil mass and in joints), and rise in the base groundwater table.

In the southern distressed zone, the hydrogeological setting, together with the observation of localised seepages above the persistent discontinuity (Section 3.5), suggest that the effect of water ingress on the upper groundwater regime in the soil profile was primarily responsible for the movement.

The mode of deformation in the southern distressed zone (i.e. primarily slope movements with no well-developed scarps) and the supporting theoretical analyses suggest that there was a small margin against overall slope failure. Ground movements within the slope appear to have developed over many years, probably at times of intense rainfall. These movements result from reductions in suction, and hence strength, coupled with localised increases in cleft water pressures. They take the form of relatively small, irreversible sliding displacements along discontinuities, including en echelon sliding between core slabs.

8.5 Debris Mobility

The low mobility of the northern slope failure is related to a number of factors. These include the relatively deep-seated and large-scale nature of the slide, the release of cleft water pressures during displacement due to dilation and opening of pre-existing cracks, the buttressing effect of the rocky bund at the toe of the central portion of the slope and the more stable configuration of the slope after movement. For the southern distressed zone, the internal displacements took place along slickensided discontinuities so that the behaviour is essentially ductile.

9. CONCLUSIONS

It is concluded that the 25 August 1999 landslide at slope No. 11NW-B/C90 in Shek Kip Mei was probably caused by the build-up of adverse transient groundwater conditions following the prolonged, severe rainfall that preceded the failure. The rainfall was probably amongst the worst rainstorms experienced by the slope since its formation in the 1950's and was comparable with that of 1997.

The northern distressed zone of the slope comprised a deep-seated rotational failure of low mobility. The southern distressed zone underwent relatively small ground movements of a complex nature involving internal distortions and displacements of the ground mass. The degree of interaction between the two zones is uncertain but is thought to be small.

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The adverse groundwater conditions resulted principally from water ingress via surface infiltration through defects in the chunam cover and subsurface seepage from the regional groundwater flow regime in the bedrock. Surface infiltration through the unauthorized cultivation area above the cut slope could also have some contribution by allowing water ingress into the slope.

Water ingress due to surface infiltration is likely to have led to reduction in suction in the ground mass, development of cleft water pressures in pre-existing tension cracks and elevated water pressures in discontinuities. The rise in the base groundwater table in the northern part of the slope was predominantly a result of the regional groundwater flow regime in the bedrock, although surface infiltration through the cracked chunam cover probably had some contribution as well.

The following unfavourable factors were identified:

(a) the slope is situated partly within the area of a large relic landslide, and formerly of deep erosion,

(b) the presence of persistent adversely orientated discontinuities with continuous weak infill, whose degree of lateral continuity (more than 60 m) is not commonly observed in Hong Kong,

(c) progressive deterioration of the slope condition,

(d) presence of pre-existing tension cracks and old drainage lines renders the slope vulnerable to loss of soil strength arising from infiltration via local defects in the slope surface cover, and

(e) unauthorized cultivation above the slope, which probably resulted in increased surface infiltration.

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

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	8.5	05:15 on 23.8.99	<2
15 minutes	24	05:55 on 23.8.99	<2
1 hour	59	06:05 on 23.8.99	<2
2 hours	83	06:20 on 23.8.99	<2
4 hours	117	07:20 on 23.8.99	<2
12 hours	178	11:55 on 24.8.99	<2
24 hours	296	11:55 on 24.8.99	5
48 hours	514	11:35 on 24.8.99	23
4 days	641	07:50 on 25.8.99	31
7 days	641	07:50 on 25.8.99	17
15 days	756	07:50 on 25.8.99	12
31 days	986	08:00 on 25.8.99	13

Notes:
(1) Return periods were derived from Table 3 of Technical Note No. 86, using Gumbel's Equation (Lam & Leung, 1994).
(2) Maximum rolling rainfall was calculated from 5-minute data.
(3) The use of 5-minute data for durations between 2 hours and 31 days results in better data resolution but may slightly over-estimate the return periods using Lam & Leung (1994)'s data, which are based on hourly and daily rainfall for these durations.
(4) Assumed time of failure at 08:00 on 25 August 1999.

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Table 2 - Summary of Classification and Index Test Results

Sample Location	Depth (m)	Material Type	Simple Type	Particle Size Distribution				Atterberg Limits		
				Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL(%)	PL(%)	PI(%)
AP1	0.6 to 0.9	CDG	Bulk	47	47	4	2	-	-	-
AP1	0.8	CDG	Bulk	37	51	8	4	-	-	-
AP1	1.3	CDG	Bulk	38	50	8	4	-	-	-
AP2	0.8	CDG	Bulk	45	46	5	4	-	-	-
AP2	1.1	CDG	Bulk	42	38	12	8	-	-	-
AP4	1.3 to 1.6	CDG	Block	22	60	12	6	49	34	15
AP4	1.3 to 1.6	CDG	Block	13	52	23	12	47	29	18
AP4	1.3 to 1.6	CDG	Block	30	52	11	7	56	34	22
AP6	0.9 to 1.2	CDG	Block	37	44	12	7	55	33	22
AP6	0.9 to 1.2	CDG	Block	12	34	29	25	46	28	18
AP6	0.9 to 1.2	CDG	Block	32	51	11	6	48	30	18
AP6	1	CDG	Bulk	18	50	19	13	-	-	-
AP7	0.4 to 0.7	CDG	Block	38	47	8	7	52	29	23
AP7	0.4 to 0.7	CDG	Block	30	55	9	6	43	28	15
TP2A	2.2 to 2.5	CDG	Block	32	61	5	2	-	-	-
TP2A	2.7 to 3	CDG	Block	35	58	5	2	-	-	-

Legend:

LL	Liquid Limit	CDG	Completely decomposed granite
PL	Plastic Limit	AP1	Trail pit No. AP1
PI	Plasticity Index		

Note: See Figure 17 for the locations of samples.

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Table 3 - Triaxial Compression Test Results for Completely Decomposed Granite

Sample Location	Depth (m)	Material Type	Sample Type	Moisture Content before Testing (%)	Dry Density before Testing (Mg/m^3)	Particle Density (Mg/m^3)	Type of Test	p' (kPa)	q (kPa)
DH3	2.38 to 2.58	CDG	Mazier	26	1.483	2.61	CUM	57.1 98.6 255.7	38.6 64.3 154.3
DH3	5.39 to 5.59	CDG	Mazier	28.5	1.435	2.61	CUM	90 182.9 320	61.4 120 195.7
DH3	8.77 to 8.97	CDG	Mazier	28	1.447	2.62	CUM	132.9 242.1 405	90 156 237.9
DH4	0.88 to 1.08	CDG	Mazier	22.4	1.529	2.62	CUM	84.3 118.6 201.4	63 82.9 124.3
DH4	1.98 to 2.18	CDG	Mazier	24	1.487	2.62	CUM	81.4 148.6 277.1	60 101.4 171.4
DH4	6.15 to 6.35	CDG	Mazier	22.4	1.477	2.61	CUM	94.3 237.1 488.6	68.6 165.7 311.4
DH4	10.84 to 11.04	CDG	Mazier	17.6	1.647	2.61	CUM	196.4 389.3 717.9	142.9 260.7 450
DH4	14.15 to 14.35	CDG	Mazier	19.3	1.567	2.61	CUM	168.6 345.7 605.7	117.1 234.3 380
DH5	3.08 to 3.28	CDG	Mazier	32.8	1.363	2.63	CUM	47.1 80 166.4	30 48.6 97.9
DH5	6.16 to 6.36	CDG	Mazier	26.9	1.461	2.63	CUM	70.7 158.6 400.7	47.1 107.1 244.3
DH8	2.28 to 2.48	CDG	Mazier	30.4	1.395	2.63	CUM	84.3 137.1 252.9	60 90 157.1
DH8	3.38 to 3.58	CDG	Mazier	38.4	1.283	2.62	CUM	50 92.9 192.8	32.9 60 120

Legend:

CDG Completely decomposed granite
CUM Consolidated undrained multi-stage triaxial compression test
DH3 Vertical drillhole No. DH3
$p' = \frac{1}{2}(\sigma_1' + \sigma_3')$
$q = \frac{1}{2}(\sigma_1' - \sigma_3')$, where σ_1' and σ_3' are the major and minor principal effective stresses respectively

Notes: (1) See Figure 17 for the locations of samples.
(2) All samples of CDG extracted for the triaxial compression tests were outside the distressed zone of the slope.

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Table 4 - Results of Direct Shear Box Tests on Slickensided Relict Discontinuities Infilled with Kaolin and Manganese Oxide Deposits

Sample Location	Depth (m)	Material Type along Shearing Plane	Sample Type	Direction of Shearing	Vertical Stress at the Beginning of Test (kPa)	Maximum Shear Stress (kPa)	Horizontal Displacement at Maximum Shear Stress (mm)	Vertical Displacement at Maximum Shear Stress (mm)	Vertical Stress at Maximum Shear Stress (kPa)
TT6	1	Discontinuity with kaolin and manganese oxide deposits	Block	Along slickenside	20	8.85	1.48	-0.02	20.39
TT6	1	Discontinuity with kaolin and manganese oxide deposits	Block	Along slickenside	30	11.11	0.66	-0.04	30
TT6	1	Discontinuity with kaolin and manganese oxide deposits	Block	Along slickenside	35	12.02	3.26	-0.51	37.08
TT6	1	Discontinuity with kaolin and manganese oxide deposits	Block	Along slickenside	40	16.46	1.40	-0.08	41.66
TT6	1	Discontinuity with kaolin and manganese oxide deposits	Block	Along slickenside	45	15.28	0.48	-0.06	45.33

Legend:

TT6 Trial trench No. TT6

Notes:
(1) See Figure 17 for the locations of samples.
(2) The rate of shearing was 0.08 mm/min.
(3) The positive vertical displacement denotes dilation and a negative vertical displacement denotes compression.

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Table 5 - Results of Double-ring Infiltration Tests

Infiltration Test No.	Location	Steady-state Infiltration Capacity (mm/hr)
IT1	WSD construction site	0.45
IT2	WSD construction site	0.45
IT3	WSD construction site	2.25
IT4	Cultivation area	24.3
IT5	Cultivation area	28.8
IT6	Old drainage line	39.5

Notes: (1) See Figure 17 for the locations of infiltration tests.
(2) The infiltration capacity corresponds to the condition with water being maintained at a depth of 25 mm above the soil surface subject to infiltration.

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Table 6 - Results of Percolation Tests

Test No.	Location	Time for 150 mm Depth of Water to Drain Away at Steady-state (min)	Rate of Drop in Water Level at Steady-state (mm/hr)
PT1	Old drainage line	15	600
PT2	Cultivation area above the distressed slope	34	265
PT3	Old drainage line	43	209
PT4	Cultivation area above the distressed slope	45	200
PT5	Old drainage line	64.5	140
PT6	Old drainage line	20	450
PT7	Disrupted chunam above the distressed slope	28	321
PT8*	WSD construction site	Water level dropped 36 mm in 400 min	5.4
PT9*	WSD construction site	Water level dropped 91 mm in 185 min	29.5
PT10	Natural hillside to the north of slope No. 11NW-B/C91	62	145
PT11	Natural hillside to the north of slope No. 11NW-B/C91	94	96
PT12	Natural hillside to the north of slope No. 11NW-B/C91	80	113

Notes:
(1) See Figure 17 for the locations of percolation tests.
(2) All percolation tests were carried out inside trial pits of size 1.5 m x 1.5 m x 0.5 m depth.
(3) The rate of the drop in water level for test marked* was very slow in that the 150 mm depth of water did not fully drain away during the test.

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Note: Base Map is extracted from Survey Sheet No. 11NW-B

Figure 1 - Location Plan

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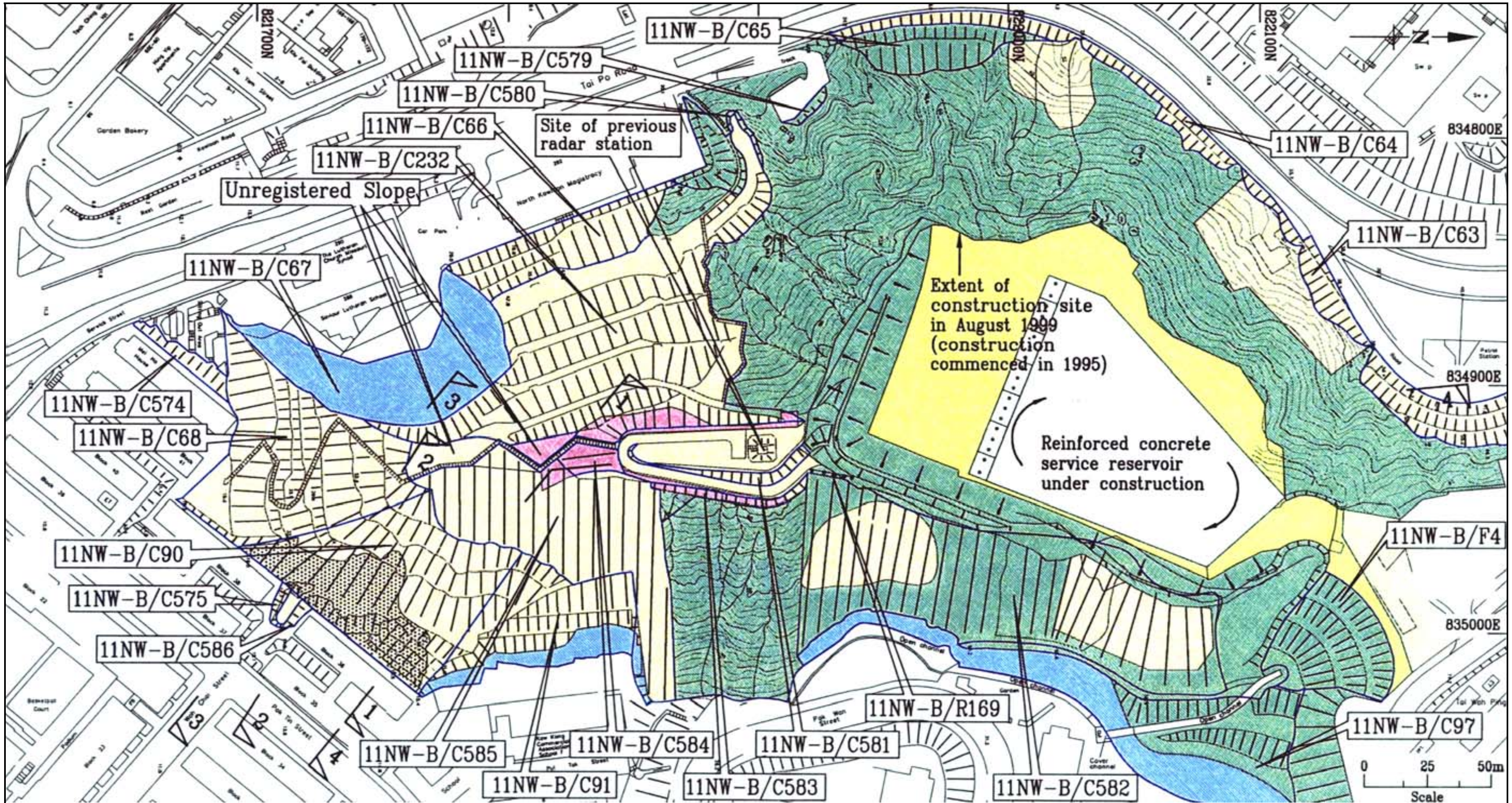

Legend:

11NW-B/C90	Registered feature number		Distress area		Rock Slope		Slope covered by vegetation
	Slope boundary		Surface with rigid cover		Exposed soil surface (here)		Cutivation area

Note: Base Map is extracted from 1:1000 scale Survey Sheet No. 11NW-9C.

Figure 2 - Site Plan

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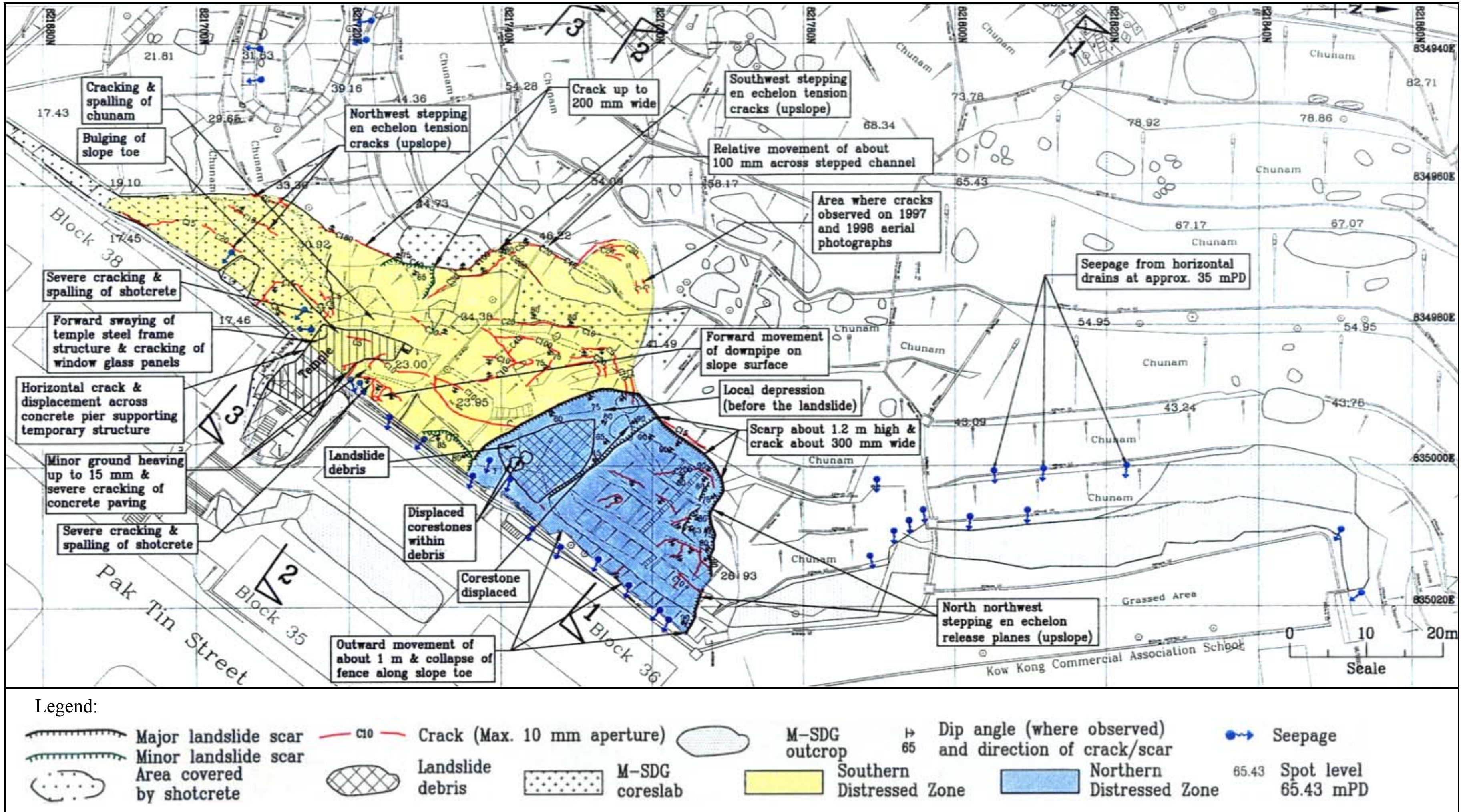

Notes: (1) Base map is based on topographic survey carried out in September and October 1999.
(2) Observations on the distress and seepage locations were made by FMSW betweeen 26 August and 10 September 1999.
(3) The chunam cover within the distressed areas was severely cracked and spalled.
(4) M-SDG coreslabs are tabular and laterally extensive bodies of slightly to moderately decomposed granite.

Figure 3 - Plan of the Landslide

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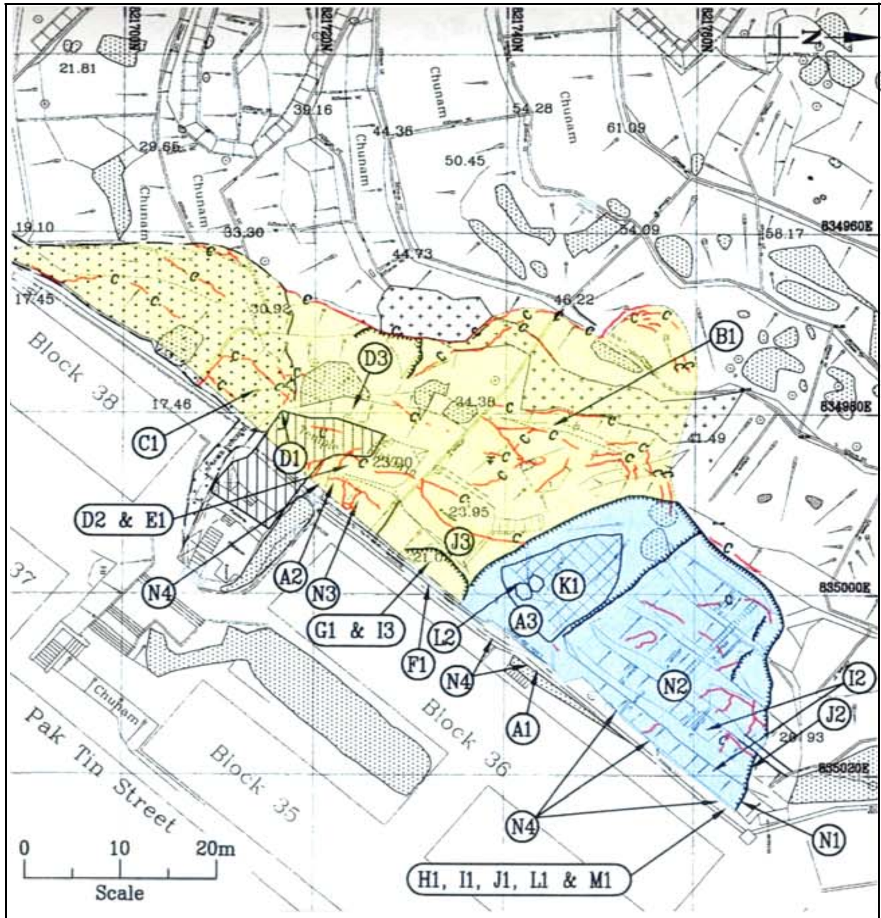

	Observation	Time of Observation
A1	Perennial seepage from the slope toe	General observations over the past years
A2	Seepage from the slope toe	
A3	Ponding after heavy rainfall	
B1	Corestone of 2 m^3 fell from the slope	6 October 1995
C1	Cracking of the shotcrete surface	About June 1997
D1	Minor bulging of the slope toe	About July & August 1997
D2	Cracking of the concrete pavement	
D3	Cracking of chunam	
E1	Ground heaving (maximum 15 mm) along the old crack mentioned in D2	First noted about 8:00 a.m. 25 August 1999 (probably occurred between 7:00 p.m. of 24 & 8:00 a.m. of 25 August 1999)
F1	Seepage from the slope toe	About 10:00 a.m. 25 August 1999
G1	1 m diameter scar on the slope	About 11:00 a.m. 25 August 1999
H1	Minor heaving of the concrete pavement and slight tilting of the fence along the slope toe	About 4:00 p.m. 25 August 1999
I1	The ground heaving mentioned in H1 increased to about 150 mm	About 4:30 p.m. 25 August 1999
I2	Cracking of the chunam surface	
I3	The scar mentioned in G1 enlarged to about 2 m x 2 m x 1 m depth	
J1	The ground heaving mentioned in H1 & I1 increased to about 300 mm	About 5:00 p.m. 25 August 1999
J2	A few pieces of chunam surface fell down from the slope	
J3	Abnormal noise 'King Kong, King Kong' from the slope	
K1	A loud noise from the slope & landslide debris of wet soil and boulders at slope toe	About 5:10 p.m. 25 August 1999
L1	The fence mentioned in H1 tilted to about 45°	About 6:20 p.m. 25 August 1999
L2	Two boulders (about 1 m diameter each) fell down from the slope with soil debris	
M1	The fence mentioned in H1 collapsed on the ground	About 7:30 p.m. 25 August 1999
N1	Failure scar of about 3 m x 2 m x 1 m depth at the toe of the northern end of the slope	About 8:00 p.m. 25 August 1999
N2	Severe cracking and bulging of the chunam surface	
N3	Severe cracking and bulging of the chunam surface	
N4	Significant seepage along the slope toe	

Legend:

Landslide scar mapped on 26, 27 & 28 August 1999
Area covered by shotcrete
M-SDG outcrop
M-SDG coreslab
65.43 Spot level 65.43 mPD
Crack mapped during 26 to 31 August 1999
Landslide debris noted on 26 August 1999
Southern Distressed Zone
Northern Distressed Zone
(A1) Location of observation

Note: Observations shown in this figure were based on information provided by witnesses.

Figure 4 - Key Observations Made on and before 25 August 1999

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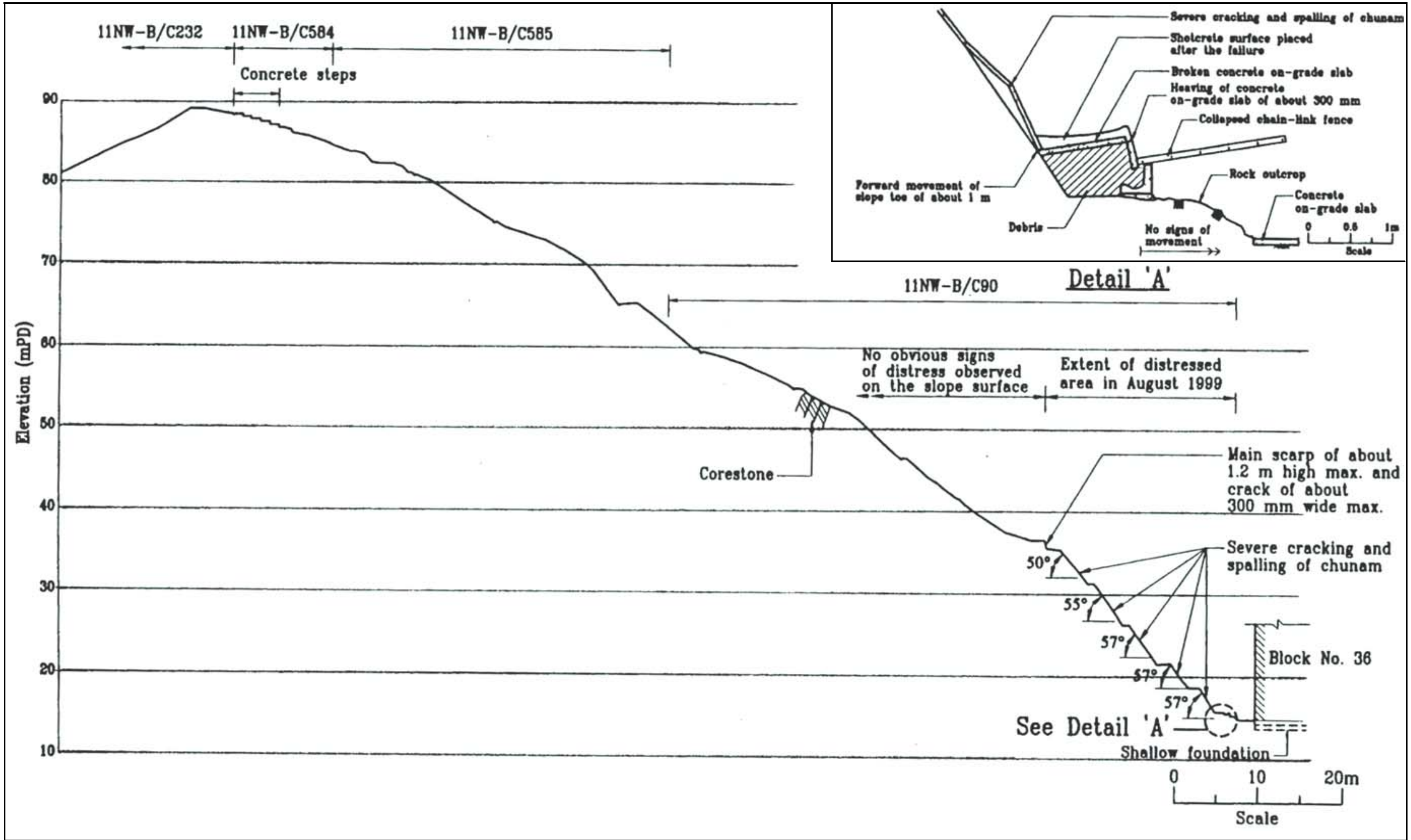

Note: See Figure 2 for location of section line.

Figure 5 - Topographic Section 1-1 through the Northern Distressed Zone

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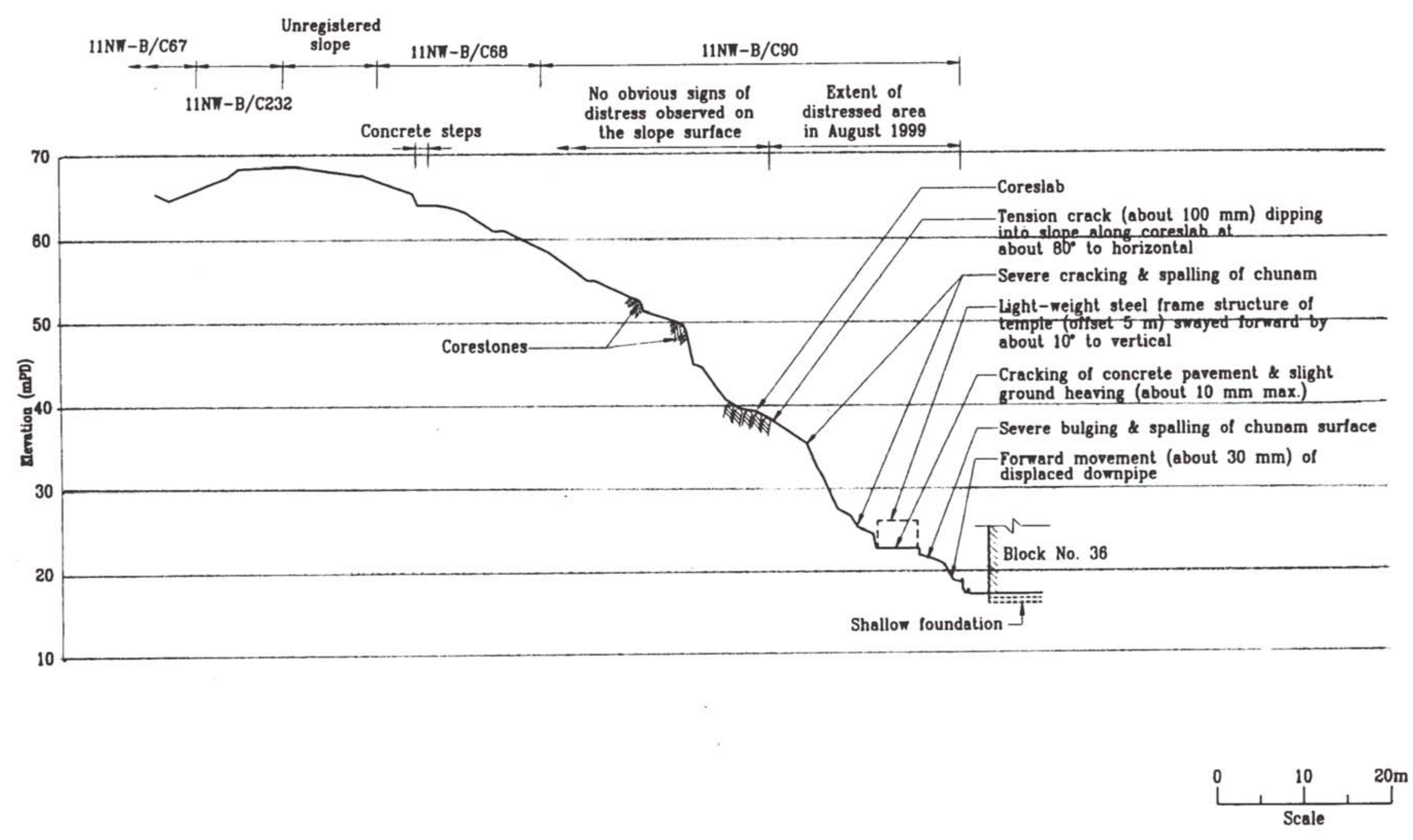

Note: See Figure 2 for location of section line.

Figure 6 - Topographic Section 2-2 through the Southern Distressed Zone

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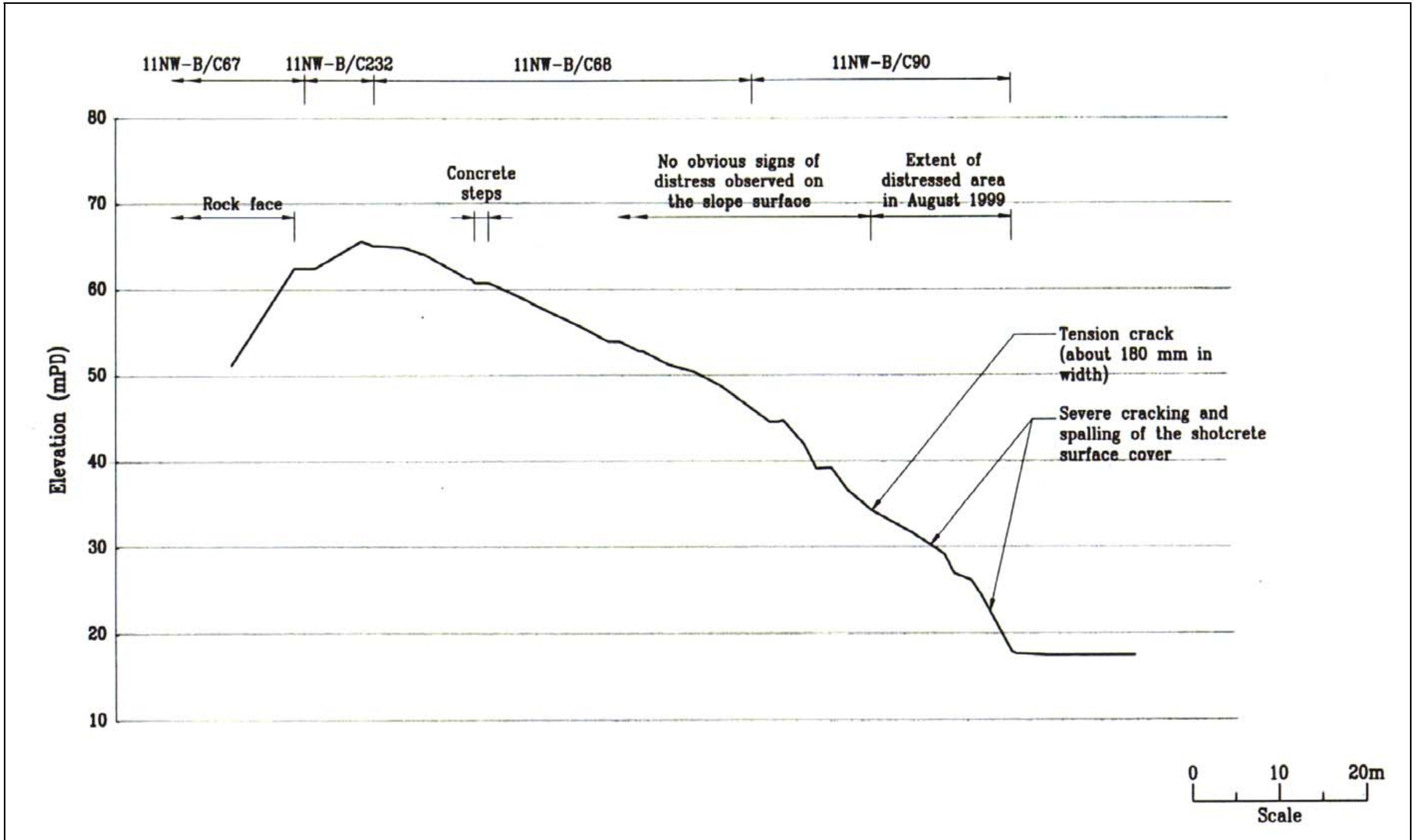

Note: See Figure 2 for location of section line.

