SECTION 2 : DETAILED STUDY OF THE LANDSLIDE AT FUNG WONG SERVICE RESERVOIR ON 9 JUNE 1998

Fugro Scott Wilson Joint Venture

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FOREWORD

This report presents the findings of a detailed study of the landslide (GEO Incident No. K 98/6/13) that occurred on the morning of 9 June 1998, at the southwestern corner of Slope No. 11NE-A/C21, adjoining Fung Wong Service Reservoir, to the west of Shatin Pass Road. Debris from the landslide blocked the 3 m-wide access track between the toe of the slope and the edge of the reservoir, and some debris spilled over into the reservoir. There were no casualties as a result of 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. It is one of a series of reports produced during the consultancy by Fugro Scott Wilson Joint Venture (FSW). The report was written by Mr M Hughes and reviewed by Mr Y C Koo. The assistance of the GEO in the preparation of the report is gratefully acknowledged.

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1. INTRODUCTION

On the morning of 9 June 1998, a landslide (GEO Incident No. K 98/6/13) occurred at the southwestern corner of Slope No. 11NE-A/C21, adjoining Fung Wong Service Reservoir, to the west of Shatin Pass Road (Figure 1 and Plate 1). Debris from the landslide blocked the 3 m-wide access track between the toe of the slope and the edge of the reservoir, and some debris spilled over into the uncovered reservoir, which is below ground level. No serious damage was caused by the landslide, but a section of the reservoir was drained following the incident, disrupting its operation. There were no casualties as a result of the landslide.

Following the landslide, Fugro Scott Wilson Joint Venture (FSW), the 1998 Landslide Investigation Consultants, commenced a study of the failure 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 FSW. Recommendations for follow-up actions are reported separately.

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 inspections, desk study and analysis.

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

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

(b) aerial photograph interpretation (API),

(c) analysis of rainfall records,

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

(e) engineering analyses of the failed slope, and

(f) diagnosis of the probable causes of failure.

2. THE SITE

2.1 Site Description

The location of the landslide is shown in Figure 1. The ground that failed comprises a cut slope and part of the natural terrain above that lie within the Fung Wong Reservoir compound.

The cut slope was registered as No. 11NE-A/C21 in May 1977 by the consultants appointed by the Government to prepare the 1977/78 Catalogue of Slopes. It comprises three distinct sections, namely, the northern, western and southern sections that lie approximately perpendicular to each other (Figure 1). The northern portion has been cut into sidelong ground that dips in a southerly direction at an angle of about 30°, while the southern portion has been cut along an approximately east-to-west trending ridgeline. The ridgeline and the terrain to the north bound a valley that has been excavated to form the western slope portion.

The northern slope is about 24 m-high and is divided into three separate 8 m-high batters by 1 m-wide berms, at elevations of about 192 mPD and 200 mPD. The lowest batter is cut into slightly to moderately decomposed granite (S-MDG) and stands at an angle of about 75° to the horizontal. The middle and upper batters both stand at an angle of about 60° to the horizontal and desk study indicates that they are cut into completely and completely to highly decomposed granite (C-HDG) underlying bouldery colluvium.

The western slope is about 19 m-high and is divided into two batters by a 1 m-wide berm that follows on from the upper berm of the adjoining northern slope, i.e. at an elevation of about 200 mPD. The lower batter is about 16 m-high and stands at an angle of about 70° to the horizontal, while the upper batter is about 3 m-high and stands at an angle of 40° to the horizontal. The junction between the lower and upper cut slopes is thought to represent the boundary between S-MDG and colluvium respectively.

The southern slope has a maximum height of about 33 m and is generally divided into three separate batters by 1 m-wide berms at elevations of about 200 mPD and 208 mPD (Figures 2 and 3). The lowest batter is about 15 m-high and is cut into S-MDG, while the height of the upper batter varies from a minimum of 3 m to a maximum of about 10 m and is cut into C-HDG. The intermediate batter is 8 m-high and marks the transition between soil and rock. The lowest batter stands at an angle of about 75° to the horizontal, while the middle and upper batters both stand at an angle of about 60° to the horizontal.

The profile of the uppermost batter of the southern slope changes over a length of about 15 m at the extreme western end of the southern cut slope. This is the location where the current landslide occurred. In this area there is a local depression that lies at a shallower angle than the majority of the southern cut slope, such that part of the uppermost batter is thought to have stood at an angle of between 40° and 50° to the horizontal prior to failure.

The majority of all portions of the cut slope are covered by surface protection, with shotcrete covering much of the lower and middle batters of the southern slope and all of the northern and western slopes. Chunam surface protection, is apparent over the upper batter of the southern slope. Localised outcrops of moderately and moderately to slightly decomposed rock are evident through the surface protection. Other forms of slope treatment work comprising soil nails, rock bolts and localised buttressing works are evident along the northern and western slope portions. A network of surface drainage channels has been installed along the intermediate berms and along the toe and portions of the crest of the slope.

According to the "Systematic Identification of Maintenance Responsibility of Slopes in the Territory" (SIMAR), the maintenance responsibility for Slope No. 11NE-A/C21 lies with the Water Supplies Department (WSD).

2.2 Water-carrying Services

Private and Government utility owners were contacted for details of their installations. This revealed that no water-carrying services are present in the vicinity of the landslide.

2.3 Site History

2.3.1 General

The site history has been established from aerial photograph interpretation (API) together with a review of desk study information. A record of the photographs examined and a plan (Figure A1) summarising the findings of the API are presented in Appendix A.

2.3.2 History of Development

The earliest available photographs, taken in 1949, show that Shatin Pass Road has been formed, but that the area forming the reservoir site comprises natural ground. The area along the ridgeline at the south-western corner of the reservoir, where the current landslide occurred, appears to be an existing natural depression. This may mark an area of relict instability as the depression appears concave and the valley floor is hummocky in nature suggesting the presence of colluvium.

The 1963 photographs show that the reservoir is under construction. The cut slope, 11NE-A/C21, has been formed and the layout is consistent with that which was observed during recent site visits, showing that no major alterations have been made since. However, the platform area present along the ridgeline above the landslide is not apparent in these photographs. A large area of colluvium is evident immediately to the north of the reservoir, above the northern cut slope. Also, bouldery colluvial deposits are evident in the valley floor above the western cut slope.

The southern portion of the cut slope is clearly visible in the 1963 photographs, and it can be seen that at the extreme western end of the slope, where the current landslide occurred, the uppermost batter cuts into the concave depression noted in the 1949 photographs. A diagonal line, which appears to be a continuation of the footpath descending from the ridgeline, can be seen cutting across the uppermost batter. The portion of the batter below this diagonal line appears to stand at a steeper angle than the portion above the line. Furthermore, the area below the line appears slightly lighter in colour than the area above. This gives the overall impression that the section below the line has been cut to a slope angle similar to the adjoining area to the east (i.e. 60^o). While the section above the line has been cleared of vegetation, but not cut back significantly, such that it stands at or close to its natural slope angle of about 30^o to 40^o.

Where the southern portion of the cut slope intersects the footpath descending from the ridgeline a new section of footpath can be seen arcing across the valley floor between the southern and northern cut slopes. This appears to be at approximately the same elevation as the crest of the uppermost batter in the south-western corner. A dark line can be seen extending from the back of the crest of the uppermost batter slope along the footpath. This line has been confirmed on site to be a surface drainage channel linking into the main drainage channel along the northern boundary of the reservoir. An area characterised by a light tone can be seen below the southern end of the footpath and drainage channel, which probably represents minor spoil associated with the formation of these features.

No signs of instability were observed in the vicinity of the current landslide from the 1963 photographs.

The next set of photographs, taken in 1967, show numerous darker areas along the western cut slope and at the western corner of the northern cut slope, which appear to be indications of surface water/seepage flows, but no indications of surface water/seepage flows are evident along the southern slope. No other significant changes are evident in these photographs.

Further indications of instability are evident in the 1973 photographs. A small wedge type failure can be seen in the uppermost batter immediately to the east of the south-western corner and immediately adjacent to the location of the current failure. It is apparent from recent site visits that this feature has been treated by what appears to be concrete dentition/buttressing.

Between the 1973 and the 1990 photographs, there is no significant change in appearance of the southern slope in the vicinity of the current landslide. The footpath along the ridgeline appears to have widened appreciably in certain areas, possibly due to erosion, and in 1981 and 1982 the section of footpath directly above the landslide site has been widened to form a long narrow rectangular area. This area is interpreted as man-made due to the uniform profile and shape, which corresponds to the platform area identified on site during the current study. Also, an access ladder can be seen on the southwestern cut slope in the 1981 photographs.

In the 1976 photographs, light coloured patches below the crest of the uppermost batter of the southern slope can be seen. These may be an indication of surface erosion due to overtopping of the drainage channel at the slope crest. Observations made on site show that the crest channel is blocked and that the footpath descending from the ridgeline probably provides a preferential flow path directing surface water runoff to the slope crest. This supports the notion of surface water overtopping the crest channel and causing erosion.

A further observation from the 1976 photographs is that in the south-western corner of the reservoir the upper portion of the uppermost batter appears to be vegetated, whereas the lower portion is smooth as if covered by surface protection. Subsequent photographs also indicate that this area is vegetated, although it is difficult to be conclusive due to shadow effects in this area.

In the 1990 photographs, a light coloured diamond-shaped area is evident on the upper soil slope at the junction of the southern and western slopes. This area may indicate a recent landslide or erosion feature, or repair thereof.

There is no significant change in appearance of the south-western corner of the reservoir in subsequent photographs.

2.4 Previous Inspections and Studies

In May 1977, the consultants appointed by Government to prepare the 1977/78 Catalogue of Slopes, inspected the cut slope. In the field sheet prepared for the feature following the inspection (Binnie and Partners, 1977), the slope was registered as No. 11NE-A/C21. There were no apparent signs of seepage or distress recorded on the field sheet and no previous incidents of instability were noted.

A photograph of the site attached to the field sheet shows that all of the mid batter and part of the lower batter of the southern slope were covered by surface protection, interpreted as chunam. The majority of the upper batter is also protected, but a small area at the top of the batter in the immediate vicinity of the current landslide appears to be vegetated. An area of new surface protection is apparent over approximately a 20 m-long section of the middle batter at the extreme western end of the southern slope (Figure 2). No records were found to indicate why this area had been provided with new surface protection.

In November 1986, the Planning Division of the Geotechnical Control Office (GCO) completed a Stage 1 Study of the cut slope (GCO, 1986a). The report presents the findings of a stability assessment comprising a detailed field inspection, desk study and geotechnical appraisal based on the available information, but no ground investigation was carried out. Field inspections at the time revealed that the lower rock slopes on the southern and western sections had recently been treated with shotcrete, but that the northern rock slope still remained unprotected. The remaining batters, which are predominantly cut in soils, were all covered by chunam surface protection. Desk study records from the report show that in June 1986 a failure occurred in the natural terrain at the extreme eastern end of the northern slope. The GEO Incident Report (K86/6/28) recorded the landslide as involving about 50 m^3 of material, which blocked the footpath at the toe of the slope. No records of instability in the vicinity of the current failure were identified in the Stage 1 Study Report.

Preliminary stability assessment was undertaken using a variety of techniques, including empirical analysis using CHASE graphs (GCO, 1982), slope stability analysis by Janbu's simplified method and kinematic analysis. The empirical method of analysis was used to assess the stability of the upper soil faces of the southern cut slope. The height versus slope angle plot for granite slopes indicated that the soil slope fell outside of the "stable zone", and on the height versus discriminate function plots the results show that the slope lay in the "moderate risk zone" of general and face instability.

The southern soil slope was also assessed by Janbu's simplified method assuming dry conditions, as was the northern slope. A sensitivity analysis was undertaken using a range of shear strength parameters. This revealed that strength parameters of $c' = 14$ kPa and $\phi' = 42^o$, for H-CDG were required to achieve an acceptable factor of safety (FOS), i.e. >1.2, for the southern slope.

Rock slope stability was assessed using kinematic analysis. The northern slope and central portion of the southern slope were assessed as having few discontinuities, giving a massive appearance, and towards the eastern end of the southern slope the discontinuities were generally widely spaced creating large blocks. These sections were considered to be stable. The western slope and the western end of the southern slope were noted as having medium spaced discontinuities. A small number of dips were measured and plotted on a stereonet. This showed that there was a potential for planar sliding failure as well as wedge failure.

The report concluded that for the soil slopes along the southern section the strength parameters required to achieve an acceptable FOS, as identified by the simplified Janbu analysis, can reasonably be expected from the slope-forming materials of H-CDG. However, these conclusions did not appear to have taken into consideration the presence of relict discontinuities in the slope.

Regarding the rock slopes, it was concluded that the pattern of discontinuities along the northern and southern slopes was generally favourable to stability and major failures were not foreseen. However, for the western rock slope the discontinuity pattern was not considered favourable to stability. Accordingly, the report recommended a Stage 2 Study for the northern and western slopes, but that the southern slope did not require further investigation.

The cut slope was included in the 1988/89 Landslip Preventive Measures (LPM) Programme, and in May 1989 the Design Division of GEO (formerly GCO) completed a Stage 3 Study (GCO, 1989). Given the recommendations made in the Stage 1 Study Report, no further consideration was given to the southern slope, such that only the northern and western slopes were considered in the Stage 3 Study. This report concluded that the stability of both slope sections was below the current standard and that upgrading works were required. Soil nailing in conjunction with local trimming work was specified for the northern slope, while stabilisation works for the western slope would include raking drains, rock bolts/dowels and removal of loose and overhanging sections, the details of which were to be determined on site. The upgradinng works recommended by the Stage 3 Study Report were completed under contract GC/88/06 in January 1991.

An API was undertaken by the Planning Division of the GEO as part of the Stage 3 Study. The report identified isolated incidents of instability along the northern slope, but no incidents of instability were noted in the southern slope.

A Stage 1 Study Report by the Design Division, dated 11 June 1991, noted no signs of distress for the entire feature, and recorded that LPM works had recently been completed.

In mid-1992, the GEO initiated a consultancy agreement, entitled "Systematic Inspection of Features in the Territory" (SIFT), to search systematically for features not included in the 1977/78 Catalogue of Slopes and to update information on previously registered features, by limited site inspections and studying aerial photographs. The cut slope was categorised as a Class C1 feature under the SIFT project, i.e. "Assumed formed pre-1978 or illegally formed", and allocated Feature No. 11NE-A/C21.

In July 1994, the GEO commenced a 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 Slope Catalogue. The GEO's consultants for the SIRST project inspected the feature in February 1998. The SIRST Report identified the slope to be in a fair condition with slight to moderate signs of seepage and no signs of distress. It also recorded "Inferred Past Instability" as being "none".

In July 1995, Slope No. 11NE-A/C21 was inspected by consultants Fugro, Mouchel and Rendel (FMR), appointed by the Highways Department (HyD) for a project entitled "Roadside Slope Inventory and Inspections" to carry out Engineer Inspections of about 4,000 HyD slopes. No signs of distress were observed, but it was noted that “Routine maintenance not carried out satisfactorily because surface debris not removed”. Due to the “very steep” nature of Slope No. 11NE/A/C21, recommendations for a stability assessment were made.

3. DESCRIPTION OF THE LANDSLIDE

3.1 Time of the Landslide

The first recorded sighting of the landslide was at about 9:00 a.m. on 9 June 1998, by workers from WSD’s term contractor, Union Contractor, as they arrived on site to their temporary offices that morning. No eyewitness accounts of the failure were identified.

Based on the above and the rainfall data at the time, a best estimate is that the failure occurred sometime between 5:00 a.m. and 9:00 a.m. that morning when the storm was at its highest intensity.

3.2 Description of the Landslide

A detailed plan and cross-section of the landslide are shown in Figures 2 and 3 respectively. Photographs of the landslide are presented in Plates 1 to 12.

