INVESTIGATION OF SOME SELECTED LANDSLIDES IN 1998 (VOLUME 4)

GEO REPORT No. 111

Fugro Scott Wilson Joint Venture

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

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PREFACE

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

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

R.K.S. Chan
Head, Geotechnical Engineering Office
August 2001

EXPLANATORY NOTE

This GEO Report consists of two Landslide Study Reports on the investigation of selected slope failures that occurred in 1998. The investigations were carried out by Fugro Scott Wilson Joint Venture (FSW) for the Geotechnical Engineering Office as part of the 1998 Landslide Investigation Consultancy.

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

(a) establishment of an improved slope assessment methodology,

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

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

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

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

SECTION 1 :
DETAILED STUDY OF THE LANDSLIDE AT THE HONG KONG STADIUM ON 24 MAY 1998

Fugro Scott Wilson Joint Venture

This report was originally produced in July 1999
as GEO Landslide Study Report No. LSR 2/99

FOREWORD

This report presents the findings of a detailed study of the landslide (GEO Incident No. HK 98/5/2) that occurred on the northeast-facing slopes bounding the Hong Kong Stadium on 24 May 1998. Debris from the landslide was deposited primarily onto the stadium perimeter access road below although some debris was also noted on the adjacent Level 3 podium of the stadium. No fatalities or injuries were reported following the landslide.

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

The report was prepared as part of the 1998 Landslide Investigation Consultancy (LIC) for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 74/97. This is one of a series of reports produced during the consultancy by Fugro Scott Wilson Joint Venture (FSW). The report was written by Mr G Taylor and reviewed by Mr Y C Koo. The assistance of the GEO in the preparation of the report is gratefully acknowledged.

Y C Koo
Project Director/Fugro Scott Wilson Joint Venture

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1. INTRODUCTION

Some time between 2:00 p.m. and 3:15 p.m. on 24 May 1998 a minor landslide (GEO Incident No. HK 98/5/2) occurred on a slope to the southwest of the Hong Kong Stadium, So Kon Po, Hong Kong Island (Figure 1 and Plate 1). The landslide occurred within Slope Nos. 11SW-D/CR227 and 11SW-D/C1007 (formerly part of Slope No. 11SW-D/N13 in the 1977/78 Catalogue of Slopes). Debris from the landslide partially blocked the stadium perimeter access road below. In addition, some debris was deposited on the adjacent Level 3 podium of the stadium. No fatalities or injuries were reported as a result of the landslide.

Following the landslide, Fugro Scott Wilson Joint Venture (the 1998 Landslide Investigation Consultants) carried out a detailed study of the landslide for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 74/97. This is one of a series of reports produced during the consultancy by Fugro Scott Wilson Joint Venture (FSW).

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

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

(a) desk study, including a review of relevant documentary records and old plans relating to the history of the site,

(b) aerial photograph interpretation (API),

(c) interviews with witnesses of the landslide and other concerned persons,

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

(e) analysis of rainfall data,

(f) engineering analyses of the slope that failed, and

(g) diagnosis of the probable causes of failure.

2. THE SITE

2.1 Site Description

The Hong Kong Stadium is located within a valley in the So Kon Po urban area (Figure 1). The southern portion of the stadium was formed by cutting into the foothills of the approximately north-south trending spurs, which flank the valley. As a result the stadium is bounded on three of its four sides by cut slopes and retaining walls.

The 24 May 1998 landslide occurred on the northeast-facing slopes bounding the southwest corner of the stadium and affected registered Slope Nos. 11SW-D/CR227 and 11SW-D/C1007 (formerly part of Slope No. 11SW-D/N13 in the 1977/78 Catalogue of Slopes). Slope No. 11SW-D/CR227 is a soil/rock cut slope, approximately 160 m in length and between 25 m and 30 m high in the area affected by the landslide. The slope comprises a near-vertical rock face about 20 m high, overlain by highly to completely decomposed granite sloping at between 45° and 80°. The toe of the rock slope is faced by a granite-dressed masonry wall, about 7 m high. Portions of both the rock face and soil slopes above have been stabilised in the past, including the use of concrete buttressing, dowel bars and soil nails.

Slope No. 11SW-D/C1007 above, comprises a cut slope with a chunam surface protection layer which was in a generally poor condition at the time of FSW's inspections during May and June 1998. Above this slope is the stadium boundary fence, beyond which is the So Kon Po Cottage Area.

The present failure occurred in a concave portion of the slope in the weathered profile near the soil/rock interface, at the junction between the above registered slope features (Plate 1).

According to correspondence between the District Lands Office/Hong Kong West, the Urban Services Department, the GEO and Wembley International (Hong Kong) Limited the Urban Council is responsible for the maintenance of the slopes affected by the recent landslide and the Architectural Services Department (Arch SD) is the maintenance agent.

As part of this investigation both Government and private utility providers were contacted for details of any services in the vicinity of the landslide. The responses indicate that no utilities, in particular water-bearing services, are located within the immediate vicinity of the landslide. Hong Kong Telecom noted that one of their underground ducts appears to traverse the landslide scar. However, no evidence of this duct was observed during the present study.

2.2 Site History

2.2.1 General

The history of the site was traced from the appropriate aerial photographs (Table 1) and from a review of other available documentary information. The locations of the features referred to in this Section are shown in Figure 2.

2.2.2 History of Development and Past Instabilities

Prior to the construction of the original stadium in the late 1950s/early 1960s, the site of the present Hong Kong Stadium and surrounding area remained largely undeveloped.

Numerous, relatively minor surface erosion scars were, however, noted on the undeveloped moderately to densely vegetated colluvial hillsides in the 1949 aerial photographs.

As part of the construction of the original stadium, extensive cutting was made into the foothills of the spurs, which flank the valley, including Slope Nos. 11SW-D/CR227 and 11SW-D/N13 (the mid-portion of which is now referred to as Slope No. 11SW-D/C1007 in the New Catalogue of Slopes recently compiled by the GEO). The So Kon Po Cottage Area was also established during this period at the crest of the southwestern spur bounding the valley.

Several small erosion scars are evident on the southwestern slopes from the 1976 aerial photographs, one of which is close to the location of the 1998 landslide. Three landslide incidents on the upper soil portion of these slopes were reported to the GEO in 1982 and 1983. Failure No. 31/11SW-D in 1982 involved about 5 m^3 of soil. Failure No. 30/11SW-D in 1983 involved approximately 30 m^3 of both soil and rock whilst Failure No. 32/11SW-D (also in 1983) comprised a boulder fall involving about 3 m^3 of material (Figure 2). Failure No. 30/11SW-D occurred immediately to the west of the 1998 landslide site.

Temporary slope repair works were subsequently designed, and siteworks supervised, by Fugro (Hong Kong) Limited (FHK), for the Architectural Office Maintenance Branch (AOMB). These works, generally of a reinstatement and maintenance nature, included vegetation clearance, making good surface drainage as well as the repair of any defective portions of the chunam surface protection layer for the soil portions and the provision of several concrete buttresses, dowels and rock bolts to the rock portions of these slopes (FHK, 1985a). The above works were carried out between December 1984 and April 1985.

At about the same time, August 1984, Slope No. 11SW-D/CR227 was considered for inclusion in the Government's Landslip Preventive Measures (LPM) Programme. The slope was deleted from the LPM Programme in January 1985 on the grounds that slope repair works, “of a temporary nature”, were being carried out, which, upon completion, would improve the stability of the slope, thus reducing the priority for LPM works. Also, Non-Development Clearance (NDC) of squatters was programmed to take place in early 1993.

Further slips/erosion scars are noted on the 1986 aerial photographs. However, there are no records of these in the GEO's landslide database. One of these scars was located close to the site of the 1998 landslide. Another possible erosion scar was noted on the 1991 aerial photographs. This scar was also situated adjacent to the 1998 landslide.

Also in 1991, the Royal Hong Kong Jockey Club (the Jockey Club in short) proposed “to build a new stadium” on the site of the existing facility. On completion, the new stadium was to “be handed over to (the) Government”. The Arch SD assumed the role of “Client” in order to streamline procedures, “liaise between the Government and (the Jockey) Club on technical problems concerning the stadium and at the same time exercise the necessary Buildings Ordinance function”. However, the Arch SD have advised that they were not responsible for granting consent, acknowledging as-built records or certifying completion of the slope upgrading works.

In July 1992 during the stadium redevelopment, the Island Division of the GEO informed Arch SD that the existing slopes around the stadium, including Slope No. 11SW-D/CR227, needed to be studied to ensure compliance with current geotechnical standards and any necessary upgrading works should be carried out as part of the redevelopment project. The engineering consultant for the new stadium project, Ove Arup and Partners (OAP), subsequently proposed, during March and April 1993, upgrading works for these slopes, including provision of soil nails and cutting back of the near-vertical soil faces to flatter angles. These works were completed in 1994.

Also in 1994, the problem of persistent foul water flow onto Slope Nos. 11SW-D/CR227 and 11SW-D/N13 was raised by OAP (Figure 2). The Housing Department was subsequently requested by the GEO to investigate this matter and rectify the situation. No records have been found during the present detailed study to date confirming that this work has been done.

2.3 Previous Studies and Assessments

2.3.1 Slope Registration

The concerned slopes were registered as Nos. 11SW-D/CR227 and 11SW-D/N13 in the 1977/78 Catalogue of Slopes, by consultants engaged by the Government of Hong Kong. The first recorded inspections of these features were carried out in March 1978 during the compilation of the catalogue. At the time of inspection the condition of Slope No. 11SW-D/CR227 was assessed as "fair at lower elevation(s)" and "poor at higher (elevations)". Substantial seepage was noted on the rock portion of the slope at the south of the feature as was cracking of the chunam surface protection layer generally. In addition, a slip scar was observed on the slope. The condition of Slope No. 11SW-D/N13 was assessed as fair. No signs of seepage were recorded. However, signs of distress in the form of boulders and erosion, associated with squatters, as well as large slip scars were noted.

2.3.2 Stage 1 Study by GCO

In the second half of 1979 a Stage 1 Study Report was prepared by the Existing Slopes Division of the Geotechnical Control Office (GCO, renamed GEO in 1991) for Slope No. 11SW-D/CR227. At that time it was noted that water may be discharging onto the face of the cut slope at its southern end ("a steady flow was noticed at this end") from a U-channel which ran approximately parallel to the crest of the slope. This water was thought to be domestic wastewater from the cottage area above. In addition, within the upper part of the slope the chunam covering was observed to be badly cracked and the slope face extensively eroded. Where the slope was thickly vegetated, the forward leaning nature of the trees, some of which were almost horizontal, was considered to be "possible evidence of substantial creep of the soil slope". The consequence of failure of the slope was assessed as "very high" at that time, particularly when the stadium was operational. The report concluded that a Stage 2 Study Report should be undertaken, concentrating, in particular, on the soil portion of the slope.

2.3.3 Detailed Study by Fugro

Following the landslides of 1982 and 1983 (see Section 2.2.2), FHK undertook an investigation of the slopes for the AOMB (FHK, 1985b) and concluded that Slope Nos. 11SW-D/CR227 and 11SW-D/N13 were not up to current geotechnical standards (see Section 4.2 for details of the ground investigation undertaken for this study). As part of this investigation FHK carried out visual inspections of the slopes and noted that portions of Slope No. 11SW-D/N13 were potentially unstable. FHK also recorded that "heavy and persistent seepage occurs at various points along the rock face, coming out along joints and at the soil/rock interface". At this time the minimum factor of safety considered acceptable for the slope, classified as "an existing low risk slope", was 1.1. The presence of adversely orientated relict joints was noted in places within the soil portion of the slope, including the location of the 1998 landslide, during FHK's field mapping. In addition, a "zone of highly weathered material at the crest of the rock slope" within Slope No. 11SW-D/CR227, including the 1998 landslide site, was identified as that "which presents the greatest potential" for future failures. The subsequent geotechnical report, issued in October 1985, recommended that the soil portion of the slope, essentially Slope No. 11SW-D/N13, be trimmed back to an inclination of 50°, with 1.5 m wide berms at 7.5 m maximum vertical intervals. Surface drainage was also to be provided. The report also stated the need to maintain both the surface protection layer and the surface water drainage network to prevent future infiltration of rainwater into the surficial slope-forming materials leading to an increase in pore pressure along the adversely orientated relict joints. However, it appears that such routine maintenance work was not subsequently carried out. In their assessment of these slopes, FHK adopted shear strength parameters c' = 8.5 kPa, ϕ' = 39°, for the completely weathered (decomposed) granite. The groundwater table was taken as generally about 1 m above rockhead level in the vicinity of the landslide (see Appendix A for details of the stability analyses undertaken during this study).

The Island Division of the GEO regarded FHK's report as an out-of-turn Stage 2 Study and requested the Design Division of the GEO to treat the slope accordingly, i.e. out of priority in the LPM Programme. As noted in Section 2.2.2 above, temporary slope repair works were carried out, including stabilisation measures for the rock slope. The soil portion of the slope, however, was not trimmed back to 50° and neither was it included within the LPM Programme.

2.3.4 Detailed Study by Ove Arup

In March 1993, as part of the Hong Kong Stadium Redevelopment Project, OAP submitted a design report on their proposals for upgrading the slopes bounding the stadium, including slopes, Nos. 11SW-D/N13 and 11SW-D/CR227 (OAP, 1993a). At the location of the present failure, it was proposed that the upper part of the slope be cut back to an angle of not greater than 60°. At other locations, portions of the slopes were to be stabilised by means of soil nails as well as trimming back. OAP observed that the concerned slopes did not show any signs of instability during the heavy rainfall of May 1992.

In response to the GEO's earlier comments, OAP submitted a further report in April 1993 providing supplementary information on their proposals and the results of additional analyses (OAP, 1993b). The revised stability analysis indicated factors of safety generally in excess of, or close to, 1.2 (see Appendix B). This report was accepted by the GEO in May 1993. In both these submissions the cut back portions of the slope are shown with an angle of inclination of approximately 50^{o}. Shear strength parameters c' = 8.5 kPa, $\phi' = 39°$, were adopted by OAP for the completely decomposed granite. The groundwater table was taken to be between 1 and 2 m above rockhead level in the vicinity of the 1998 landslide. The geological models used by OAP in their stability analyses for Sections 5-5 and 6-6, which are approximately 12 m and 3 m to either side of the 1998 landslide respectively, are given in Figure 3.

The present study has established that the slope was not cut back in accordance with the approved design (see Section 8.3).

2.3.5 SIFT and SIRST Studies and Maintenance Inspections

In 1992, the GEO initiated the consultancy agreement entitled "Systematic Inspection of Features in the Territory" (SIFT), which aimed to update the information contained in the 1977/78 Catalogue of Slopes based on studies of aerial photographs and limited site inspections. Slope No. 11SW-D/CR227 was noted as having been formed "pre-1961" and assigned to Class "C1", i.e. a slope "formed or substantially modified before 30.6.78". A large portion of Slope No. 11SW-D/N13 was included under SIFT Feature No. 11SW-15D/S190 which was also assigned to Class "C1". The chunam covering of this feature was considered on inspection at this time to be in a generally poor condition. The northern and southern ends of Slope No. 11SW-D/N13 were deleted during the compilation of the New Catalogue of Slopes whilst the mid-portion of this feature was re-registered as Slope No. 11SW-D/C1007 with corresponding new slope boundary.

In 1994, the GEO initiated the consultancy entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST) to update the 1977/78 Catalogue of Slopes and prepare a New Catalogue of Slopes. The record for field inspections carried out on 4 September 1995 for Slope No. 11SW-D/CR227 indicated no signs of distress. Heavy seepage, however, was observed both at "mid height & above" and "near (the) toe" of the slope, as were previous failure scars. The slope was classified "(as of) 30/04/95 (as) Status-LPM completed".

From the middle of 1996 up to the present time, routine maintenance inspections of the slopes bounding the stadium have been carried out. These inspections were undertaken by Wembley International (Hong Kong) Ltd up to about the middle of 1998 and from then onwards by the Arch SD. Apart from the routine maintenance works undertaken to the slopes following these inspections, no other slope problems appear to have been identified.

Slope No. 11SW-D/C1007, was inspected by the SIRST team on 20 October 1995. No signs of distress or seepage were noted on the slope at that time. This slope was also inspected on 15 July 1997. The condition of the slope was assessed as fair during the inspection. No signs of distress, past instabilities or seepages were observed at this time. The presence of potentially leaky services was, however, noted, although no signs of leakage were evident on inspection (a water main and a sewer were found at the crest of the slope).

Slope No. 11SW-D/CR227 was inspected again by the SIRST team, on 27 October 1997. The condition of the slope face was assessed as fair at this time. "Reasonable" signs of distress were observed "near (the) crest (within the) mid-portion (and) at (the) toe (of the slope)", (based on a grading system comprising the categories severe, reasonable, minor and none). Heavy seepage was also noted "at or above mid-height or from several rock joints". The surface water drainage channels were observed to be "partially blocked" on the slope itself and "blocked" at the slope toe. In addition, the presence of potentially leaky services was noted on the slope although no signs of leakage were evident during the inspection.

3. THE LANDSLIDE

3.1 Time of Failure

The landslide was reported to the GEO by the Police at 3:43 p.m. on 24 May 1998 following a 999 call they received at 3:15 p.m. A representative of the GEO subsequently inspected the landslide between 4:20 p.m. and 5:10 p.m. (see Appendix C for the GEO Incident Report). The first inspection of the landslide by FSW was made on 28 May 1998. Subsequent inspections were made at the beginning of June 1998 to map the landslide scar.

Unfortunately, there were no eye-witnesses to the landslide, thus the condition of the slope immediately before failure is not known.

A total of 13 concerned persons, including a security guard at the stadium, were interviewed by FSW following the landslide. Unfortunately, they could not provide much useful information regarding the failure.

3.2 Description of the Landslide

The landslide was located in a concave depression towards the crest of Slope No. 11SW-D/CR227 (Plates 1 and 2). The landslide involved the failure of about 5 m^3 of decomposed material near the soil/rock interface. The majority of the debris was deposited on the stadium perimeter access road some 25 m below. However, a small amount of debris landed on the pedestrian walkway of the Level 3 podium of the stadium.

The landslide scar was approximately 6 m wide and 6 m long, with a maximum depth of about 0.5 m. The profile of the scar varied from sub-vertical in the upper portion to an inclination of approximately 45° in the lower half of the scar (Figure 4 and Plate 3). Two sub-vertical relict joint planes, orientated parallel and normal to the strike of the slope face with strike and dip of typically 014°-039°/60°-75° and 308°-355°/75°-86° respectively, were observed. In addition, an adversely orientated relict joint plane was mapped in the lower portion of the landslide scar with a dip angle of typically between 35° and 45° and a generally northeasterly orientation. This adversely orientated relict joint plane was occasionally kaolin-coated (Plate 4).

The surficial deposits exposed in the landslide scar comprised essentially corestone-bearing decomposed granite near the soil/rock interface of rock mass weathering zone PW 0/30 (see Appendix C). A pegmatite dyke-vein was observed near the head of the landslide scar and further deposits of pegmatite were noted within the scar below. The generally highly to moderately decomposed granite corestones were concentrated in the lower half of the landslide scar. The remainder of the material exposed in the landslide backscarp consisted of completely decomposed granite, typically described as medium dense silty slightly clayey sand.