Figure 7 - Topographic Section 3-3 through the Southern Distressed Zone

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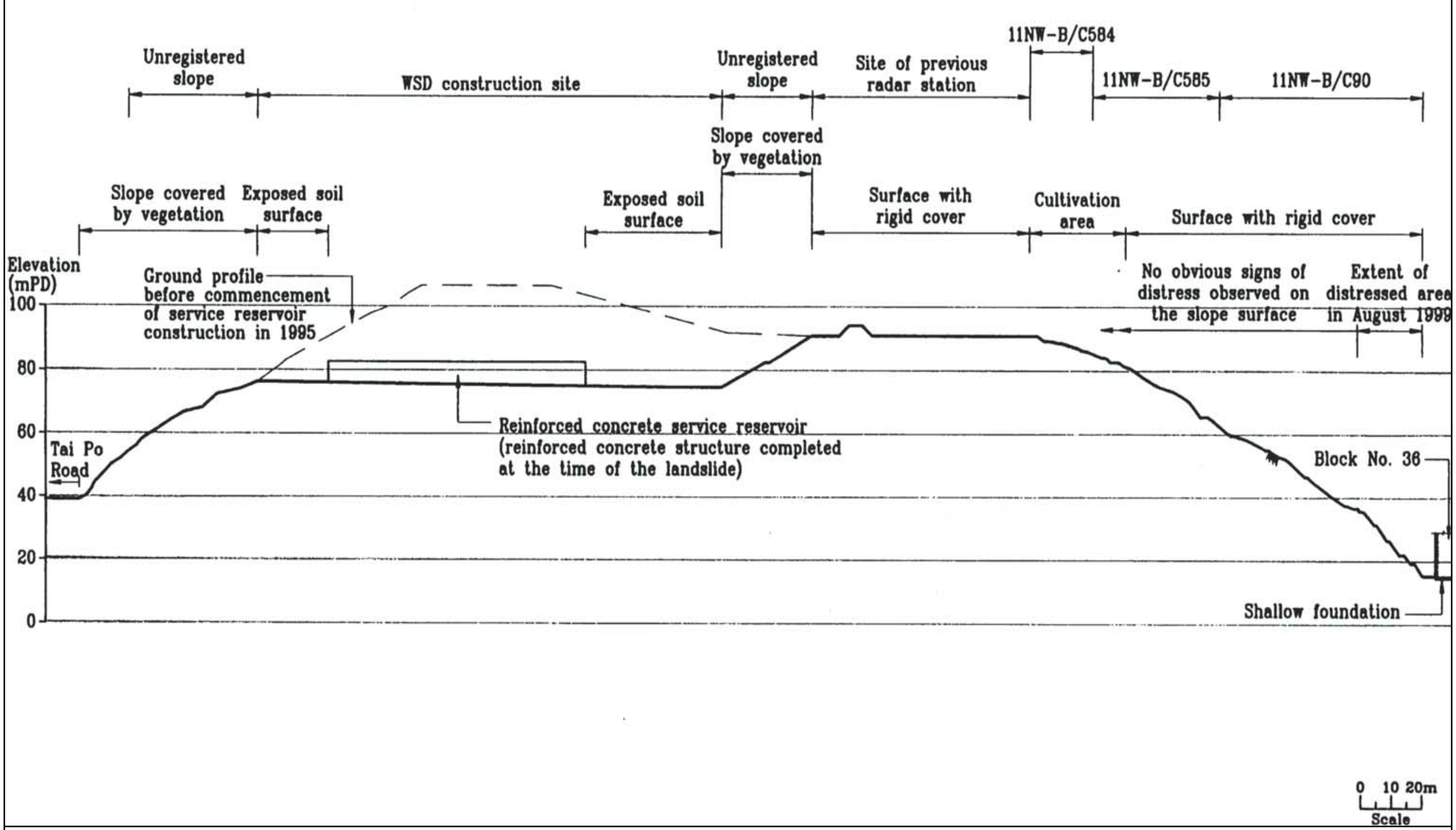
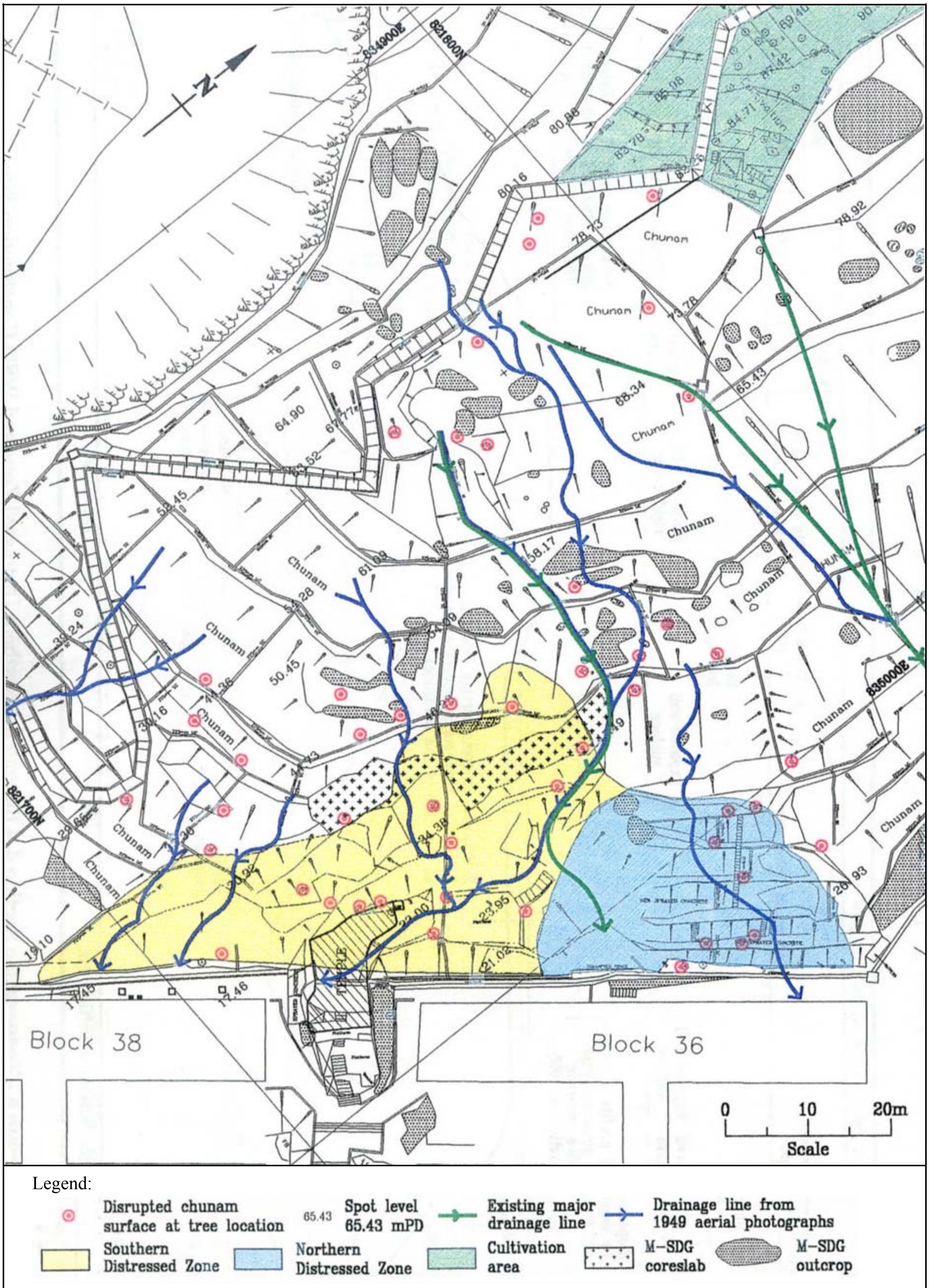


Note: See Figure 2 for location of section line.

Figure 8 - Topographic Section 4-4 through the Northern Distressed Zone and WSD Construction Site



Note: The chunam cover within the distressed areas was severely cracked and spalled.

Figure 9 - Major Drainage Lines Within and Above the Distressed Slope

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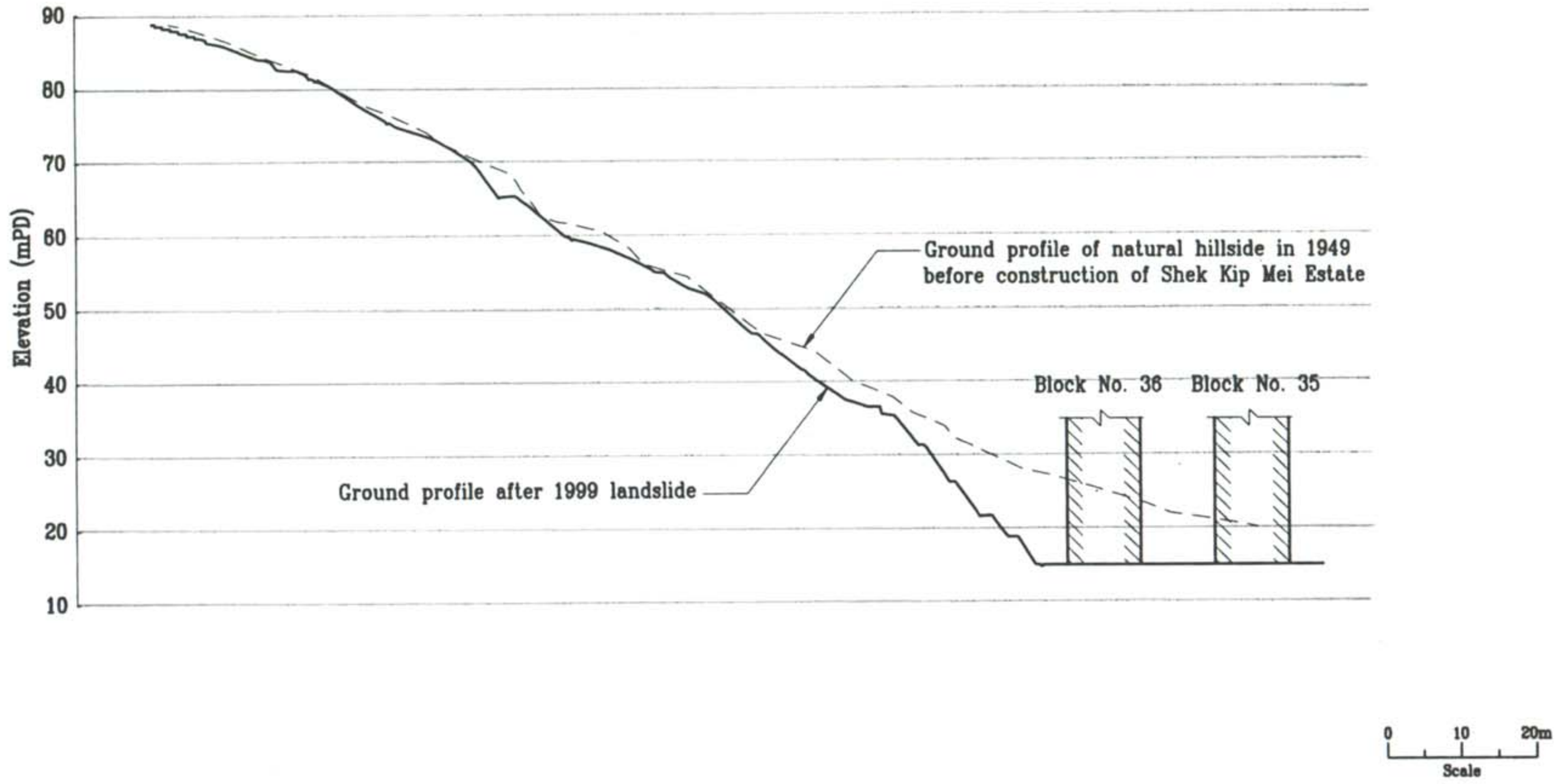

Notes: (1) See Figure 2 for location of section line.

(2) Ground profile before construction of Shek Kip Mei Estate was determined by photogrammetric survey based on aerial photographs Nos. 6009 and 6010 taken on 19 May 1949.

Figure 10 - Formation of Slope Profile at Section 1-1

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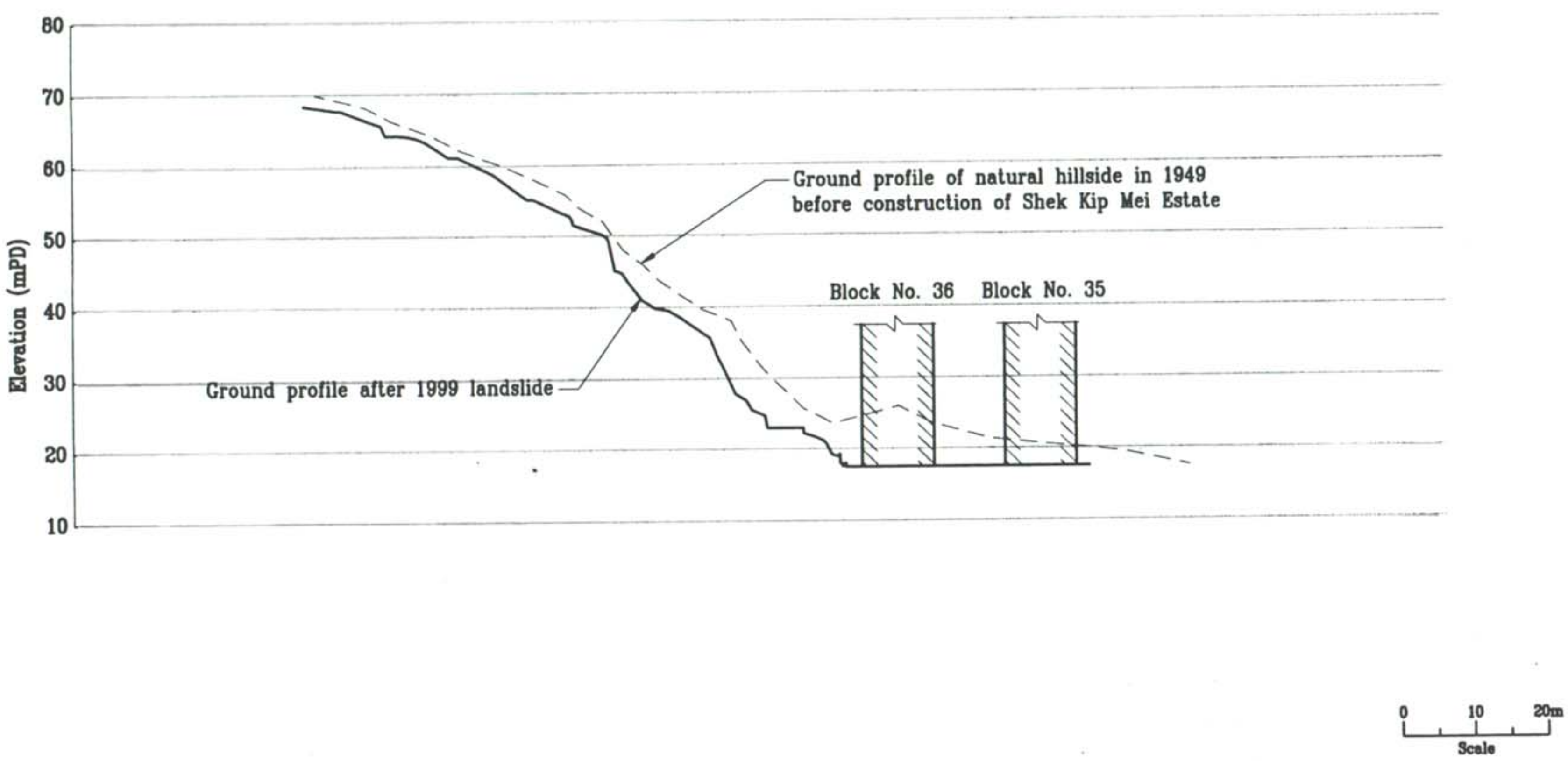
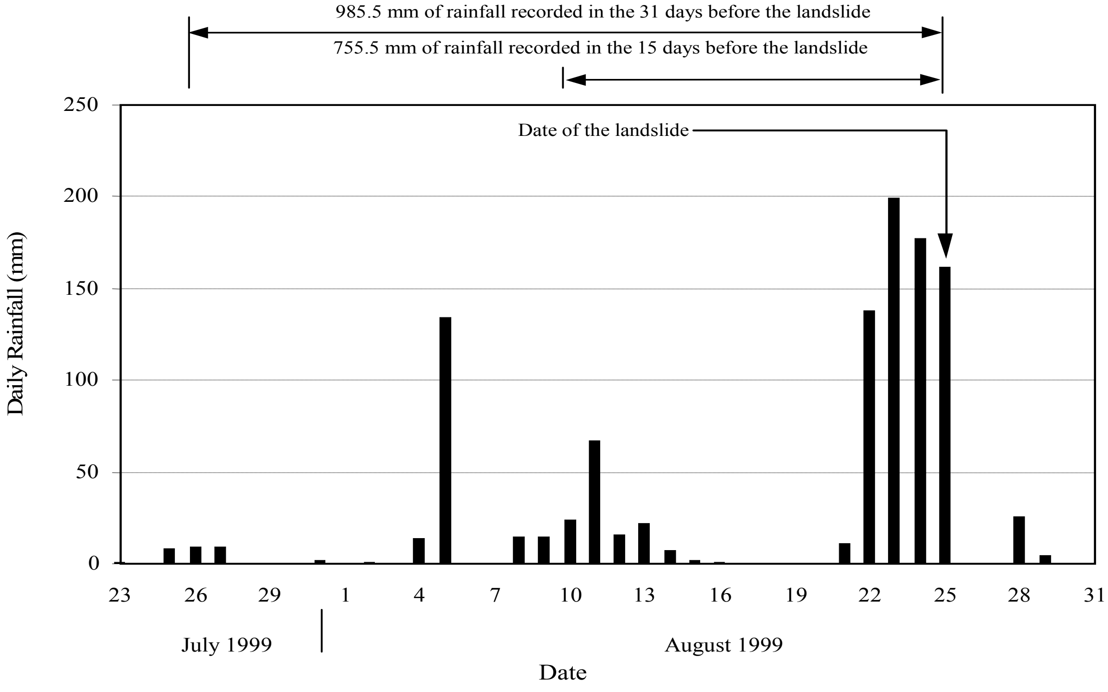
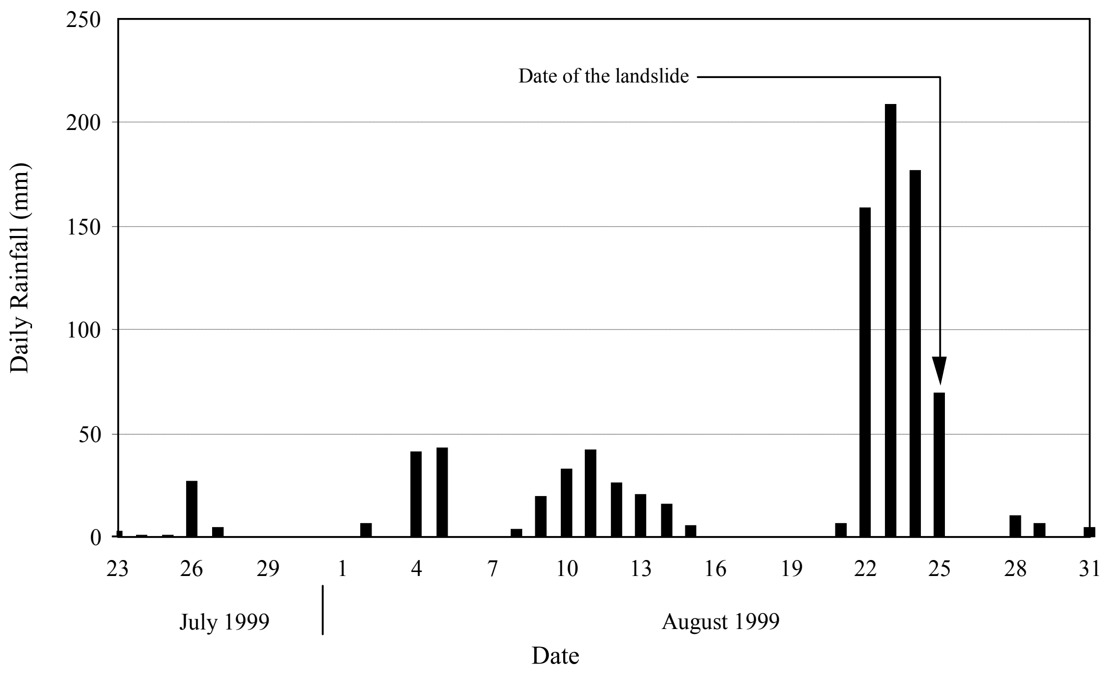


Notes: (1) See Figure 2 for location of section line.
(2) Ground profile before construction of Shek Kip Mei Estate was determined by photogrammetric survey based on aerial photographs Nos. 6009 and 6010 taken on 19 may 1949.

Figure 11 - Formation of Slope Profile at Section 2-2



(a) Daily Rainfall Recorded between 23 July 1999 and 31 August 1999 at GEO Raingauge No. K06



(b) Daily Rainfall Recorded between 23 July 1999 and 31 August 1999 at HKO Raingauge No. R01

Figure 12 - Daily Rainfall Records of GEO Raingauge No. K06 and HKO Raingauge No. R01

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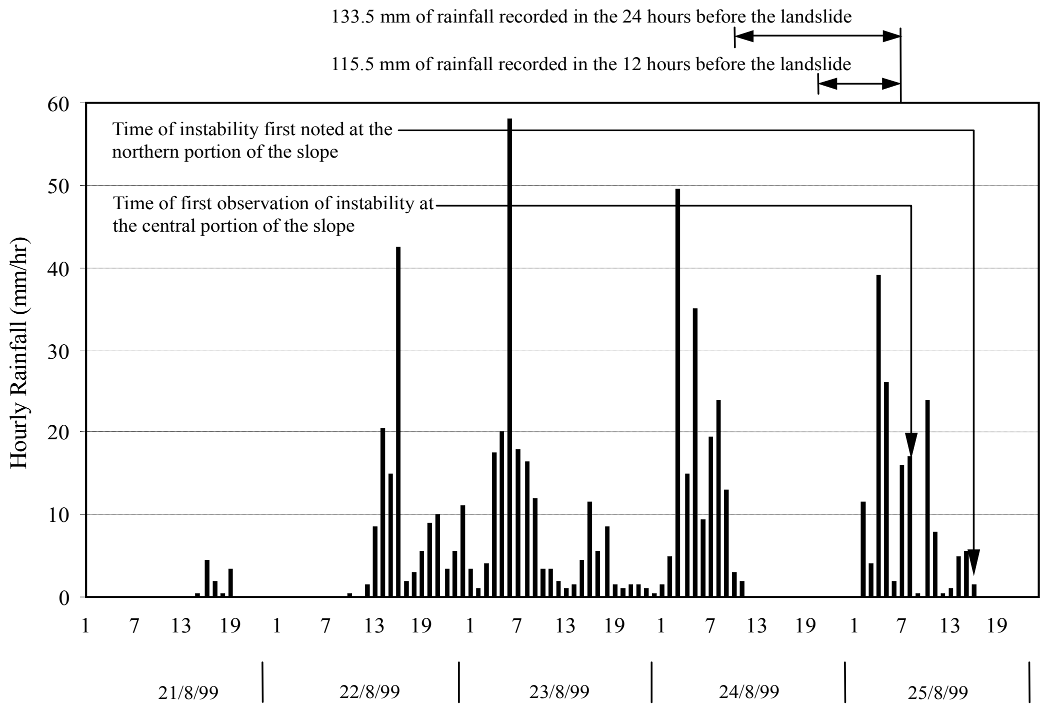
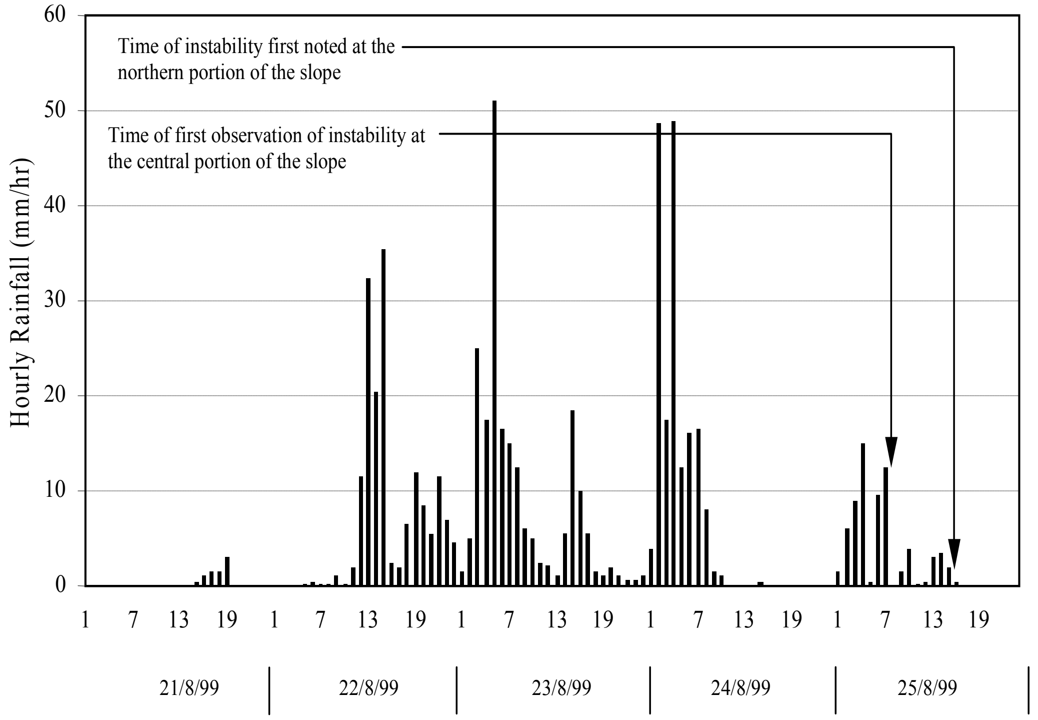


(a) Hourly Rainfall Recorded between 21 and 25 August 1999 at GEO Raingauge No. K06



(b) Hourly Rainfall Recorded between 21 and 25 August 1999 at HKO Raingauge No. R01

Figure 13 - Hourly Rainfall Records of GEO Raingauge No. K06 and HKO Raingauge No. R01

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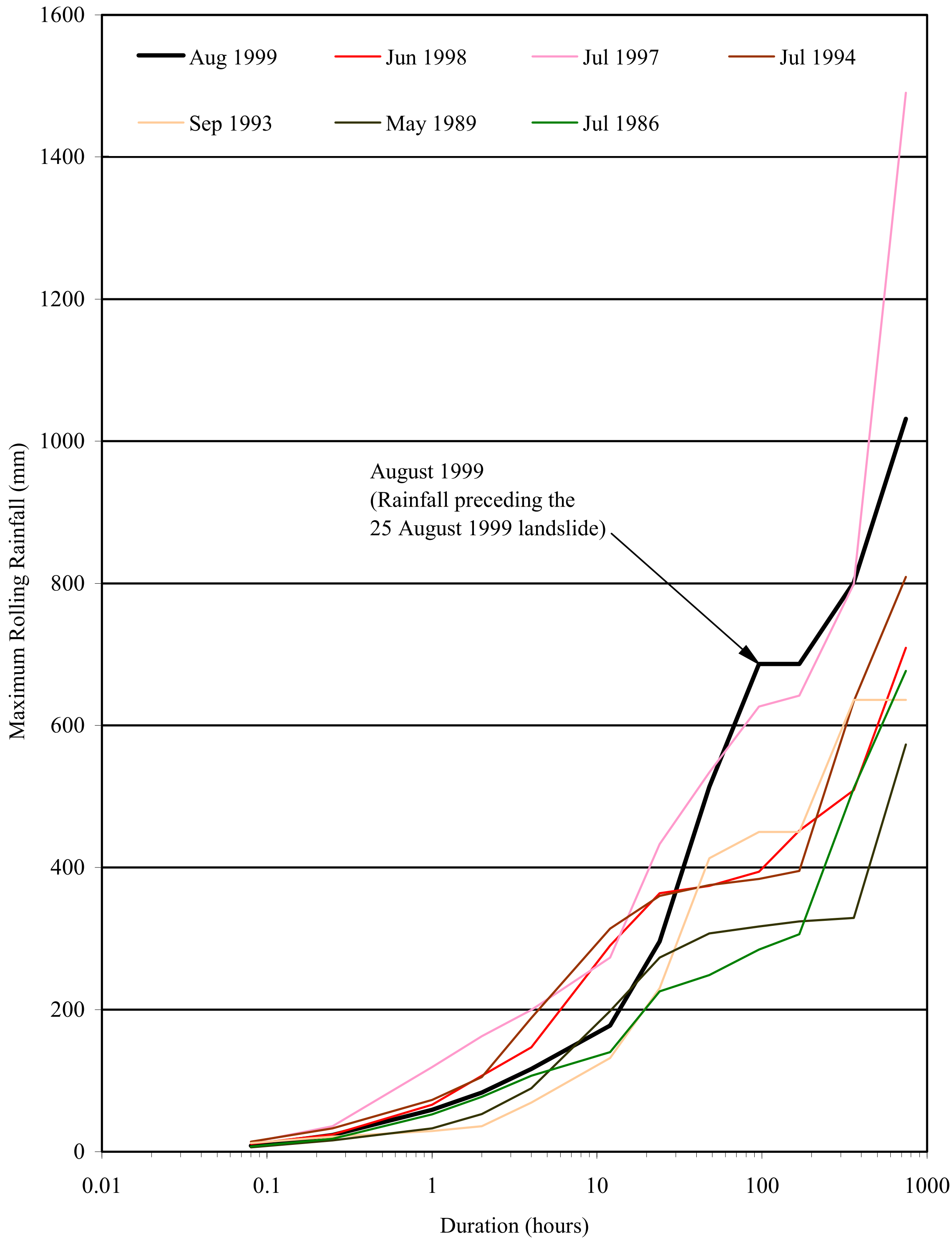

Note: Rainfall data are taken from 5-minute rainfall database maintained by GEO.

Figure 14 - Maximum Rolling Rainfall at GEO Raingauge No. K06 for Major Rainstorms between 1982 and 1999

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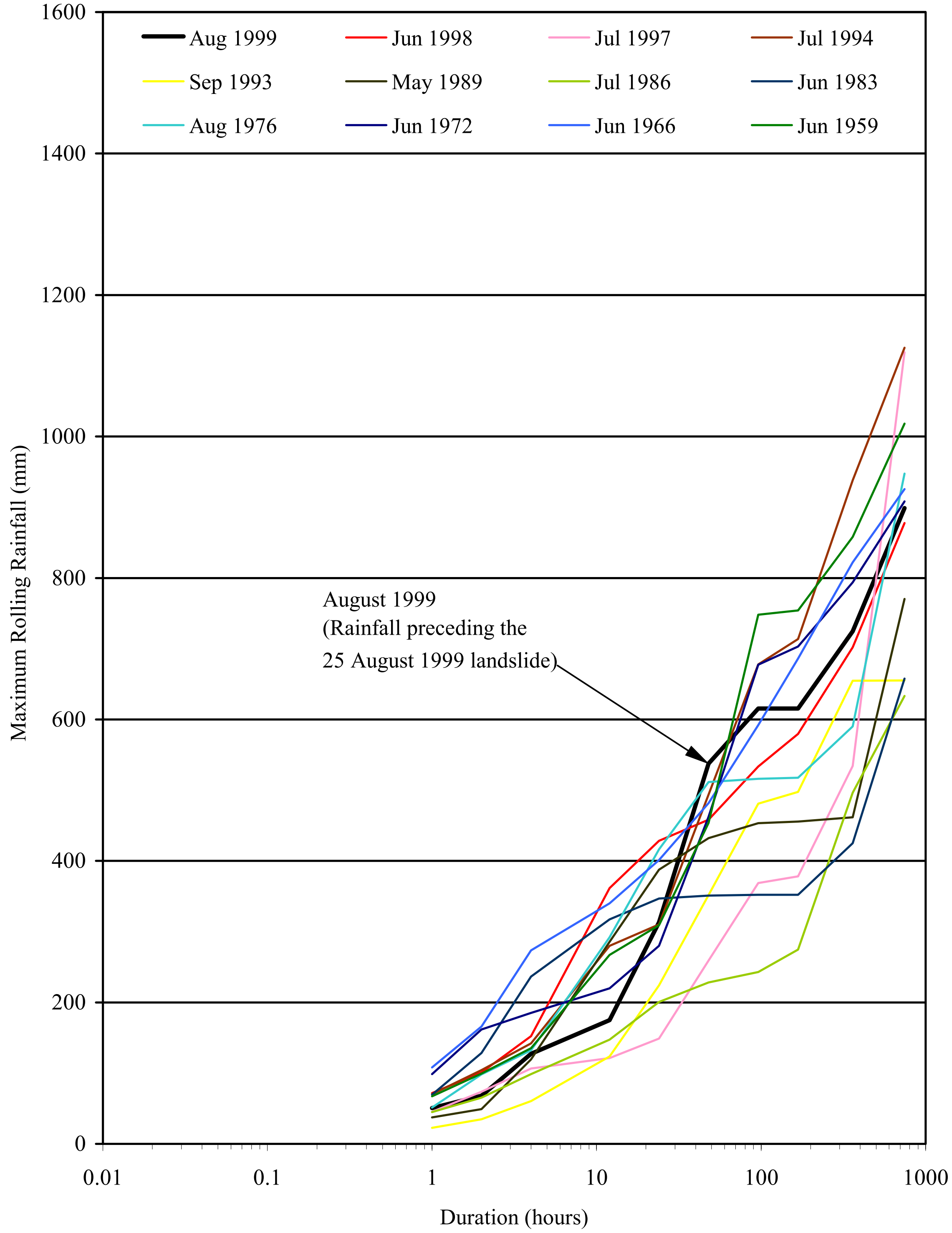

Note: Rainfall data are taken from 1 hour rainfall database maintained by the Hong Kong Observatory.

Figure 15 - Maximum Rolling Rainfall at HKO Raingauge No. R01 for Major Rainstorms between 1959 and 1999

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Legend

BH1	Drillhole by Bachy Soletanche Group (1998)	BH1	Drillhole by Lam Geotechics Ltd. (1988)
BH1	Drillhole by Enpack (HK) Ltd. (1997)	SKM-2	Drillhole by Enpack (HK) Ltd. (1977)
BH-1	Drillhole by Bachy Soletanche Group (1993)		

Note: See Map is extracted from 1:1000 scale Survey Sheet No. 11NW-9C.

