一九九五年八月十三日
柴灣翡翠道山泥傾瀉事件報告
REPORT ON THE FEI TSUI ROAD LANDSLIDE OF 13 AUGUST 1995

土力工程處報告系列第188號

GEO REPORT No. 188

黎佐賢爵士及土力工程處

Sir John Knill & Geotechnical Engineering Office

香港特別行政區政府

土木工程拓展署

土力工程處

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING AND DEVELOPMENT DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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First published, August 2006

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. The GEO Reports can be downloaded from the website of the Civil Engineering and Development Department (http://www.cedd.gov.hk) on the Internet. Printed copies are also available for some GEO Reports. For printed copies, a charge is made to cover the cost of printing.

The Geotechnical Engineering Office also produces documents specifically for publication. These include guidance documents and results of comprehensive reviews. These publications and the printed GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the second last page of this report.

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

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FOREWORD

This Report is presented in two volumes. Volume 1 contains the independent findings of Sir John Knill on the Fei Tsui Road landslide of August 1995 and the lessons to be learnt from it. Volume 2, prepared by the Geotechnical Engineering Office of the Civil Engineering Department, presents the detailed findings of the landslide investigation. The contents of Volume 2 have been reviewed and agreed by Sir John Knill who relies on them in his own assessment given in Volume 1.

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VOLUME 1: INDEPENDENT REVIEW OF THE INVESTIGATION BY THE GEOTECHNICAL ENGINEERING OFFICE

Sir John Knill
Berkshire, the United Kingdom

This report was originally produced in February 1996 as Report on the Fei Tsui Road Landslide of 13 August 1995

CONTENTS

1. INTRODUCTION

Intense rainfall occurred in Hong Kong on 12 and 13 August 1995 in the wake of Typhoon Helen; there were reports of over 120 landslides. One of these landslides, at Fei Tsui Road in Chai Wan, caused the burial of the road in slide debris. There was one fatality and one person was injured.

The Geotechnical Engineering Office (GEO) of the Civil Engineering Department commenced an investigation of the landslide on the morning of 13 August 1995. The results of this investigation are reported in an accompanying document (GEO, 1996).

It was decided to have an independent technical review of the GEO investigation, which the writer was invited to carry out, reporting to the Government of Hong Kong. Three visits were made for this purpose over the periods 5-8 September, 31 October-3 November and 27 November-1 December 1995. During these visits the site was inspected twice, and discussions were held with GEO regarding the investigation programme and the results as progressively achieved. An advanced draft of the GEO report was reviewed directly with the GEO.

2. DESCRIPTION OF LANDSLIDE

The landslide took place in a cut slope excavated into the northern side of a spur of volcanic rocks which extends in a east-north-east direction down from the flanks of Mt Collinson. The site formation was carried out between 1972 and 1976 as part of the construction of the Hing Wah Estate Phase II development. Fei Tsui Road was built on the northern side of the slope which had a maximum height of about 27 m and an overall slope angle of about 60°. The Chai Wan Salt Water Service Reservoir which was constructed in 1959 is located on the crest of the spur immediately to the south of the cut slope. To the east of the Reservoir the top of the spur above the crest of the slope is composed of flat vegetated land which was, up to 1991, occupied by a squatter settlement. The southern flank of the spur is formed by a valley with a small stream which exits at the eastern end of Fei Tsui Road.

The slope was excavated with a narrow berm at about two thirds of its height, the upper part of the slope being chunamed at the time of construction; the lower part of the slope consists of exposed rock. The slope face at the time of the landslide was covered with vegetation. In front of the slope, and separating it from Fei Tsui Road, there is a 12 m wide strip of level ground which was fenced-off, being partly covered by grass and partly used for temporary storage of drainage equipment.

Since the excavation of the slope four small failures have taken place in 1985, 1986, 1987 and 1993. The debris from all these failures had been confined within the limits of the level ground in front of the rock face.

The landslide took place in two stages, this observation being well-documented by eye witnesses. The first failure occurred at about 0055 on 13 August at the eastern end of the eventual landslide immediately adjacent to the site of the 1993 failure. The failure involved some tens of cubic metres of rock and, like the four previous small failures, the debris was confined to the level ground. The second, major, failure occurred about 20 minutes later

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cutting 40 m back into the slope and forming a scar with a maximum length of nearly 90 m long and with an average vertical depth of about 15 m. About 14,000 m^3 of debris slid out of the scar crossing the fenced-off level ground and Fei Tsui Road, entering a playground and mounding up against the side of the Chai Wan Baptist Church to a maximum depth of 6 m. The maximum extent of the debris mound from the toe of the slope was 33 m, and the mound had a length of just over 90 m along Fei Tsui Road indicating that little lateral spreading occurred.

The general appearance of the landslide after failure suggested that this was likely to be a translational failure with a rock mass sliding outwards on a surface inclined northwards out of the slope. The subsequent investigations have confirmed this interpretation. Although this mechanism is a well-known form of rock slope instability, large failures of this type are unusual in Hong Kong and, indeed, the Fei Tsui Road landslide is the largest recorded rapid onset failure in a permanent cut slope in the Territory.

3. REVIEW OF GEOTECHNICAL ENGINEERING OFFICE REPORT

The GEO report will now be reviewed on a section-by-section basis.

> “1. Introduction
>
> The introduction to the report provides the setting to the landslide event and outlines the main components of the investigations carried out. The method and scale of the investigation was wholly appropriate to the nature of the landslide.
>
> 2. Description of the Site
>
> The site description, together with the figured maps and photographs, provides an adequate factual description of the site which is on Government land. Attention is drawn to the interpretative nature of the layout of the surface drainage system, and the observations made with regard to blockage of drains.
>
> 3. Description of the Landslide
>
> The description of the landslide covers the geometrical form of the landslide, and the nature and distribution of the resulting debris. The unusual size of the Fei Tsui landslide is highlighted. Particular note should be taken of the runout distance of the debris which was longer than would be expected for landslides in Hong Kong, indicating greater mobility of the slipped material at the time of failure. If the slide debris had behaved in accord with common rain-induced landslides in Hong Kong, then the main mass of debris would have probably

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extended to the middle of Fei Tsui Road.

Reference is made to the exposure of the 24-inch subsurface salt water main as a result of the landslide which was observed to be discharging water after the failure.

4. History of the Site

The site development history is outlined in this section. There is some uncertainty as to the reason for the presence of the strip of level ground below the slope but it is believed that consideration had been given at some stage to the eventual widening and extension of Fei Tsui Road to the southern side of Hong Kong Island. A consequence of this situation was that, although the slope was registered, and subject to various reviews and to the normal GEO procedures, it was physically detached from immediate proximity to Fei Tsui Road or to buildings.

Reference is made to the four small failures which occurred in the slope, two of which were formally reported to the Geotechnical Engineering Office (Geotechnical Control Office before 1991). In each case the debris was retained within the limits of the level ground in front of the slope. In the case of both the 1987 and 1993 incidents the base of the failure was about 10 m above ground level and involved the upper part of the slope. Inspection of the contemporary records and photographs indicates that the base of these failures was the same as that associated with the 13 August 1995 landslide. In the case of the 1993 failure fly rock broke some of the windows in the Baptist Church. As these windows were at a relatively high level, the landslide would have had to hit the ground with some force to generate fly rock on a steep enough trajectory. This could suggest that the failure process was by toppling.

5. Analysis of Rainfall Records

A rain gauge is located on the roof of Wo Hing House about 220 m north of the landslide and the records which are available from this gauge since August 1979 were used to carry out an evaluation of the intense rainfall to which the slope was subjected in relation to previous events.

The rainfall in Hong Kong in August 1995 was the highest ever recorded in the month of August based on records at the Royal Observatory, the rainfall being particularly heavy in the early part of the month. The rainfall recorded by the gauge was the highest recorded at that site for any rainstorm event for

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durations exceeding 7 days. The rainfall intensities for periods shorter than 12 hours are comparable to the highest intensities recorded in previous events.

6. Sequence of Failure

There were a number of witnesses to the landslide, including police officers, so that a reliable reconstruction of the timing of the two-stage sequence (described in Section 2. above) has proved possible.

7. Characterisation of the Subsurface Conditions at the Site

7.1 Geology

The geological conditions have been investigated by means of mapping of the landslide area after most of the debris had been removed, trenches and boreholes. The landslide took place in volcanic rocks grading from predominantly completely to highly decomposed tuff at the top of the slope to predominantly moderately to slightly decomposed tuff in the lower part of the slope. The rock mass is jointed with two through-going steep joint sets dipping west-north-west and north-east respectively; these sets together define the backscarp of the landslide.

The geological feature of greatest interest in the slope is the presence of an extensive layer of kaolinitic clay, up to 0.6 m in thickness and associated with clay veining, derived from alteration of tuff. This layer undulates, having a northerly dip varying in angle from 10 to 25°; some of the undulation may be the result of offsetting by small-scale faulting. The characteristics of this kaolinitic layer has been studied in particular detail by GEO.

The kaolinitic clay layer has been interpreted as a shear zone parallel to the original bedding of the tuff. The original material within the layer could well have been altered and deformed during the intrusion of the Kowloon granite Pluton which outcrops about 70 m north of the landslide. There is a set of joints which dip northwards at 10° to 25°, approximately parallel to the clay layer. The basal surface of the landslide on which the displacement took place did not follow the kaolinitic layer precisely. In the upper part of landslide the surface is above the layer (within altered tuffs with kaolinite veins) but, in the lower part, layer and surface appear to be the same, although much of the clay was stripped off by the sliding debris.

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7.2 Soil and Rock Properties

A comprehensive series of classification, strength and consolidation tests has been carried out on the materials within the landslide that are relevant to the stability of the slope including the kaolinitic clay layer and veins, altered tuff and weathered volcanic joints.

A range of shear strength values has been established dependent on the relative proportion of kaolinite, and associated veining, with the altered tuff. The lower bound value of $\phi' = 22°$, $c' = 0$ was assigned to the situation where shearing was through soil with a higher kaolinite content, and $\phi' = 29°$, $c' = 0$ was considered representative of the kaolinite-rich altered tuff layer which formed the basal failure surface of the landslide.

7.3 Groundwater Conditions

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Two groundwater regimes are recognised at the site, a deeper regional water table, and a shallower perched water table retained by the kaolinitic clay layer at a higher level.

The regional water table, as based upon observations in boreholes drilled after the landslide, is 4 to 8 m below the kaolinitic clay layer. As noted earlier in this report (Section 2) the site is located on the side of a spur extending out from the higher ground to the south-west. In such a topographic situation the groundwater flow from the higher ground will tend to be diverted away the spur, and so any mound in the groundwater level beneath the spur will need to be sustained by local infiltration.

A perched water table was developed on top of the kaolinitic clay layer, and there is documentary evidence of minor seepage having been observed in the rock face at or above the level of this layer. Seepage, which increased with rainfall, was also observed above the kaolinitic clay layer within the landslide area during the investigations. As there was no evidence of significant seepage at the toe of the backscarp of the landslide, about 5 m above the kaolinitic clay layer, it is concluded that the perched water level was probably in the range 1 to 4 m above the kaolinitic clay layer.

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8. Conditions of Chai Wan Salt Water Service Reservoir and the Associated Water Main System

The Salt Water Reservoir is briefly described and, as based on the pumping record, it is recognised that there were no signs of abnormal operation nor major leakages before 0115 on 13 August 1995 which was the time of the second, main, failure. A leakage test was carried out in the west compartment (the east compartment is being kept empty for safety reasons) and no measurable leakage was detected.

A 21 m section of the 24-inch salt water main was severed during the landslide and water discharged for some time into the slope debris. All the dislodged pipe sections were retrieved and no signs of any old or sustained leakage through the pipes or joints were found.

These observations demonstrate that gross leakage from the Salt Water Reservoir was not a factor in triggering the landslide.

Nevertheless, a programme of chloride determinations was carried out on both soil and water samples in order to establish the extent of any salt water seepage. Elevated chloride contents were found in the regional groundwater in boreholes DH9 about 20 m north of the Reservoir and DH16 about 90 m north-east of the Reservoir indicating that seepage has almost certainly occurred from the Reservoir over a period of years. There are also significant chloride contents in seepages discharging from the cut slope west of the landslide, and in a seepage above the landslide basal surface. Soil samples from the extreme western part of the landslide have higher chloride contents which probably represent contamination by the salt water from the severed pipeline. In contrast the chloride contents of soil samples from the main part of the landslide and the eastern back scarp are lower and these soils were almost certainly not contaminated by the discharge from the fractured 24-inch water main. The presence of chloride in the seepages, above expected natural levels, and in the soil samples from the main part of the landslide basal surface, provides confirmatory evidence of the role of the kaolinitic clay layer in developing a perched water table condition.

9. Engineering Analyses

The engineering stability analyses carried out are based on the assumption that translational sliding took place on a surface inclined at 20° out of the rock slope with detachment from the backscarp on weathered volcanic joints. This is a reasonable geological model for the site conditions.

If no water pressure was assumed to be acting on the slide mass then failure would occur if the angle of shearing

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resistance (ϕ') was 25° or less. Such a value for ϕ' is within the range of measurement but less than the value (29°) considered to be a reasonable operational value.

The analysis has assumed that perched water conditions would give rise to effective water heads on the slide surface of 1, 2 and 4 m, which would theoretically give rise to failure if the values of ϕ' for the slide surface were 26.5°, 28.0° and 31.5° respectively. Hence it is concluded that, on the basis of reasonable assumptions regarding the perched water table conditions, instability could theoretically occur through translational failure.

The backscarp of the landslide is defined by two sets of steep joints and behind the crest of the failure there is some surface cracking on directions similar to these sets, although the extent of these cracks has yet to be explored laterally or in depth. In conditions of heavy rain such cracks may become rapidly filled with water and so an alternative groundwater assumption can be made that, for a ϕ' of 29° for the slide surface, the slide mass could be displaced by water pressure acting within steep, open joints within the whole of the backscarp. Under such circumstances the upper 9 to 10 m of the joint system would have to be filled with water for theoretical instability to be achieved. It is possible that this process could be a contributory factor to the slope failure process.

10. Diagnosis of the Landslide

This section provides a comprehensive overview of the slope failure process as derived from the results of the investigations carried out. The writer is in agreement with this analysis.

The Fei Tsui landslide was the result of the effect of elevated groundwater conditions, following an exceptionally wet period of some days length and during an intense rainstorm, on a continuous, bedding-controlled layer infilled by relatively weak materials. Several of the features associated with the landslide are unusual although not exceptional: the extremity of the rainfall event and its antecedent history, the existence of a bedding-controlled planar layer in volcanic rocks, the presence of significant amounts of kaolinitic clay, and the size of this translational failure.

The first, small landslide could have resulted from toppling or a combination of sliding and toppling. This initial movement may have had a role in relation to a progressive

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unlocking of the process which then permitted the subsequent, larger failure to take place a short time after. Within that period of twenty minutes there may have been a series of progressive, albeit minor readjustments which contributed to the eventual failure mechanism and relatively long runout of the failed debris.

Although evidence has been identified of chloride contamination in the groundwater and soil which can almost certainly be ascribed to long-term seepage from the Salt Water Reservoir, there is no evidence that gross leakage contributed in any way to the triggering of the landslide.

11. Conclusions

The conclusions are a very brief re-statement of the cause of the landslide with which the writer is in agreement."

4. CONCLUSIONS ON GEOTECHNICAL ENGINEERING OFFICE REPORT

The investigation carried out by the Geotechnical Engineering Office into the Fei Tsui Road landslide has been comprehensive, having been executed in a professional manner. The Report accurately reports the conclusions of the investigation, and reaches a logical conclusion as to the contributory factors to, and cause of, the landslide. The writer is in agreement with the report on all essential matters.

5. LESSONS TO BE LEARNT

The Fei Tsui landslide has not identified any new feature not previously recognised in geological or landslide prevention practice within Hong Kong. However, there are features relevant to the cause of the landslide and the history of the site which deserve to be highlighted.

5.1 Structural Controls on Landsliding in Volcanic Rocks

The failure surface of the Fei Tsui Road landslide was bedding-controlled and no translational failures of this scale and type have been reported previously in Hong Kong. As the volcanic rocks of Hong Kong have been folded, any bedding that is present may result in unfavourable combinations of bedding dip and direction, and topography. There is the possibility that the type of condition which exists at Fei Tsui Road may occur elsewhere.

The review of the site history (Appendix A, A.2.2) has identified previous documentation on the site. A contemporary report on the slope written at the time of site formation refers to "a prominent horizontal weathered seam midway between the road and the

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berm (plate 8)" "approximately 100 mm wide and at least 50 m long". The "plate 8", taken in 1976, which is presented as Plate 2 to the GEO Report, illustrates the site of the Fei Tsui landslide. The very obvious planar geological structure observable in this 1976 photograph is presumably the "prominent horizontal weathered seam". However, it is obvious that this structure is not horizontal but rather it is a throughgoing feature dipping out of the face at an angle which might potentially lead to sliding. Even though the slope was looked at by other organisations subsequently, the possibility that this geological structure might induce a large translational failure does not appear to have been explored.

With this hindsight knowledge, it is necessary to recognise that continuous discontinuities dipping outwards at angles of 20° (and possibly less) from rock faces can result in translational failures.

5.2 Mineralogical Controls on Landsliding in Volcanic Rocks

The presence of kaolinite is not unusual in altered or weathered rocks in Hong Kong. In this case what is unusual is that the kaolinite was present as a relatively thick, continuous layer. Such layers might not be recovered by conventional diamond drilling, being washed away within the water flush, unless their presence was suspected and special measures taken to recover the material.

At this stage the factors which resulted in the development of the kaolinitic clay layer associated with altered tuffs at Fei Tsui Road cannot be stated with sufficient precision to enable such materials to be predicted.

5.3 Multiple Small-scale Failures

Four small-scale failures occurred at Fei Tsui Road over the period of two decades since the original site formation. In some other cases of slope instability in rock, such failures have proved to be the harbinger of an eventual larger failure.