The landslide debris included cobbles and boulder-sized corestones of highly to moderately decomposed granite in a silty sand matrix.

The upper portion of the chunamed slope face adjacent to the landslide scar had a face angle of approximately 70° whilst the lower portion had a shotcrete surfacing and a face angle ranging from 40° to 50°. A recent application of cement grout/mortar (prior to the 1998 landslide) over the lower portion of the chunam surfacing was also noted. The overall morphology of the surrounding slopes suggests the possible occurrence of past failures, which is consistent with the observations from the API carried out during this study and the findings of the associated desk study.

Extensive root growth from trees located at the slope crest was also observed along the surface of rupture in the upper portion of the landslide scar (Plates 5 and 6).

The landslide scar was noted as being generally moist at the time of the initial FSW inspection four days after the landslide. It should be noted that minimal rainfall, about 15 mm on 25 May 1998, was recorded at the nearest raingauge to the landslide site in the interim period. No subsurface seepages were observed. Seepages were, however, noted generally on the lower rock cut face throughout the study period. The morphology of the failure scar together with the condition of the debris observed on inspection suggest that the failure probably did not involve washout by concentrated surface water flow.

As the landslide involved the localised detachment of surficial material from a steep soil slope above the rock face of Slope No. 11SW-D/CR227 the concept of travel angle of debris (Wong & Ho, 1996a), is not considered to be of relevance in this case.

Following the landslide, the stadium perimeter access road was closed to through traffic, as was the pedestrian walkway on podium Level 3. Urgent repair works comprising the removal of loose materials and chunam within the vicinity of the landslide scar, trimming back oversteep portions of the slope and the provision of a shotcrete surface protection layer were also carried out at this time. The stadium perimeter access road was re-opened to through traffic on 25 May 1998.

Upon the further advice of the GEO, 78 nos., 32 mm diameter, mostly 7 m long, soil nails were installed at 2 m nominal spacing within the slope face and a shotcrete covering applied, in the vicinity of the 1998 landslide. These works were undertaken by the Arch SD during the second half of 1998.

4. SUBSURFACE CONDITIONS

4.1 General

The ground conditions at the site were determined using information obtained from both the desk and field studies. The desk studies included a review of the available documentation supplemented by API, whilst field studies included post-failure geological mapping.

4.2 Geology and Previous Ground Investigations

The Hong Kong Geological Survey's 1:20 000 scale map, Sheet 11 Hong Kong and Kowloon Solid and Superficial Geology, indicates that the site is underlain by fine-grained Mesozoic granite of Jurassic-Cretaceous age. Jointing within the granite is shown as dipping at 65° in a generally north-northwesterly direction. In addition, a basalt dyke is shown as trending in a generally northwest-southeast orientation across the northern portions of Slope Nos. 11SW-D/CR227 and 11SW-D/N13. Debris flow deposits have also been mapped in the valley floor below.

The Geotechnical Area Studies Programme (GASP) Report 1, Hong Kong and Kowloon (Geotechnical Control Office, 1987), produced for regional appraisal and outline and strategic planning purposes at a scale of 1:20 000, indicates that no past instabilities have been noted within the area affected by the 1998 landslide or its immediate environs. The accompanying engineering geology map, whilst generally confirming the stratigraphy identified above, also indicates that a geological photolineament traverses the southern parts of Slope Nos. 11SW-D/CR227 and 11SW-D/N13, and the stadium, trending in a generally northnortheast-southsouthwest direction. A further geological photolineament, trending in an approximately northwest-southeast direction and also crossing the stadium, has been identified about 100 m to the north of these slopes.

There have been two ground investigations in the general vicinity of the failed slope (Bachy Soletanche, 1983 and Lam Geotechnics, 1991). FSW have used information from these investigations, together with post-failure site observations to prepare a geological section through the landslide site (Figures 2 and 4).

The closest drillhole to the 1998 landslide is drillhole BH13, located at the crest of the slopes. This drillhole encountered a thin layer of fill, approximately 0.4 m thick, overlying about 1.5 m of extremely weak completely/highly decomposed fine-grained granite, typically described as a yellowish brown silty fine-grained sand. Underlying this horizon of decomposed granite was a shallow layer, about 0.5 m thick, of very weak to moderately strong highly to moderately decomposed fine-grained granite. Moderately decomposed granite was encountered at about 59 mPD.

Drillhole BH9, situated at the toe of the slopes below the 1998 landslide site, encountered a thin layer of fill overlying very strong grey slightly decomposed to fresh fine-grained granite.

Drillhole BH7, situated adjacent to the footpath above the slopes currently under study and some 35 m to the northwest of the landslide scar, encountered a 4 m thick mantle of essentially highly decomposed granite overlying slightly decomposed medium- to coarse-grained granite. Drillhole BH6, situated at the junction between the rock face and soil portion of slope, No. 11SW-D/CR227, some 18 m to the northeast of BH7, recovered 4 m of highly to moderately decomposed granite overlying moderately to slightly decomposed coarse-grained granite.

Drillholes BH4, located on the footpath above the slope and some 40 m to the south of the landslide site, and BH5, located in the southeast corner of the slope some 25 m to the east of BH4, encountered fine- to medium-grained, slightly decomposed granite almost immediately below the ground surface.

The ground conditions in the vicinity of the landslide site were interpreted by FHK (1985b) from face mapping and ground investigation to comprise approximately 5 m completely decomposed granite overlying about 10 m completely to highly decomposed granite, which in turn overlies slightly decomposed granite (see Appendix A). OAP's geological models (see Figure 3) for the area in close proximity to the landslide as presented in their geotechnical submissions to the GEO (OAP, 1993a & 1993b) are generally consistent with FHK's geological models.

Examination of the landslide scar generally confirmed the above sequence. The completely and highly decomposed granites were mapped within the landslide scar as being derived from medium- to coarse-grained granite. Additionally, detailed geological mapping revealed the presence of an adversely orientated relict joint plane, occasionally kaolin-coated, dipping at angles between 40° and 45°, as well as two sub-vertical relict joint sets.

4.3 Groundwater Conditions

The closest groundwater monitoring instrumentation to the site of the 1998 landslide are the standpipe installed in drillhole BH9 and the standpipe piezometer installed in drillhole BH13 (Figure 2). Both instruments were installed with tips below rockhead, at 8.4 m and 7.6 m below the ground surface respectively. These instruments were monitored between 2 and 19 September 1991. The water levels recorded at this time in the standpipe indicate a groundwater level close to the ground surface whilst the levels recorded in the standpipe piezometer indicate a water level on the slope of about 6.8 m below ground level. This suggests that the water level in the soil portion can be very high locally or alternatively that there was a blockage in the standpipe.

In addition, observation wells were installed in drillholes BH4, BH5, BH6 and BH7. Standpipe piezometers were also installed in drillholes BH6 and BH7. The observation wells were installed with tips below rockhead at 15.0 m, 15.7 m, 15.0 m and 19.0 m below the ground surface respectively whilst those of the standpipe piezometers were installed just above the soil/rock interface at 4.5 m and 4.25 m below the ground surface respectively. These instruments were monitored between 12 September 1983 and 8 December 1984.

The water level recorded in the observation well installed in drillhole BH7 is in general agreement with the monitoring records of the adjacent drillholes. The standpipe piezometer which was also installed in this drillhole, however, recorded water levels within the highly decomposed granite generally about 3 m below ground level suggesting the potential for the development of transient elevated water pressures in these surficial deposits.

The available groundwater monitoring records together with post-failure field observations indicate that, in general, within the vicinity of the landslide, there is no permanent high groundwater table within the slope (Figure 4).

5. SURFACE WATER DRAINAGE ABOVE THE LANDSLIDE

No surface drainage channels exist within the immediate vicinity of the landslide scar. However, an old concrete kerb, apparently constructed as an intercepting drain, is located approximately 8 m above the backscarp of the landslide (Plate 7). This drain and the area immediately behind were noted as being overgrown with vegetation during FSW's post-landslide field inspections. Much domestic refuse has also been dumped in the area generally (Figures 4 and 5). In addition, a 2 m length of this intercepting drain, directly above the landslide scar, was noted by FSW as being slightly dislodged and partially broken, possibly providing a means for overland flow to get onto the slope below during rainstorms (Figure 5, and Plates 8 and 9).

A system of surface channels, carrying both storm and foul water, is present in the So Kon Po Cottage Area above (Figure 5). Whilst generally in a good condition, apart from one section that was blocked by leaves along the footpath outside cottages 161-165, these drains have sharp bends at certain locations which may lead to overtopping during intense rainstorms.

6. RAINFALL

The nearest GEO automatic raingauge to the landslide is raingauge No. H07 located on the roof of Block C/D of the Leighton Hill Flats, Broadwood Road, about 450 m to the northwest of the site (Figure 1). The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the GEO. For the purposes of rainfall analysis it has been assumed that the landslide took place at approximately 2:15 p.m. on 24 May 1998. This corresponds to the peak hourly rainfall recorded on that day.

The daily rainfall recorded by this raingauge for one month preceding and seven days following the event is presented in Figure 6. The corresponding hourly data for the period from 1:15 p.m. on 22 May 1998 until 10:15 p.m. on 24 May 1998 is also shown on Figure 6. This plot clearly shows that the short-term duration rainfall had an intense peak of about 70 mm/hr between 1:15 p.m. and 2:15 p.m. on 24 May 1998. Isohyets of rainfall for the 24-hour period preceding the landslide are given in Figure 7.

Table 2 presents the estimated return periods for the maximum rolling rainfalls recorded at raingauge No. H07 for selected durations based on historical rainfall data recorded at the Hong Kong Observatory (Lam & Leung, 1994). The 1-hour maximum rolling rainfall, a short-term duration storm, was the most severe with a corresponding return period of about 5 years. Whilst it is acknowledged that this simplified method of analysis does not necessarily give the true rainfall return period for a particular site as several contributory factors are not taken into account (Wong & Ho, 1996b), it does, nonetheless, provide an indication of the likely relative severity of the various rainfall characteristics assessed.

Based on the above analysis, it can be noted that the rainstorm on 24 May 1998 although periodically intense was not particularly severe. This observation is confirmed by Figure 8 which presents a comparison between the pattern of rainfall preceding the 1998 landslide and that of selected previous major rainstorms recorded at raingauge No. H07 since its installation in the mid-1980s.

7. THEORETICAL STABILITY ANALYSIS

Theoretical stability analyses were carried out to assist in the diagnosis of the probable causes and mechanisms of the landslide. The purpose of these analyses was to investigate the likely range of operational shear strength parameters along the slip plane corresponding to different groundwater conditions at the time of failure.

A cross-section through the landslide site on which stability analyses were carried out, based on the rigorous solution of Morgenstern & Price (1965), is presented in Figure 9. The pre-failure slope profile was established from topographical survey plans whereas the geometry of the landslide surface was based on site measurements made by FSW and post-failure topographical survey by Henry Chan Surveyors Limited.

The soil strength parameters assumed for the decomposed granite have been based on the generalised parameters given for these materials in Table 8 of Geoguide 1 (GEO, 1993), the results of laboratory tests carried out on the matrix material (Geotechnics & Concrete Engineering, 1983) as well as an assessment of the nature of the material observed in the field. In addition, cognisance has been taken, in particular, of the presence of an adversely orientated, kaolin-coated relict joint plane within the lower part of the landslide scar. The soil strength parameters adopted in the stability analyses for the decomposed granite, are also given in Figure 9. The range adopted includes approximately the values for the shear strength parameters adopted by both FHK and OAP in their earlier analyses. Various assumptions of transient elevated water pressures above the observed surface of rupture were also considered during the stability analyses. These transient elevated water pressures could result from perching above a relatively impermeable layer or seepage pressure due to subsurface flow.

The results of the stability analyses are summarised in Figure 10. The presence of an adversely orientated, locally kaolin-coated, relict joint plane, which partly controlled the failure, suggests that the likely operational value of c' along the observed surface of rupture was relatively low. Given this, it is also likely that a relatively modest increase in water pressure would have been sufficient to initiate failure.

It should be noted that given the shallow nature of the failure and the possible presence of locally open joints due to the influence of tree root growth, it is possible that the development of cleft (joint) water pressure also played a role in causing the failure in addition to, or instead of, the build-up of elevated water pressure within the soil mass. Nevertheless, the general observation is that the failure could have resulted from a relatively modest increase in water pressure.

8. PROBABLE CAUSES OF THE LANDSLIDE

8.1 Factors Contributing to the Landslide

The correlation between rainfall and the probable time of failure demonstrated previously in Section 6 indicates that the landslide was most likely to have been triggered by rainfall. Furthermore, based on the information collected during this study, it is postulated that the landslide is attributable to a combination of the following principal factors:

(a) the oversteep slope profile within the vicinity of the landslide (65°-75°) and presence of adversely orientated, kaolin-coated relict discontinuities within the weathered rock mass,

(b) the development of transient elevated pore water pressures within the corestone-bearing granitic saprolite following rainfall, and

(c) inadequate slope maintenance.

8.2 Probable Causes of the Landslide

The landslide occurred in corestone-bearing decomposed granite standing at an angle of between 65° and 75° in an area with a history of past instability. The generally poor condition of the chunam cover (See Section 2.1) would have rendered the slope more susceptible to direct infiltration (Plate 10). The extensive tree root growth evident could also have wedged open the soil mass, particularly the adversely orientated relict joint (Plates 5 and 6).

General blockage of the intercepting drain near the slope crest due to inadequate maintenance could have resulted in an increased discharge onto the slope below and provided an additional source of water (Plate 7). In addition, the dislodged and locally broken concrete kerb, part of the intercepting drain above the backscarp of the present failure, may have directed surface water flow onto the landslide site, further promoting water ingress (Plates 8 and 9).

Water ingress into the near-surface material would have wetted up the soil mass. Localised pore water pressures would thus develop as a result of seepage flow through the soil mass, along preferential flow paths, or possible filling up of joints wedged open by root growth. Given the presence of local geological weaknesses (i.e. the kaolin-coated relict joint plane) as well as the oversteep nature of the slope geometry, a relatively modest increase in pore water pressure was probably sufficient to destabilise the slope locally. This is consistent with the findings of the supporting theoretical stability analyses.

Additionally, the occurrence of the 1998 landslide during a period of rainfall that was less severe than previous rainstorms suggests that there may have been changes in environmental factors or that the slope could have suffered progressive deterioration, possibly exacerbated by inadequate maintenance.

Poor detailing of the surface drainage provision (e.g. sharp bends) in the So Kon Po Cottage Area at the crest of Slope No. 11SW-D/C1007 is liable to lead to overtopping during intense rainstorms, which may be a possible further source of upstream water flow directed to the landslide site. The contribution of such overtopping to the 1998 landslide is, however, uncertain.

8.3 Other Observations

The slope upgrading proposals made by OAP, and subsequently approved by the GEO, comprised cutting back portions of the slopes, including the area of the 1998 landslide, and providing soil nails in other areas as appropriate. A Geotechnical Engineer from OAP was on site during the stadium redevelopment works to deal with matters such as the slope upgrading works.

Site observations show that the part of the slope affected by the current landslide has not been cut back to a shallower angle in accordance with the approved drawing. No records have been found of re-submissions to the GEO by OAP covering this amendment to the approved design. The Arch SD have advised that neither details of this amendment nor any record plans for the slope upgrading works have been submitted to them.

According to OAP following the landslide, the slope that failed was not cut back as per the approved drawing because "this section of the slope was in fact rock and would therefore not have been required to be cut back as if it were a soil slope" (OAP, 1999). This is different to the geological models presented for the adjacent sections, 5-5 and 6-6, in the design submissions made by OAP. Detailed mapping of the landslide scar by FSW after the landslide revealed that the failure occurred in saprolite which is consistent with the previous geological models postulated in the 1985 and 1993 detailed studies by FHK and OAP respectively and the Incident Report prepared by the GEO shortly after the failure (see Appendix C).

A review of the available information indicates that there is a clustering of past failures at and in close proximity to the 1998 landslide site. The form of the 1998 landslide appears to be similar to these past failures and similar failures may occur in future if no slope improvement works are carried out.

9. CONCLUSIONS

It is concluded that the landslide, which occurred on 24 May 1998 at Slope Nos. 11SW-D/CR227 and 11SW-D/C1007, was triggered by rainfall. The failure affected a portion of the slope that had been subject to detailed stability assessment but on which no upgrading works were carried out in accordance with the approved design.

The 1998 landslide was probably caused primarily by transient elevated water pressures in the near-surface slope-forming materials. The oversteep nature of the slope face, presence of an adversely orientated relict joint plane with weak infill and inadequate slope maintenance are contributory factors to the failure. The area in the vicinity of the landslide has experienced repeated failure, which may be an indication of inherent weakness of the material and/or unfavourable groundwater conditions.

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Table 1 – Summary of Site Development from Aerial Photograph Interpretation

Year	Photographic Reference No.	Altitude (feet)	Observations (refer to Figure 2)
1945	681/6, 4066, 4067	20,000	The site of the current Hong Kong Stadium is largely undeveloped and comprises a valley and associated foothills to the south and west. The northern portion of the site of the current stadium is however, under cultivation, as were some of the lower foothills. Several small valleys are also evident, draining in a generally southwest-northeast direction down from the western spur. The present site of the So Kon Po Cottage area is undeveloped at this time.
1949	6116, 6117, Y01423, Y01424	8,600	Extensive drainage works have been constructed in the valley (running in a generally southwest-northeast direction across the centre of the present site of the Hong Kong Stadium) and the foothills to the west, draining in general from west to east. Cultivated plots are also evident on the eastern floodplain of the valley. Numerous minor surface erosion scars are evident on the surrounding moderately to densely vegetated hillsides.
1961	F43.81/600, 0127, 0128	30,000	The original Hong Kong Stadium has now been constructed within the valley as have the associated cut slopes along the south-western edge of the stadium. The lower portion of these cut slopes appears to be almost vertical and cut in rock. The upper portion is at a shallower angle and appears to have been cut in soil. In addition, a lighting tower has been constructed approximately half way up these slopes whilst a drain is visible above the northern portion of the crest of the slopes. A perimeter road/path borders the edge of the stadium and runs along the toe of the cut slopes. The So Kon Po Cottage area has also been developed to a layout very similar to its present form.

Year	Photographic Reference No.	Altitude (feet)	Observations (refer to Figure 2)
1963	7061, 7062 8019, 8020	2,700 3,700	A crest drain is evident bordering the cut slopes under study as is a toe drain which is visible at the northern end of the cut slopes. Several small features are visible along the lower sections of the cut slope which may be downpipes associated with the masonry walls at the base of the cut slope. The lighting tower platform is evident on the slope and there appears to be a rock outcrop/overhang to the south of this platform approximately halfway up the slope.
1967	Y13281, 13282	6,250	The slopes are largely in shadow, with trees well-established on the upper portion of the slope.
1972	1833, 1834	2,500	Bare patches (possible erosion scars) are evident at the southern end of the stadium. The slopes under study appear well-vegetated. Additional buildings are evident on the hillside above these cut slopes to the north of the So Kon Po Cottage area.
1976	12642, 12643	4,000	There appears to be standing water on the perimeter track beneath the cut slopes. A wall is visible along the lower section of the southern portion of these slopes. Some small erosion scars are also evident at the southern and northern ends of these slopes, and also at about the location of the present failure.
1977 1978 1979 1980 1980 1981	18434 19684 20459, 20460 23867, 23868 26912, 26913 29837, 29838 32413, 32492 37401, 37154	4,000 9,000 4,000 4,000 4,000 4,000 4,000 4,000	No significant changes apparent. The slopes are often obscured by vegetation and/or shadow.
1982	40667, 40668 43054, 43055 45336, 45337	4,000 3.500 4,000	The slopes are largely obscured by vegetation and/or shadow. Major site formation works are ongoing on the ridge to the west (and south-west) of the stadium.