Figure 16 - Plan of Pre-1999 Ground Investigation

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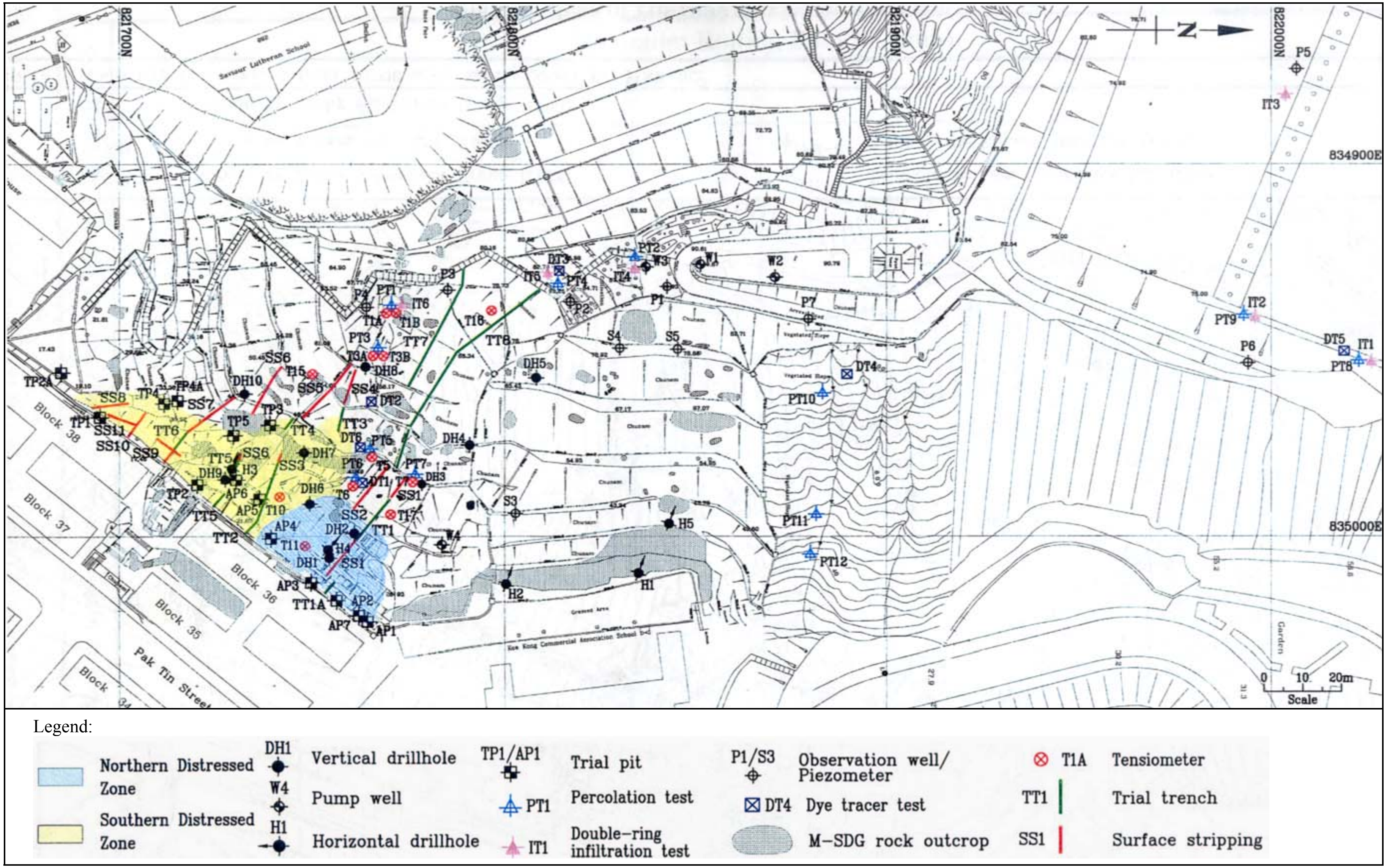

Figure 17 - Plan of Ground Investigation after the Landside

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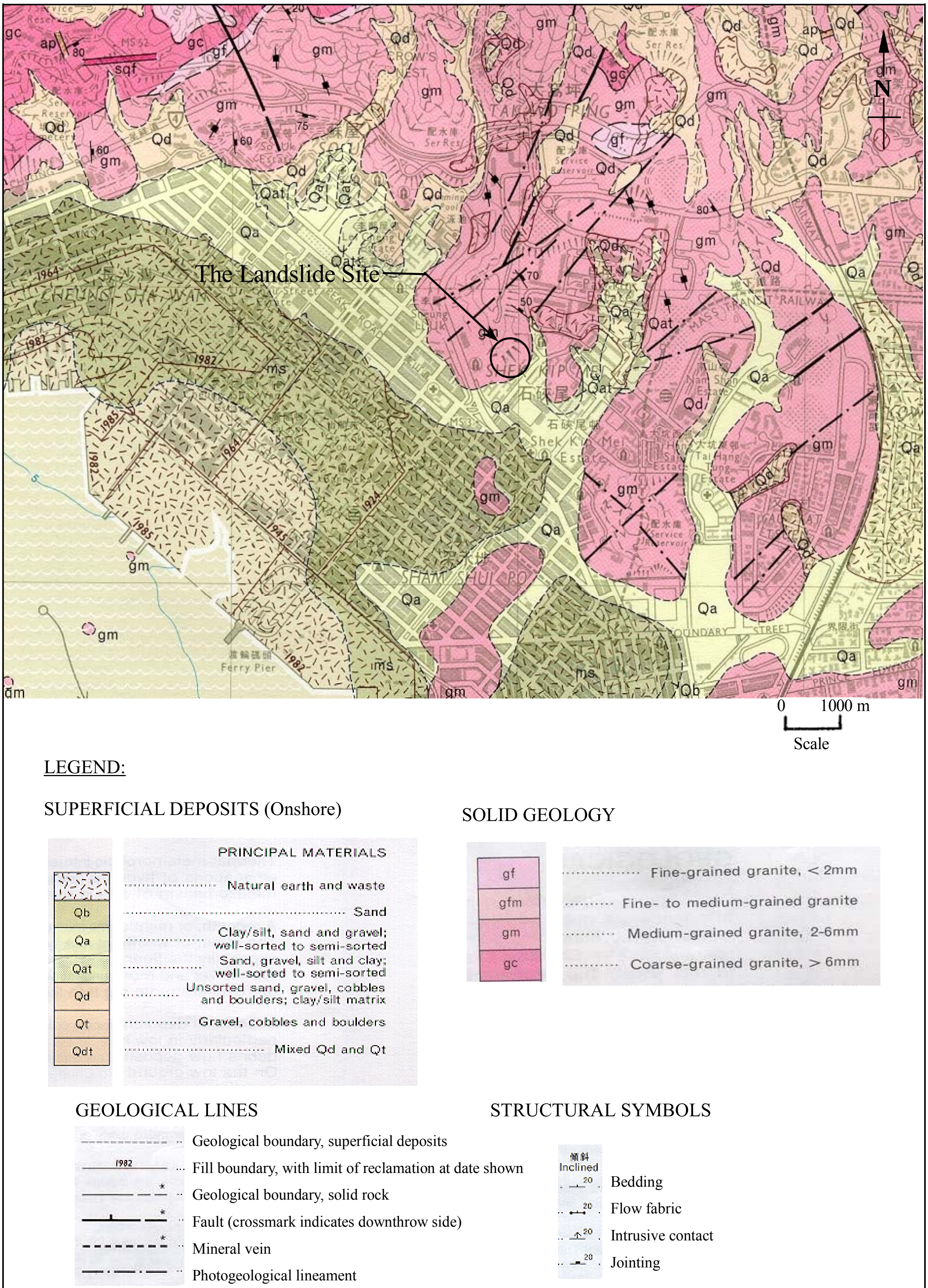

Note: Extracted from Hong Kong Geological Survey Sheet 11 (1:20,000) Map Series HGM20 (GCO, 1989).

Figure 18 - Regional Geological Map

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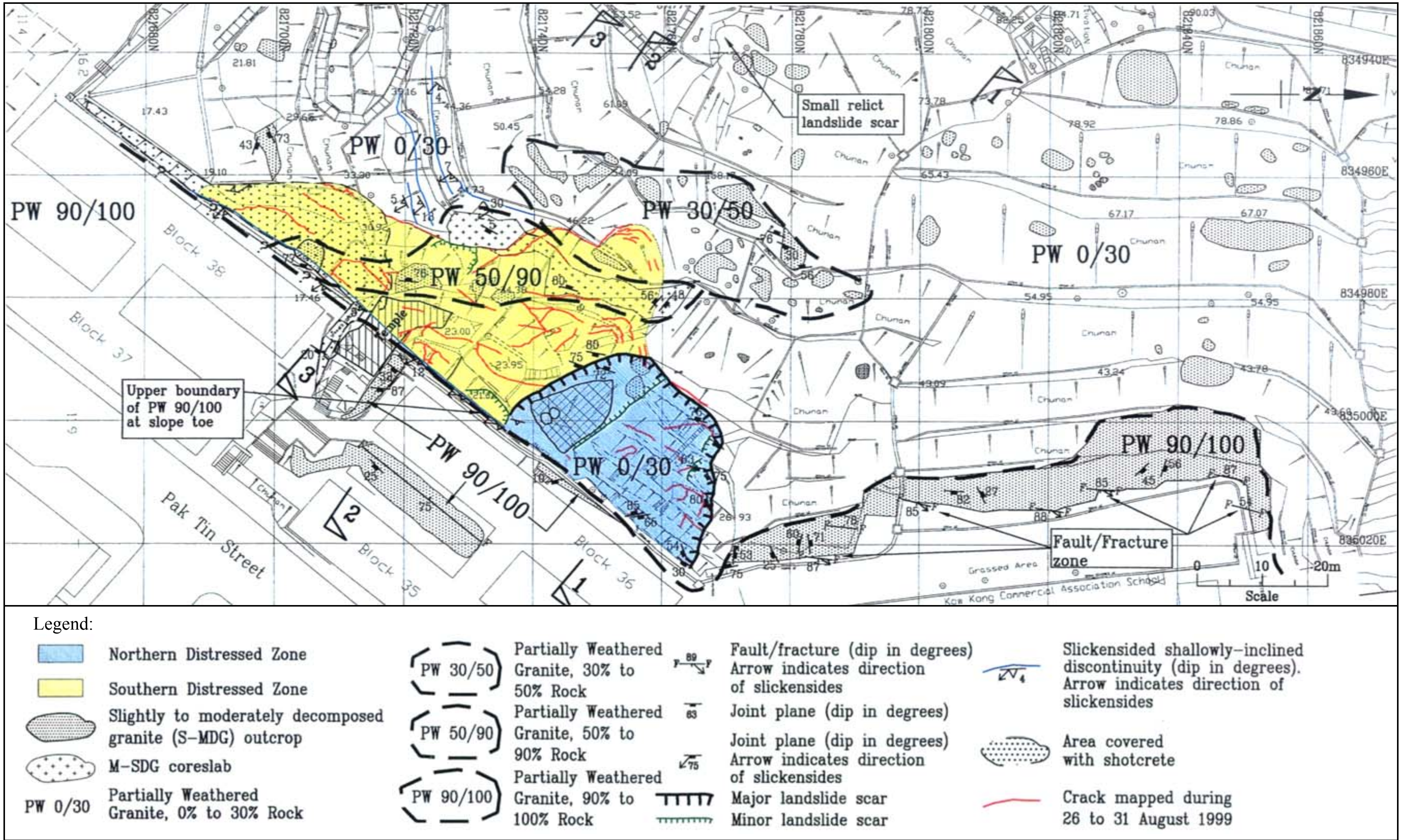

Note: The information presented is based on ground investigation works and field mapping carried out by FMSW after the 25 August 1999 landslide.

Figure 19 - Geological Plan of the Landslide Area

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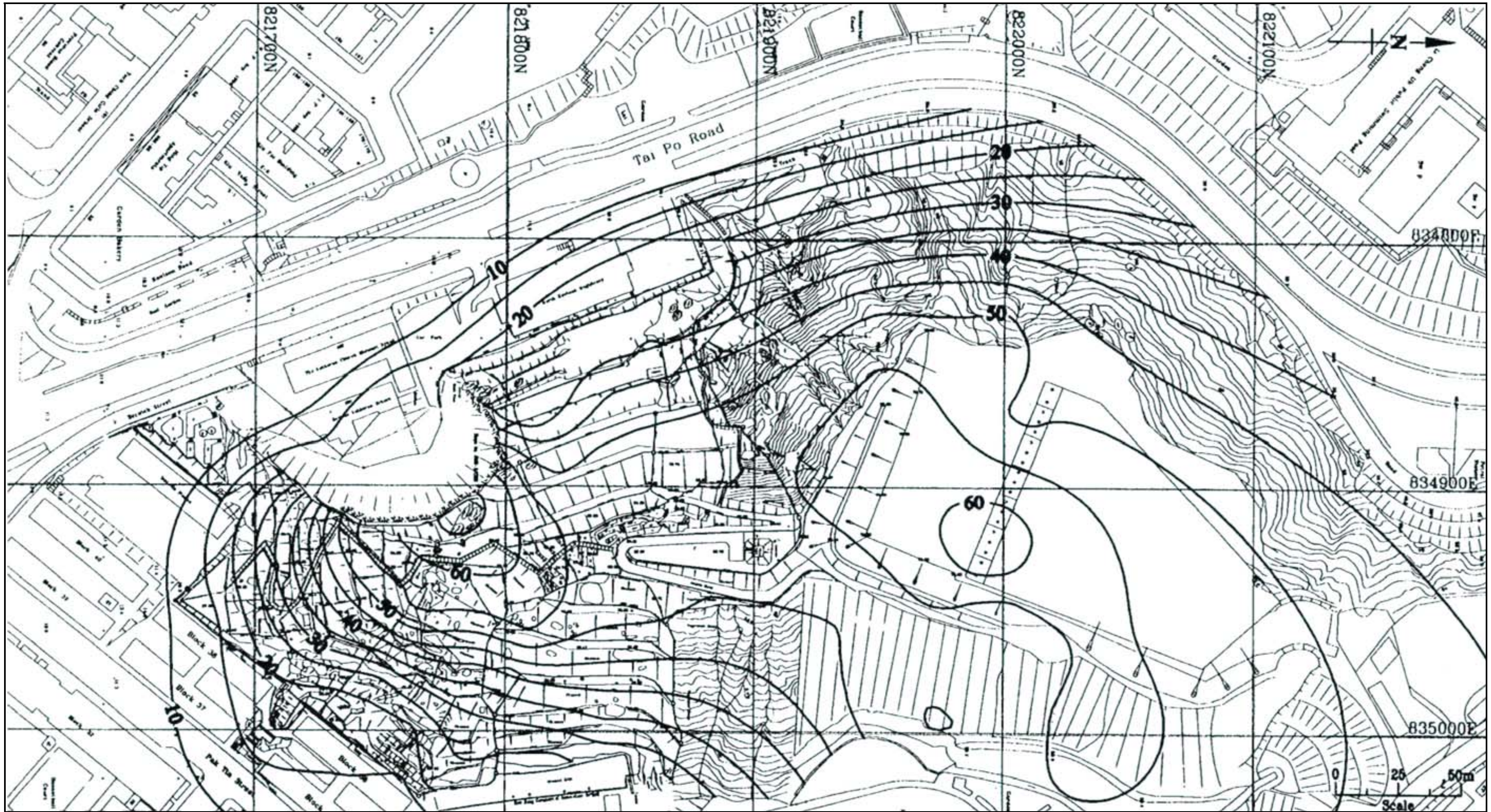

Legend:
—10— Contour line of +10 mPD representing top of Grade III or better rock

Notes: (1) Base Map is based on topographic survey carried out in September and Octorber 1999.
(2) Inferred top of Grade III or better rock is based on results of drillholes, geological mapping and geoghysical survey.

Figure 20 - Inferred Bedrock Contour Plan

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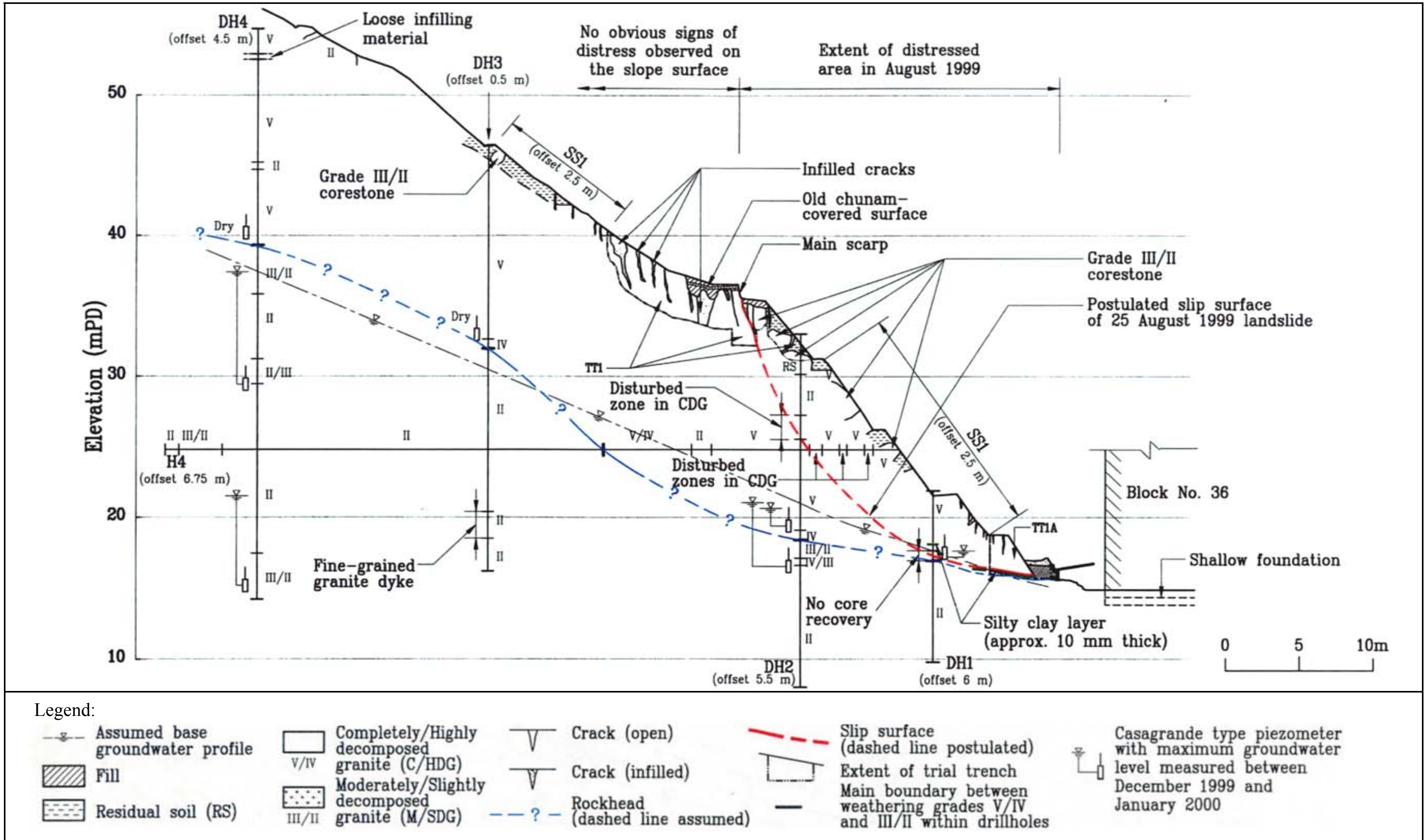

Note: Piezometers were installed in December 1999. The base groundwater table was based on the highest water levels measured in piezometers betweeen December 1999 and January 2000.

Figure 21 - Inferred Geological Profile at Section 1-1

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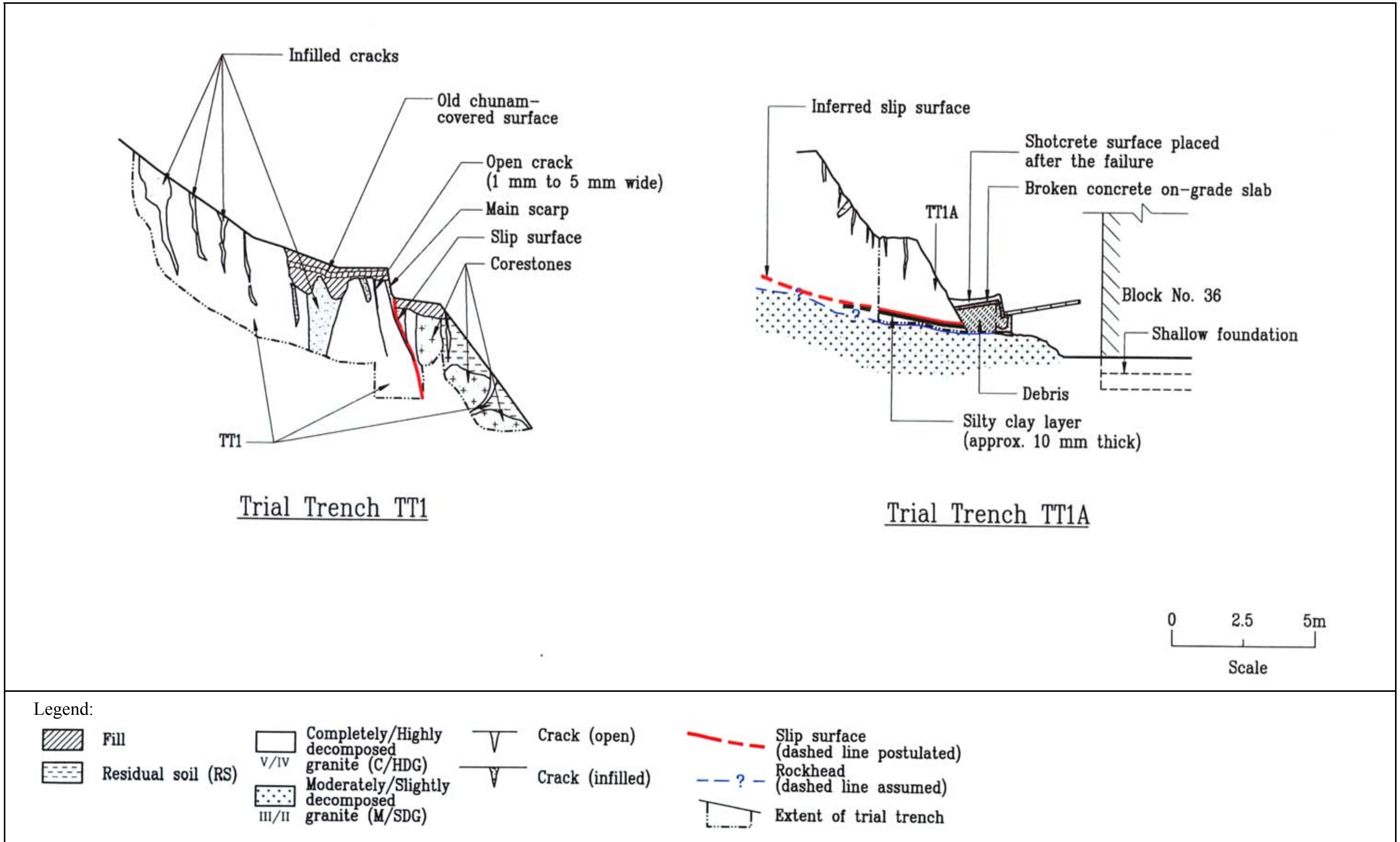

Note: See Figure 21 for locations of trail trenches.

Figure 22 - Main Observations in Trial Trenches at Section 1-1

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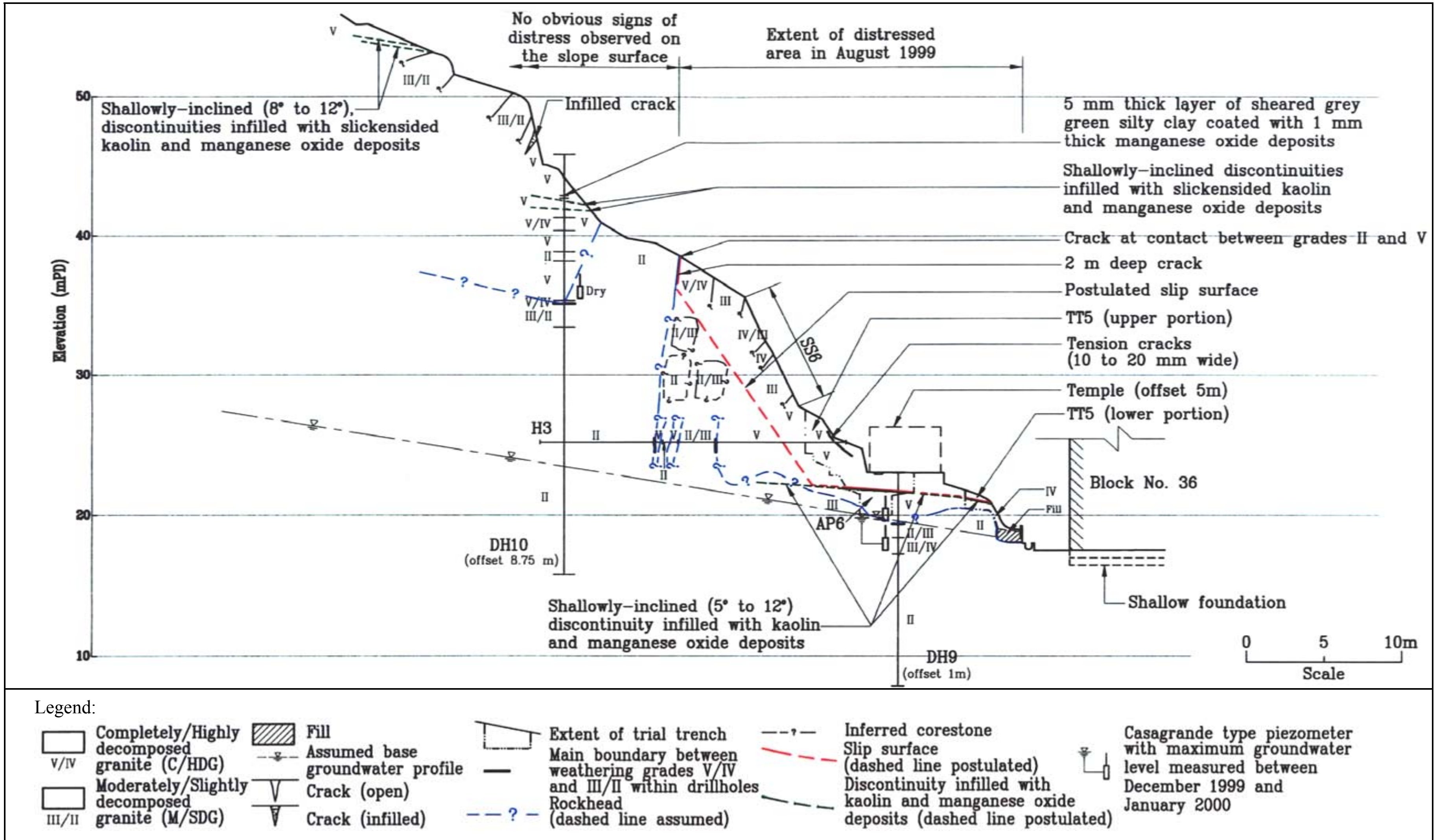
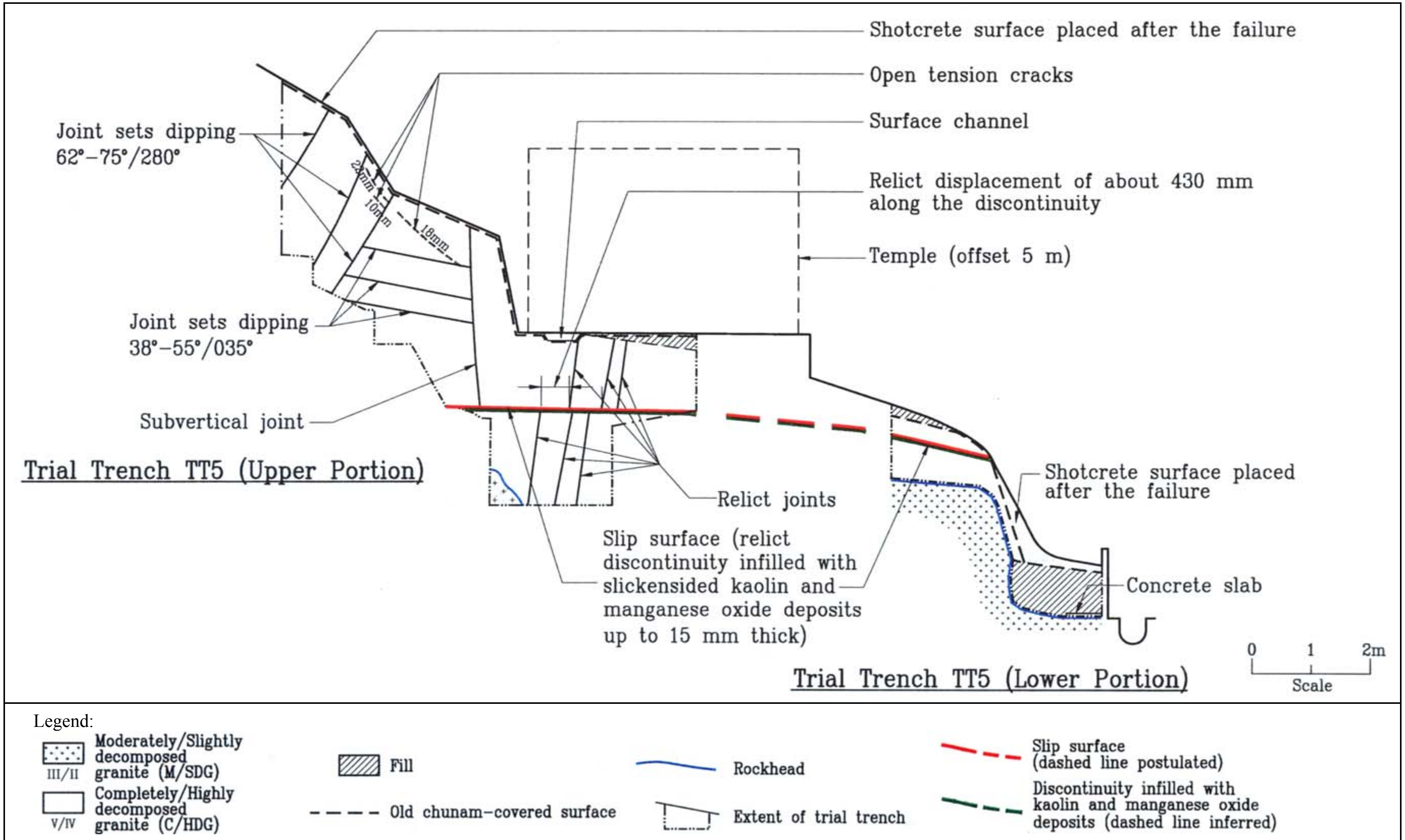


Note: Piezometers were installed in December 1999. The base groundwater table was based on the highest water levels measured in piezometers between December 1999 and January 2000.

Figure 23 - Inferred Geological Profile at Section 2-2



Note: See Figure 23 for location of trial trench TT5.

Figure 24 - Main Observations in Trial Trenches at Section 2-2

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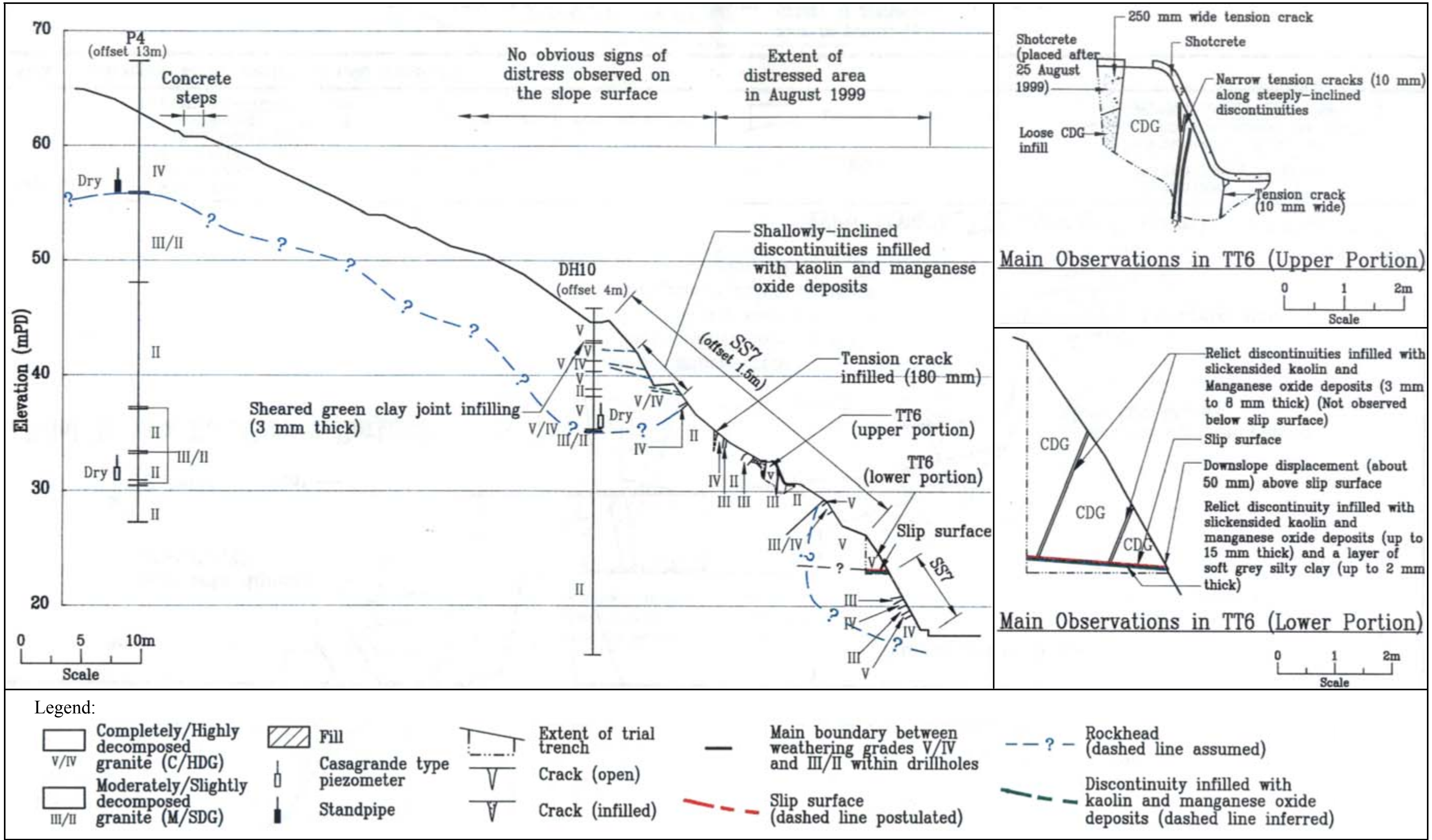
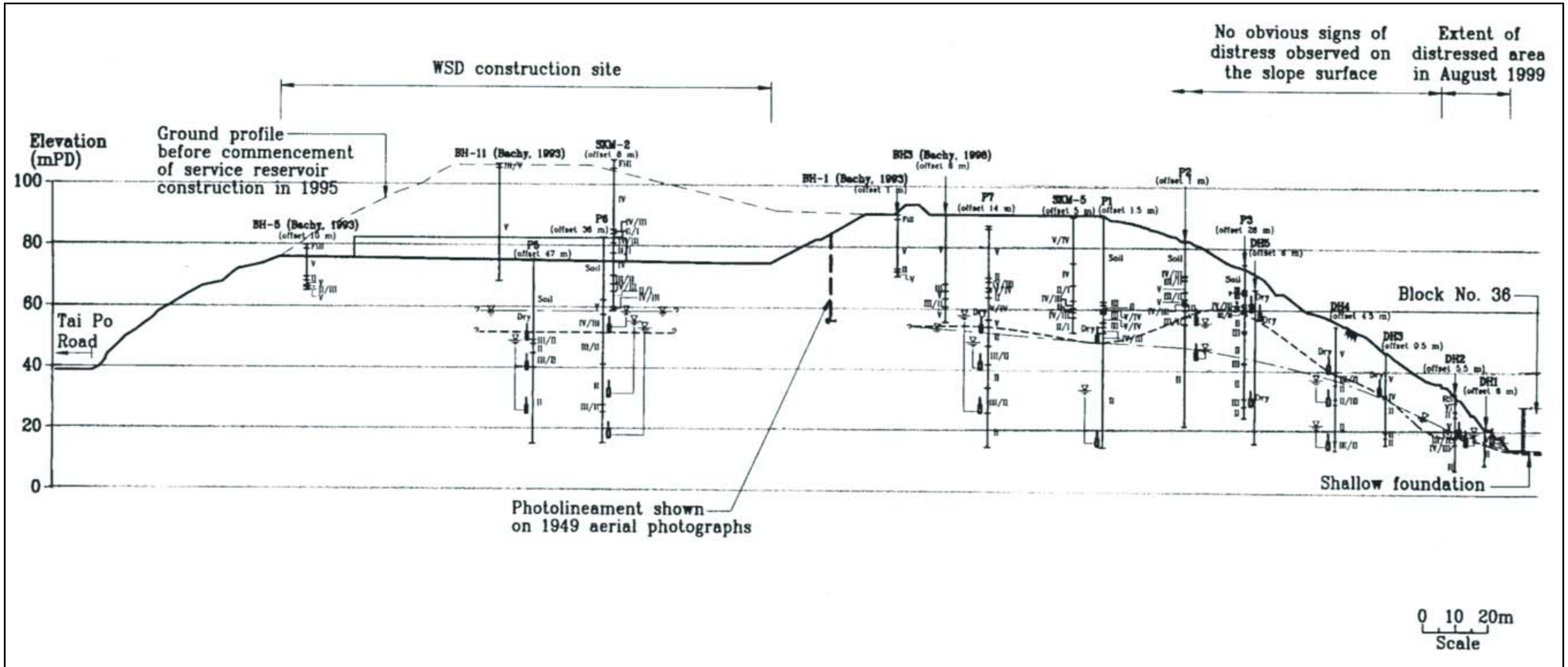


Note: Piezometers were installed in December 1999. The base groundwater table was based on the highest water levels measured in piezometers between December 1999 and January 2000.

