SECTION 4: DETAILED STUDY OF THE LANDSLIDE AT VILLE DE CASCADE, FO TAN, SHATIN ON 3 JULY 1997

Halcrow Asia Partnership Ltd

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FOREWORD

This report presents the findings of a detailed study of a landslide located on a man-made slope south of Ville de Cascade, Fo Tan, Shatin. On 3 July 1997 a minor failure (GEO Incident No. ME97/7/42) occurred near the crest of the man-made slope. Debris from the failure remained on the slope and no fatalities or injuries were reported. Following clearance of vegetation from the slope an extensive area of landsliding was identified, that extended the full height of the slope, about 27 m, and was about 50 m wide.

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 instability. The scope of the study included site reconnaissance, desk study, ground investigation and analysis. Recommendations for follow-up actions are reported separately.

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

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1. INTRODUCTION

On the morning of 3 July 1997, a minor failure (GEO Incident No. ME97/7/42) occurred near the crest of a registered cut slope (No. 7SW-B/C123) to the south of Ville de Cascade, Fo Tan, Shatin (Figure 1 and Plate 1). Debris from the failure remained on the slope and no fatalities or injuries were reported. Following clearance of vegetation from the slope an extensive area of surface cracking and deformation was identified adjacent to the minor failure. The distressed area (referred to as the landslide in this report) extended the full height of the slope, about 27 m, and was about 50 m wide (Figure 1).

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

The key objectives of the study were to document the facts about the landslide, which includes the minor detachment and significant distress associated with a large-scale instability, present relevant background information and establish the probable causes of the instability. The scope of the study included site reconnaissance, desk study, ground investigation 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) review of relevant documents relating to the history of the site and the sequence of events leading to the landslide,

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

(c) interviews with witnesses,

(d) analysis of rainfall records,

(e) detailed ground investigation to determine the subsurface conditions at the site by drilling, probing, excavation of trial pits, in-situ testing, groundwater monitoring and laboratory testing,

(f) establishment of representative hydrogeological and geological models for the slope,

(g) theoretical slope stability analysis, and

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

The ground investigation works associated with this detailed study were undertaken in conjunction with the GEO's Landslip Preventive Measures (LPM) Programme for the design and implementation of remedial works to the slope. At the time of preparation of this report, remedial works to the slope were in progress.

2. THE SITE

2.1 Site Description

A plan of the landslide is shown in Figure 2. The landslide occurred on the western portion of Slope No. 7SW-B/C123 beneath a lay-by on Sui Wo Road. Ville de Cascade is located at the toe of the slope below the landslide. The minor failure on 3 July 1997 occurred above the eastern part of the landslide and to the west of a pedestrian stairway that crossed the slope (Figure 2).

At the site of the landslide, the slope was formed in soil with a covering of shrubs and grass with occasional trees. To the west, tree growth was more pronounced and beyond the stairway to the east, the slope was predominantly covered by shotcrete and chunam. The slope profile at the landslide comprised three batters separated by 1 m- to 3 m-wide berms. The upper and lower batters were inclined at about 32° to the horizontal, and the middle batter at about 36°. Slope drainage comprises U-channels along the toe, crest and berms connected by stepped channels at about 50 m intervals.

The slope is predominantly formed in soil but rock is locally exposed, particularly to the east of the stairway. Parts of the upper batter and the western end of the slope are formed of fill, which is up to 3 m thick (Figure 3).

According to the "Systematic Identification of Maintenance Responsibility of Slopes in the Territory" (SIMAR) consultancy, the Highways Department (HyD) is responsible for the maintenance of the section of Slope No. 7SW-B/C123 affected by the 1997 landslide.

2.2 Site History

The site history was established from an interpretation of a sequential series of aerial photographs (Table 1), and a review of available documentary records. The history of development of the site is shown in Figure 3.

In 1949 the site was natural terrain with a northwest-aligned river valley (A) running parallel and adjacent to the present position of the toe of the slope. The slope site was a northeast-facing natural hillside crossed by valleys with tributary streams in the central portion (B) and western end (C). On the 1949 aerial photographs, a large relict landslide is visible on the hillside at a location that in plan overlapped the eastern flank of the 1997 landslide. Two smaller landslide scars are visible close to the 1997 landslide on the 1963 aerial photographs (Figure 3). The relict landslide was about 50 m wide with possible debris lobes that extended into the river. The inclination of the natural hillside at the site of the relict landslide was about 25°.

In 1976, site formation works were in progress on platforms above and below the slope which was under construction. By 1977 the slope and the surrounding platforms were substantially completed. The configuration of the slope is similar to that of the present day, except that the westernmost part was originally at a lower level (Figure 3). Rock exposures are visible on the 1977 aerial photographs, particularly in the central portion of the slope.

In 1982, some modification of the slope profile had taken place, with part of the upper eastern batter reprofiled, removing a small spur. The western end of the slope had been built-up by placement of fill into the tributary valley (C), see Figure 3. The stairway located in the central section of the slope was under construction.

In 1984, building works had started on the lower platform with the construction of Shatin College, and by 1986, Greenwood Terrace and Ville de Jardin were under construction on the upper platform. By 1988, Ville de Cascade located beneath the western slope was substantially completed. Aerial photographs indicate that parts of the slope toe had been cut-back to a height of about 2 m during the construction of Ville de Cascade (Figure 3). It would appear from aerial photographs that the excavation was unsupported at the time.

2.3 Previous Studies

A review of previous studies and reports, summarised in Appendix A, shows that the slope and surrounding area originally formed part of a borrow area for reclamation works at Shatin in the 1970s. The slope and adjacent platforms were subsequently incorporated into Area 41A of the Shatin New Town development proposals, as shown on the proposed layout plan issued by the New Territories Development Department (NTDD) in 1978.

In 1979, in response to NTDD's proposed layout plan, the Geotechnical Control Office (GCO, renamed GEO in 1991) requested its consultant, Binnie and Partners (B & P), who were responsible for carrying out geotechnical checking of new works, to undertake an appraisal of the development proposals. B & P (1980) reported that a "full stability check (is) required" for Slope No. 7SW-B/C123.

In 1979, Maunsell Consultants Asia Ltd. (MCA) produced a slope stability report of the proposed development for NTDD. The assessment included aerial photographic interpretation, geotechnical site survey, site investigation comprising boreholes and trial pits (see Section 5.1) and laboratory tests. Groundwater conditions were established from monitoring of piezometers installed in boreholes and from field observation of seepages. Shear strength parameters for completely decomposed granite of $c' = 13$ kPa and $ø' = 39°$ were determined from consolidated single- and multi-stage triaxial testing of undisturbed borehole samples.

For slope 7SW-B/C123, MCA recommended minor reprofiling of the eastern slope, and re-turfing and installation of horizontal drains on the central and western slope. Stability analysis undertaken for the western portion of the slope, the nearest to the 1997 landslide, assumed that the slope comprised completely decomposed granite with moderately to slightly decomposed granite at between 12 m and 7 m below the slope surface (section X – X in Figure 3 and Figure B1 in Appendix B). The results of the analysis gave a minimum factor of safety of 1.53, for a "2 to 3 m rise in the measured highest wet season piezometric levels". This corresponded to a piezometric surface between 3 m and 6 m below the slope surface and sub-parallel to the base of the saprolite.

In 1980, B & P commented that they "endorsed MCA's proposals but have also recommended further investigation and design". In particular, B & P recommended that the extent and stability of the fill identified in the central and western slope in the areas, where valleys B and C crossed the slope respectively (Figure 3), required further investigation, and that for the eastern slope "recommended remedial works should be expanded to ensure stability".

Supplementary reports on slope stability were produced by MCA in June 1980 and May 1981 which included results of GCO probing and further stability analyses. The 1981 report proposed remedial works for the western and central parts of Slope No. 7SW-B/C123 that included "removal of all loose fill and turf down to in-situ decomposed granite, laying a 700 mm filter blanket... (and) reprofiling the slope to a gentler gradient with compacted fill" (Figure 3).

B & P considered the proposed remedial works in the western and central slope satisfactory provided the extent of the loose fill was "confirmed during construction". B & P also agreed that the proposed works on the eastern slope, submitted by MCA in 1980 and comprising trimming of the soil slope and stabilisation of potentially unstable rock, resulted in "acceptable stability".

The as-built drawings of the slope were issued by MCA in December 1983. Following a request by GCO for clarification of the completed slope remedial works, MCA provided further information. On receipt of this information, GCO stated that "design of all slopes has been accepted by this office". The completed western part of the slope is shown in Plate 2.

The slope was not registered in the 1977/78 Catalogue of Slopes. In 1992, the GEO initiated the consultancy agreement entitled "Systematic Inspection of Features in the Territory" (SIFT) which, inter alia, aims to identify features not registered in the 1977/78 Catalogue of Slopes and update information on existing registered slopes based on studies of aerial photographs. In June 1996, the SIFT study identified the cut slope as SIFT class "C1", indicating the slope had been formed or substantially modified before 1978. The cut slope was allocated SIFT No. 7SW-10D/S50.

In 1994, the GEO commenced the consultancy agreement entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST) to systematically update the 1977/78 Catalogue of Slopes and to prepare the New Catalogue of Slopes. The SIRST project registered the cut slope as No. 7SW-B/C123. During an inspection in December 1996 by the SIRST consultants, signs of seepage on the slope were noted (location not given) but no signs of distress were identified and no emergency actions on the slope were considered necessary.

In June 1997, as part of the GEO's LPM Programme, HAP was requested to undertake a Stage 3 Study of the slope.

2.4 Previous Failures

Three landslides occurred in the natural terrain prior to slope formation, as shown on aerial photographs taken in 1949 and 1963 (Figure 3 and Section 2.2). GEO's Natural Terrain Landslide Inventory records no landslides within the vicinity of the site.

Following its formation in 1977, several minor failures occurred on the slope, with a number of failures being in the vicinity of the 1997 landslide (Figure 3). An inspection of the slope by MCA in 1979 identified a possible failure scar on the eastern part of the slope, which was considered to have occurred during slope formation.

In 1982, several shallow failures occurred on the western slope. The largest of these occurred in the western part of the middle batter and was about 5 m wide by 10 m long and was described by the GCO as "an area of surface instability" (see Appendix A). The GCO subsequently reported the failure to MCA. In 1984, during a site inspection, the GCO observed a shallow failure involving mainly surface erosion (Plate 2 and Figure 3) on the upper western batter (see Appendix A), and MCA was subsequently notified. By 1986 this shallow failure had enlarged to cover parts of the upper two batters with an extent of about 20 m by 5 m.

3. THE LANDSLIDE

3.1 Sequence of Events

The establishment of the sequence of events leading to the discovery of the full extent of the 1997 landslide is based on site observations by HAP and GEO, and from a review of records and interviews with persons involved. There were no eye-witnesses to the landslide incident.

The first indication of distress in the slope was a minor failure noticed at about 10:00 hours on 3 July 1997 by an employee of the Ville de Cascade management. According to accounts given by the employee to HAP, seepage and displacement at the toe of the slope were observed. A Landslip Warning had been issued by the GEO at 06:25 hours on 2 July 1997, which was cancelled at 08:40 hours on 5 July 1997.

The failure was reported to the GEO at 12:00 hours on 3 July 1997, who subsequently inspected the site on 7 July 1997 (Plate 1). At the time of the inspection the slope was densely vegetated and only limited access was possible. The GEO advised HyD to fence off the affected area, cover the failure scar with sheeting, remove loose soil debris, trim back the slope surface, provide a hard surface protection, and clear and repair surface channels, as necessary.

On 27 June 1997, as part of the LPM Programme, GEO requested its consultant, HAP, to include the slope in HAP's current LPM contract (GE/96/06). The slope was subsequently inspected by HAP on 16 and 31 July 1997. During the inspections it was observed that there was bulging on the lower slope and that "superficial movement of soil had caused cracking of the U-channel" at the toe of the bottom batter, downslope of the 3 July incident (HAP, 1997). Dense vegetation prevented further detailed inspection at that time.

On 7 August 1997, the minor failure in the upper part of the slope was inspected by HAP as part of the LIC. Prior to the inspection, the upper two batters of the slope, including the minor failure scar, had been partly shotcreted as part of the urgent repair works carried out by HyD (Figure 2). HAP also observed seepage from the lower batter.

By 28 October 1997, the lower batter was cleared of vegetation and a ground investigation for the LPM works commenced. On 7 November 1997, inspection of the vegetated part of the upper batter to the west of the minor failure (Figure 2) revealed a tension crack and landslide scarp. Following a joint site inspection by GEO and HAP on 12 November 1997 it was noted that the landslide probably extended throughout the full height of the slope and it was therefore agreed that stabilisation measures should be fast-tracked to facilitate early completion.

Prior to completion of the stabilisation measures, emergency works including crack monitoring, sealing of the tension crack and repair of damaged surface drains, were implemented. GEO also notified the management of Ville de Cascade that when a Landslip Warning was in operation, they should prevent all access to the road at the toe of the landslide, and requested that occupants of the lower two levels of the apartments below the landslide should vacate their rooms facing the slope.

3.2 Description of the Landslide

The description of the landslide was based on walk-over surveys of the site by HAP and GEO and observations from trial pits during the ground investigation following the 1997 landslide. The observations (Figure 2) show an extensive area of displaced and cracked ground downslope and to the west of the minor failure of 3 July 1997, encompassing a slope area of about 2 500 m^2. If the failure surface extends to the base of the saprolite (see Sections 5.2 and 6), then the landslide volume is estimated to be up to about 10 000 m^3.