Year	Photographic Reference No.	Altitude (feet)	Observations (refer to Figure 2)
1983 (Oct)	49622, 49623	4,000	Small erosion scar present near the crest of the slope close to the location of the 1998 failure. The majority of the slopes are obscured by shadow and vegetation.
1984	56620, 56684, 56685	4,000	Vegetation has re-established itself on the erosion scar noted in 1983. Site works on the ridge to the west of the stadium are well-advanced.
1985	66032, 66033 A03784, A03785	4,000 4,000	Vegetation has been partly cleared on the slopes under study and some trimming works and shotcreting, especially on the upper sections of the slopes, appear to have been carried out.
1986 (Sept)	A06006, A06007	4,000	There are now many additional dwellings on the hillside to the north of the So Kon Po Cottage area. Possible erosion scars noted on the concerned slopes to the north and south of the lighting tower platform.
1986 (Nov)	CN1063, CN1064	5,000	There appears to be some fresh shotcrete/chunam below the crest, around mid-section, of the slopes under study.
1987	A10354, A10355	4,000	The slopes are well-vegetated and largely in shadow. High rise development is now present on the ridge to the west of the stadium. No significant changes apparent.
1988	A14491, A14492	4,000	Good quality photographs. There appears to be a patch of chunam/shotcrete on the slopes in the same position as one of the possible erosion scars noted in 1986. Also, some vegetation clearance has been carried out adjacent to the crest of the slopes under study, in front of Cottage No. 129C.
1991	A30940, A30941 A27734, A27735 A28061, A28062, A28063	4,000 4,000 4,000	A possible small erosion feature is visible to the south of the lighting tower. The slopes are heavily vegetated.

Year	Photographic Reference No.	Altitude (feet)	Observations (refer to Figure 2)
1992	A32504, A32505	4,000	The Hong Kong Stadium Redevelopment Project has commenced. The concerned slopes appear largely unchanged.
1993	A33854, A33855 A36991, A36992	2,500 4,000	The stadium is almost completed to its present form. The concerned slopes appear to be unaffected but are largely obscured by the new stadium roof.
1994 1995 1996 1997	CN7910, CN7911 CN12640, CN12641, CN12707 CN14104, CN14105 CN15558, CN15557 CN17055, CN17056 CN18537, CN18538	4,000 3,500 3,500 4,000 4,000 4,000 4,000	The concerned slopes are largely obscured by the new stadium roof and thick vegetation. No significant changes evident.

Table 2 - Maximum Rolling Rainfall at GEO Raingauge No. H07 and Estimated Return Periods for Different Durations Preceding the Landslide of 24 May 1998

Duration	Maximum Rolling Rainfall (mm)	End of Period (Hours)	Estimated Return Period (Years)
5 Minutes	11.0	13:40 on 24 May 1998	2 Years
15 Minutes	29.5	13:40 on 24 May 1998	3 Years
1 Hour	69.5	14:15 on 24 May 1998	5 Years
2 Hours	75.0	14:15 on 24 May 1998	< 2 Years
4 Hours	77.0	14:15 on 24 May 1998	< 2 Years
12 Hours	99.5	14:15 on 24 May 1998	< 2 Years
24 Hours	102.5	14:15 on 24 May 1998	< 2 Years
2 Days	102.5	14:15 on 24 May 1998	< 2 Years
4 Days	117.5	14:15 on 24 May 1998	< 2 Years
7 Days	140.0	14:15 on 24 May 1998	< 2 Years
15 Days	224.5	14:15 on 24 May 1998	< 2 Years
31 Days	492.5	14:15 on 24 May 1998	< 2 Years

Notes :

1 Return periods were derived from Table 3 of Lam & Leung (1994).

2 Maximum rolling rainfall was calculated from 5-minute data for durations up to 48 hours, and from hourly rainfall data for longer rainfall durations.

3 The use of 5-minute data for durations between 2 hours and 48 hours results in better data resolution, but may slightly over-estimate the return periods using Lam & Leung (1994)'s data, which are based on hourly rainfall for these durations.

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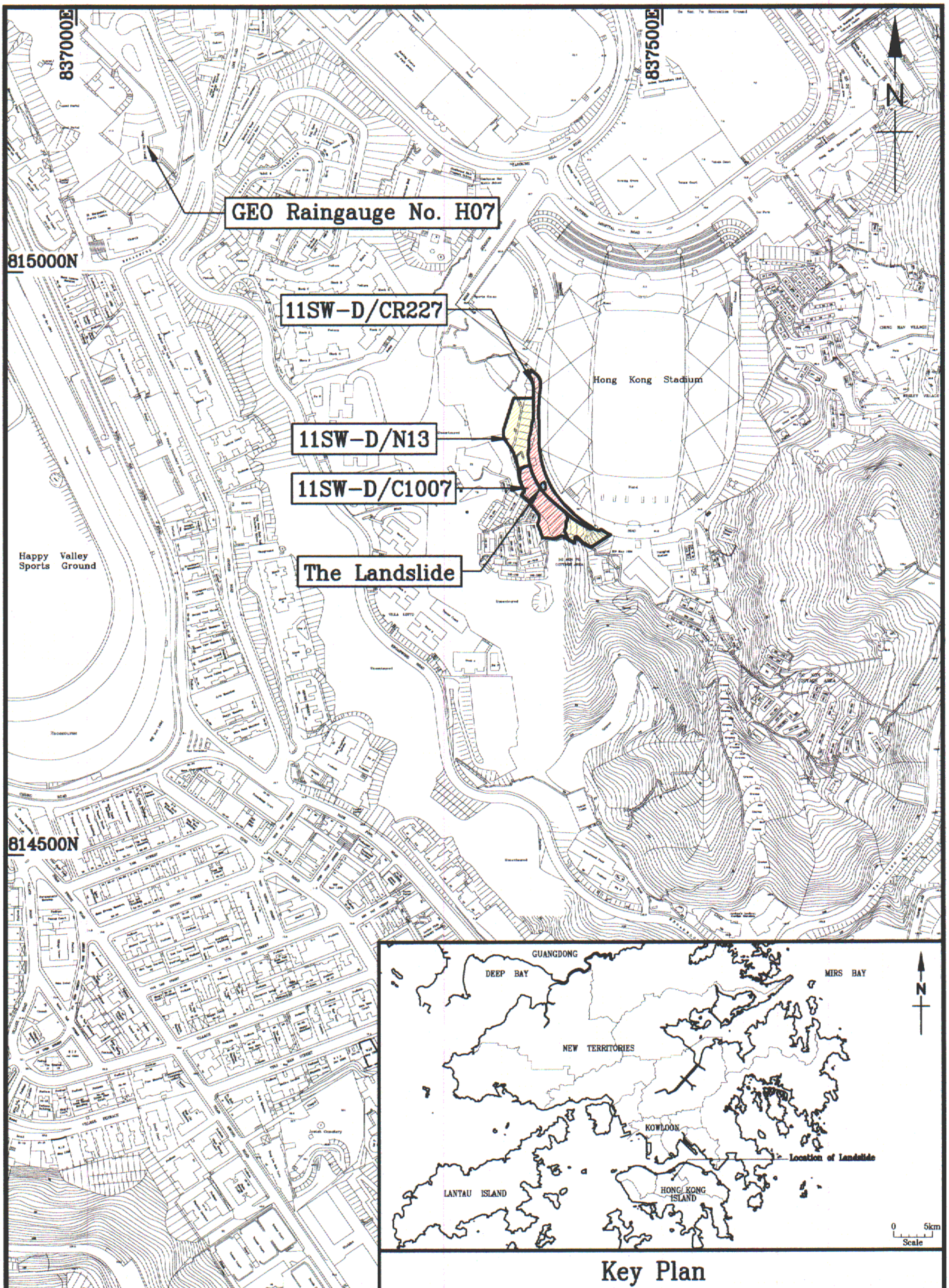
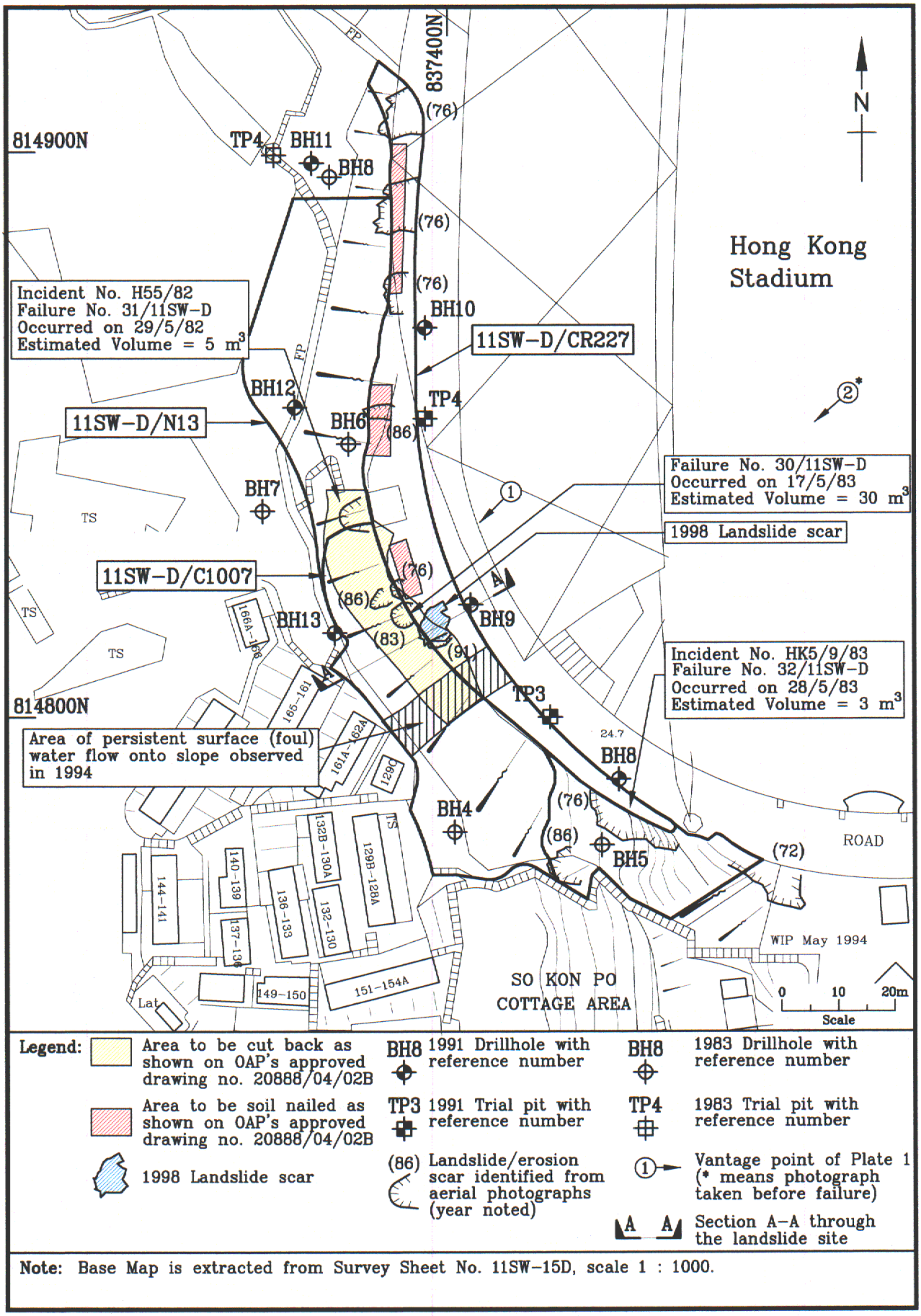


Note: Base Map is extracted from Survey Sheet Nos. 11SW-B, 11SW-D, 11SE-A & 11SE-C, scale 1 : 5000.

Figure 1 - Site Location Plan



Note: Base Map is extracted from Survey Sheet No. 11SW-15D, scale 1 : 1000.