Figure 25 - Inferred Geological Profile at Section 3-3



Legend:

Assumed base groundwater profile	RS Residual soil (RS)	P1 Drillhole by Bachy Soletanche Group (1999)	BH2 (Bachy, 1998) Drillhole by Bachy Soletanche Group (1998)
III/II Moderately/Slightly decomposed granite (M/SDG)	Inferred rockhead	DH1 Drillhole by Gammon Construction Ltd. (1999)	BH2 (Enpack, 1997) Drillhole by Enpack (HK) Ltd. (1997)
V/IV Completely/Highly decomposed granite (C/HDG)	Casagrande type piezometer/ stanpipe with maximum groundwater level measured between December 1999 and January 2000	SKM-2 Drillhole by Enpack (HK) Ltd. (1977)	BH-1 (Bachy, 1993) Drillhole by Bachy Soletanche Group (1993)

Notes: (1) See Figure 2 for location of section line.
(2) Piezometers were installed in December 1999. The base groundwater table was based on the highest water levels measured in piezometers between December 1999 and January 2000.

Figure 26 - Inferred Geological Profile at Section 4-4

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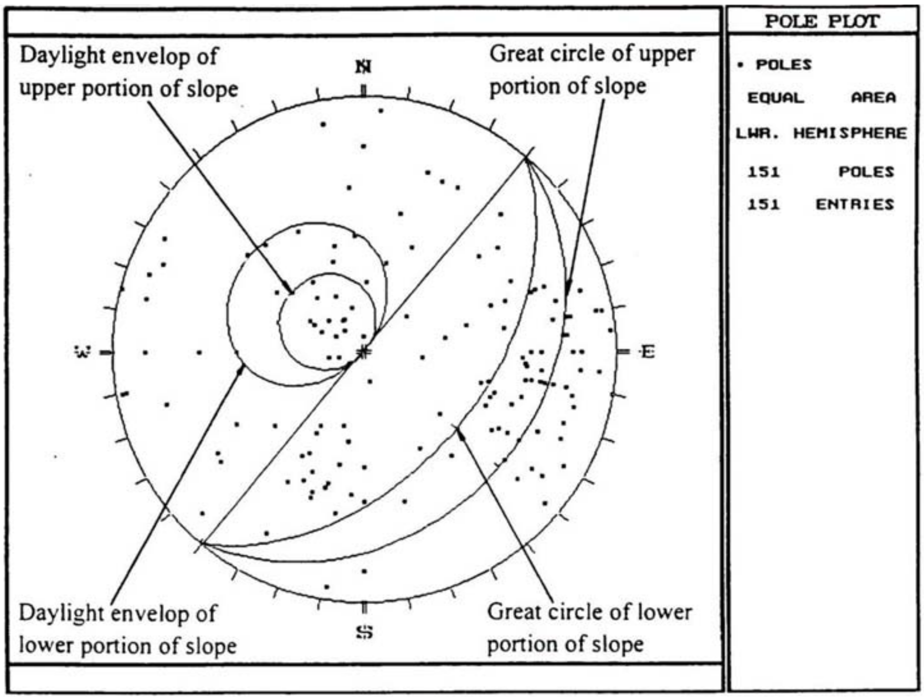
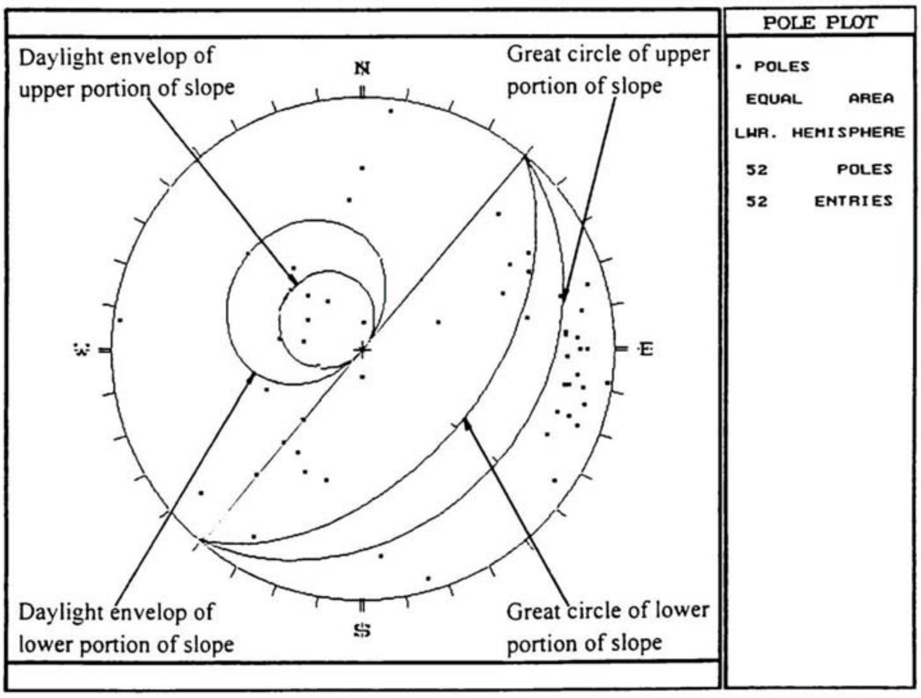


(a) Relict Discontinuities Infilled with Kaolin and Manganese Oxide Deposits



(b) Slickensided Relict Discontinuities Infilled with Kaolin and Manganese Oxide Deposits

Note: Average dip angles and dip directions of the slope profile before the 1999 failure are 30° / 130° and 50° / 130° for the upper and lower portions of slope No. 11NW-B/C90 respectively.

Figure 27 - Stereoplots for the Measured Orientations of Discontinuities in Highly to Completely Decomposed Granite

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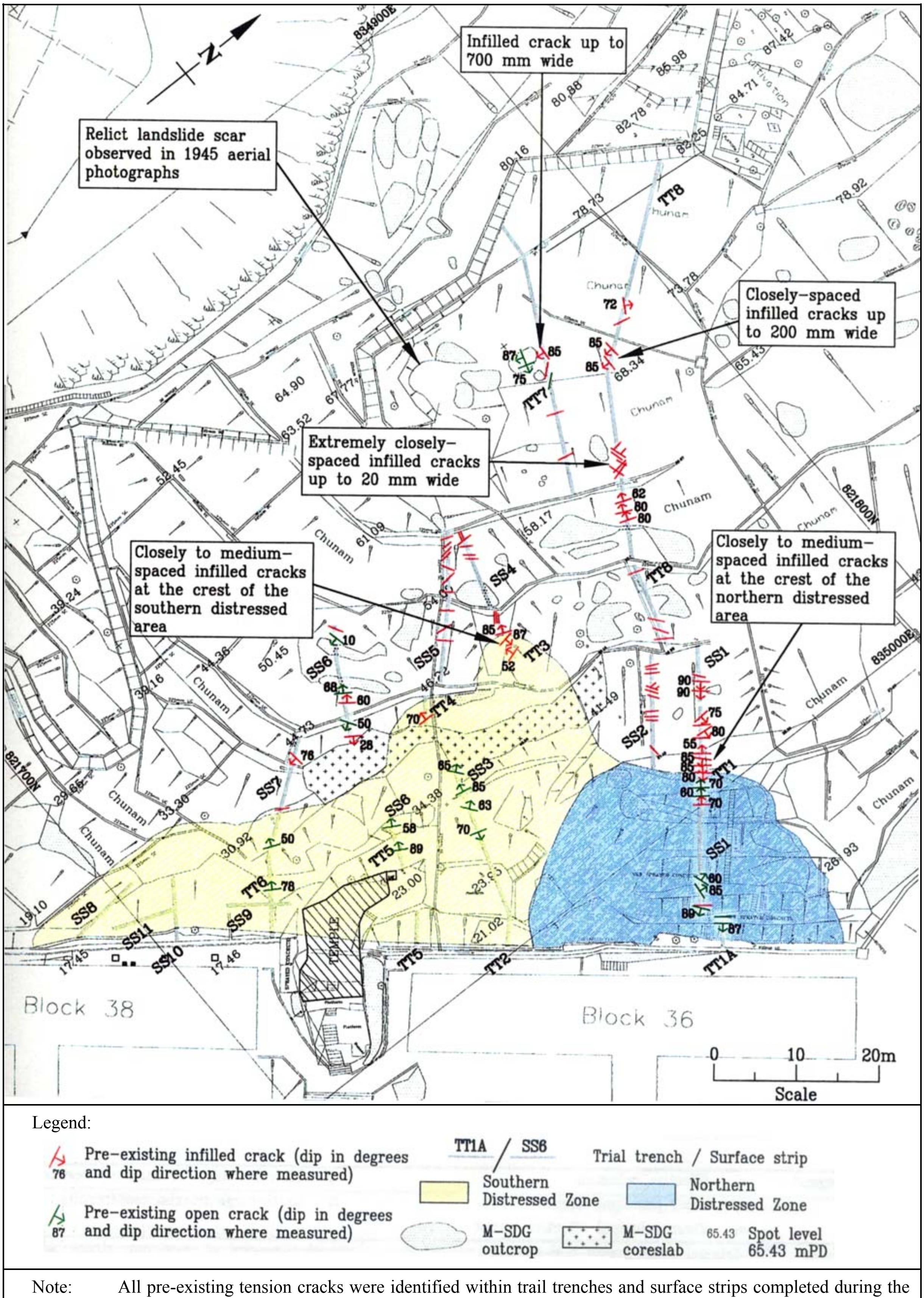

Note: All pre-existing tension cracks were identified within trail trenches and surface strips completed during the subsurface ground investigation.

Figure 28 - Locations of Pre-existing Tension Cracks

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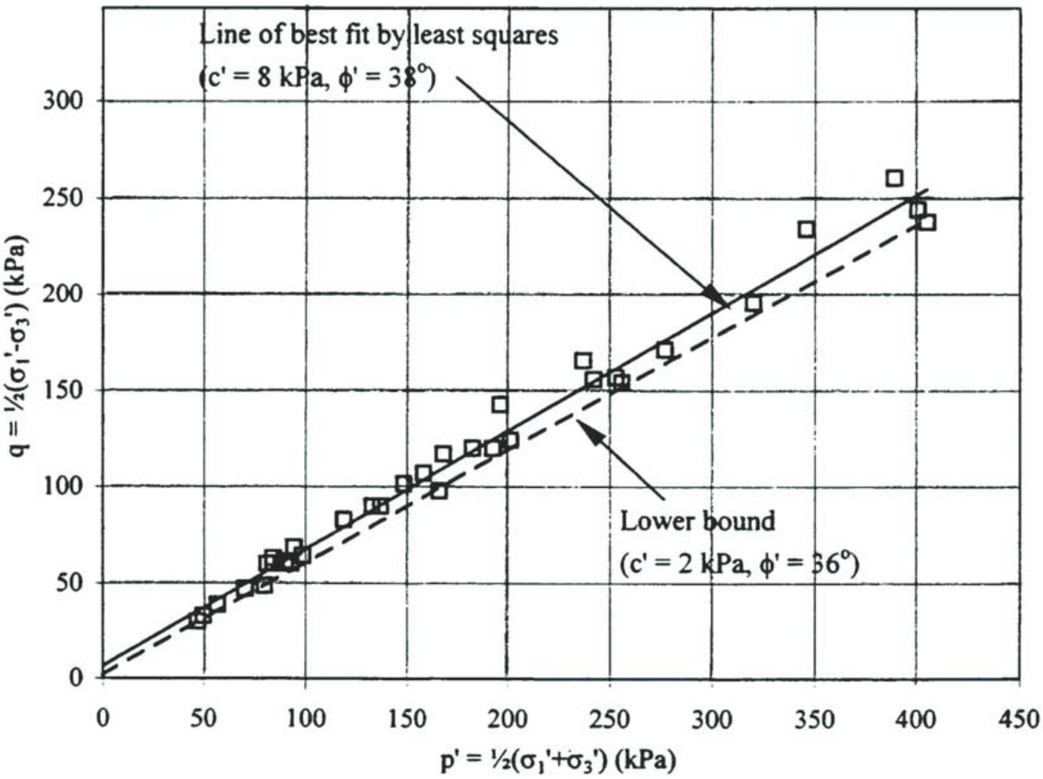

Legend:

c'	Effective cohesion	ϕ'	Effective angle of shearing resistance
σ_1'	Major principal effective stress	□	Multi-stage triaxial test results
σ_3'	Minor principal effective stress		

Notes: (1) Traixial compression test results correspond to those at the point of maximum stress obliquity, i.e. maximum stress ratio σ_1' / σ_3'.
(2) Only test results corresponding to p' up to 400 kPa have been included.

Figure 29 - Results of Triaxial Compression Tests on Completely Decomposed Granite

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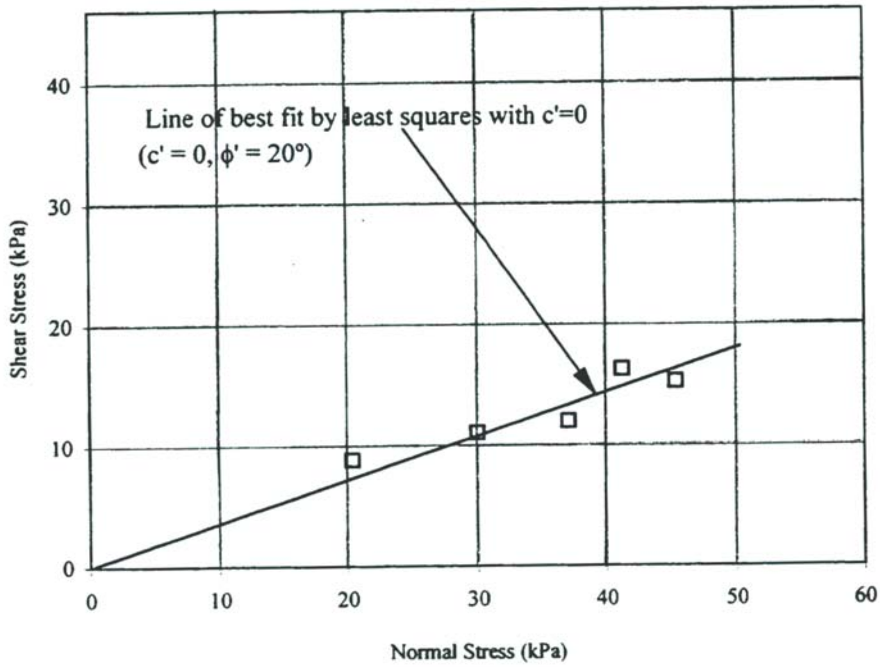

Legend:

c'	Effective cohesion
□	Single-stage direct shear box test results
ϕ'	Effective angle of shearing resistance

Notes: (1) Specimens were sheared along the joint plane and along the slickensided direction.
(2) Direct shear box test results shown in this figure correspond to those at the point of peak shear stress.

Figure 30 - Results of Direct Shear Box Tests on Slickensided Discontinuity Infilled with Kaolin and Manganese Deposits

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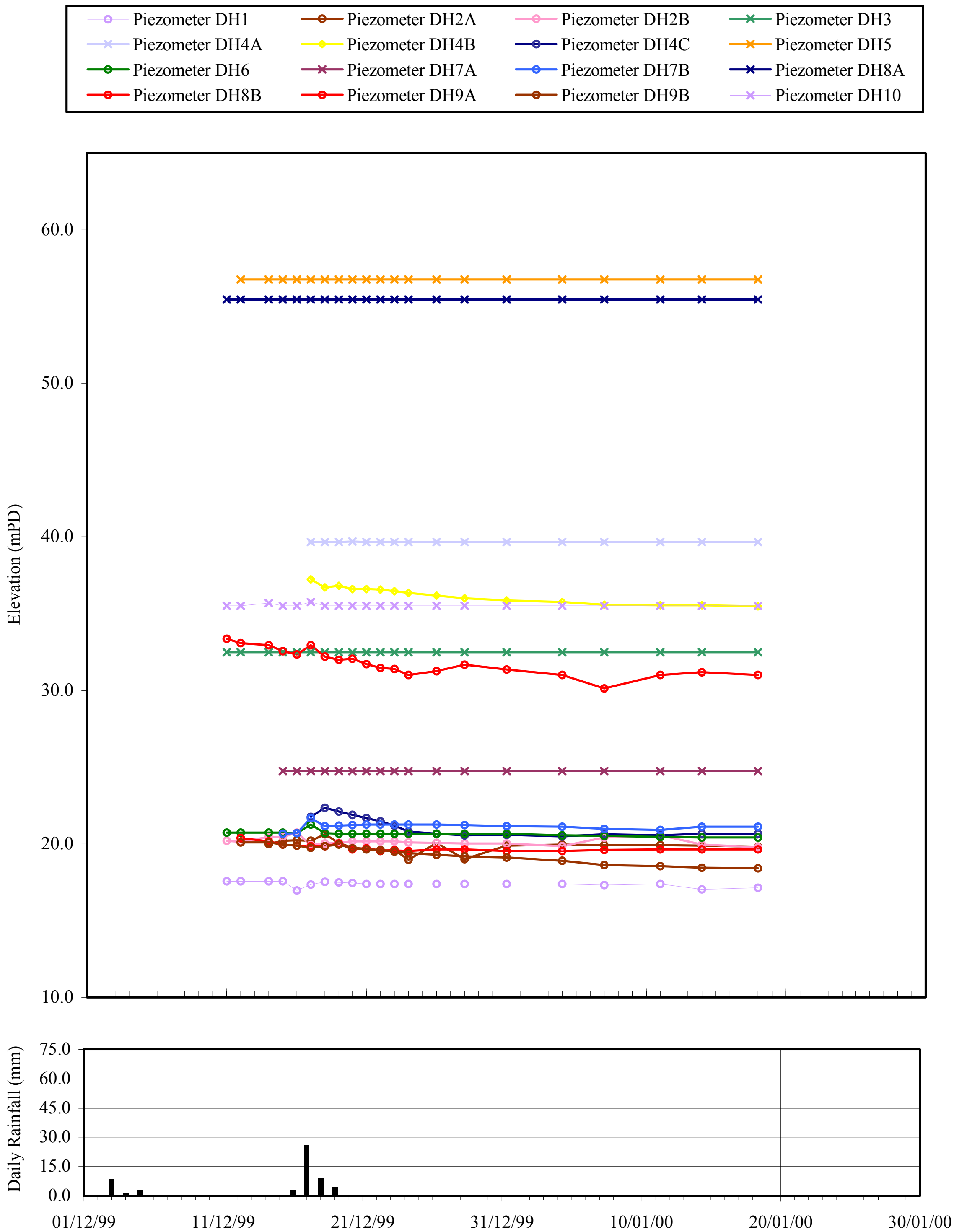

Legend:
Measured groundwater level in piezometer
Bottom level of piezometer which was dry during monitoring

Note: See Figure 17 for locations of piezometers.

Figure 31 - Records of Groundwater Monitoring

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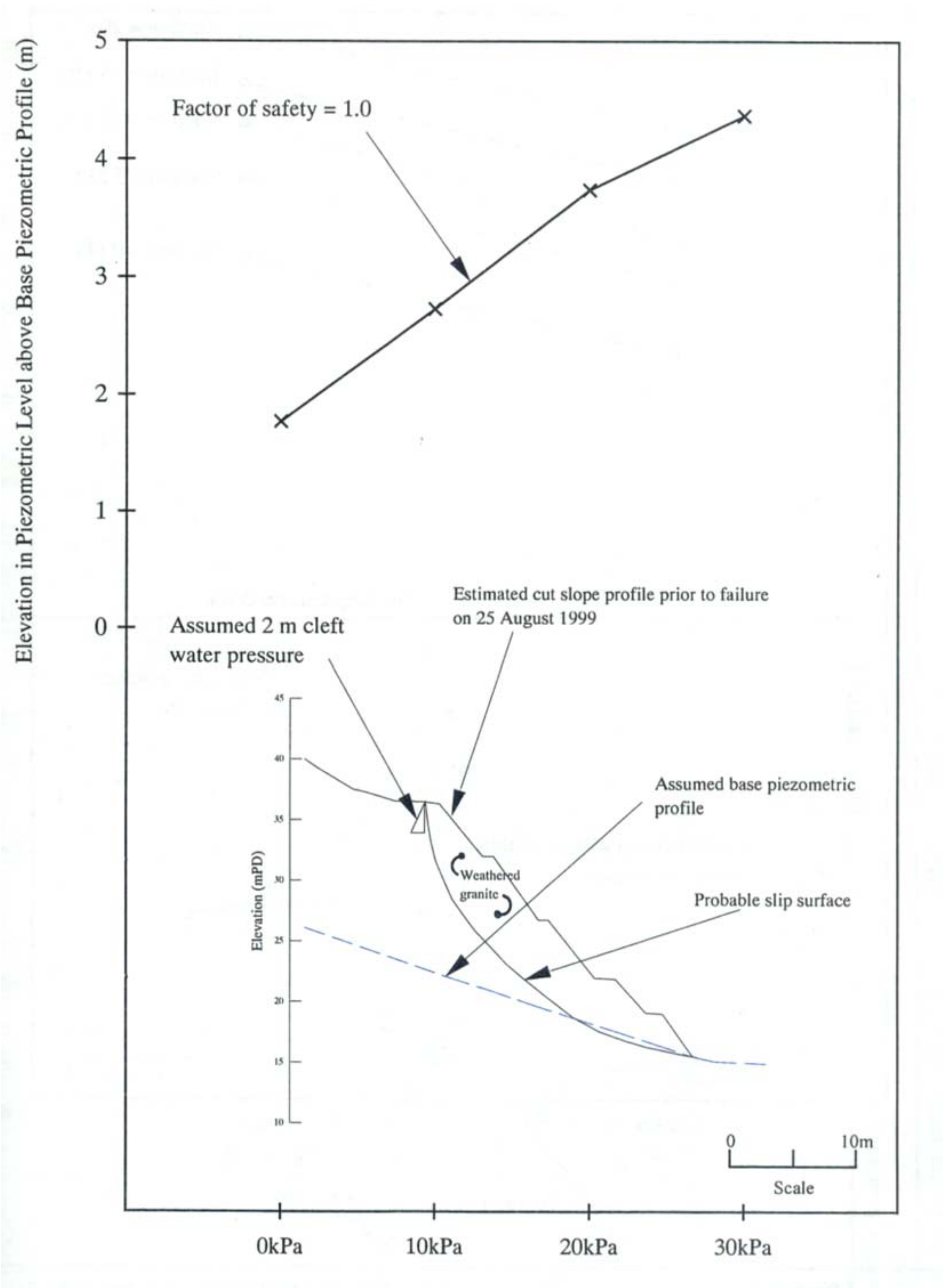

Soil Suction in the Ground Mass

Notes:
(1) The inferred geological profile at Section 1-1 given in Figure 21 was selected for analyses.
(2) The rigorous method of Morgenstern & Price (1965) was used in the stability analyses.
(3) The shear strength parameters for the weathered granite are c' = 8kPa, $\phi' = 38^{\circ}$.
(4) 2 m cleft water pressure was assumed in the stability analyses.

Figure 32 - Results of Slope Stability Analyses for the Northern Distressed Zone

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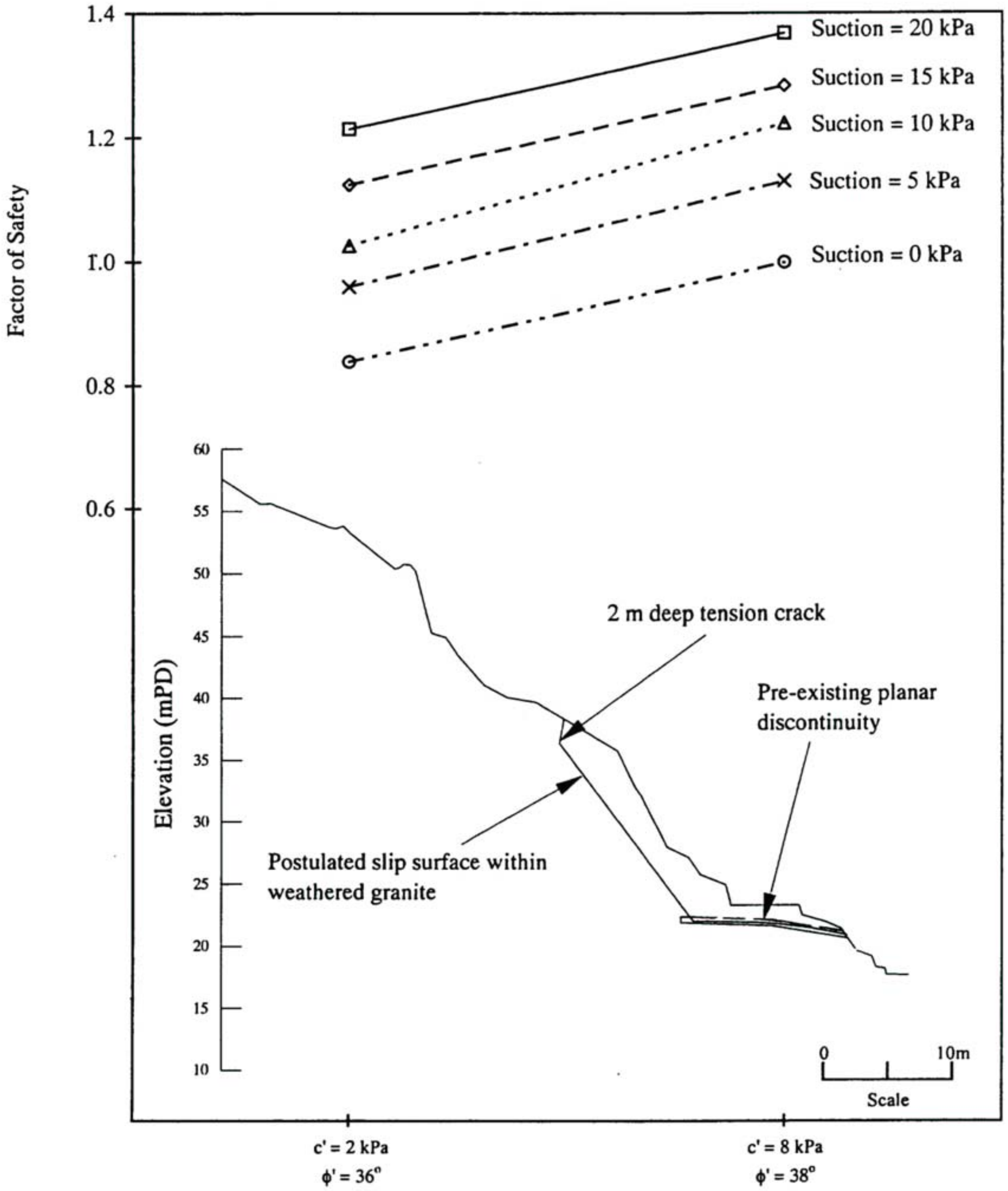

Shear Strength of the Ground Mass

Legend:
c' Effective cohesion
ϕ' Effective angle of shearing resistance

Notes:
(1) Inferred geological profile at Section 2-2 given in Figure 23 was selected for analyses.
(2) The shear strength parameters of the pre-existing, planar discontinuity are : c' = 0, ϕ' = 20°.
(3) Water table at 0.3 m above the pre-existing, planar discontinuity based on site observation of seepages.
(4) The tension crack was assumed to be filled with water.
(5) The rigorous method of Morgenstern & Price (1965) was used in stability analyses.

Figure 33 - Results of Slope Stability Analyses for the Southern Distressed Zone

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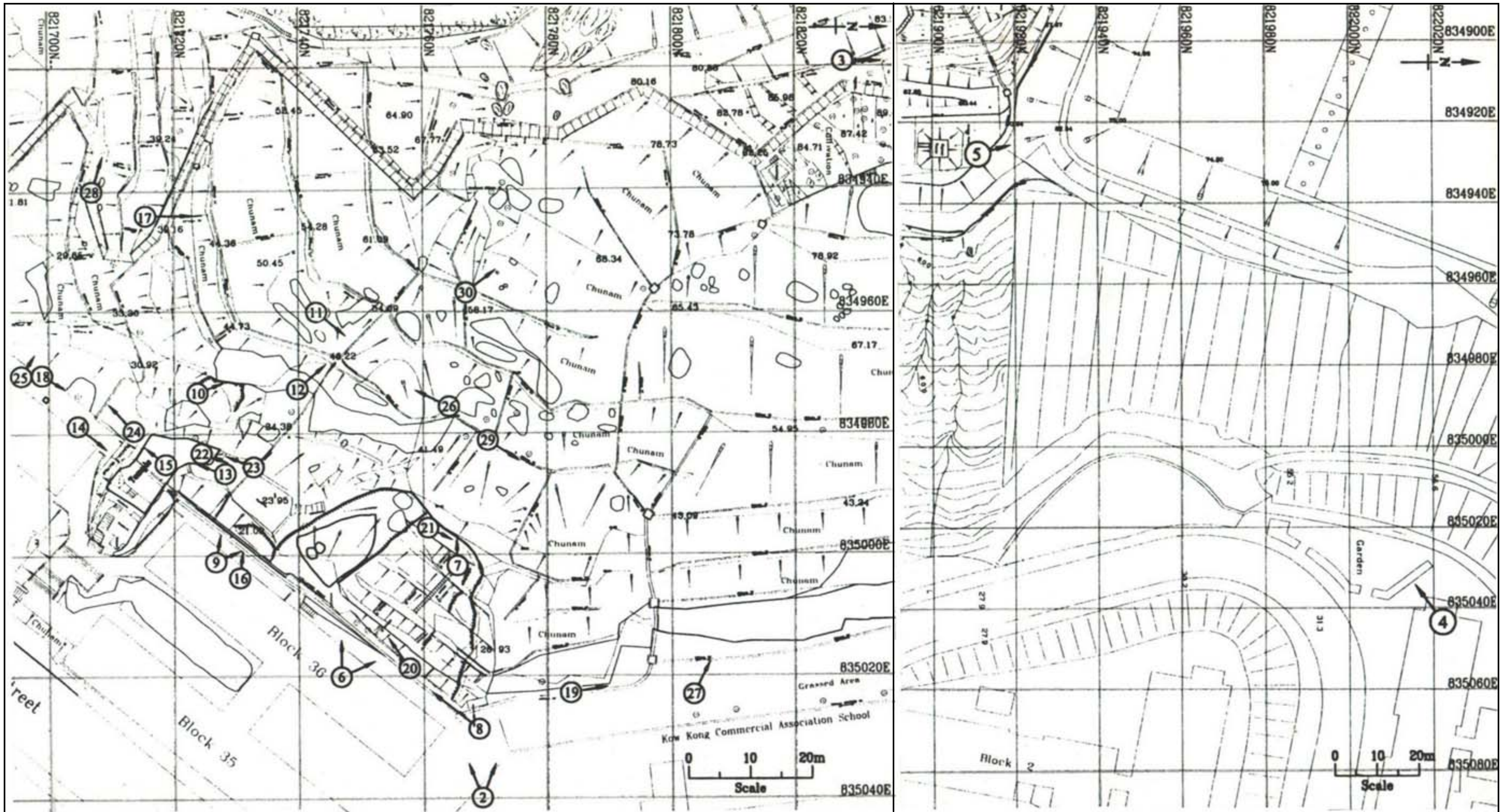

Legend:

Approximate location from which Plate 2 was taken

Note: Base map is based on topographic survey carried out in September and October 1999.

Figure 34 - Location Plan of Photographs Taken

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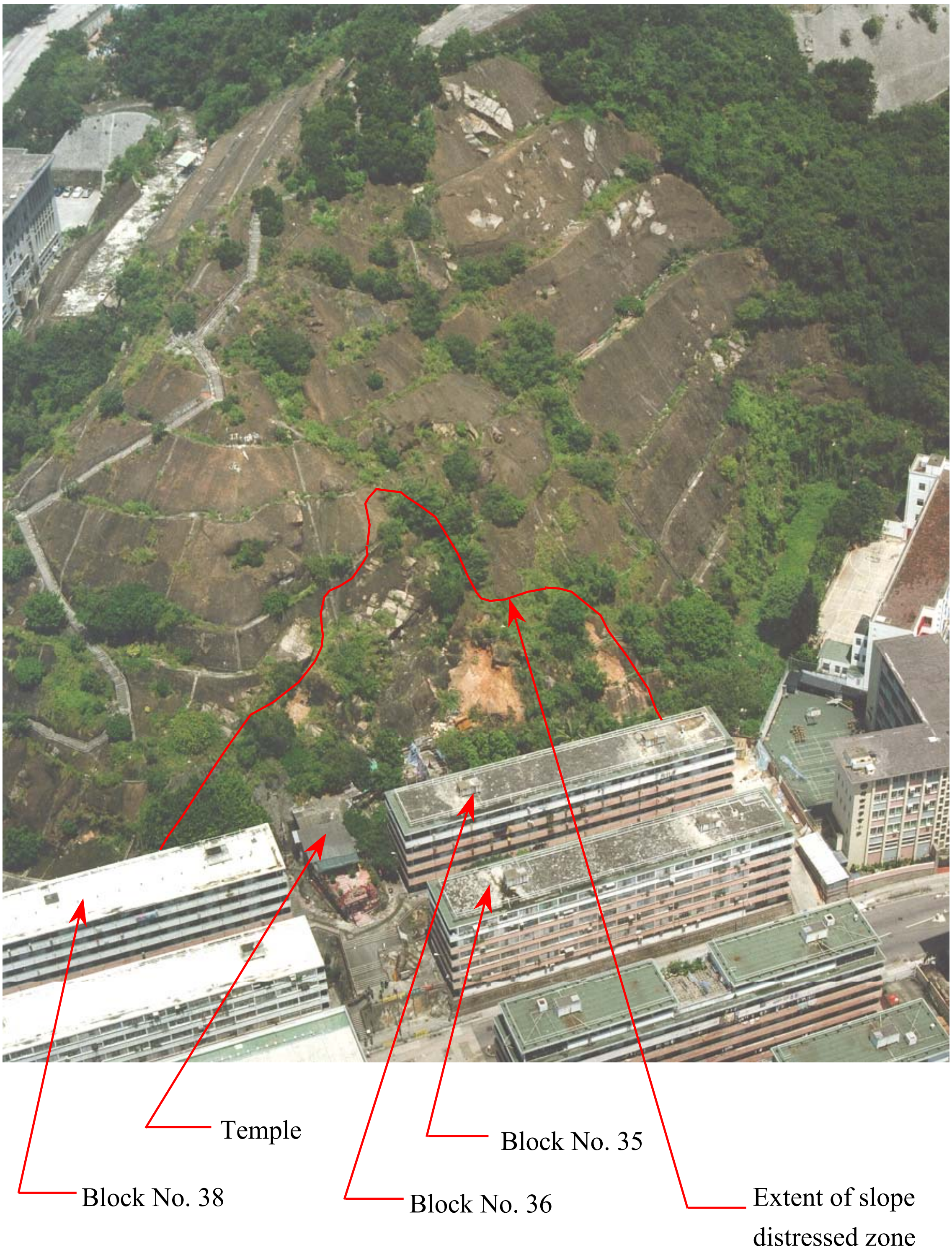

Plate 1 - Oblique Aerial View of the Landslide Site (Photograph Taken on 26 August 1999)

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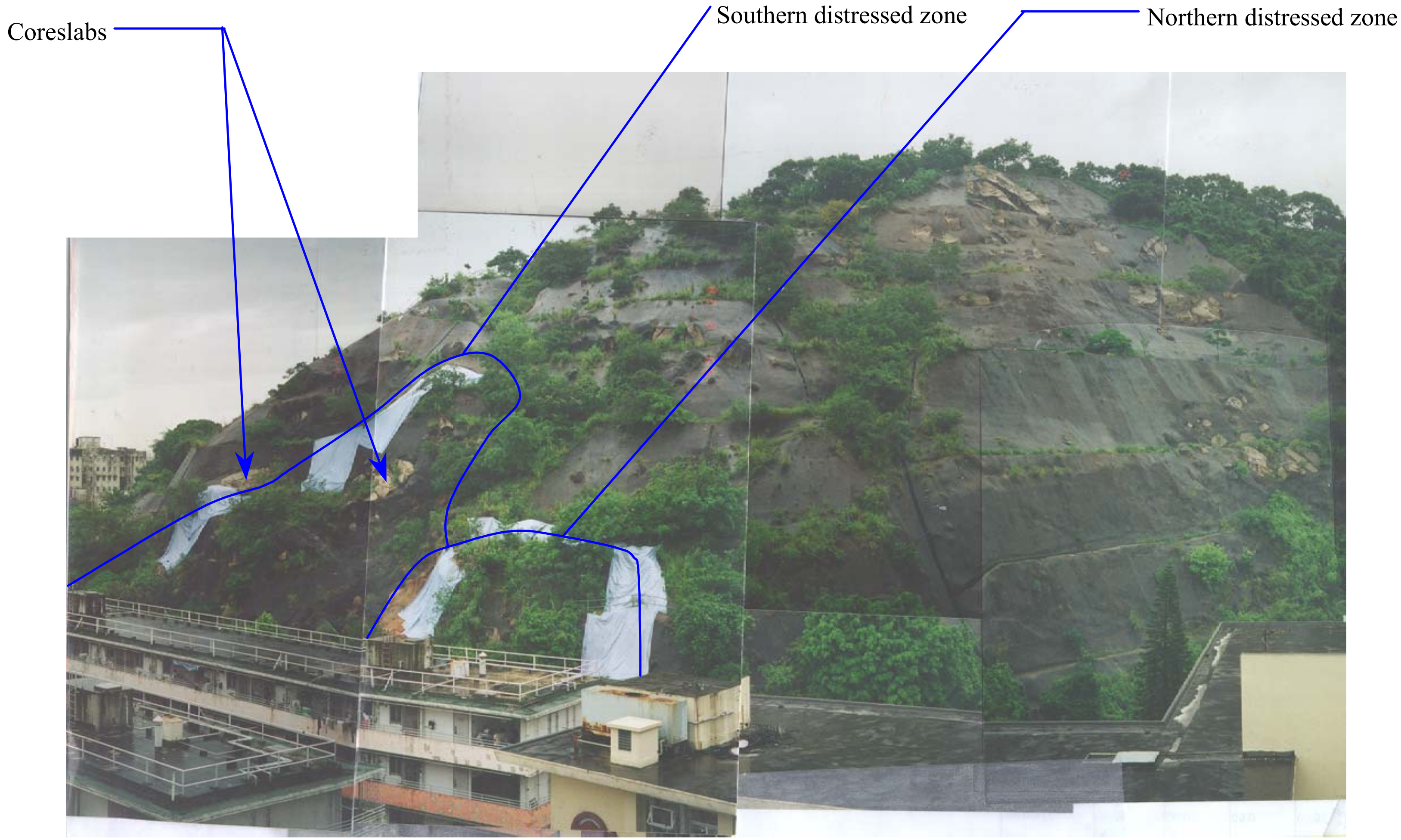

Plate 2 - General View of the Distressed Slope (Photograph Taken on 28 August 1999).
See Figure 34 for location.