The extent and magnitude of the slope movement based on observations of ground deformations and measured vertical displacement are shown in Figure 2. The greatest vertical displacement was concentrated along the western scarp with a subsidence trough of about 500 mm extending to the lower berm. Further subsidence occurred in the lower batter immediately behind the bulged area.

The minor failure of 3 July 1997 was apparently a shallow slide within principally very clayey and silty sand fill. The dimensions of the failure were estimated to be 5 m wide, 5 m long and up to 0.6 m deep, with a volume of about 15 m^3. The failed material came to rest immediately downslope of the rupture surface (Figure 2 and Plate 1). The cause of the failure is not known, though surface water infiltration was indicated in the GEO Incident Report and localised loose fill deposits are known to exist in the vicinity (see below). It is also noted that the location of the toe of the rupture surface of the minor failure lies along an easterly extension of the main landslide scarp (Figure 2). It is possible that the minor failure was associated with the development of the main landslide scarp.

The upper western limit of slope movement was defined by the landslide scarp that ran parallel to the slope crest for about 14 m before trending obliquely down the slope for 25 m to the middle batter where its surface expression faded and disappeared (Figure 2). Displacements across the scarp were about 1 m vertically and up to about 0.5 m horizontally. The eastern extent of the scarp was masked by shotcrete placed following the urgent repair works by HyD.

HAP noted that the scarp appeared predominantly fresh during the inspection on 7 November 1997 (Plate 3). However, it is uncertain whether there had been earlier movement along the scarp. Elsewhere, at the location where the scarp turns downslope, the ground surface is hummocky and lies within a depression (Figure 2). This area coincides with the 1984 and 1986 failures identified from interpretation of aerial photographs (Section 2.4). Subsequent excavation at this location in April 1998 for the LPM works revealed a local area of loose fill.

Inspection of the berm drainage through the landslide area showed horizontal displacement of surface channels, estimated to be up to about 200 mm (Plates 4 and 5). A topographical survey indicated vertical displacement along the channels of up to about 500 mm.

At the toe of the slope, bulging was evident over about a 30 m length that resulted in the crushing of the toe drain and small-scale collapses of soil (Plate 6). Upslope of the bulged area, the lower batter showed signs of disturbance with areas of subsidence and cracking up to 50 mm wide. Trial pits Nos. TP6, TP9, TP10 and TP11 excavated in the lower batter encountered sub-vertical open relict joints up to 120 mm wide. Within trial pit No. TP9, open joints partially infilled by the overlying fill were evident (Plate 7). A 2 m-high concrete wall along the toe of the slope and the paved area in Ville de Cascade showed no evidence of distress.

Trial pits Nos. TP1, TP4 and TP15 were excavated along the line of the main scarp to investigate its nature and persistence (Figure 2). Trial pit No. TP1 encountered about 1 m of fill overlying completely decomposed granite. Within the completely decomposed granite there was a sub-vertical open relict joint about 80 mm wide (Plate 8). Displacement of a quartz vein across the open joint indicated about 200 mm vertical movement. There was no discernible trace of corresponding movement in the overlying fill indicating that movement on the joint possibly predated placement of fill.

The western part of the landslide scarp, exposed in trial pit No. TP4, exploited a pre-existing relict joint in completely decomposed granite that was open to a depth of about 3 m below the ground surface (Plate 9). Displacement across the joint varied but was up to 400 mm horizontally and about 200 mm vertically. Trial pit No. TP15, located at the western end of the scarp, exposed a sub-vertical crack within the fill with a horizontal displacement of up to 20 mm. The crack extended about 0.8 m below the ground surface.

Seepage at the toe of the landslide was initially reported following the minor failure on 3 July 1997. The seepage and general dampness at this location have remained throughout the period of this investigation.

4. RAINFALL

The nearest GEO automatic raingauge No. N02 is located at Shun Wo House, Wo Che Estate, approximately 1 km to the southeast of the landslide. Slope movement was first observed at the landslide site on 3 July 1997. The daily rainfall recorded in June and July 1997, together with the hourly rainfall from 30 June to 3 July 1997 is presented in Figure 4.

A total of 1375 mm of rainfall was recorded by raingauge No. N02 in the 31 days before the minor failure of 3 July 1997. The 12-hour and 24-hour rainfalls preceding the landslide are 143.5 mm and 292.5 mm respectively. The hourly rainfall preceding 3 July 1997 shows a prolonged period of rainfall starting at around 04:00 hours on 2 July 1997, and continuing up to the time the failure was first observed. The peak intensity of clock hourly rainfall occurred at between 05:00 hours to 06:00 hours on 2 July 1997 (Figure 4).

As the exact timing of the landslide is not known, the estimated return periods for maximum rolling rainfall for various durations have been compiled from the preceding rainstorm up to the time when the minor failure was noticed on 3 July 1997 (Table 2). The estimated return periods are based on rainfall data recorded at the Hong Kong Observatory (Lam & Leung, 1994) and as such the return periods can only be considered as indicative for the landslide site.

The estimated return periods for durations of 2 hours or greater ranged from about 20 years to about 150 years, the latter corresponding to the 31-day duration rainfall.

The maximum rolling rainfall for selected durations preceding 3 July 1997 has been compared with previous severe rainstorms recorded at raingauge No. N02 since its installation in February 1980 (Figure 5). The maximum rolling rainfall for the recent rainstorm exceeds those of previous rainstorms at the raingauge for all durations.

5. SUBSURFACE CONDITIONS

5.1 General

Several ground investigations were carried out at the site from 1972 onwards, as shown on Figure 6. The 1972 ground investigation (Gammon (HK) Ltd., 1972) was undertaken for the purpose of establishing the suitability of the site as a borrow area. This included four boreholes (KD series) located within the vicinity of the slope. Subsequent investigations between 1977 and 1979 (ED series) (Koken Boring Machine Co. Ltd., 1979) and in 1980 (G series) (Material Testing Laboratory, 1980) were undertaken on behalf of MCA as part of the geotechnical appraisal of slopes within Area 41A. Further investigations for building developments adjacent to the slope were carried out in 1982 for Shatin College (Anon, 1982), and 1985 for the Ville de Jardin and Ville de Cascade developments (Fugar Engineering and Construction Co. Ltd., 1985).

The ground investigation following the 1997 landslide (Figure 6) comprised fourteen boreholes, fifteen trial pits, ten surface strips and forty-five GCO probes (Goldram Engineering and Development Ltd., 1998). The boreholes were advanced in soil using continuous Mazier sampling or by alternate Standard Penetration Tests and Mazier sampling with air-foam flush and, in rock, by coring using a T2-101 core barrel.

Standpipe piezometers were installed in thirteen boreholes within saprolite, at the saprolite/moderately to slightly decomposed granite interface and within moderately to slightly decomposed granite at depths ranging from 2.5 m to 24.5 m below ground level.

Impression packer tests to determine joint orientations were carried out in eight boreholes within moderately to slightly decomposed granite at depths ranging from 0.6 m to 20.1 m below ground level. This was supplemented by a downhole acoustic televiewer to determine the in-situ characteristics and orientation of joints in a further three boreholes.

Following completion of the 1997 landslide ground investigation, laboratory shear strength and classification tests were carried out on selected soil samples. In addition, Mazier samples from a number of boreholes were extruded, split and photographed prior to detailed logging.

5.2 Geology

The Hong Kong Geological Survey 1:20 000-scale sheet 7 (GCO, 1986) shows that the slope is underlain by medium- and coarse-grained granite. The contact zone between the two rock types trends at about 050° and crosses the central part of the slope. The landslide site is shown to be underlain by medium-grained granite.

A representative geological cross-section through the landslide based on the 1997 landslide ground investigation is shown in Figure 7.

Most of the boreholes and trial pits in the landslide site encountered a layer of fill, to a maximum depth of 2.5 m, though typically the depth was less than 1 m. Fill exposed in trial pits was generally loose to very dense, brownish yellow clayey/silty fine- to coarse-grained sand, with many fine- to coarse-grained gravel and occasional boulder- and cobble-sized fragments of granite.

The dominant material at the landslide site is a saprolite of extremely weak, orange brown mottled black medium-grained completely decomposed granite, with medium to closely spaced joints. Within the landslide site the thickness of completely decomposed granite was found to range from 2 m to 12 m, with the greatest thickness recorded in the western mid-slope, reducing to about a few metres at the toe (Figure 7).

The depth to moderately to slightly decomposed granite (taken as bedrock in this study) increases generally across the landslide site from east to west (Figure 7). The depth to bedrock beneath the middle batter varied from 4 m on the eastern margin of the site to 12 m on the western margin and within the site it was 7 m to 9 m.

The granite has been locally altered over a depth of 3.3 m to 6.1 m below ground level in borehole No. BH2 (Figure 2). The alteration comprises partial mineral dissolution in moderately decomposed granite resulting in a porous structure with an inter-connecting network of voids of 2 mm to 60 mm diameter (Plate 10). This is possibly the result of localised hydrothermal alteration from migration of hot fluids along lines of weakness during a later phase of granitic intrusion.

The presence of several localised kaolinite-rich zones in boreholes Nos. BH6, BH8 and BH11 also suggests possible hydrothermal alteration. Between 5 m and 7 m below ground level in borehole No. BH8, these zones were more pronounced and up to 350 mm thick. The kaolinite-rich zones identified from Mazier samples recovered from boreholes could not be traced laterally into adjacent boreholes.

5.3 Structural Geology

The dominant discontinuity set within the slope, as measured in boreholes and established from surface strips and trial pits in the recent ground investigation, is inclined at about 10° to 35° to the north-northeast. These discontinuities are sub-parallel to the original natural hillside. A secondary discontinuity set dips steeply into the slope, at about 60° to 80° in a southwesterly direction (Figure 8a).

The structural fabric of the completely to highly decomposed granite was determined from trial pits and splitting and logging of a total length of about 30 m of Mazier samples. Joints within the granite were found to be typically closely spaced and occasionally very closely spaced. Most of the joints showed iron and manganese staining with some damp soft to firm black clayey silt infill generally about 5 mm thick. Several joints had a stiff to firm grey/white clayey occasionally sandy silt infill (probably kaolinitic), with a maximum thickness of 20 mm but generally less than 5 mm.

Slickensided joints, associated with displaced ground, were recorded in trial pits Nos. TP4, TP9 and TP15. Only one slickensided joint was identified from a Mazier sample recovered from borehole No. BH10 between 5.45 m and 6.45 m below ground level. No readily identifiable sheared surface or zone of movement could be identified from the other Mazier samples. Several slickensided infilled joints were identified from rock core samples retrieved from a few metres below the top of moderately to slightly decomposed granite in borehole Nos. BH10 and BH4, which were drilled outside the area of the distressed area. The origin of these latter slickensided joints is uncertain.

An analysis of kaolin-infilled and slickensided discontinuities (Figure 8b) shows a similar distribution to the overall discontinuity pattern (Figure 8a). Most of the kaolin-infilled and slickensided discontinuities are associated with the dominant discontinuity sets. Individual discontinuities however could not be traced between adjacent boreholes.

5.4 Soil Properties

Laboratory tests were carried out on both disturbed and undisturbed soil samples recovered from the boreholes and trial pits for the ground investigation following the 1997 landslide. The tests included particle size distribution, Atterberg limits, density determination and single- and multi-stage triaxial compression tests.

The shear strength properties of the completely decomposed granite matrix were determined using consolidated undrained triaxial compression tests. The test results (Figure 9) show a range of shear strength parameters that broadly fall within the typical range of published data (GEO, 1993).

5.5 Groundwater Conditions

The groundwater conditions at the site were evaluated from a review of available groundwater monitoring records and site observations. The sources of information included:

(a) monitoring of groundwater in borehole No. ED5 by MCA (see Figure 6) between August 1980 and June 1981,

(b) recent observations of seepage on the slope, and

(c) monitoring of groundwater in boreholes Nos. BH1 to BH3, and BH5 to BH11 by Goldram Engineering and Development Ltd and HAP from October 1997 to March 1998.

The base water table is between 1 m and 5 m above rockhead in the lower part of the slope and becomes progressively deeper and is within bedrock beneath the upper part of the slope (Figure 7). Piezometers in boreholes Nos. BH2, BH3 and BH9 in the lower slope indicated hydraulic continuity across the rockhead interface. Piezometric levels in the lower slope recorded during the monitoring period remained above rockhead with a fluctuation of less than about 1 m. The perennial seepage and dampness at the toe of the slope are likely to be fed by groundwater from this source.

Piezometers were installed in borehole Nos. BH8 and BH10 in both the slightly to moderately decomposed granite and the completely decomposed granite. Although the upper piezometers were generally dry, piezometric levels of between 0.15 m and 1 m above rockhead were recorded in boreholes BH8 and BH10 respectively (Figure 7). The piezometric levels recorded in the lower piezometers were well below rockhead indicating a transient perched water table. Whilst the mass permeabilities of both the completely decomposed and bedrock granite are similar, the presence of an open steeply-dipping joint system in the completely decomposed granite, as revealed in trial pits Nos. TP1 and TP9, is likely to increase the rate of infiltration and flow through the completely decomposed granite. It is noted that the presence of the open tension crack and the possible opening of joints during the landslide movement would have further increased water infiltration into the slope which could have rendered the post-failure piezometer monitoring records not representative of the pre-failure conditions.

The base water table in borehole No. ED5, located in the upper slope about 10 m to the west of the 1997 landslide (Figure 6), was monitored for a 10-month period that included parts of the 1980 and 1981 wet seasons. The maximum piezometric level recorded on 17 August 1980 was some 4 m higher than the typical dry season level. The rainstorm associated with the maximum piezometric level in borehole No. ED5 in August 1980 was notably less severe than that recorded between 2 and 3 July 1997 (Figure 5).