The landslide of 9 June 1998 occurred at the westerly end of the southern portion of Slope No. 11NE-A/C21 and comprised two morphological units (Figure 2):

(a) a wedge-shaped surface of rupture, partially controlled by a relict discontinuity with an adverse dip and orientation, and

(b) a fan-shaped debris trail covering the lower cut slopes and the access track at the toe of the feature.

The estimated failure volume based on the size of the failure scar was about 120 m^3.

The landslide produced a wedge-shaped scar of about 19 m in length and 12 m in width, with a maximum depth of about 2.5 m (Plates 1 to 3). Due to the height of the failed section (between 16 m and 30 m above ground level), a detailed inspection of the failure scar could not be undertaken until a scaffold access had been erected. This was not completed until September 1998 and therefore objective observations regarding seepage and water flows could not be made shortly after the failure. However, no soil pipes were observed upon inspection of the back scar and there were no obvious signs of significant vegetation disturbance or erosion.

Detailed inspection of the rupture surface revealed that the eastern side of the wedge failure was formed by a persistent, essentially planar surface believed to be a relict discontinuity (Plates 3 to 5). This plane had a slightly convex profile with an overall dip in a northerly direction of between 50° and 55° to the horizontal. The material forming this face comprised completely decomposed granite, with evidence of kaolin over portions of the failure surface (Plates 5 and 6).

A more undulating plane of variable dip involving both concave and convex breaks in profile formed the western side of the landslide (Plate 3). This plane comprised a steeper upper portion, standing at about 60° to the horizontal, that graded into a larger and more dominant shallower lower portion, standing at about 45° to the horizontal. Both surfaces dipped in an east-north-easterly direction and also comprised completely decomposed granite. The surface of the steeper upper portion was not as smooth as that exposed on the eastern plane, giving the impression that this portion of the landslide may have failed through intact material. The shallower lower portion also appeared as if it had failed through intact material, but much of this portion was covered by landslide debris making it difficult to assess accurately whether this part of the landslide had failed along a relict discontinuity or through intact material.

At the interface between the planes it was noticeable that a clayey seam had developed. This seam had the same dip and orientation as the easterly failure plane and was 10 mm to 20 mm thick (Plate 7).

A sub-vertical face trending roughly parallel with the crest channel formed the crown of the landslide (Plate 8 and Figure 2). This plane had a maximum depth of about 3 m and was roughly triangular in elevation.

Debris from the landslide formed a fan-shaped deposit at the toe and over the lower portion of the cut slope, blocking the 3 m-wide access track bounding the perimeter of the reservoir. Some material had spilled over into the reservoir itself, and it is probable that more material would have entered the reservoir, but for the presence of a perimeter fence around the reservoir, which trapped much of the debris. Inspection of the material showed that it comprised a loose silty sand with fine quartz gravel fragments, that was dry at the time of inspection. The travel angle of the landslide debris (Wong & Ho, 1996a), measured from the crown of the landslide to the fence at the toe is about 46°, but this does not reflect the actual mobility of the debris because of the influence of the fence. The surface of the main debris lobe towards the base of the slope was relatively smooth, with no abrupt changes in profile, suggesting that the landslide was a "single phase" event that probably occurred rapidly.

Inspection of the area immediately adjacent to the landslide showed that the lower portion of the upper batter stood at a steeper angle than the upper portion (Plate 4), giving the impression that the lower portion had been cut, with the upper portion at or near its natural angle. Also, along the crest channel at the crown of the landslide, it was apparent that the channel intersected a narrow track approximately 1 m-wide cut into the slope (Figure 2 and Plates 8 and 10). Although this feature is difficult to identify on the photographs, it was clearly visible on site. The line of this track joins up with an existing footpath higher up the slope (Plate 9), suggesting that prior to formation of the cut slope the two were connected and formed a continuous footpath down the side of the ridgeline. These findings corroborate the observations from the API (Section 2.3.2).

Chunam surface protection covered intact portions of the western side of the upper batter adjacent to the landslide, but a vegetated slope surface adjoined the eastern side.

Chunam also covered areas adjoining the landslide in the middle batter. The chunam layer was in a fair condition, but was quite thin being about 15 mm thick.

The crest channel directly above the crown of the landslide, which starts at approximately the upper eastern corner of the rupture surface, was completely blocked at a distance of about 12 m from its starting point (Plates 10 and 11). The debris blocking the 300 mm-wide channel appeared to be slope wash material transported from the higher ground above. Further investigation showed that the channel connected into the main drainage line above the northern boundary of the reservoir, but that a considerable section (approximately 20 m to 30 m) had become completely filled with debris. Consequently, surface water flowing into this section of the channel could not drain away along its intended route. Instead, it would overtop the channel once sufficient water had entered this blocked section of surface drainage.

It was also noted that an area of open ground has been formed along the ridgeline directly above the landslide, which based on API was formed some time between 1981 and 1982 (Figure 2 and Plate 12). This area has been levelled to form a platform about 30 m long by 5 m wide and is unprotected, being open to infiltration and could provide a source for rainwater runoff.

4. SUBSURFACE CONDITIONS

4.1 Geology and Geomorphology

The area surrounding the landslide site is dominated by a sequence of prominent ridgelines trending in a generally northwest to southeast direction creating a series of parallel valleys. Natural drainage lines are evident along the floor of the valleys, which appear to be covered with bouldery colluvium.

When looking at the valley in which the reservoir was formed, it can be seen that a number of failure scars are evident along the ridgeline that forms the southern slope (Figure 4). In particular, a concave depression is evident at the southwestern corner of the reservoir where the current landslide occurred. Aerial photographs taken prior to formation of the reservoir show that a lobe of material is apparent in the drainage line at the toe of the depression, suggesting that this feature marks the scar of a past landslide. Larger fans of what appear to be colluvium are also evident to the northwest of the reservoir.

The published geological map (GCO, 1986b) indicates that the geology of the area comprises fine and medium-grained granite. Large fans of debris flow deposits are shown to exist immediately to the northwest and to the east of the reservoir.

The geological map shows a photogeological lineament crossing the landslide site in approximately an east-west direction along the same line as the northern cut slope. This appears to be a fault bisecting the area into a southerly portion of medium-grained granite and a northerly portion of fine-grained granite.

The geological map also shows a 400 m-long intrusion of fine-grained granite, approximately 100 m, to the southwest of the landslide site, trending in a northeast to southwest direction.

4.2 Previous Ground Investigations

No ground investigation records were recovered for the area where the landslide occurred, but investigation works were undertaken above the northern portion of the cut slope as part of the Stage 3 Study under the 1988/89 LPM Programme. These works comprised four drillholes, five chunam strips and two trial pits. These identified a layer of bouldery colluvium of maximum thickness 13 m overlying fine to medium and coarse-grained granite of varying degrees of decomposition. A more detailed description of the materials encountered on site is provided in the following section.

4.3 Current Investigation

Detailed field mapping carried out following the landslide showed that the eastern face of the failure was formed by a persistent planar surface mapped as extremely to very weak, reddish pink, speckled black, completely decomposed medium-grained granite (CDG) with kaolin and manganese veins. The plane covers an area approximately 3 m to 4 m wide and 15 m long and is believed to be a relict discontinuity, portions of which had a thin (1 mm to 2 mm) infilling of kaolin exposed (Plates 5 and 6), giving the surface of the discontinuity a smooth appearance. Also present over a small section of the relict discontinuity towards the crest of the failed section was a thin layer (less than 5 mm thick) of clayey material, possibly infill (Plate 6). A residual soil layer approximately 1 m thick was evident along the eastern edge of the failure overlying the CDG.

The western face of the wedge failure comprised extremely to very weak, pinkish white and orangish brown, speckled black, completely decomposed medium-grained granite (CDG) with manganese and iron-stained veins. The western face had an undulating profile, that gave the appearance that this side of the wedge failure had possibly sheared through intact material rather than along a relict discontinuity.

At the interface between these two planes a zone of more decomposed material, comprising a brown clayey silty sand, typically 10 mm to 20 mm thick, was present. This seam is evidence of preferential decomposition along the relict discontinuity, probably caused by infiltrating surface water.

Mapping of the remaining portion of the southern cut slope revealed discontinuity surfaces dipping in a similar direction and at a similar angle to that of the relict discontinuity exposed in the failure scar (Figure 5 and Table 1). These discontinuities occurred at regular intervals along the southern slope portion in both soil and rock slope portions (Plates 13 and 14), and were typically widely to very widely spaced. One such discontinuity forms a prominent exposure in the lower batter (Figure 5). It is thought that these discontinuities probably represent sheeting joints, possibly formed following erosion of surface material over the years. Along the soil slope portion of the southern cut slope the surface of the discontinuities inspected were found to be infilled with kaolin, which in places was about 2 mm to 3 mm thick. Furthermore, there was evidence of slickensiding along one of these discontinuities that dipped steeply into the slope in a southwesterly direction (Plate 15).

4.4 Deduced Ground Conditions

Based on the available information the dominant material involved in the landslide was CDG, which has been shown to have a network of kaolin and manganese veins within the fabric of the material. There was also evidence in the form of other infill material along the relict discontinuity (eastern failure plane) to suggest that there may have been previous movements towards the crown of the failed section of the slope leading to a slight opening across the discontinuity that has since become infilled. Alternatively, this infill may be further evidence of preferential weathering along the relict discontinuity.

Inspection of the slope immediately adjacent to the landslide indicates that the relict discontinuity forms part of a system of medium to widely-spaced joints present along the southern slope portion. Many of these discontinuities are infilled with kaolin and to a lesser extent manganese. In-situ weathering and/or precipitation of kaolin and manganese/iron oxide minerals originating from overlying deposits may also be responsible for these deposits (Irfan & Woods, 1988). There is also isolated evidence in the form of slickensides dipping steeply into the slope in a southwesterly direction to indicate past movement along some of these kaolin-infilled surfaces, although no slickensiding was noted over the kaolin-infilled discontinuity forming the eastern failure plane.

4.5 Groundwater Conditions

The topography of the area surrounding the landslide site is such that the catchment serving this area is relatively small, and it is not considered to be large enough to sustain a high groundwater table. Accordingly, even following the heavy rains during the storm of 9 June 1998, it is considered that the main groundwater table remained well below the floor of the landslide.

It is thought that the setting is favourable to a build-up of transient water pressures in the soil mass, particularly along relict discontinuities, as a result of direct infiltration and surface water flows over the portion of unprotected natural terrain above the landslide area, as well as over the failed area itself, further promoting infiltration.

5. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge is No. K07, which is located at Oi Wai House, Tse Oi Estate, some 800 m west of the landslide. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone link to the GEO. These records have been analysed to determine the characteristics of the rainstorm associated with the time of the landslide.

The daily rainfalls recorded by the raingauge in May and June 1998, together with the hourly rainfalls from 8 to 9 June 1998, are shown in Figure 6. The storm was concentrated around the day of 9 June 1998, with particularly heavy rainfall experienced in the morning between 05:30 hrs and 09:30 hrs.

Isohyets of rainfall for the 24-hour period prior to the first sighting of the landslide (09:00 hrs on 9 June 1998) for the whole of the SAR are shown on Figure 7.

Table 2 shows the maximum rainfall intensities and their estimated return periods for different durations for the main storm of 9 June 1998. The maximum 2 hour rainfall of 137.5 mm was the most severe, which corresponded to a return period of about 8 years (Lam & Leung, 1994).

A comparison of the patterns of previous heavy rainstorms affecting the landslide site is shown in Figure 8. Although only slightly greater than previous events, it is evident that the rolling rainfall for a period up to 4 hours preceding the current landslide was the most severe of that experienced by the area since installation of raingauge No. K07 in 1983. However, given that the slope was formed before 1963, the 9 June 1998 rainstorm, which was only moderately heavy, may not be the most intense in the history of the slope.

Whilst return periods for various durations within the storm have been assessed based on the method given by Lam & Leung (1994), it is recognised that this method does not necessarily give the true return period for a particular site (Wong & Ho, 1996b). However, it does provide an objective ranking of the relative severity of the different rainfall characteristics and different rainstorms.

6. ANALYSIS OF SLOPE STABILITY

Theoretical stability analyses were undertaken to assist in the diagnosis of the probable causes and triggering factors of the landslide. These analyses were used to investigate the effects of increasing pore water pressure on the factor of safety (FOS) of the slope assuming a range of likely mass shear strength parameters.

Due to the wedge type nature of the landslide, the failure has been analysed assuming a sliding mechanism, as outlined in Hoek & Bray (1981). In the analysis an increase in water pressure along the eastern failure plane was modelled to simulate increased infiltration along the relict discontinuity, but no pore water pressure was assumed to act along the western failure plane, which is believed to have failed through in-situ material.

A range of different shear strength parameters was assumed for the eastern (discontinuity) and western (intact) failure surfaces, namely, $c' = 0$ kPa and $\phi' = 30^o$ to 35^o for the eastern plane and $c' = 7$ kPa to 11 kPa and $\phi' = 40^o$ for the western failure plane respectively. These values are based on generally accepted ranges in Hong Kong (e.g. Siu & Premchitt (1988) and Geoguide 1 (GEO, 1993)) and are considered to reflect the likely range of shear strength parameters corresponding to the nature of materials mapped on site.

The results of the analysis are summarised in Figure 9. These show that the wedge is stable in dry conditions for the range of shear strength parameters adopted. However, as water pressure builds-up along the relict discontinuity the factor of safety falls below unity.

Kinematic analyses were also undertaken for the range of joint measurements taken along the southern slope (Figure 5 and Table 1). These measurements are plotted on an equatorial equal-area stereonet. This showed that even given a high angle of friction ($\phi' = 45^o$) along the joint planes there would still be the potential for plane and wedge failures (Figure 10).

7. PROBABLE CAUSES OF THE LANDSLIDE

The occurrence of the landslide during the rainstorm of 9 June 1998 indicates that the landslide was most probably triggered by rainfall. The geometry of the landslide suggests that it involved a shallow wedge-shaped failure in completely decomposed granite that was partially controlled by a persistent relict discontinuity with relatively weak infill material.

Information gathered during site visits and from the desk study phase of the investigation suggests that a number of contributory factors probably combined to cause the landslide, including :

(a) the presence of a flat area formed along the ridgeline above the landslide allowing possible ponding and generating significant runoff onto the landslide site during heavy rainfall that was not controlled by an appropriate drainage system,

(b) the presence of an unprotected area of natural terrain below the flat area of ground along the ridgeline and directly above the crown of the landslide, which allows ingress of water during rainfall,

(c) the presence of adverse, partly kaolin and manganese infilled, relict joints in the soil mass, and

(d) the build-up of pore pressure along the relict joints.

It is postulated that during the rainstorm of 9 June 1998, direct infiltration of surface water together with sub-surface seepage from infiltration further upslope led to a preferential increase in transient water pressures along the relict joints identified in the soil mass. The flat area along the ridgeline provided an area for direct infiltration as well as creating the potential for runoff. This would have led to increased water flows down the unprotected portion of the natural terrain as well as along the face of the cut slope due to the blocked channel along the crest of the cut slope, thereby exacerbating infiltration.

The eastern failure plane was dominated by an adversely orientated, persistent and planar discontinuity that in places was covered by a thin layer of kaolin. The failure scar also exposed clayey material, typically 10 mm to 20 mm thick, along the discontinuity, indicating that infiltration and weathering were ongoing along this plane. The shear strength along this persistent relict discontinuity would have been less than the mass shear strength of the CDG material, thereby forming a structural weakness within the soil cut slope. This adverse geological feature appears to have been a significant factor in the failure, both in terms of relatively low shear strength and being favourable to a build-up of pore water pressure along the joint.

A further contributory factor to the failure is inadequate slope maintenance, as evidenced by the completely blocked drainage channel at the crest of the failure.