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Plate 3 - View of the Unauthorized Cultivation Area at the Hilltop above the Distressed Slope (Photograph Taken on 10 September 1999). See Figure 34 for location.

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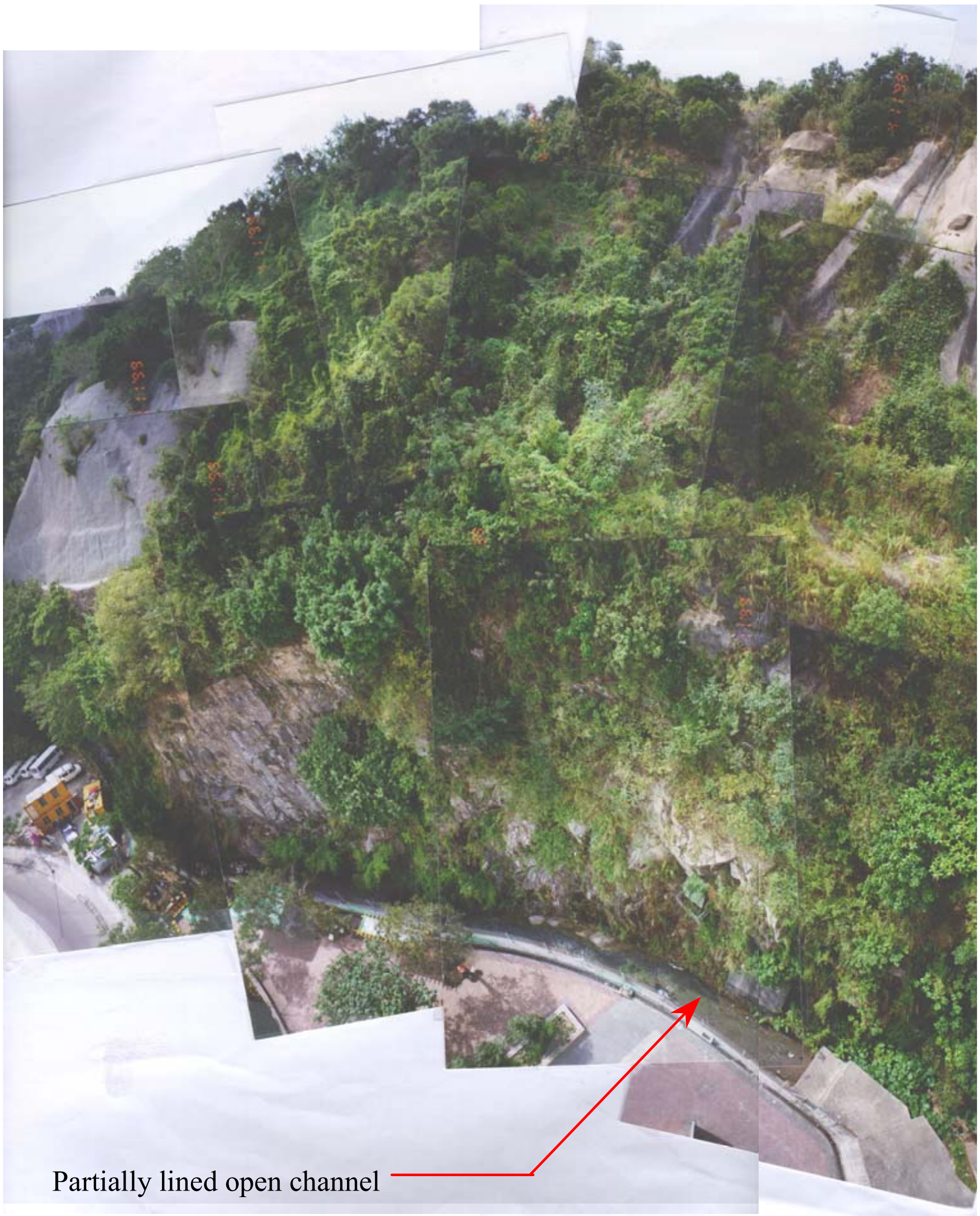

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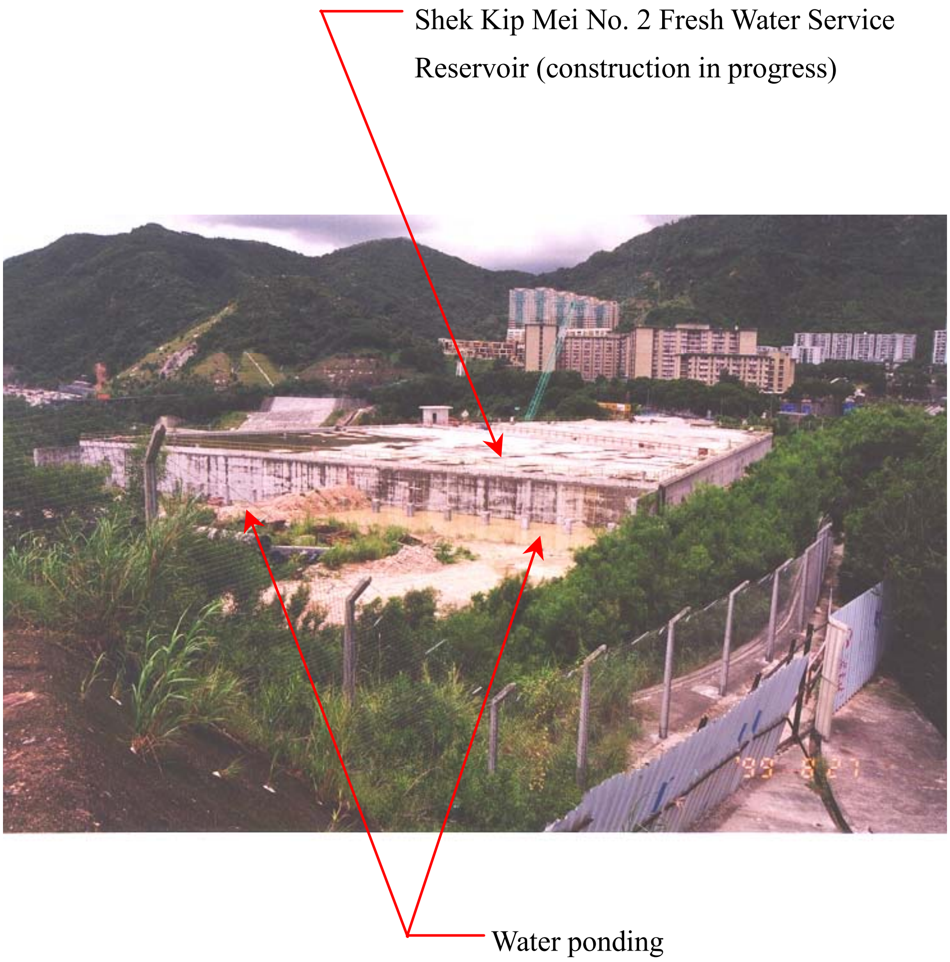

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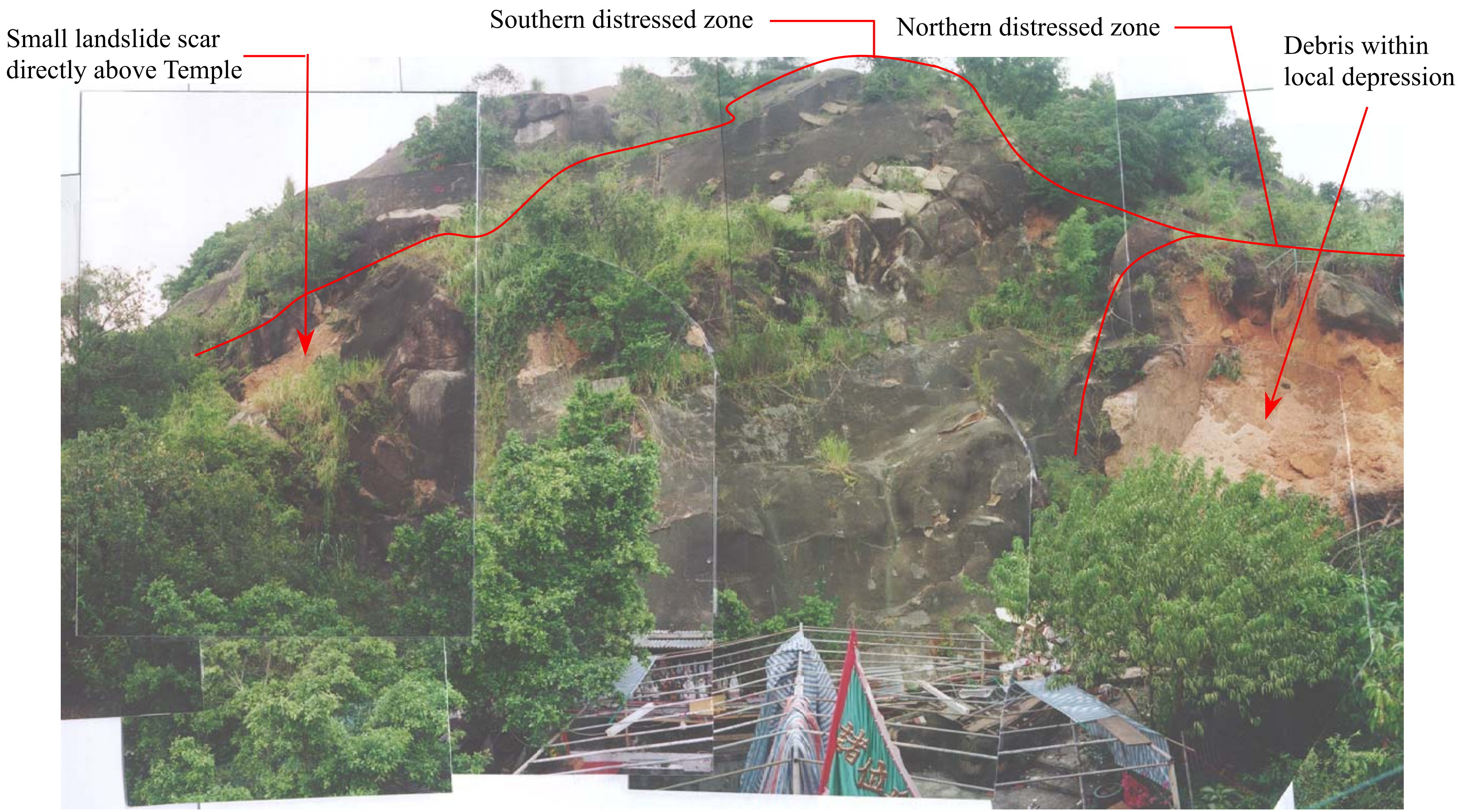

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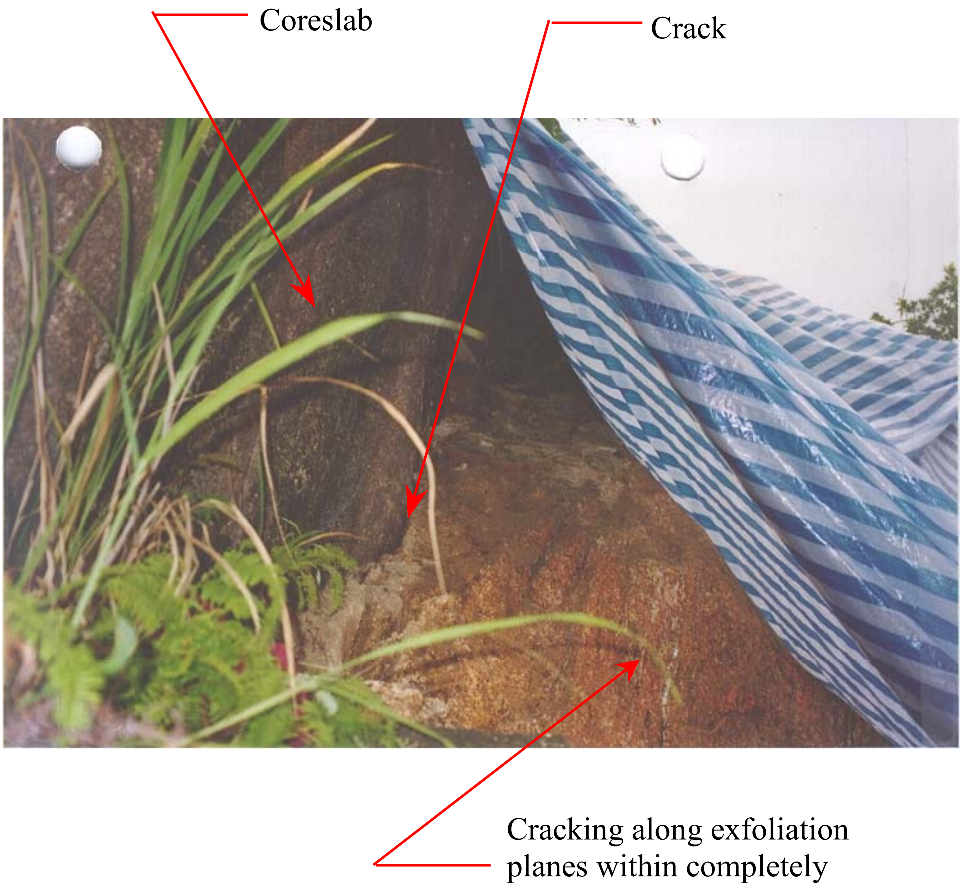

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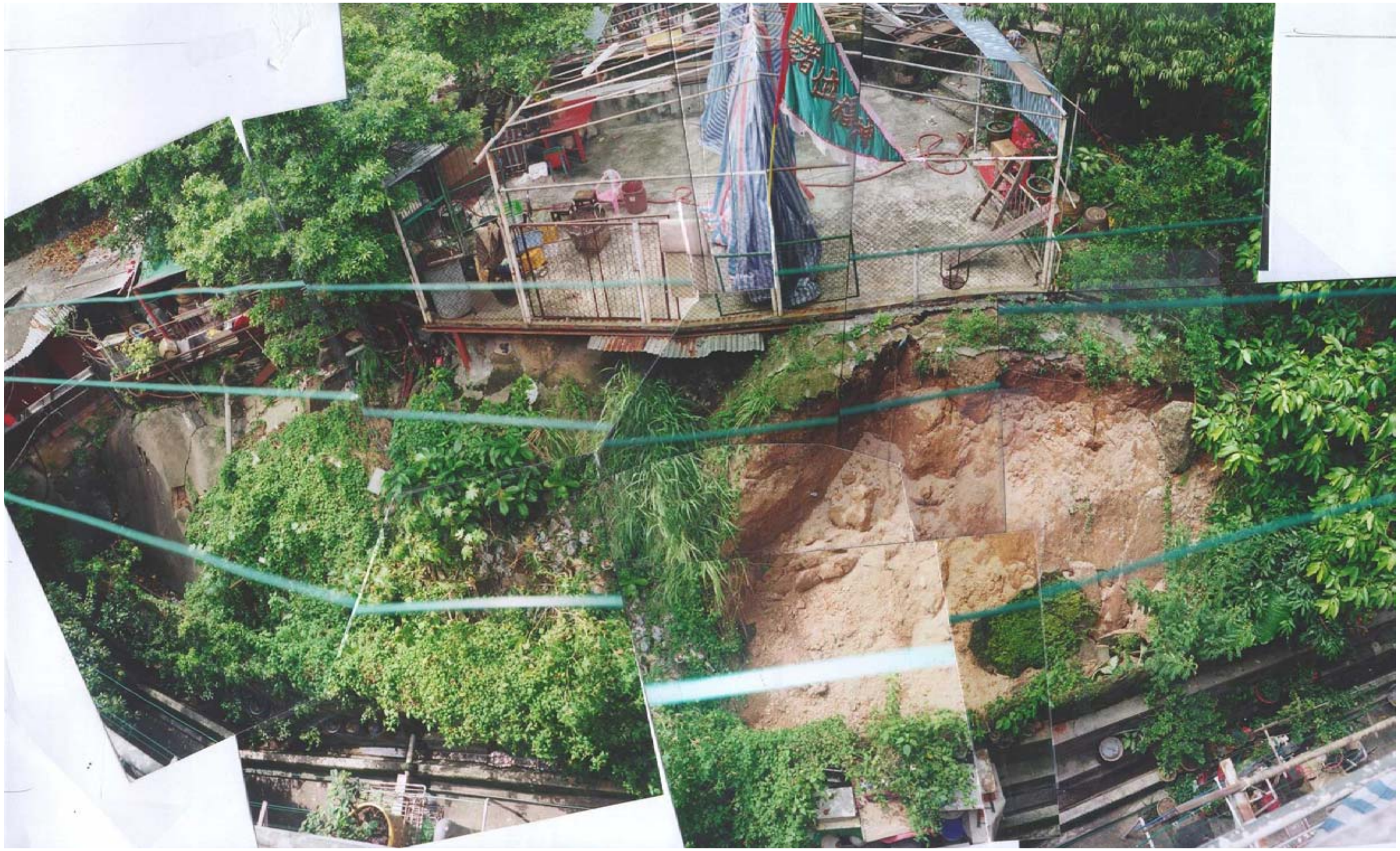

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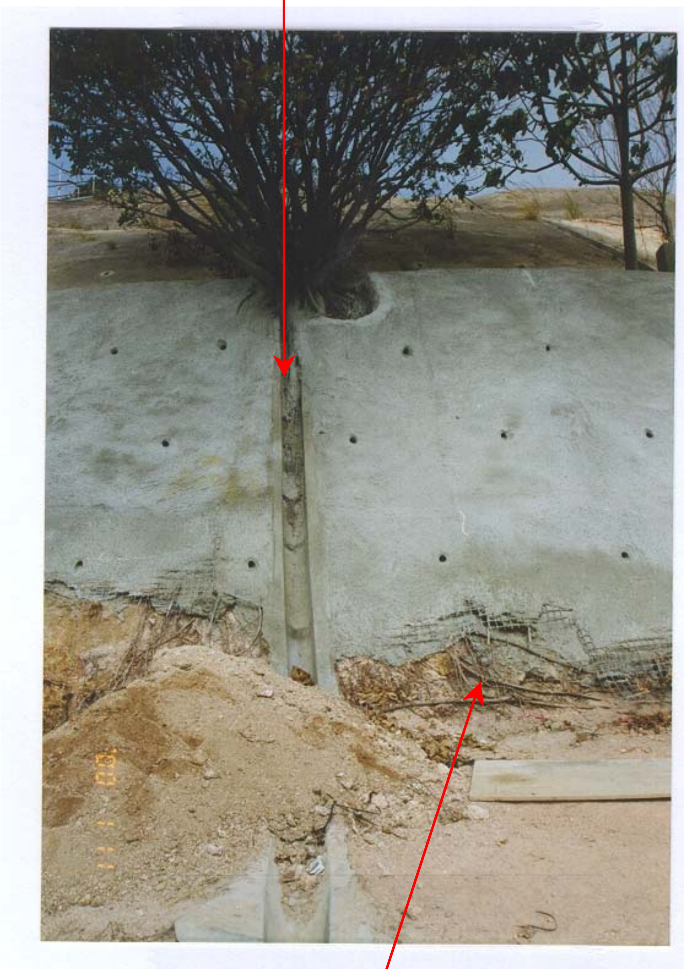

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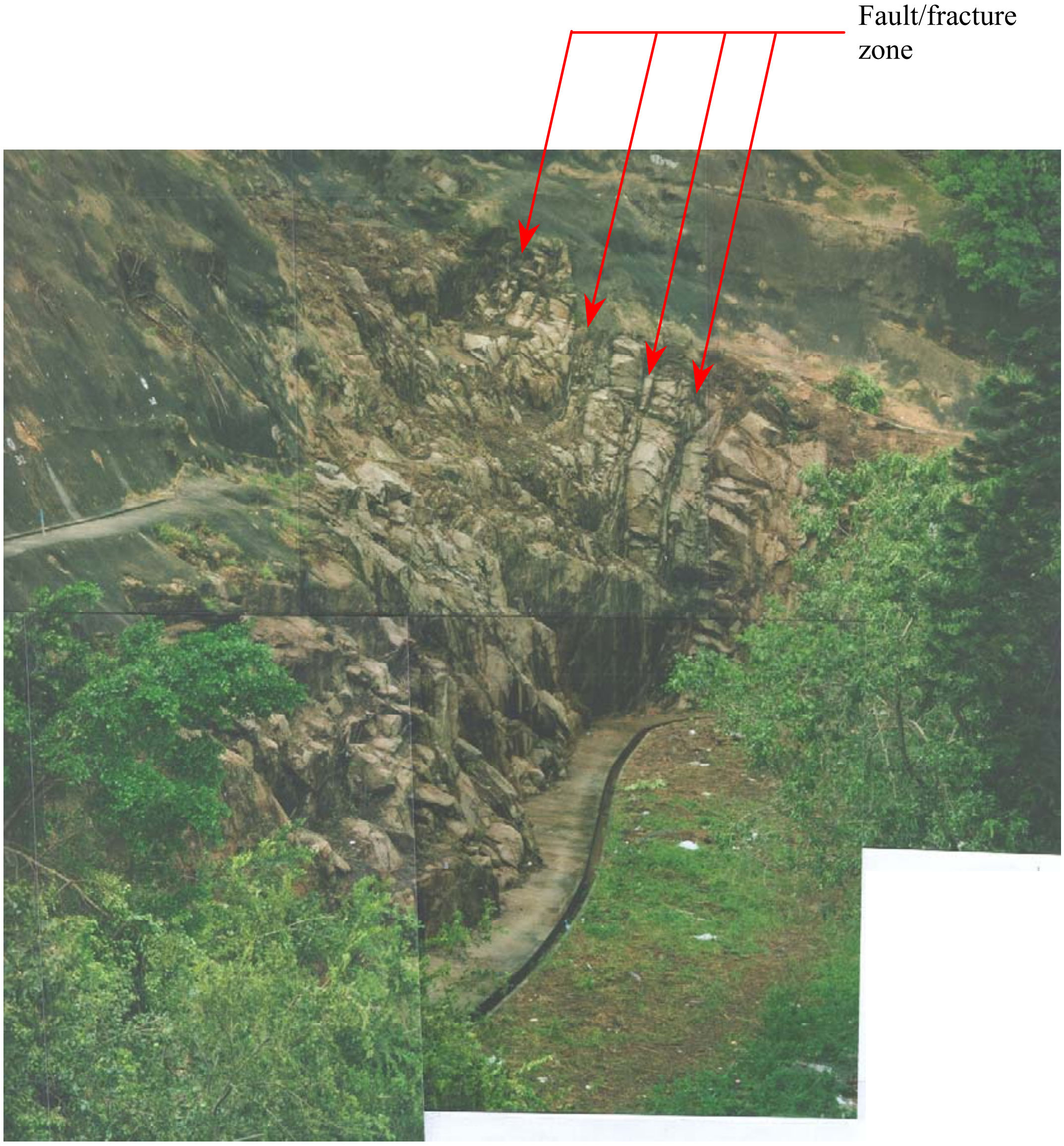

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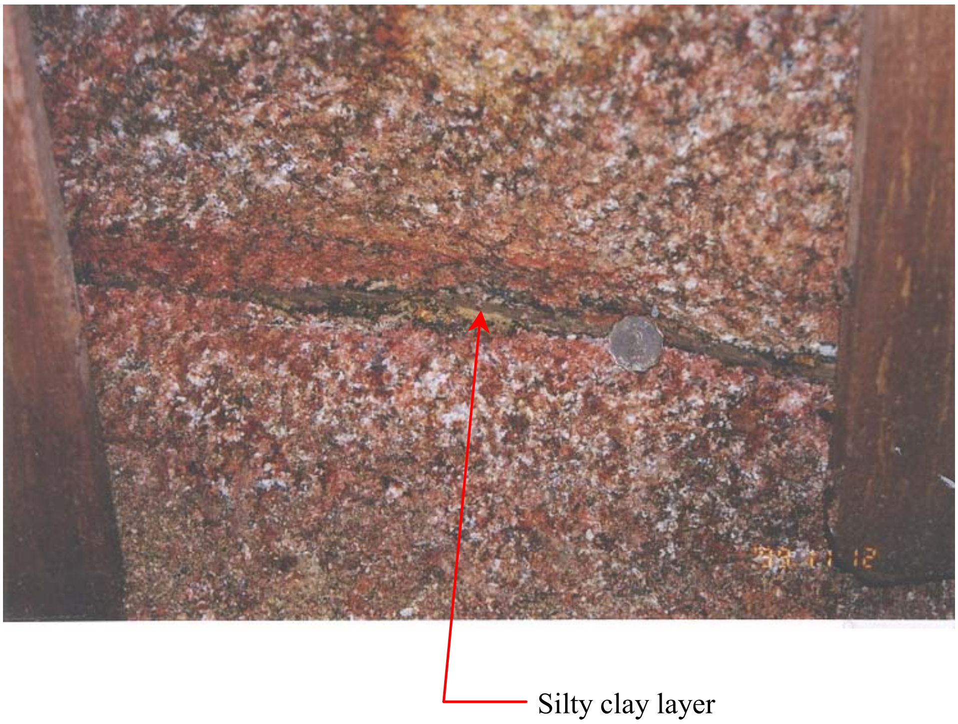

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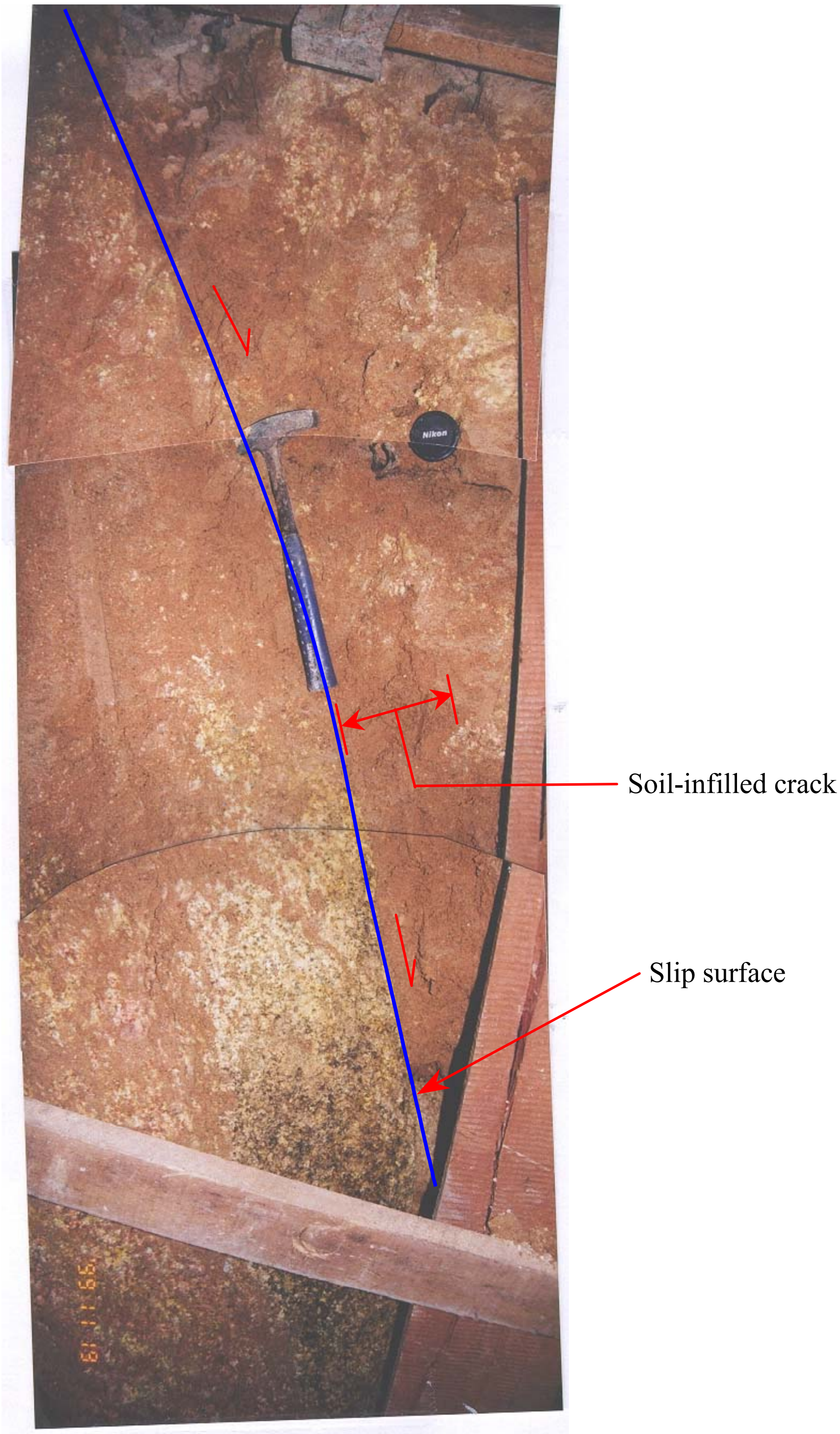

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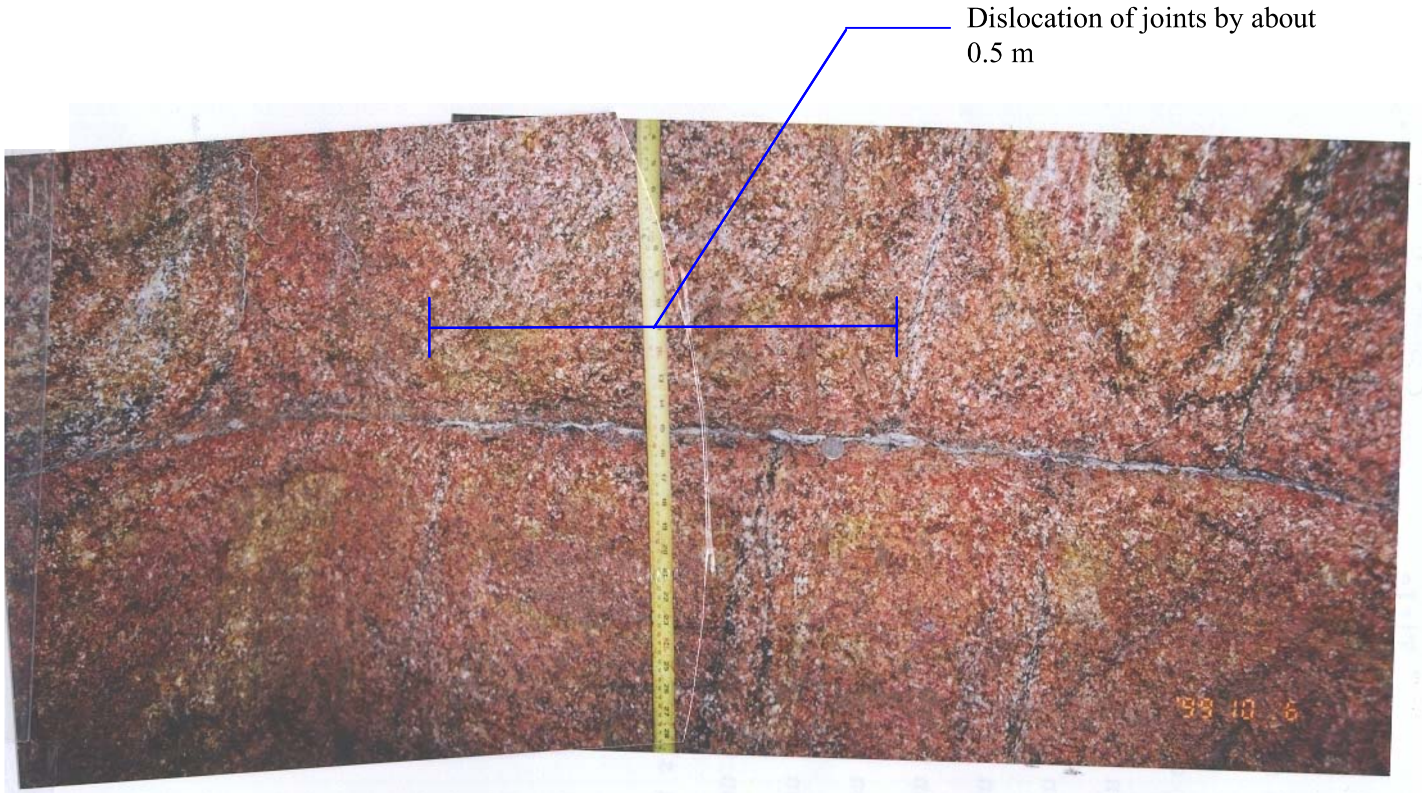

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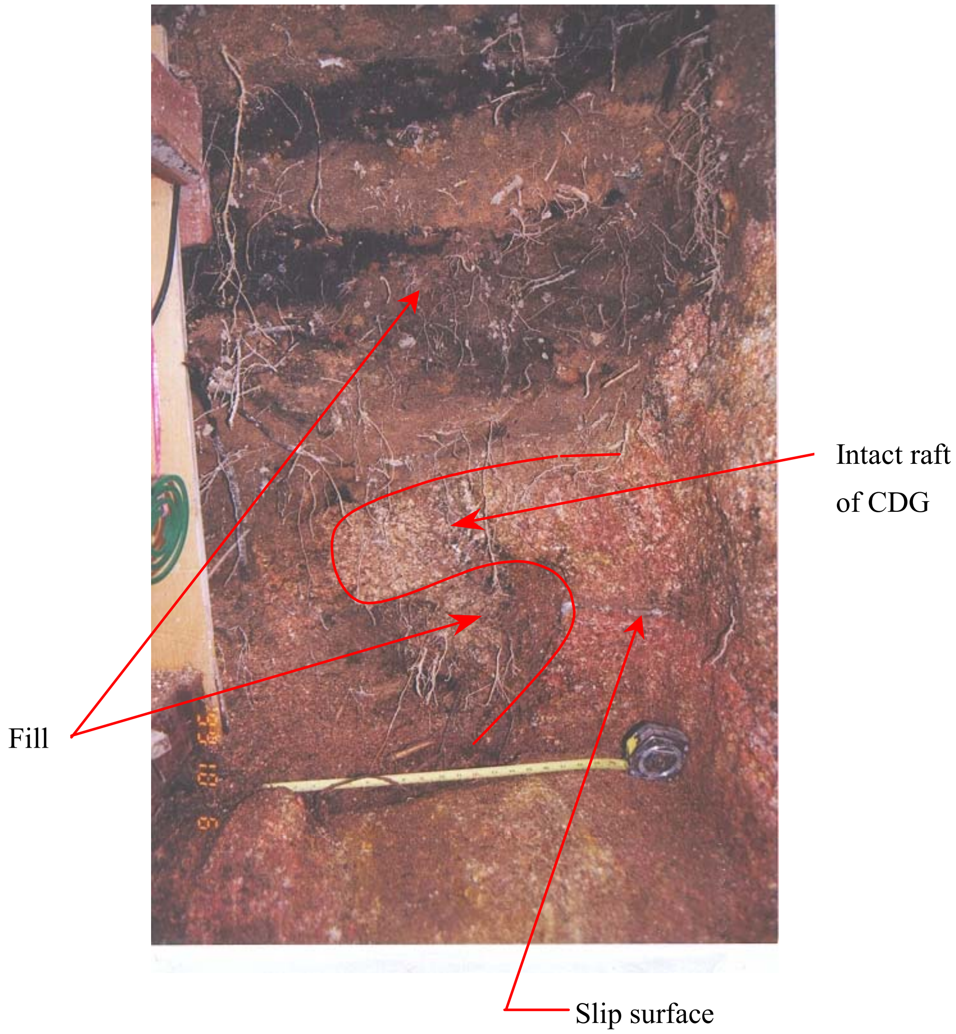