Methods should be identified, within the GEO integrated approach towards landslide prevention, whereby multiple failures over a period of years at a single site can be identified for appropriate forensic study.

5.4 Leakage from Service Reservoirs

Although in this case gross leakage from the Salt Water Reservoir did not contribute to triggering the landslide, there was evidence of soil and ground water contamination through long term seepage. The possibility needs to be borne in mind that leakage from service reservoirs can influence local groundwater conditions, and this may in turn influence slope stability.

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6. REFERENCES

GEO (1996). Report on the Fei Tsui Road Landslide of 13 August 1995. Volume 2: Findings of the Landslide Investigation. Geotechnical Engineering Office, Hong Kong, 68 p.

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VOLUME 2: FINDINGS OF THE LANDSLIDE INVESTIGATION

Geotechnical Engineering Office
Civil Engineering Department
Hong Kong Government

This report was originally produced in February 1996
as Report on the Fei Tsui Road Landslide of 13 August 1995

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EXECUTIVE SUMMARY

On 13 August 1995, a landslide occurred at the slope opposite Chai Wan Baptist Church, Fei Tsui Road, and resulted in one fatality and one injury. The landslide involved the sudden collapse of part of registered cut slope No. 11SE-D/C42 and the land above the crest of the cut slope adjacent to the Chai Wan Salt Water Service Reservoir.

A comprehensive investigation into the landslide was carried out by the Geotechnical Engineering Office (GEO) during the period August to December 1995. This detailed study included review of documentary information, analysis of rainfall records, interviews with witnesses to the landslide, site survey, ground investigation, examination of the role of the service reservoir and the water main system in the landslide, theoretical stability analyses and diagnosis of the causes of failure.

The investigation concluded that the landslide was probably primarily caused by an increase in water pressure in an extensive and weak layer of clayey soil in the slope, following the extremely heavy and prolonged rainfall that preceded the failure.

Details of the investigation and its findings are given in this report on the landslide.

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CONTENTS

1. INTRODUCTION

In the early morning of 13 August 1995, a landslide occurred on the slope opposite Chai Wan Baptist Church, Fei Tsui Road, Chai Wan (Plate 1 & Figure 1). A 90-m long section of Fei Tsui Road was buried by the debris from the landslide. The landslide resulted in one fatality, and one other person was slightly injured.

After the landslide, the Geotechnical Engineering Office (GEO) of the Civil Engineering Department conducted a detailed investigation into the failure. A Progress Report was issued on 28 September 1995 (Geotechnical Engineering Office, 1995).

The investigation was carried out during the period August to December 1995, and comprised the following key tasks:

(a) review of all known relevant documents relating to the development of the site and the sequence of events leading up to the landslide,

(b) analysis of the rainfall records,

(c) interviews with witnesses to the landslide,

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

(e) geological mapping,

(f) execution of a comprehensive programme of ground investigation by drilling, insitu testing and laboratory testing,

(g) examination of the role of the Chai Wan Salt Water Service Reservoir and the water main system in the landslide, and

(g) theoretical stability analyses of the slope that failed.

This report presents the findings of this investigation. Full details of the investigation work undertaken and the results obtained are contained in a set of documents, which is placed in the Civil Engineering Library on the First Lower Ground Floor of the Civil Engineering Building.

2. DESCRIPTION OF THE SITE

The location of the landslide at Fei Tsui Road is shown in Figure 2. The ground that failed comprised part of a cut slope and the land above the crest of the cut slope adjacent to the Chai Wan Salt Water Service Reservoir.

The cut slope was registered as No. 11SE-D/C42 by the consultants engaged by

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Government to prepare the Catalogue of Slopes in 1977. Before the landslide, the cut slope inclined towards the north at an average angle of about 60° to the horizontal, with a maximum height of about 27 m. Rock was exposed at the lower part of the cut slope, and the upper portion was covered with chunam. A photograph of the cut slope taken after its formation in mid-1970s, which shows the condition of the rock face and the chunamed slope surface at the time, is shown in Plate 2. Photographs of the site taken in 1994 show that the cut face was covered extensively by unplanned vegetation (Plate 3).

The Chai Wan Salt Water Service Reservoir is a mass concrete water retaining structure located immediately to the southwest of the landslide area. The reservoir measures about 25 m by 40 m by 5.4 m deep and is not roofed. The reservoir is surrounded by a fill embankment (Figure 1), with a maximum height of about 6 m and an inclined surface of about 30° to the horizontal. The embankment was registered as fill slope No. 11SE-D/F27 in the Catalogue of Slopes in 1977.

This fill embankment and the ground between it and the cut slope were covered by vegetation before the landslide.

To the southeast of the landslide the natural ground forms a spur trending east-northeast, with ground levels falling southwards towards a valley. This spur is an abandoned squatter area, on which there are remains of concrete floor slabs and paved floor surfaces (Figure 3).

In front of the cut slope that failed on 13 August 1995 is a strip of flat open space, about 12 m wide, adjacent to Fei Tsui Road. Apart from the western portion, which was allocated to the Drainage Services Department (DSD) on 27 April 1995 for use as a temporary storage compound, the strip of land was vacant (Figure 1).

The section of Fei Tsui Road below the landslide was about 7.3 m wide, with a pedestrian pavement about 3.3 m wide along the northern side of the road.

The probable layout of the surface drainage system of the site, as interpreted from the available documentary records and site observations after the landslide, is shown in Figure 3. It was found from inspections after the landslide that the surface drainage channels on the unfailed section of the cut slope to the east of the landslide were blocked by humus and black-stained silty soil (Plate 4). The crest channel on the cut slope to the west of the landslide was partly filled with fallen leaves and silty soil, to a depth of about 20 mm.

The ground that failed in the landslide comprises Government land. The western part of the cut slope is allocated to the DSD, and part of the land above the cut slope is within the site boundary of the Chai Wan Salt Water Service Reservoir (Figure 1).

Regarding slope maintenance, the DSD advised that “general site clearance including the removal of debris on the surface channels within the DSD’s allocated area was carried out from 29.4.1995 to 3.5.1995. Regular site inspection have been carried out on the allocated area to ensure that all the surface channels are functioning properly without blockage”. As for maintenance works at the service reservoir, the Water Supplies Department (WSD) advised that the reservoir “has been regularly maintained” and the most recent “Grass cutting and clearance of surface channels” before the landslide was carried out on 30 December 1994.

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3. DESCRIPTION OF THE LANDSLIDE

A photograph of the landslide taken on the morning of 13 August 1995 is shown in Plate 1. A cross-section through the landslide location is given in Figure 4.

According to the accounts of witnesses, the landslide involved two phases of failure. The main failure occurred at about 1:15 a.m. on 13 August 1995 and was preceded by a minor failure about 20 minutes earlier. The average depth of the landslide was about 15 m, which is deep compared with other rain-induced slope failures in Hong Kong. About 14 000 m^3 of debris were released in the landslide. As shown in Figure 2, the landslide debris covered the lower part of the cut slope, the open space in front of the slope and Fei Tsui Road, with some deposited onto the playground across the road. Part of the debris piled up against the south-western corner of the Chai Wan Baptist Church to a maximum height of about 6 m. The maximum horizontal travel distance of the debris was about 70 m, as measured from the crest of the landslide. The maximum width of the debris mound was about 90 m.

The landslide debris comprised predominantly coarse gravel-size to boulder-size joint-bound blocks of moderately and highly decomposed tuff in a matrix of clayey silty fine sand to sandy clayey silt, together with some man-made materials (Figure 5). The latter included sections of 24-inch and 6-inch diameter asbestos cement pipes, broken pieces of concrete surface channel, broken sections of a masonry wall, concrete blocks and catchpits, chain-link fencing, galvanized iron pipes, cables and damaged lamp posts. Many of the broken channels were filled with humus and black-stained soil (Plate 5), which indicates that they had been blocked for some time before the landslide.

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Some cracks, with a maximum width of about 15 mm, were observed at the paved ground surface next to the crest of the failure in the abandoned squatter area. However, it is not known whether the cracks were formed before or after the landslide.

A review of the available records of landslides in Hong Kong since 1984 reveals that about 0.1% of the reported incidents had a failure volume of more than 5000 m^3. The Fei Tsui Road landslide, with a volume of about 14 000 m^3, was very unusual in terms of the scale of the failure. It is indeed the largest reported fast-moving cut slope failure in Hong Kong over the last decade.

The mobility of a landslide debris can be gauged by the inclination of the line that joins the distal end of the debris and the crest of the landslide. The smaller the angle, the more mobile is the debris. For common rain-induced landslides on cut slopes in Hong Kong, this angle is generally greater than 30° (Wong & Ho, 1996). However, the angle was 24° for the Fei Tsui Road landslide, which indicates that the landslide debris in this failure was more mobile than is commonly observed in cut slope failures.

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A truncated 24-inch diameter underground salt water main was exposed near the crest of the landslide scar (Plate 1 & Figure 2). Another truncated underground water main, which is 6 inches in diameter, was also exposed near the crest of the back scarp at about 4 m to the east of the truncated 24-inch water main (Figure 5). Witnesses observed that water was discharging from the 24-inch water main onto the landslide area on the morning of 13 August after the failure. The witnesses interviewed by the GEO, however, did not report having noticed the presence of the truncated 6-inch water main.

4. HISTORY OF THE SITE

The site history, summarised in Appendix A, was traced from the aerial photographs of the site and from a review of other available documentary information.

The earliest available aerial photographs, which were taken in 1924, show that the site was situated on an east-northeast trending spur and was undeveloped at that time. The Chai Wan Salt Water Service Reservoir was constructed in 1959. The cut slope was formed between 1972 and 1976 by the Architectural Office (AO) (reorganised as the Architectural Services Department in 1986), as part of construction for the Hing Wah Estate Phase II development.

A number of small-scale landslides had occurred previously at the cut slope (Figure 6).

Two landslides, which occurred in 1987 (Plate 6) and in 1993 (Plate 7), with failure volumes of about 50 m^3 and 30 m^3 respectively, were reported to the Geotechnical Control Office (GCO) (renamed GEO in 1991). In each incident, the landslide occurred at the upper part of the cut slope, and the base of the failure daylighted at about 10 m above the level of Fei Tsui Road. The landslide debris came to rest in the flat open space in front of the slope, except that some windows of the Baptist Church were reported to have been damaged by small pieces of fly rock in the 1993 landslide.

A slope failure can be observed in the 1985 aerial photographs, and another landslide in 1986 is indicated on a plan in a GEO file. No other information about these incidents can be found. The scale of these two landslides was apparently smaller than the 1987 and 1993 incidents referred to in the preceding paragraph.

5. ANALYSIS OF RAINFALL RECORDS

An automatic raingauge (No. H14) is located on the roof of Wo Hing House in Hing Wah Estate, about 220 m to the north of the landslide (Figure 1). The daily rainfalls recorded by the raingauge in July and August 1995, together with the hourly rainfalls from 11 to 13 August 1995, are shown in Figure 7.

Rain was heavy from the morning of 12 August to the time of the landslide. The 12-hour and 24-hour rainfalls before the landslide were 231 mm and 370 mm respectively. The peak 60-minute rainfall was 94.5 mm, which was recorded between 11:30 p.m. on 12 August and 0:30 a.m. on 13 August.

A total of 1 303 mm of rain was recorded by raingauge No. H14 in the 31 days before the landslide. This exceeds the highest calendar monthly rainfall ever recorded by the raingauge at the Royal Observatory since records began in 1884. Analysis of the return periods of the rainfall intensities of this rainstorm for different durations based on historical rainfall data at the Royal Observatory shows that the 31-day rainfall was the most extreme, with a corresponding return period of about 95 years.

Shown in Figure 8 is a comparison between the pattern of the rainfall prior to the 1995 landslide and those of previous major rainstorms affecting the site since installation of

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raingauge No. H14 in 1979. It can be seen that the rain that preceded the landslide was the highest recorded by the raingauge for durations in excess of 15 days. For rainfall durations of 7 days or less, the rainfall intensities of this rainstorm were comparable to those experienced previously.

6. SEQUENCE OF FAILURE

The sequence of failure was re-constructed from accounts given by twelve witnesses, from records of the incident by the Royal Hong Kong Police Force and the Fire Services Department (FSD), and from site observations made by the GEO after the landslide.

The Fei Tsui Road landslide comprised two phases of failure. At about 0:55 a.m. on 13 August 1995, a small landslide occurred at the eastern part of the cut slope, opposite the Chai Wan Baptist Church. A police officer reported the incident to the Police Chai Wan Regional Console at 0:56 a.m. This failure was sudden and likely to have involved the upper part of the cut slope at the eastern side of the site, probably at two locations that were a few metres apart. The debris was deposited within the vacant space, and the scale of failure was probably in the order of several tens of cubic metres. It is possible that this first phase of the landslide was similar, in terms of the size and extent of failure, to the landslides reported in 1987 and 1993.

At about 1:15 a.m., the second phase of the failure, which was the main failure that resulted in casualties, took place suddenly. This failure extended to the whole of the landslide area, including the cut slope and the ground above. The debris slid down and buried the open space and Fei Tsui Road in a matter of seconds. A 24-inch diameter water main was exposed near the crest of the back scarp, and water was observed on the morning of the landslide to be discharging from this truncated water main onto the landslide area. This failure was reported to the Police by a member of the public at 1:17 a.m., and the FSD was informed by the Police at 1:18 a.m.

7. SUBSURFACE CONDITIONS AT THE SITE

7.1 General

The subsurface conditions at the site were determined using information from desk and field studies. The desk study comprised a review of existing data, whilst the field study included geological mapping and ground investigation.

Geological mapping of the site commenced on 14 August 1995 and continued as the landslide debris was removed during emergency repair works and during ground investigation.

Ground investigation commenced on 17 August 1995 and the majority of the works were carried out after the completion of the emergency repair works in mid-September 1995. The ground investigation comprised 11 vertical drillholes, 3 inclined drillholes, 2 observation wells, 9 trial pits and 4 trial trenches (Figure 9).

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7.2 Geology

The geological features observed at the site after the landslide by the Hong Kong Geological Survey are shown in Figure 10. A section showing the typical stratigraphy through the landslide site is shown in Figure 11.

The geology at the landslide area comprises weathered volcanic rock overlain by a layer of fill up to about 3 m thick at the top of the slope. The weathered rock consists of completely to slightly decomposed tuff. The thickness of completely and highly decomposed tuff ranged from about 4 m on the eastern side of the landslide area up to about 11 m on the western side.

The rocks at the site were mapped by the Hong Kong Geological Survey as the Shing Mun Formation of the Repulse Bay Volcanic Group, according to the 1:20 000 scale geological map for the area (Geotechnical Control Office, 1986). The slightly decomposed rock at the base of the cut slope was sampled during the geological survey, and a detailed description of a thin section of a hand specimen is given in the geological memoir (Strange & Shaw, 1986). The rock is described as a eutaxitic, lapilli-bearing tuff. Geological work on site during the present investigation has confirmed that this typifies the predominant lithology at the site.

The weathered tuff is pervasively jointed. Two persistent, closely-spaced to medium-spaced, generally tight, rough and planar, and steep (60° to 85°) joint sets dipping west-northwest and northeast are dominant. These joint planes form the lateral release surfaces and the back scarp of the landslide (Plate 8), and on inspection on 14 August 1995, they appeared to have been only recently exposed. A third low-angle set of joints, dipping predominantly to the north at 10° to 25°, was also identified.

Across the site, the lower 5 m to 7 m of the cut slope is predominantly composed of moderately to slightly decomposed tuff. It did not form part of the landslide (Plate 9).

A notable feature of the site is a laterally-extensive layer of kaolinite-rich altered tuff dipping approximately to the north at about 10° to 25°. It is considered that the basal slip surface of the landslide was developed mainly along this layer. Across the site, the layer is offset by a series of small north-trending faults (Figure 10), which have the effect of stepping the layer upwards to the west. Vertical offsets are mostly less than a metre at each fault, but one fault at the western side of the landslide has a vertical offset of up to about 3 m.

At the unfailed portion of the cut slope adjoining the eastern edge of the landslide, the kaolinite-rich altered tuff layer is about 0.5 m thick and is overlain by moderately to slightly decomposed tuff. At this locality, the layer dips approximately 20° north and is highly kaolinised and completely decomposed, with abundant kaolinite veins (Plate 10). The thickness of the veins ranges from 2 mm to 20 mm, and some of them are sub-parallel to the orientation of the layer.

When the landslide debris was removed, similar altered tuff was found in the eastern and central part of the base of the landslide (Plate 11). The thickness of the layer in this area is up to about 0.6 m, although the upper part has been eroded in places by the landslide.

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In the western part of landslide, the landslide basal surface exposed in the trial trench is locally steeper, dipping at about 30° to the north. The altered tuff layer at the basal surface has been substantially scoured by the failure, with moderately to slightly decomposed tuff observed directly beneath the landslide debris. Remnants of the layer (Plate 12) were found on top of moderately to slightly decomposed tuff just below the landslide basal surface near the back scarp and the toe of the landslide.

At the base of the western back scarp, the altered tuff layer is more 'diffuse'. It is thicker (about 3 m), less kaolinised, and although predominantly completely to highly decomposed, includes some moderately decomposed material. Kaolinite veins are comparatively less abundant, commonly occurring near the top of the layer (Plate 13).

The layer was identified in the drillholes to the east and west of the failure. The layer was also identified in drillholes close to the back scarp but is absent in a drillhole about 16 m to the south. This shows the possibility of the layer extending laterally into the ground adjoining the landslide, but the degree of alteration and amount of kaolinite-veining may vary at different locations.

Regarding the origin of the altered tuff layer, the Hong Kong Geological Survey advises that several processes have contributed to the development of the clay-rich layer. The layer originated as a shear zone parallel to the original bedding and/or fabric of the tuff. Early alteration and faulting of the layer were probably caused by the intrusion of the nearby Kowloon Pluton (70 m to the northwest of landslide). Differences between eastern and western parts of the layer are probably due to original differences in lithology of this heterogeneous sequence, and consequent differences in shearing and the effects of hydrothermal fluids. Most recently, the layer has, like other parts of the rock mass, been affected by a long period of near-surface weathering.