Considering the above, it is postulated that the base water table at the time of the 1997 landslide was probably at least 4 m higher than the maximum recorded from subsequent post-landslide monitoring. It is also considered that a transient water table perched above rockhead in the upper part of the slope would also have formed during the rainstorm of 2 and 3 July 1997.

6. THEORETICAL STABILITY ANALYSIS

Theoretical stability analysis was carried out to determine the probable range of operating shear strengths and credible groundwater conditions that could lead to deep-seated failure in the slope. The range of likely shear strength parameters considered was c' = 5 kPa to 10 kPa and ø' = 25° to 37° based on site observations, laboratory testing and experience of similar materials in Hong Kong. The lower bound shear strength would apply in the case where the failure surface passed through a series of kaolin-infilled discontinuities or other zones of weakness. The upper bound shear strength parameters (c' = 10 kPa and ø' = 37°) are typical of those for a matrix of completely decomposed granite.

The stability analyses were carried out using the rigorous method of Morgenstern and Price (1965) using a representative pre-failure cross-section (Figure 10), which is based on the geological cross-section given in Figure 7.

In the analysis the rises in piezometric level have been taken relative to the piezometric surface deduced from the groundwater observations made during the 1997/98 dry season (Figure 10).

For shear strength parameters that would apply for kaolin-infilled discontinuities (c' = 10 kPa and ø' = 25°), a piezometric rise of 2 m would have been required to reduce the factor of safety to unity (Figure 11). Limited groundwater monitoring between August 1980 and June 1981 has shown that the recorded piezometric level August 1980 was some 4 m above the dry season level. Given the severity of the rainstorm on 2 and 3 July 1997, it is probable that an even higher rise in the piezometric level could have occurred at the time of failure. For instance a rise of 5 m could have brought the slope into a state of limiting equilibrium with average operational shear strength parameters (c' = 5 kPa and ø' = 35°).

7. PROBABLE CAUSES OF THE LANDSLIDE

7.1 Diagnosis of Failure Trigger and Key Factors Causing the Instability

The close correlation between the severe rainfall and reporting of the landslide incident suggests that the failure was triggered by rainfall.

The presence of a significant tension crack as well as localised bulging at the toe of the slope, cracking of the slope surface, horizontal displacement and cracking of U-channels suggests that material detachment and the extensive signs of distress could be surface expressions of the development of deeper-seated instability.

In the diagnosis of the causes of the instability, the contributions of the key factors need to be considered, namely, the influence of relict landsliding, groundwater conditions, the presence of unusually weak material and the influence of excavation at the toe of the slope during development of Ville de Cascade.

7.2 Possible Influence of Relict Landsliding

A relict landslide was interpreted from the 1949 aerial photographs (Section 2.2) and in plan it partly overlaps the eastern part of the 1997 distressed area. Such relict landsliding could adversely affect material strength and the groundwater regime. However, it is important to note that the partial overlapping of the relict landslide and the 1997 distressed area does not necessarily imply that the former would have affected the present cut slope given that much material (up to 10 m) was removed during slope formation. Notwithstanding this, the occurrence of a large-scale relict landslide, particularly where the terrain was only gently inclined (about 25°), highlights potential instability problems at the site and could provide an indication of inherent weakness of the terrain and/or unfavourable groundwater conditions. Such destabilising factors could still operate and affect the present distressed portion of the slope.

7.3 Groundwater Conditions

The groundwater regime in the weathering profile, particularly where the material has been subjected to displacement and disturbance, is likely to be complicated and highly variable, with infiltration and subsurface seepage through preferential flow paths and open cracks and joints. Only a limited amount of groundwater monitoring was carried out after the failure but it is possible that the groundwater conditions could have been altered by the instability and hence may not be representative of that prevailing prior to the instability. Although there is much uncertainty regarding the precise groundwater model, there is however evidence of high groundwater conditions, viz. piezometer monitoring over a limited period during the design of the slope, the presence of horizontal drains and seepage observations in the past and during the detailed landslide study. A review of the piezometer data obtained during the original design period indicates that the highest reading of piezometer ED5 in August 1980 was about 4 m higher than the typical dry season level. The rainstorm in August 1980 was notably less severe than that recorded between 2 and 3 July

1997, and hence an even higher rise in piezometric levels was most likely at the time of the 1997 landslide.

7.4 Possible Presence of Unusually Weak Material

The 1997 ground investigation for the LPM works proved the presence of kaolin-infilled adversely orientated discontinuities, which could have played a significant role in causing instability.

Theoretical stability analyses considered potential slip surfaces extending from the tension crack to the toe of the slope. The results suggest that deep-seated instability within the deeply-weathered profile, given fairly typical shear strength parameters and high groundwater conditions, is credible. Although it is not possible to be conclusive about the groundwater conditions at the time of the instability and there is much uncertainty about the persistence of kaolin-infilled discontinuities, it would appear that the operational strength was not particularly low involving unusually weak material. However, it is likely that the presence of locally weak material (e.g. subvertical infilled joints) had contributed to bringing about a lower mass shear strength than that of the matrix strength.

7.5 Possible Influence of Excavation at the Toe of the Slope

During construction of the Ville de Cascade development in 1988, the toe of the slope downslope of the landslide site was locally cut to a steeper angle. This involved removal of only a small amount of material, from the toe to a height of about 2 m. There were no reported incidents of instability between 1988 and 1997 and in the circumstances it is considered that the effect of this local excavation on overall slope stability would have been slight.

7.6 Discussion

It is postulated that the cause of the instability is primarily related to elevated groundwater pressures in response to severe rainfall, superimposed on a high groundwater regime.

The presence of the significant tension crack and extensive signs of distress suggests the possibility of deep-seated instability, which is corroborated by the supporting theoretical analyses. The mode of the instability consists of slope displacement with localised slope detachments. This may be related to the relatively large-scale and deep-seated nature of the instability of a deep weathering profile, variability in piezometric response of the unstable soil mass (both spatially and temporally), release of matrix and discontinuity pore water pressure as the ground mass dilates and joints open up during slope displacement and the fact that the material is not particularly brittle. The comparatively shallow inclination of the slope with a limited driving force may also have been a contributory factor to the relatively ductile response as opposed to a fast-moving uncontrolled failure with major slope detachment.

The extensive ground deformation and instability occurred on a slope which had previously been subjected to a detailed stability assessment based on site-specific ground investigations and laboratory testing. Groundwater monitoring was carried out over a limited period of time and it would appear that the design assumptions in respect of groundwater during intense rainfall were less severe than those realised in practice. It is also possible that the shear strength parameters of c' = 13 kPa and ø' = 39° (Section 2.3) adopted in the design assessment were optimistic.

Although the tension crack was not identified for some 4 months after the reported landslide incident, its relatively fresh appearance suggests that it probably formed at the same time as the failure in July 1997, during a period of severe rainfall.

There are no maintenance records for the horizontal drains at the site. It has not been possible to establish whether deterioration of horizontal drain performance was a contributory factor in the landslide.

8. CONCLUSIONS

It is concluded that the 1997 landslide was triggered by severe rainfall.

It is possible that the landslide together with the tension crack and signs of distress on the slope could be surface expressions of the development of deeper-seated instability. The cause of the instability is probably primarily related to elevated groundwater pressures in a deep weathering profile in response to severe rainfall.

The cut slope was previously subjected to a detailed assessment based on site-specific ground investigation and laboratory testing. However, it would appear that the assumptions with respect to groundwater levels during intense rainfall were less severe than those realised in practice.

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Table 1 – Summary of Aerial Photographic Interpretation

Year	Photographic Reference No.	Altitude (feet)	Observations (see also Figure 3)
1949	Y02064, Y02065	5 800 ft	Clarity of photographs is poor, though the main features of the site can be discerned. The area is natural terrain with the site of the slope (7SW-B/C123) a northeast hillside facing a stream valley (A). The stream is aligned along and adjacent to the toe of the slope. A relict landslide is present on the hillside, characterised by a depression about 50 m wide beneath which are lobate features within the stream (A). Two minor streams, aligned northeast to southwest, cut through the central (B) and western (C) portion of the slope site. Agricultural fields are present in the lower reaches of stream C. Minor drainage gullies are present in the natural topography to the east of the slope site.
1963	Y09191, Y09192	4 000 ft	Site is natural terrain. Two minor landslides are present within the western section of the site adjacent to the landslide identified from the 1949 aerial photograph. A terrace of material extending into the stream (A) is present along the western toe of the hillside beneath the failures. The terrace is partly lobate and has been incised by stream action, and is likely to have been formed by debris from the identified failures.
1976	12546, 12547	4 000 ft	Earthworks have commenced within the platform areas above and below the slope site. Site remains unaltered, except for a haul road linking the platform workings cutting through the central eastern portion of the slope site.
1977	20117	4 000 ft	Platform areas in the vicinity of the slope are substantially complete. Construction of the slope is complete with a similar configuration as the present day slope. The slope consists of three batters with surface protection separated by access berms. Stepped channels and the stairway are not present. Rock outcrops visible particularly in central area of slope.
1980	30762, 30763	4 000 ft	Slope remains unaltered. Western-most part of the slope appears to have shallow fill deposit within the location of stream C. No buildings are present.
1982	44590, 44591	10 000 ft	Stairway under construction and stepped channels built. The upper eastern-most part of the slope has been reprofiled, removing a small spur feature. Western end of slope has been extended into stream valley (C) by placement of fill.
1984	56847, 56848	4 000 ft	A shallow failure is visible on the western part of the slope and minor erosion is visible in upper batter and along upper berm. Construction of Shatin College underway.
1986	A04860, A04861	4 000 ft	Further shallow failure and erosion at the same location as 1984. Construction of Greenwood Terrace and Ville de Jardin underway. Shatin College appears to have been completed.
1988	A14033, A14034	4 000 ft	Construction of Ville de Cascade development substantially complete. Sections of the slope toe in the west have been cut-back, allowing the construction of Ville de Cascade internal road. As part of the development a further platform, used as a recreational area, has been constructed adjoining the western end of the slope.
1991	A27118, A27119	4 000 ft	Minor surface erosion adjacent to stairway on slope.
1995	CN10892, CN10893	2 500 ft	Some minor seepage staining visible beneath area of vegetation on the eastern part of the slope. Eastern part of slope beyond stairway has a surface protection. The western part of the slope beyond the stairway is heavily-vegetated, particularly in the western-most part.

Table 2 – Maximum Rolling Rainfall at GEO Raingauge No. N02 for Selected Durations Preceding the 3 July 1997 Minor Failure and The Corresponding Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	12.5	06:15 hours on 2 July 1997	1
15 minutes	34.5	06:15 hours on 2 July 1997	1
1 hour	124	06:25 hours on 2 July 1997	1
2 hours	185.5	07:00 hours on 2 July 1997	78
4 hours	229.5	07:00 hours on 2 July 1997	24
12 hours	367	15:00 hours on 2 July 1997	33
24 hours	587.5	03:00 hours on 3 July 1997	132
2 days	660.5	04:00 hours on 3 July 1997	98
4 days	699.5	10:00 hours on 3 July 1997	46
7 days	740	09:00 hours on 3 July 1997	37
15 days	856	10:00 hours on 3 July 1997	23
31 days	1375	10:00 hours on 3 July 1997	151