Figure 2 - History of Site Development

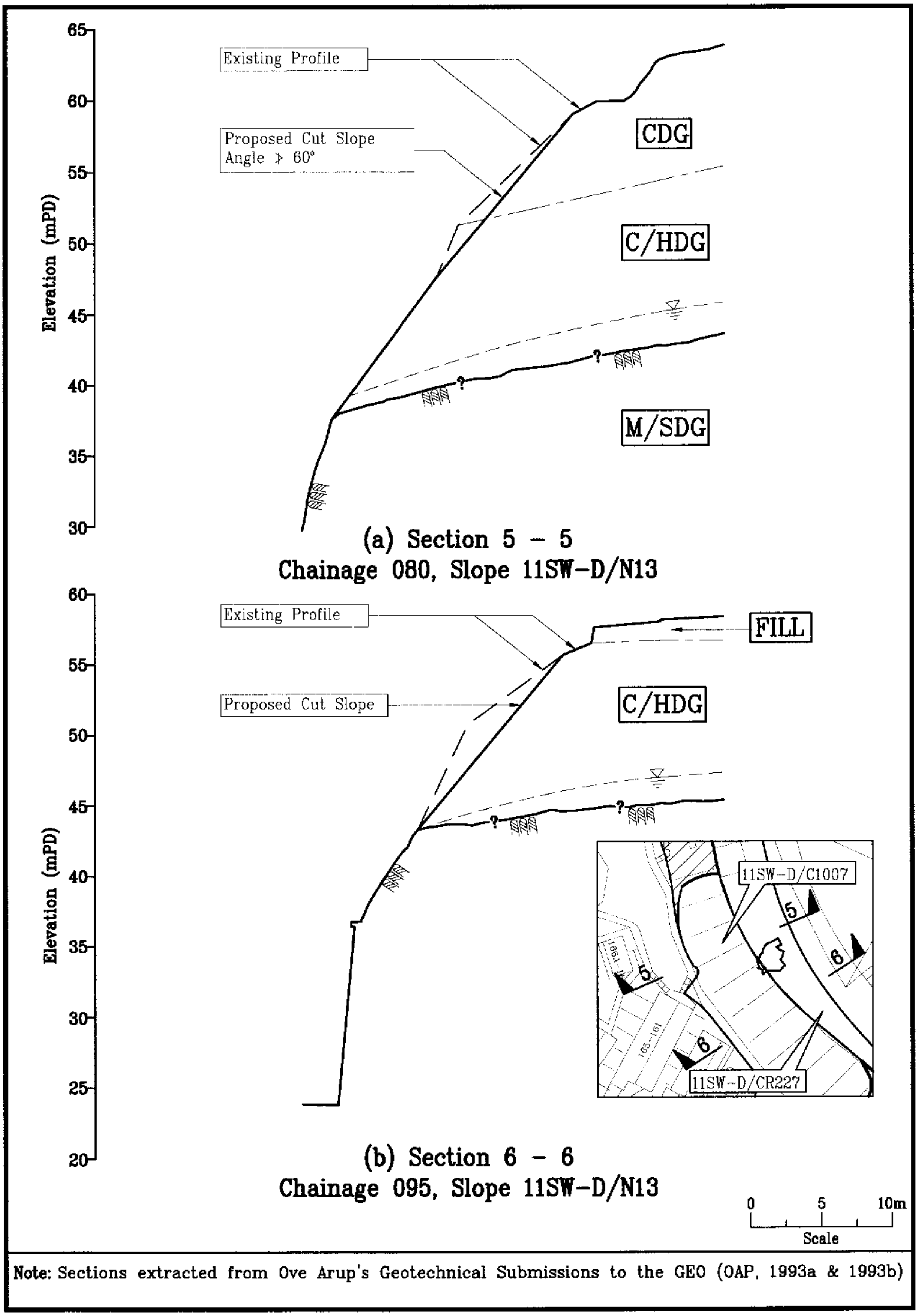


Figure 3 - OAP's Geological Models

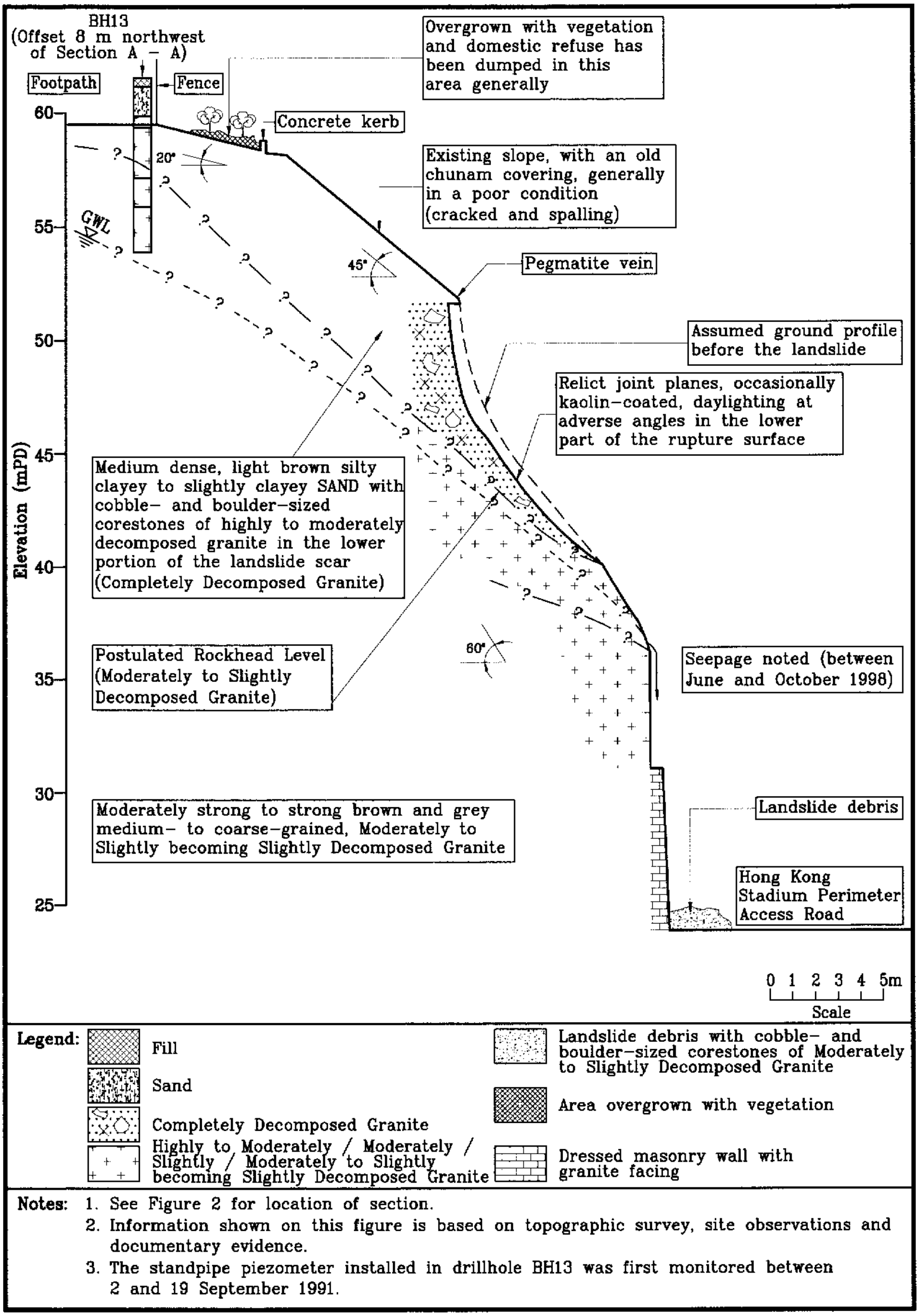


Figure 4 - Section A - A Through the Landslide

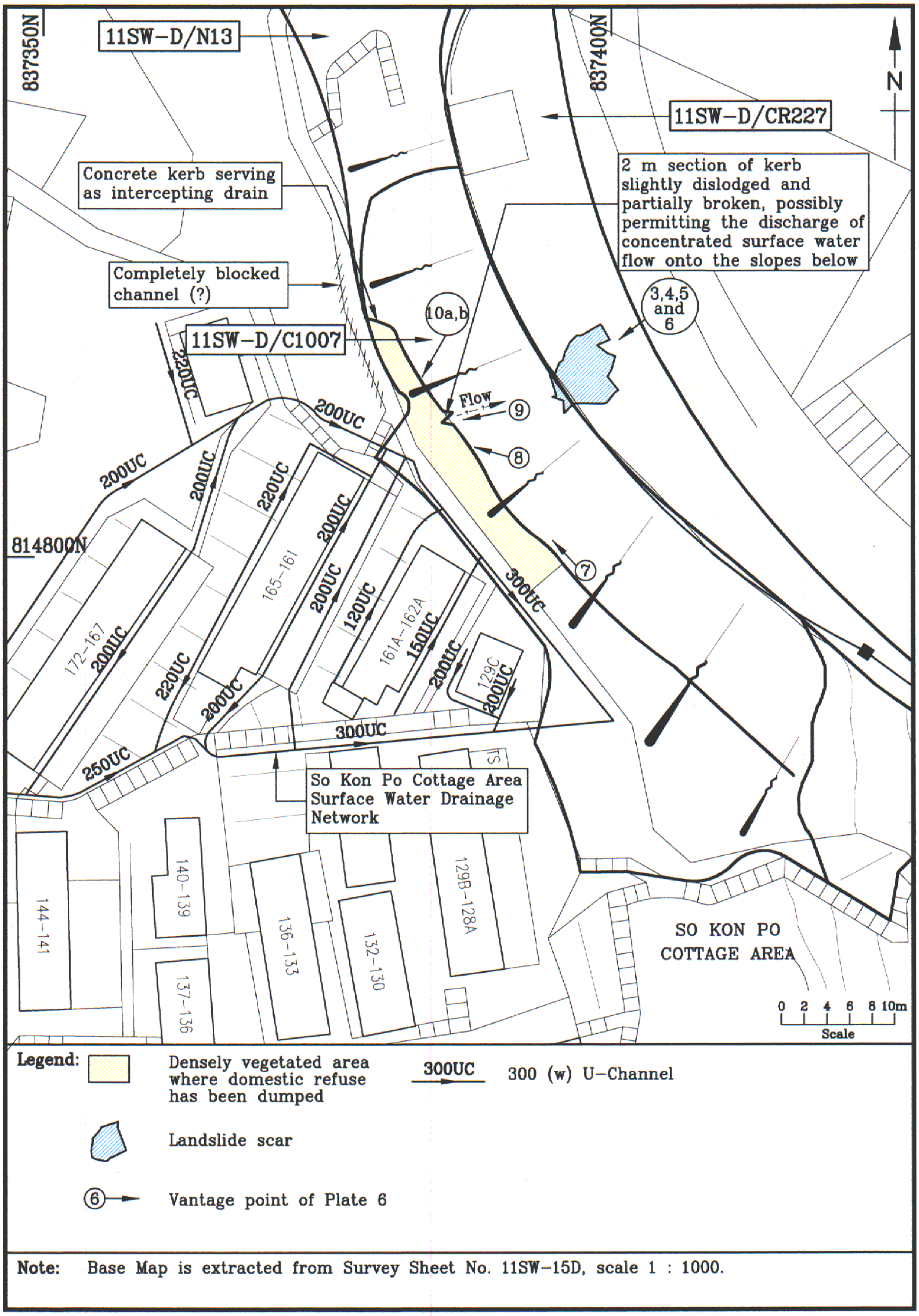
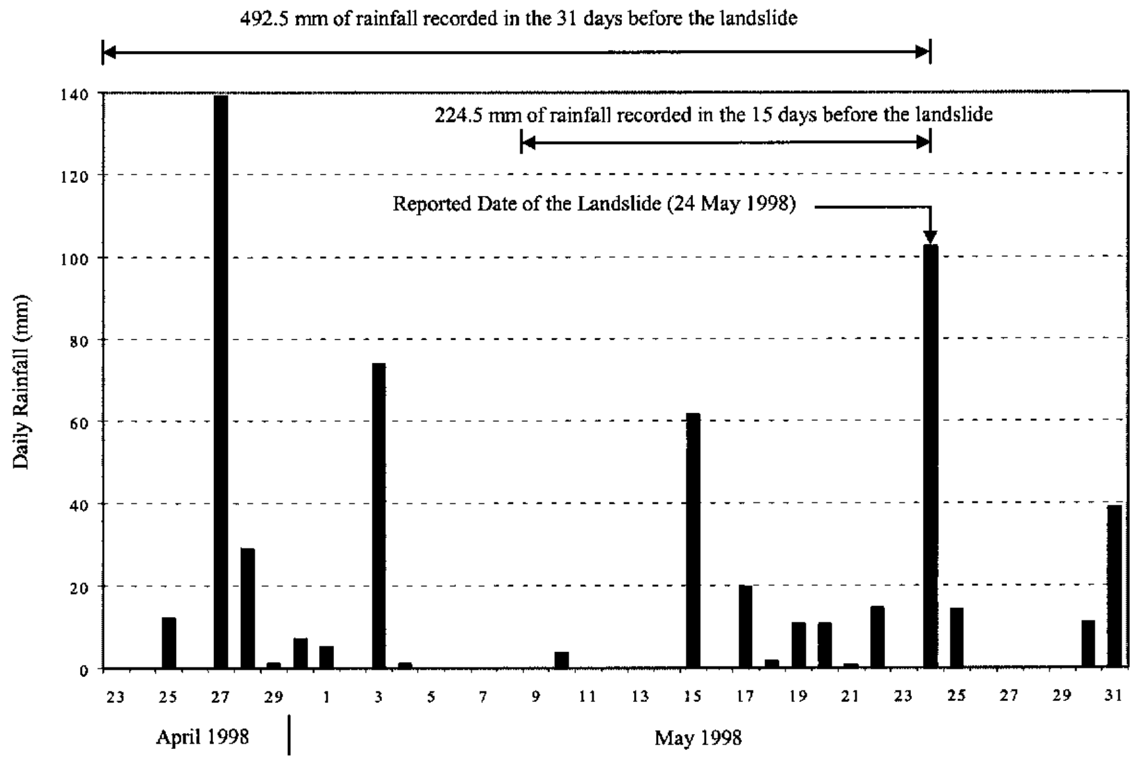
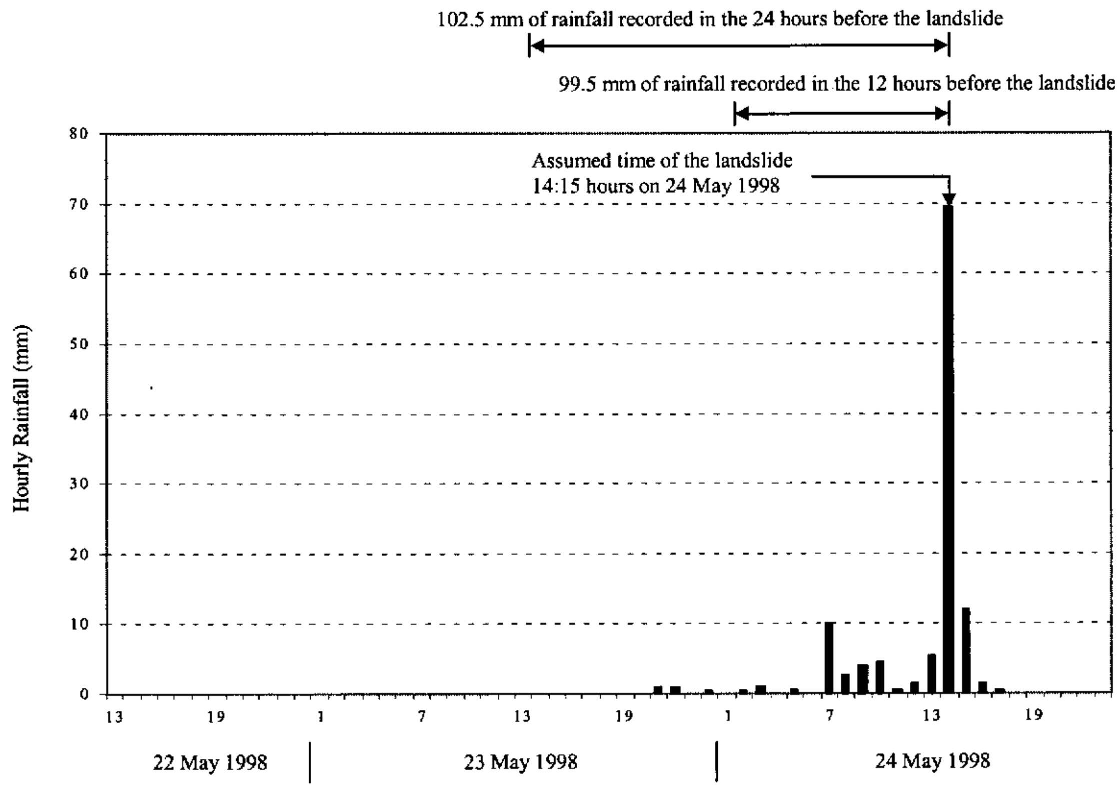


Figure 5 - Surface Water Drainage Networks

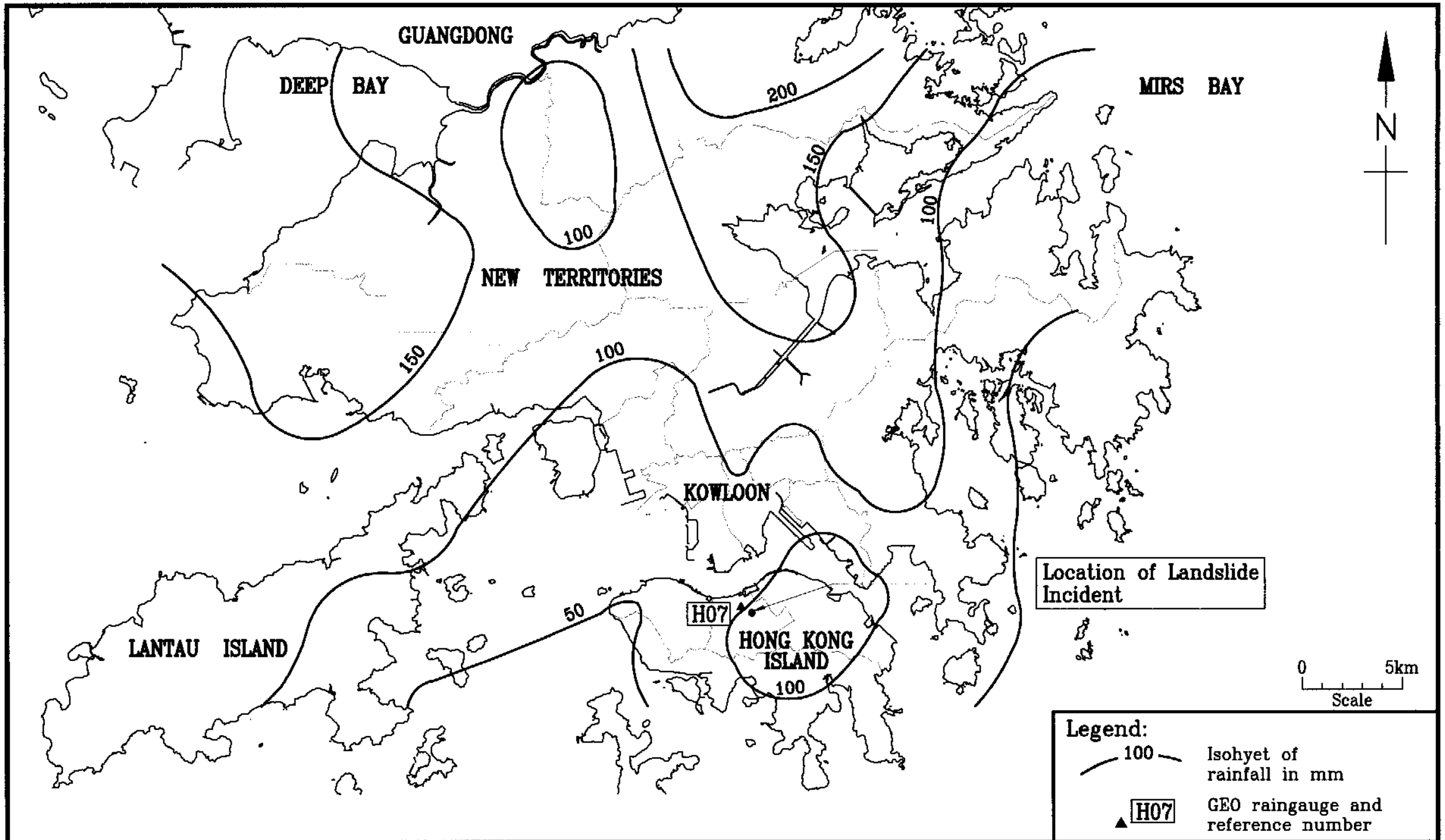


(a) Daily Rainfall Recorded between 23 April and 31 May 1998



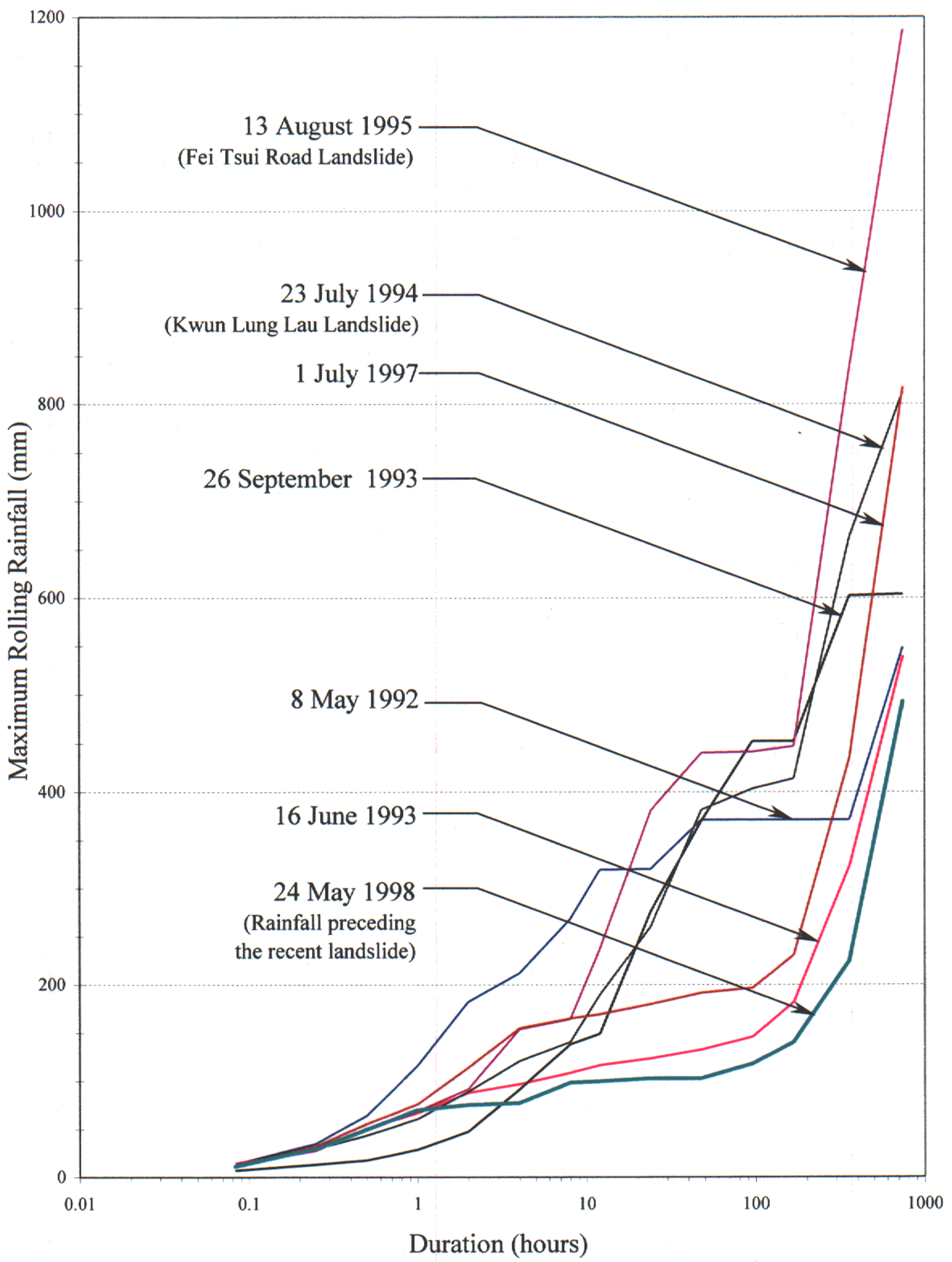
(b) Hourly Rainfall Recorded between 13:00 hours on 22 May and 24:00 hours on 24 May 1998