7.3 Soil and Rock Properties

A comprehensive series of geotechnical laboratory tests was conducted on soil and rock samples retrieved during the ground investigation. The tests included particle size distribution tests, Atterberg limits tests, direct shear tests, triaxial compression tests and oedometer tests. These tests aimed to determine the geotechnical properties of the kaolinite-rich altered tuff and the weathered volcanic joints, which formed the basal slip surface and the back scarp respectively.

Particle size distribution and Atterberg limits tests were carried out in accordance with Chen (1994). The average fines (i.e. clay and silt) content of altered tuff excluding kaolinite veins was found to be 71%, and that of kaolinite veins in the altered tuff to be 92%. The plasticity index of the fines of altered tuff and kaolinite veins ranged from 9 to 18, and the liquid limit ranged from 29 to 50. This indicates that the materials were of low to intermediate plasticity, which is consistent with typical properties of kaolinite.

The shear strength properties of altered tuff were assessed by direct shear tests (Head, 1982) and consolidated undrained triaxial compression tests (Head, 1986). The properties of weathered rock joints were assessed by direct shear tests according to the method recommended by Hencher & Richards (1989). The test results and the shear strength

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parameters of the materials determined from the line of best-fit by the least squares method, are shown in Figures 12 & 13.

For altered tuff samples without kaolinite veins, the angle of shearing resistance (ϕ') was found by direct shear tests to be 34°, with cohesion intercept (c') of 10 kPa. For altered tuff samples with kaolinite veins aligned to the direction of shearing in direct shear tests, the average ϕ' was 29°, with zero c'. The lower-bound ϕ' value was 22°, which corresponds to the situation for which the shearing was through soil with high clay content.

The majority of the specimens tested were remnants of kaolinite-rich altered tuff recovered from trial trenches after the landslide. There is a possibility that the part of the altered tuff layer which actually controlled the landslide, and which consequently was removed in the failure, was weaker than the samples tested. In view of the extensive presence of kaolinite veins, many of which were adversely oriented in the altered tuff layer with respect to the landslide direction, the shearing resistance of the layer would have been governed principally by the kaolinite veins. Overall, for the purposes of theoretical stability analyses (Section 9), the average strength parameters of the altered tuff samples with kaolinite veins (i.e. ϕ' of 29° and c' of zero) are considered representative for the kaolinite-rich altered tuff layer that formed the basal slip surface of the landslide
.

The average ϕ' for weathered volcanic joints was found by direct shear tests to be 35°, with zero c'. These parameters are considered representative for the persistent and rough and planar joints that formed the back scarp of the landslide.

The consolidation properties of the samples taken from the altered tuff layer, which are expressed as a coefficient of consolidation assessed from oedometer tests (Head, 1982) and from the consolidation phase of direct shear tests, ranged from 28 m^2/year to 172 m^2/year. Given that the layer was generally less than one metre thick, it is considered that drainage would have taken place sufficiently rapidly in response to changes in loading and groundwater conditions, such that there would have been no significant excess pore water pressure at the onset of failure.

In addition to the soil and rock tests described above, standard chemical analyses were undertaken on soil and water samples to help assess the likely source of water that existed in the ground. The findings are described in Section 8.

7.4 Groundwater Conditions

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

(a) pre-landslide groundwater monitoring data for two drillholes (No. C8 & No. C9, Figure 6) over the period between March 1976 and June 1978, and for another two drillholes (No. P1 & No. P2, Figure 6) in February and March 1982,

(b) pre-landslide observations made in previous inspections of

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the slope that water was seeping out of the slope at a weathered seam midway between the toe of the slope and the berm halfway up the cut face (Sections A.2.2, A.2.3 & A.2.4, Appendix A),

(c) post-landslide groundwater monitoring data for 11 vertical drillholes (No. DH1 to No. DH10 & No. DH4A, Figure 9) and 2 observation wells (No. DH16 & No. DH17, Figure 9) over a period between September and December 1995, and

(d) post-landslide observations that water was seeping out of the landslide debris deposited above the altered tuff and exposed in the trial trenches, that the extent and amount of seepage increased with rainfall, and that there were no signs of significant seepage at the back scarp exposed in the landslide.

Based on the above information, it was postulated that two groundwater regimes existed at the site at the time of the landslide, viz. a regional groundwater table within the rock mass below the altered tuff layer, and a perched water table in the weathered volcanics overlying the altered tuff layer.

The groundwater monitoring results indicate that the regional groundwater level was about 4 m to 8 m below the basal surface of the landslide. It is considered that this groundwater table was unlikely to have been above the base of the landslide at the time of the failure, and hence it could not have had any significant effect on the landslide.

The perched groundwater regime operating in the altered tuff layer and the ground above is likely to have been an important factor in causing the landslide. The presence of such a perched water table is supported by seepage observations before and after the landslide. It is also consistent with the geological setting of the site. The altered tuff layer, and in particular the interface between the layer and the underlying weathered rock where water flow is confined to rock joints, constitutes a low-permeability boundary. This boundary would have impeded downward flow of water that entered the ground from surface infiltration during heavy rain and from other sources, and resulted in the development of a perched water table.

The lack of signs of significant seepage at the back scarp exposed in the landslide suggests that the perched water table was unlikely to have built up to the level of the exposed back scarp, which is about 4 m to 5 m above the altered tuff layer. As a best estimate, the perched water level was in the range of 1 m to 4 m above the altered tuff layer when the landslide took place.

Surface infiltration could also have resulted in the ingress of rain water into the weathered rock joints at or near the back scarp, thereby causing a transient elevated water pressure in the rock joints and adverse effects on the stability of the slope.

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8. CONDITIONS OF CHAI WAN SALT WATER SERVICE RESERVOIR AND THE ASSOCIATED WATER MAIN SYSTEM

The layout of the Chai Wan Salt Water Service Reservoir and its water main system near the landslide area is shown in Figure 14. The reservoir consists of four mass concrete side walls and a floor slab, with a central concrete partition dividing the reservoir into east and west compartments. According to WSD’s records, two underground water mains traversed the western part of the site before the landslide. The 24-inch diameter water main was constructed of asbestos cement pipes about 4 m long, connected by joints which comprised asbestos cement sleeves and rubber sealing rings. The water main supplied salt water from the service reservoir to the Chai Wan area. The 6-inch diameter water main had been abandoned and capped off at a point near the toe of the cut slope (Figure 14) since 1986.

The salt water service reservoir was inspected by the WSD and GEO on the afternoon of 13 August 1995. There was no apparent movement at any of the exposed joints of the side walls of the reservoir.

On 21 August 1995, the GEO received a report (WSD, 1995) on the landslide prepared by the WSD on 16 August 1995. The report stated that “There was no previous record of burst/leak in the affected 24"(600mm) and 6"(150mm) S.W. mains. Pressure in these sections of pipes very close to the outlet of the S/R is estimated to be less than 10 metres according to the design”.

WSD’s records of water levels in the reservoir before the landslide and the number of pumps in use for pumping water to the reservoir are shown in Figure 15. No signs of any abnormal operation or gross leakages indicated by a rapid drop in water level before the time of the main failure can be seen from the records. WSD (1995) stated that “Basing on the past inspection record and up to date operation record of the Chai Wan S.W. S/R, Chai Wan S.W. P/S and the supply and distribution network, it can be concluded that the service reservoir, the pumping station and the associated water mains were under normal operating conditions and were performing satisfactorily up to the time when the landslide occurred”.

A leakage test was carried out on the west compartment of the reservoir by the WSD on 17 August 1995. This involved filling the compartment with water to 1.3 m deep and monitoring the water level over a period of four hours. According to the WSD, the water level monitoring was accurate to 0.1 mm, which corresponded to a volume of about 0.05 m^3 in the compartment. No measurable loss was detected by the WSD in the leakage test. No leakage test was carried out on the east compartment, which has been kept emptied since the landslide for safety reasons.

A 21-m long section of the 24-inch diameter water main was truncated during the landslide. All the severed pipe sections of the water main were retrieved from the landslide debris, and were jointly inspected by the WSD and GEO on 7 September 1995. The pipes were found to be generally intact, apart from some fresh breakages.

According to WSD's records, the section of the abandoned 6-inch water main that was severed in the landslide was connected to the service reservoir before the slope failure. Only 2.4 m of the water main were recovered by the WSD from the debris and less than 20 m of the water main were identified by the GEO from photographic records of the landslide debris.

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This is far short of the approximate 55-m length according to WSD's records of the water main alignment (Figure 14). Hence, there are uncertainties about the actual alignment and condition of this water main.

For the purposes of assessing the likely extent of any salt water ingress into the ground, chemical tests were carried out on 37 soil samples and 44 water samples to determine the chloride content, in accordance with the procedures given in American Public Health Association (1992) and BSI (1990) respectively.

The results of the tests are shown in Figure 16. Salt water taken from the service reservoir had a chloride content of 11 000 to 19 000 mg/l. The chloride content of a water sample taken from the stream course in the valley to the southeast of the landslide, where any influence of seepage from the reservoir would have been minimal, was 42 mg/l. These may be taken as references for assessment of the degree of salt water ingress into the groundwater in the vicinity of the landslide.

A high chloride content of 4 500 mg/l was found in two water samples collected from about 8 m and 11 m below the ground surface from a drillhole about 20 m to the north of the reservoir (Figure 16). This indicates probable seepage of salt water from the service reservoir. The chloride content of water samples collected from two drillholes across Fei Tsui Road and from seepage locations in the vicinity of the landslide area ranged from 120 mg/l to 1 300 mg/l, which is symptomatic of presence of salt water within about 10% by volume in the groundwater. This suggests that seepage from the reservoir was a probable source of water in the vicinity of the landslide. The salt water seepage might have contributed to the wetting of the altered tuff layer, though it was probably a less significant source of water in the build up of a high perched water table compared with water infiltration during heavy rain.

The results of the soil chloride content tests are shown in Figure 17. High chloride contents ranging from 0.07% to 0.17% were found in soil samples recovered from the toe of the western part of the back scarp exposed after the landslide, along which salt water discharging from the reservoir through the truncated 24-inch water main would have flowed after the landslide. However, the chloride contents of soil samples taken from the back scarp and the debris at other locations in the landslide area were low, with an average chloride content of about 0.03%. Therefore, there are no indications that seepage from the reservoir was a significant source of groundwater in the vicinity of the landslide, which is consistent with the findings from chloride content tests on water samples.

9. THEORETICAL STABILITY ANALYSES

Theoretical stability analyses were carried out to assist the diagnosis of the mechanism and causes of the landslide. These analyses aimed to determine the likely range of shear strength parameters of the altered tuff layer at the time of the landslide, corresponding to different perched water levels above the layer.

Information obtained from the post-failure ground investigation field work, laboratory testing, and site observations and measurements was used in the analyses. A representative cross-section of the landslide site and the input parameters adopted in the analyses are shown

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in Figure 18. A perched water table up to 4 m above the altered tuff layer at the time of the landslide was assumed in the analyses.

The results of the analyses are summarised in Figure 19. For a factor of safety of 1.0, the angle of shearing resistance (i.e. ϕ') of the altered tuff layer was found to be 26.5°, 28° and 31.5°, for a perched water table of 1 m, 2 m and 4 m respectively. This range of ϕ' values agrees well with that determined from laboratory testing (Figure 13). Given the development of a perched water table, the slope would theoretically become unstable, involving a translational failure through shearing along the altered tuff layer.

Sensitivity analyses were carried out to examine the effects of water pressure on the weathered volcanic joints that formed the back scarp of the landslide. The best-estimate shear strength parameters were adopted for the altered tuff layer and the weathered volcanic joints. It was found that, in the absence of a perched water table, a 9 m to 10 m head of hydrostatic water pressure would be required at the rock joints to result in failure. Thus, the rock joints at the back scarp would have to have been filled with water to a great depth and lateral extent for this to have had a significant effect in triggering the landslide.

10. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

Based on the information collected from this investigation, it is postulated that the Fei Tsui Road landslide was caused by the two principal factors of :

(i) the extensive presence of weak material in the body of the slope, and

(ii) increase in groundwater pressure following the prolonged heavy rainfall.

The landslide was controlled by a persistent kaolinite-rich altered tuff layer which formed the basal slip surface, with the steep and adversely oriented planar joints in the weathered tuff forming the lateral release surfaces and back scarp of the landslide. It was found from the laboratory testing and geological mapping carried out after the landslide that the planar joints, and the altered tuff layer in particular, are much weaker than the bulk of the slope-forming material. These are considered to have combined to give rise to a deep translational failure mechanism involving shearing along the gently-dipping altered tuff layer.

The landslide occurred after a period of exceptional prolonged rainfall that was the heaviest recorded near the site since 1979 when the raingauge No. H14 was installed. The heavy rain is likely to have led to the development of a perched water table above the altered tuff layer. This scenario is supported by site observations, and it is consistent with the hydrogeological regime at the site. A rise in the perched water table would have increased the water pressure in the altered tuff and resulted in a reduction of the material shear strength, and thereby culminated in the landslide.

The possibility of the landslide being triggered by a build up of water pressure in the rock joints at or near the back scarp was also examined. Analyses have shown that, in the absence of a perched water table, the rock joint at the back scarp would have to have been

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filled with water to a great depth and lateral extent prior to the landslide. This scenario is considered less probable than that of the development of a perched water table. However, the possible presence of some water pressure within the rock joints cannot be excluded, and this might have combined with the perched water table in triggering the landslide.

The landslide involved two distinct phases of failure. Whilst the two factors described above are assessed to be the principal causes of the landslide, other factors that could conceivably have had some contribution in triggering the failure are discussed in the following paragraphs.

The first phase of the landslide occurred at about 0:55 a.m. on 13 August 1995. The failure was confined to the eastern part of the section of slope that slipped in the main landslide, with a failure volume in the order of several tens of cubic metres. Inadequate slope maintenance, with possible slope deterioration and convergent water ingress because of blockage of surface drainage channels, is a possible contributory factor to the first phase of local failure.

The second phase of the landslide at about 1:15 a.m. was the main failure. Partial loss of support and release of side restraint as a result of the first phase of the landslide could have contributed to the triggering of the main failure, although the possibility that the whole of the landslide area was on the verge of a global failure at the time when the local slope collapse occurred cannot be excluded.

There was no evidence of any abnormal operation of the service reservoir, nor any indication of gross leakage from the reservoir and the associated water main system before the main failure of the landslide.

The re-constructed sequence of events which it is believed led to the landslide is illustrated in Figure 20.

The Fei Tsui Road landslide was an unusual cut slope failure in terms of the size of the failure and the large travel distance of the debris. These are likely to be principally related to the presence of the extensive altered tuff layer that governed the landslide, which has not been commonly observed in other landslides in Hong Kong. The persistent and weak altered tuff layer, which was about 15 m below the crest of the cut slope, is thought to have rendered the large deep failure possible. The extremely heavy and prolonged rainfall preceding the landslide favoured the build up of water pressure over a large area at some depth below the ground surface, and hence triggered the large-scale failure.

The inclination of the line joining the distal end of the debris to the crest of a landslide reflects the debris mobility and the apparent angle of friction of the material that governs debris movement. For common cut slope failures in Hong Kong, this angle is generally between 30° and 40°, which is compatible with the range of ϕ' for typical Hong Kong soils and rocks (Wong & Ho, 1996). For the Fei Tsui Road landslide, the angle was only 24°. This value is comparable to the strength of the altered tuff layer determined from laboratory tests. Other possible contributory factors that might have led to a more mobile debris movement include the large volume of failure, and the gently-dipping nature of the basal slip surface, which resulted in a lower inclination trajectory with less energy loss of the debris upon impact with the ground in front of the slope.

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11. CONCLUSIONS

It is concluded that the 1995 Fei Tsui Road landslide was probably primarily caused by elevated water pressure in an extensive and low strength kaolinite-rich altered tuff layer in the slope, following the extremely heavy and prolonged rainfall that preceded the failure.

12. REFERENCES

American Public Health Association (1992). Standard Methods for the Examination of Water and Wastewater. (18th edition). Section 4 : Inorganic non-metallic constituents. American Public Health Association, 135 p.

Binnie (1977). Government of Hong Kong Landslide Study Phase IIC Report on the Stability of Slopes in the Chai Wan Area. Binnie & Partners (Hong Kong) Consulting Engineers for Government of Hong Kong, 64 p. plus 12 Plates, 6 Appendixes and 23 Drawings.

BSI (1990). Methods of Test for Soils for Civil Engineering Purposes. (BS1377 : 1990). Part 3 : Chemical and Electro-chemical Tests. British Standards Institution, London, 44 p.

Chen, P.Y.M. (1994). Methods of Tests for Soils in Hong Kong for Civil Engineering Purposes (Phase 1 Tests). Geotechnical Engineering Office, Hong Kong, 91 p. (GEO Report No. 36)

Geotechnical Control Office (1986). Hong Kong and Kowloon: solid and superficial geology. Hong Kong Geological Survey, Map Series HGM 20, Sheet 11, 1:20 000 scale. Geotechnical Control Office, Hong Kong.

Geotechnical Engineering Office (1995). Progress Report on the Investigation of the 13 August 1995 Landslide at Fei Tsui Road Chai Wan. Geotechnical Engineering Office, Hong Kong, 16 p.

Head, K.H. (1982). Manual of Soil Laboratory Testing. Volume 2 : Permeability, Shear Strength and Compressibility Tests. Pentech Press, New York, 747 p.

Head, K.H. (1986). Manual of Soil Laboratory Testing. Volume 3 : Effective Stress Tests. Pentech Press, 1238 p.

Hencher, S.R. & Richards, L.R. (1989). Laboratory direct shear testing of rock discontinuities. Ground Engineering, vol. 22, no. 2, pp 24-31.