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

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

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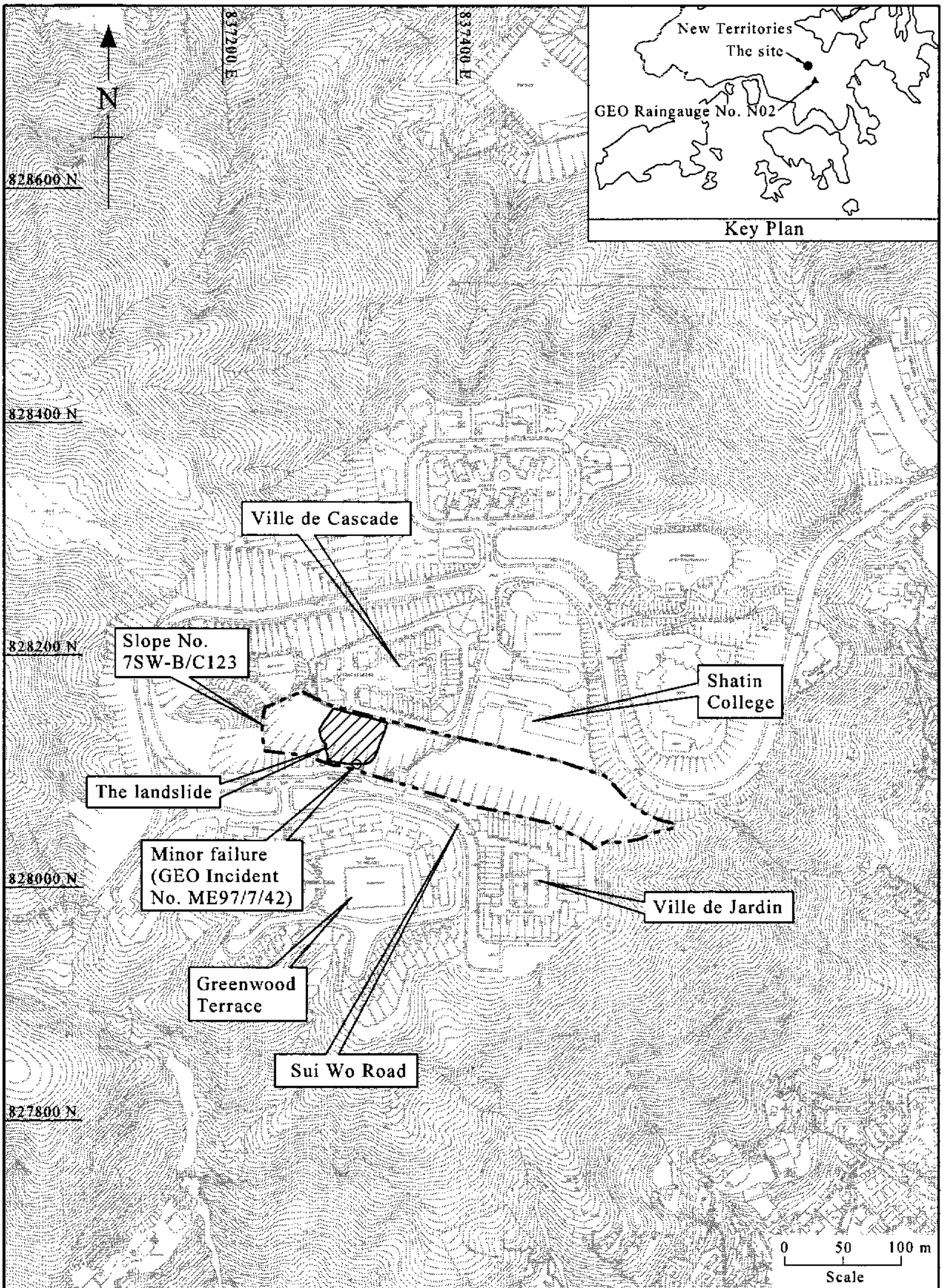
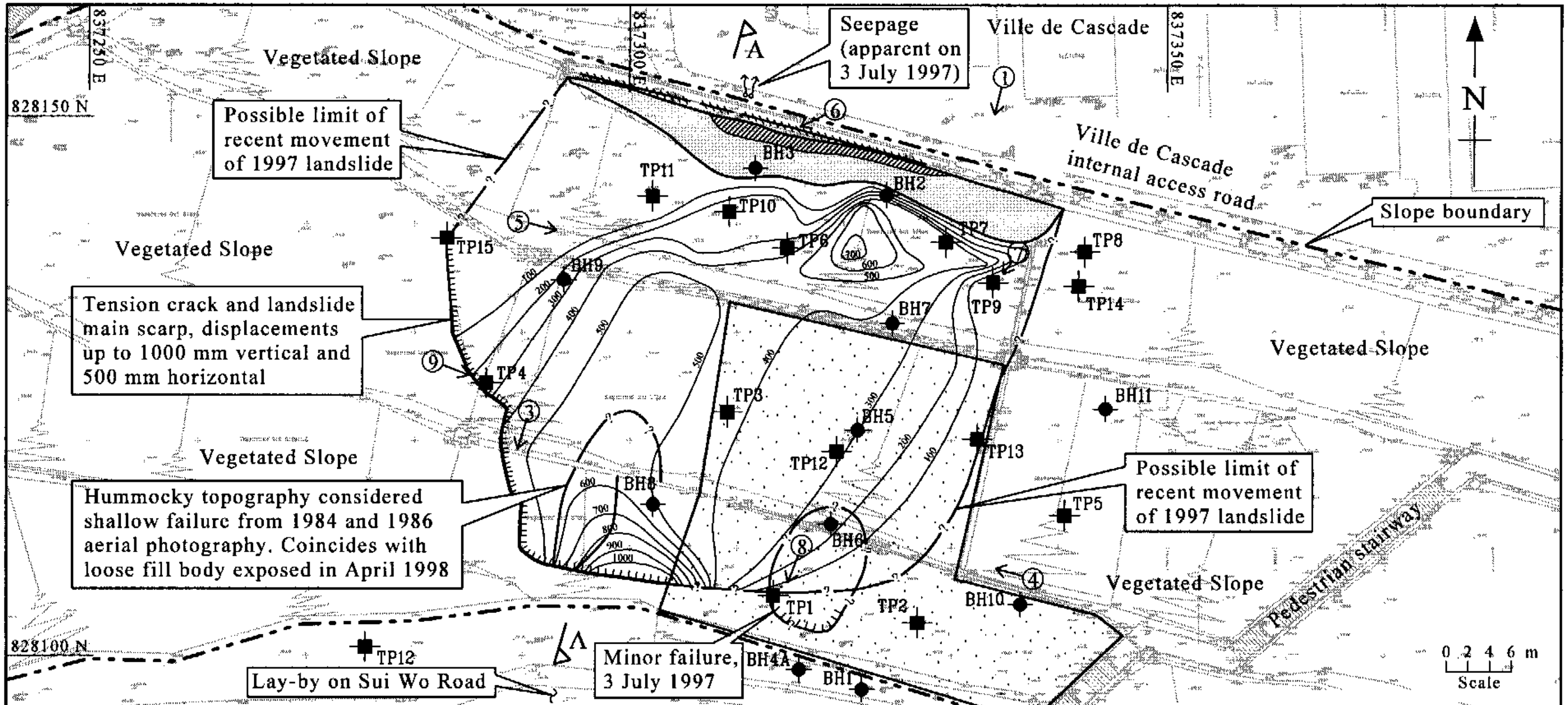


Note : Base map is extracted from Survey Sheet Nos. 7SW-10B, 7SW-10D, 7SE-6A and 7SE-6C dated October 1992 (Scale 1:1 000).

Figure 1 - Site Location Plan



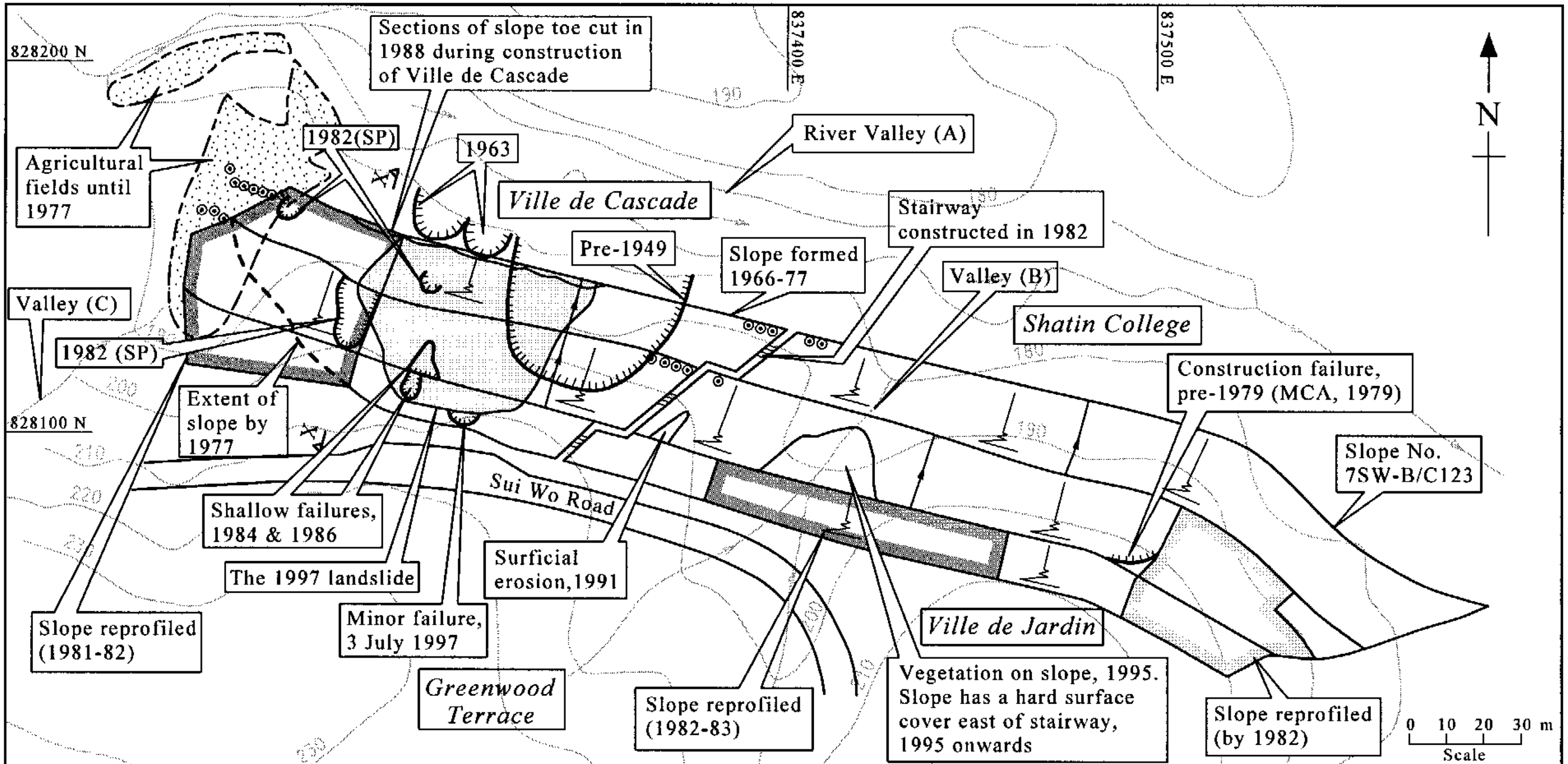
Legend :

Bulging slope surface	Crushed channel	① Plate No.1 location and orientation	Approximate vertical slope displacement with contour label in mm (see note 2)
BH5 Borehole BH5, Goldram Engineering and Development Ltd., 1997/98	TP5 Trial pit TP5, Goldram Engineering and Development Ltd., 1997/98		Recent shotcrete covering by HyD contractor (July to August 1997)
Tension crack and landslide main scarp	Area of minor seepage and dampness	A A Cross-section A-A	

Notes : (1) Boundary of the 1997 landslide is based on site observations and ground investigation.

(2) Vertical slope displacement contours have been determined from: level surveys along drainage channels and landslide scarp, observation, and comparison with the likely as-constructed slope profile.

Figure 2 - Plan of the Landslide

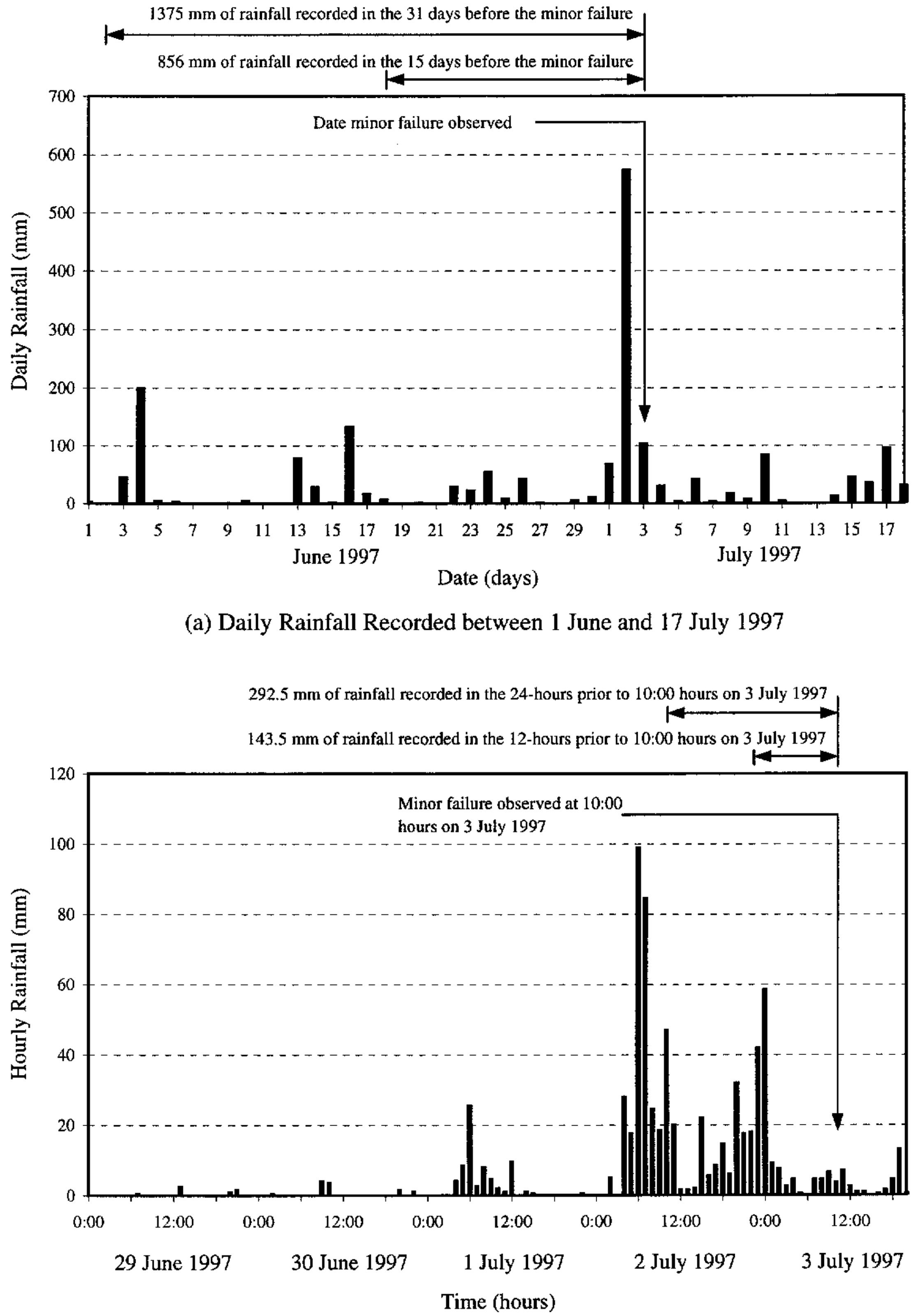


Legend:

- Slope reprofiled (MCA, 1983)
- Extent of slope reprofiling by removal of loose fill and replacement with compacted fill (MCA, 1983)
- *Shatin College* — Approximate location of Shatin College
- 1982(SP) — Landslide scar, identified from 1982 site photograph
- 1963 — Landslide scar, identified from 1963 aerial photograph
- X X — Stability section x - x analysed by MCA (1979-81)
- ⊙ — Horizontal drain shown on as-built drawing (MCA, 1983)

Notes :
(1) Development history based on API and documentary record.
(2) The boundary of the 1997 landslide is based on site observations and ground investigation.
(3) Outline of slope superimposed onto pre-existing natural terrain contours.