Figure 6 – Rainfall Recorded at GEO Raingauge No. H07



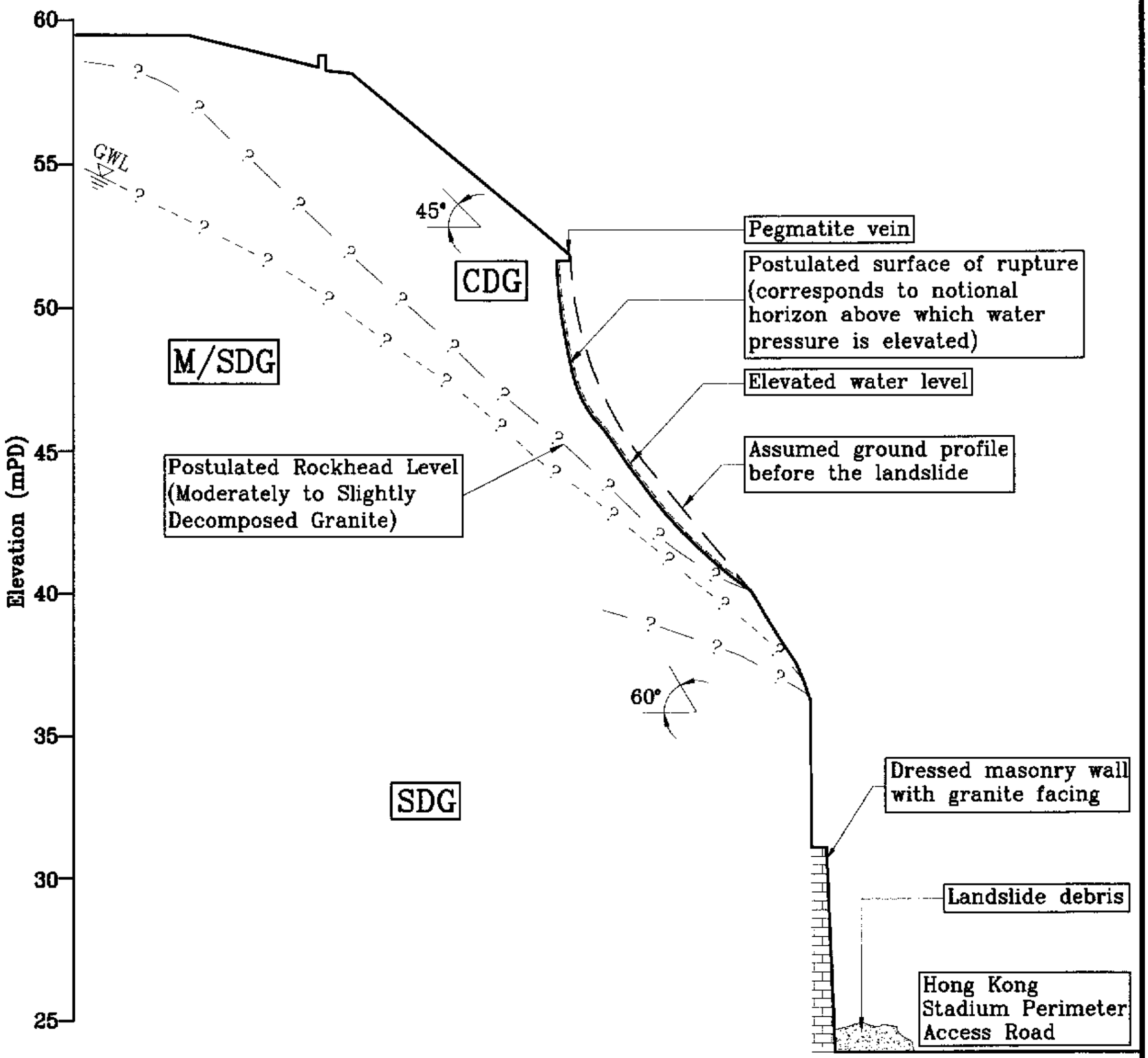
Note: Isohyets of rainfall for the period from 2:20 p.m. on 23 May 1998 until 2:15 p.m. on 24 May 1998.

Figure 7 - Rainfall Distribution in the 24 hour Period Preceding the Landslide of 24 May 1998



Note : Rainfall data are 5-minute statistics from GEO raingauge No. H07

Figure 8 - Maximum Rolling Rainfall Preceding the Landslide of 24 May 1998 and Selected Previous Major Rainstorms Recorded at GEO Raingauge No. H07



Section A – A Through the Landslide Site

Material	Range of Parameters Used in Analysis		
	Representative Saturated Mass Shear Strength		Unit Weight (kN/m^3)
	Effective Cohesion c' (kPa)	Effective Angle of Shearing Resistance (ø')	
Corestone–bearing Completely Decomposed Granite with kaolin–coated relict joint planes	2 – 8	30° – 40°	19

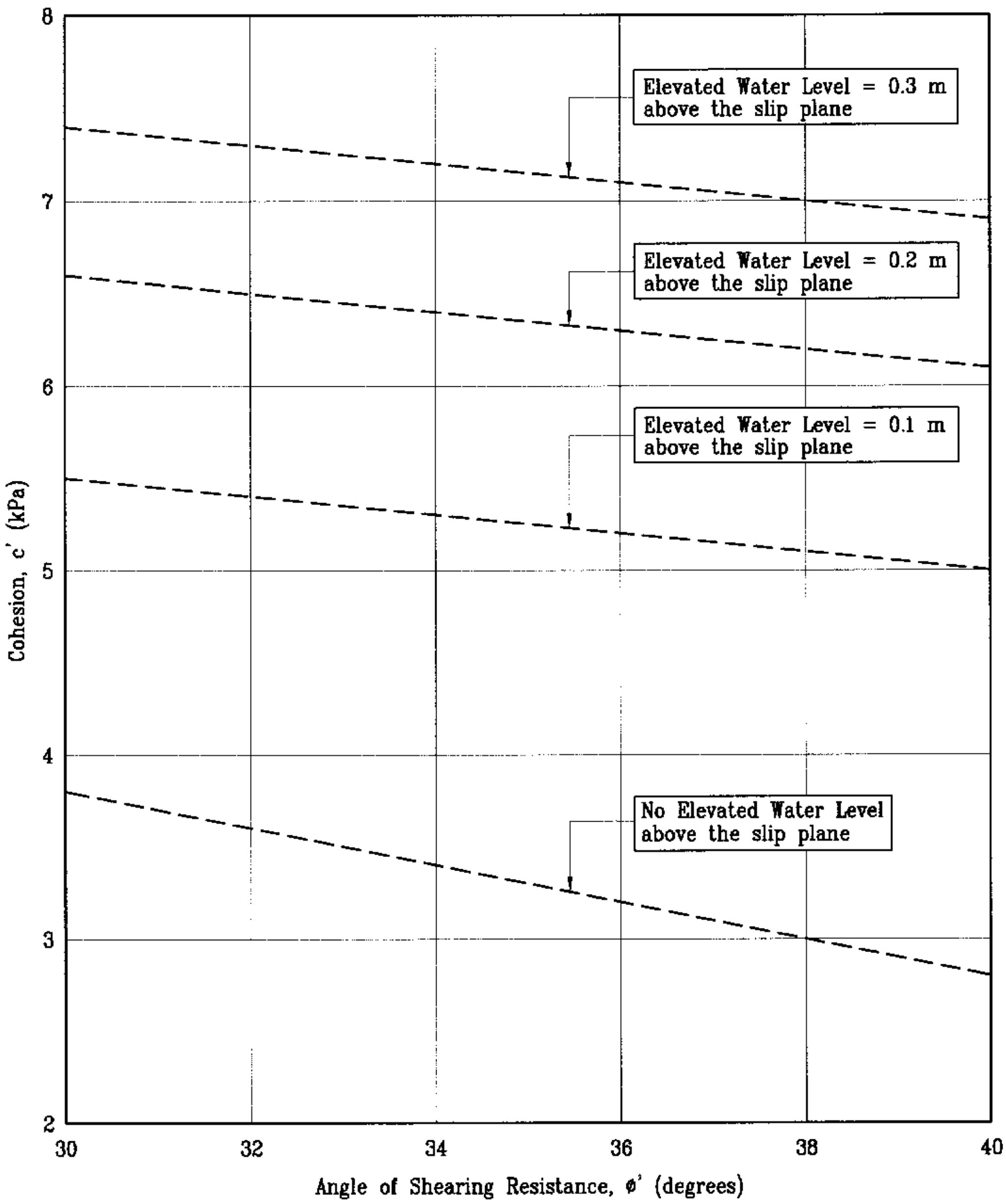
Scale: 0 1 2 3 4 5 m

Legend:

- Wall
- Landslide debris
- CDG Completely Decomposed Granite
- M/SDG Moderately to Slightly Decomposed Granite
- SDG Slightly Decomposed Granite

Notes:
1. See Figure 4 for location of section.
2. Information shown on this figure is based on site observations and topographic survey.

Figure 9 - Representative Cross-section of the Landslide for Theoretical Stability Analyses



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

Figure 10 - Summary of Theoretical Stability Analyses

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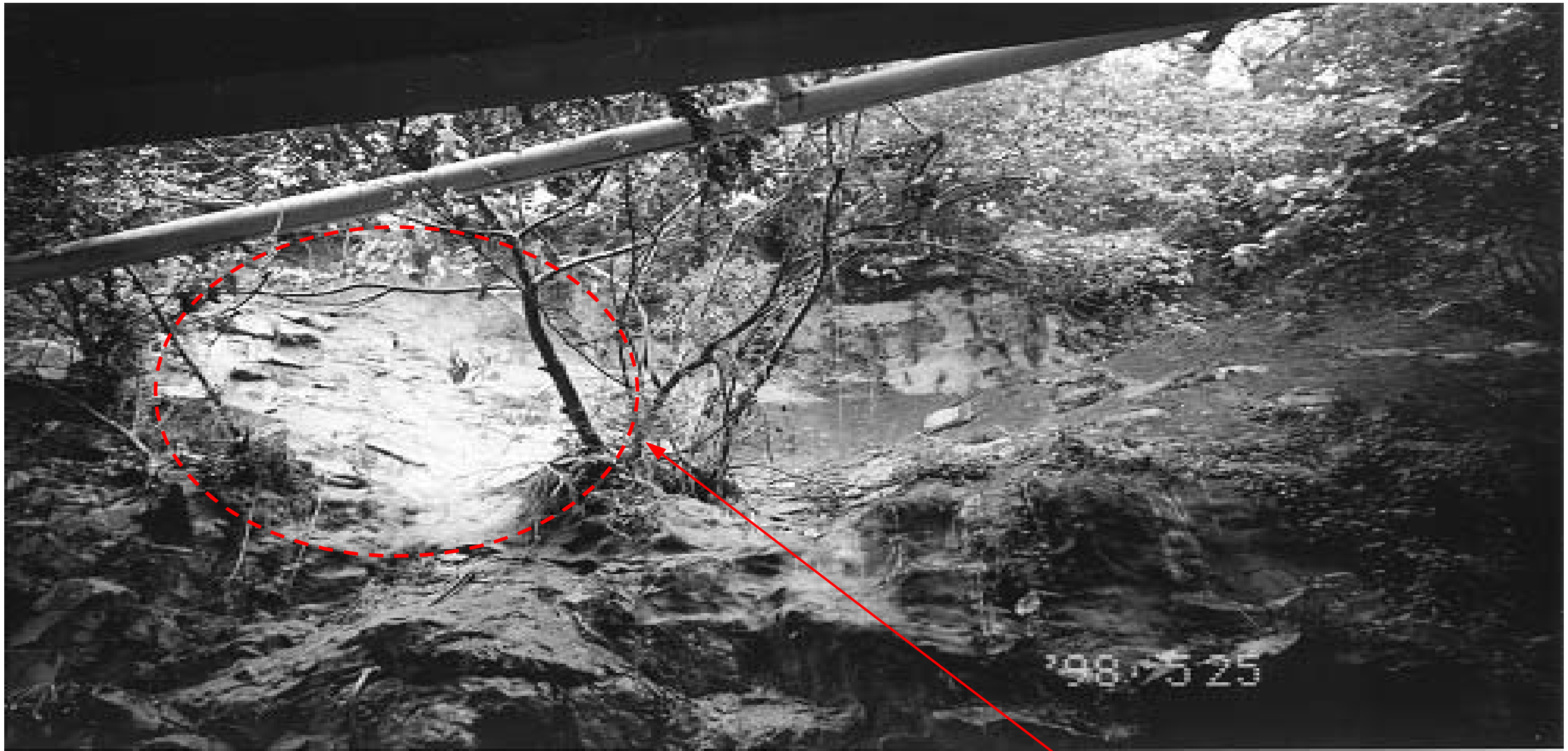


Plate 1 – The Landslide of 24 May 1998
(Photograph taken on 25 May 1998)

Approximate Outline of
the 1998 Landslide Scar

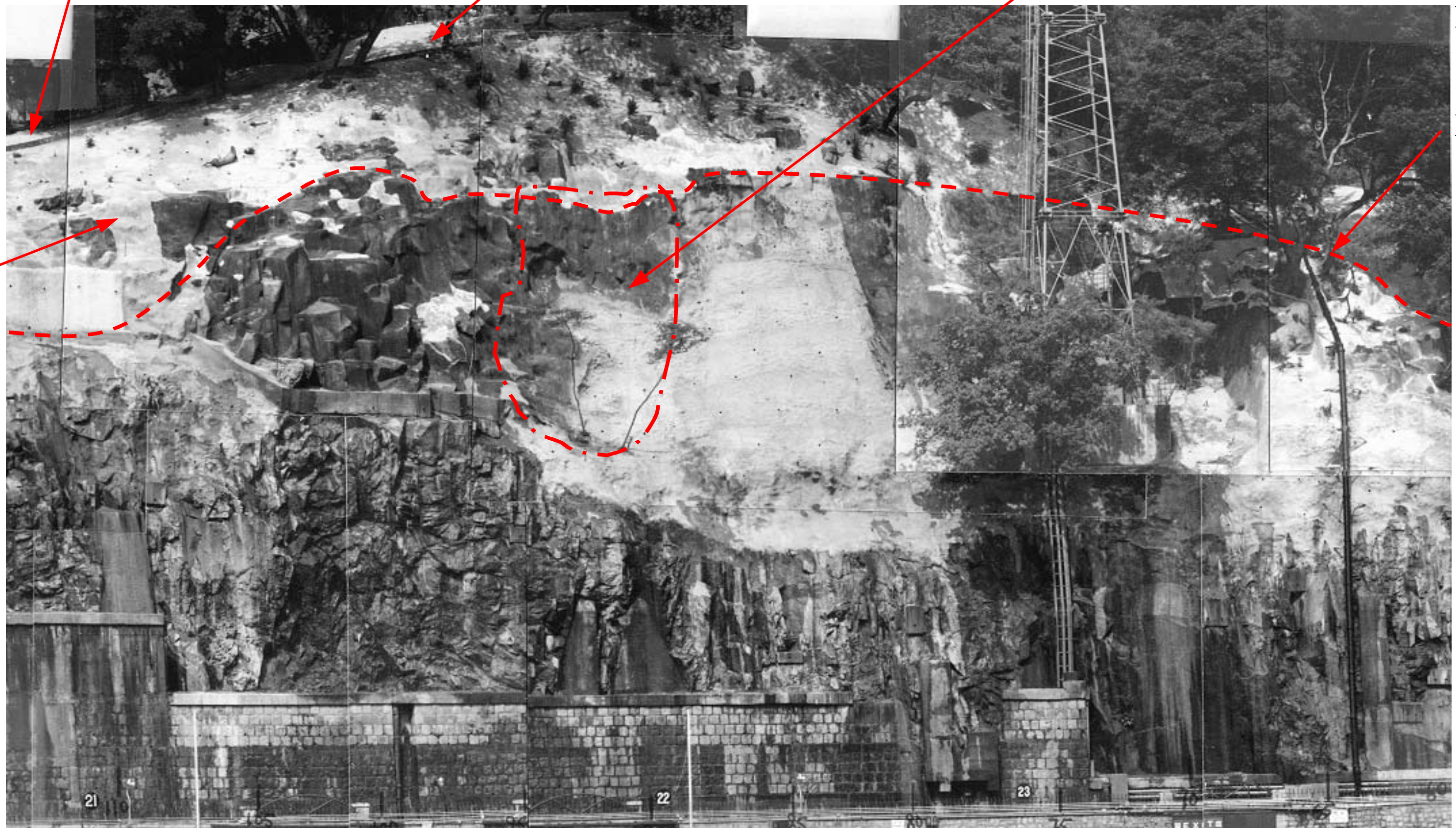


Plate 2 – Front View of the Slopes in 1985
(Photographs reproduced from Fugro (HK) Ltd's As-built Record Report dated June 1985)

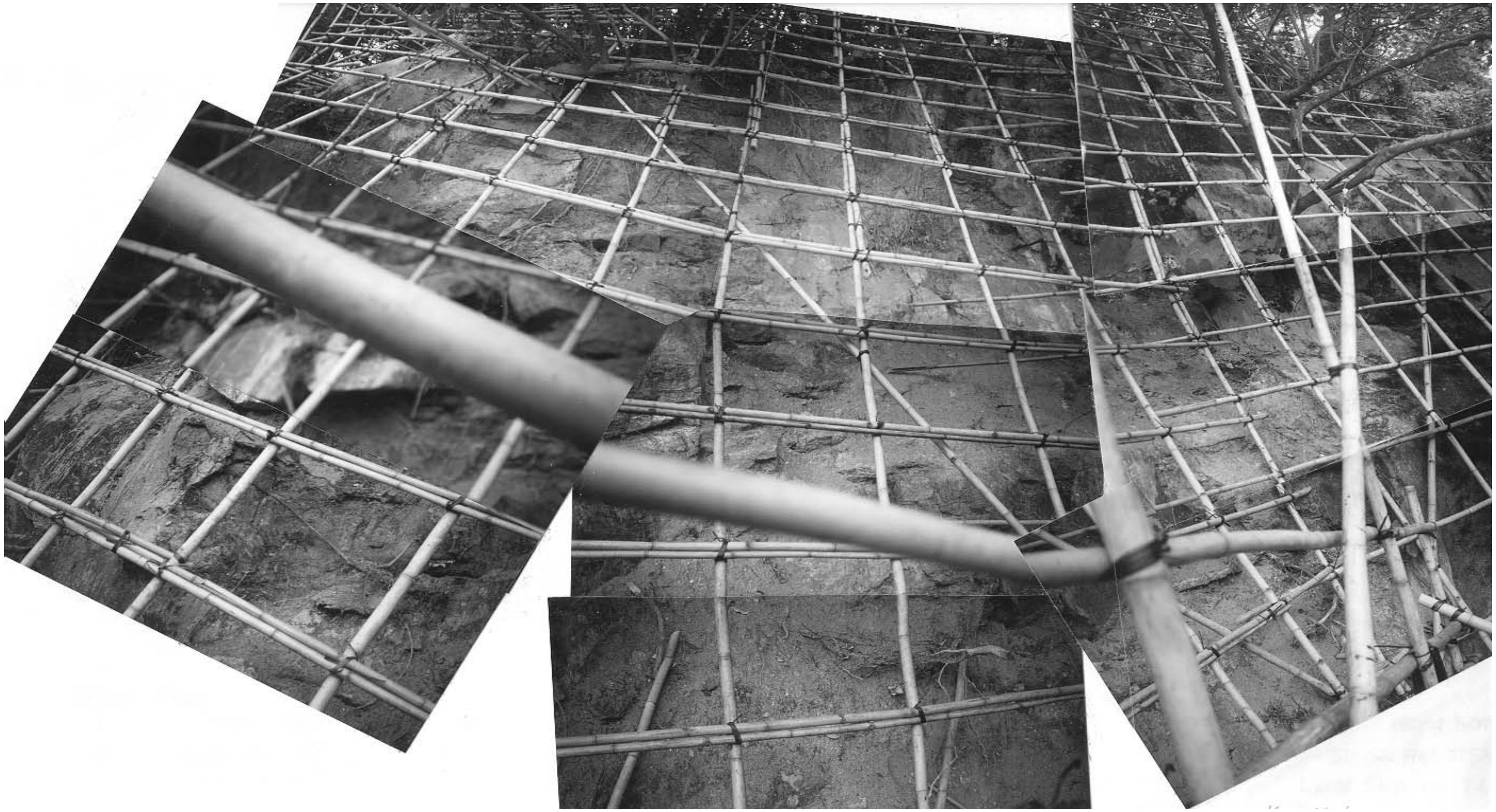


Plate 3 – General View of Landslide Scar
(Photographs taken on 24 May 1998)