Strange, P.J. & Shaw, R. (1986). Geology of Hong Kong Island and Kowloon. Geotechnical Control Office, Hong Kong, 134 p. (Hong Kong Geological Survey Memoir No. 2)

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Water Supplies Department (1995). Report on the Landslide at Fei Tsui Road, Chai Wan on 13.8.95. Water Supplies Department, Hong Kong, 7 p. plus 3 Drawings, 6 Appendixes and 11 Plates.

Wong, H.N. & Ho K.K.S. (1996). Travel distance of landslide debris. Proceedings of the Seventh International Symposium on Landslides, Trondheim, Norway, in press.

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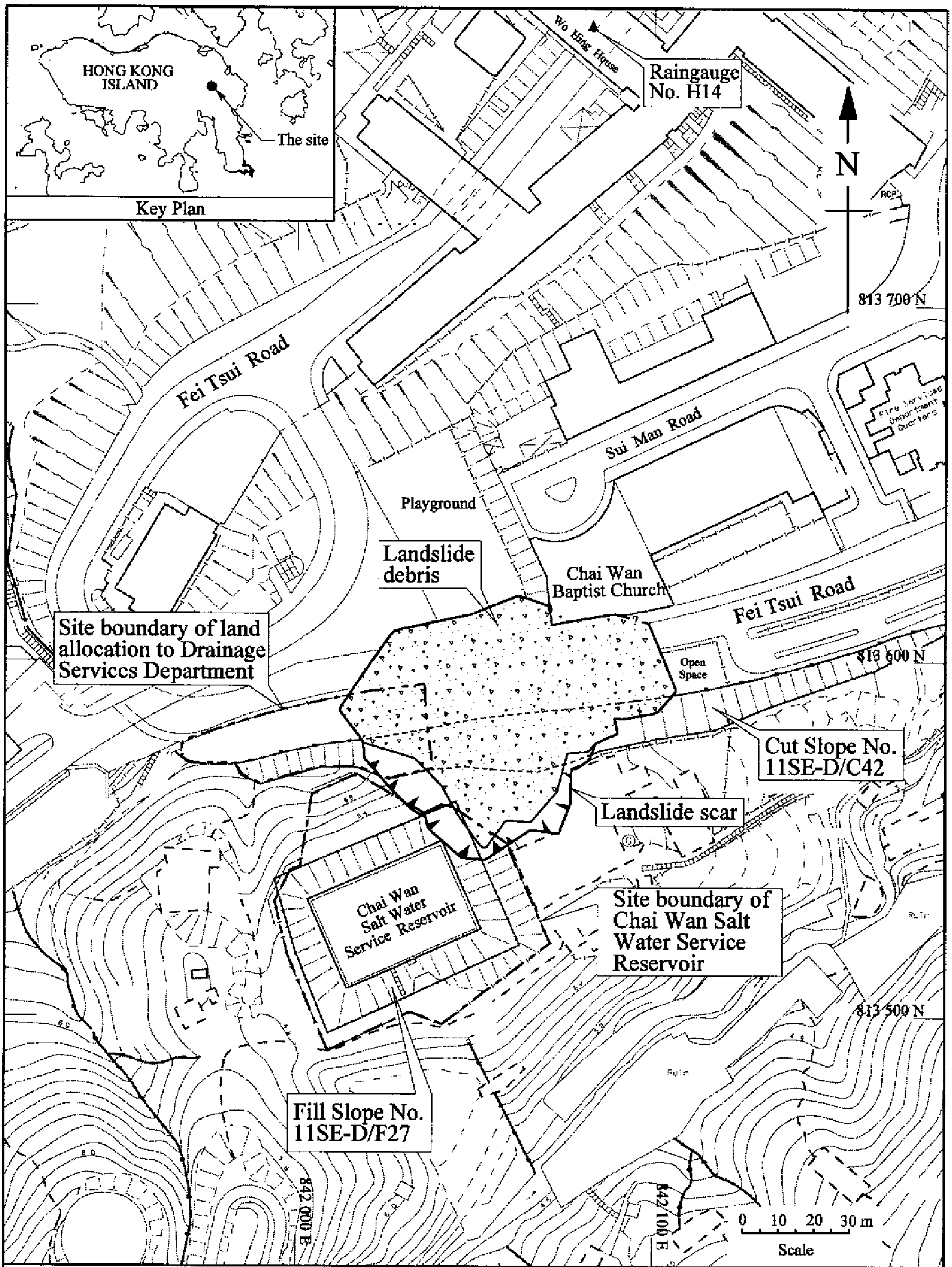

Notes :
(1) Base map is extracted from Survey Sheets No. 11-SE-18D & No. 11-SE-19C, dated 29 September 1992, scale 1 : 1000.
(2) Site boundary of Chai Wan Salt Water Service Reservoir is according to the location of the boundary fence of the service reservoir.

Figure 1 - Site Location Plan

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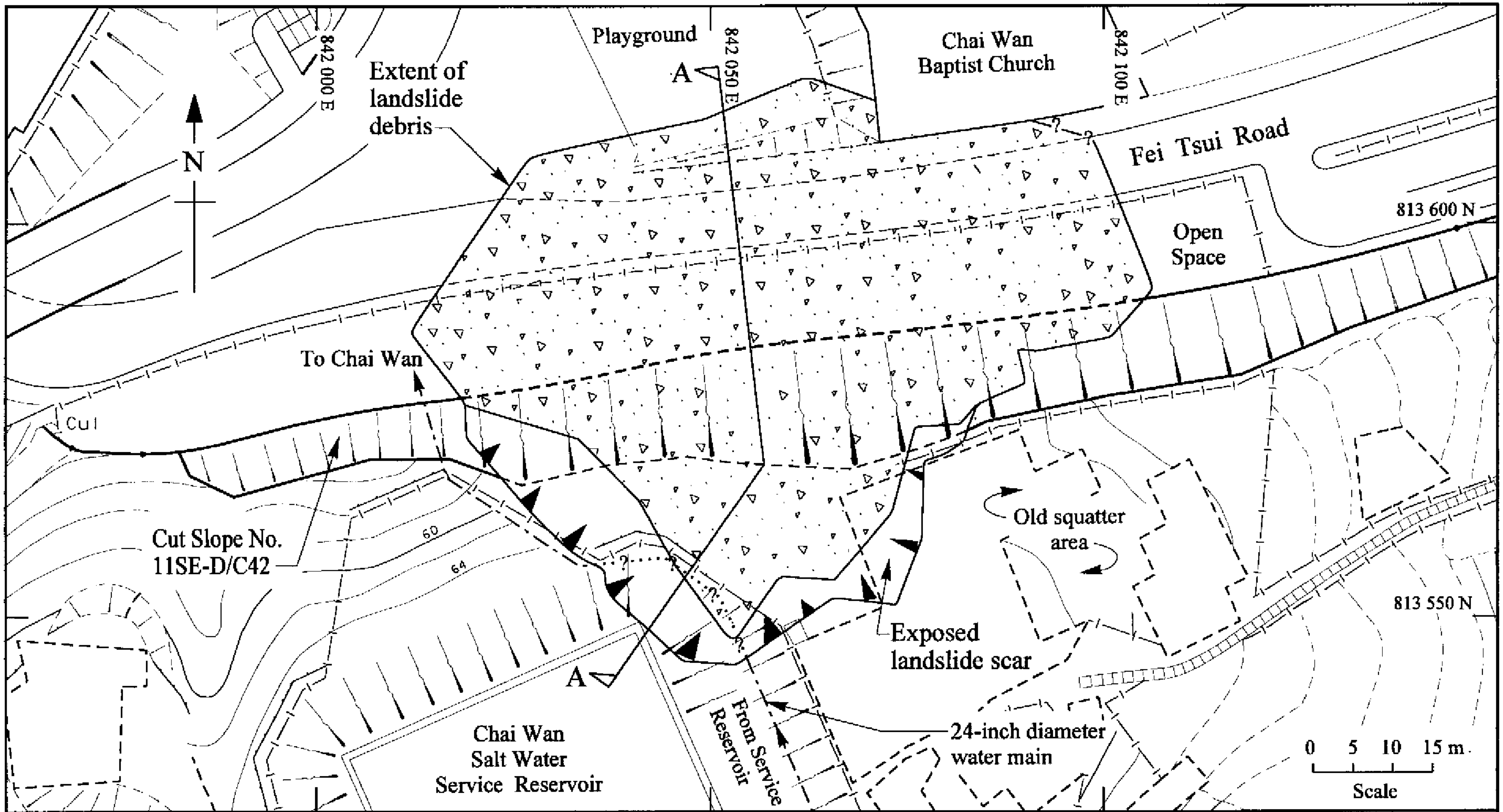

Notes :
(1) See Figure 4 for Section A-A.
(2) Information shown in this figure is based on topographic survey, geological mapping, field observations and documentary records.
(3) Layout of other water mains in the vicinity shown in Figure 14. It is not given in this figure for clarity.

Figure 2 - Plan of the Landslide

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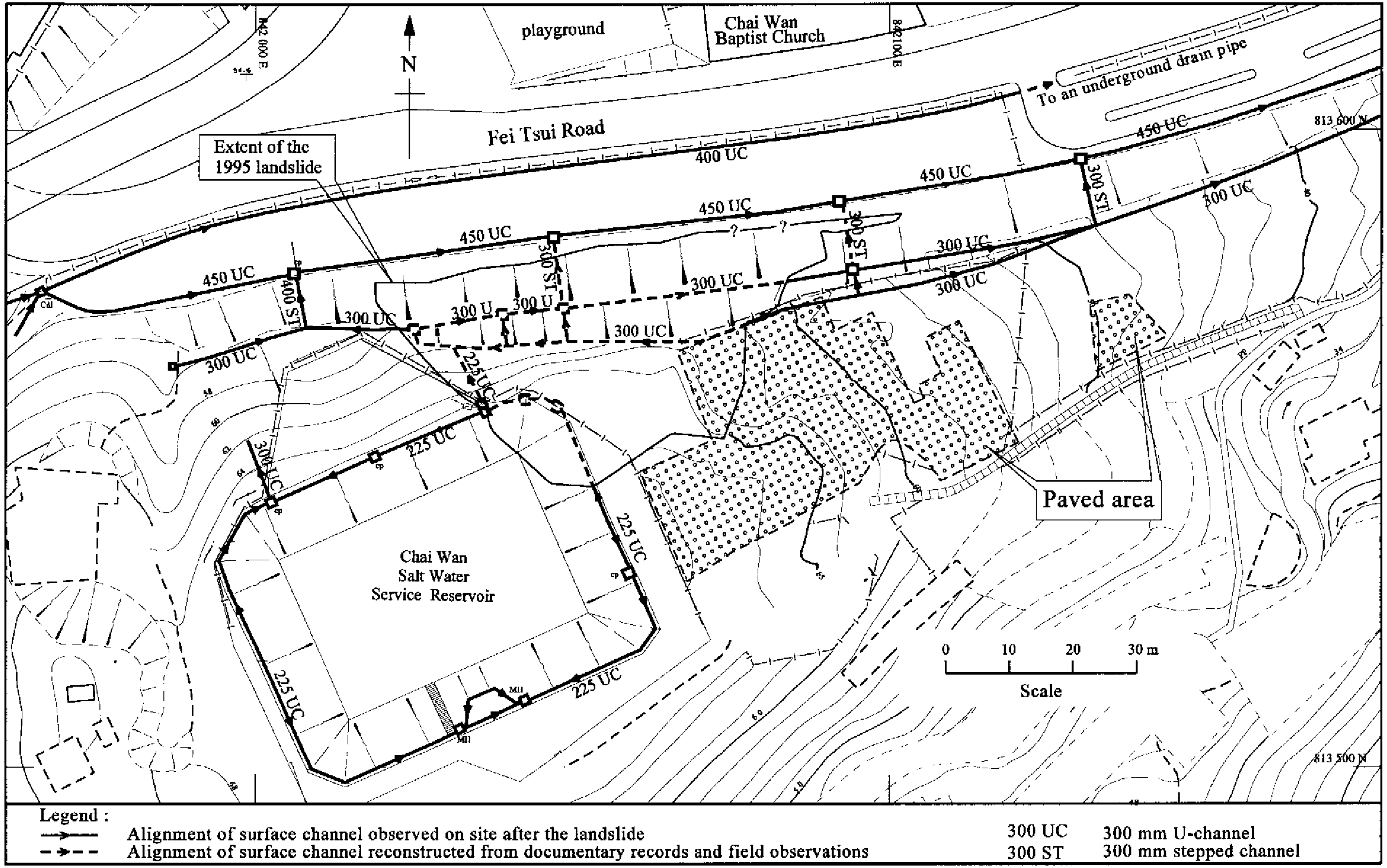

Figure 3 - Probable Drainage Layout of the Site

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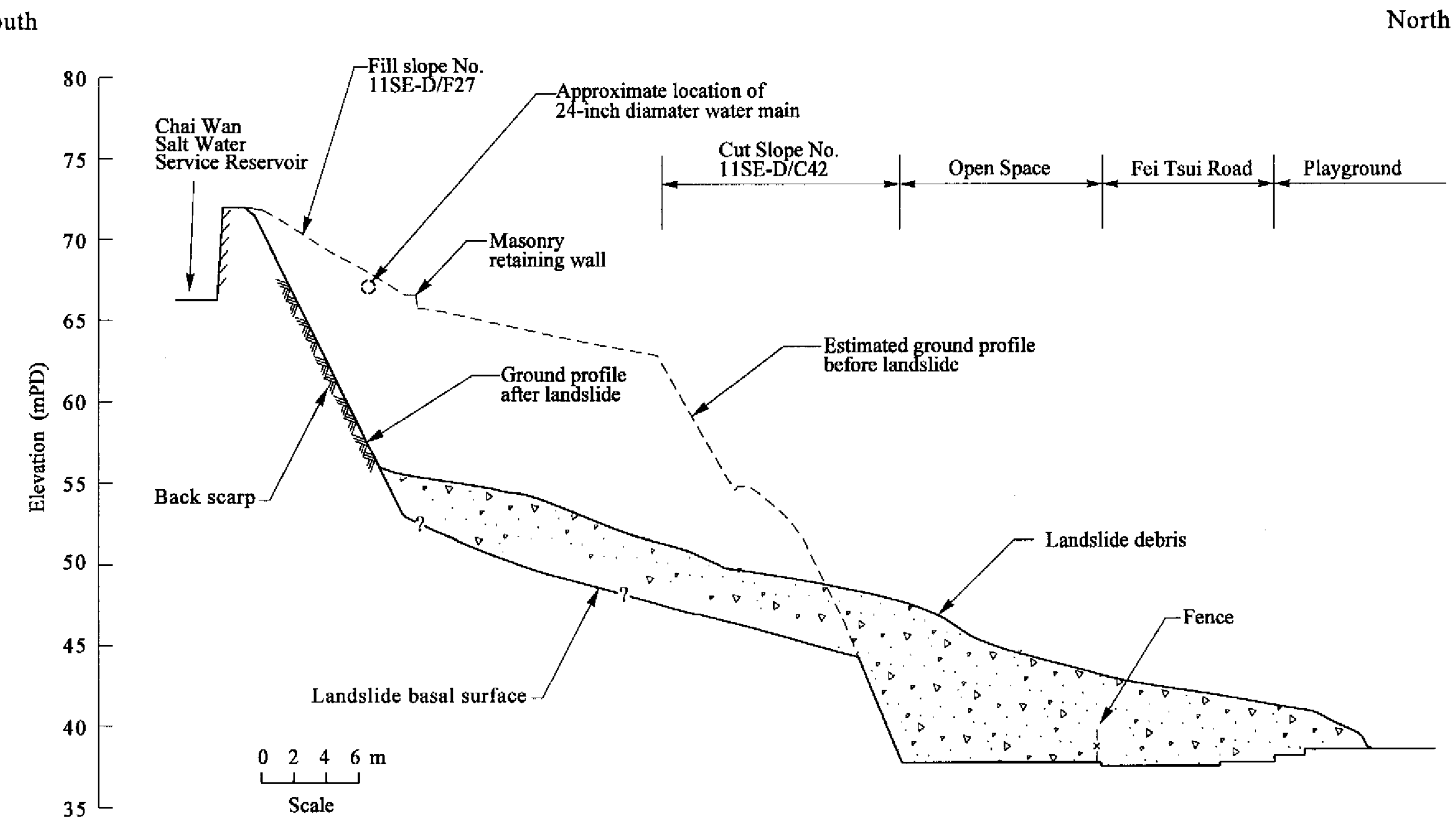

Notes : (1) See Figure 2 for location of section.
(2) Information shown in this figure is based on topographic survey, geological mapping, field observations and documentary records.
(3) The location of the abandoned 6-inch diameter water main, which cannot be ascertained, is not shown in this figure.