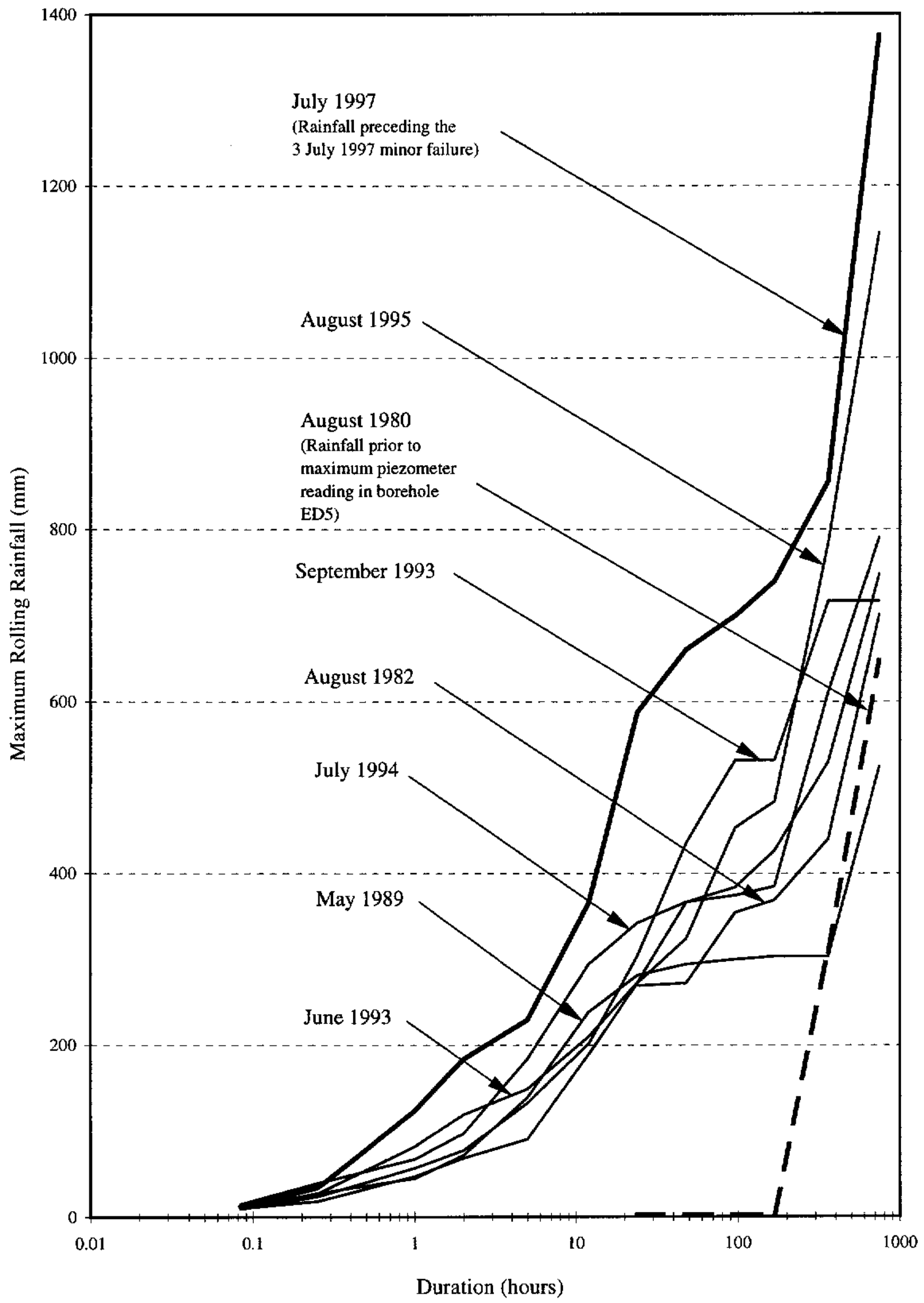
Figure 3 - Site Plan and History of Development



(a) Daily Rainfall Recorded between 1 June and 17 July 1997

(b) Hourly Rainfall Recorded between 29 June and 3 July 1997

Figure 4 - Rainfall Recorded at GEO Raingauge No. N02



Note: Rainfall preceding August 1980 is based on daily rainfall data from 17 August 1980.

Figure 5 - Maximum Rolling Rainfall at GEO Raingauge No. N02 for Major Rainstorms

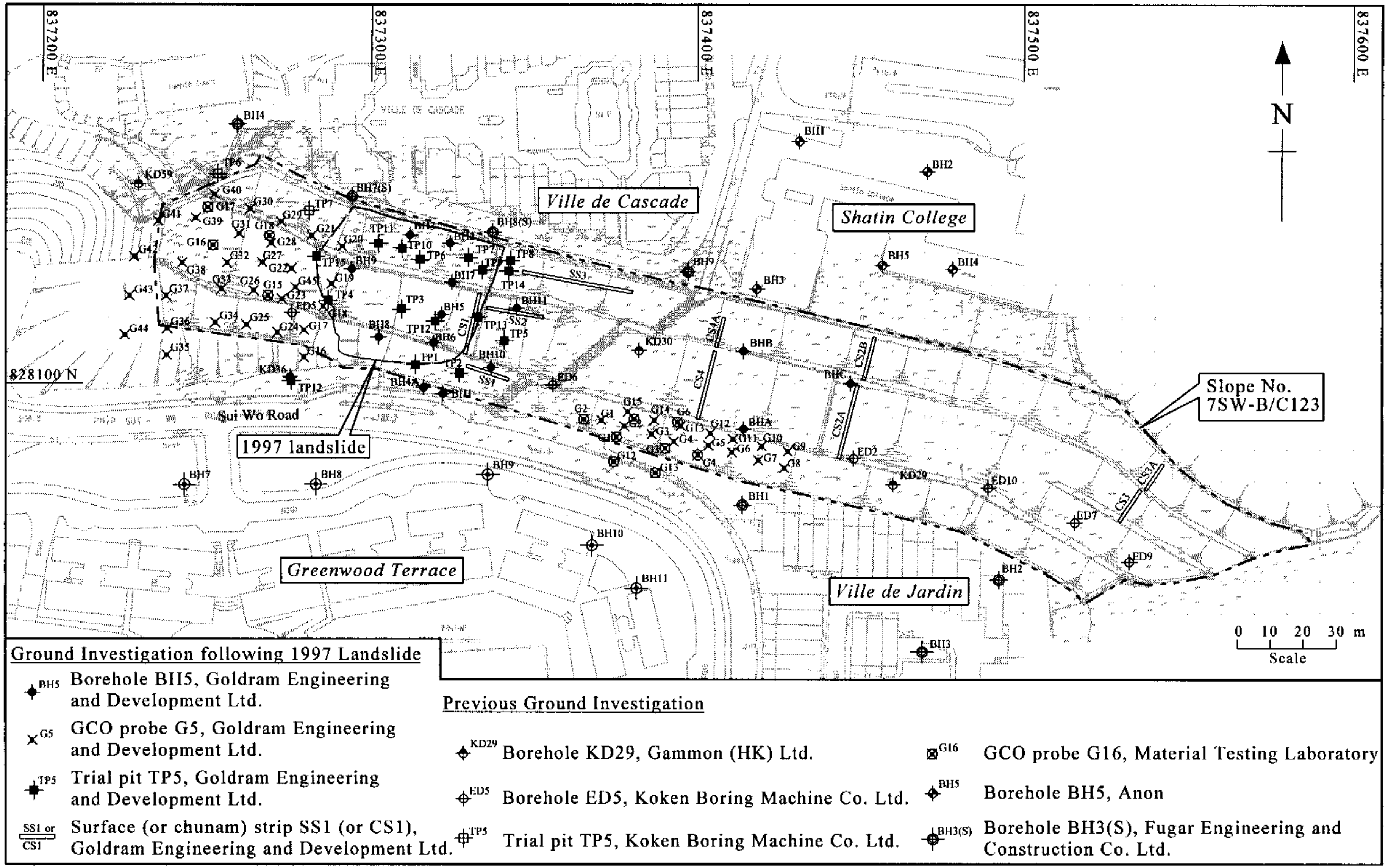
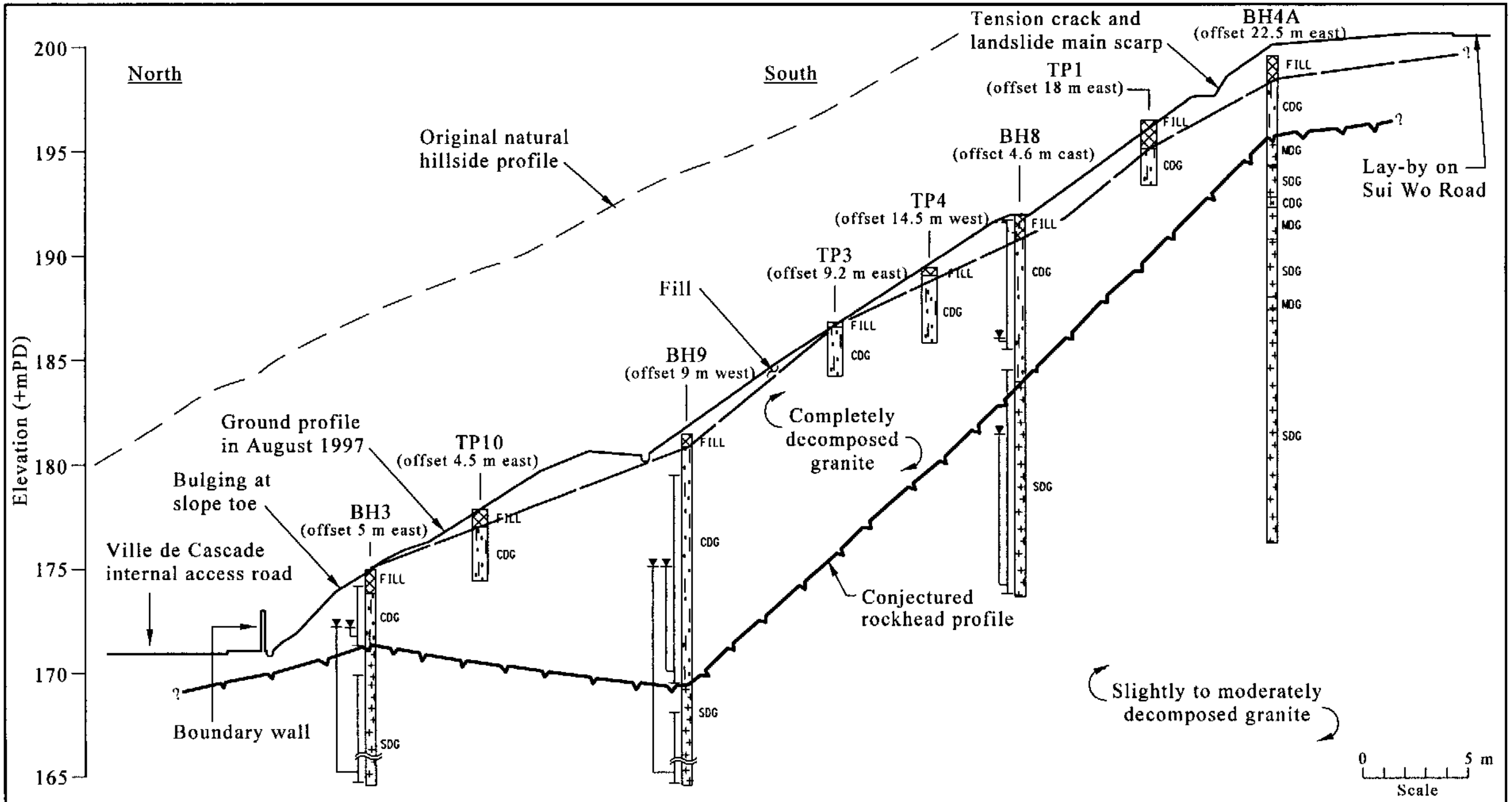