8. DISCUSSION

It is noted that the presence of relict structure and the possibility of a build-up of water pressure within the CDG were not considered in the detailed Stage 1 Study undertaken by the Planning Division of the GEO (GCO, 1986a). This study concluded that for the soil sections of the southern slope, "The soil strength parameters required to achieve an acceptable factor of safety can reasonably be expected from the slope forming materials of highly to completely weathered granite". The required strength parameters, arrived at by back analysis assuming dry conditions, were $c' = 14$ kPa and $\phi' = 42^{o}$.

Additionally, it is apparent from low level aerial photographs that there has been a clustering of past failures in the immediate vicinity of the present landslide site. This may be an indication of locally weak terrain and/or unfavourable groundwater conditions. The past performance of the slope, the site setting (which is favourable to water ingress into the landslide site) and the structural geology (in particular the adverse geological features) do not appear to have been considered in detail in the previous Stage 1 Study.

9. CONCLUSIONS

It is concluded that the landslide on 9 June 1998 was triggered by moderately heavy rainfall with a return period of about 8 years.

The landslide was partially controlled by the presence of a persistent relict discontinuity over the eastern portion of the failure. The failure was probably caused by the build-up of transient water pressures in the soil mass, particularly along relict discontinuities. It occurred at a location that has a history of minor instabilities.

Inadequate slope maintenance is also a contributory factor to the landslide.

The landslide occurred at the southern section of slope No. 11NE-A/C21, which was subject to a stability assessment and was subsequently excluded from upgrading works carried out for the remaining northern and western portions of the slope.

10. REFERENCES

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

Siu, K.L. & Premchitt, J. (1988). GCO Internal Report. Landslide Studies 1987 : Cho Yiu Estate. SPR 3/88, Geotechnical Control Office, Hong Kong, 62p

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

Wong , H.N. & Ho, K.K.S. (1996b) Thoughts on the Assessment and Interpretation of Return Periods of Rainfall. Discussion Note DN 2/96, Geotechnical Engineering Office, Hong Kong, 19 p

LIST OF TABLES

Table 1 - Summary of Joints Measured Along the Southern Cut Slope

Joint Set Reference	Dip Direction/Dip Angle (º)				
	1	2	3	4	Average
J_A	352/53	350/55	355/60	355/50	353/55
J_B	010/55	005/50	015/43	010/50	010/50
J_C	030/50	020/45	-	-	025/48
J_D	008/60	015/65	-	-	012/63
J_E	020/66	010/74	-	-	015/70
J_F	075/20	065/25	-	-	070/23
J_G	053/70	054/70	-	-	054/70
J_H	250/78	260/72	-	-	255/75
J_I	335/65	335/70	-	-	335/68
J_J	290/84	294/82	-	-	292/83
J_K	070/90	070/80	-	-	070/85
Note: For location of joint sets see Figure 5					

Table 2 - Maximum Rolling Rainfall Records at GEO Raingauge No. K07 for Selected Durations Preceding 9 June 1998 Landslide and the Corresponding Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	12	05:50 hours on 9 June 98	2
15 minutes	31	05:55 hours on 9 June 98	4
1 hour	83.5	06:40 hours on 9 June 98	4
2 hours	137.5	06:55 hours on 9 June 98	8
4 hours	166	08:45 hours on 9 June 98	6
12 hours	223.5	09:00 hours on 9 June 98	4
24 hours	238.5	09:00 hours on 9 June 98	3
48 hours	245	09:00 hours on 9 June 98	< 2
4 days	284	09:00 hours on 9 June 98	< 2
7 days	349	09:00 hours on 9 June 98	< 2
15 days	448	09:00 hours on 9 June 98	< 2
31 days	705.5	09:00 hours on 9 June 98	3

Notes : (1) Return periods were derived from Table 3 of Lam and Leung (1994).

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

(3) The use of 5-minute rainfall 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.

LIST OF FIGURES

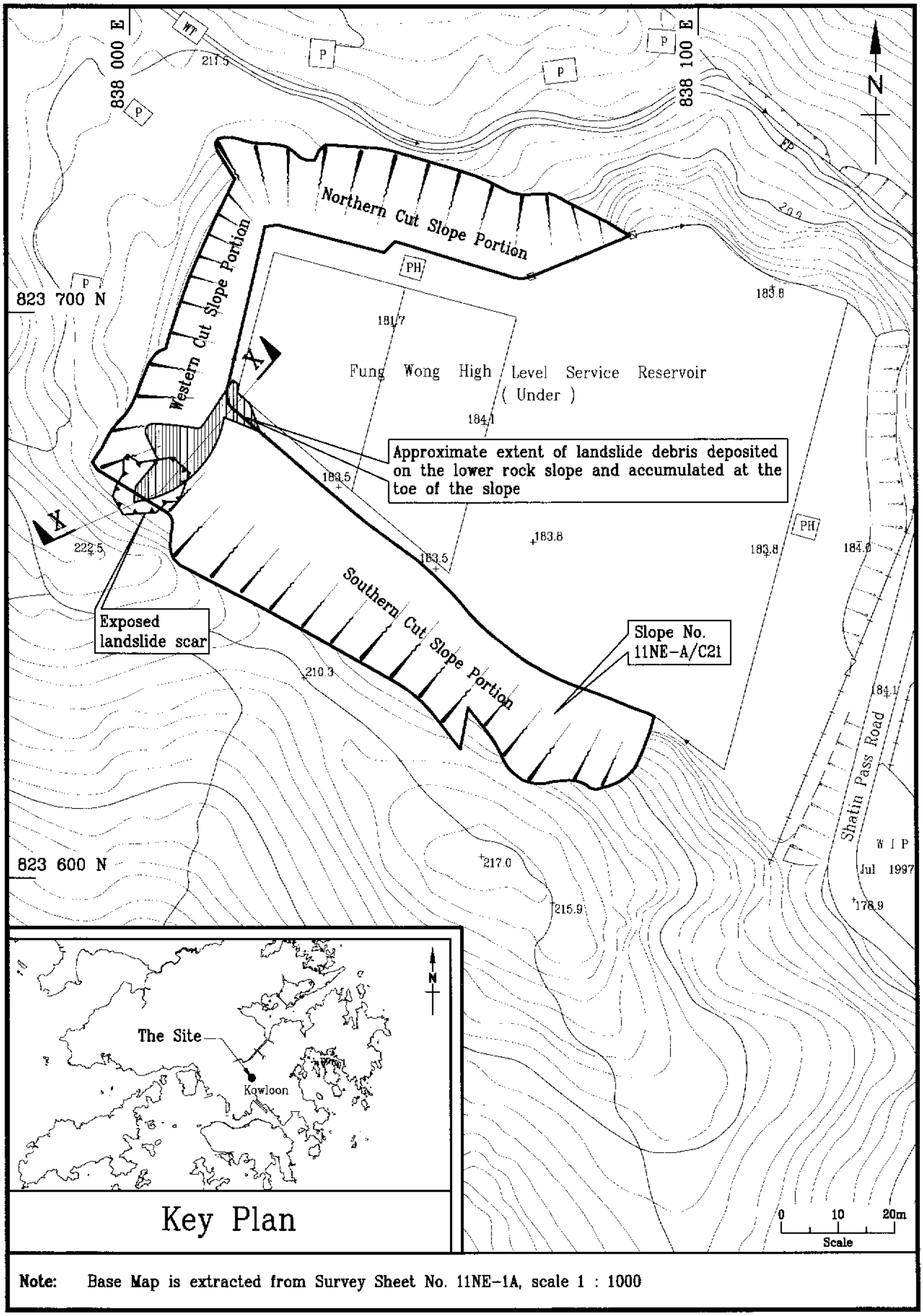


Figure 1 - Site Location Plan

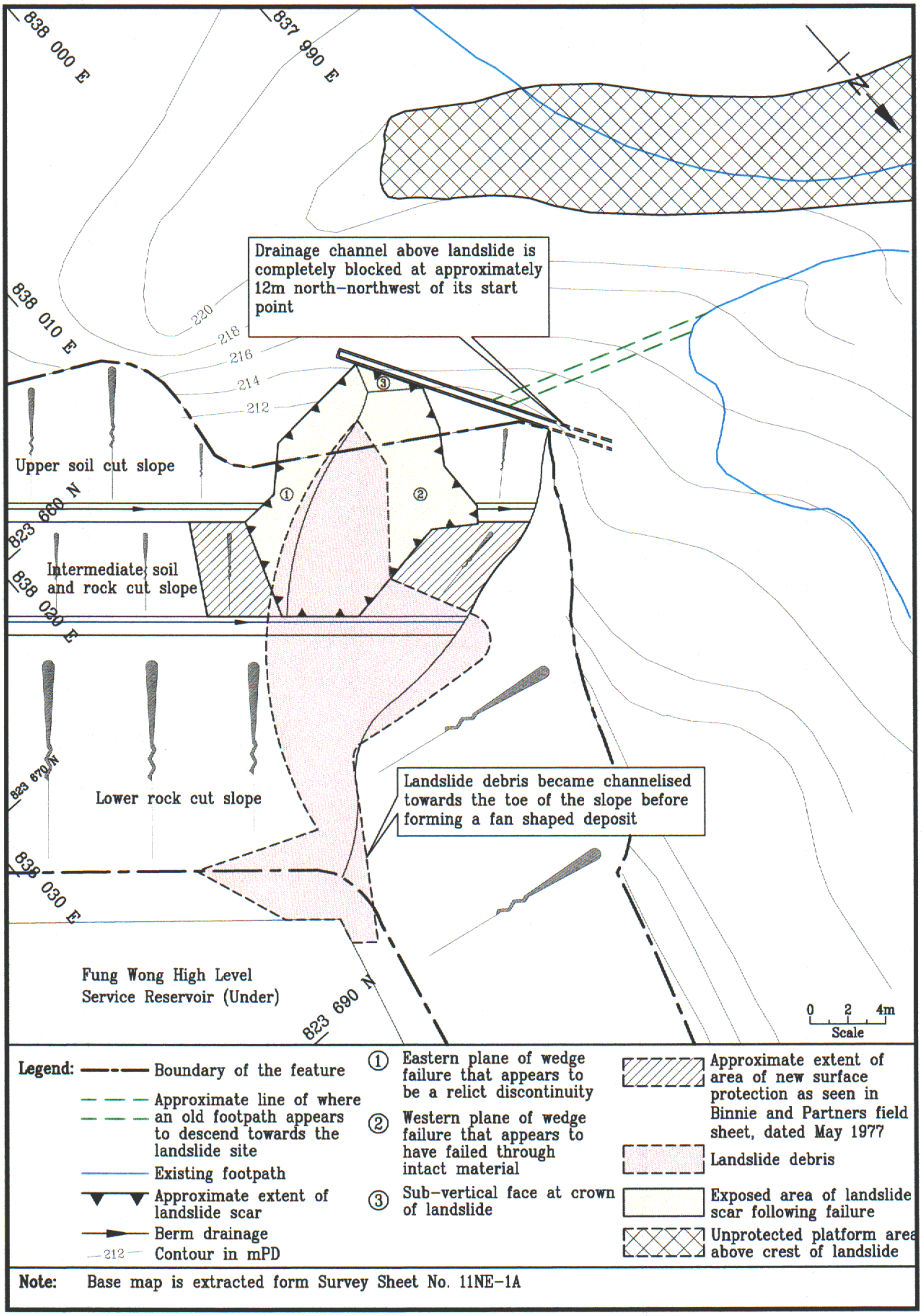


Figure 2 - Plan of the Landslide Site

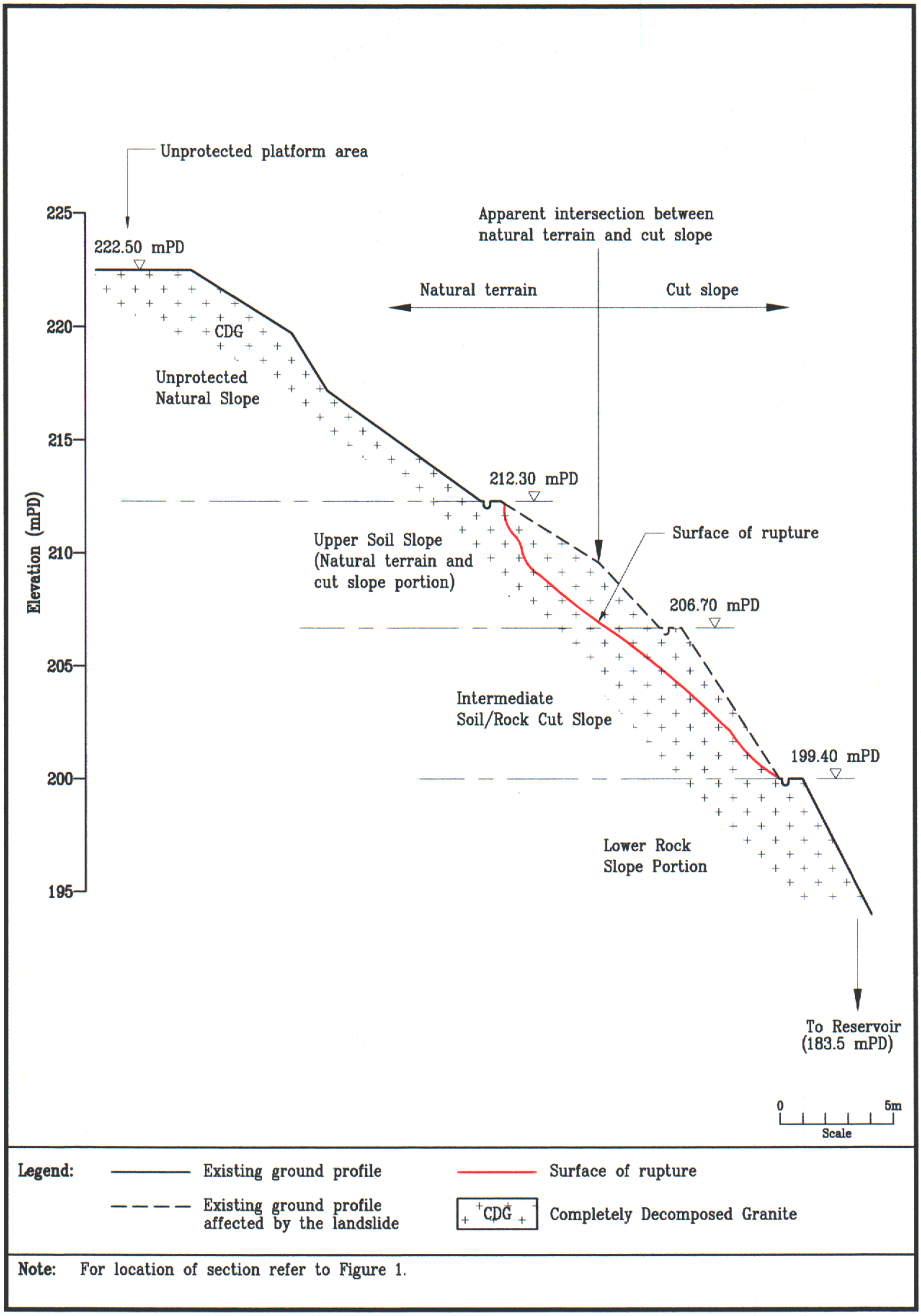
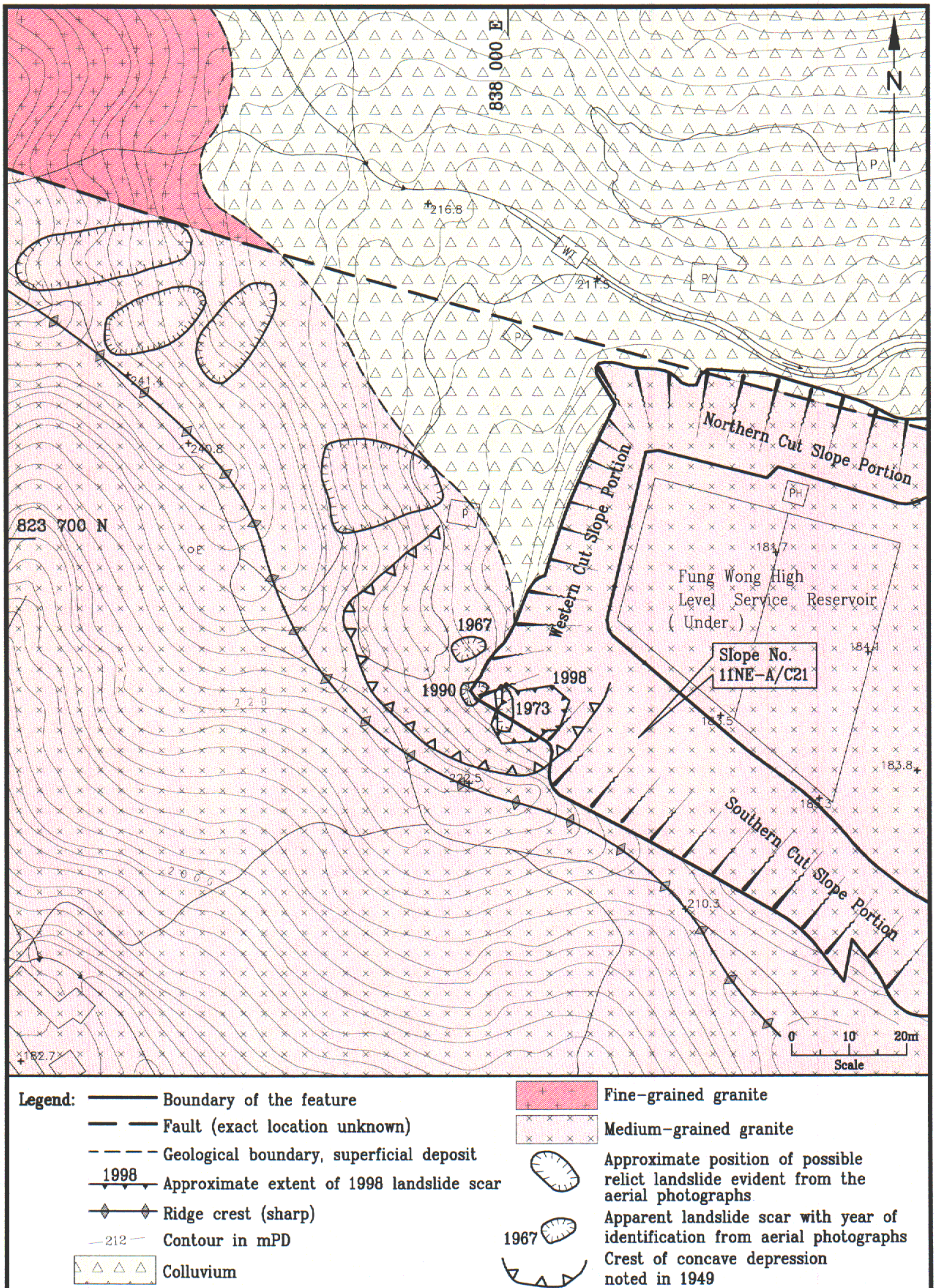