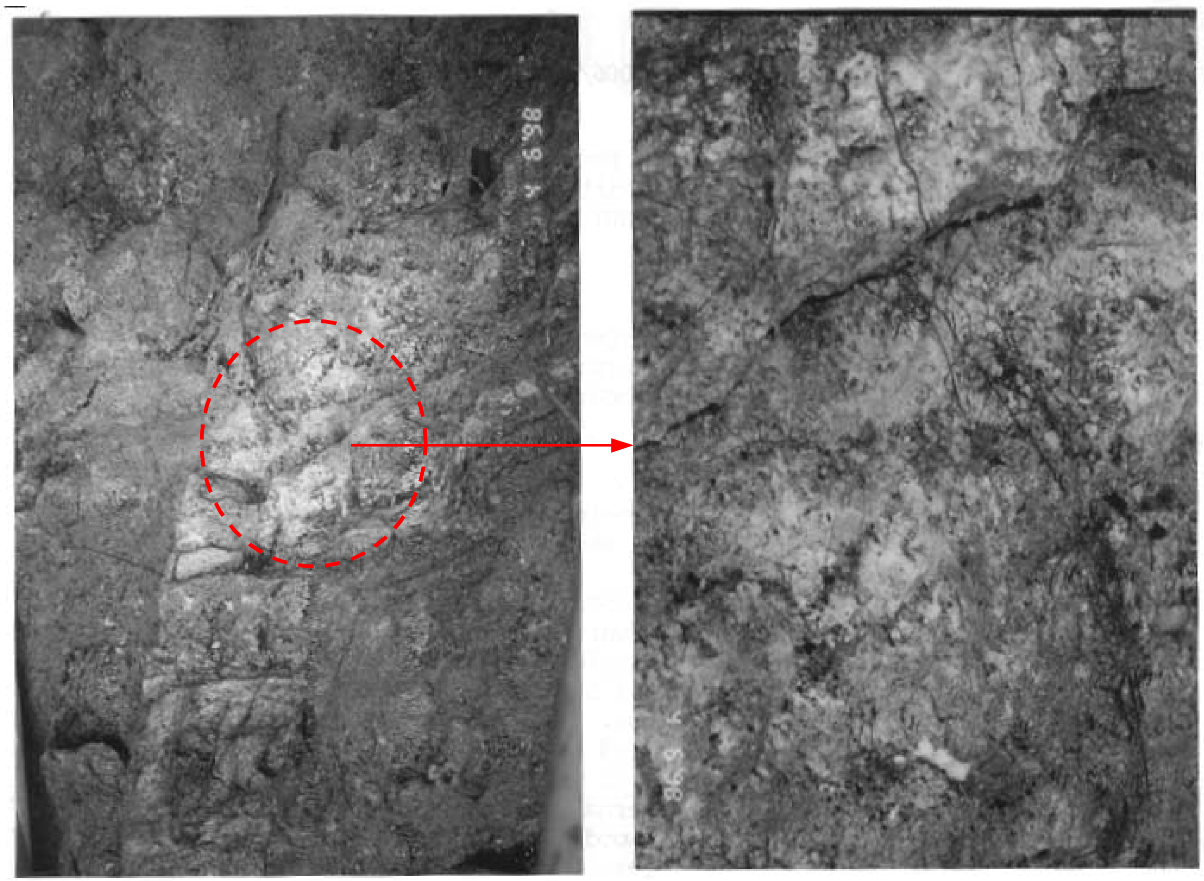


Plate 4 – Kaolin Patches Noted on Joint Planes within the Completely Decomposed Granite
(Photographs taken on 4 June 1998)

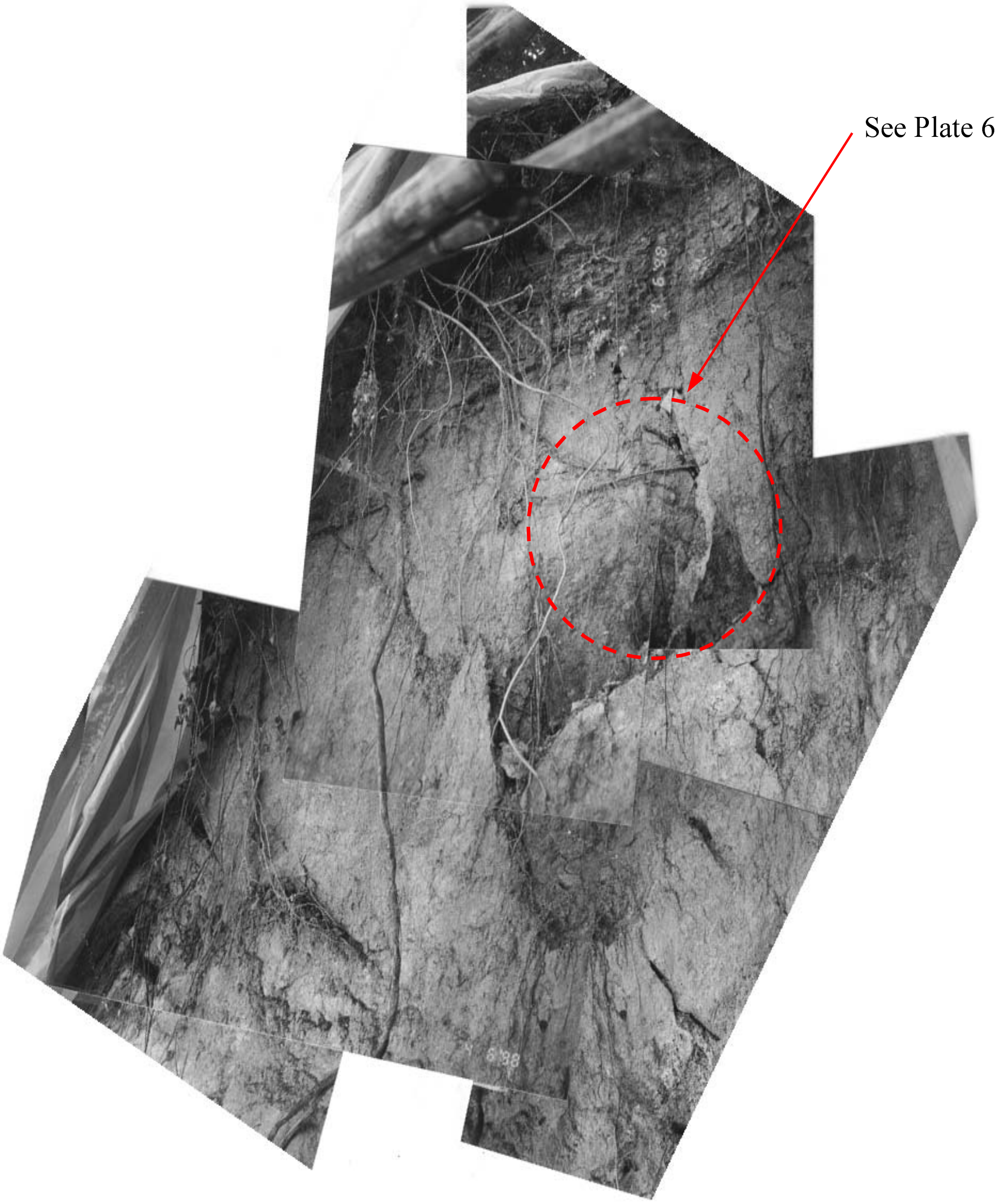


Plate 5 – Completely Decomposed Granite and Highly to Moderately Decomposed Corestones Exposed in the Upper Portion of the Landslide Scar. Note the Extensive Root Growth. (Photographs taken on 4 June 1998)

Plate 6 – View of Root Growth within the Landslide Backscarp and a Manganese-stained Relict Joint (Photograph taken on 4 June 1998)

Plate 7 – Blocked Drainage 8 m above Slope No. 11SW-D/C1007
(Photographs taken on 5 June 1998)

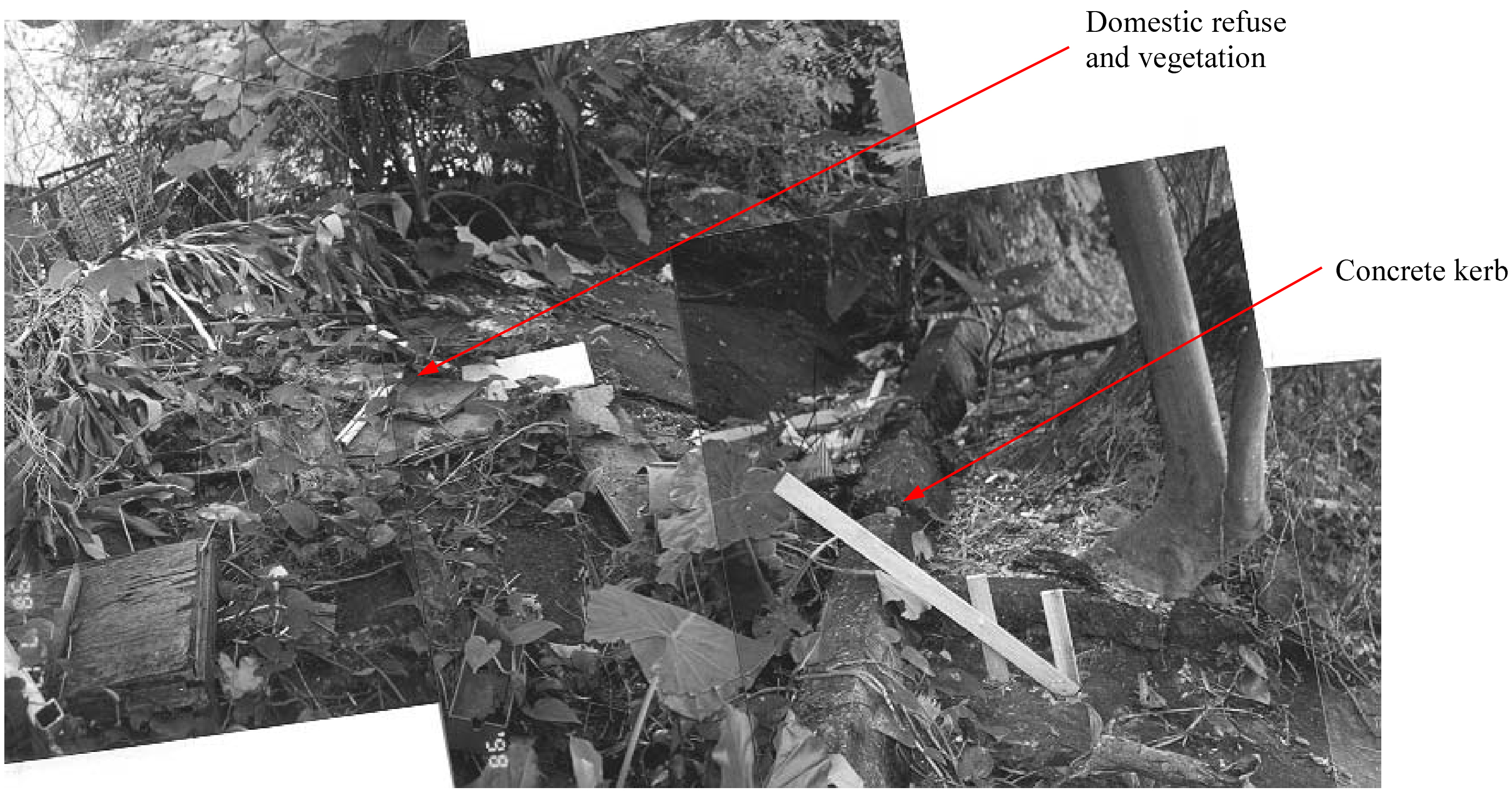


Plate 8 – Dislodged and Partially Broken Section of Concrete Kerb
(Photographs taken on 17 July 1998)

Plate 9 – Close-up View of Dislodged and Partially Broken Section of Concrete Kerb
(Photographs taken on 17 July 1998)

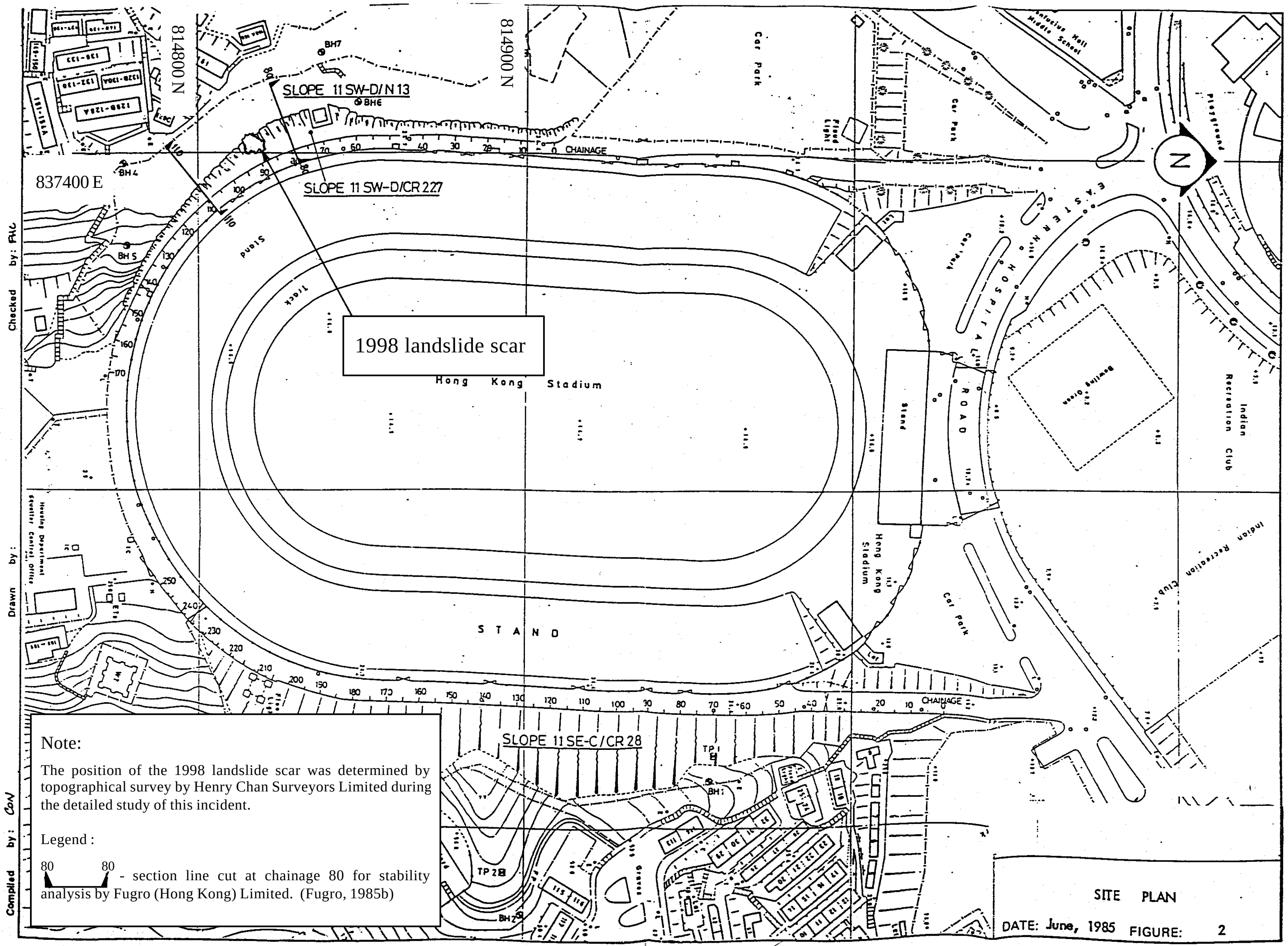
a)

b)

Plate 10 – General Views of Surface Covering to Slope above the Landslide Scar
(Photographs taken on 5 June 1998)

APPENDIX A

FUGRO'S STABILITY ANALYSIS



Note:

The position of the 1998 landslide scar was determined by topographical survey by Henry Chan Surveyors Limited during the detailed study of this incident.