Figure 4 - Section A-A

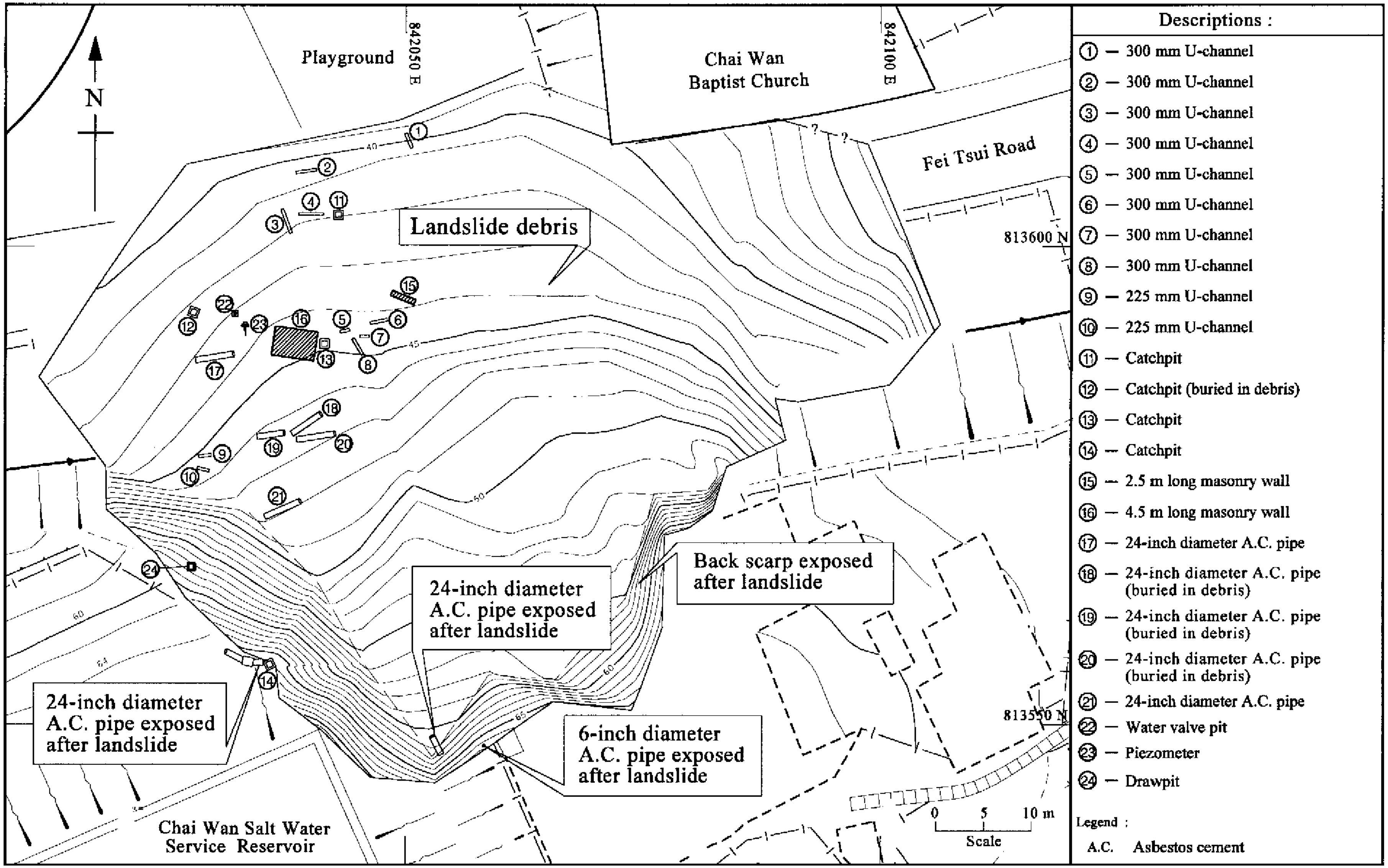

Figure 5 - Locations of Man-made Materials in the Landslide Debris

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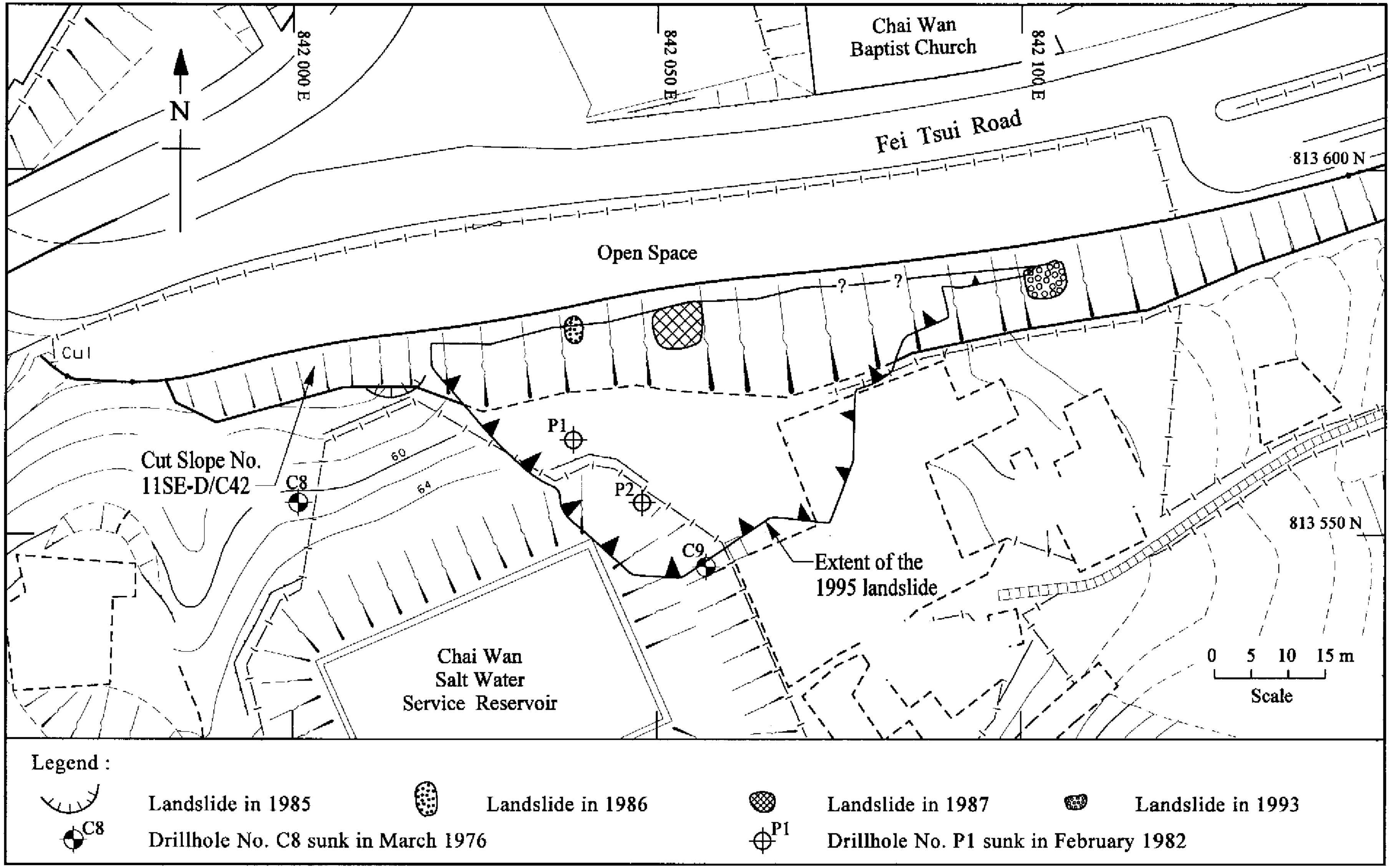
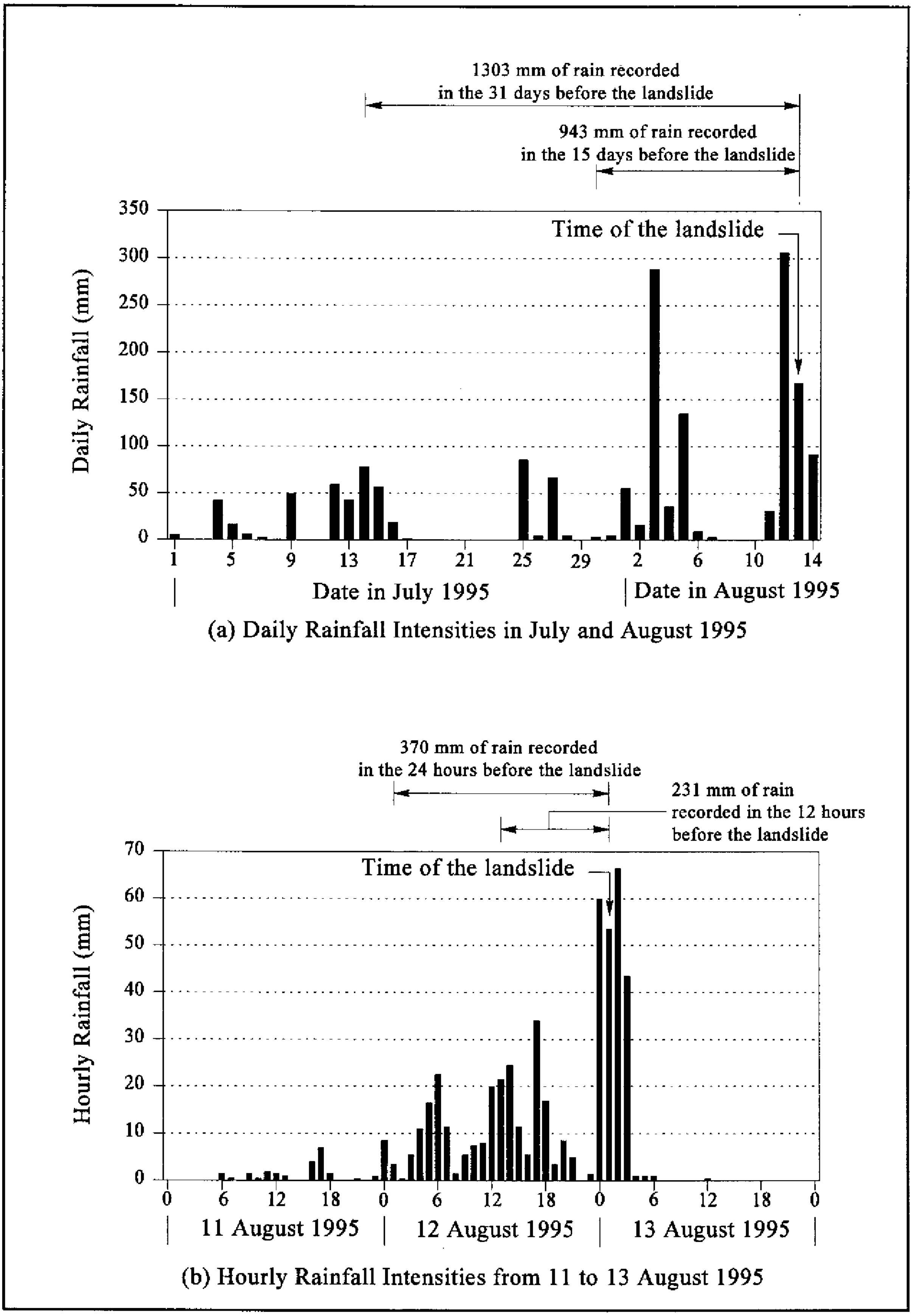


Figure 6 - Locations of Previous Landslides at Slope No. 11SE-D/C42



(a) Daily Rainfall Intensities in July and August 1995

(b) Hourly Rainfall Intensities from 11 to 13 August 1995

Figure 7 - Rainfall Records of GEO Raingauge No. H14

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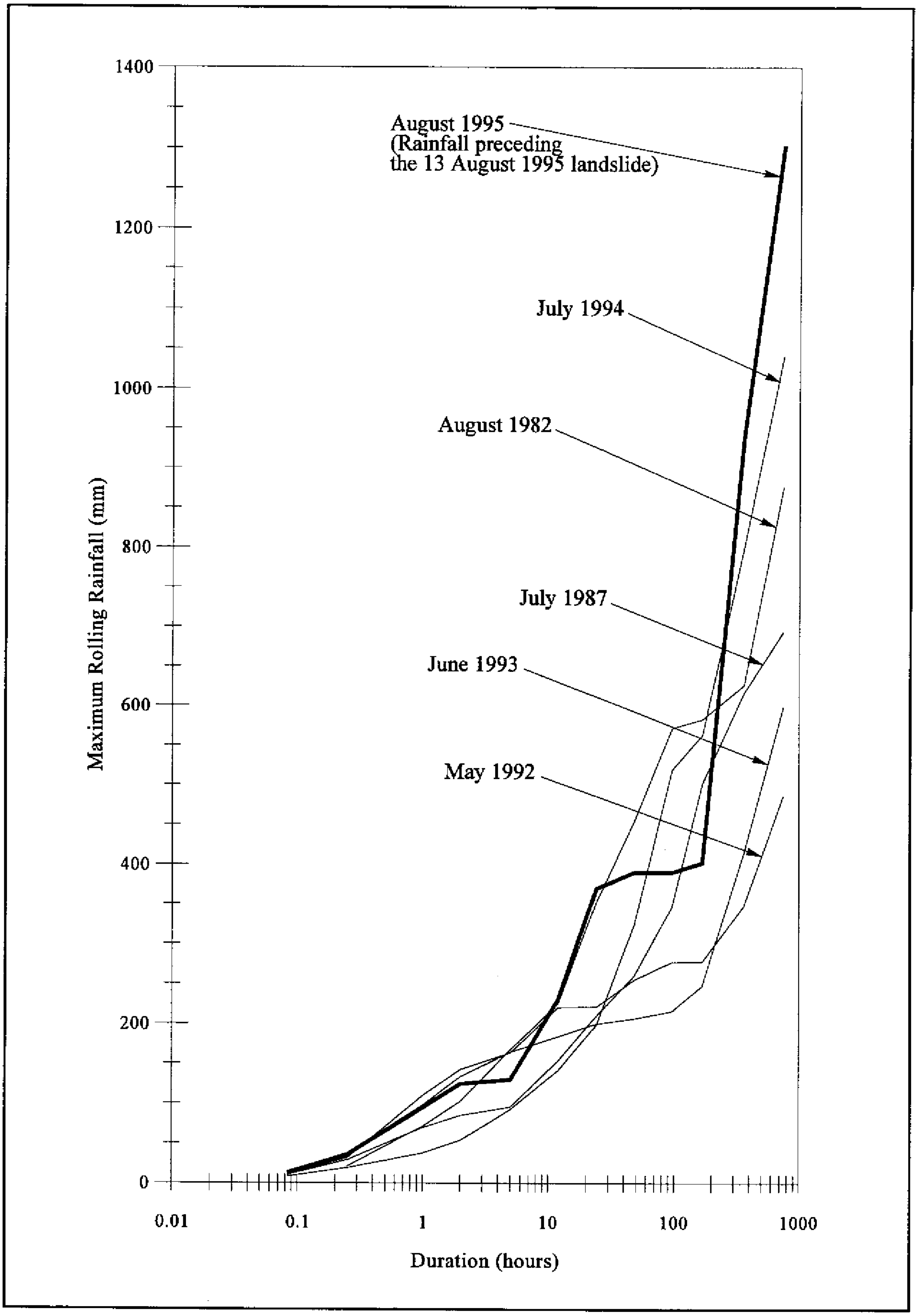

Figure 8 - Maximum Rolling Rainfalls at GEO Raingauge No. H14 for Major Rainstorms

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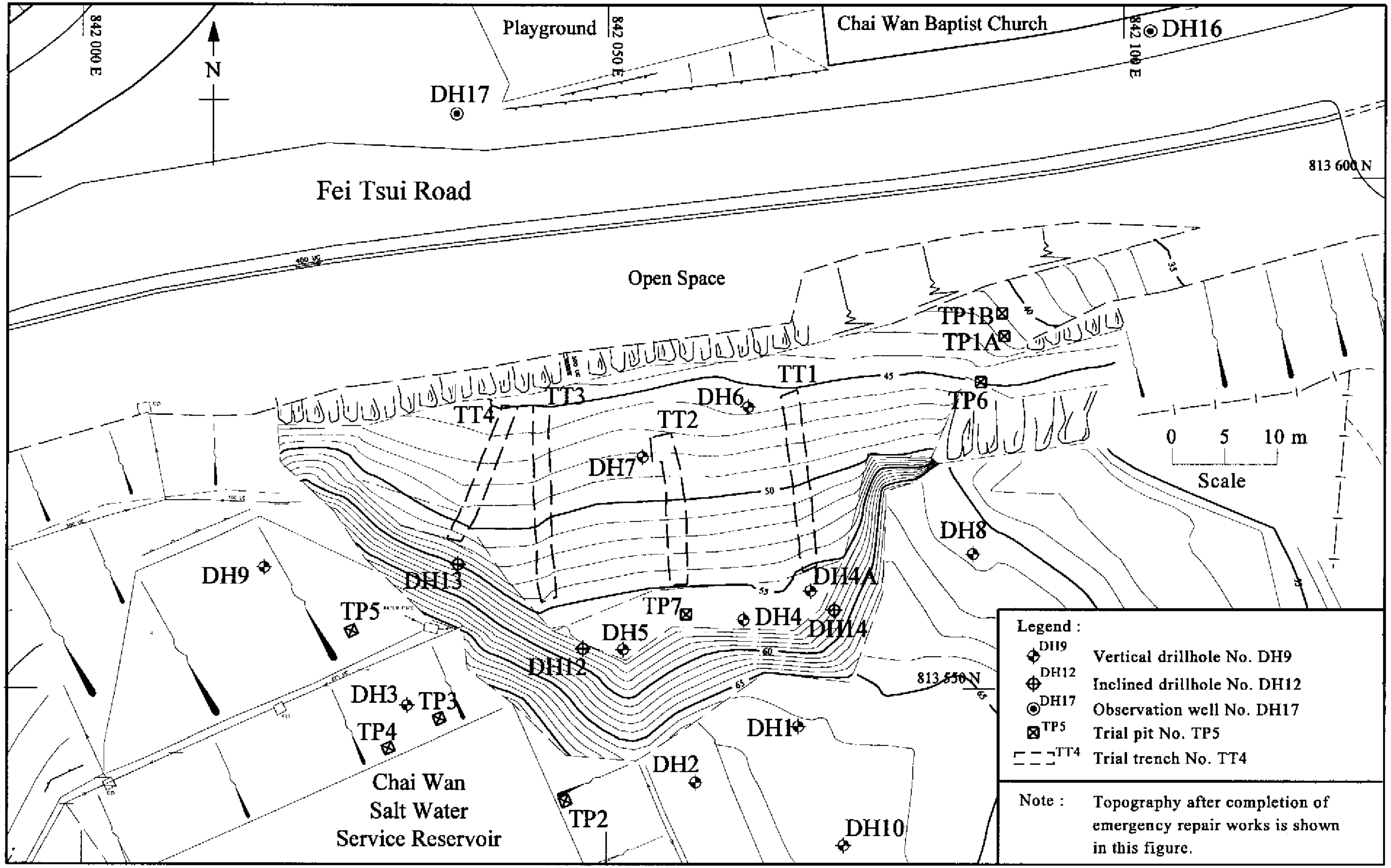