Figure 3 - Cross-section X-X through the Landslide Site



Note: Base Map is extracted from Survey Sheet No. 11NE-1A, scale 1 : 1000

Figure 4 - Geological and Geomorphological Map of the Landslide Site

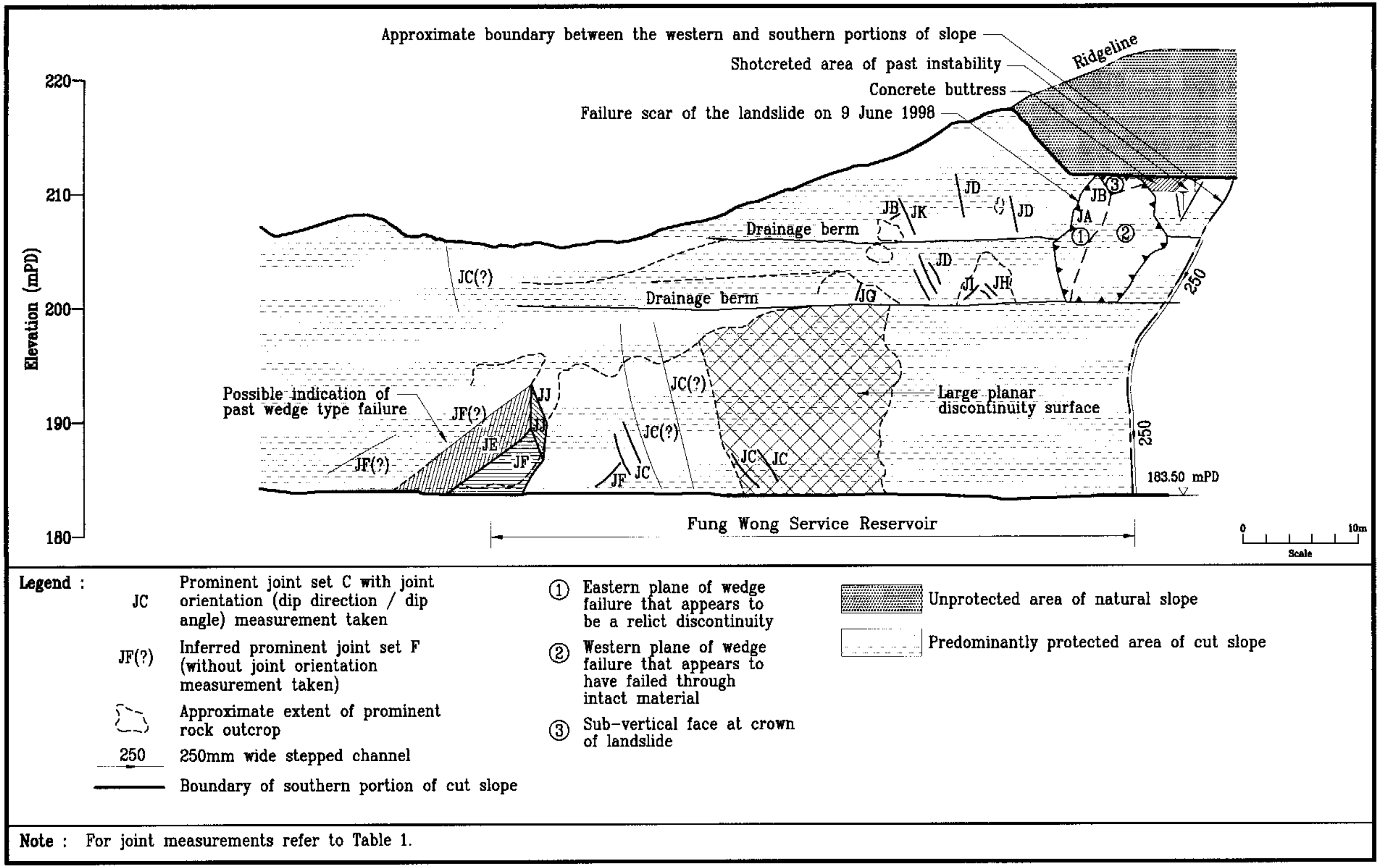


Figure 5 - Elevation of the Southern Cut Slope Showing Locations of Joint Sets

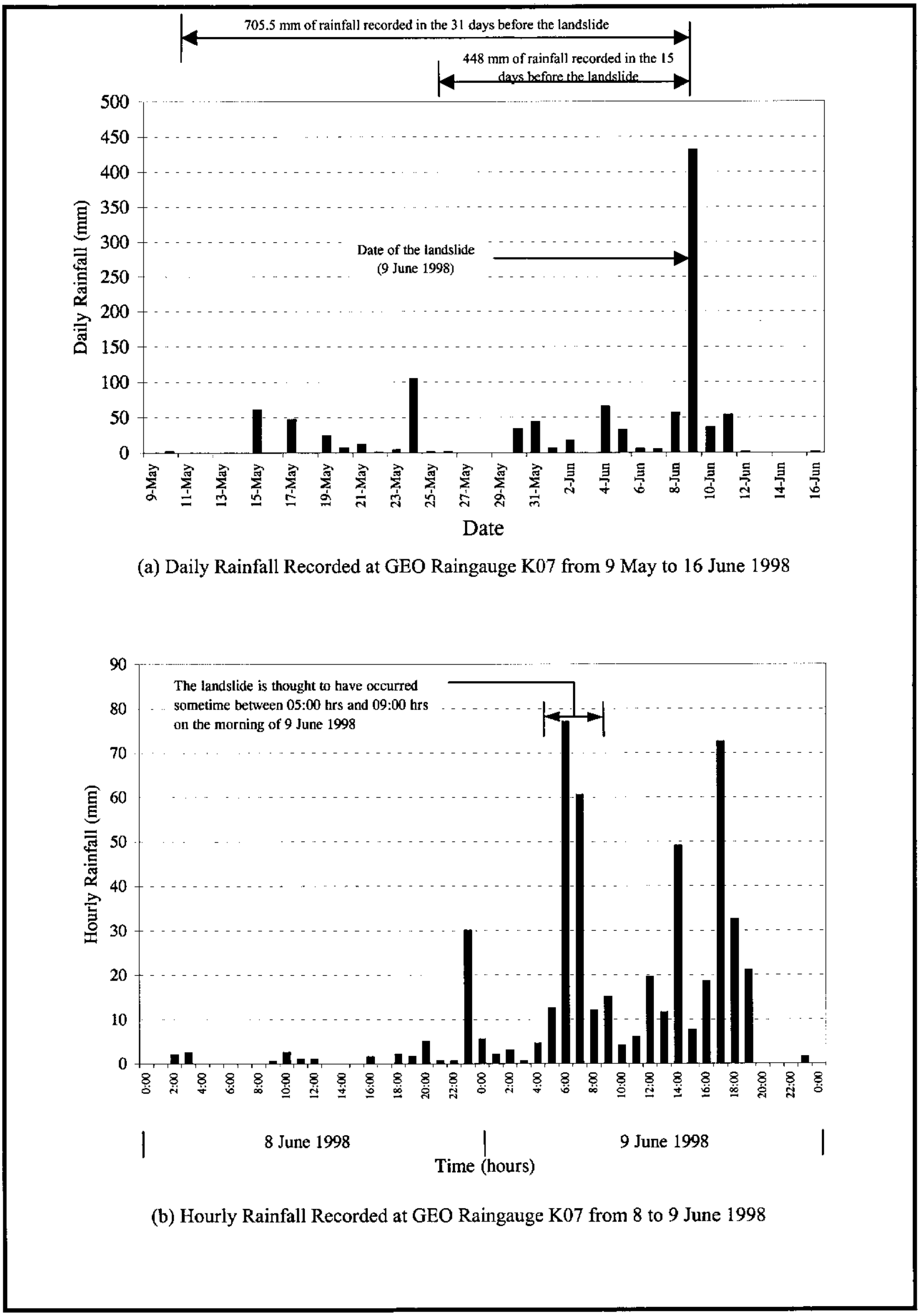
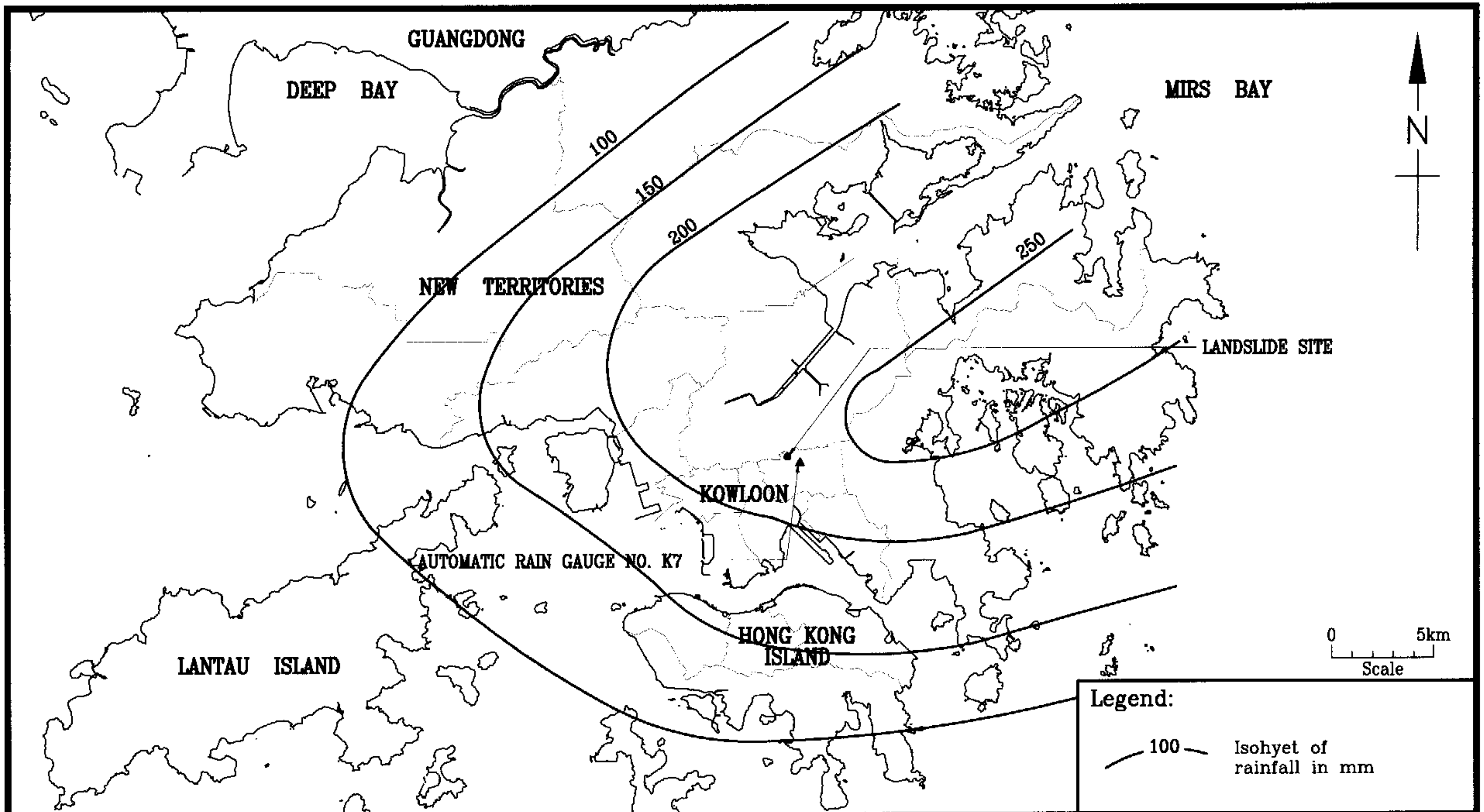
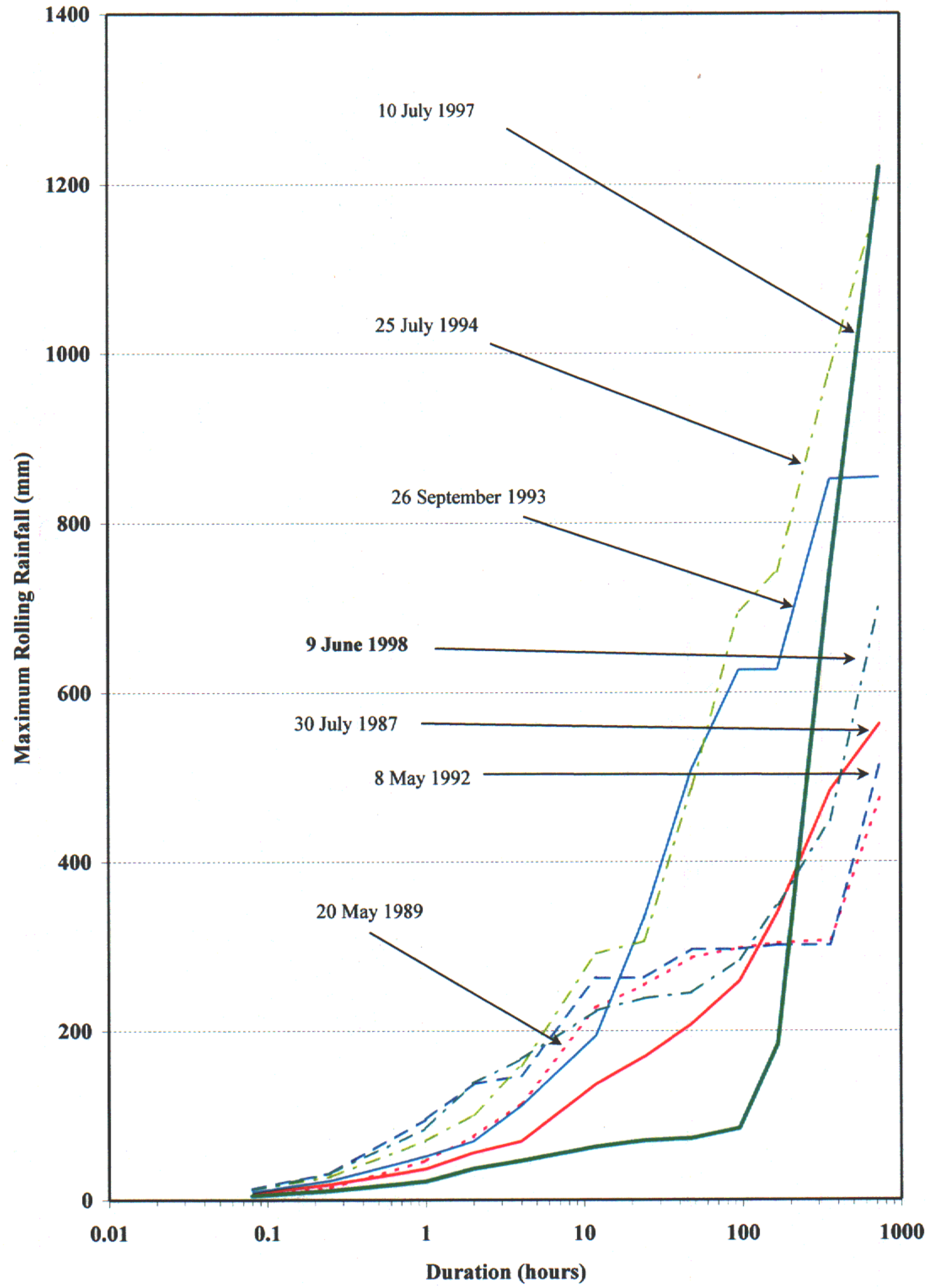