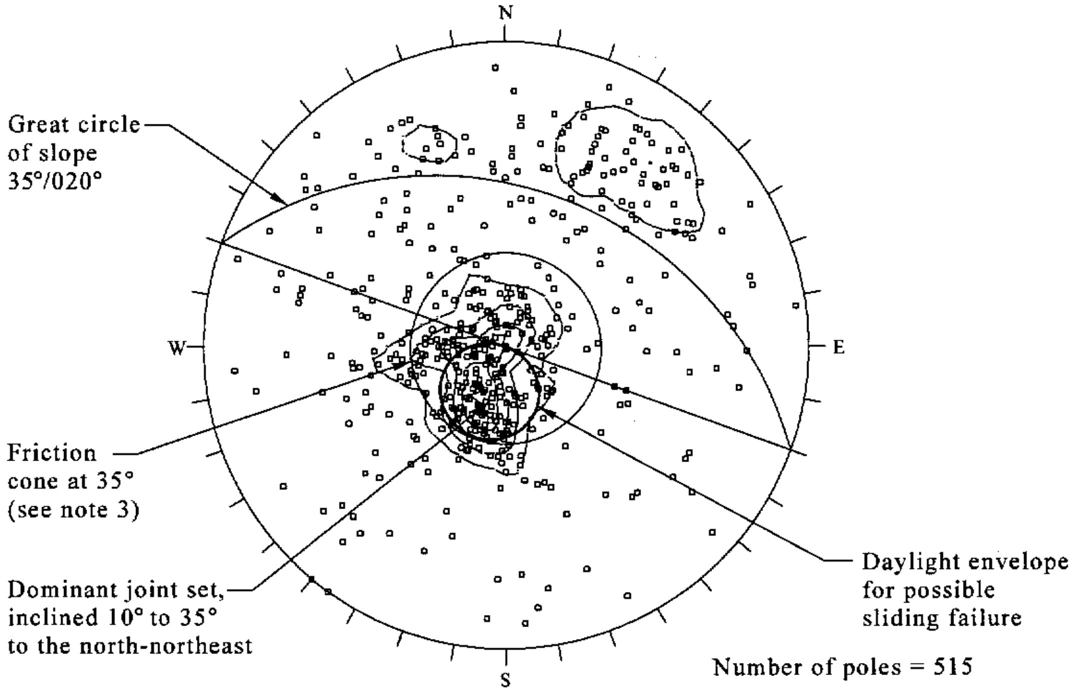
Figure 6 - Site Investigation Location Plan



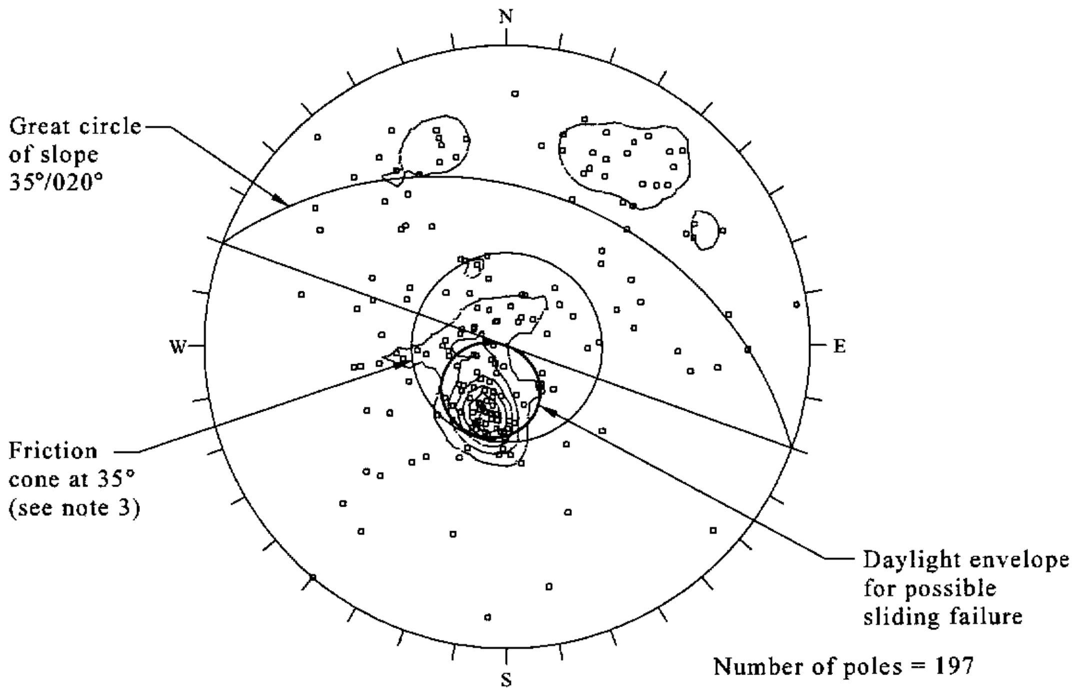
Legend : Fill | Completely decomposed granite (CDG) | Moderately to slightly decomposed granite (M to SDG) | Maximum measured water level

Notes : (1) Information shown in this figure is based on topographic survey, ground investigation and site observations by HAP.
(2) See Figure 2 for location of section.
(3) Water level is based on limited piezometer monitoring period from 30 October 1997 to 6 March 1998.

Figure 7 - Geological Cross-section A-A through the Landslide



(a) Discontinuity data from boreholes, surface strips and trial pits in ground investigation following 1997 landslide

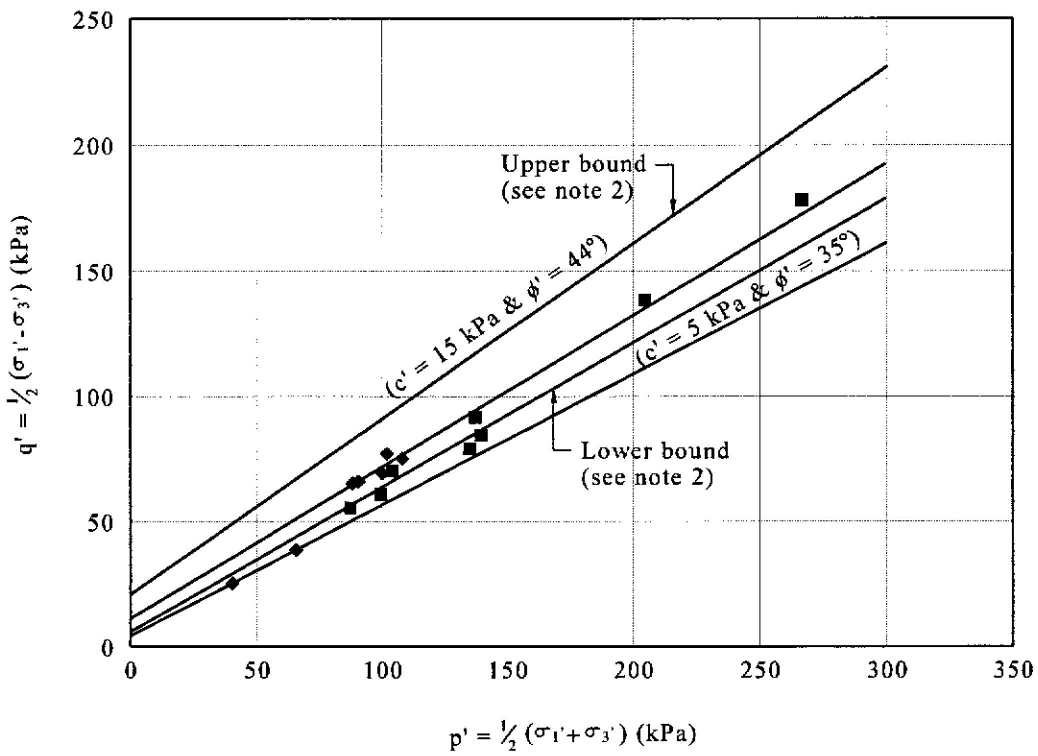


(b) Discontinuities with kaolin infill or slickensides from boreholes and trial pits in ground investigation following 1997 landslide

Legend : ▫ Pole ——— Contour of pole concentration

Notes :
(1) Stereoplot of equal angle lower hemisphere.
(2) Discontinuity data from recent ground investigation following 1997 landslide.
(3) Friction angle of 35° is shown for comparison purposes only.

Figure 8 - Stereographic Projection of Discontinuities within Granite at the Landslide Site



Legend :

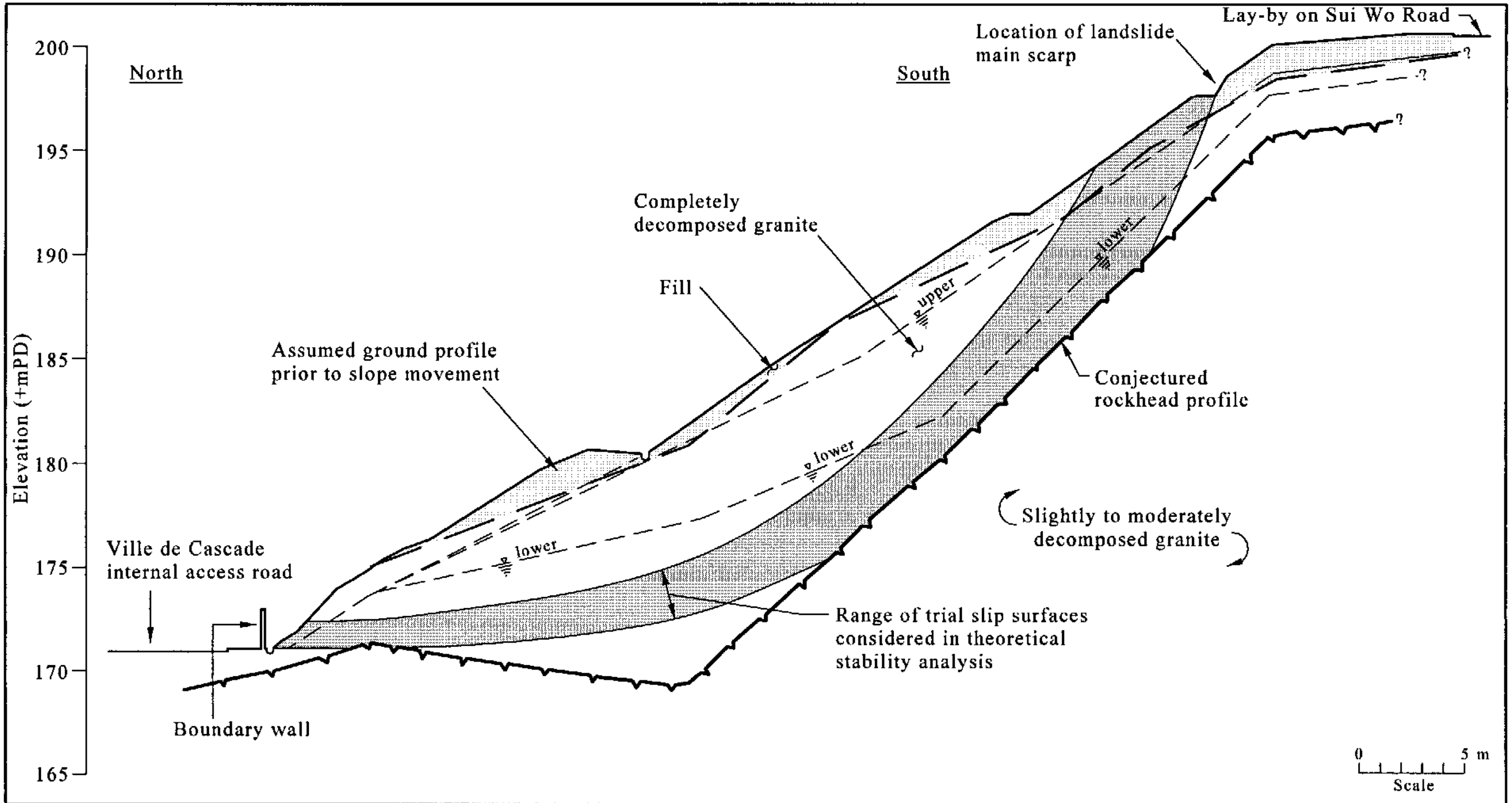
■ Single-stage test — c' Cohesion — σ_1' Major principal effective stress

◆ First stage of multi-stage test — ϕ' Angle of shearing resistance — σ_3' Minor principal effective stress

Notes : (1) Test results correspond to the point of maximum stress ratio, i.e. σ_1'/σ_3'.

(2) Upper and lower bound parmeters are based on GEO (1993).