Figure 9 - Location Plan of Ground Investigation Works

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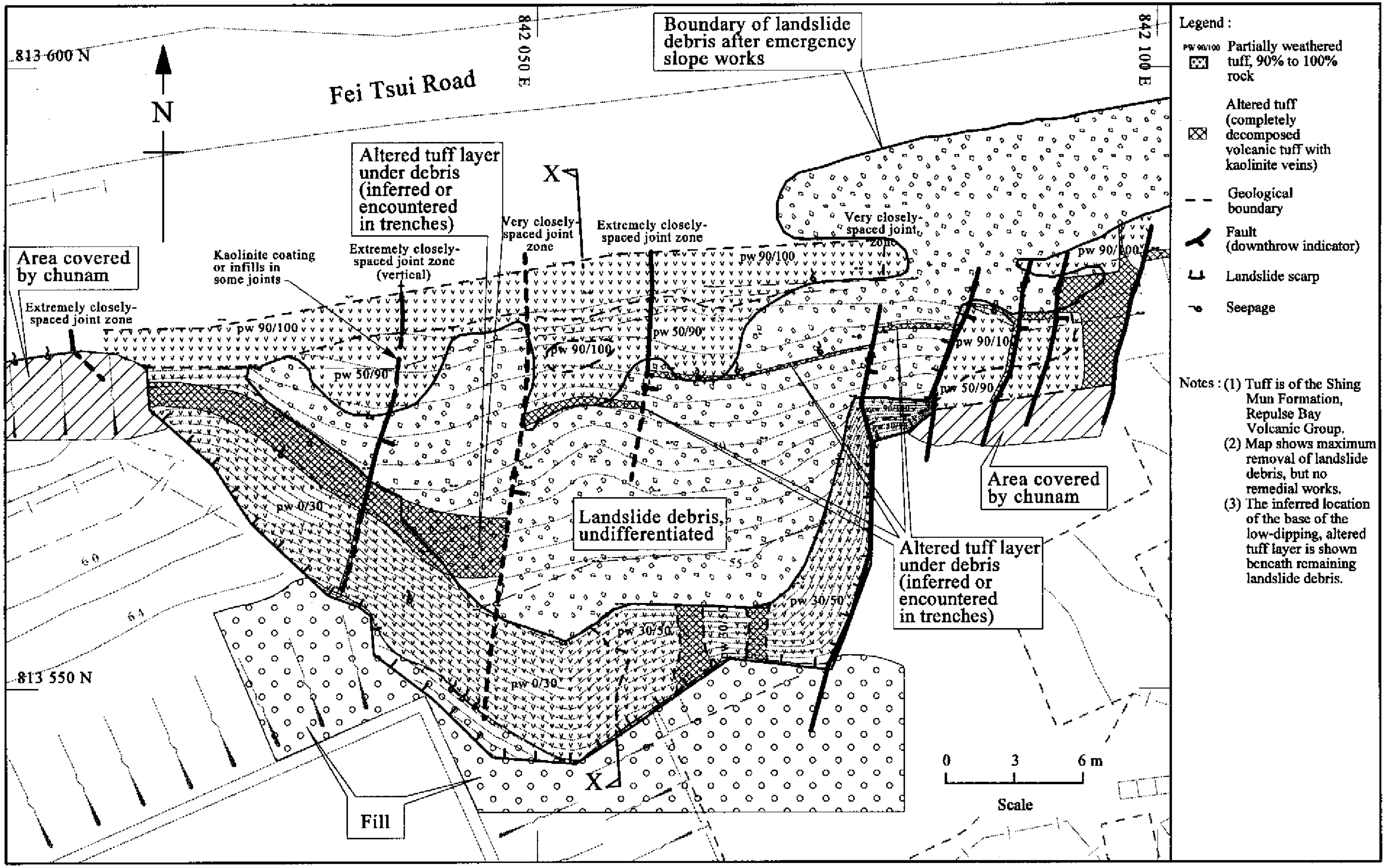
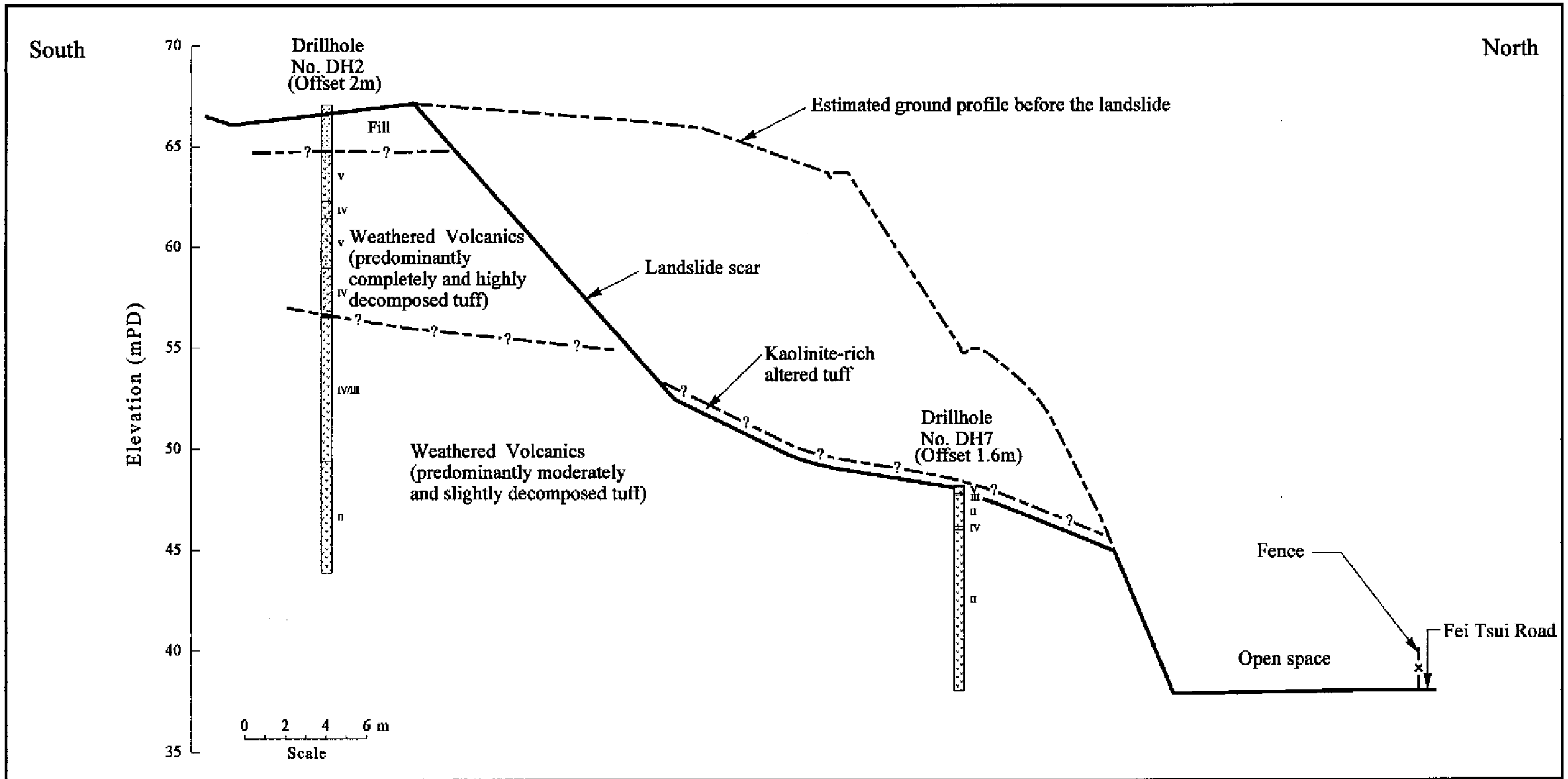


Figure 10 - Geological Map of the Landslide



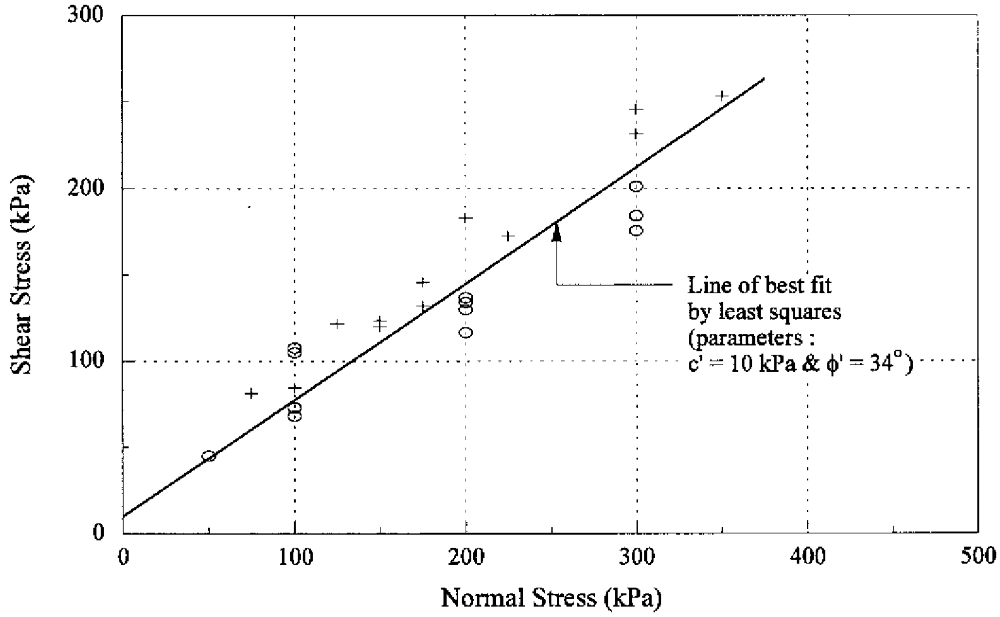
Legend :

V	Completely decomposed tuff		IV	Highly decomposed tuff		III	Moderately decomposed tuff
II	Slightly decomposed tuff			Fill			

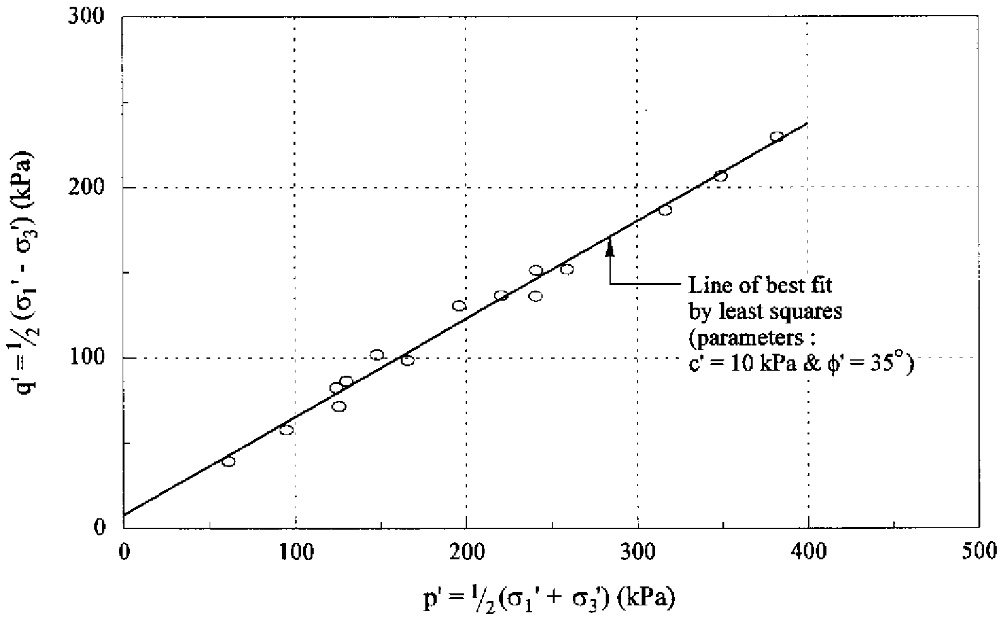
Notes : (1) See Figure 10 for location of section.

(2) Information shown in this figure is based on topographic survey, ground investigation, geological mapping and documentary records.

Figure 11 - Section X-X Showing the Typical Stratigraphy through the Landslide Site



(a) Results of Direct Shear Tests



(b) Results of Consolidated Undrained Triaxial Compression Tests with Pore Water Pressure Measurement

Legend :

+ Single-stage test	σ_1' Major principal effective stress	c' Cohesion
○ Multi-stage test	σ_3' Minor principal effective stress	ϕ' Angle of shearing resistance

Note : Direct shear test results shown in this figure correspond to those at the point of peak shear stress, and triaxial compression tests results correspond to those at the point of maximum stress obliquity, i.e. maximum stress ratio σ_1' / σ_3'.

Figure 12 - Direct Shear and Triaxial Compression Test Results for Altered Tuff without Kaolinite Veins

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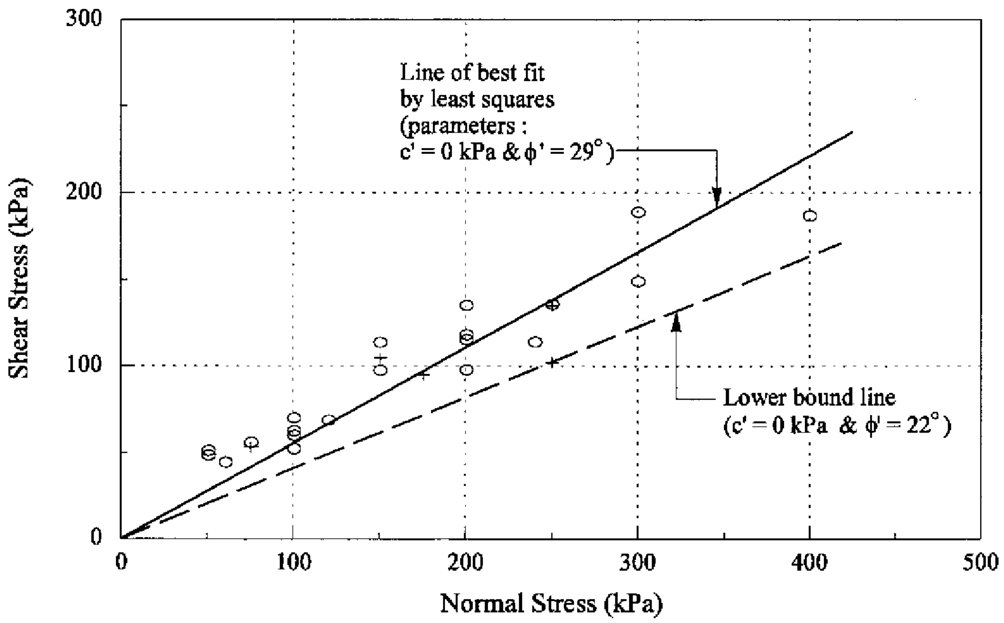
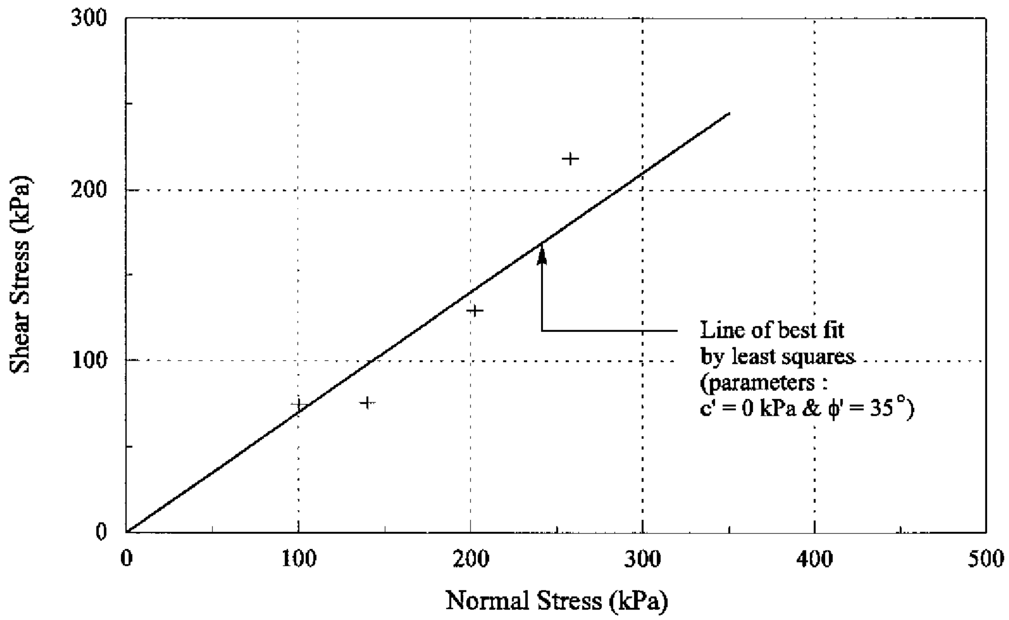


(a) Test Results for Altered Tuff with Kaolinite Veins



(b) Test Results for Weathered Volcanic Joints

Legend :

+	Single-stage test	c'	Cohesion
○	Multi-stage test	ϕ'	Angle of shearing resistance

Notes :
(1) Zero cohesion intercept was assumed in fitting the line of best fit by least squares method through the data points.
(2) For plot (a), samples were oriented to align kaolinite veins along the direction of shearing.
(3) Direct shear test results shown in this figure correspond to those at the point of peak shear stress.