Figure 6 - Daily and Hourly Rainfall Recorded at GEO Raingauge No. K07



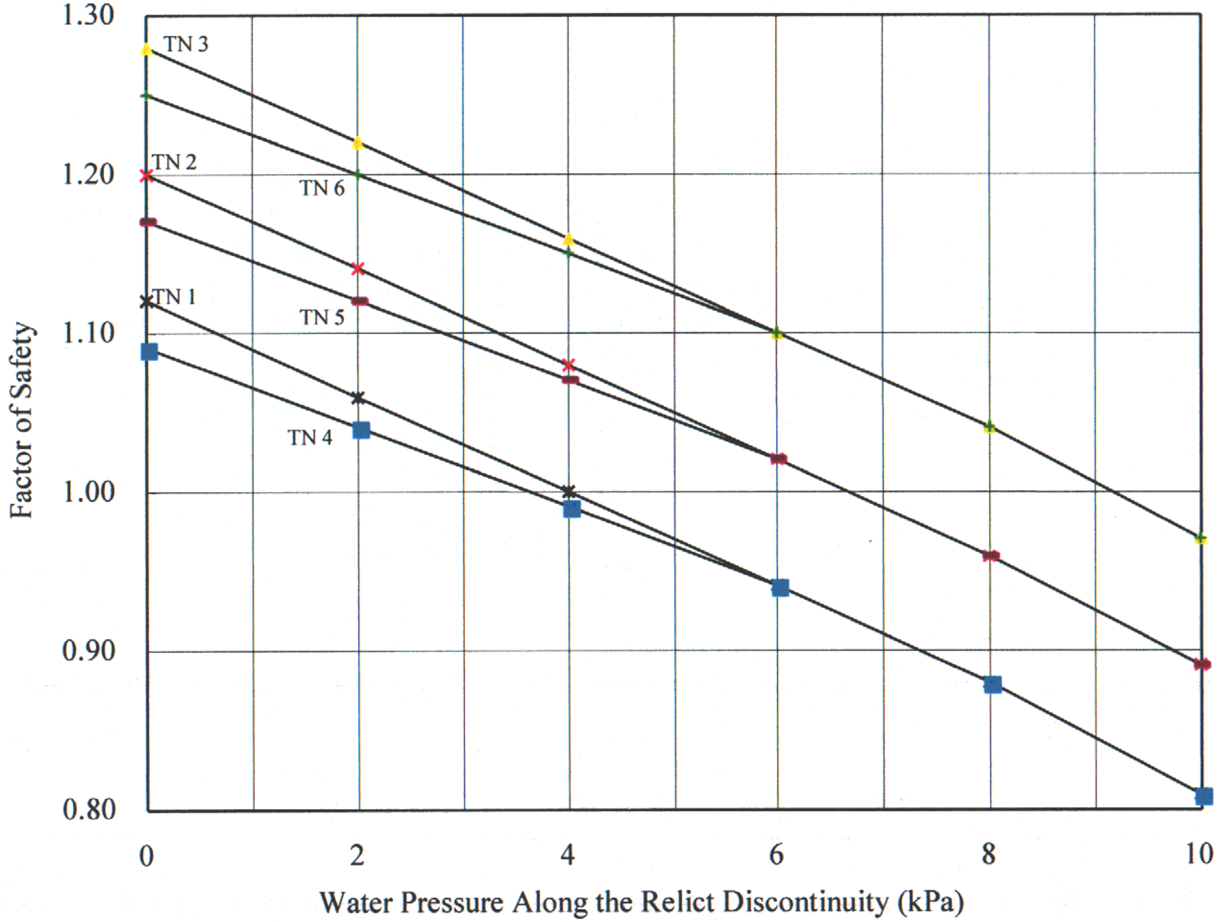
Note: Date from GEO raingauges

Figure 7 - Isohyet of 24-hour Rainfall Prior to the Landslide



Note: Rainfall data are 5-minute statistics from GEO raingauge No. KO7

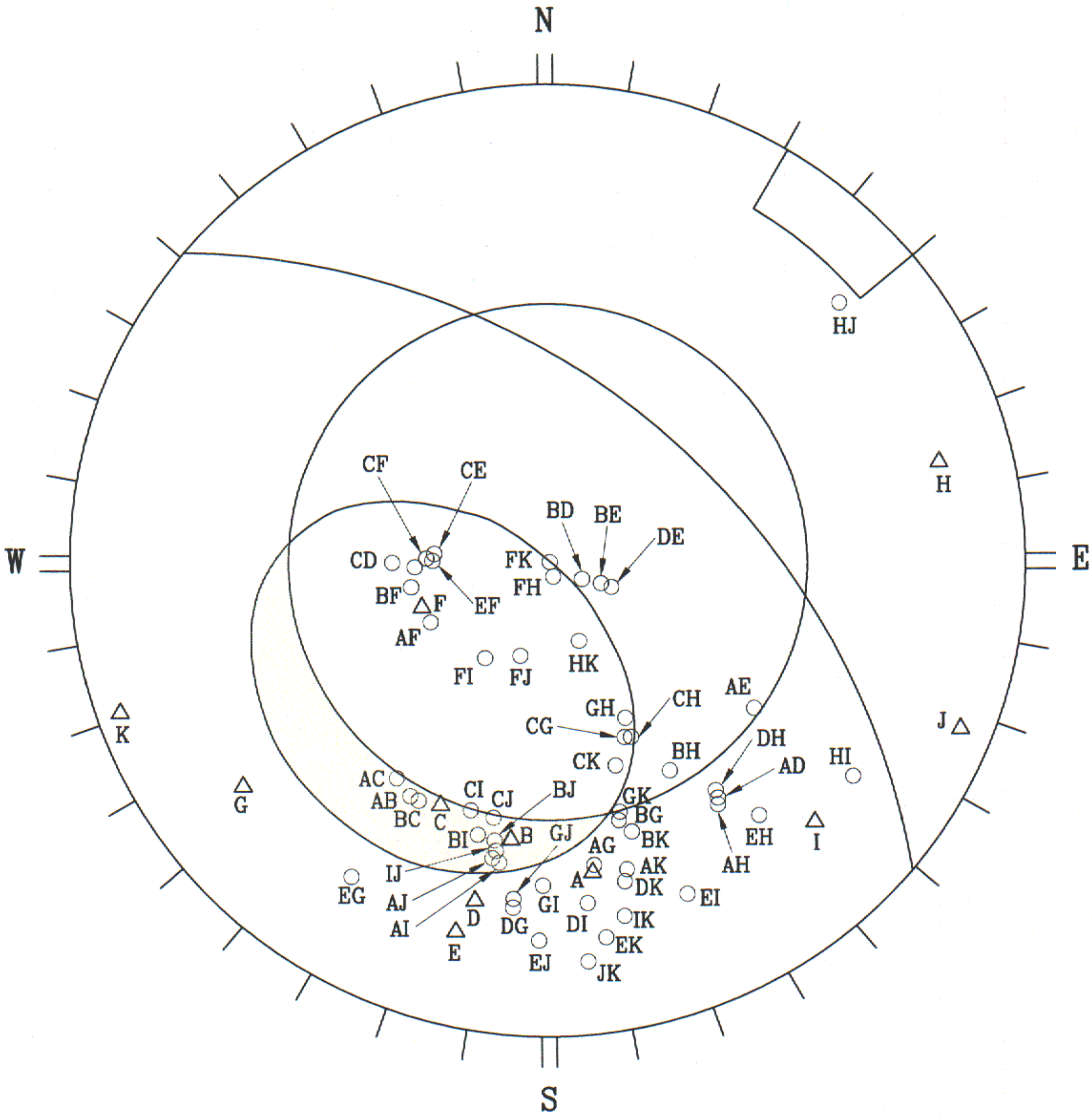
Figure 8 - Maximum Rolling Rainfall at GEO Raingauge No. K07 for Major Rainstorms



	Plane 1		Plane 2	
Trial Number	c' (kPa)	φ' (°)	c' (kPa)	φ' (°)
TN 1			7	
TN 2	0	35	9	40
TN 3			11	
TN 4			7	
TN 5	0	30	9	40
TN 6			11	

Note : Plane 1 represents the steeper easterly failure plane and Plane 2 represents the shallower westerly failure plane, as shown on Figure 2.

Figure 9 – Summary of Results of Stability Analyses



Legend:

A △ Pole of Joint Set A

AB ○ Pole of Line of Intersection between Joint Sets A and B

Zone of Potential Instability

Notes:

(1) For dip direction/dip angle of each joint set see Table 1.

(2) Dip direction of 040° for slope modelled in stereoplot.

(3) Dip dire of slope 060° for slope modelled in stereoplot.

(4) Friction circle of 45° assumed for stereoplot.

Figure 10 - Summary of Kinematic Analyses

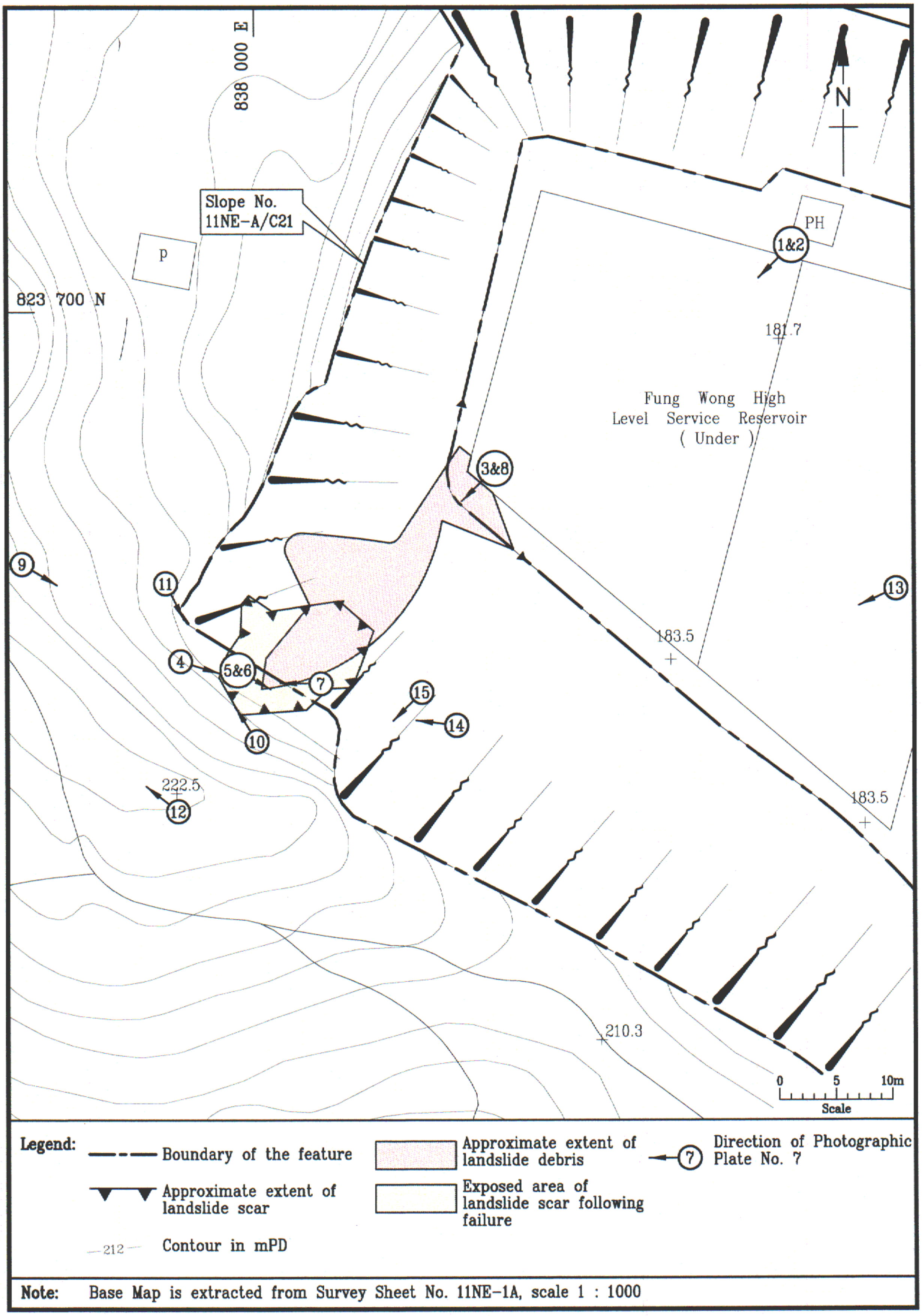


Figure 11 - Plan of the Landslide Site Showing the Direction of Photographic Plates