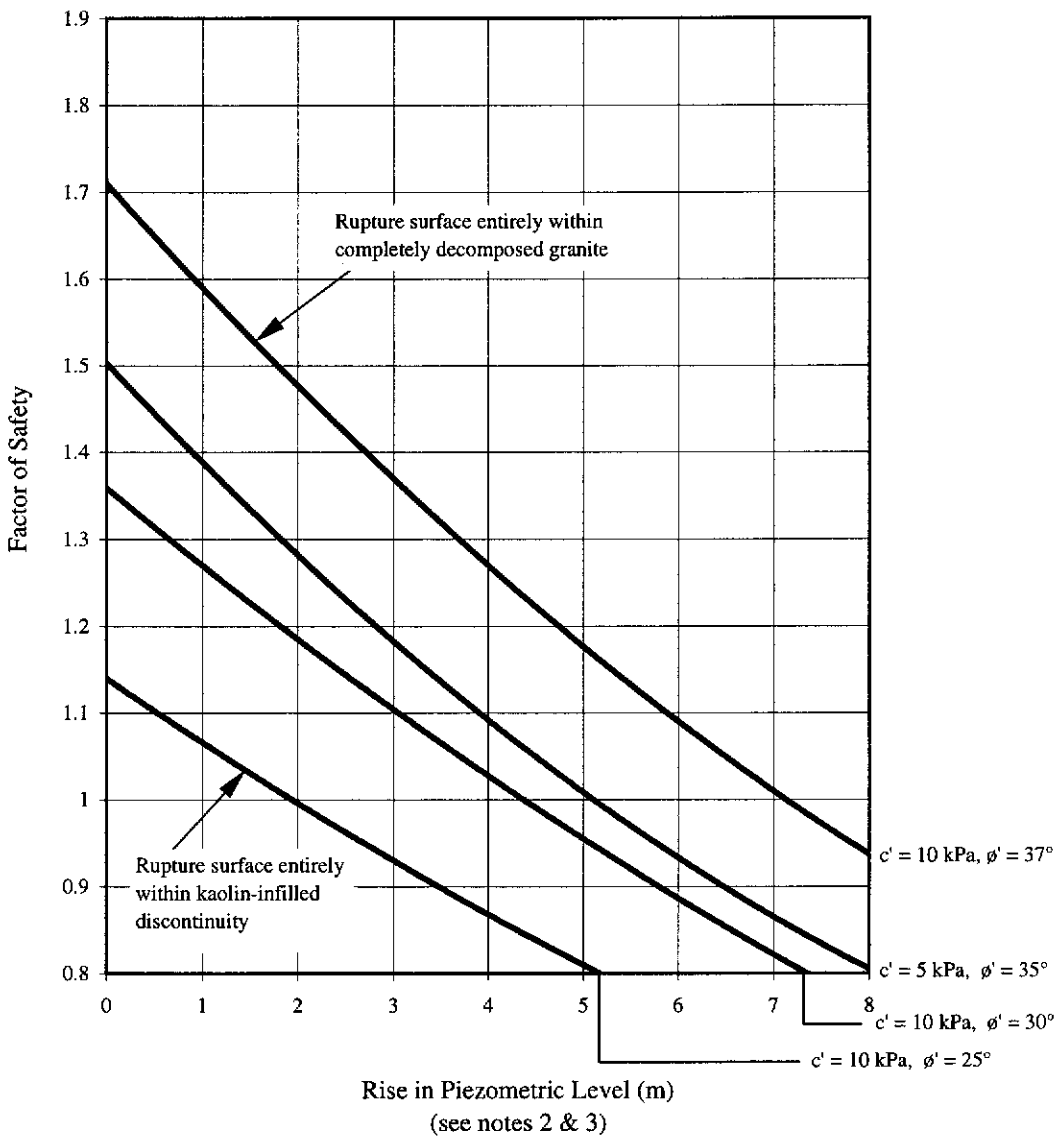
Figure 9 - Triaxial Compression Test Results for Completely Decomposed Granite



Legend : Assumed initial piezometric level (see note 2) | upper Piezometric level at limiting equilibrium for assumed upper bound shear strength | lower Piezometric level at limiting equilibrium for assumed lower bound shear strength

Notes :
(1) Shallow failures within the fill are not considered.
(2) For assumed initial groundwater condition, see Section 6.
(3) Based on section A-A, see Figure 7.

Figure 10 - Slope Model for Theoretical Stability Analysis



Legend:

—— Variation of factor of safety with change in piezometric level for constant c' and ø'

c' apparent cohesion

ø' effective angle of shearing resistance

Notes: (1) For cross-section, see Figure 10.

(2) For initial groundwater condition in the slope (corresponding to zero peizometric rise), see Figure 10 and Section 6.

(3) Rise in piezometric level is measured in the central part of the slope.

(4) Soil strength parameters shown above are illustrative of the likely range and should not be interpreted as the actual soil strength.

Figure 11 - Results of Theoretical Stability Analysis

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Plate 1 - View of Minor Failure (GEO Incident No. ME97/7/42) on Slope No. 7SW-B/C123 (Photograph Taken on 7 July 1997)

Plate 2 - View of Western Part of Slope No. 7SW-B/C123 (Photograph Taken on 29 May 1984)

Plate 3 - View of Recent Displacement across Western Part of Tension Crack (Photograph Taken on 2 February 1998)

Plate 4 - Displacement of U-Channel, shown in Background along Upper Berm (Photograph Taken in November 1997)

Plate 5 - Displacement of U-Channel along Lower Berm (Photograph Taken on 29 October 1997)

Plate 6 - Bulging and Collapse of Slope Toe (Photograph Taken on 30 October 1997)

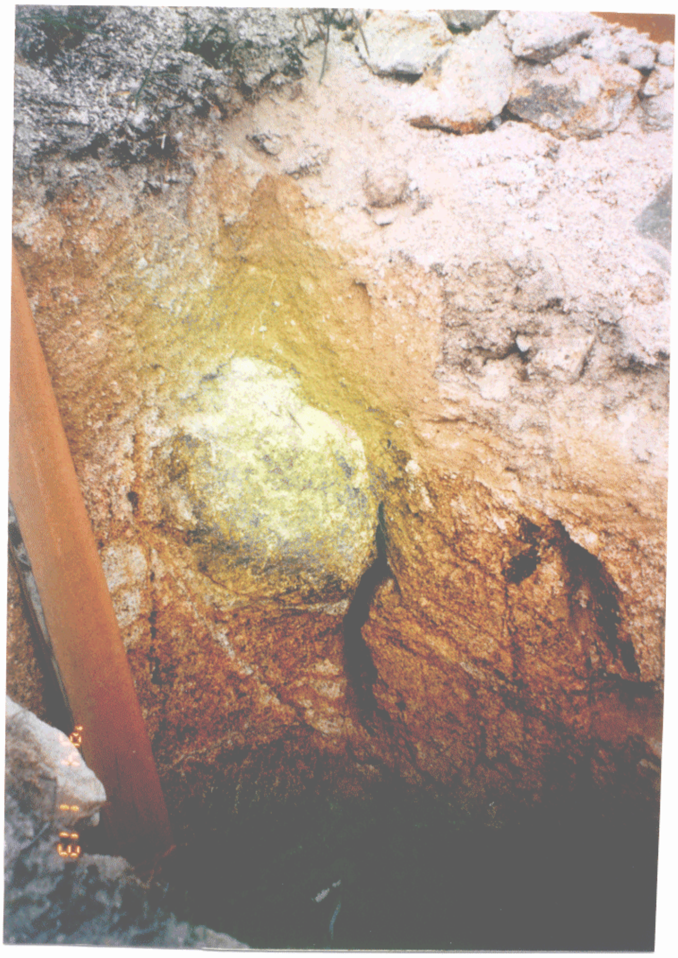


Plate 7 - Partially Infilled Open Joints within Trial Pit No. TP9
(Photograph Taken in November 1997)

Plate 8 - Open Relict Joint about 80 mm Wide in Completely Decomposed Granite below Fill in Trial Pit No. TP1 (Photograph Taken on 27 November 1997)

Plate 9 - Tension Crack Exploiting Pre-existing Relict Joint in Completely Decomposed Granite in Trial Pit No. TP4 (Photograph Taken on 24 November 1997)

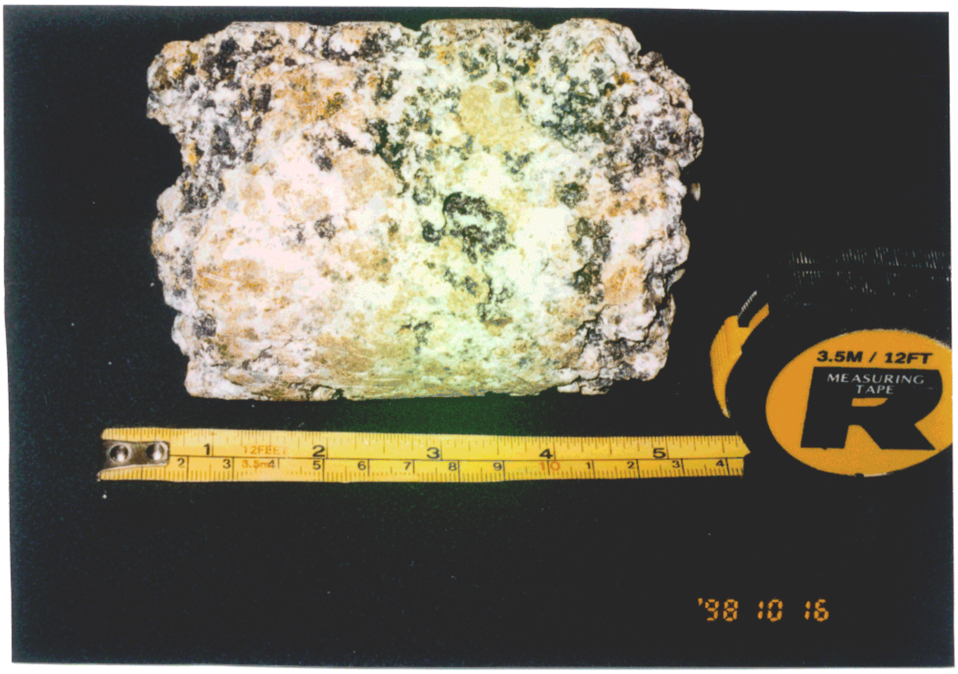


Plate 10 - View of Partial Mineral Dissolution in Moderately Decomposed Granite Retrieved from Borehole No. BH2 Between 3.3 m and 6.1 m Depth (Photograph Taken in October 1998)

APPENDIX A

SUMMARY OF SITE HISTORICAL DOCUMENTS

Table A1 – Summary of Site History Documents (Sheet 1 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1978	Sha Tin New Town Development, Planning Area 41A – Layout Plans No. STP 78/006C, Explanatory Statement	NTDD	CEDW	Slope No. 7SW-B/C123 is located in Shatin New Town Development Planning Area 41A. This area was designated as a borrow area in conjunction with the reclamation for the development of Shatin Race Course.
1979	Geotechnical Checking of New Works, Sha Tin – Planning Area 41A (Correspondence)	B & P	GCO	B & P, as the GCO checking engineers for new works, were requested to report on the geotechnical aspects of Area 41A. The report is based on site observation and B & P's knowledge of the area. In view of the proposed residential usage B & P considered " ......a full geotechnical assessment should be made of the present configuration of platforms and slopes....". Particular comments were given on the platform areas and their adjacent slopes, with regards to slope No. 7SW-B/C123 the following comments were given. The western slope was 30 m high at 1 on 1.5 although locally steepened to 1 on 1. "Seepage at toe" was noted. B & P recommend a "full stability check required." The eastern slope was 30 m high in soil with chunam surfacing. Typically at 1 on 1.5, but locally steeper. B & P recommend that a "full stability check required."

Table A1 – Summary of Site History Documents (Sheet 2 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1979	Report on Slope Stability Assessment, Sha Tin Area 41A	MCA	NTDD	MCA, as consultants to NTDD undertook a geotechnical investigation of Area 41A. This report presented the results and recommendations of the geotechnical investigation. The investigation included mapping weathering states, lithology, areas of groundwater seepage, condition of slope protection work and drainage on existing cut slopes. The report divided slope No. 7SW-B/C123 into three areas: the western end up to the lay-by in Sui Wo Road is referred to as "Slope G", the central area is "Slope F", and; the eastern end is "Slope E". Slope stability assessment of soil portions of the slope used soil shear strength parameters of $c' = 13$ kPa and $ø' = 39°$ and bulk unit weight of 19 kN/m^3. Groundwater assumptions were based on a "minimum of 2 to 3 m rise in the measured highest wet season piezometric levels". The 1997 landslide occurred on Slope F and partly extended into Slope G (Figure 3). The observations of MCA on the overall slope are given below. <u>Slope E (eastern slope)</u> The upper slope is in "soil" with slightly weathered granite in the lower slope. It was observed that "groundwater seepage occurs, as is evident from field observations, throughout the year from piezometric level monitoring". Stability analysis gave a factor of safety of 1.30 for the soil slope. Recommended remedial works included trimming of soil slope and stabilisation of potentially unstable blocks.

Table A1 – Summary of Site History Documents (Sheet 3 of 12)

Year	Document Title	By Whom	To Whom	Remarks
				Slope F (central slope) A cross-section reproduced in Figure A1, and shown as X – X on plan in Figure 3, located on the western side of the 1997 landslide, shows rockhead at 7 m to 14 m beneath the surface overlain by completely decomposed granite. Maximum measured piezometric level (from borehole ED5) was 3 m above rock-head. Stability analysis showed "that the slope has a minimum factor of safety of 1.5 against deep-seated or shallow failures". It was also noted that there was "constant seepage….along the pre-existing stream course [see Figure 3] and dampness is evident in adjoining areas on the slope". Recommended remedial works included horizontal drains installed in old stream course. Slope G (western slope) It was noted that this slope was "mainly fill". The slope "was part of an old haul road and this was removed by simply spreading out the fill material. From the original contours the maximum fill depth is around 6 m…..". No recommended remedial works given, except "re-turf part of slope".

Table A1 – Summary of Site History Documents (Sheet 4 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1980	Geotechnical Checking of New Works, Shatin New Town Area 41A, Assessment of Stability and Remedial Work (Correspondence)	B & P	GCO	B & P, as the GCO's checking engineers for new works, reviewed the findings of MCA's 1979 report. B & P stated, "Generally we have endorsed MCA's proposals but have also recommended further investigation and design". B & P's general comments concerning slope 7SW-B/C123 were: Groundwater was "monitored periodically [in Area 41A] throughout the 1979 wet season and more frequently, after significant storms….. it is unlikely the peak response has been recorded". However, B & P considered the MCA design "piezometric profile 2 to 3 m above and parallel to the inferred soil/rock interface….to be reasonable". Soil and rock parameters used by B & P were based on their previous work in the area. The parameters used for soil were $c' = 10$ kPa and $ø' = 40°$. A summary of checking results noted that for : <u>Slope E</u> "Soil slope – the extent of MCA's recommended remedial works should be expanded to ensure stability. Rock slopes – essentially stable. The extent of MCA's proposed remedial bolting may require expansion on closer inspection of sheeting joints". <u>Slope F</u> "Cut slope satisfactory at 30 m high at 1 on 1.5 with F>1.4. Extent and stability of fill slope yet to be proved. Remedial works probably required". <u>Slope G</u> "Deep seated stability in residual soil gives F> 1.5. Investigation of the fill slope to determine extent, density and interface with original ground required. Stability analysis may indicate remedial work to be necessary".