Figure 13 - Direct Shear Test Results for Altered Tuff with Kaolinite Veins and for Weathered Volcanic Joints

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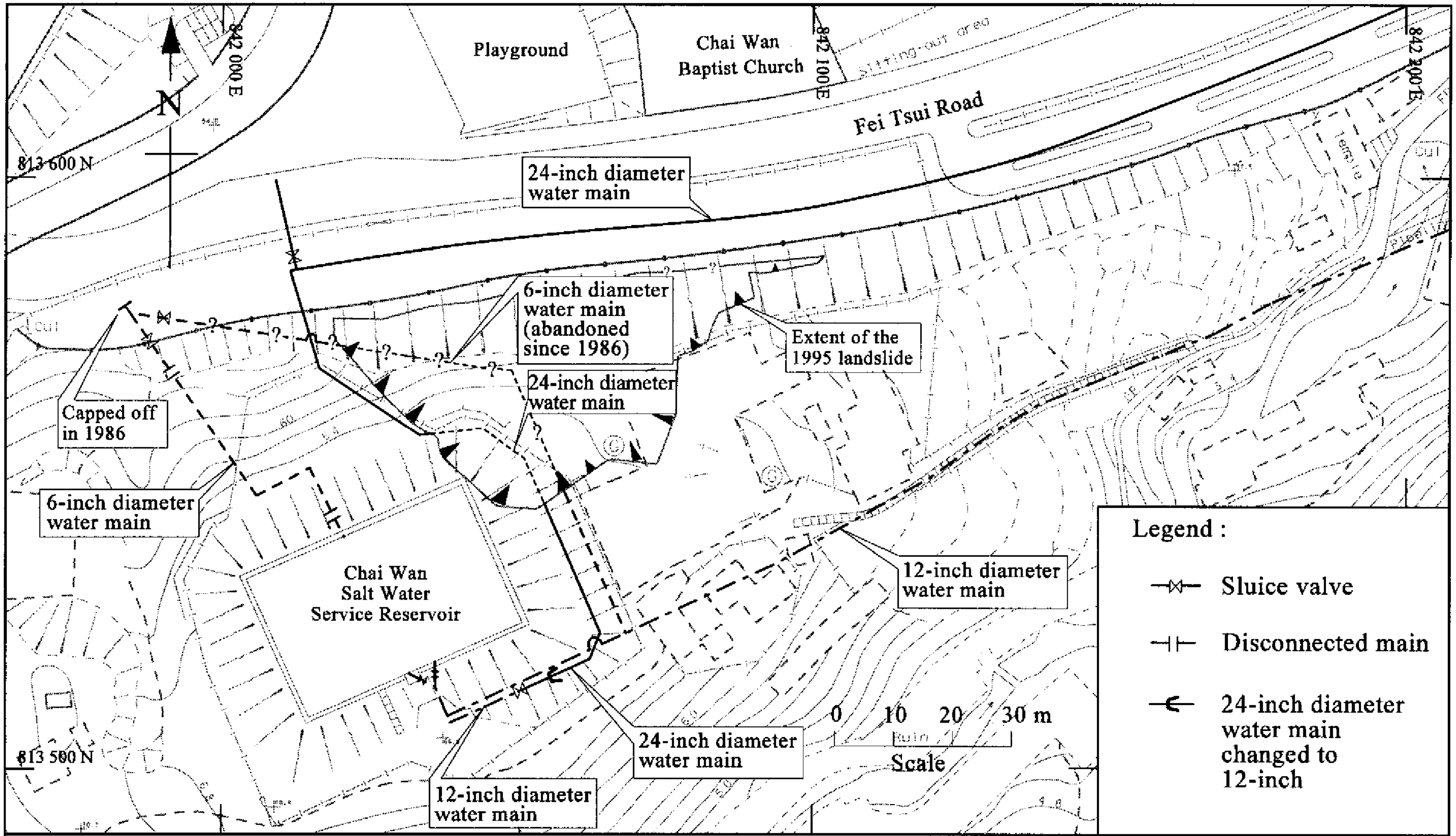
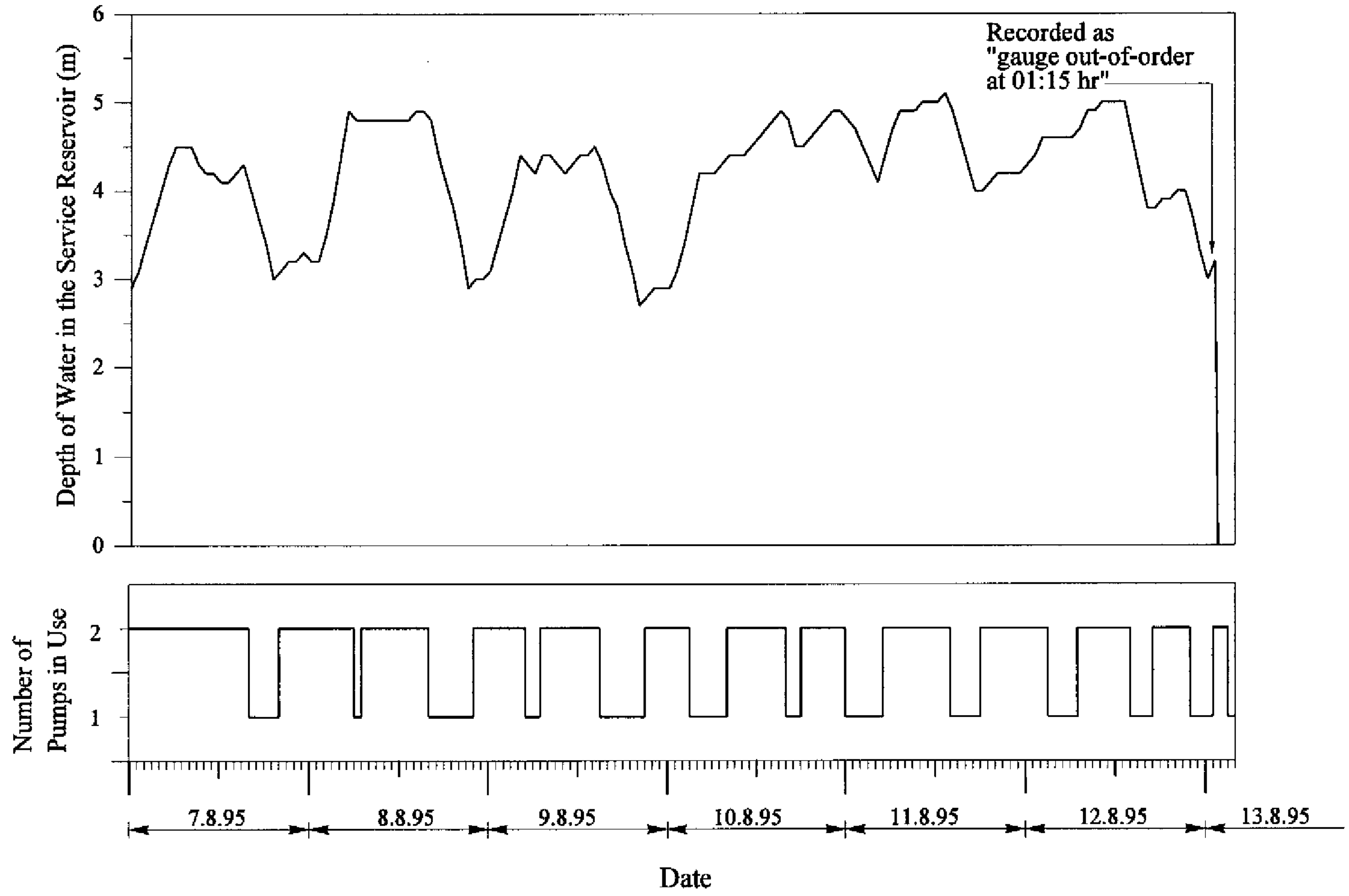


Note : The water main alignments based on information from Water Supplies Department, but alignment of the section of the 24-inch diameter water main severed in the landslide slightly adjusted based on findings from site observations.

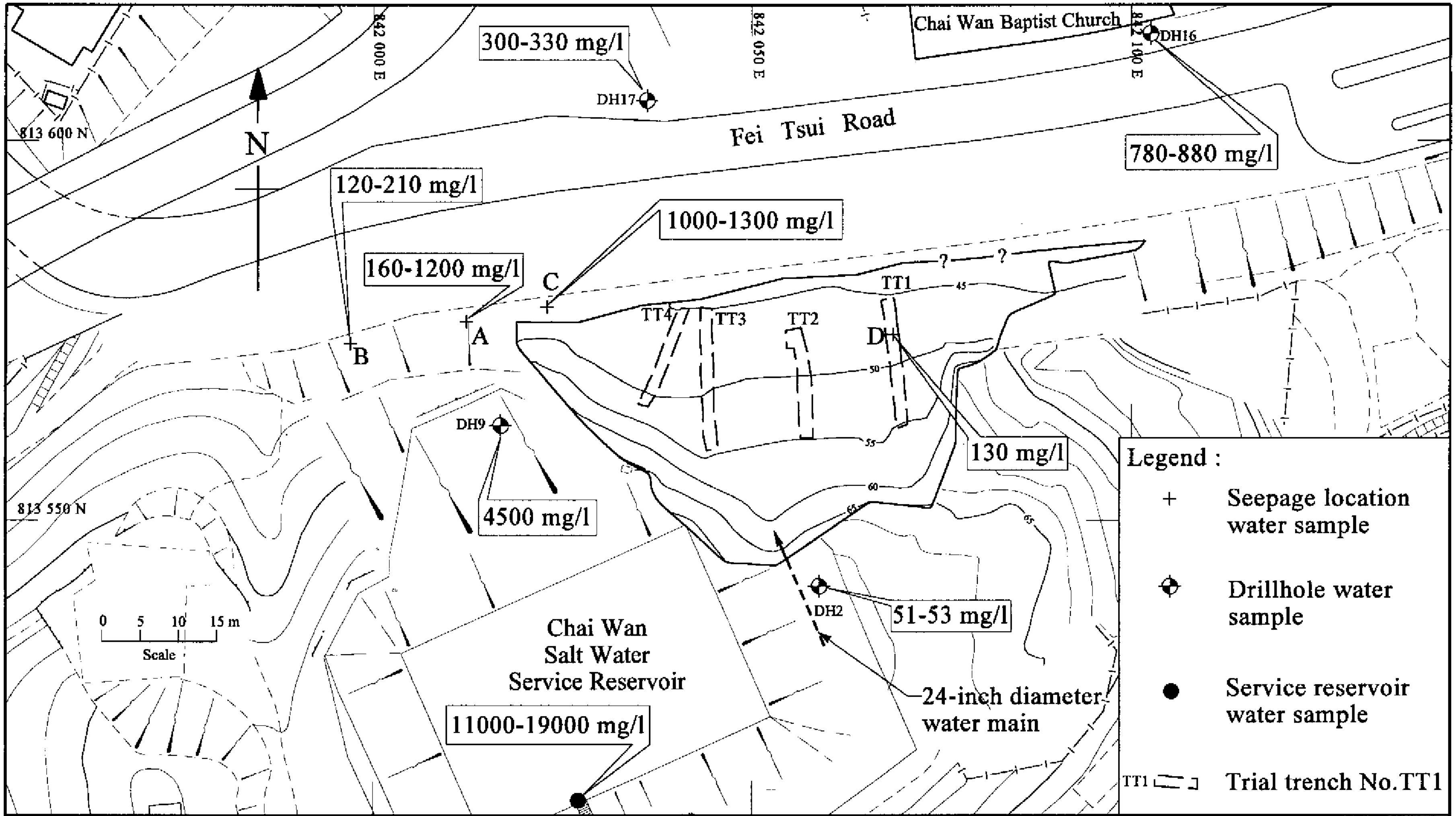
Figure 14 - Layout of Water Mains before the Landslide



Notes : (1) The pumps are housed in Chai Wan Salt Water Pumping Station. Depth of water in the service reservoir shown is for both its east and west compartments.

(2) This figure was based on information obtained from Water Supplies Department (1995).

Figure 15 - Records of Water Levels in the Service Reservoir and Number of Pumps in Use



Notes : (1) Chloride contents of water samples shown in mg/l.
(2) A, B and C are seepage locations at the slope face. D is seepage above the kaolinite-rich altered tuff layer.
(3) The chloride content of a water sample taken from the stream course in the valley at about 200 m to the southeast of the landslide was found to be 42 mg/l.
(4) Topography after completion of emergency repair works is shown in this figure.

Figure 16 - Results of Chloride Content Tests on Water Samples

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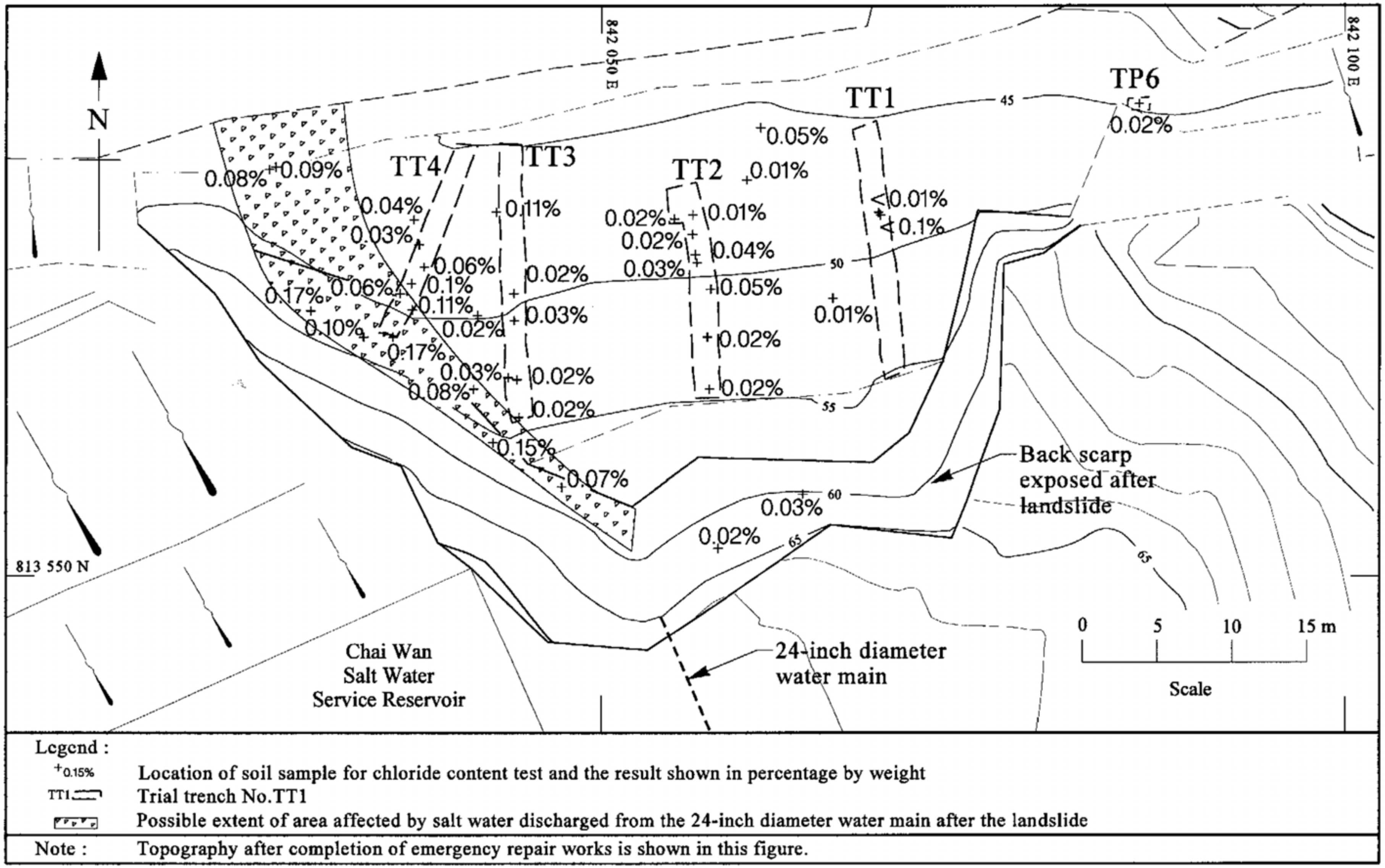
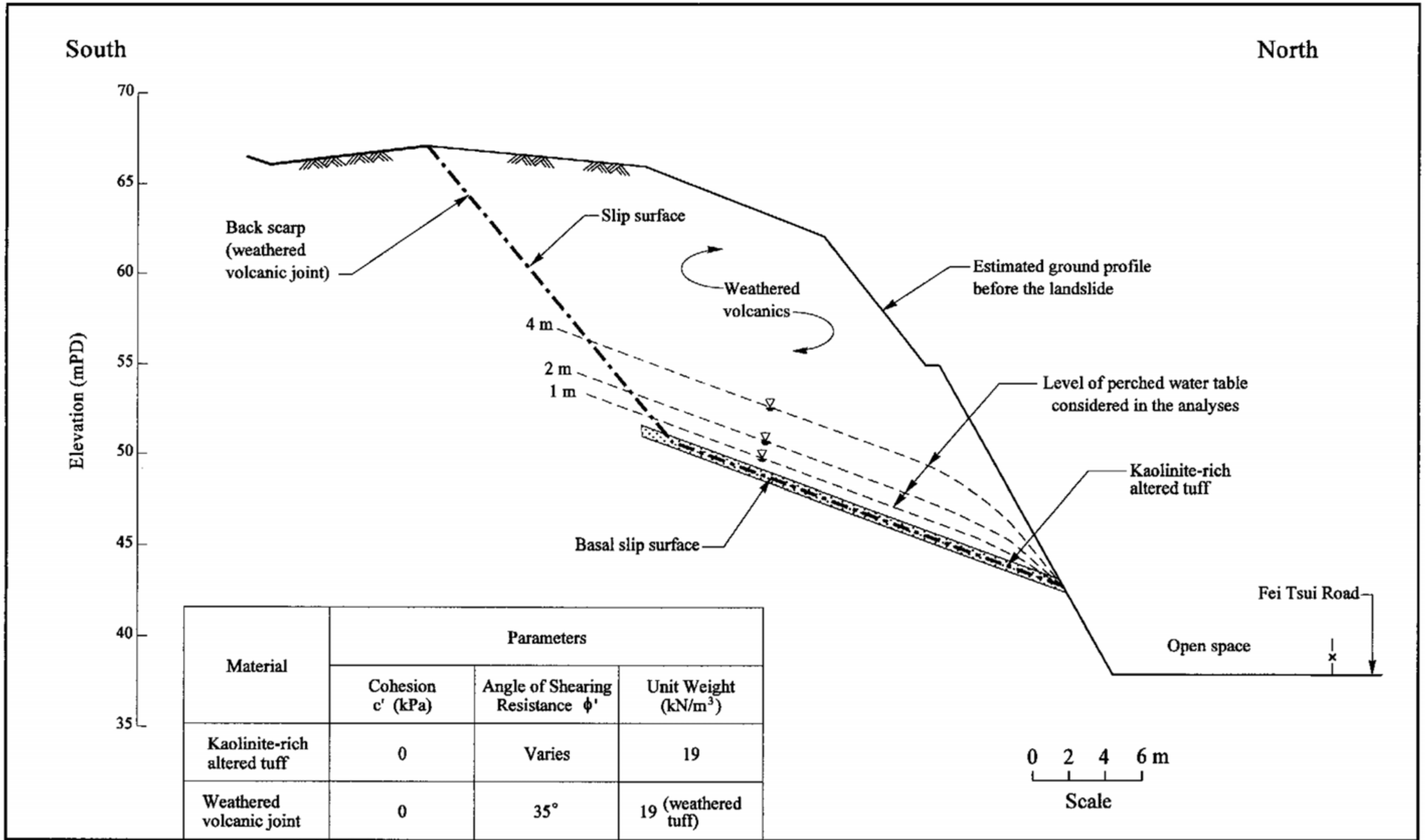


Figure 17 - Results of Chloride Content Tests on Soil Samples



Material	Parameters		
	Cohesion c' (kPa)	Angle of Shearing Resistance φ'	Unit Weight (kN/m³)
Kaolinite-rich altered tuff	0	Varies	19
Weathered volcanic joint	0	35°	19 (weathered tuff)

Note : The case of no perched water table was also analysed.