LIST OF PLATES

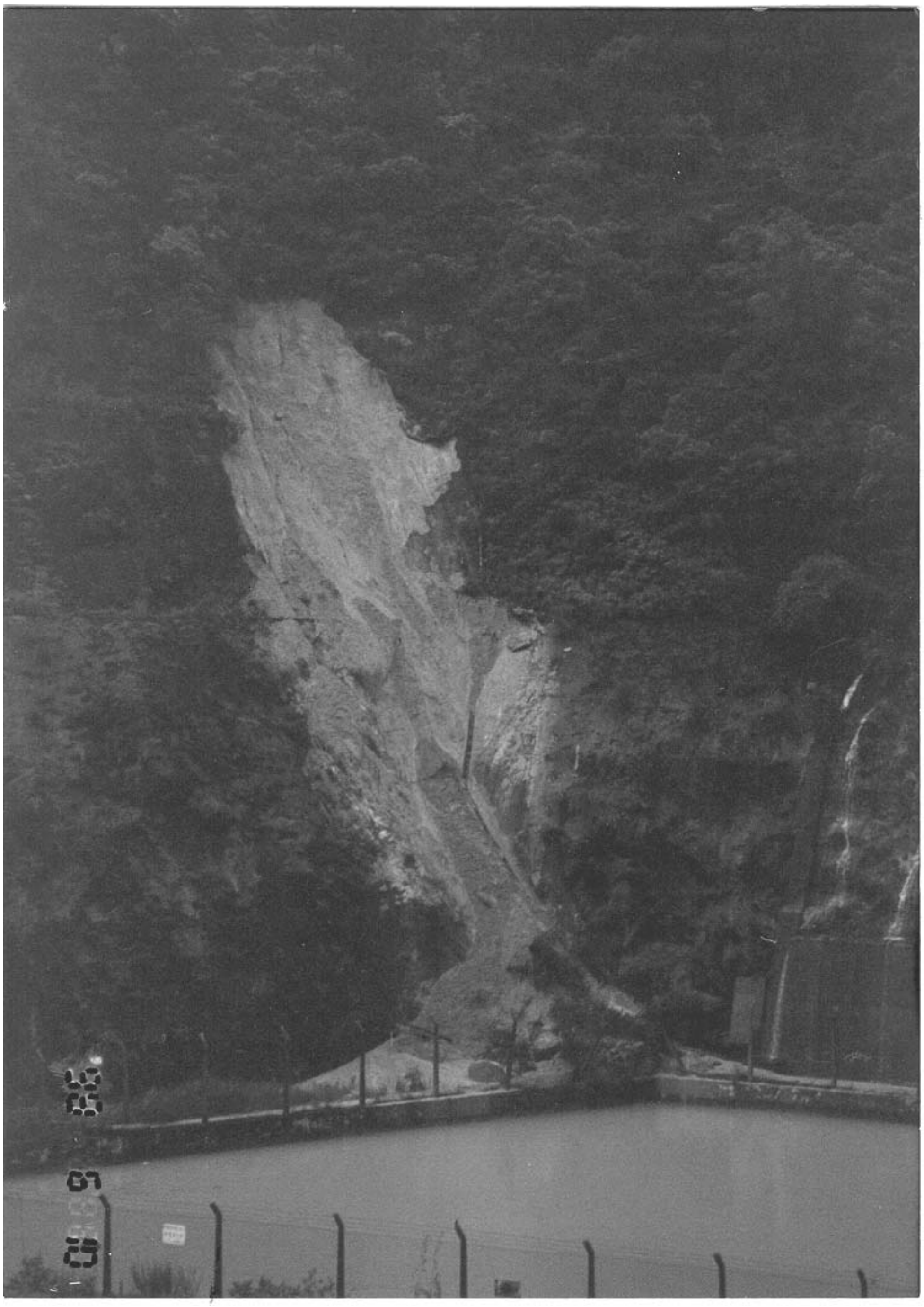


Plate 1 – General View of the Landslide
(Photograph taken on 10 June 1998)

Note: See Figure 11 for Location of Photograph.

Plate 2 – General View of the Landslide
(Photograph taken on 30 June 1998)

Note: See Figure 11 for Location of Photograph.

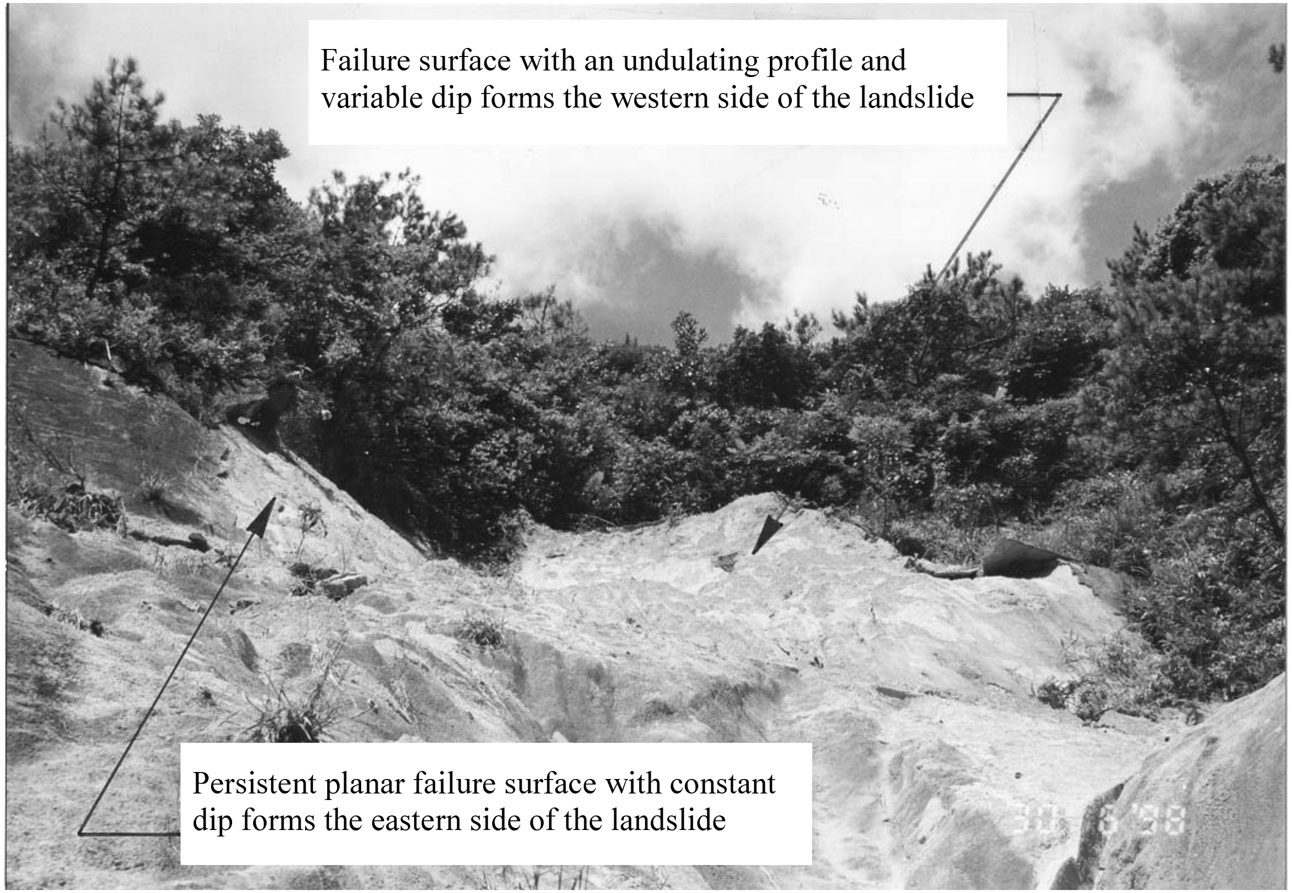
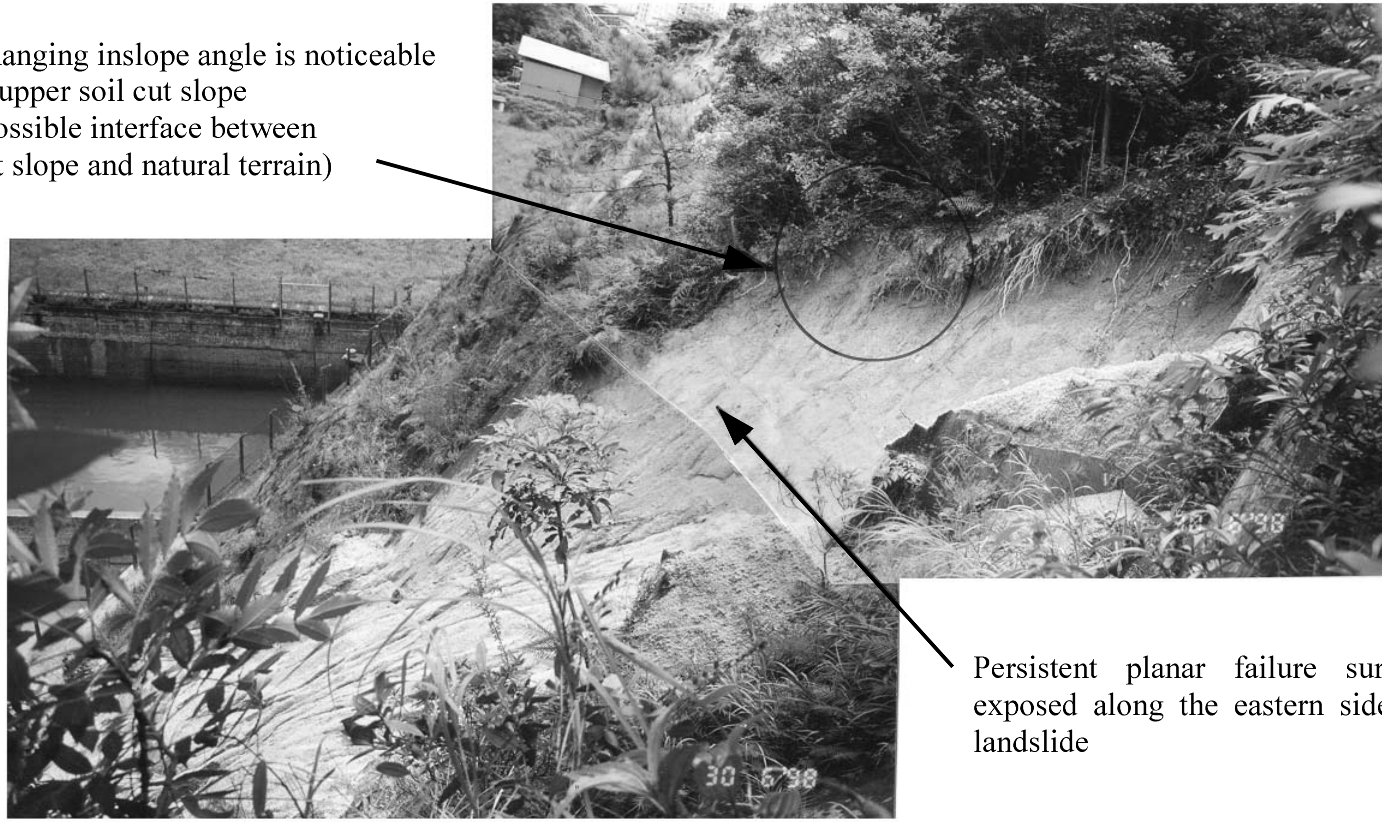


Plate 3 – Up-slope View of the Landslide
(Photograph taken on 30 June 1998)

Note : See Figure 11 for Location of Photograph.

Changing inslope angle is noticeable in upper soil cut slope (Possible interface between cut slope and natural terrain)



Persistent planar failure surface is exposed along the eastern side of the landslide

Plate 4 – View of the Eastern Failure Plane from the Crown of the Landslide
(Photograph taken on 30 June 1998)

Note : See Figure 11 for Location of Photograph.

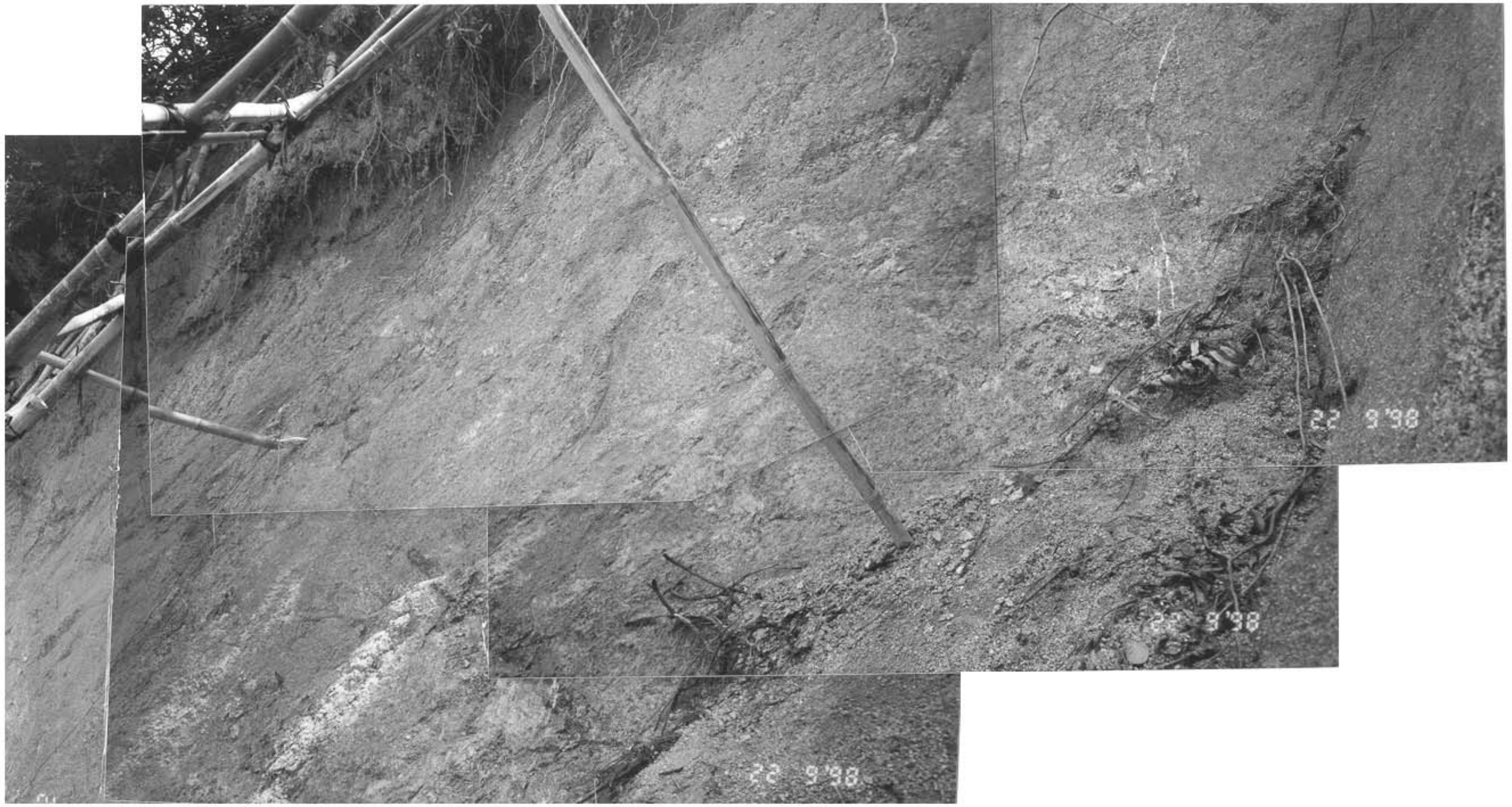


Plate 5 – Close-up View of the Eastern Failure Plane
(Photograph taken on 22 Septembe 1998)

Note : See Figure 11 for Location of Photograph.