Table A1 – Summary of Site History Documents (Sheet 5 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1980	Sha Tin Area 41A, Slope Stability Assessment (Correspondence)	MCA	B & P, GCO, NTDD	Response to B & P's recommendations and comments on 1979 MCA report. MCA noted that compliance with B & P's recommendations would require "a considerable amount of further investigation and stability analysis". For Slope E, MCA agreed with the recommendations of B & P. For Slope F and G the extent of fill bodies needed to be investigated further and "this will be carried out by a combination of probing, trial pits and drillholes.
1980	Geotechnical Checking of New Works, Sha Tin Area 41A	B & P	GCO	Minutes of meeting with GCO, B & P, MCA and NTDD to "agree the extent of further investigation, design and remedial work considered necessary to raise the area to meet current standards". For slopes F and G, "doubts cast as to extent and stability of fill….will be investigated by MCA. Initially the GCO probe and then trial pits to be used". With respect to required geotechnical remedial works for Area 41A, "It was agreed that none of the previously agreed remedial work ….. was likely to affect the Town Plan as detailed on drawings STP 78/006E".

Table A1 – Summary of Site History Documents (Sheet 6 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1980	Supplementary Report on Slope Stability Assessment in Area 41A	MCA	B & P, GCO, NTDD	Report presented the results and calculations of further site investigation and additional slope stability analyses in Area 41A. Slope F GCO probe tests carried out, and extent of fill defined, see Figures 3 and 6. Stability analysis "with assumed values of c' = 0 kPa and ø' = 36° for the existing soil condition reveals a minimum FOS of 1.31. Area of loose fill will be compacted…..". Slope G GCO probe tests undertaken and the extent of the fill defined, see Figures 3 and 6. Stability analysis "…..using same strength parameters as Slope F and assuming a conservative position of a perched ground water table gives a [factor of safety] FOS 1.26. We recommend recompaction [of] the fill with the provision of a drainage blanket underneath……further investigations with GCO probe should be carried out to define the extent of fill behind the top of the slope….".

Table A1 – Summary of Site History Documents (Sheet 7 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1980	Geotechnical Checking of New Works, Sha Tin New Town Area 41A, Assessment of Stability and Remedial Works (Correspondence)	B & P	GCO	Comments on MCA Supplementary Report on Slope Stability. Slope E B&P noted that they "have not seen stability analyses of this proposal and request it be submitted". Slope F "…the lateral extent of the fill has yet to be confirmed. Without piezometric data, the assumed perched water table 1.5 m above the 'probed' base of the fill cannot be deemed conservative. Provision of adequate underdrainage should be considered…Calculations indicating final FOS following remedial works are also outstanding for this slope". Slope G "…proposed recompaction with drainage underblanket appears satisfactory although a final stability check should be submitted to confirm this…. However, we are concerned that more critical fill slopes may exist in this area, particularly below the present access road [Sui Wo Road]...".

Table A1 – Summary of Site History Documents (Sheet 8 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1980	Sha Tin New Ton Area 41A, Assessment of Stability and Remedial Works (Correspondence)	MCA	B & P, GCO, NTDD	Reply to B & P comments. With respect to slope 7SW-B/C123, it was noted that : Slope E "Stability analysis of the recent slope was submitted in the original submissions". Slope F In response to B & P's comment that the fill body was not adequately defined, MCA considered, "GCO probes sufficiently defined the extent, depth and degree of compaction of the fill such that a stability check could be carried out and remedial work recommended". Slope G MCA, in order to ensure no further fill was present, stated that, "Further investigation of the extent of fill will be carried out during the remedial work contract". In response to B & P's comment that more critical fill slopes may be present beneath Sui Wo Road, above Slope G, MCA stated that, "We do not consider further more critical fill slopes may exist and would be pleased to receive their location".
1980	Geotechnical Checking of New Works, Sha Tin Area 41A (Correspondence)	B & P	GCO	B & P comments on MCA's reply. Slope E Considering MCA's reply, B & P "accept that this will result in an adequate factor of safety being achieved". Slope F & G B & P noted MCA's reply and awaited "a design submission demonstrating that an adequate FOS will be achieved in the recompacted slopes". With regard to possible more critical fill slopes, B & P "should be pleased to receive an assessment of stability for the slopes below the present access road [Sui Wo Road]".

Table A1 – Summary of Site History Documents (Sheet 9 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1981	Sha Tin New Town, Area 41A, Design Submission on Slopes (Report)	MCA	B & P, GCO, NTDD	The purpose of the report was to reassess the stability of fill and cut slopes which remained outstanding from previous submissions. Slope F The proposed remedial works consist of "removal of all the loose fill and turf down to in-situ decomposed granite, laying a 700 mm filter blanket compacted in layers, reprofiling the slope to a gentler gradient with compacted fill, returfing the recompacted fill slope surface and reinstating surface drainage" (Figure 3). Stability analysis shows a minimum factor of safety of 1.55 on the proposed reconstructed slope. Slope G A similar proposed remedial works included "removal of all the loose fill and turf down to in-situ decomposed granite, laying a 700 mm filter blanket, reprofiling the slope to a gentler gradient with compacted fill, turfing the slope surface and reinstating surface drainage" (Figure 3). Stability analysis gave a minimum factor of safety of 1.55 on the proposed reconstructed slope.
1981	Geotechnical Checking of New Works, Sha Tin Area 41A (Correspondence)	B & P	CGE/NW	Comments on MCA's design submission. Slope E Having reviewed MCA's original submission, B & P "agree that this results in acceptable stability (F = 1.4) being obtained". Slope F B & P noted that, "The lateral extent of the loose fill will be confirmed during constructions". And that, "the proposed recompaction with filter [is] acceptable". Slope G Similarly, B & P noted that , "The lateral extent of the loose fill will be confirmed during construction". And that, "the proposed remedial works [are] satisfactory".

Table A1 – Summary of Site History Documents (Sheet 10 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1981	Geotechnical Checking of New Works, Shatin – Area 41A (Correspondence)	B & P	GCO	B & P handed over the geotechnical checking of slopes in Area 41A to GCO.
1982	Geotechnical Checking of New Works, Site Visit Report Sheet	GCO	-	A site visit report by GCO, noted that on Slope G "…there is an area of surface instability in the second batter" (Figure 3).
1983	Sha Tin New Town, Contract No. 626/80, Formation, Roads & Drainage & Slope, Remedial Works in Area 41A, 42, 43, 46 & 16B Area 41A – Slopes A, B, C: Slope N (Correspondence)	MCA	NTDD	Issue of as-built drawings for Area 41A, including Slope E, F and G.
1984	Sha Tin New Town Contract No. 626/80, Formation, Roads & Drainage & Slope, Remedial Works in Areas 41A, 42, 43 and 46 (Correspondence)	GCO	NTDD	A review of the as-built drawings noted for Slope G that, "The extent of fill, on assumption that it exists, has not been shown on the drawings".
1984	Sha Tin New Town Contract No. 626/80, Slope Remedial Works in Areas 16B, 41A, 42, 43 and 46 (Correspondence)	MCA	NTDD	In reply to GCO's comment on the as-built drawings, MCA stated that, "GCO's assumption that fill material still exists in this slope G is not valid. Actually, all the loose fill material had been removed. Therefore, our calculated factors of safety still apply".

Table A1 – Summary of Site History Documents (Sheet 11 of 12)

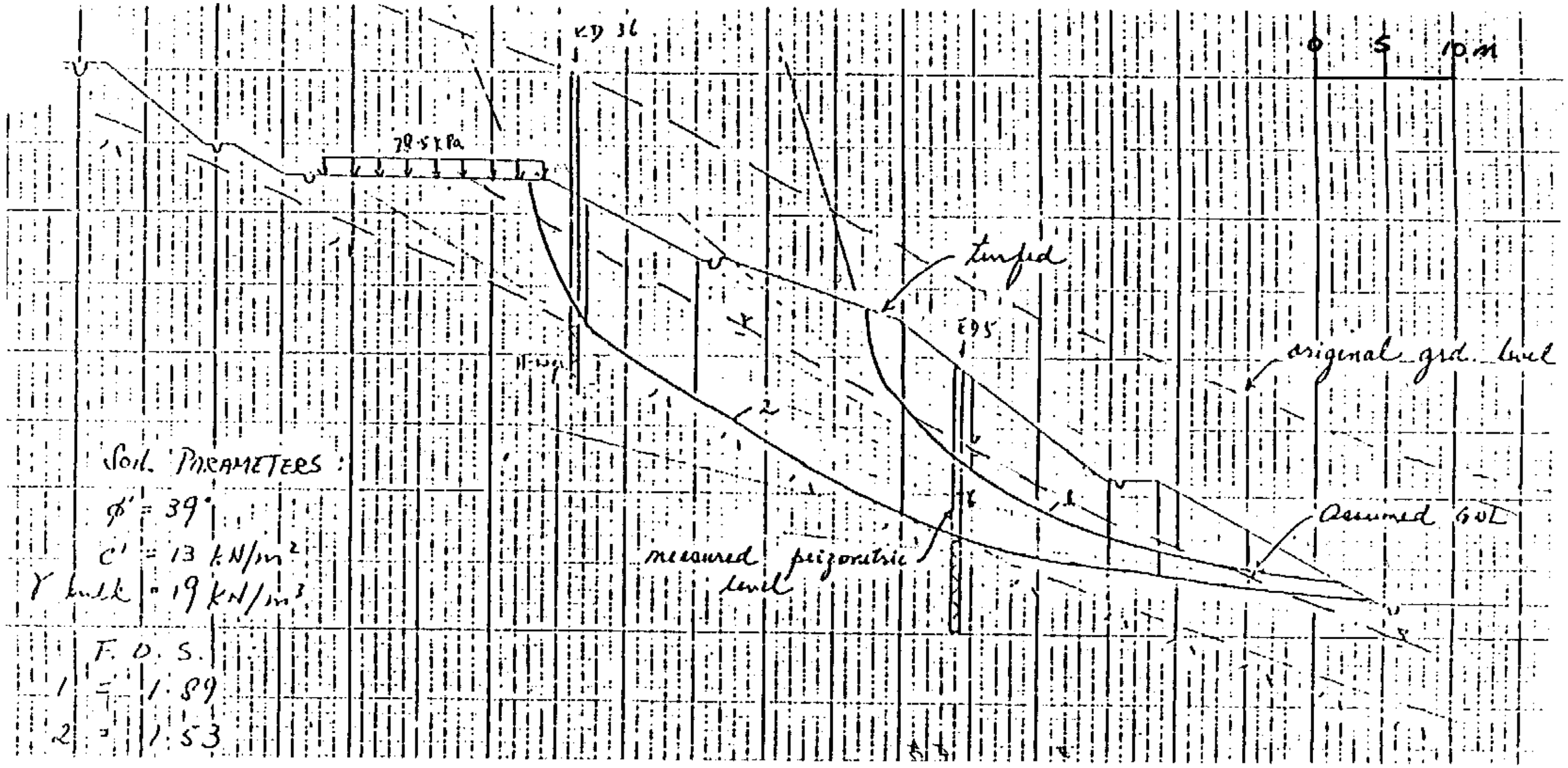
Year	Document Title	By Whom	To Whom	Remarks
1984	Sha Tin New Town, Contract No. 626/80, Slope Remedial Works in Areas 16B, 41A, 42, 43 and 46 (Correspondence)	GCO	NTDD	GCO's comments on MCA's reply. A recent inspection of Slope G "revealed that there was a slip in Slope G [Figure 3] ....need to substantiate that all fill has been removed from this slope, with records showing the determination of in-situ/fill boundary". GCO went on to say that, "provided all fill has been removed, ...no further requirements for long term monitoring in view of MCA's comments".
1984	Sha Tin New Town, Contract No. 626/80, Formation, Roads & Drains & Slope Remedial Works in Areas 41A, 42, 43, 46 & 16B (Correspondence)	MCA	NTDD	In response to GCO above, MCA stated that, "The slip referred to ....is surface erosion and is located about 15 m away from the boundary of Slope G. The eroded area is made up of very dense in-situ CDG where grass was not growing well. We do not consider that this surface erosion should be correlated with remedial work in Slope G and we insist that all loose fill in that slope has been removed". The location of the slip, as shown on photographs, indicates that it is outside the area of compacted fill as shown on MCA's drawings.
1984	Sha Tin New Town, Contract No. 626/80, Formation, Roads & Drains & Slope Remedial Works in Areas 41A, 42, 43, 46 & 16B (Correspondence)	GCO	NTDD	In response to MCA's comments above, the GCO "noted and accepted".

Table A1 – Summary of Site History Documents (Sheet 12 of 12)

Year	Document Title	By Whom	To Whom	Remarks
1985	Sha Tin New Town, Contract No. 626/80, Formation, Roads & Drains & Slope Remedial Works in Areas 41A, 42, 43, 46 & 16B, Handover of Slope Remedial Work (Correspondence)	GCO	CEH/NT	With reference to handover of slopes in Area 41A, the GCO stated that, "The design of all slopes has been accepted by this office". Furthermore the GCO stated that, "Proposed remedial works have been accepted on Slope A to J inclusive". Slope No. 7SW-B/C123 comprising Slopes E, F and G.

APPENDIX B

STABILITY ASSESSMENT CARRIED OUT FOR SLOPE DESIGN IN 1979



Note : For location of cross-section X - X refer to Figure 3.

Figure B1 - Stability Cross-section X - X Assessed by MCA