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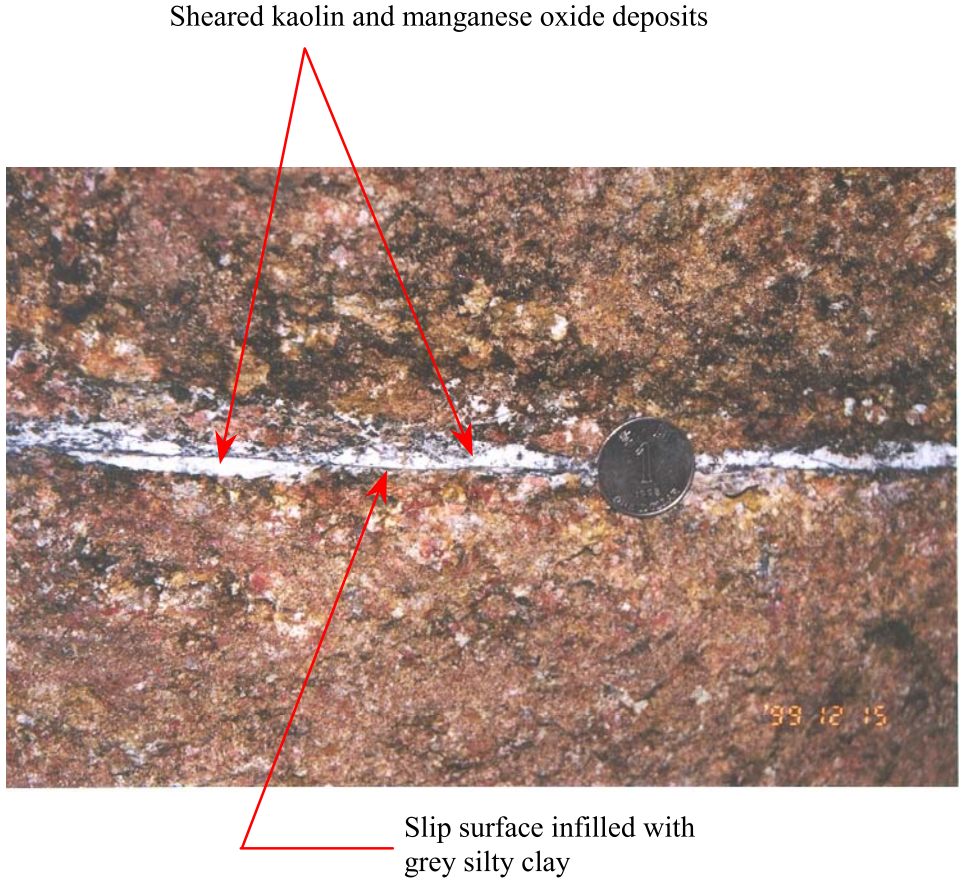

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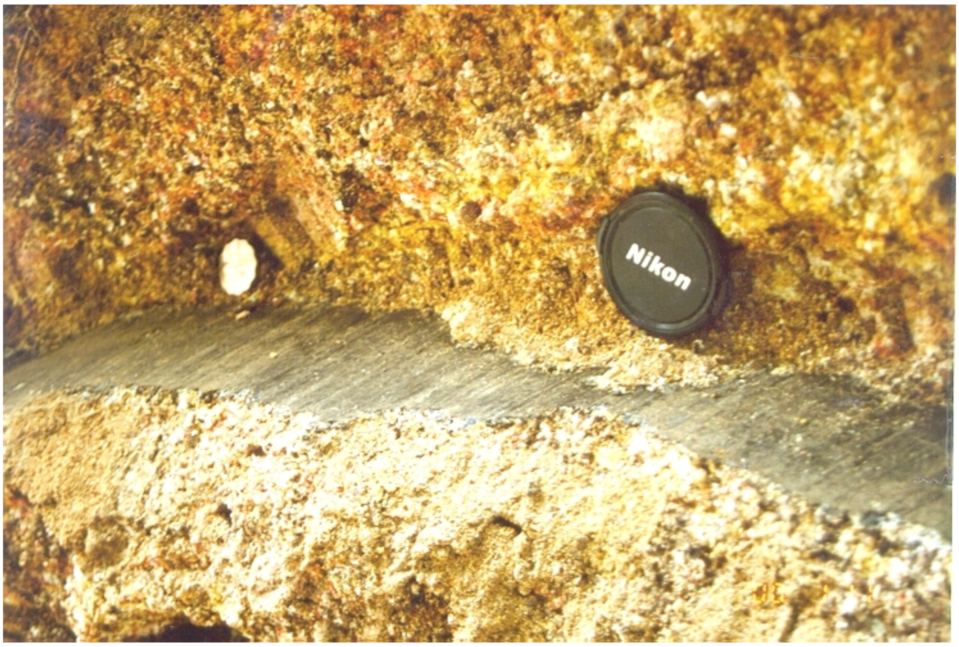

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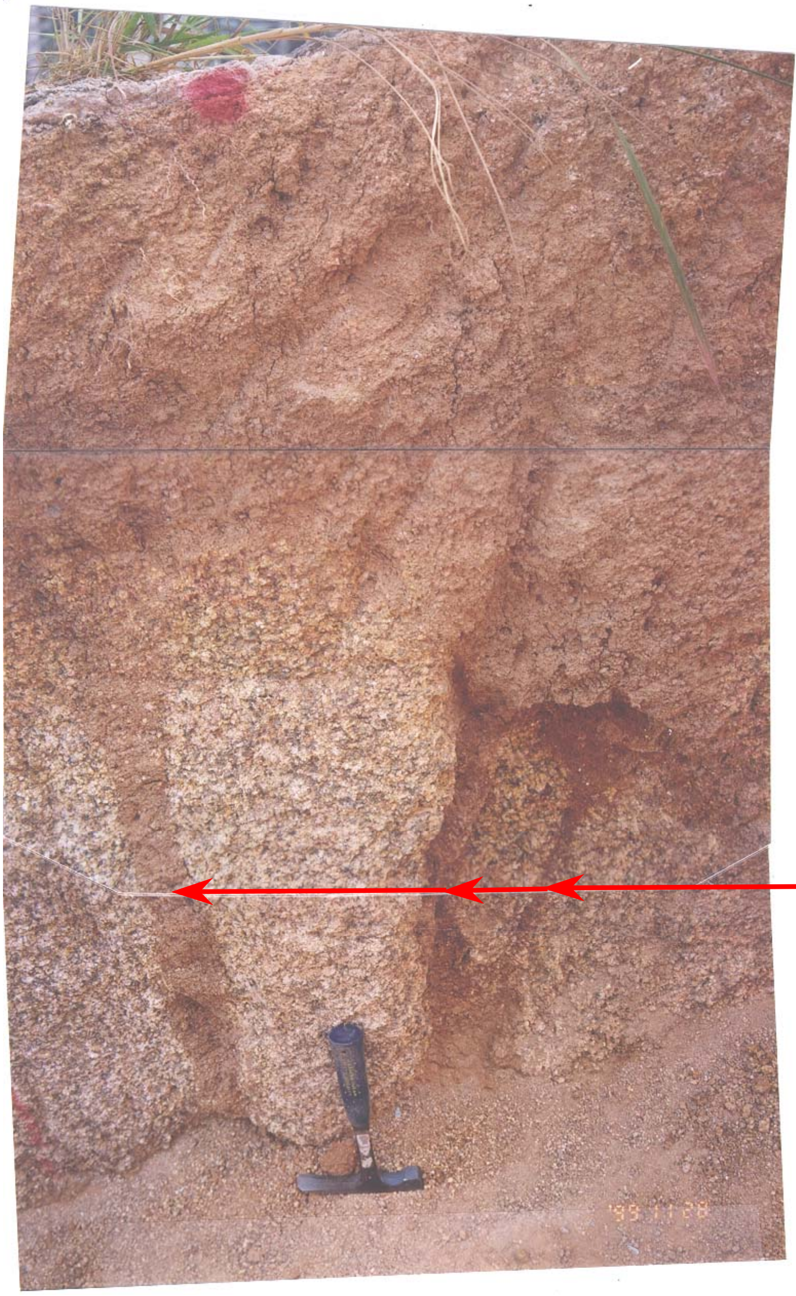

Pre-existing
soil-infilled
tension cracks

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APPENDIX A

SUMMARY OF SITE HISTORY

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A.1 INTRODUCTION

The history of the site has been determined from:

(a) old topographic maps,

(b) interpretation of a sequence of aerial photographs taken between 1945 and 1999, and

(c) documentary records from files from the GEO, the Lands Department and the Hong Kong Housing Authority (HKHA).

A plan showing the locations of various features discussed in this Appendix, including the slope registration numbers, is given in Figure 2 of the report. The parties responsible for the maintenance of features adjacent to slope No. 11NW-B/C90 are shown in Table A1.

Locations of references made in the following discussions are shown in Figures A1 to A5.

A.2 HISTORY OF SITE DEVELOPMENT

The earliest known available aerial photographs of the site were taken in 1945. At that time the Shek Kip Mei Estate was not constructed. The hillside to the north of the Shek Kip Mei Estate and to the east of Tai Po Road, was characterised by a rounded ridgeline trending north-south (see Section 6.2 of the report). The hillside was one of several similarly trending ridgelines in this area, which comprised many deeply incised drainage lines probably created by erosion due to surface runoff. The crest of the ridge was relatively flat up to the site of the radar station erected between 1963 and 1964. A photolineament associated with a low point on the ridge and coinciding with gullies to east and west was noted to the south of the WSD service reservoir now under construction (Figure A1).

Aerial Photos Nos. Y605–6 (20,000 ft)

Several areas on the hillside to the north of the Shek Kip Mei Estate appear to have been affected by erosion and landsliding. The most noteworthy of these, and of most relevance to the 1999 landslide incident at cut slope No. 11NW-B/C90 (Figure 2), is the large concavity at the rear of the current Block No. 36 of the Shek Kip Mei Estate (Figure A1).

The next available aerial photographs taken in 1949 are of better quality and resolution than those taken in 1945, and the main features are shown on Figure A1.

Aerial Photos Nos. Y1736-7 and Y1710-1 (8,000 ft)

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The concavity at the rear of the present-day Block No. 36 of the Shek Kip Mei Estate appears as a large depression which is probably a relic landslide scar. The depression cannot be confirmed on the 1945 aerial photographs due to their poor resolution. The western margin of the depression is represented by a steep, linear face trending north northeast-south southwest. This linear feature probably corresponds to the main area of outcropping, tabular and laterally-extensive corestones (referred to as ‘coreslabs’) identified to the northwest of the temple on the present-day slope configuration. The northern and eastern margins of the concavity formed a generally south-facing slope below a small spur. The spur appeared to have been composed predominantly of soil and was delineated to the north by a drainage line leading to the northern portion of the present-day slope No.11NW-B/C90 behind Block No. 36. Two drainage lines were also located above the intersection between the western and northern margins of the relic landslide scar, one extended upslope towards a small landslide scar and a minor one across the coreslabs along the western margin.

Below the margins of the relic landslide scar the area was terraced for agricultural purposes (Figure A1). Several boulders were noted within the terraces below the drainage line which intersected the western margin. The geometry of the scar at this time suggests that the direction of landslide debris movement was from north to south.

Some small buildings were noted along the toe of the southeast and west portions of the hillside.

By November 1954 Blocks Nos. 36 to 41 (present-day numbering) of Shek Kip Mei Estate had been constructed. The areas now occupied by cut slopes Nos. 11NW-B/C574 and 11NW-B/C68 (Figure 2) had been trimmed. The hillside behind Blocks Nos. 36 and 38 was mostly obscured by shadow, and as such it is not clear as to the extent of the excavation involved in this area.

Aerial Photos Nos. Y2685-6 (29,200 ft)

The 1956 photographs are of better resolution than those of 1954, however the slopes to the west of Blocks Nos. 36 and 38 were partially obscured by shadow and it is unclear as to the extent of cutting to slope No. 11NW-B/C90. However, the northern and southern portions of cut slope No. 11NW-B/C90 appeared to have been partially formed in 1956 (Figure A2).

Aerial Photos Nos. Y3368-9 (16,700 ft)

The small spur, which formed the northern and eastern margins of the large concavity, had been mostly excavated to accommodate Block No. 36 (Figure A1). The spur was flanked on either side by drainage lines, which were slightly cut back by the excavation. The positions of the drainage lines in relation to the recent survey plan of the area are shown in Figure A2. The drainage line to the north of the relic landslide scar was partially excavated during the formation of slope No. 11NW-B/C90.

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The upper portion of the concavity did not appear to have been significantly altered following the construction of Blocks Nos. 36 and 38. However, the lower portion of the concavity and the terraces had been substantially excavated to accommodate Blocks Nos. 36 and 38. As such, cut slopes had also been formed opposite Block No. 38 and the southwest portion of Block No. 36 (both of which form part of slope No. 11NW-B/C90). The two Blocks were divided by a trimmed portion of the original slope, which dipped towards Pak Tin Street. Prior to slope formation this area was mostly buried beneath the terraces.

The 1959 photographs show that cut slope formation was still on-going at slopes Nos. 11NW-B/C68 and 11NW-B/C90.

Aerial Photos Nos. Y4619-20 (40,000 ft)

By 1961 the southern portion of slope No. 11NW-B/C91 had been trimmed (Figure A2). The northeastern portion of slope No. 11NW-B/C90 appeared to have been formed and it could have already been at this stage in 1959. This portion of slope comprised four to five batters (the lowest batter was in shadow). The southern portion also appeared to have been formed with a complex set of slope batters and berms. The extent of surface protection applied to this slope cannot be clearly discerned.

Aerial Photos Nos. Y4908-9 (30,000 ft)

The photographs taken in January 1963 are of excellent quality and clearly show a series of drainage channels and catchpits on cut slopes Nos. 11NW-B/C68 and 11NW-B/C90. Both cut slopes appear to be covered with surface protection, probably chunam. The northern portion of slope No. 11NW-B/C90 comprised five batters that can be clearly seen. Slope No. 11NW-B/C91 was still under formation with excavation on going. The Kow Kong Commercial Association School building was also under construction to the east of slope No. 11NW-B/C91.

Aerial Photos Nos. Y8705-6 (3,900 ft) and Y8021-2 (2,700 ft)

In 1964 slope No. 11NW-B/C91 had been formed (Figure A2) and appeared to be covered with surface protection. A radar station was erected along the main ridgeline to the south of the east-west trending photolineament (Figure A1).

Aerial Photos Nos. Y12928-9 (12,500 ft)

In 1967 the Saviour Lutheran School was under construction at the toe of cut slope No. 11NW-B/C67.

Aerial Photos Nos. Y13388-9 (6,250 ft)

In 1968 and 1969 some deterioration of the chunam surfaces and very thin and scattered vegetation were noted on slopes Nos. 11NW-B/C68 and 11NW-B/C90.

Aerial Photos Nos. Y14251-2, Y14266-7, Y14926-7 and Y14973-4 (Flying height unknown)

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A landslide scar was noted from the 1973 photographs towards the top of slope No. 11NW-B/C91 (Figure A2). Debris comprising mostly scattered boulders was deposited at the toe of the slope on the open platform. The scar was located immediately adjacent to an older and slightly larger landslide scar, which may have occurred during cut slope formation. At this time slope No. 11NW-B/C90 was covered by chunam with patchy vegetation.

Aerial Photos Nos. 6891-2 (3,000 ft) and 8010-1 (12,500 ft)

By 1974 parts of slope No. 11NW-B/C68 had been cleared of vegetation and resurfaced. There had been further excavation at the top of slope No. 11NW-B/C91 following the failure noted in 1973 (Figure A2), and at the top of the southern portion of slope No. 11NW-B/C91 adjacent to slope No. 11NW-B/C90. The chunam appeared to have been resurfaced within the concavity immediately to the south of the northern portion of slope No. 11NW-B/C90 affected by the 1999 instability (Figure A2). Vegetation comprising bushes covered the upper portion of the northern part of slope No. 11NW-B/C90. The present temple structure appeared to have been constructed in the form of a small hut on the platform between Blocks Nos. 36 and 38.

Aerial Photos Nos. 9340-1 (3,000 ft) and 9782-3 (12,500 ft)

In 1975 the small hut (i.e. the initial stage of the temple structure) was extended towards the southeast and northeast.

Aerial Photos Nos. 11502-3 (2,500 ft) and 11992-3 (12,500 ft)

In 1976 some minor repair works had been completed to a small portion of the second batter on slope No. 11NW-B/C68 (Figure A2).

Aerial Photos Nos. 14759-60 (2,500 ft) and 15961-2 (12,500 ft)

Between 1976 and January 1978 there were no significant changes to the hillside.

Aerial Photos Nos. 20746-7 (12,500 ft)

By December 1978 major slope work had begun on the hillside behind Blocks Nos. 36 and 38. Slope No. 11NW-B/C68 had been trimmed, resurfaced and a staircase was under construction, which zigzagged upslope towards the ridgeline. New surface cover was also applied to the upper portion of slope No. 11NW-B/C90 and within part of the concavity to the west of Block No. 36. Slopes Nos. 11NW-B/C585 and 11NW-B/C584 (i.e. the present cultivation area shown in Figure 2) were under formation involving cutting to the natural hillside above slopes Nos. 11NW-B/C90 and 11NW-B/C91 (Figure A2).

Aerial Photos Nos. 24125-6 (4,000 ft)

Slope work appeared to have been started on slope No. 11NW-B/C67 as indicated by scaffolding erected on portions of the slope at the rear of the Saviour Lutheran School.

The ridgeline was also being re-formed/excavated, as was the hillside above slope No. 11NW-B/C66, which was being cut back to form slope No. 11NW-B/C232.

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The temple structure was extended slightly to the southeast along rocky outcrop and some slope works (involving trimming, possibly some local filling and application of surface protection) were noted in the area to northeast of the temple structure.

By 1979 the formation of slope No. 11NW-B/C232 and the works at slope No. 11NW-B/C68 were mostly completed. Slope No. 11NW-B/C585 was further trimmed above slope No. 11NW-B/C91 towards the ridgeline. The toe of slope No. 11NW-B/C90 opposite the southwestern part of Block No. 36 and the slope portion to the west of Block No. 38 (Figure A2) were darker in tone than the rest of the slope and were unlikely to have been affected by these slope works. The surface of slope No. 11NW-B/C584 was covered with grass at this time. The radar station was situated at the northern end of a paved platform, which was formed along the ridge above slopes Nos. 11NW-B/C581, 11NW-B/C583, 11NW-B/C584 and 11NW-B/C232.

Aerial Photos Nos. 26699-27000 (6,000 ft) and 28073-4 (10,000 ft)

In 1980 scaffolding was no longer apparent on slope No. 11NW-B/C67, and the slope works being carried out in 1979 appeared to be complete. The configuration of slopes Nos. 11NW-B/C90, 11NW-B/C91, 11NW-B/C583, 11NW-B/C584, 11NW-B/C585, 11NW-B/C586, 11NW-B/C575, 11NW-B/C68, 11NW-B/C67, 11NW-B/C66 and 11NW-B/C232 (Figure 2) were very similar to that noted prior to the landslide incident in 1999.

Aerial Photos Nos. 30117-8 (4,000 ft) and 33469-70 (10,000 ft)

Between 1982 and 1990 there was a small increase in vegetation cover on slope No. 11NW-B/C90 (Figure A3).

Aerial Photos Nos. 44533-4 (10,000 ft), 47243-4 (20,000 ft), 51409-10 (10,000 ft), 54010-1 (4,000 ft), 56987-8 (4,000 ft), A01760-1 (10,000 ft), 67185-6 (10,000 ft), A04469-70 (4,000 ft), A06284-5 (4,000 ft), A08192-3 (10,000 ft), A09508-9 (4,000 ft), A09536-7 (4,000 ft), A12165-6 (10,000 ft), A13700-1 (10,000 ft), A14662-3 (4,000 ft), A19634-5 (10,000 ft), A20916-7 (4,000 ft) and A23635-6 (4,000 ft)

In April 1992 the vegetation on slope No. 11NW-B/C90 appeared to be dense, especially in the concavity opposite Block No. 36 and to the north of the temple. Trees were noted on the northwest portion of slope No. 11NW-B/C584, which was mostly covered with grass and a few bushes (Figure A3).

Aerial Photos Nos. A30036-7 (10,000 ft) and A30444-5 (4,000 ft)

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By August 1992 a portion of the lower part of the slope at the rear of Block No. 38 had been resurfaced (Figure A2), probably with shotcrete. No landslide scars were observed at this portion.

Aerial Photos Nos. CN3108-9 (3,000 ft) and A33225-6 (10,000 ft)

In 1994, slopes Nos. 11NW-B/C583 and 11NW-B/C584 and the uppermost part of slope No. 11NW-B/C232 remained mostly covered with grass and a few bushes (Figure A4). More tree growth was noted on the northwest portions of the slope. The vegetation within the drainage lines on slope No. 11NW-B/C90 was more dense (Figure A4).

Aerial Photos Nos. A38078-9 (5,000 ft), A39255-6 (4,000 ft) and CN6163-4 (10,000 ft)

By September 1995 excavation into the hilltop area for the construction of the WSD service reservoir had started. Also, a platform was under construction next to the temple at this time. Several small structures were erected along the area to the northeast of the temple structure, and some steps had been constructed above this area.

Aerial Photos Nos. CN11323-4 (3,500 ft) and CN12271-2 (10,000 ft)

In 1996 slopes Nos. 11NW-B/C583, 11NW-B/C584 and the uppermost part of slope No. 11NW-B/C232 were mostly covered with dense trees, although a bare patch of soil was noted at the end of the staircase (Figure A4). Unauthorized cultivation of these slopes (as seen following the August 1999 instability) probably started between 1995 and 1996 when the ground surface was mostly obscured by tree cover. The platform to the northeast of the temple structure was formed and a small wall erected on the upslope side.

Aerial Photos Nos. CN13549-50 (4,000 ft), CN14732-3 (5,000 ft), CN15583-4 (5,000 ft) and CN15737-8 (5,000 ft)

Between 1997 and 1999 the vegetation covering slope No. 11NW-B/C90 became progressively denser, particularly within the drainage lines, and more extensive in areal terms (Figure A4). On the photographs taken in November 1997, one to two cracks were noted in the slope surface cover at the north side of the crest of the southern distressed zone of the August 1999 instability, trending northeast-southwest for a few metres (Figure A2).

Aerial Photos Nos. CN17246-7 (4,000 ft), CN18909-10 (4,000 ft), CN20730-1 (3,500 ft), CN22189-90 (4,000 ft) and CN22366-7 (5,500 ft)

Photographs taken in October 1998 showed two cracks located in the slope surface cover at similar positions as those noted in 1997.

A.3 PREVIOUS ASSESSMENTS

A.3.1 Slope Registration in the 1977/78 Catalogue of Slopes

In May 1977, the cut slope in which the 1999 landslide incident occurred was registered as slope No. 11NW-B/C90 by the consultant, Binnie & Partners (Hong Kong) (B&P), engaged by the Government to prepare a catalogue of sizeable man-made cut slopes, fill slopes and retaining walls (now commonly known as the 1977/78 Catalogue of Slopes). Then inspection report recorded that no seepage was observed at that time.

B&P, 1977

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A.3.2 North Kowloon Landslide Study

In November 1973, B&P was commissioned by the Public Works Department to report on slope stability in the North Kowloon area covering, inter alia, Resettlement Estates which included the present slope No. 11NW-B/C90 and the natural hillside above. The report stated that "Although the slope appears in good condition, its toe is very close to the buildings and it should be inspected after heavy rainfall. A phase II stability analysis is needed". B&P, 1973

A.3.3 SWKP's Area Geotechnical Study

In June 1973, the Highways Office (renamed Highways Department, HyD, in 1989) engaged Scott Wilson Kirkpatrick & Partners (SWKP) to carry out a geotechnical study of the slopes in the Shek Kip Mei area, including slope No. 11NW-B/C90. Appendix I (SWKP, 1977)

SWKP carried out site visits but "owing to its low priority and non-availability of funds in the financial year 1974/75, the investigation was discontinued". In December 1975, SWKP resumed the study at the request of the Highways Office. In January 1977, SWKP prepared a geotechnical report on the study which comprised site inspections and stability analyses on selected slopes. The report stated that "No sub-soil investigation was carried out as site inspection indicated that the soil contained numerous random boulders". Section 2 (SWKP, 1977) Section 4 (SWKP, 1977)

Under the "Soil Slopes" section, the report discussed the results of stability analyses on slopes Nos. 11NW-B/C66 and 11NW-B/C91 together with the natural hillside above (Figure 2). Section 5 (SWKP, 1977)

For slope No. 11NW-B/C66 and the natural hillside above, the report stated that with the assumptions of dry condition and an angle of internal friction of 35° for completely decomposed granite, results of back analyses indicated that a "minimum "cohesion" of some 520 1b/ft^2 [i.e. about 25 kPa] would be required to maintain a factor of safety of unity. We [SWKP] believe that if the slopes are protected from ingress of water, the soil masses will have a pseudo-cohesion (which includes negative pore-pressure or soil suction) in excess of the above value. However, as the foot of this slope is extremely close to the rear wall of the Magistracy we [SWKP] consider it advisable to cut the top of the slope to increase the stability of the hillside". The report recommended that "The upper section of the slopes behind the Magistracy ……. require cutting to slopes of about 40°, complete with 5-ft wide berms, chunam surfacing and drainage channels". Section 5.1 (SWKP, 1977) Section 10 (SWKP, 1977)

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For slope No. 11NW-B/C91 and the natural hillside above, the report stated that "Analysis of the western soil slope above the bed rock indicates that the stability of the slope needs improving" and "water seepage is evident at the rock/soil interface". The report recommended that "the upper section of the slopes behind ……. the Kow Kwong School require cutting to slopes of about 40°, complete with 5 ft wide berms, chunam surfacing and drainage channels" and "horizontal drainage holes of minimum length 15 ft (but penetrating the rock mass by at least 10 ft), inclined up at some 5° from the horizontal should be provided at a level 3 ft above the first bench (i.e. around 3 ft above soil/rock interface). The spacing of the drains should initially be 10-15 ft and additional holes provided where excessive seepage is found (either in the dry or rainy seasons)".

Section 5.2 (SWKP, 1977)

Section 10 (SWKP, 1977)

For slope No. 11NW-B/C90, the report presented two cross-sections, one at the northern portion of the slope and one near the central portion through the temple. No stability analysis on the slope was presented in the report but a "probable potential critical slip surface" at about 12 m below the slope surface was indicated on the northern cross-section. The report stated that the slope is "generally flatter than the slopes analysed [i.e. slopes Nos. 11NW-B/C66 and 11NW-B/C91, together with the natural hillside above]" and "do not form potential hazards". The report did not recommend any specific works to this slope except that two "future drainage extension" at the natural hillside above the cut slope were shown on the Remedial Works Plan for the study area. The report stated that "it is not considered that this need be built in the immediate future. However, a close watch of these slopes is necessary and improved drainage/protective works should be constructed as and when significant erosion is apparent".

Section 5.3 & Figures 4 & 5 (SWKP, 1977)

Section 9 & Figure 8 (SWKP, 1977)

The report provided the following general recommendations which were relevant to slope No. 11NW-B/C90:

Section 10 (SWKP, 1977)

(a) "all exposed corestones and loose rock should be removed or stabilised",

(b) "all existing chunam surfacing should be examined in detail and any sub-standard material renovated",

(c) "the summit of the hill requires regrading and the platform so formed turfed and, possibly, "shrubbed". If maintenance of this vegetation is likely to be a problem, the area could be sealed by either a 3-in thick concrete slab or 2-in thick bituminous carpet",

(d) "the hydraulic properties of all existing channels and cascades should be reassessed and, where necessary, increased in size

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to discharge a 50-year storm",

(e) "slope drainage and protection, including backfilling of gulleys, should be extended as necessary as erosion progresses", and

(f) "the importance of adequate maintenance of the chunamed, turfed or exposed rock slopes, and drainage channels and structures cannot be over-emphasised. To facilitate maintenance adequate access should be provided".

A.3.4 B&P's Phase IIC Landslide Study

In January 1978, B&P prepared a report, under the Phase IIC Landslide Study commissioned by the Geotechnical Control Office (GCO, renamed Geotechnical Engineering Office (GEO) in 1991), on the stability of slopes in the Shek Kip Mei area, including slope No. 11NW-B/C90. B&P, 1978

The study comprised field inspections, ground investigation (5 boreholes and 4 trial pits), rock joints survey for rock slopes, examination of aerial photographs and records in government departments, together with stability analyses on selected slopes. One of the boreholes (No. SKM-5) was drilled in the area above slope No. 11NW-B/C90 (see Figure 16 of the report) but no ground investigation was carried out on this slope. Section 2.2 & Appendix II (B&P, 1978)

The report stated that the cut slopes between the North Kowloon Magistracy and Kow Kong Commercial Association School (i.e. slopes Nos. 11NW-B/C66, 11NW-B/C67, 11NW-B/C68, 11NW-B/C90 and 11NW-B/C91 (Figure 2)) and the natural hillside above had been studied by SWKP in 1977 and "In general, we [B&P] endorse the recommendations made in that [SWKP's] report". Section 2.6 (B&P, 1978)

The report presented results of stability analyses on slopes Nos. 11NW-B/C68 and 11NW-B/C91. Sections 6.3 & 7.3 (B&P, 1978)

For slope No. 11NW-B/C68, the report noted that "The present slope is very close to the original shape of the hillside" and "The face is about 80 m wide and 50 m high with an average slope of 32° and appears to have been trimmed before a complete cover of chunam was placed" and commented that "The chunam has cracked in many places but regular maintenance work appears to have been carried out leaving the slope in fair condition". Section 6.1 (B&P, 1978) The results of stability analyses, with assumptions of $c' = 0$, $\phi' = 38°$ for completely decomposed granite (CDG) and no soil suction, indicated that the critical slip surface at about 5 m below the slope surface had a factor of safety of 1.38. The factor of safety would Section 6.3 (B&P, 1978)

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increase to 1.40 with an assumed soil suction of 0.7 kPa. Dry condition was adopted in the stability analyses as "The ground water level appears to be well below the [slip] surfaces considered". The report recommended that "All cracks in the chunam should be repaired" and "Weepholes should be installed in the lower third of the slope and all U channels should be cleared out and repaired". Section 6.4 (B&P, 1978)

For slope No. 11NW-B/C91, the report stated that "Three factors combine to make this a particularly dangerous slope, these are:- Section 7.1 (B&P, 1978)

(i) seepage in the grade III granite,

(ii) the abnormally high (37 m) cut in grade III to V granite,

(iii) the steep angle (35°) of the natural hill side above the cut".

The report further commented that the "chunam is in poor condition and small failures have occurred near the top". Stability analyses were carried out on a cross-section comprising about 17 m of grade IV/V rock overlying layers of grade III and grade II rocks. The report stated that "Evidence from borehole SKM5 and seepage at the surface indicate that there may be a water table along the granite grade II rock. The rise in groundwater level, due to a wetting band from an extreme storm will probably remain within the thickness of the grade III rock". The results of the analyses, with the assumptions of no soil suction, $c' = 0$, $\phi' = 38°$ for the grade V rock and groundwater table below the soil mantle, indicated that the minimum factor of safety was 0.61. For a factor of safety of 1.2, a soil suction of 34 kPa would be required. The report recommended that "The required suction can be reduced by cutting back the slope sufficiently to provide drainage berms at 8 m vertical intervals and an overall of 54°. Chunam protection immediately above the top of the slope will prevent saturation of the most critical failure surface by a descending wetting band. Psychrometers should be installed to check that the required soil suction, 29 kPa, is maintained". Section 7.3 (B&P, 1978)

For slope No. 11NW-B/C90, the report stated that no seepage could be observed on the slope and "Generally the formation of the slope is similar to C68 [slope No. 11NW-B/C68] except that the trimming at the foot of the slope has been more extensive exposing an uneven surface of grade III rock. In the northern half there is a cut 15 m high 25 m long with four intermediate berms and slopes of 60°. Since the rock exposed at the toe is mainly grade III, with persistent and extensive bands of grade II, the arrangement of the rock joints controls the stability. The principal sets of joints are near vertical with a dip direction into the slope and no other joints combine with them to make any potential failure mechanisms". Sections 4.7 & 6.2 (B&P, 1978) The report did not present any stability analysis for this slope. The report recommended that "All cracks in the chunam should Sections 6.4 (B&P, 1978)

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be repaired", "Weepholes should be installed in the lower third of the slope and all U channels should be cleared out and repaired" and "Major trimming and protection works will be required above slope C90 [slope No. 11NW-B/C90]".

A.3.5 GCO Planning Division Stage 1 Study

The GEO is responsible for carrying out a preliminary geotechnical assessment of catalogued man-made slopes and retaining walls, which were formed before the GCO (renamed GEO in 1991) was established, in accordance with a risk-based priority ranking system under the Landslip Preventive Measures (LPM) Programme. The purpose of the preliminary geotechnical assessment is to establish whether further action (such as detailed studies, urgent repair works, service of a Dangerous Hillside Order, etc.) is warranted.

A 'Stage 1 Studies Summary Report' was prepared for slope No. 11NW-B/C90 in September 1985 by the Planning Division of the GCO during the stage of selecting slopes for preliminary geotechnical assessments. The Summary Report made reference to the studies by SWKP and B&P (see Sections A.3.3 and A.3.4) and noted that a Stage 1 Study (i.e. preliminary geotechnical assessment) will not be carried out for this slope in view of the past studies and that it was "Checked by GCO/GCB in November 1977".

GCO, 1985

The history of past actions on slope No. 11NW-B/C90 was further reviewed by the Mainland East Division of the GCO in December 1986. Having cognizance to the past geotechnical studies by SWKP in 1977 and B&P in 1978 and the subsequent slope works by the Highways Office following these studies, the file recorded that "No further action is thus required".

GEO
File ref. GCMW 2/E1/11NW-B/C90

A.3.6 Systematic Inspection of Features in the Territory

In mid-1992, the GEO commenced a consultancy agreement entitled "Systematic Inspection of Features in the Territory" (SIFT), to search systematically for features not included in the 1977/78 Catalogue of Slopes and to update information on previously registered features, by studying aerial photographs together with limited site inspections. In August 1995, the SIFT study assigned the slope No. 11NW-B/C90 to Class 'C1', i.e. a slope "formed or substantially modified before 30 June 1978".

GEO, 1995B

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A.3.7 Systematic Identification and Registration of Slopes in the Territory

The GEO commenced a consultancy agreement entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST), in July 1994 to update the 1977/78 Catalogue of Slopes and to prepare the New Catalogue of Slopes. The GEO's consultants for the SIRST project inspected slope No. 11NW-B/C90 in September 1996. The SIRST report recorded the presence of fine cracks in the chunam cover at the mid-portion and toe of the slope. The report also recorded signs of seepage and that stepped channels were cracked.

GEO, 1996

A.3.8 Inspections by the HKHA

The HKHA commissioned consultants to carry out Annual Geotechnical Inspections (AGI) in 1980, 1982, 1984 and 1986 on slope No. 11NW-B/C90. After the establishment of the Vesting Order (VO) boundary for Shek Kip Mei Estate in June 1985 by the Lands Department, HKHA's consultants have carried out a Special Area Inspection (SAI) in 1993 and AGI in 1995, 1997 and 1998, on the lower part of slope No. 11NW-B/C90, as well as on slopes within the VO boundary.

B&P, 1980, 1982 & 1984, D&M, 1986, FH, 1994, GW, 1996 and ACL, 1997 & 1998

During these inspections, disrupted chunam cover, disrupted drainage channels and unplanned vegetation were observed. Details of the observations and recommendations presented in the inspection reports are summarised in Table A2. The key observations made are summarised in Figure A5.