Plate 6 – View of Kaolin Infilling along the Eastern Failure Plane
(Photograph taken on 23 September 1998)

Note: See Figure 11 for Location of Photograph.

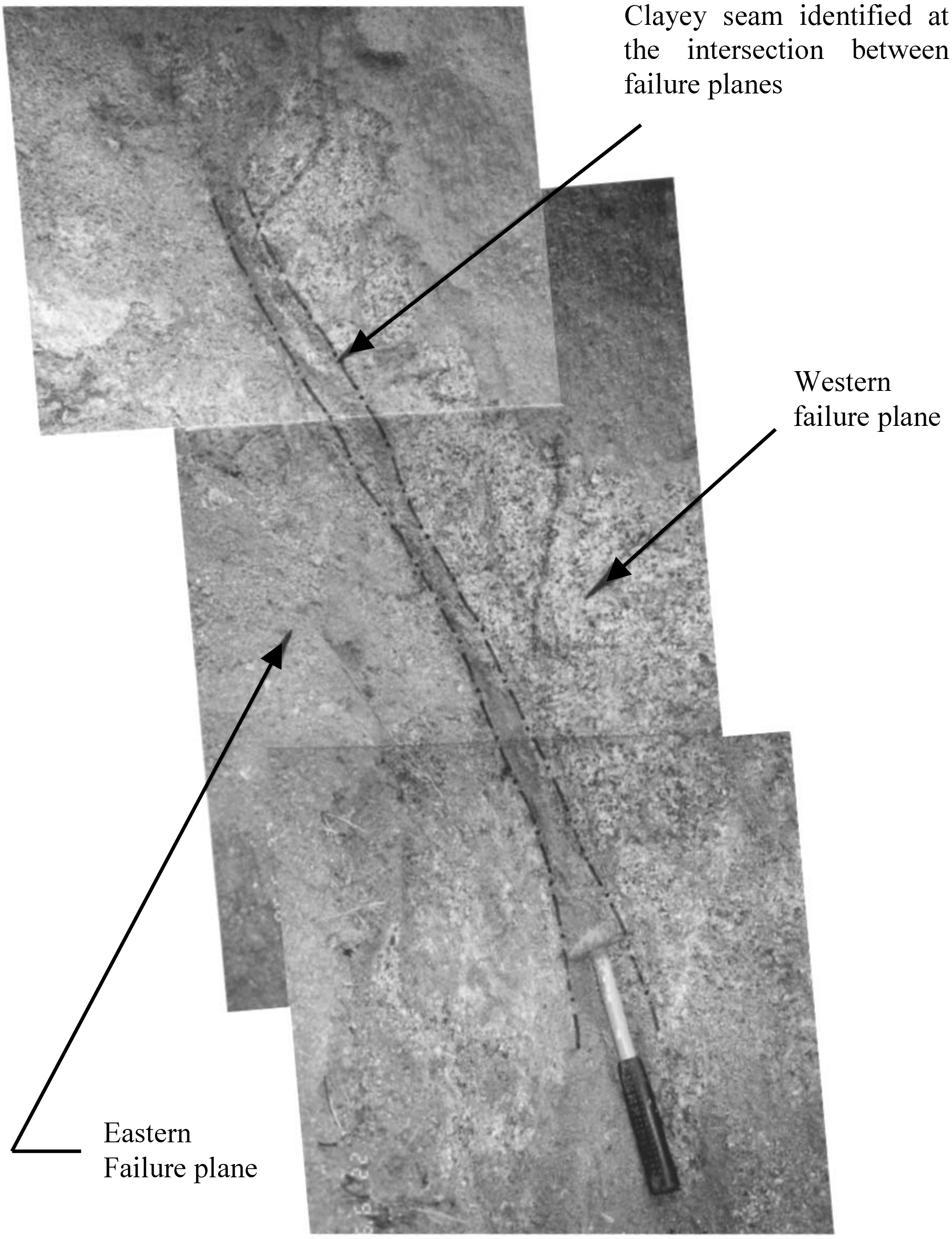


Plate 7 – View of Clayey Seam at the Intersection between Failure Planes
(Photograph taken on 22 September 1998)

Note: See Figure 11 for Location of Photograph.

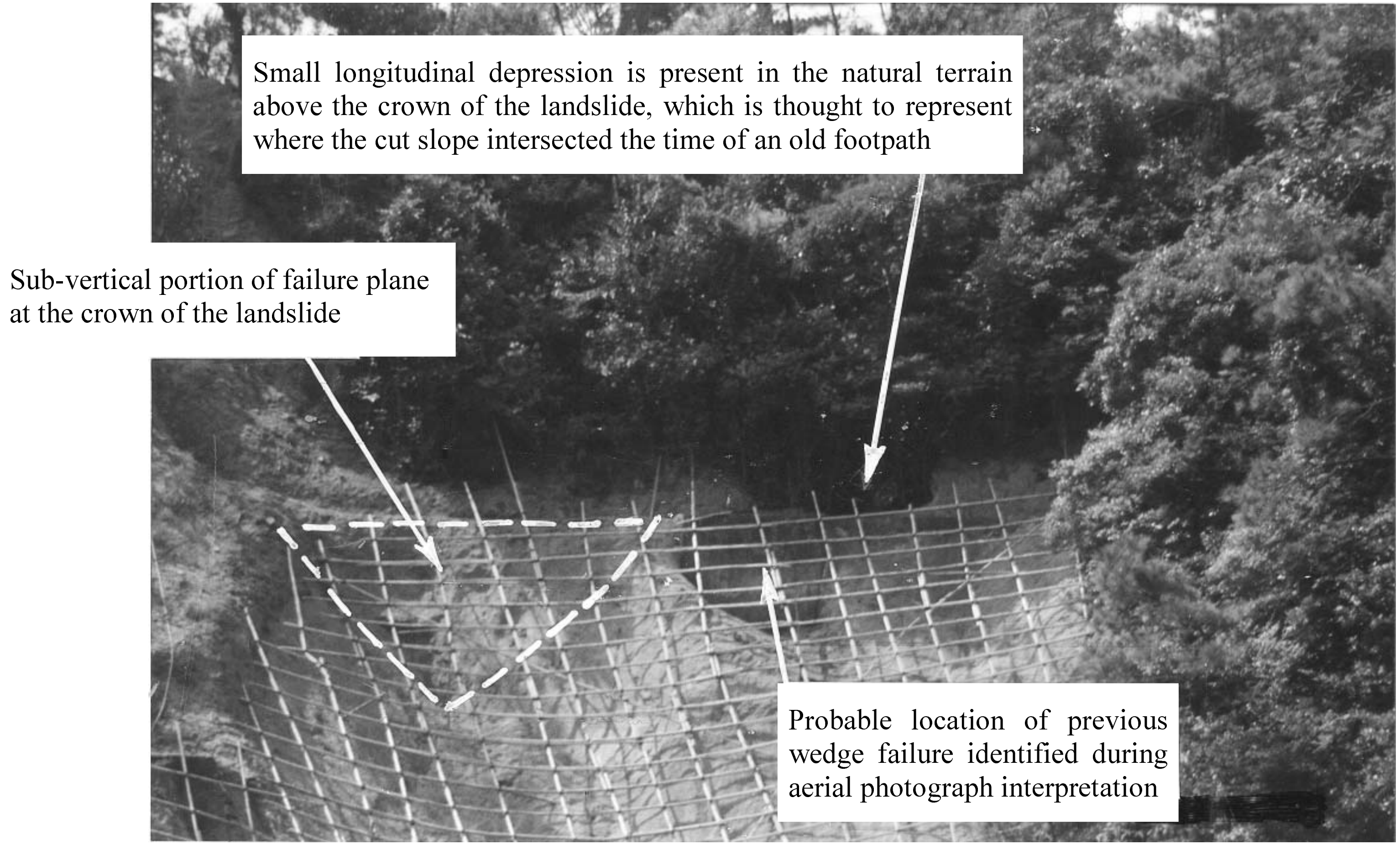


Plate 8 – View of the Upper Portion of the Failure Following Clearance Works
(Photograph taken on 14 October 1998)

Note : See Figure 11 for Location of Photograph.

Plate 9 – View of Existing Footpath that Descends from the Ridgeline
(Photograph taken on 12 October 1998)

Note: See Figure 11 for Location of Photograph.

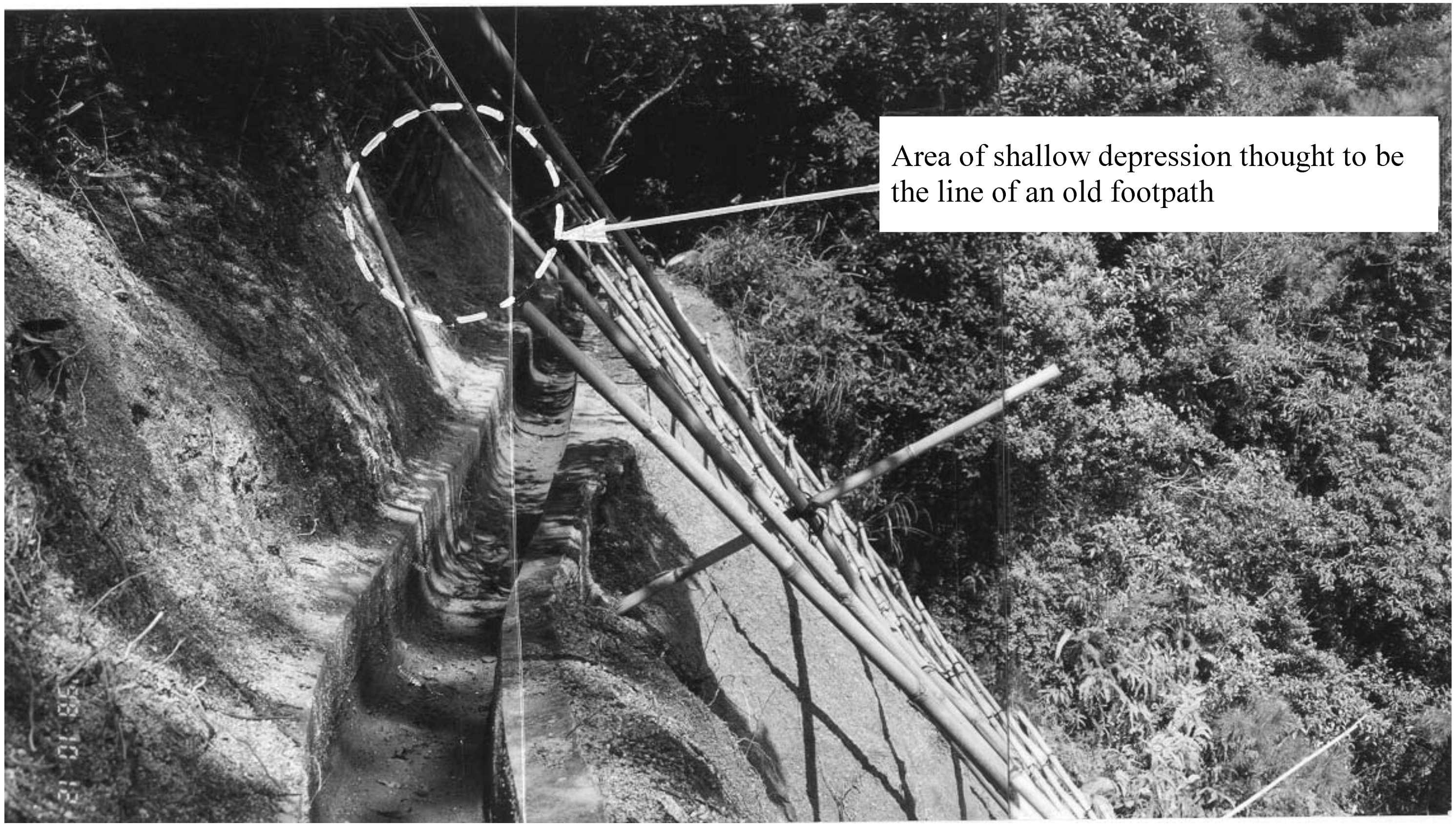


Plate 10 – View of the Drainage Channel above the Crown of the Landslide
(Photograph taken on 12 October 1998)

Note : See Figure 11 for Location of Photograph.

Channel has been completely blocked and totally obscured over a length of between 20 m and 30 m by what appears to be slope wash material

Plate 11 – View of the Drainage Channel above the Western Side of the Landslide
(Photographs taken on 12 October 1998)

Note : See Figure 11 for Locations of Photographs.

Natural terrain descends relatively steeply towards the cut slope and area of the landslide below

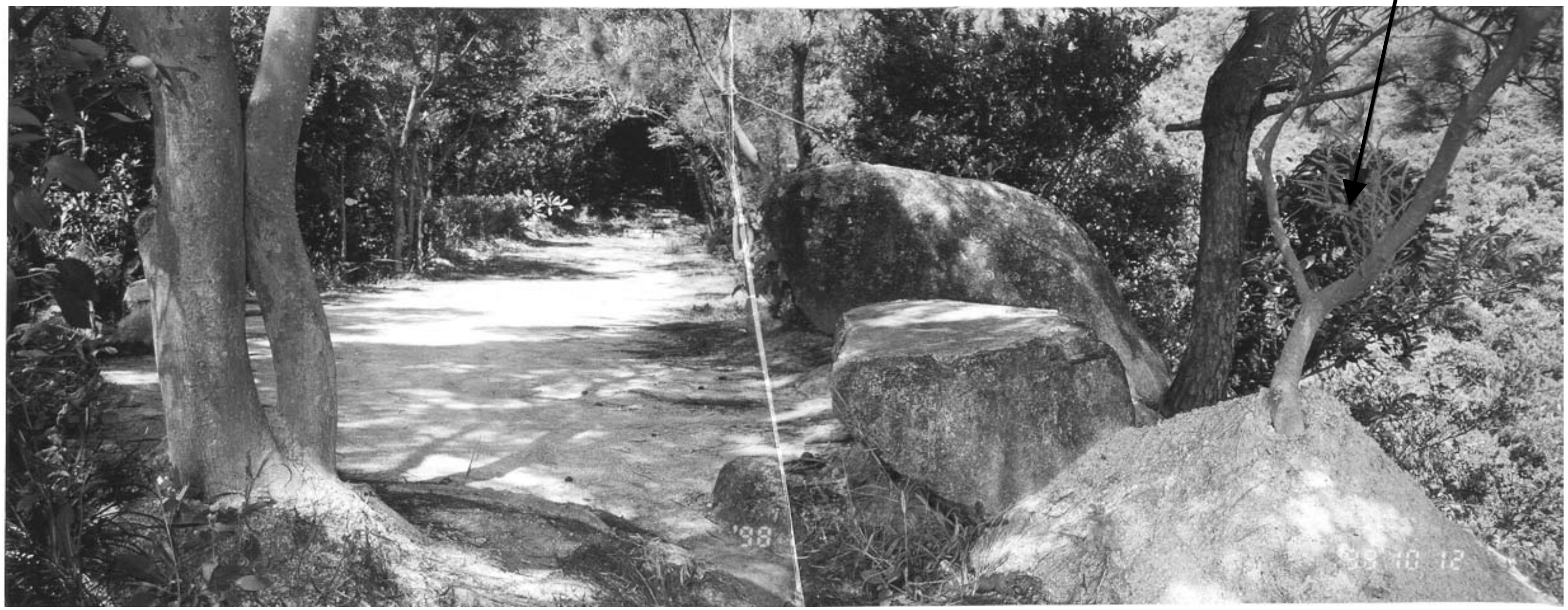


Plate 12 – View of Area above the Landslide Site
(Photograph taken on 12 October 1998)

Note : See Figure 11 for Location of Photograph.