Legend :

80 ⌊___⌋ 80 - section line cut at chainage 80 for stability analysis by Fugro (Hong Kong) Limited. (Fugro, 1985b)

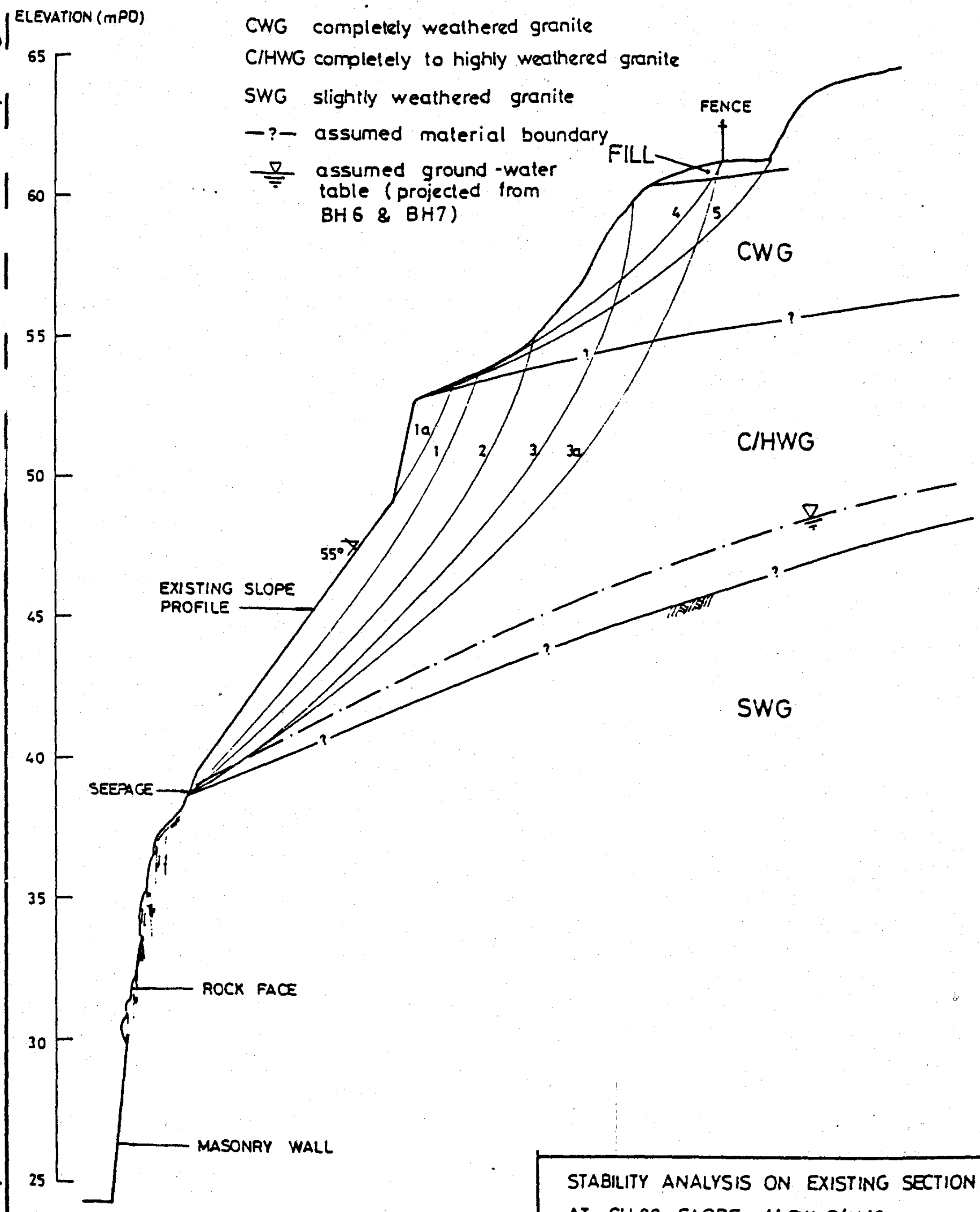
Slip surface	Factor of safety	
1	0.66	(1.28)*
2	0.78	(1.15)
3	0.84*	(1.11)
4	0.85	(1.42)
5	1.05	(1.68)
1a		(1.69)*
3a		(1.13)*

Material	Shear strength parameters		Bulk density
	C' (kPa)	Ø' (deg)	(kN/m³)
FILL	0	32	19
COLL	3.5	35	19
CWG	0 (8.5)	37 (39)	19
C/HWG	0 (8.4)	44	19

* GENERALISED FACTOR OF SAFETY

NOTE :- ( ) DENOTES REVISED PARAMETERS AND FACTOR OF SAFETY AS A RESULT OF BACK ANALYSIS

LEGEND :-
CWG completely weathered granite
C/HWG completely to highly weathered granite
SWG slightly weathered granite
—?— assumed material boundary
assumed ground-water table (projected from BH6 & BH7)



STABILITY ANALYSIS ON EXISTING SECTION AT CH 80, SLOPE 11 SW-D/N 13

Date: June 1980 FIGURE: 7

Compiled by: DN　　Drawn by: CON　　Checked by: FHC

LEGEND :-

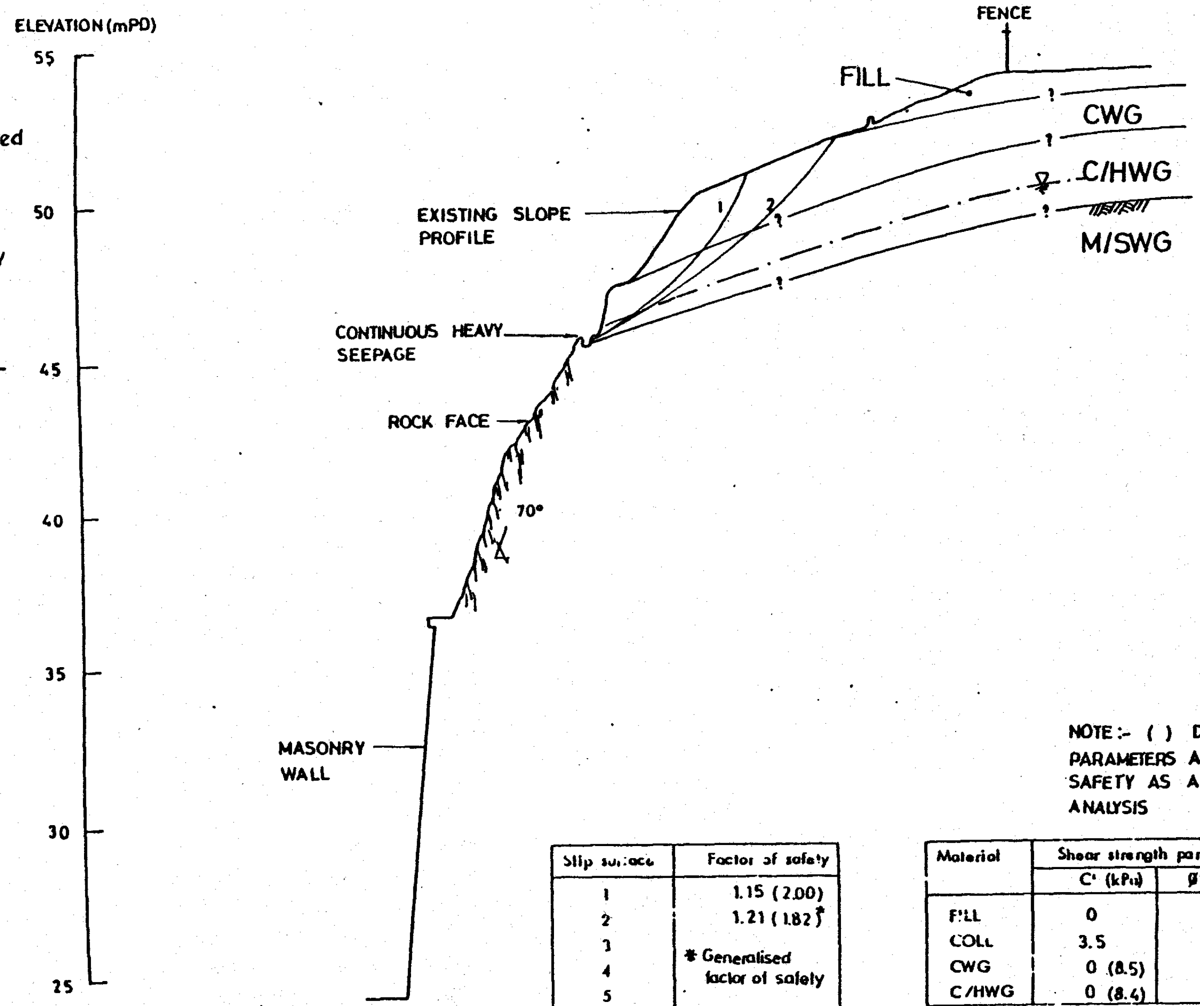
CWG completely weathered granite

C/HWG completely to highly weathered granite

M/SWG moderately to slightly weathered granite

—?— assumed material boundary

∇ assumed ground-water table



NOTE :- ( ) DENOTES REVISED PARAMETERS AND FACTOR OF SAFETY AS A RESULT OF BACK ANALYSIS

Slip surface	Factor of safety
1	1.15 (2.00)
2	1.21 (1.82)*
3	
4	
5	

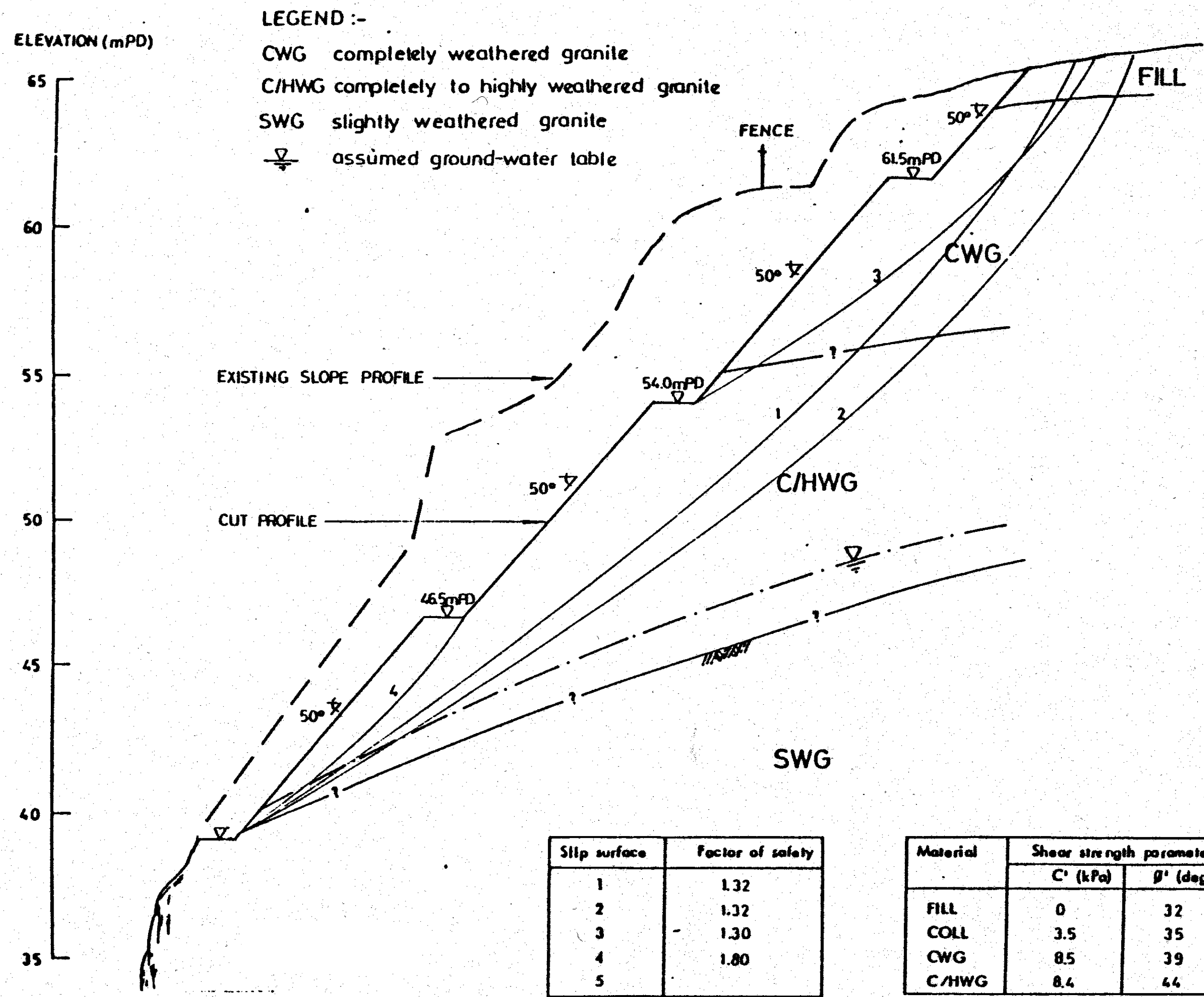
* Generalised factor of safety

Material	Shear strength parameters		Bulk density (kN/m^3)
	C' (kPa)	∅' (deg)	
FILL	0	32	19
COLL	3.5	35	19
CWG	0 (8.5)	37 (39)	19
C/HWG	0 (8.4)	44	19

STABILITY ANALYSIS ON EXISTING SECTION AT CH 110, SLOPE 11 SW-D/N 13

Date: June 1985　　FIGURE: 8

Compiled by: [illegible] Drawn by: Checked by: FR[illegible]



Slip surface	Factor of safety
1	1.32
2	1.32
3	1.30
4	1.80
5	

Material	Shear strength parameters		Bulk density (kN/m^3)
	C' (kPa)	Ø' (deg)	
FILL	0	32	19
COLL	3.5	35	19
CWG	8.5	39	19
C/HWG	8.4	44	19

STABILITY ANALYSIS ON CUT SECTION
AT CH 80, SLOPE 11SW-D/N13

DATE: June 1985 FIGURE: 12

Compiled by: CAN   Drawn by: CAN   Checked by: FHC

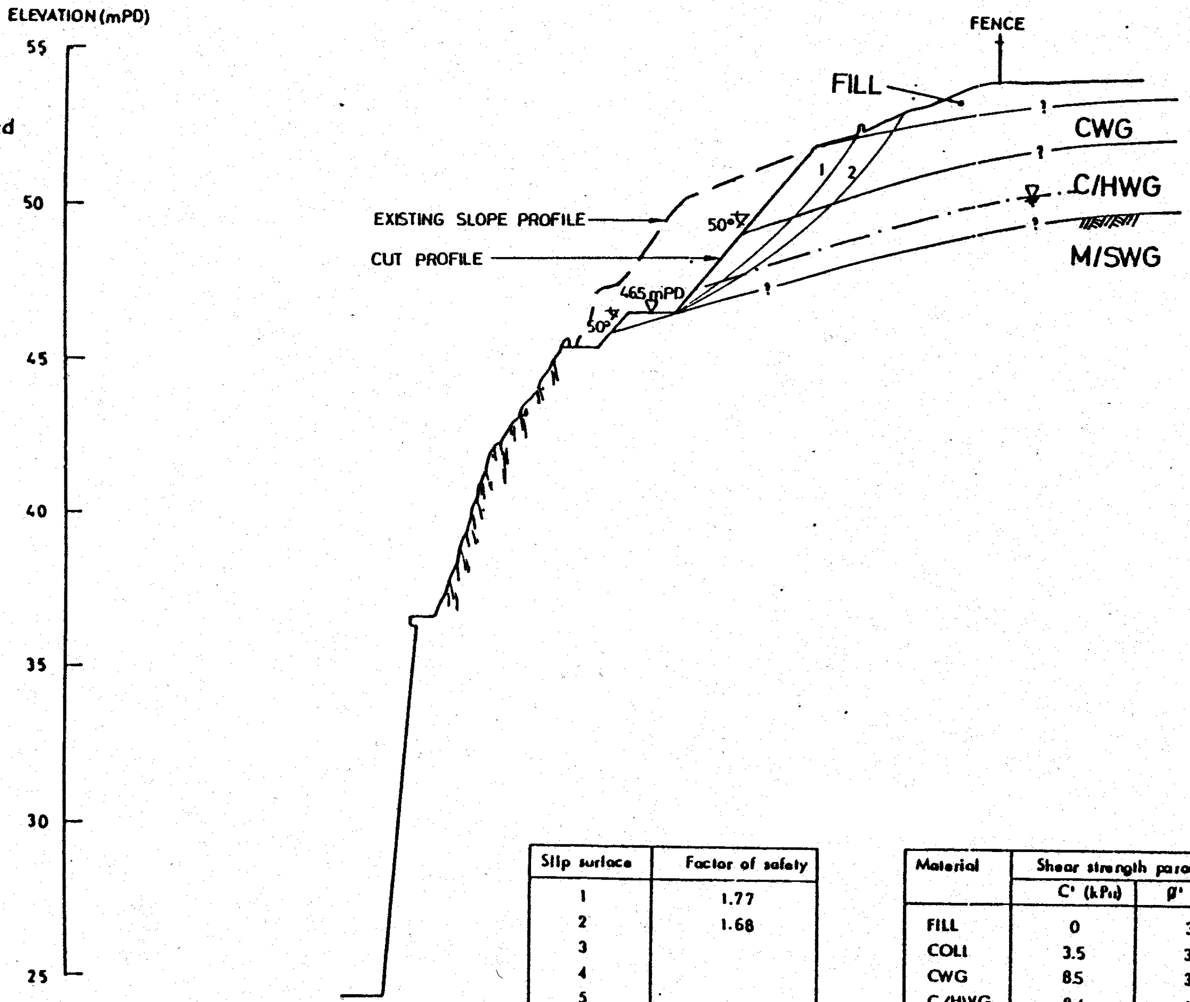
LEGEND :-

CWG completely weathered granite

C/HWG completely to highly weathered granite

M/SWG moderately to slightly weathered granite

▽ assumed ground-water table



Slip surface	Factor of safety
1	1.77
2	1.68
3	
4	
5	

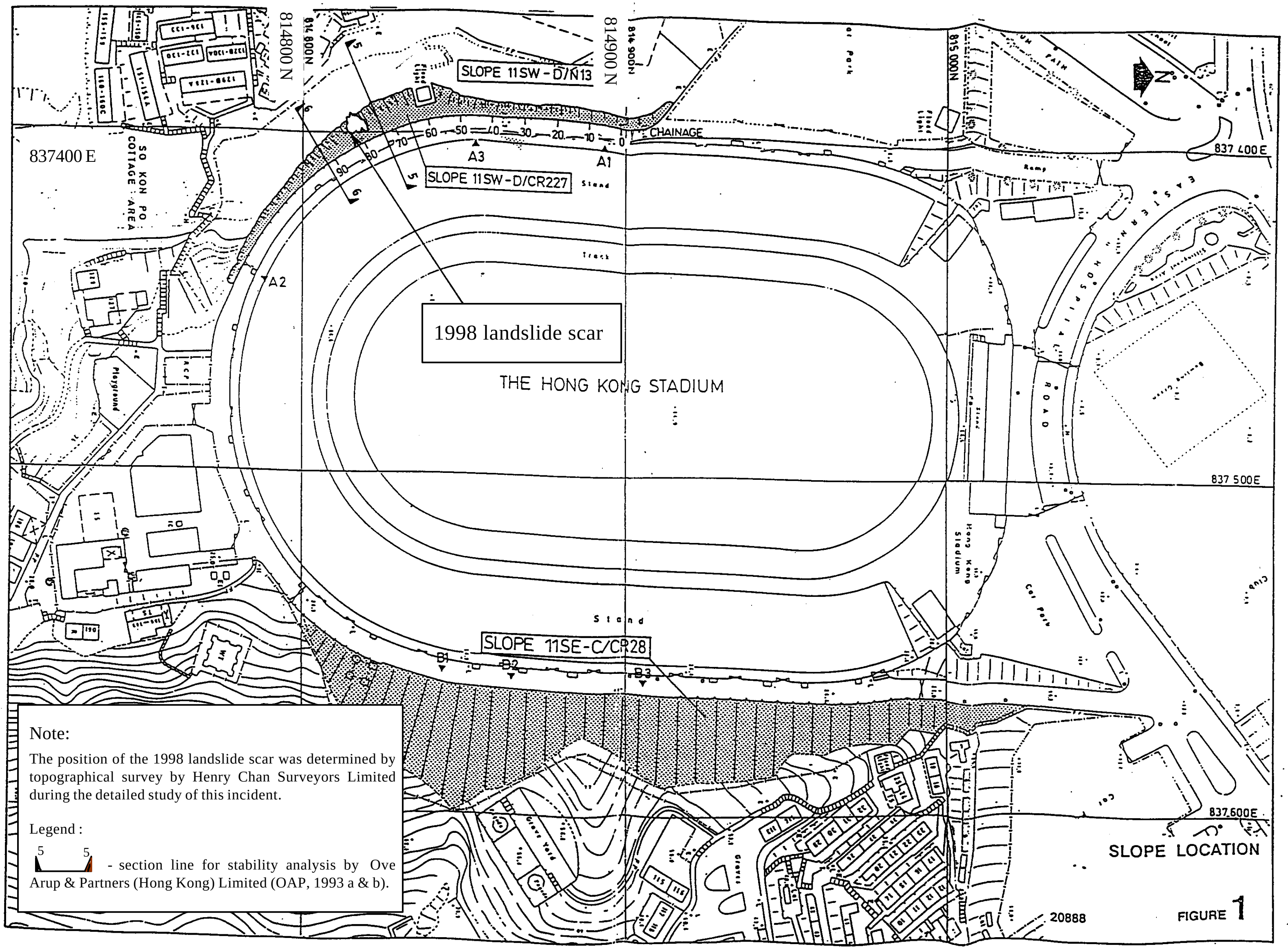
Material	Shear strength parameters		Bulk density (kN/m³)
	C' (kPa)	Ø' (deg)	
FILL	0	32	19
COLL	3.5	35	19
CWG	8.5	39	19
C/HWG	8.4	44	19