A.3.9 Design of the Shek Kip Mei No. 2 Fresh Water Service Reservoir

Advice on the geotechnical design of the reservoir was provided to Water Supplies Department (WSD) by the GEO's Advisory Division. The effect of construction on the cut slopes and hillside in the vicinity of the reservoir site was considered. The cut slopes adjacent to the Shek Kip Mei Estate, which are about 200 m away from the reservoir site, were considered unlikely to affect or be affected by the construction of the service reservoir and, hence, were not assessed. According to WSD, no unusual geological or hydrogeological features were encountered during the excavation.

GEO
File ref.
GCA2/C7/82

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A.4 PREVIOUS SLOPE WORKS

A.4.1 Previous Slope Works by the Highways Office

The Highways Office prepared a set of drawings for the proposed slope works at the Shek Kip Mei area. The proposed works presented in the drawings were accepted by the GCO in November 1977. The works were subsequently carried out under Public Works Department Contract No. 668 of 1977. The works that were relevant to slope No. 11NW-B/C90 and its immediately adjacent area included the following:

GEO File ref. GCMW 2/B2/7

Highways Office (Kowloon Div.), 1980

(a) partial replacement/repair of existing chunam surface channels, removal of vegetation and clearing of surface channels on slope No. 11NW-B/C90,

(b) removal of exposed corestones and boulders on the hillside above the cut slope,

(c) regrading of the hillside above the cut slope to the present-day profile (the regraded slope was registered as slopes Nos. 11NW-B/C584 and 11NW-B/C585 by the SIRST consultants in 1997), and

(d) provision of surface channels, chunam surface protection (approximate extent covering the present slope No. 11NW-B/C585) and tuffing (approximate extent covering the present slope No. 11NW-B/C584) on the regraded slope.

The works carried out covered all the items of works recommended by SWKP (1977) and B&P (1978).

A.4.2 Previous Slope Maintenance Works

Based on HKHA's records, general slope maintenance works were carried out on slope No. 11NW-B/C90 in 1981, 1983, 1984 and 1985 before the VO boundary for Shek Kip Mei Estate was vested in the HKHA in June 1985 by the Lands Department. In 1991 and 1992, slope maintenance works were carried out at the lower part of the slope behind Block No. 38.

HKHA Maintenance Division File ref. 5/42/1/26, (B&P, 1982), (B&P, 1984) and (D&M, 1986)

Following an inspection by in-house staff, HD referred their observation of "minor slope defects" at Slope Nos. 11NW-B/C90 and 11NW-B/C68 to the GEO in June 1993 to arrange "to repair these defects before the rainy season". Given that the slopes were subsequently confirmed by the District Lands Office (DLO) as being on unallocated Government land, the GEO requested DLO in August 1993 to "initiate

HD memo to GEO dated 10/6/1993

DLO, LD memo to GEO & copied to HD dated 5/8/1993

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the necessary action to carry out the maintenance works for the captioned slopes [Nos. 11NW-B/C90 and 11NW-B/C68], in accordance with the Category 5(f) of Appendix A in the Lands and Works Technical Circular No. 9/87". — GEO memo to DLO dated 12/8/1993

Recommendations on slope maintenance works made after the AGI carried out in 1997 and 1998 (see Section A.3.8) were referred to the DLO to initiate the necessary action. Lands Department wrote to HD on 1 June 1998 advising that HD was responsible for maintenance of slope No. 11NW-B/C90 according to the SIMAR project. — ACL letters to DLO dated 7/11/1997 & 6/1/1999; EM, LD memo to DLO, LD & copied to HD dated 1/6/1998

There are no records of any slope maintenance works having been carried out since 1993.

A.5 PREVIOUS INSTABILITY

On 6 October 1995, a portion of an exposed corestone of about 2 m^3 fell down from the central portion of the slope No. 11NW-B/C90. The detached rock rested on the platform north of the temple. The location of the corestone before the failure is shown in Figure 4 of the report. — GEO, 1995A

GEO inspected the failure on the same day and recorded that the failure was a "rock fall".

Slope reinstatement works, comprising covering the scar with chunam and removal of the fallen rock, were subsequently carried out by the HyD at the recommendation of the GEO.

A.6 REDEVELOPMENT PROGRAMME FOR SHEK KIP MEI PUBLIC HOUSING ESTATE

According to HD, part of Shek Kip Mei Estate (Blocks Nos. 1 to 18 and 25 to 41), together with other selected public housing estates, were included in the Comprehensive Redevelopment Programme since 1987 for clearance before 2005. Blocks Nos. 35 to 41 of the Shek Kip Mei Estate were first included in the '1997/1998 Five Year Redevelopment Programme' by HD in May 1997 for clearance in 2001/2002. Subsequent to the strong request from local residents for local reception at Pak Tin Estate, the redevelopment programme of Blocks Nos. 35 to 41 was revised in June 1999 under the '1999/2000 Five Year Redevelopment Programme' from 2001/2002 to 2003/2004.

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Table A1 - Maintenance Responsibilities of Registered Features Adjacent to Slope No. 11NW-B/C90

Registered Feature No.	Party Rresponsible for the Maintenance of the Feature	Maintenance Agent	'SIMAR' Endorsement Date
11NW-B/C63	Highways Department	Highways Department	3 March 1997
11NW-B/C64	Highways Department	Highways Department	3 March 1997
11NW-B/C65	Highways Department	Highways Department	3 March 1997
11NW-B/C66	Architectural Services Department	Architectural Services Department	7 Aprril 1997
11NW-B/C67	Architectural Services Department & Lands Department	Architectural Services Department & Lands Department	4 May 1999
11NW-B/C68	Lands Department & Urban Services Department	Architectural Services Department & Lands Department	4 May 1999
11NW-B/C91	Lands Department	Lands Department	2 June 1997
11NW-B/C97	Housing Department	Housing Department	7 Aprril 1997
11NW-B/C232	Architectural Services Department & Lands Department	Architectural Services Department & Lands Department	5 January 1998
11NW-B/C574	Urban Services Department	Architectural Services Department	7 Aprril 1997
11NW-B/C575	Housing Department	Housing Department	3 March 1997
11NW-B/C579	Lands Department	Lands Department	2 June 1997
11NW-B/C580	Lands Department	Lands Department	2 June 1997
11NW-B/C582	Water Supplies Department	Water Supplies Department	2 June 1997
11NW-B/C583	Water Supplies Department	Water Supplies Department	5 August 1997
11NW-B/C584	Lands Department	Lands Department	5 August 1997
11NW-B/C585	Lands Department	Lands Department	5 August 1997
11NW-B/C586	Housing Department	Housing Department	3 March 1997
11NW-B/F4	Housing Department	Housing Department	7 July 1997
11NW-B/R169	Water Supplies Department	Water Supplies Department	2 June 1997

Notes: (1) Maintenance responisblities of the above features are based on the "Systematic Indentification of Maintenance Responsibility of Slopes in the Territory" (SIMAR) commissioned by the Lands Department.
(2) See Figure 2 of the report for locations of the above registered features.

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 1 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
Annual Geotechnical Inspection (Inspection Date: January 1980) (Binnie & Partners (Hong Kong) (B&P), 1980)					
<u>Observations given in inspection report</u> - "Cracking with preferred orientation" in southern portion and mid-height of central portion of slope - "Seepage at toe" of northern portion - "Surface is overgrown with boulders and rubbish" in lower central portion **<u>Observations interpreted by FMSW</u>** - Unplanned vegetation on slope surface in lower central portion	Photos Nos. 22a,27a,26a, 28a,23a & 24a (B&P,1980) Photo No. 23a (B&P,1980)	- Improve access - Seal cracks to act as telltales - Clear blocked channels	Sheet 1 (B&P,1980)	- Inspection covers the full extent of slope No. 11NW-B/C90	Sheet 2 (B&P,1980)
Annual Geotechnical Inspection (Inspection Date: June 1982) (Binnie & Partners (Hong Kong) (B&P), 1982)					
<u>Observations given in inspection report</u> - "Berm channel of inadequate capacity" in upper central portion - "Erosion on slope surface" in upper southern portion <u>Lower southern portion</u> - "Local disruption of chunam surface due to root action" - "Disrupted stepped channel on berm" - "Disruption at toe surface" - "Disruption of chunamed surface" - "Minor vegetation behind stepped channel" - "Disruption of chunam, minor vegetation and boulders on surface" - "Minor vegetation and disruption of chunamed surface"	Photos Nos. 6a,14a,24a, 25a,26a,7a, 8a, 9a,13a,10a, 12a,16a, 21a,27a,15a, 17a,18a,20a, 22a, 23a & 28a (B&P,1982)	- Repair disrupted chunamed surfaces - Seal cracks in chunam & concrete - Provide concrete surfacing - Clear channels - Improve access for inspection - Remove random vegetation - Remove loose boulders and debris on slope surface - Monitor the movement of cracks in chunam surface	Sheet 1 (B&P,1982)	- Inspection covers the full extent of slope No. 11NW-B/C90 - The report stated that "The slope [No. 11NW-B/C90] has been resurfaced in December 1981"	Sheet 1 (B&P,1982) Sheet 2 (B&P, 1982)

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 2 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
<u>Lower northern portion</u> - "Toe channel disrupted" - "Stepped channel disrupted at toe" - "Toe channel ….. berm channel blocked by debris" - "Minor failure scar" <u>Lower central portion</u> - "U-channel ……. toe channel blocked by debris" - "Loose block under temple" - "Toe channel cracked" - "Debris and vegetation behind toe wall" - "Dense vegetation- unable to inspect" **<u>Observations interpreted by FMSW</u>** - Repaired cracks on chunam surface in lower central portion - Patches of dark tone on chunam in lower northern portion - Unplanned vegetation in lower central portion	Photos Nos. 2a,4a,19a, 20a,30a,36a & 37a (B&P, 1982).				
Annual Geotechnical Inspection (Inspection Date: June 1984) (Binnie & Partners (Hong Kong) (B&P), 1984)					
<u>Observations given in inspection report</u> - "Local disruption on chunam surface" in lower southern portion and upper northern portion - "Channels blocked by construction materials" in lower central portion **<u>Observations interpreted by FMSW</u>** - Cracks on chunam surface in lower	Photos Nos. 6-2, 5-10 & 5-4 (B&P,1984) Photos Nos.	- Repair disrupted chunam surfaces - Provide access for inspection - Carry out Annual Geotechnical Inspection	Sheet 1 (B&P,1984)	- Inspection covers the full extent of slope No. 11NW-B/C90 - The report stated that "Maintenance work according to AGI 1982 was carried out in 9/83. Latest maintenance work was carried out in 4/1984 after routine inspection by	Sheet 2 (B&P,1984) Sheet 1 (B&P,1984)

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 3 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
southern portion and lower central portion repaired with mortar. - Patches of dark tone noted on chunam in lower northern portion	6-0 & 6-3, (B&P,1984)			DMO **[**District Maintenance Office of Housing Department**]** Offices." and **"**Sealed cracks on the slope was monitored visually between 9/83 and 2/84 and no apparent movement of the cracks was reported"	
Annual Geotechnical Inspection (Inspection Date: August 1986) (Dames & Moore (D&M), 1986)					
<u>Observations given in inspection report</u> <u>Upper northern portion</u> - "Cracks on chunam surfaces" - "Broken chunam" surfaces - "Weed growth from disrupted chunam surfaces" - "Spalling cement mortar" on slope surface <u>Upper central portion</u> - "U-channel blocked by debris and vegetation" - "Weed growth from disrupted chunam surfaces" - "Opened rock joint" <u>Lower southern portion</u> - "Cracks on chunam surfaces - "Disruptions along U-channels" - "Vegetation growth from rock joints" <u>Lower northern portion</u> - "U-channel blocked by general debris" - "Water ponding inside catchpit" <u>Lower central portion</u> - "Disruptions on chunam surface" - "Disruptions from plastered surface of open channel"	Photos Nos. 18/5,19/32a, 18/18,18/19, 18/11,18/17, 18/20,18/23, 19/33a,18/24, 18/30a,18/27, 19/35a,19/36a , 18/26,18/10 & 19/34a (D&M,1986)	- Clear catchpit & U-channels on slope - Repair disruptions along U-channels & chunam surfaces - Remove weeds from chunam surfaces & rock joints - Seal opened rock joints - Reinstate undermined U-channel	Section 5 (D&M, 1986)	- Inspection covers the full extent of slope No. 11NW-B/C90 - The report stated that "Recommendations in the AGI report were implemented" - The report stated that "In general, the slope appears to be in satisfactory condition"	Sheet 2 (D&M, 1986) Section 5 (D&M, 1986) Sheet 1 (D&M, 1986)

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 4 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
- "Undermining of U-channel" and "cracks developed across channel" **<u>Observations interpreted by FMSW</u>** - Cracks on chunam surface in lower southern portion, mid-height central portion and lower northern portion repaired with mortar - Patches of dark tone noted on chunam in lower central point and lower northern portion - Staining on chunam surface noted in lower northern portion	 Photos Nos. 18/2,18/22, 18/8,18.9 & 18/13-15 (D&M,1986)				
Special Area Inspection in (Inspection Date: November 1993) (Fraenkel - Haswell (FH), 1994)					
<u>Observations given in inspection report</u> - "A number of cracks are observed on the chunam surface" **<u>Observations interpreted by FMSW</u>** - Unplanned vegetation growing on berm and chunam surface in lower northern portion - Seepage on slope toe near staircase at toe of northern portion - Vegetation growing on cracked chunam in lower northern portion - Patches of dark tone noted on chunam in lower northern portion	 Photos Nos. 6/17,6/18, 6/19 & 6/22 (FH,1994)	- Nil		- Inspection covers the lower part of slope No. 11NW-B/C90 - The report stated that the slope is "generally in good conditions"	Sheet 2 (FH, 1994) Section 3.2 (FH, 1994)
Annual Geotechnical Inspection (Inspection Date: July 1995) (Greg Wong & Associates Ltd (GW), 1995)					
<u>Observations given in inspection report</u> <u>Lower southern portion</u>	Photos Nos.	- Clear unplanned vegetation	(GW, 1995)	- Inspection covers the lower part of	Sheet 1

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 5 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
- "Blocked U-channel ….. and weepholes to be cleared" Lower northern portion - "Unplanned vegetation" - "Disrupted chunam surface" - "Blocked U-channel ….. and weepholes to be cleared - "Moss on chunam surface" - "Decomposed chunam surface" Lower central portion - "Unplanned vegetation" - "Disrupted chunam surface" - "Blocked U-channel ….. and weepholes to be cleared **Observations interpreted by FMSW** - Seepage from weepholes at about mid-height of shotcrete surface in lower southern portion - Unplanned vegetation on chunam surface in lower central portion and lower northern portion - Vegetation growing on cracked chunam	1/7/2, 18/7/25,1/7/3, 18/7/31, 1/7/4, 18/7/19 & 18/7/24 (GW, 1995) Photos Nos. 18/7/29, 18/7/30, 18/7/11-13, Photos Nos. 18/7/15-17, 1/7/4, 18/7/19 & 18/7/24 (GW, 1995)	- Repair disrupted chunam surface and cracked U-channel - Clear blocked weepholes and U-channel - Clear moss from chunam surface - Seal decomposed chunam surface		slope No. 11NW-B/C90 - The report stated that "the slope was in good general condition"	(GW, 1995) Section 5 (GW, 1995)
Annual Geotechnical Inspection (Inspection Date: October 1997) (Atkins China Ltd (ACL), 1997)					
Observations given in inspection report Lower northern portion - "Unplanned vegetation from berm & slope surface" - "Unplanned vegetation from disrupted	Photos Nos. 5/11,5/5,5/4, 5/36,5/13, 5/12,5/1 &	- Clear rubbish from slope surface - Eradicate unplanned vegetation from berm & slope	Sheet 2 (ACL, 1997)	- Inspection covers approximately the lower part of slope No. 11NW-B/C90 - The report stated that "DMO informed	Sheet 2 (ACL, 1997) Sheet 2

Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90
by the HKHA's Consultants (Sheet 6 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
U-channel" - "Disrupted pavement" <u>Lower central portion</u> - "Rubbish from slope surface" - "Disrupted stepped channel apron" - "Disrupted chunam surface" - "Crack in chunam surface"	5/10, (ACL, 1997)	surface - Repair disrupted U-channel, stepped channel apron, chunam surface & pavement - Seal crack in chunam surface with cement mortar		that no technical inspection has been carried out in the past since the feature is outside the estate's boundary"	(ACL, 1977)
				- Lands Department (LD) endorsed on 5 May 1997 that slope No.11NW-B/C90 had been assigned to HD for maintenance	LD memo to HD & GEO dated 18/6/1997
<u>Observations interpreted by FMSW</u> - Patches of dark tone on chunam in lower central portion and lower northern portion - Unplanned vegetation on chunam surface in mid-height of central portion and in lower northern portion - Blocked weepholes in lower northern portion	Photos Nos. 5/12,5/4-7, 5/10 & 5/2 (ACL, 1997)			- HD lodged an appeal on 31 October 1997 regarding maintenance responsibility for slope No. 11NW-B/C90 assigned by Systematic Identification of Maintenance Responsibility of Slopes in the Territory (SIMAR)	HD memo to LD dated 31/10/1997
				- ACL notified District Lands Office (DLO) of their recommendations on slope maintenance works and requested DLO to refer "to the relevant government departments to arrange for the implementation of the recommended maintenance actions"	ACL letter to DLO dated 7/11/1997
				- LD stated that slope No. 11NW-B/C90 had been "referred to the SIMAR section of the Lands Department for consideration and further referral to other departments for appropriate actions"	LD letter to ACL dated 16/2/1998
				- In response to ACL's letter to DLO dated 7/11/1997, LD stated that the party responsible for the maintenance	EM, LD memo to DLO, LD & copied to HD

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 7 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
				of slope No. 11NW-B/C90 was "HD [the result is being appealed against by HD]" and HD was requested to "take appropriate action to their responsible slopes in order to avoid future potential instability problems" - The report stated that "The slope is generally in a poor condition and major routine maintenance is required"	dated 1/6/1998 Sheet 2 (ACL, 1997)
Annual Geotechnical Inspection (Inspection Date: July 1998) (Atkins China Ltd (ACL), 1998)					
<u>Observations given in inspection report</u> <u>Lower southern portion</u> - "Disrupted shotcrete surface" - "Crack in shotcrete surface at slope toe" <u>Lower northern portion</u> - "Rubbish from slope toe" - "Hollow at slope toe" - "Disrupted planter wall surface (>0.01m^2)" <u>Lower central portion</u> - "Crack in U-channel" - "Crack in shotcrete surface" **<u>Observations interpreted by FMSW</u>** - Seepage from near slope toe of northern portion - Unplanned vegetation on berm and chunam surface in northern portion - Blocked weepholes in lower northern portion	Photos Nos. 7/32, 7/35, 7/37, 7/27, 8/2, 7/36 & 7/26 (ACL, 1998) Photos Nos. 8/3 & 8/4 (ACL,1998)	- Clear rubbish from slope toe - Fill up hollow at slope toe - Repair disrupted planter wall surface, shotcrete surface, and U-channel - Seal cracks in shotcrete surface & U-channel - Seal cracks in shotcrete at slope toe	Sheet 2 (ACL, 1998)	- Inspection covers approximately the lower part of slope No. 11NW-B/C90 - The report stated that "DMO informed that no technical inspection has been carried out in the past since the feature is outside the estate's boundary" - ACL notified DLO of their recommendations on slope maintenance works and requested DLO to refer "to the relevant government departments to arrange for their own implementation of the recommended maintenance actions" - The report stated that "the slope is generally in a fair condition only minor routine maintenance is required"	Sheet 1 (ACL, 1998) Sheet 2 (ACL, 1998) ACL letter's to DLO dated 6/1/1999 Sheet 2 (ACL, 1998)

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Table A2 - Summary of Observations and Recommendations from Inspections of Slope No. 11NW-B/C90 by the HKHA's Consultants (Sheet 8 of 8)

General Observations	Ref.	Recommendations	Ref.	Remarks	Ref.
- Vegetation growing on cracked chunam in lower northern portion					

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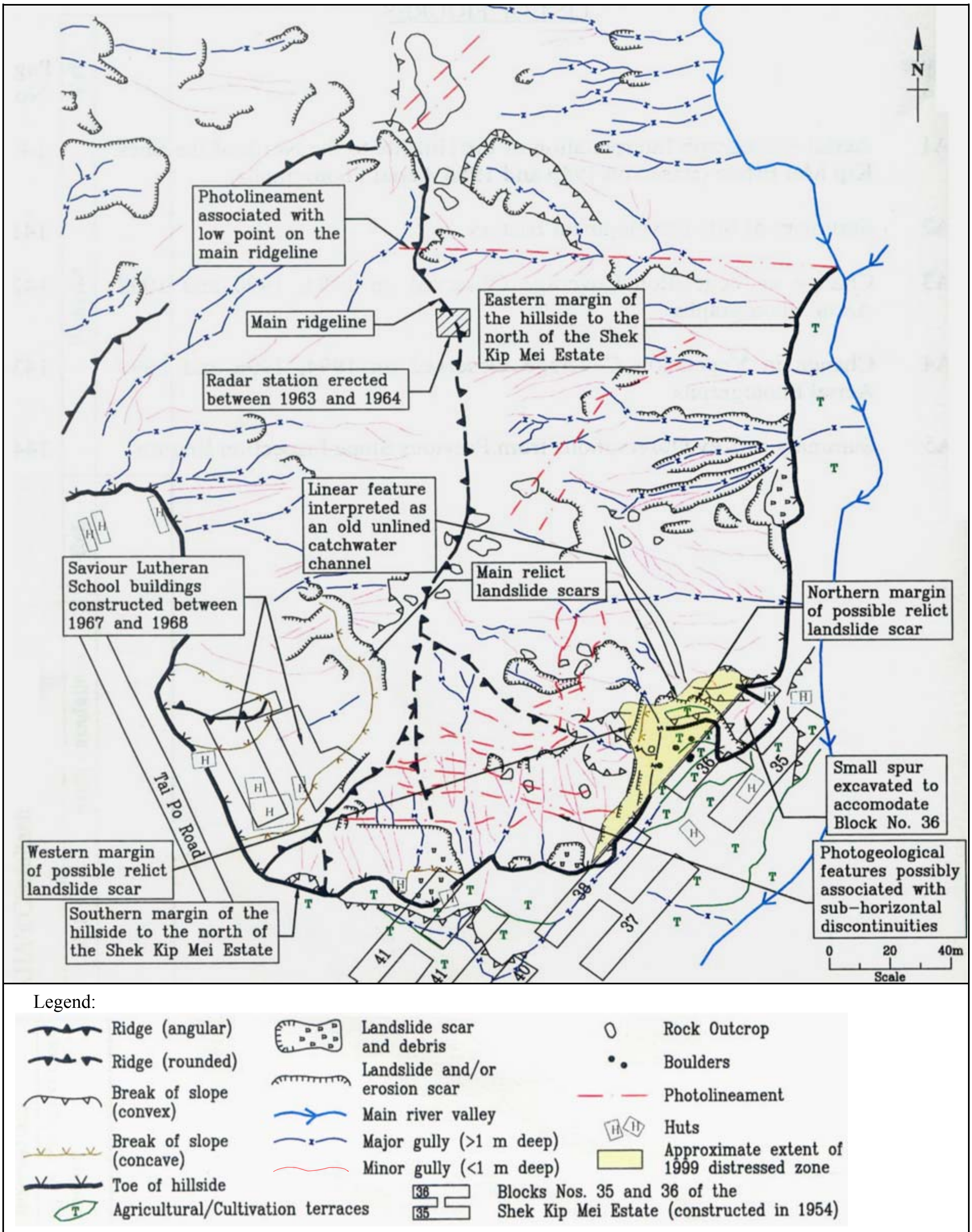

Notes: (1) The positions of Blocks Nos. 35, 36, 37, 38 and 41 of the Shek Kip Mei Estate, and the Saviour Lutheran School are extracted from 1:1000 scale Survey Sheet No. 11NW-9C.

(2) All other features, unless otherwise indicated, are interpreted from aerial photographs Nos. Y605, Y606, 6095 and 6096.

Figure A1 - Aerial Photograph Interpretation of the Hillside to the North of the Shek Kip Mei Estate (Based on 1949 and 1956 Aerial Photographs)

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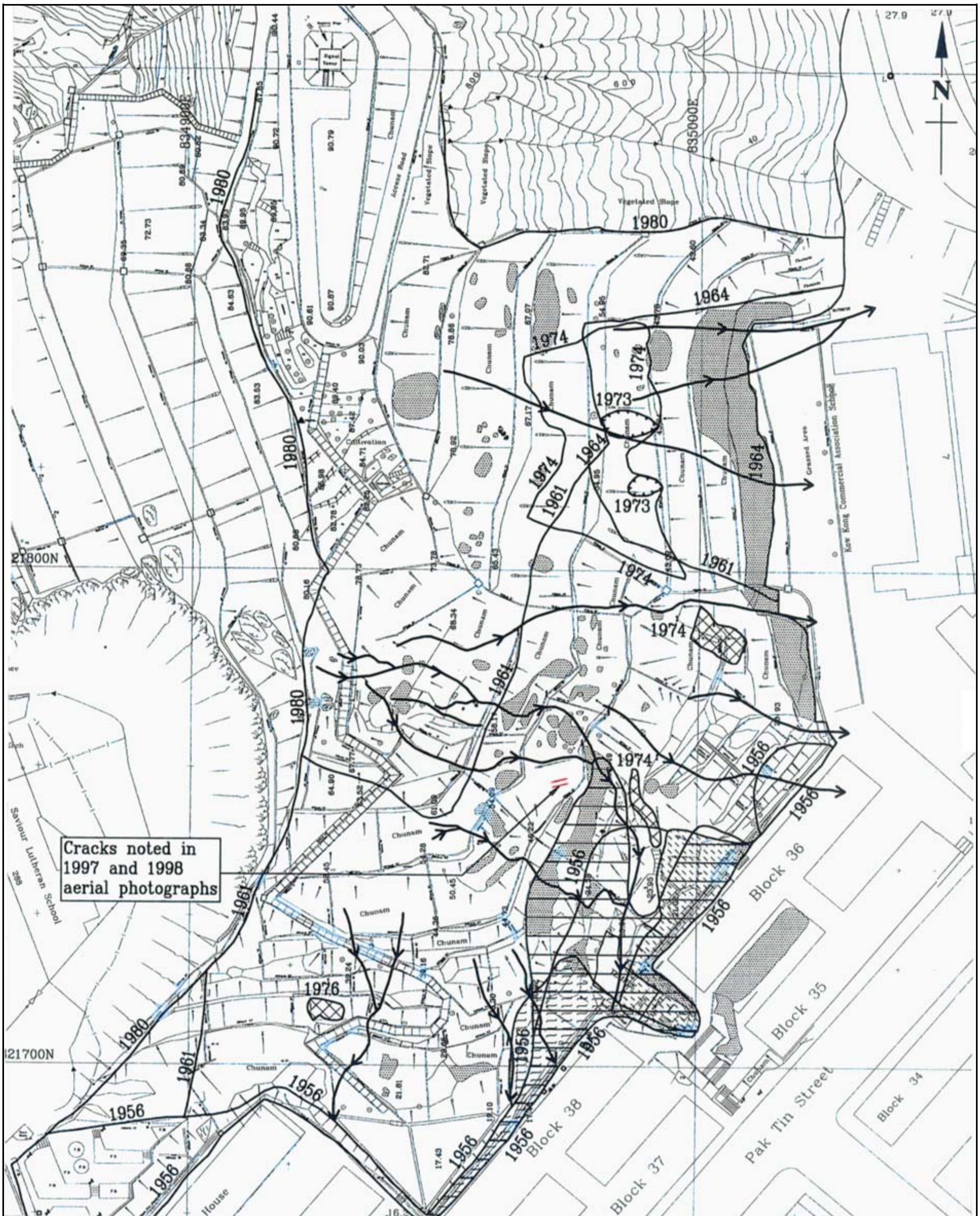

Legend:

1961	Extent of slope cutting in 1961		Agriculture terraced area in 1949		Area shotcreted in 1992
1973	Landslide scar in 1973	1976	Area resurfaced in 1976		Major drainage lines noted in 1949
	Area not affected by slopeworks between 1978 and 1980	65.43	Spot level 65.43 mPD		M-SDG rock outcrop (based on 1999 topographic survey)

Figure A2 - Summary of Site Development History

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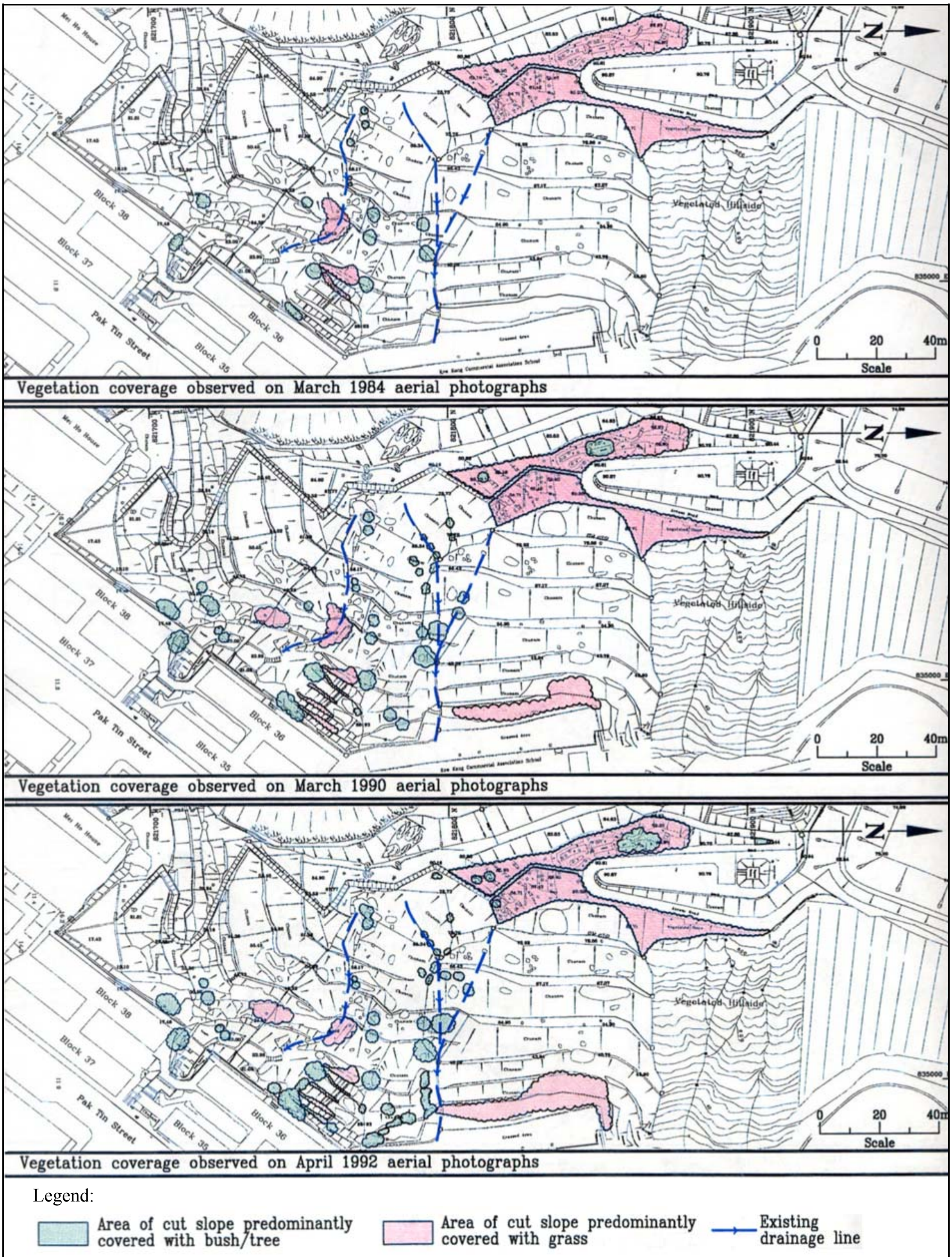

Notes: (1) Base map is based on topographical survey carried out in September 1999.

(2) Extents of vegetation are based on observations on aerial photographs Nos. 54010, 54011, A20916, A20917, A30444, and A30445.

Figure A3 - Change in Vegetation Coverage Observed on 1984, 1990 and 1992 Aerial Photographs

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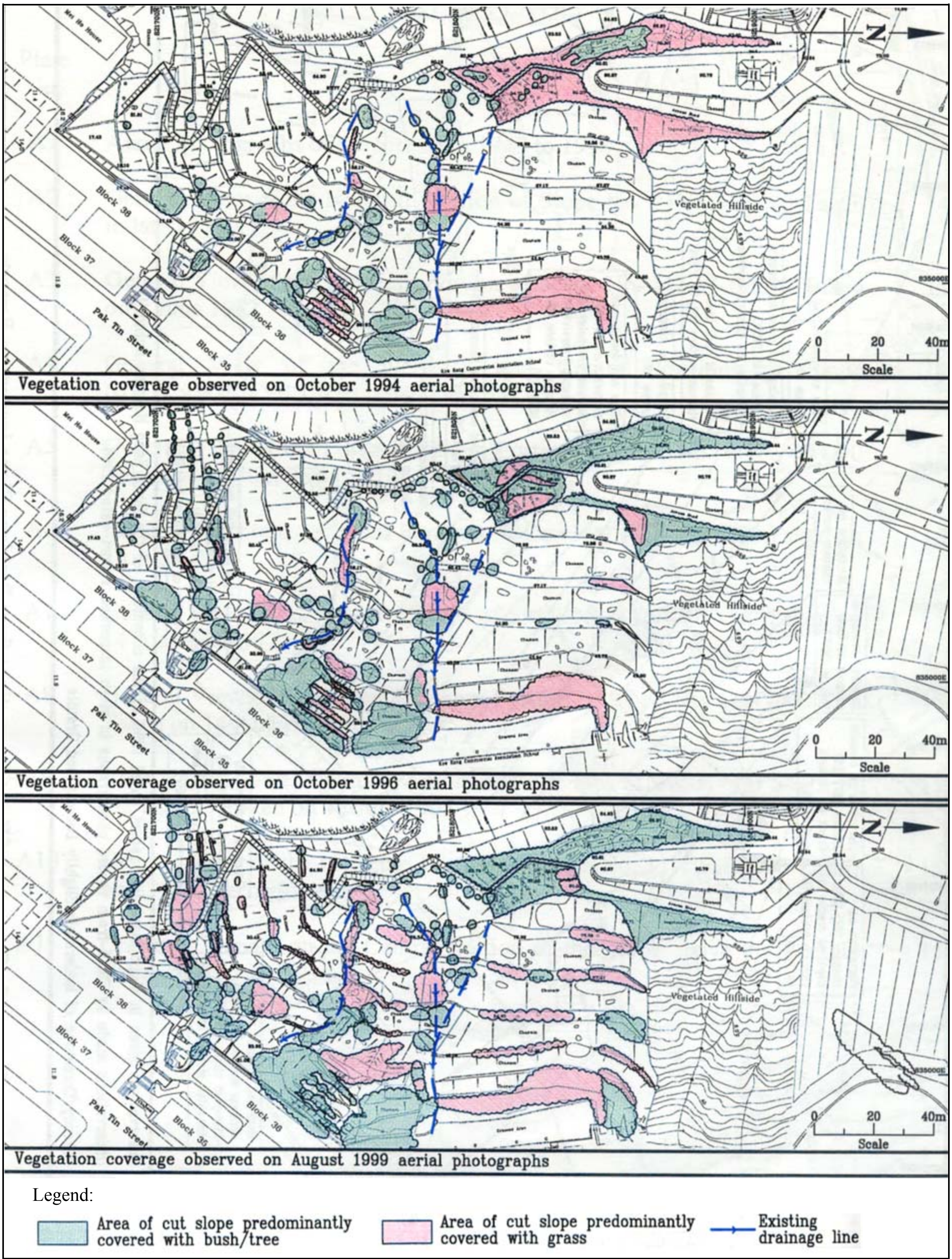

Notes: (1) Base map is based on topographical survey carried out in September 1999.
(2) Extents of vegetation are based on observations on aerial photographs Nos. A39255, A39256, CN14732, CN14733 and oblique.