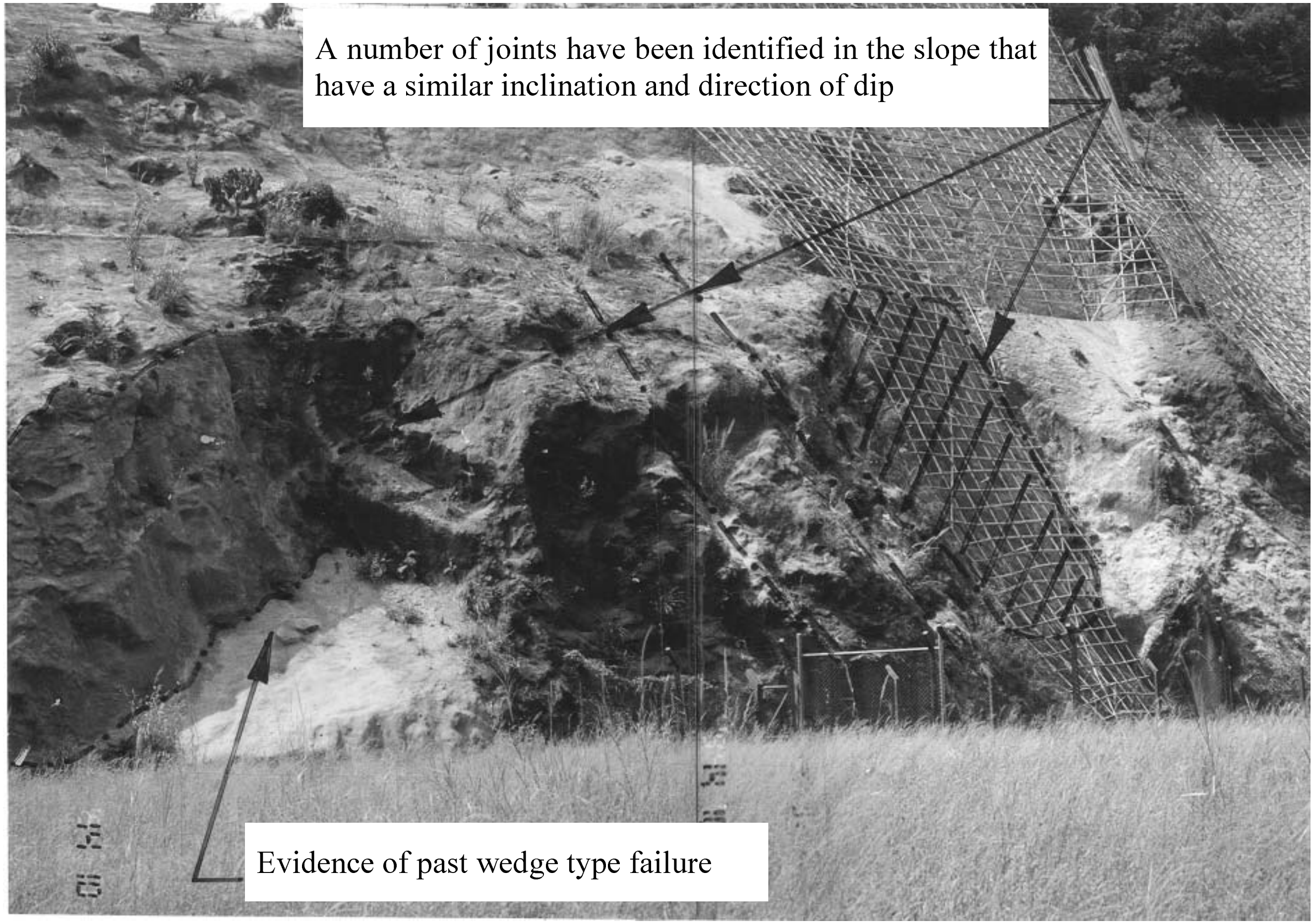


Plate 13 – General View of the Southern Portion of the Cut Slope
(Photograph taken on 14 October 1998)

Note : See Figure 11 for Location of Photograph.

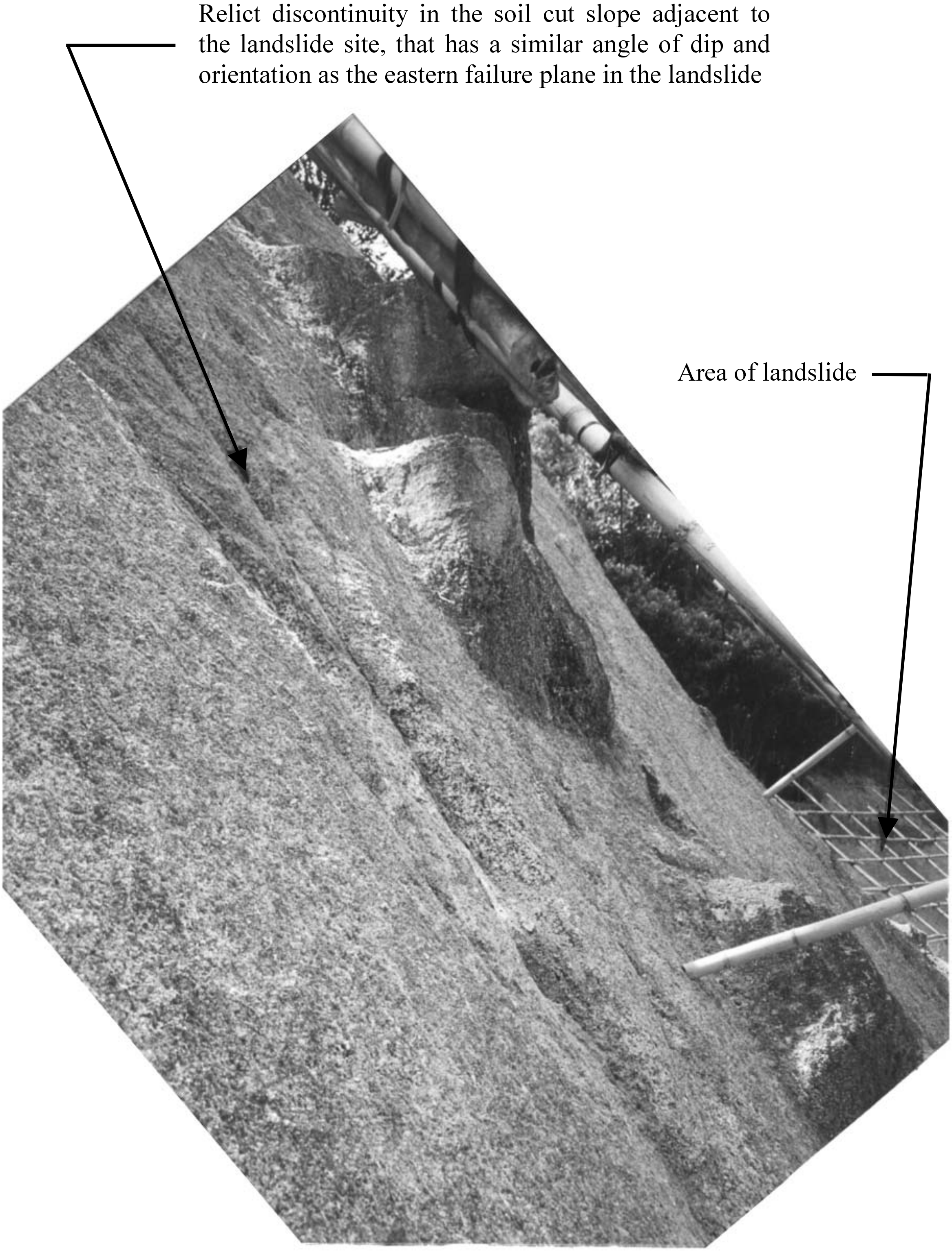
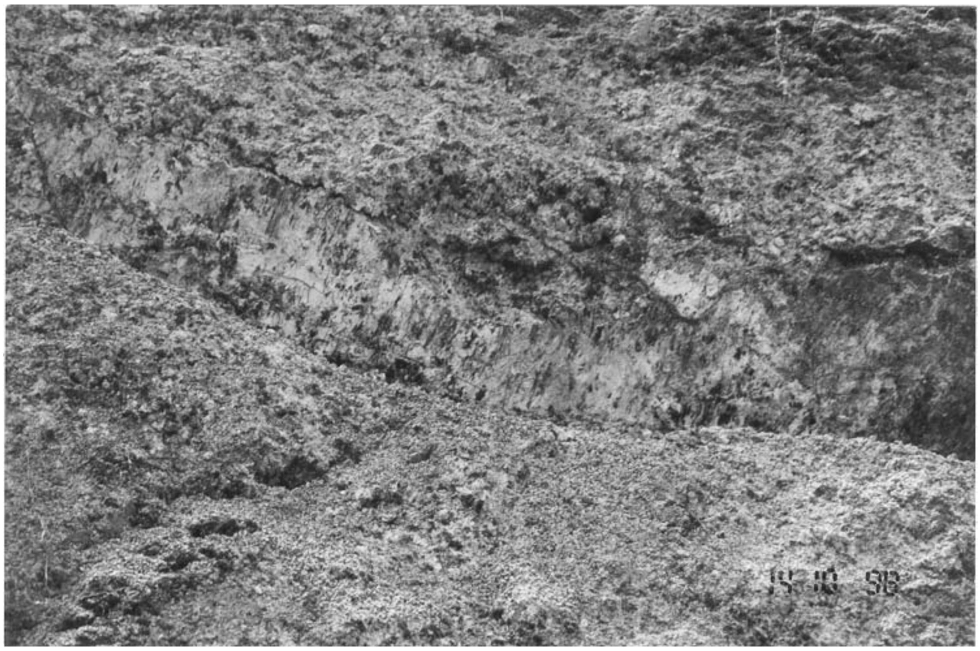


Plate 14 – View of Relict Discontinuities Present in the Upper Soil Cut Slope
(Photograph taken on 14 October 1998)

Note: See Figure 11 for Location of Photograph.



Kaolin infilled discontinuity dipping steeply into the slope in a southwesterly direction. Note the presence of slickensiding along the surface of the kaolin.

Plate 15 – View of Kaolin Infilled Joint Surfaces Exposed in the Soil Cut Slope (Photographs taken on 14 October 1998)

Note: See Figure 11 for Locations of Photographs.

APPENDIX A

AERIAL PHOTOGRAPH INTERPRETATION

A1. SUMMARY OF SITE HISTORY

The aerial photographs show that the cut slope under investigation was formed in association with the construction of the reservoir in about 1963. The cut slope comprises three distinct sections, namely the northern, western and southern sections, which lie perpendicular to each other. The photographs indicate instability at the eastern end of the northern section of the cut slope prior to 1963, as well as at the western end in 1984. In the immediate vicinity of the current landslide several indications of small landslides or active erosion are visible in 1967 and 1990. A local depression adjacent to the current failure that can be seen on site is also evident in the 1973 photographs as a wedge type feature. Indications of excessive seepage/surface water flows are generally evident in the photographs between 1963 and 1996, predominantly from the western slope.

Since the original formation of the cut slope in 1963, periodic maintenance in the form of new surface protection is evident from the photographs, otherwise, except for the failures noted above, no significant changes or modifications are visible.

A2. PHOTOGRAPHS

A full list of the photographs studied as part of the API is presented below:

DATE	ALTITUDE (Feet)	PHOTO REFERENCE
May 1949	8000	Y1809-Y1810
January 1963	2700	Y08244-Y08245
May 1967	6250	Y13421-Y13422
December 1973	3000	6856-6857
December 1974	4000	10192-10193
June 1976	2500	14302-14303
December 1977	4000	20256-20257
November 1978	4000	24108-24109
April 1980	4000	30135-30136
February 1981	5500	36586-36587
November 1984	4000	57028-57029
September 1986	4000	A06308-A06309
June 1987	4000	A09494-A09495
October 1988	4000	A14718-A14719
November 1990	4000	A23608-A23609
November 1991	4000	A27515-A27516
April 1992	4000	A30466-A30467
November 1993	4000	A36081-A36082
September 1995	3500	CN11364-CN11365
November 1996	4000	CN15839-A15840

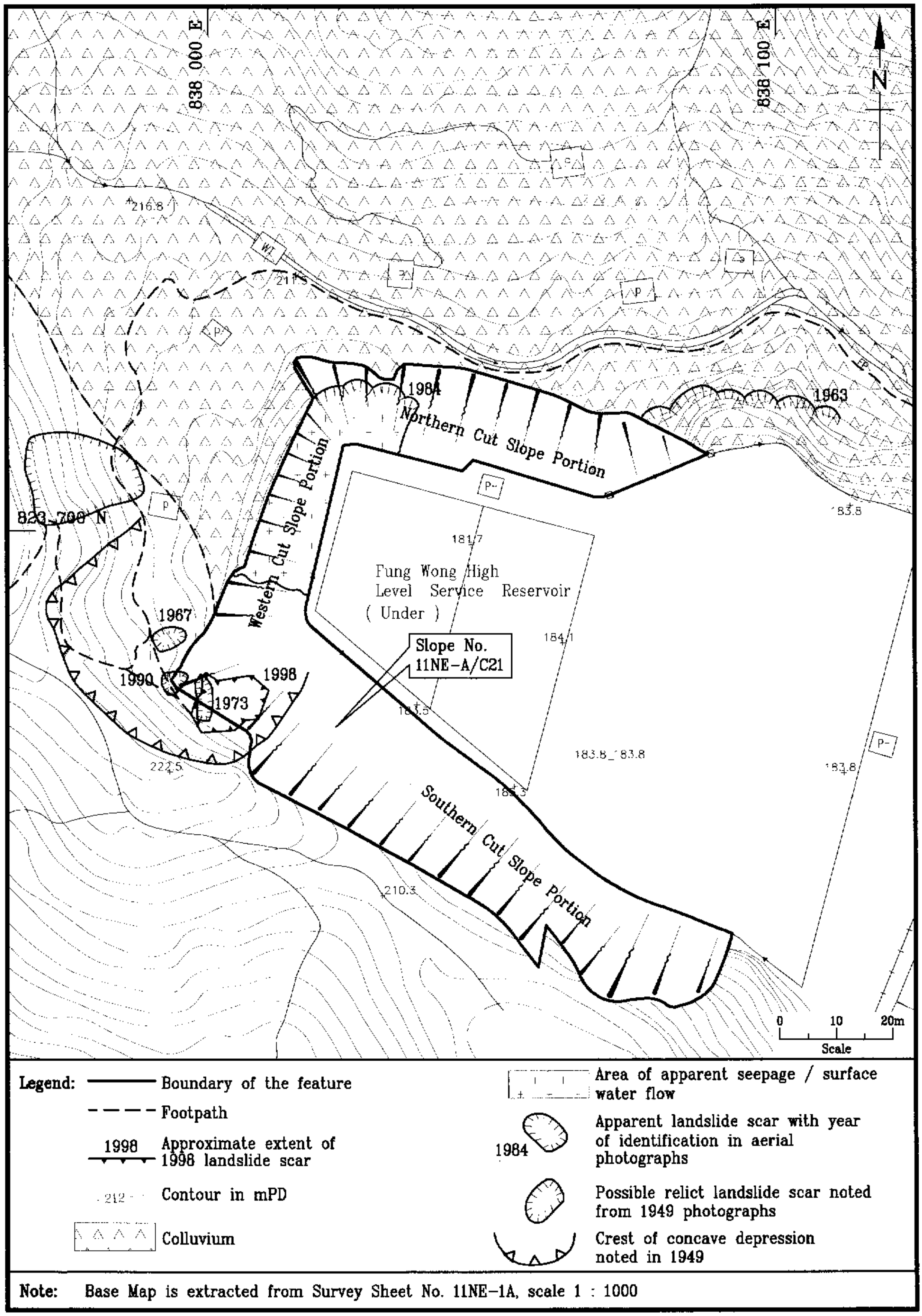


Figure A1 - History of Development of the Site