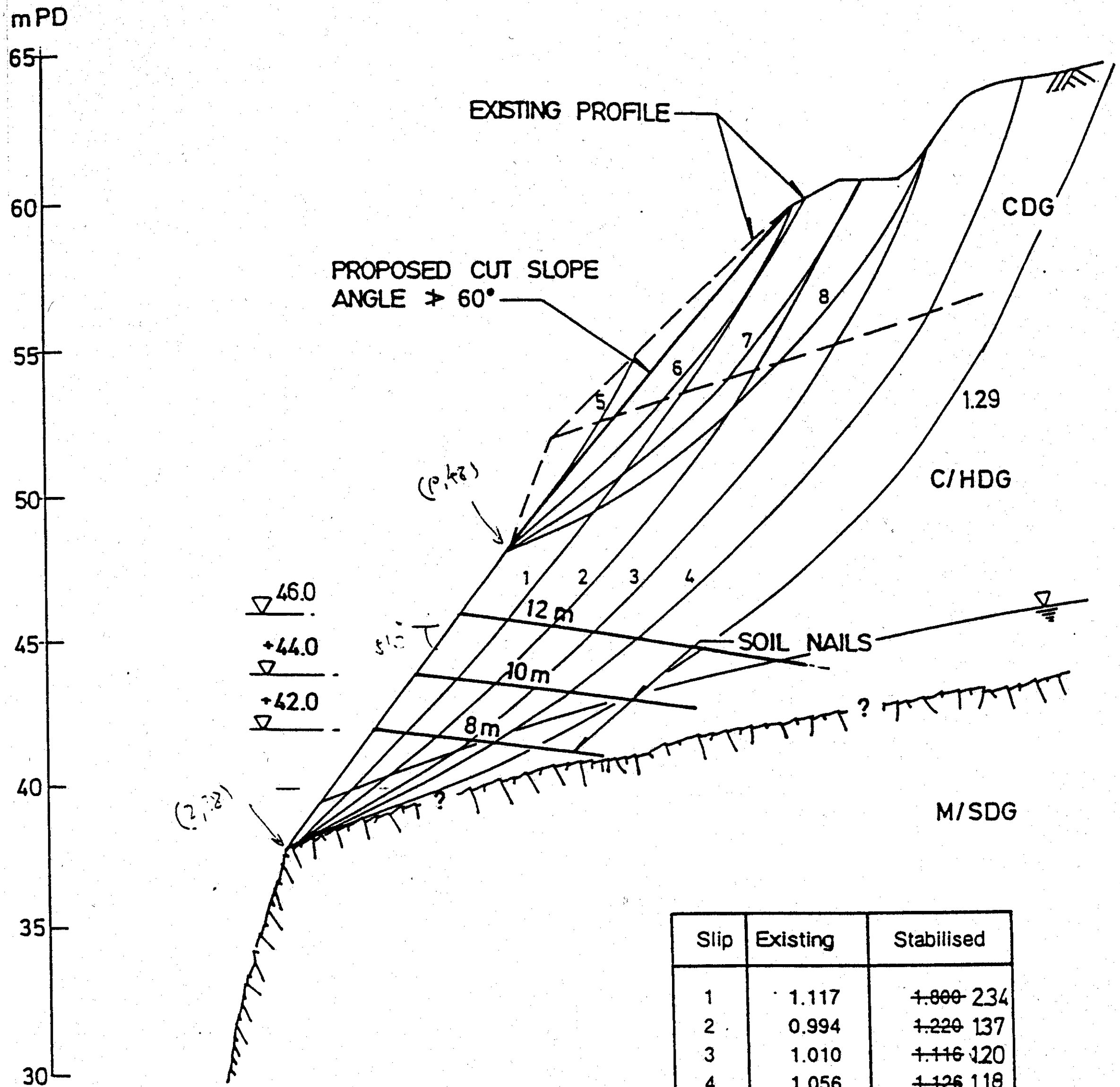
STABILITY ANALYSIS ON CUT SECTION AT CH 110, SLOPE 11SW-D/N13

Date: June 1985   FIGURE: 13

APPENDIX B

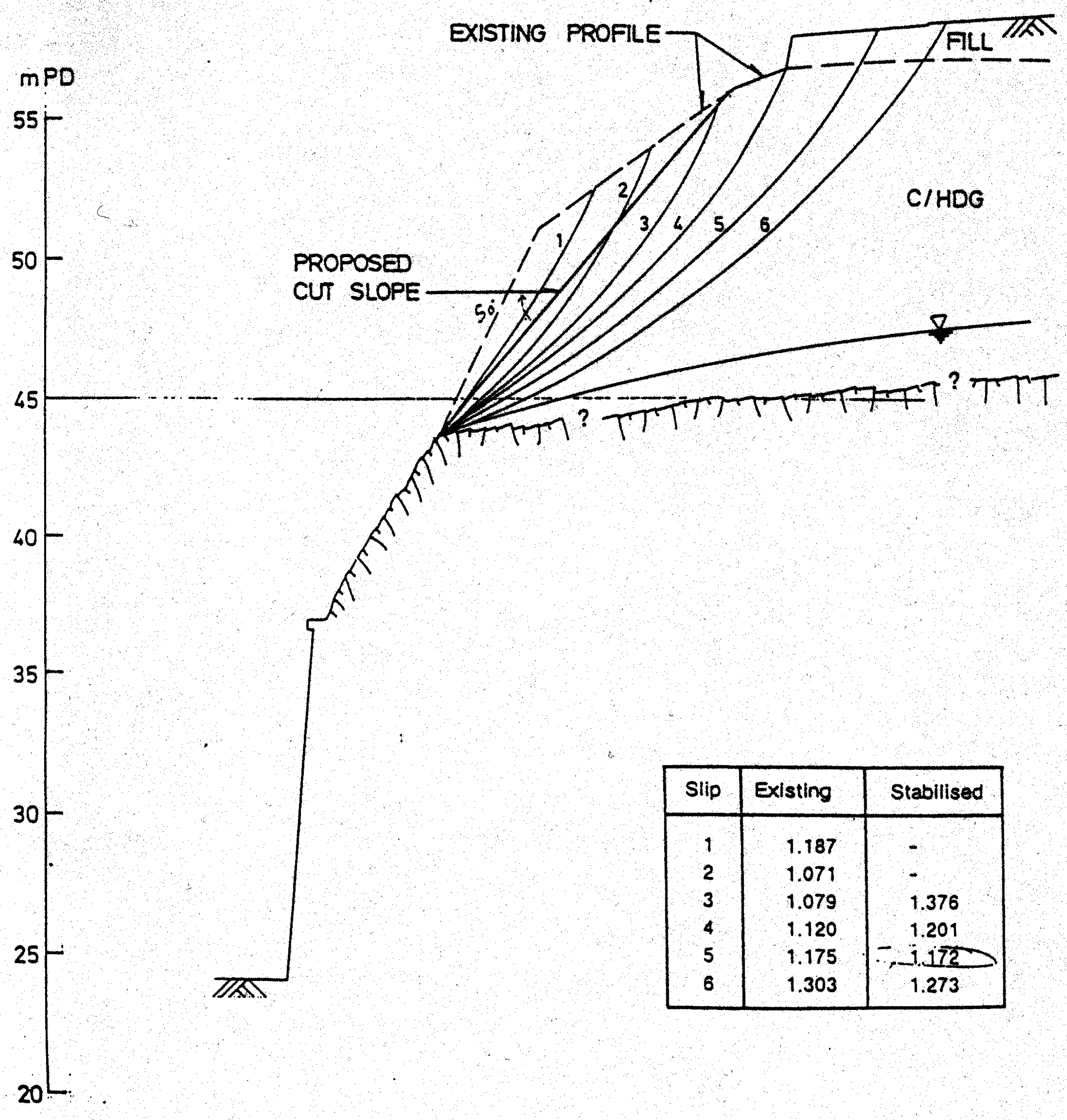
OVE ARUP'S STABILITY ANALYSIS


814800 N
814900 N
815 000N
837400 E
837 400 E
837 500 E
837 600 E
SLOPE 11SW-D/N13
SLOPE 11SW-D/CR227
SLOPE 11SE-C/CR28
CHAINAGE
A1
A2
A3
B1
B2
B3
1998 landslide scar
THE HONG KONG STADIUM
SO KON PO COTTAGE AREA
Stand
Track
Playground
EASTERN HOSPITAL ROAD
Hong Kong Stadium
Car Park
SLOPE LOCATION
20888
FIGURE 1
Note:
The position of the 1998 landslide scar was determined by topographical survey by Henry Chan Surveyors Limited during the detailed study of this incident.
Legend :
- section line for stability analysis by Ove Arup & Partners (Hong Kong) Limited (OAP, 1993 a & b).



Slip	Existing	Stabilised
1	1.117	~~1.800~~ 2.34
2	0.994	~~1.220~~ 1.37
3	1.010	~~1.116~~ 1.20
4	1.056	~~1.126~~ 1.18
5	1.449	-
6	1.252	-
7	1.186	1.361
8	1.239	1.261

Section 5-5



Slip	Existing	Stabilised
1	1.187	-
2	1.071	-
3	1.079	1.376
4	1.120	1.201
5	1.175	1.172
6	1.303	1.273

Section 6-6

APPENDIX C

GEO'S INCIDENT REPORT

URGENT BY HAND

Amended July 97

GEO INCIDENT REPORT

To: ☑ CGE/ ~~PM/ME/MW~~* (See also section 16)
☑ PM/LIC (~~Fax : 2827 8352~~) Fax : 28945780
☐ MW Div'n (Attn : SGE/SQ, for landslips affecting squatters only)

(to be filled in by District Division)	
File No.	GA 2/E2/1998 -
Incident No.	HK 98/5/2
Feature No.	11SW-D/CR227

COPY

FILL IN OR TICK OR DELETE (FOR ITEM MARKED *) AS APPROPRIATE

1. Incident Reported: Date 24 / 5 / 1998 Time 15 : 50 hrs
☑ thro' Pager No. 4554*/~~6700~~* ☐ from ETC Mr .... (ETC Incident No. ....)
☐ from Mr/~~Ms/Mrs~~ .... Tel. No. ....
of ☐ Arch SD ☐ DLO/........ ☐ BD ☐ DO/........ ☐ HD
☐ HyD/........ ☑ Police ☐ FSD ☐ WSD ☐ ....

2. Location of Failure HK Stadium access road to Chiu Man Tsuen on west side of Stadium
Co-ordinates of centre of failure........
(Attach a 1:1000 survey plan to show location of failure, and Slope Reference Number if possible)

3. Date of Inspection 24 / 5 / 1998 Inspection by CAKR
with Mr/~~Mrs/Ms~~ William Lau of HyD Tel. No. 27237226 Time arrived on site 16:20 hrs
Mr/~~Mrs/Ms~~ #A4438 of HKCD Tel. No. 28602312 Time left site 17:10 hrs
Mr/Mrs/Ms .... of .... Tel. No. ....
Mr/Mrs/Ms .... of .... Tel. No. ....
Mr/Mrs/Ms .... of .... Tel. No. ....

4. Time and Date of Failure .... : .... 24 / 5 / 98 Weather Condition at time of inspection Raining
(It is important to give exact time if possible; ask residents or others)

5. Type of Failure ☐ Significant sign of distress with no failure mass .... ☐ Washout
☑ Landslide ☐ Boulder fall from natural slope ☐ Retaining wall failure ☑ Rock fall
☐ Non landslip case (tree fall/ building collapse*/ ....) ☐ ....
(Circle ☐ to indicate principal type of failure if more than one type is involved)

6. Feature Type ☐ Natural slope ☐ Rock slope ☐ RC Retaining wall ☐ Masonry wall
☐ Fill slope ☐ Soil cut slope ☑ Soil/rock cut slope ☐ Others .... above masonry R/W
(Circle ☐ to indicate principal feature type if more than one type is involved)

FSW JV	
FILE	W15801/
REPLIED BY	
REC'D	27 MAY 1998

	ACTION	INFO	COPY
PAC			
ECM			

7. Material and Mass Description of the Feature (The Geoguide 3 classification system should be adopted)
☐ Fill .... ☐ Colluvium .... ☐ Residual Soil ....
☑ Partially Weathered Rock (✓PW 0/30, __PW 30/50, __PW 50/90, __PW 90/100, __unclear)
☐ Others ....
(Circle ☐ to indicate dominant material if more than one material is involved)

8. Scale of Failure 3 - 5 m³ (Volume of slip scar*/debris*)
For boulder fall from natural slope (Approx. trajectory to be shown in Item 15)
Number of boulders involved 2 - 3 Dimension and shape of boulders Angular up to 0.2m

9. Feature Condition
Evidence of poor state of maintenance: ☑ Damaged chunam/shotcrete/stone-pitching*, location above rock slope
☐ Bare slope surface (poorly maintained vegetation), location ....
☐ Blocked/broken drains, location .... ☐ Others .... location ....
Capacity of surface drainage system ☐ adequate ☐ inadequate ☐ not present ☑ not known
Coverage of hard protection surface against infiltration ☐ adequate ☑ inadequate ☐ not present
Surface Protection Material Chunam
Field evidence of past instability at or adjoining the failure location ☐ yes ☐ no Unknown
Groundwater seepage observed at the failure location ☑ yes (location to be shown in item 15) ☐ no

10. Possible Contributing Causes of Failure (tick one or more) ☑ Infiltration ☐ wash-out
☐ seepage behind an impermeable surface ☐ groundwater (main/perched/unclassified)* ☐ rupture of watermains
☐ Insufficient maintenance contributed to failure .... ☐ Others ....
(Circle ☐ to indicate primary cause if more than one cause is involved)

11. Consequence of Failure ☐........ person(s) killed ☐........ person(s) injured ☐ Squatter huts affected ☐ Carparks affected
☐........ lane(s) of road blocked ☐ Building lot affected ☐ Construction site affected ☐ Pedestrian pavement affected
☐ Country park affected ☐ Open space affected ☐ Building access affected ☑ 1 lane of access road alongside HK Stadium blocked

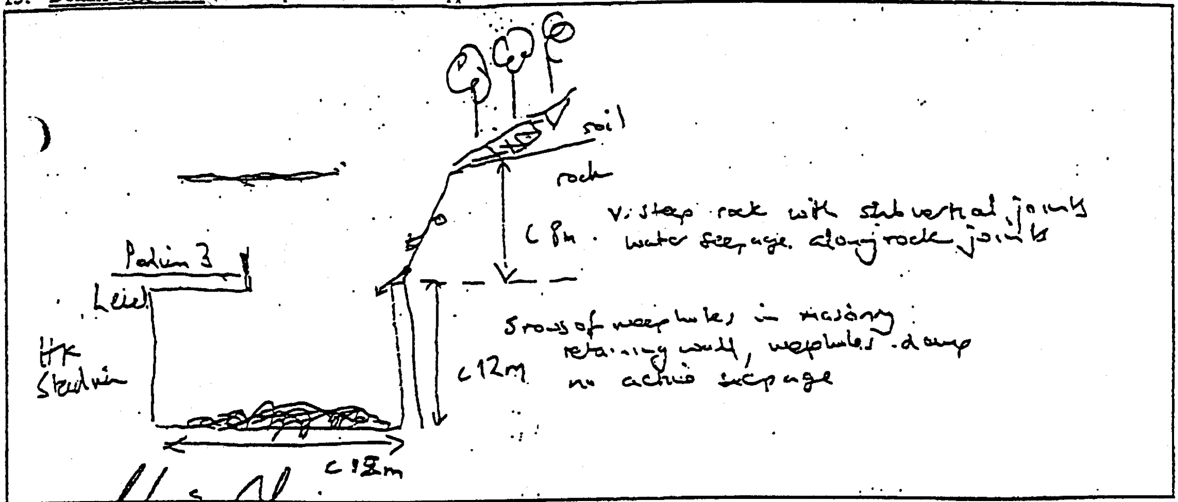
12. Immediate Advice Given To: Mr/~~Mrs/Ms~~ ......Willie Lam........ of ......HyD......

- ☐ Squatter huts permanently evacuated Hut No(s) ....
- ☐ Squatter huts temporarily evacuated Hut No(s) ....
- ☐ Flats evacuated/Closure Order Recommended. Building/Flat No. ....
- ☑ Close ......access...... lane(s) of road to podium level 3
- ☑ Close pedestrian pavement on podium level 3 of HK Stadium
- ☑ Divert surface runoff from reaching failure area
- ☑ Cover failure scar with tarpaulin properly secured against wind
- ☐ Fence off area in danger
- ☑ Trim back failure surface
- ☐ Provide surface protection (with weepholes) to trimmed failure surface
- ☐ Provide/reconstruct drainage system
- ☐ Isolate water-carrying services
- ☑ Check land status
- ☐ Buttressing
- ☐ Warn nearby occupants of possible danger during heavy rainstorm
- ☐ ....

13. Is Further Inspection by District GE Required? ☑ Yes ☐ No
if Yes, Reason ......to determine remedial works and slope ownership......

14. Is Landslip Record Card Required: ☐ Yes (for debris vol. > 50 m³ or involving death)

15. Details of Failure (location plan & section with approx. dimensions of slope height, gradients, depth of failure, debris run out etc.)



[signature]	CLIVE FRANKS GE/E4	24/5/98
Inspection Officer's Signature	NAME IN BLOCK LETTER, POST	DATE

District Action Required *(to be filled in by District Division)*

16.1 Feature to be registered (GEO Circular 8/96)? ☐ Yes ☑ No (Decision to be made by District GE) slope registered.

16.2 Feature previously studied, upgraded or checked by GEO ☑ Yes ☐ No (TO to carry out file search)

If Yes, state action taken in file GCS 3/1/789 Pt. III F(31)

Additional Remarks Stage 1 Study Completed. Stabilization works to the slope completed in 1993

[signature] YLSChan	YW CHAN	26.5-98
District GE's Signature	NAME IN BLOCK LETTER	Date
[signature] Willie Wong	WL WONG	26.5.98
District SGE's Signature	NAME IN BLOCK LETTER	Date

C.C.
- ☑ CGE/SP (with photo) thro' GGE I/M ,PGGE
- ☑ CGE/LI (with photo)
- ☐ CGE/SS (for boulder fall from natural slope only)
- ☐ CBS/SS (for private site only)

1/. One incident report should be filled in for each incident. Report to be continued on supplementary sheets if necessary.

2/. Section 1 to 15 to be filled in by Inspecting GE. On returning to the office, the copy should be despatched to CGE/District and PM/LIC asap. The remaining sections (Top box & Section 16) to be completed by District GE and despatched accordingly.

3/. Useful observation which supplement information given in the Incident Report to be made on separate sheets.

GCDIST 31 (7/97)