Figure A4 - Change in Vegetation Coverage Observed on 1994, 1996 and 1999 Aerial Photographs

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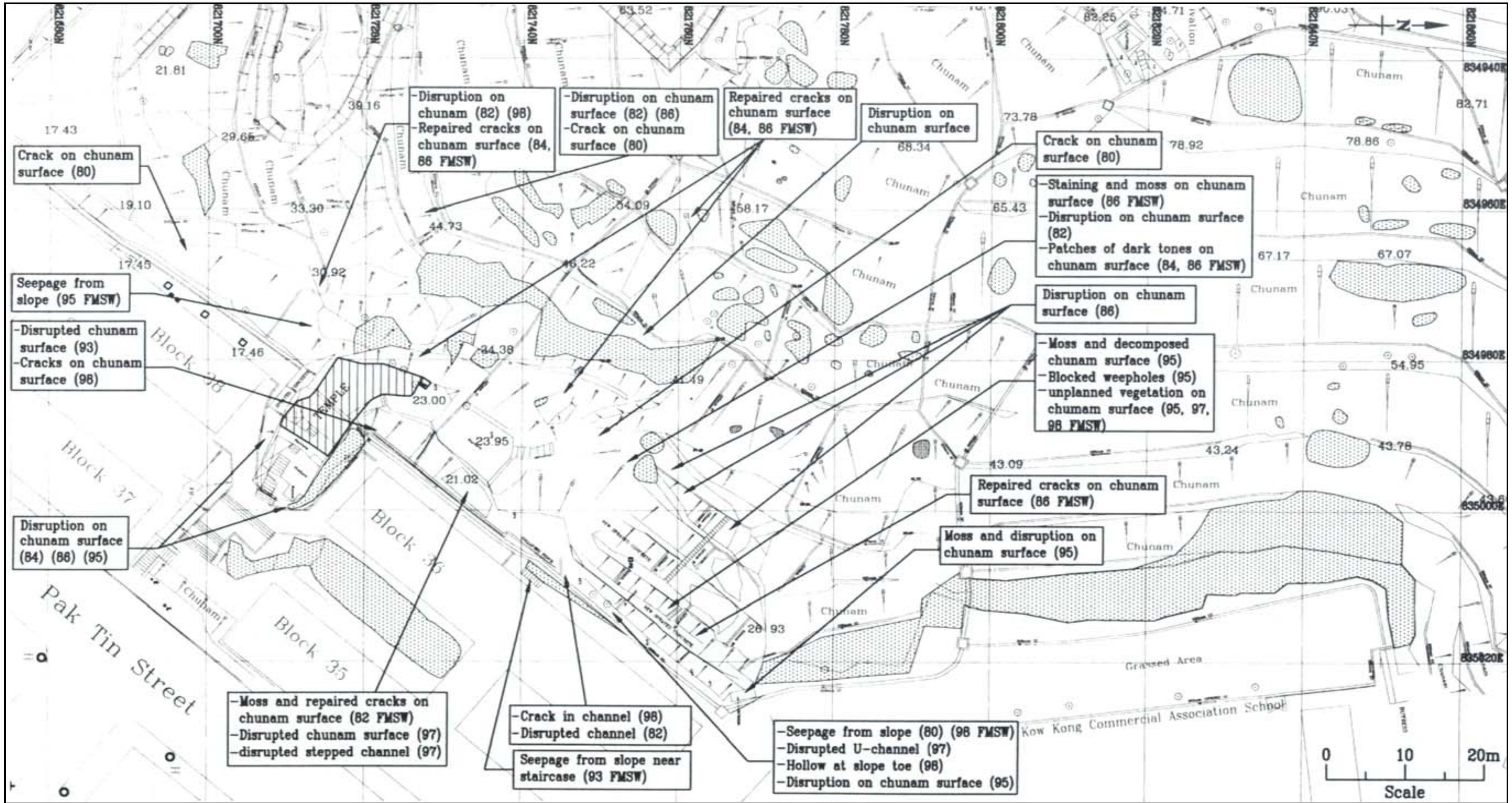

Legend:

(86) Observations given in slope inspection report dated 1986 by the consultant of the Hong Kong Housing Authority.

(86 FMSW) Observations made by FMSW from record photographs in slope inspection report dated 1986.

Outcrop/Corestone of Moderately/Slightly Decomposed Granite

Figure A5 - Summary of Key Observations from Previous Slope Inspection Reports

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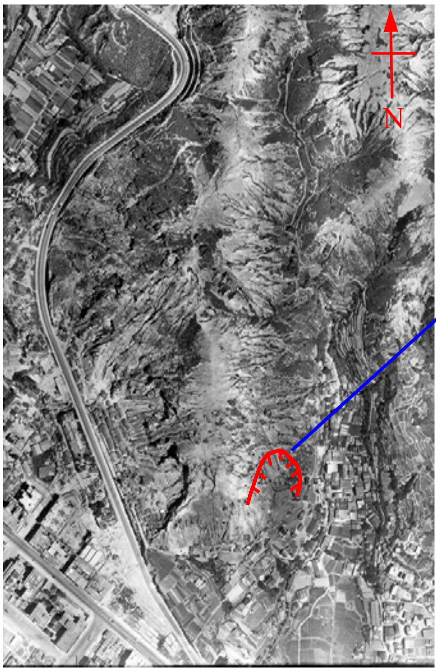
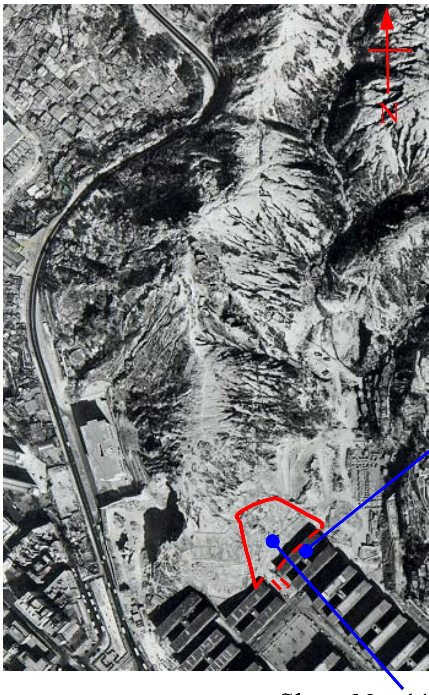
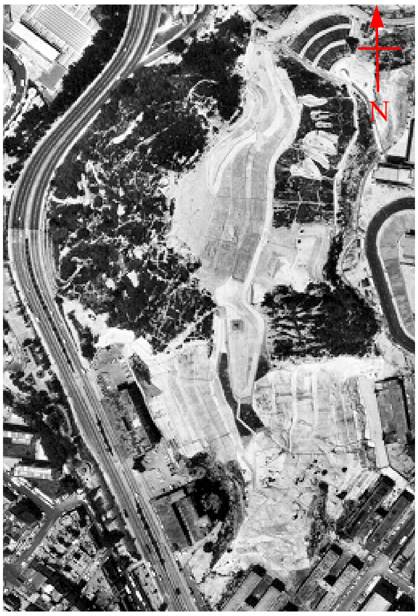


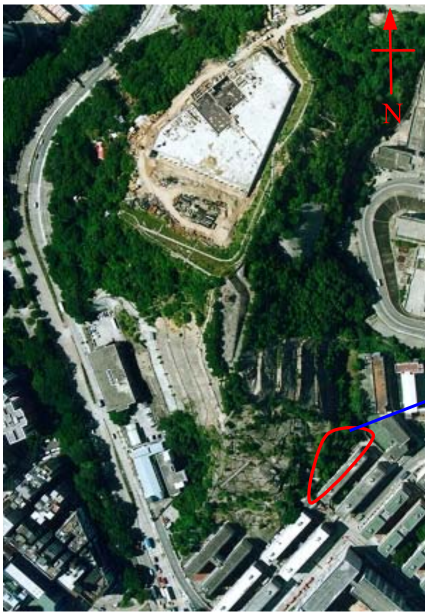
(d) 1949
Hillside prior to development of Shek Kip Mei Estate



(b) 1963
Hillside following construction of Shek Kip Mei Estate, and formation of cut slope No. 11NW-B/C90



(a) 1980
Further cut slope formation



(c) 1998
Dense vegetation noted above and within the area of distress in 1999

Plate A1 - Aerial Photographs of the Site from 1949 to 1998

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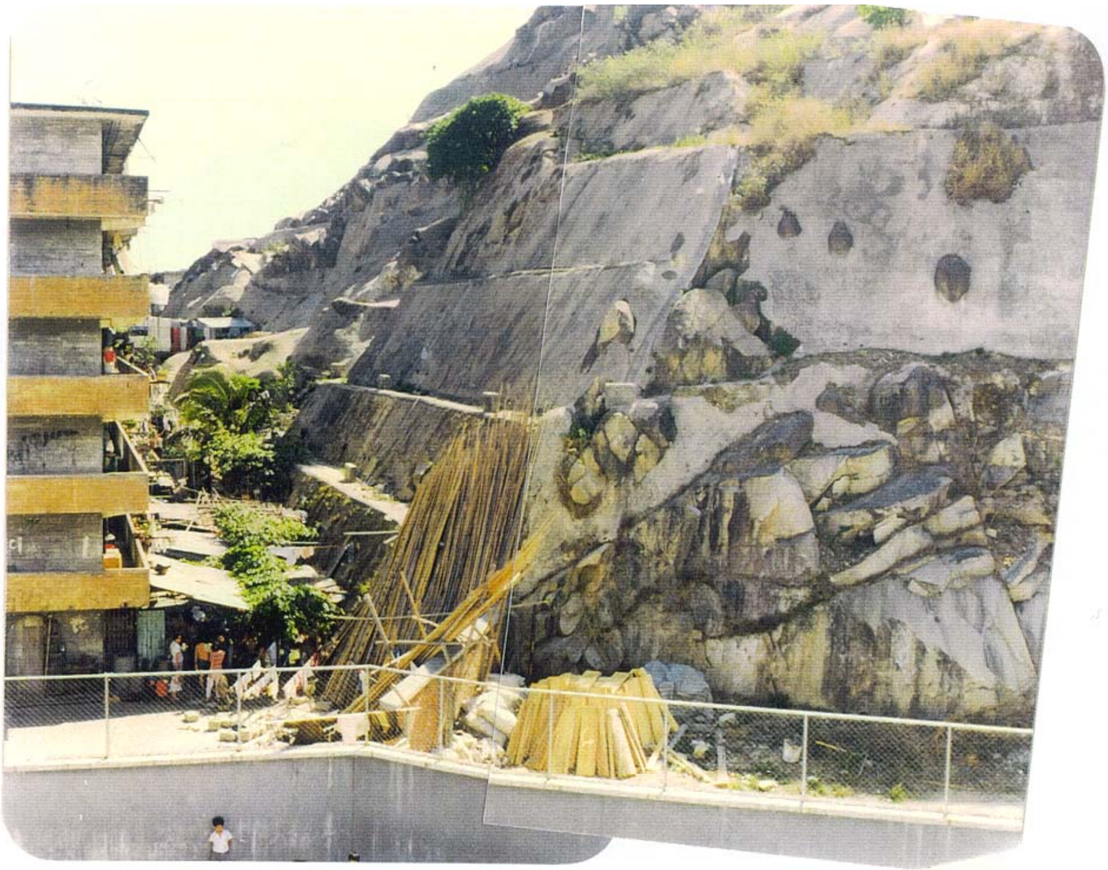

Plate A2 - General View of the Northern Portion of Slope
No. 11NW-B/C90 Taken in Late 1970's
(Reproduced from B&P, 1978).

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Plate A3 - General View of the Northern Portion of Slope No. 11NW-B/C90
Taken on 4 June 1982 (Reproduced from B&P, 1982).
Note patches of repaired chunam on the slope surface.

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Plate A4 - General View of the Northern and Central Portions of Slope No. 11NW-B/C90 Taken on 4 June 1982 (Reproduced from B&P, 1982). Note the disrupted chunam, patches of repaired chunam and chunam with dark tone on the slope surface.

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Repaired chunam along slope toe

Plate A5 - General View of the Lower Northern Portion of Slope No. 11NW-B/C90 Taken on 15 June 1984 (Reproduced from B&P, 1984). Note the repaired chunam along the toe and on the slope.

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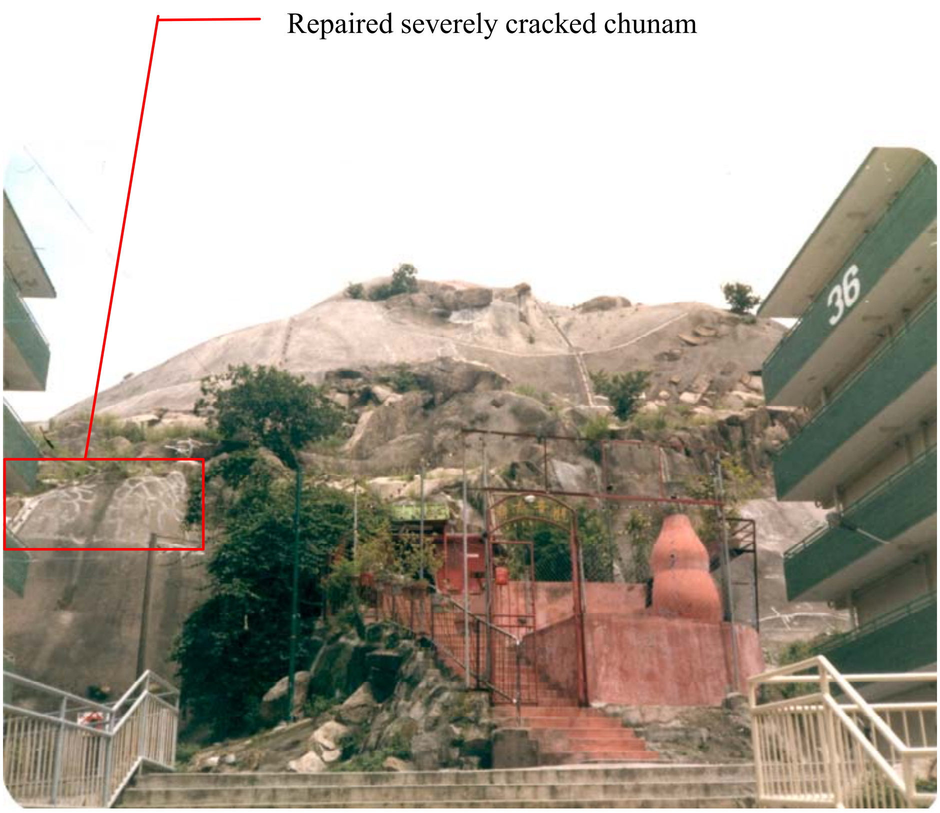

Plate A6 - General View of the Central Portion of Slope No. 11NW-B/C90 Taken on 15 June 1984 (Reproduced from B&P, 1984). Note the repaired severely cracked chunam above the approximate location of the relict slip surface observed in trial trench No. TT6 excavated after the 1999 Landslide (see Section 6.3 and Figure 25)

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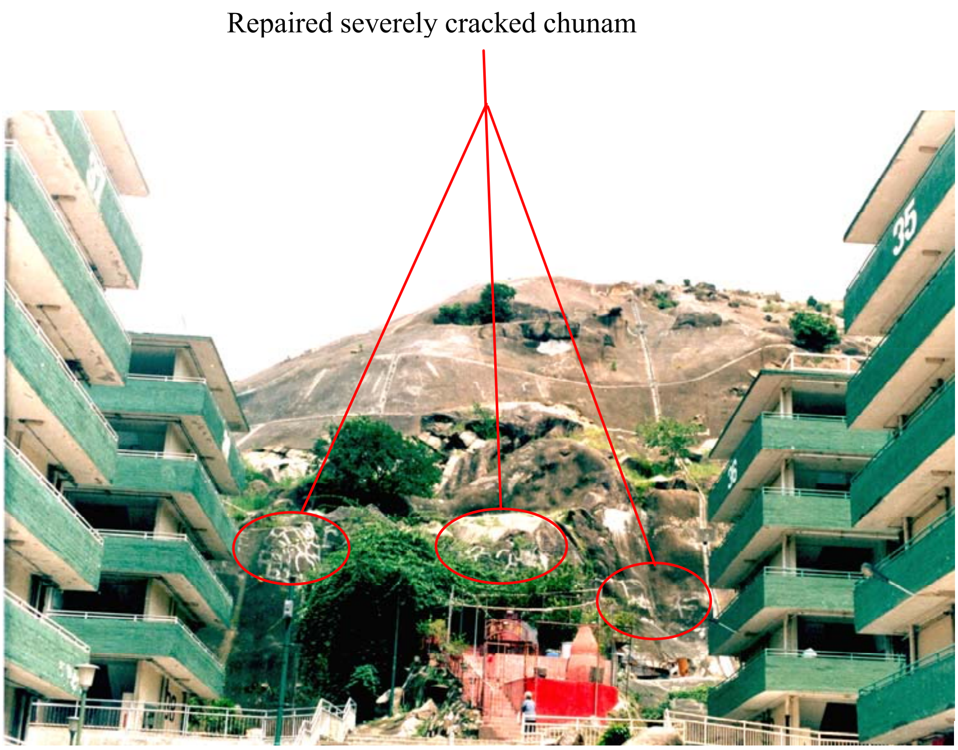

Plate A7 - General View of the Central Portion of Slope No. 11NW-B/C90 Taken on 14 August 1986 (Reproduced from Dames & Moore, 1986). Note the repaired severely cracked chunam above the approximate location of the relict slip surface observed in trial trench No. TT6 excavated after the 1999 Landslide (see Section 6.3 and Figures 23 and 25)

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Plate A8 - General View of the Lower Northern Portion of Slope No. 11NW-B/C90 Taken on 14 August 1986 (Reproduced from Dames & Moore, 1986). Note the repaired chunam and sealed cracks on the slope.

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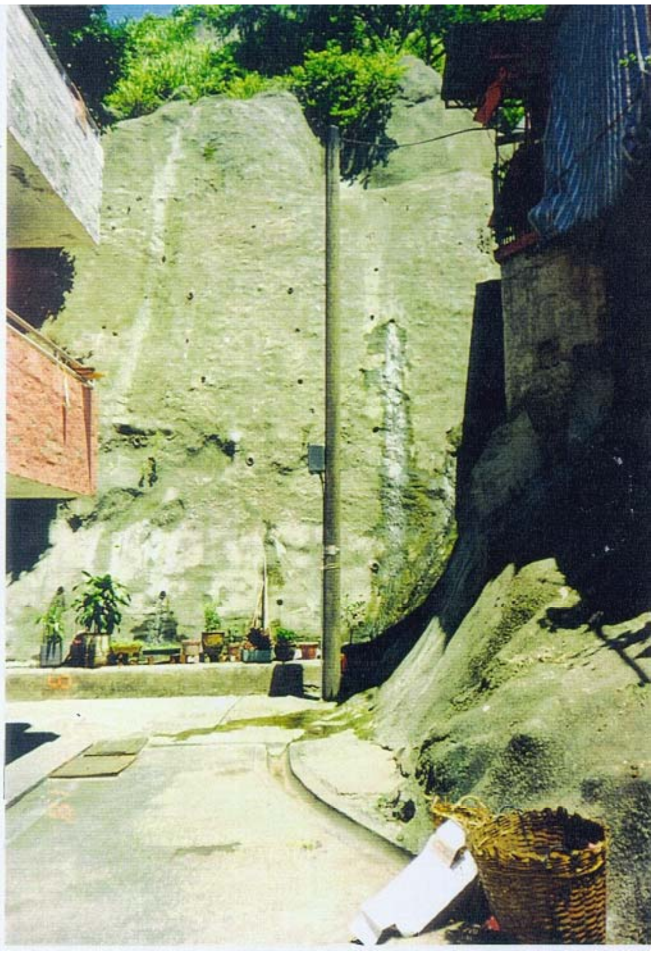

Plate A9 - View of the Lower Central Portion of Slope No. 11NW-B/C90 South of the Temple Taken on 18 July 1995 (Reproduced from Greg Wong & Associates, 1995). Note the seepage at the approximate location of the persistent infilled discontinuity observed in trial trench No. TT6 excavated after the 1999 Landslide (see Section 6.3 and Figure 25)

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Plate A10 - General View of the Northern Portion of Slope No. 11NW-B/C90 Taken on 13 October 1997 (Reproduced from Atkins China Ltd., 1997). Note the extent of chunam with dark tone and dense unplanned vegetation on the slope.

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Plate A11 - Cracked Chunam on the Central Portion of the Slope above the Temple Taken on 22 July 1998 (Reproduced from Atkins China Ltd., 1998). Note that these cracks were also observed after the 1999 Landslide and appeared to have opened up.

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VOLUME 2: LESSONS LEARNT AND RECOMMEDATIONS

Professor J.B. Burland

This report was originally produced in March 2000
as Report on the Shek Kip Mei Landslide of 25 August 1999

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CONTENTS

1. INTRODUCTION

The cut slope (No. 11NW-B/C90) behind Blocks Nos. 36 and 38, Shek Kip Mei Estate exhibited significant movement on 25 August 1999 following severe rainfall. The incident resulted in the permanent evacuation of three housing blocks.

A detailed investigation into the slope instability was undertaken by Fugro Maunsell Scott Wilson Joint Venture under the Writer's direction, on behalf of the Geotechnical Engineering Office (GEO). The findings of the investigation are documented in Volume 1 of the Report on the 25 August 1999 Shek Kip Mei landslide.

In addition to the above technical investigation, the Writer was requested by the GEO to review lessons learnt and the need for improvement in the light of the Shek Kip Mei incident. This report presents the findings of the review.

2. FACTORS CONTRIBUTING TO THE SHEK KIP MEI INCIDENT

The investigation concluded that the 25 August 1999 slope failure at Shek Kip Mei was triggered by prolonged, severe rainfall resulting in infiltration of water into the slope and raising of the base groundwater table.

The site setting of the slope is adverse in that it is located on a hillside with landsliding and deep erosion in the past, together with unfavourable geological and hydrogeoglogical features (i.e. weak infilled discontinuities and old drainage lines). The relic landslide resulted in disturbance to the slope, and could have given rise to the pre-existing tension cracks.

Apart from the above, the slope had also been influenced by the actions of man. The slope was formed by cutting into the hillside in the late 1950's. The knowledge of soil mechanics at that time was rudimentary around the world and the design process was probably empirical (i.e. based on precedents). The continued stability of the slope was most likely to have been a result of the suction preserved in the ground by the application of a surface protective cover.

The slope was assessed under a regional study in the late 1970's and some improvement works were subsequently carried out. The slope had apparently remained stable for about 20 years at the time of the study and it appears that the study considered it reasonable to rely on suction for continued stability. It should be noted that the scope of this past study did not include a detailed site-specific ground investigation on the slope. Given this constraint, the study did not properly identify the ground types or the presence of the extensive pre-existing tension cracks and weak discontinuities beneath the slope surface cover. These tension cracks would probably have rendered the slope vulnerable to even relatively minor defects in the surface cover. This is because water ingress via infiltration of rainfall through such defects and tension cracks into the slope will tend to destroy the suction and hence reduce the safety margin. In this respect, the extensive surface cover might have given a false sense of security against surface infiltration in the long term.

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Some slope maintenance works were carried out at the lower part of the slope in 1992, after which no further maintenance works had been undertaken. This led to significant progressive deterioration of the slope due to extensive unplanned vegetation causing disruption of the chunam protective cover due to root action. Lack of slope maintenance in this case was an administrative (i.e. not technical) matter associated with the identification of the most suitable owner department to take up the responsibility for the works. It is understood that possible ambiguities in the administrative guidelines on slope maintenance responsibility have now been resolved by the satisfactory completion of the Systematic Identification of Maintenance Responsibility of Slopes in the Territory (SIMAR) study and the issuing of a new internal administrative circular following the Shek Kip Mei landslide.

Human actions in the vicinity of the slope are also liable to adversely affect the hydrogeological setting and result in additional water ingress. Illegal cultivation above the slope since 1995 could have contributed to the landslide by providing an additional source of water ingress. The construction site for the service water reservoir could also have potentially contributed by increasing the catchment area; however, the evidence suggests that this was unlikely to have been significant as a direct cause of the landslide.

3. PRESENT SLOPE CLASSIFICATION AND INSPECTION SYSTEMS IN HONG KONG

3.1 Slope Classification

In Hong Kong, a comprehensive computerised catalogue of the sizeable man-made slopes and retaining walls has been compiled by the GEO between 1994 and 1998 (New Catalogue of Slopes), superseding the 1977/78 Catalogue of Slopes.

Slopes are classified under the above re-cataloguing exercise as slopes which have, or have not, been processed under the slope safety system managed by the GEO. High priority slopes are subject to stability assessment in accordance with a risk-based priority ranking system under the Landslip Preventive Measures (LPM) Programme, and upgrading works are carried out on substandard slopes to prevent failure.

Each of the registered slopes of Government ownership in the New Catalogue of Slopes is assigned to a responsible department (or party) for maintenance under the SIMAR study completed by the Lands Department in December 1999.

Where the need for upgrading works at a Government slope has been identified, the works will normally be carried out without delay unless there are development plans in the coming few years. In the latter case, subject of course to the assessed risk, the upgrading works may be shelved on grounds of cost-effectiveness (i.e. avoidance of possible abortive works). In the case of the slopes at Shek Kip Mei, there were redevelopment proposals under the Comprehensive Redevelopment Programme since 1987 and in 1997/98, Blocks Nos. 35, 36 and 38 were scheduled for clearance in 2001/2, but it is not clear whether this has had any direct bearing on any plan to maintain or upgrade the slopes concerned.

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3.2 Slope Inspections

In recognition of the importance of slope maintenance, the GEO stepped up its public education and publicity programmes and published the technical standards of good practice on slope maintenance “Geoguide 5 - Guide to Slope Maintenance” in 1995 (with the Second Edition issued in 1998). All slopes need to be regularly inspected and maintained, including those that had previously been processed under the slope safety system.

Under the current system, each of the Government slopes will be subject to regular routine inspections by technical staff. Any abnormal features noted will trigger an immediate inspection by a qualified geotechnical engineer and appropriate follow-up actions will be taken. There are also periodic inspections by qualified geotechnical engineers (i.e. Engineer Inspection) which include a review of the adequacy of past slope assessments and works, and an assessment of whether the site conditions have changed significantly (e.g. performance records in respect of landslides, new developments in the vicinity, etc.). In this respect, the findings of the Engineer Inspection will provide feedback on the performance of slopes to facilitate the determination of the need and priority for follow-up action.

At present, regular maintenance inspections of private slopes are not mandatory. The GEO is promulgating the message by public education and publicity on slope maintenance, and via the Community Advisory Unit set up since April 1999.

In addition to the above systematic inspections, all landslide incidents reported to the GEO are reviewed and selected cases are investigated to identify landslide preventive actions on slopes affected by failures and other slopes with similar adverse generic problems before the situation deteriorates. This initiative, which was introduced following the 1994 Kwun Lung Lau landslide as recommended by Professor N R Morgenstern, supplements the conventional, catalogue-based slope stability assessment process under the LPM Programme and provides a ‘safety net’ in ensuring that timely actions are taken on deserving slopes.

3.3 Review of Adequacy of the Slope Classification and Inspection Systems

The slope classification and inspection procedures of the current slope safety system in Hong Kong are one of the most comprehensive and rigorous in the world. Given the large stock of old, potentially substandard, slopes affecting the dense urban hillside development, built previously without proper geotechnical control, these procedures are appropriate.

The Writer is also satisfied with the adequacy and appropriateness of the technical standards already in place. The crux appears to be the need to ensure proper and effective implementation of the system and standards. In addition, there are other areas which warrant attention in the light of the Shek Kip Mei incident. These matters are discussed in Section 5.

4. DISCUSSION

4.1 General

This section examines the context under which key decisions were made on slope No. 11NW-B/C90 over the years before the failure on 25 August 1999.

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4.2 Reliance on Suction

In appreciating the various engineering decisions made over the years on slope No. 11NW-B/C90, it is necessary to outline the relevant key milestones in respect of the progressive advancement in slope engineering practice in Hong Kong.

As explained above, the past assessment and improvement works at slope No. 11NW-B/C90 in the 1970's essentially relied on suction prevailing in the ground. At the time, there was a concerted research effort, both by the GEO and the local tertiary institute, regarding the contribution of suction to slope stability. This interest stemmed from the fact that many of the pre-existing old slopes would have been assessed as being unstable if the presence of suction was ignored, which would have had a significant bearing on the cost-effectiveness of the follow-up actions on these slopes. Based on field measurements, suctions undoubtedly exist, but the key questions are how much and what longevity.

The comprehensive Mid-levels Studies between 1982 and 1984 undertaken by the GEO provided evidence that the suction values at some sites could drop rapidly to zero at times of heavy rainfall. In other words, the assumption of the continued existence of suction may not be reliable for long-term slope stability purposes for certain site settings. The recommended standard good practice from 1984 onwards was not to rely on suction for slope design.

As to whether it was reasonable to re-visit slopes previously designed on the basis of suction, it is necessary to consider how much and for how long suction can be maintained in the slope, which is related to the service life and the state of maintenance of the slope. This is also an issue of relative priority as discussed below.

4.3 The Question of Relative Priority

Given limited resources in tackling a large number of potentially substandard old slopes, getting the priority right is very important in order to achieve the most effective and efficient reduction of the overall risk level posed to the community.

In 1985, slope No. 11NW-B/C90 was reviewed by the GEO for possible further study under the LPM Programme. In essence, it was not considered appropriate at that time to study this slope again at the expense of another slope. This decision needs to be viewed from the perspective of assigning priorities. There were a large number of other slopes which had not been subject to any form of geotechnical studies or works since the establishment of the GEO. The decision in 1985 was intrinsically linked to the fact that improvement works were only carried out several years before that time and the slope had apparently performed satisfactorily, with no major incidents. In this respect, the Writer considers that the decision in 1985 was not unreasonable from the point of view of priority.

It is noted that at the time of the review in 1985, there was no detailed re-assessment of the adequacy of the earlier study/works. Such assessment would of course have had resource implications and hence was again a relative priority issue. The Writer also notes that a review of the adequacy of the past assessment had not been carried out in accordance with the requirements stipulated in Geoguide 5 before the failure.

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The Writer understands that the GEO had previously planned to re-visit those slopes subjected to old studies as part of the 10-year Extended LPM Programme. With hindsight, this category of slopes perhaps should have warranted earlier attention. However, as mentioned before, in the case of the slope at Shek Kip Mei, the issue was complicated by the planned redevelopment of the Estate.

4.4 The Concepts of Service Life and Residual Risk

It is not realistic to expect that engineered structures, including slopes, will last forever given that ageing and deterioration are inevitable and inexorable.

It is fundamental that risk cannot be completely eliminated. There will always be some level of residual risk, even for engineered slopes. This is because of the uncertainties in the ground conditions and rainfall loading, disproportionate cost required to reduce the risk further (i.e. money would be better spent on other risks from a cost-benefit point of view), etc.

In general, there is limited reliable information on the length of service life for different engineering measures, which will inevitably deteriorate with time. Slope maintenance will serve to reduce the rate of deterioration and maintain the existing level of stability. Conversely, lack of slope maintenance will accentuate slope deterioration and adversely affect the stability of the slope.

Like other professions, advances in geotechnical know-how and technology are made from time to time. In the case of slope engineering, knowledge about the longevity of suction was advanced in the early 1980's and more robust slope measures (e.g. soil nails) were developed in the late 1980's. These latter measures are more reliable (i.e. lower residual risk), more robust (i.e. better able to cope with unforeseen defects in the ground) and hence have a correspondingly longer service life. Because of the immense resource implications, the implementation of such improved measures will be a gradual process based on assessments of risk, relative priority and available resources. In principle, it is not unreasonable to replace the measures progressively by more robust provisions, taking into consideration the longevity of the original measures in serving their intended purposes and the performance history of the slope. As explained above, the current inspection system serves to provide feedback on the performance of slopes and reflect on the need to re-prioritize follow-up actions under the LPM Programme based on the latest findings. If properly implemented, the slope inspection system should ensure that timely actions will be taken.

The Shek Kip Mei incident has brought to light the important message that some of the slopes treated in the late 1970's may well be approaching a state deserving re-assessment of the need for further improvement works. Given this, it is now prudent to take timely action on those deserving slopes before the situation deteriorates to an extent that serious problems arise.

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5. RECOMMENDATIONS

The Writer recommends the following:

(a) Slopes processed in the early years of setting up the slope safety system in Hong Kong form a target group that deserves attention. These candidate slopes should be screened for priority attention, giving due regard to the following factors associated with the failed slope at Shek Kip Mei:

(i) large, old cut slopes affecting development or major roads, with slope studies or works carried out in the 1970's,

(ii) reliance on suction to maintain stability, and

(iii) slopes which are vulnerable to defects in the slope surface cover, such as presence of pre-existing tension cracks.

(b) The existing requirements of Engineer Inspections, which provide feedback on the review of the adequacy of past slope stability assessment/works, constitute an important element in periodic review of the slope condition for timely slope maintenance and landslip preventive action. A mechanism should be devised to maintain the quality of Engineer Inspections and to ensure regular maintenance of slopes to prevent deterioration.

(c) For Government slopes pending action under future re-development, arrangements should be put in place to regularly appraise the slope condition (e.g. more frequent Engineer Inspections), carry out the necessary enhanced maintenance and/or interim improvement works, and give warnings of any impending danger.

(d) Although unauthorized cultivation was not a key contributory factor in the Shek Kip Mei failure, such practice could nonetheless have significant adverse impact on the adjoining slopes. Effective means to discourage unauthorized cultivation and take timely actions upon discovery of unauthorized cultivation should be examined.

(e) It is fortunate that the Shek Kip Mei landslide was of low mobility when signs of instability were manifested on 25 August 1999. Prudent actions were taken to evacuate the people at risk and prevent the slope from developing into an uncontrolled failure. Improved knowledge of those

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technical factors that determine the degree of mobility would aid in the assessment of risk and prioritisation for landslip preventive action. The Writer is pleased to note that GEO has been carrying out research and development work on mobility of slope failures in Hong Kong and strongly endorses this action.

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GEO PUBLICATIONS AND ORDERING INFORMATION
土力工程處刊物及訂購資料

A selected list of major GEO publications is given in the next page. An up-to-date full list of GEO publications can be found at the CEDD Website http://www.cedd.gov.hk on the Internet under "Publications". Abstracts for the documents can also be found at the same website. Technical Guidance Notes are published on the CEDD Website from time to time to provide updates to GEO publications prior to their next revision.

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Copies of GEO publications (except maps and other publications which are free of charge) can be purchased either by:

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MAJOR GEOTECHNICAL ENGINEERING OFFICE PUBLICATIONS
土力工程處之主要刊物

GEOTECHNICAL MANUALS

Geotechnical Manual for Slopes, 2nd Edition (1984), 300 p. (English Version), (Reprinted, 2000).

斜坡岩土工程手冊(1998)，308頁(1984年英文版的中文譯本)。

Highway Slope Manual (2000), 114 p.

GEOGUIDES

Geoguide 1	Guide to Retaining Wall Design, 2nd Edition (1993), 258 p. (Reprinted, 2000).
Geoguide 2	Guide to Site Investigation (1987), 359 p. (Reprinted, 2000).
Geoguide 3	Guide to Rock and Soil Descriptions (1988), 186 p. (Reprinted, 2000).
Geoguide 4	Guide to Cavern Engineering (1992), 148 p. (Reprinted, 1998).
Geoguide 5	Guide to Slope Maintenance, 3rd Edition (2003), 132 p. (English Version).
岩土指南第五冊	斜坡維修指南，第三版(2003)，120頁(中文版)。
Geoguide 6	Guide to Reinforced Fill Structure and Slope Design (2002), 236 p.

GEOSPECS

Geospec 1	Model Specification for Prestressed Ground Anchors, 2nd Edition (1989), 164 p. (Reprinted, 1997).
Geospec 2	Model Specification for Reinforced Fill Structures (1989), 135 p. (Reprinted, 1997).
Geospec 3	Model Specification for Soil Testing (2001), 340 p.

GEO PUBLICATIONS

GCO Publication No. 1/90	Review of Design Methods for Excavations (1990), 187 p. (Reprinted, 2002).
GEO Publication No. 1/93	Review of Granular and Geotextile Filters (1993), 141 p.
GEO Publication No. 1/2000	Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls (2000), 146 p.
GEO Publication No. 1/2006	Foundation Design and Construction (2006), 376 p.

GEOLOGICAL PUBLICATIONS

The Quaternary Geology of Hong Kong, by J.A. Fyfe, R. Shaw, S.D.G. Campbell, K.W. Lai & P.A. Kirk (2000), 210 p. plus 6 maps.

The Pre-Quaternary Geology of Hong Kong, by R.J. Sewell, S.D.G. Campbell, C.J.N. Fletcher, K.W. Lai & P.A. Kirk (2000), 181 p. plus 4 maps.

TECHNICAL GUIDANCE NOTES

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