Figure 18 - Representative Cross-section of the Landslide for Theoretical Stability Analyses

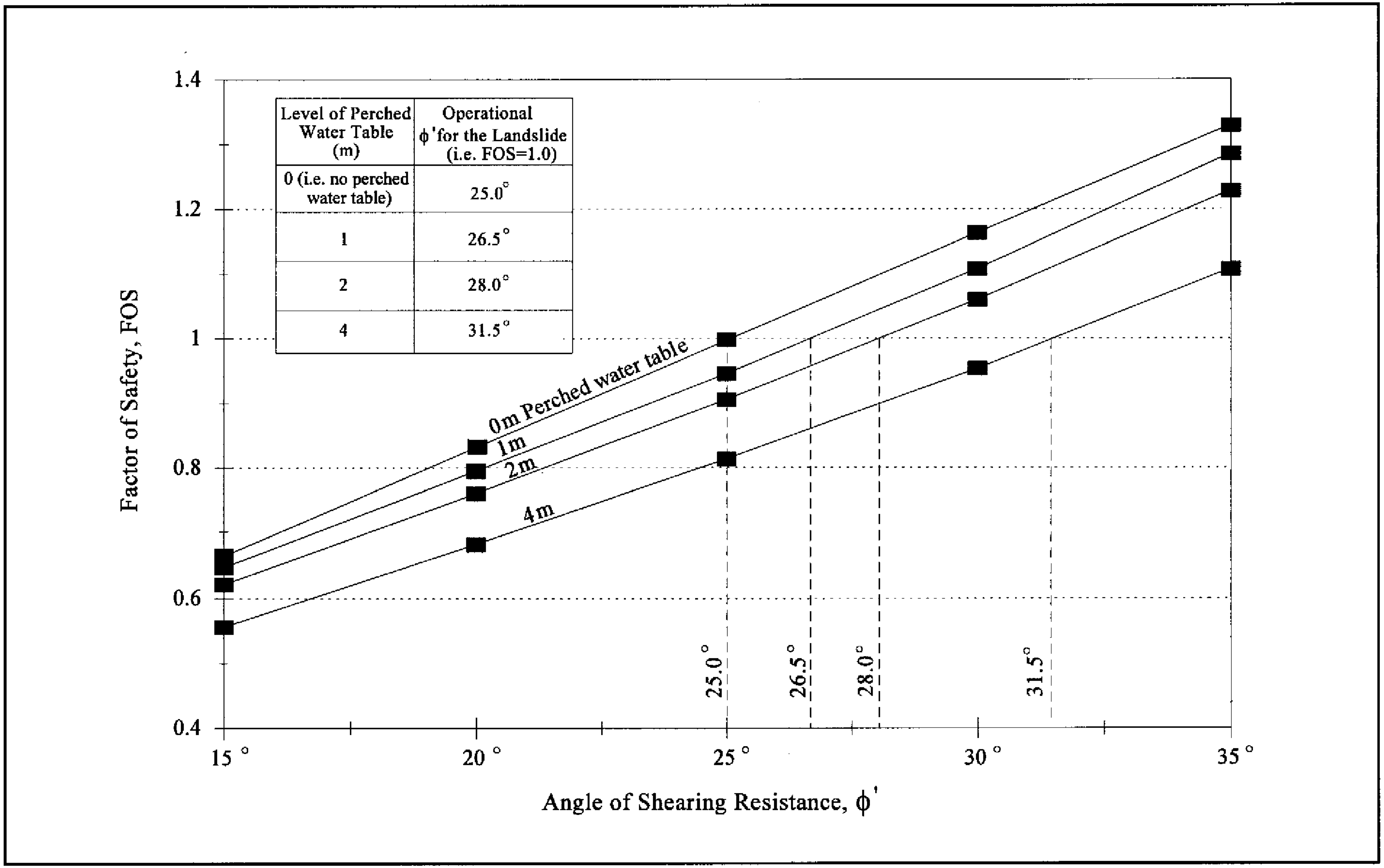


Figure 19 - Results of Theoretical Stability Analyses

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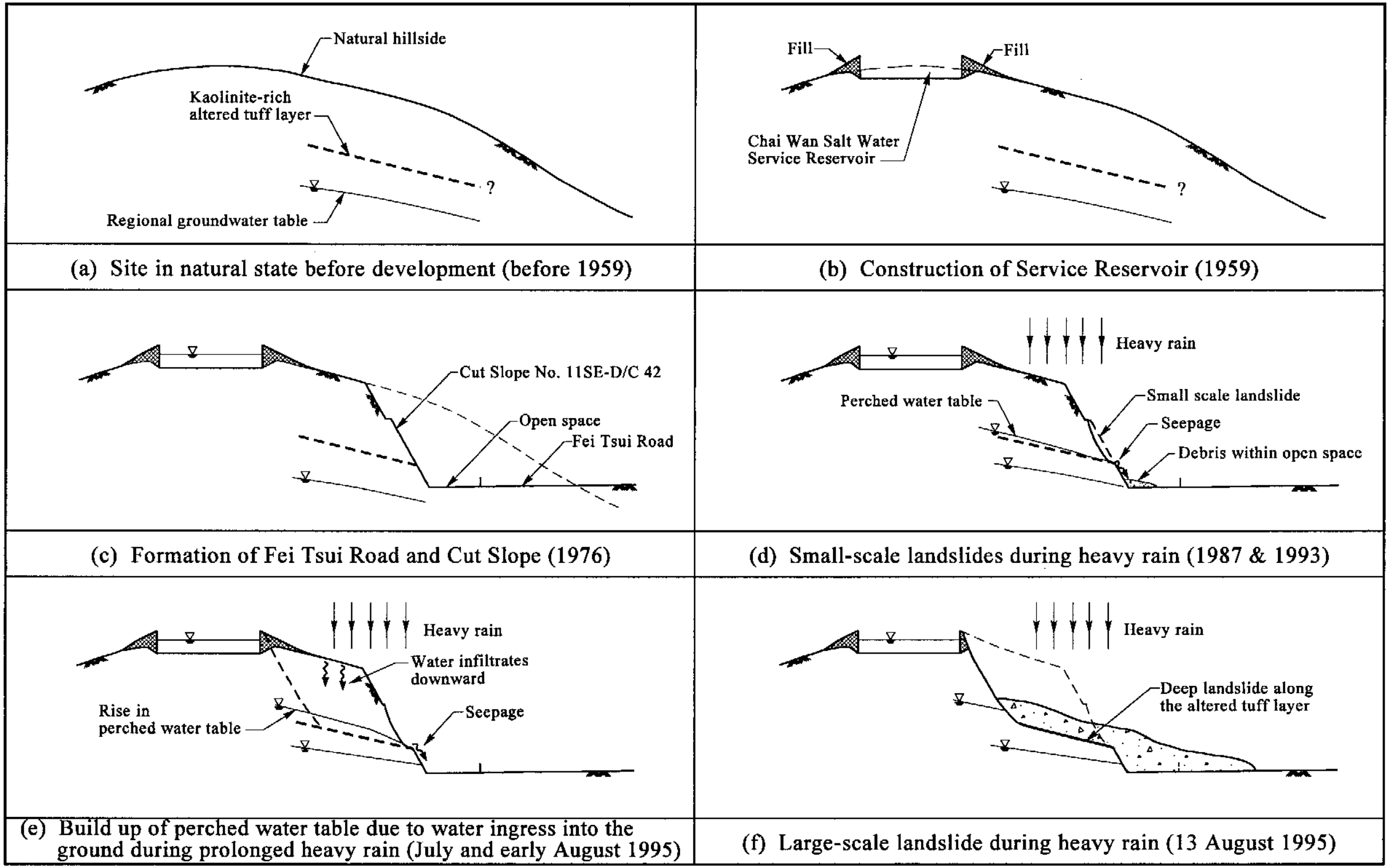

Figure 20 - Probable Sequence of Events

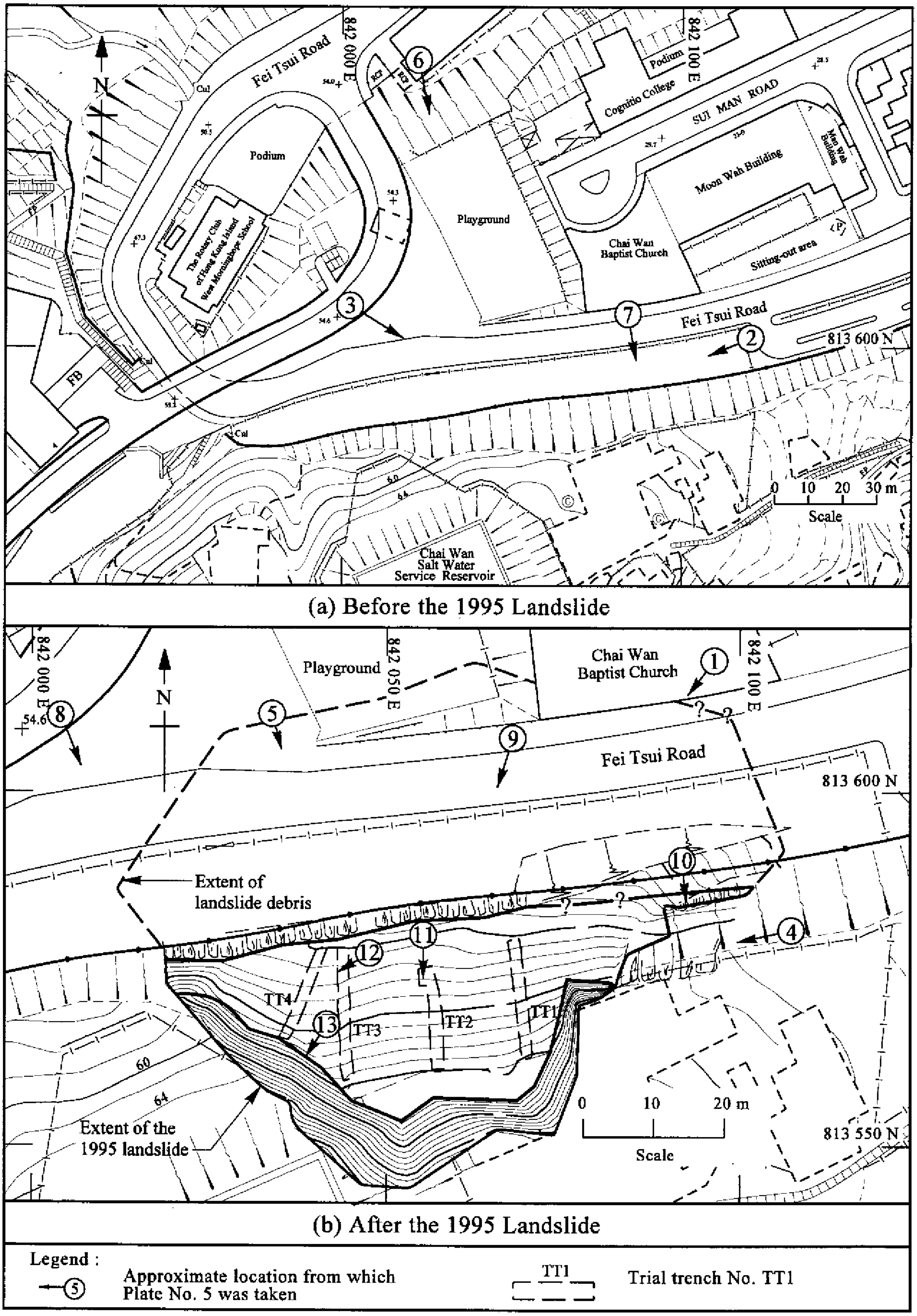

Figure 21 - Location Plan of Photographs Taken

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Plate 1 - Photograph of the Landslide Taken in the Morning of 13 August 1995 (See Figure 21 for Location)

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Plate 2 - Photograph of Cut Slope No. 11SE-D/C42 Taken after Slope Formation (Reproduced from Plate 8 of Binnie, 1977. See Figure 21 for Location)

Plate 3 - Photograph of Cut Slope No. 11SE-D/C42 Taken on 9 November 1994 (See Figure 21 for Location)

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Plate 4 - Blocked Surface Drainage Channel on the Unfailed Section of the Cut Slope (Photograph Taken on 10 October 1995. See Figure 21 for Location)

Plate 5 - Blocked Surface Drainage Channel in the Landslide Debris (Photograph Taken on 13 August 1995. See Figure 21 for Location)

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Plate 6 - View of the 1987 Landslide (Photograph Taken on 17 August 1987. See Figure 21 for Location)

Plate 7 - View of the 1993 Landslide (Photograph Taken on 27 September 1993. See Figure 21 for Location)

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Plate 8 - Weathered Rock Joints Forming the Back Scarp of the Landslide (Photograph Taken on 19 August 1995. See Figure 21 for Location)

Plate 9 - Rock Face below the Base of the Landslide (Photograph Taken on 31 August 1995. See Figure 21 for Location)

Plate 10 - Kaolinite-rich Altered Tuff at the Unfailed Portion of the Cut Slope to the East of the Landslide (Photograph Taken on 8 October 1995. See Figure 21 for Location)

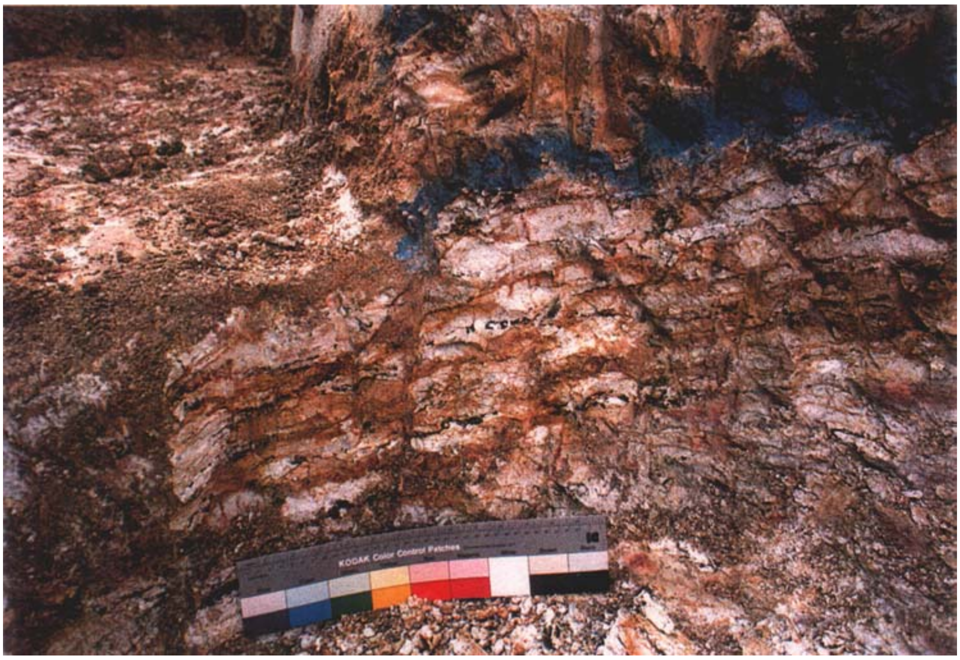

Plate 11 - Kaolinite-rich Altered Tuff at the Central Part of the Base of the Landslide (Photograph Taken on 20 October 1995. See Figure 21 for Location)

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Plate 12 - Remnants of Kaolinite-rich Altered Tuff at the Western Toe of the Landslide (Photograph Taken on 20 October 1995. See Figure 21 for Location)

Plate 13 - The 'Diffuse' Layer at the Western Back Scarp (Photograph Taken on 14 August 1995. See Figure 21 for Location)

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APPENDIX A

SUMMARY OF SITE HISTORY

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A.1 SITE DEVELOPMENT

References

The earliest available aerial photographs of the site were taken in 1924. At that time, the site was situated on an east-northeast trending spur and was undeveloped.

The Chai Wan Salt Water Service Reservoir was constructed in 1959 according to the Water Supplies Department. WSD (1995)

In the 1961 aerial photographs, squatter huts can be seen on the lower footslopes to the northeast of the service reservoir. The number of squatter huts increased in subsequent years, as is evident from the 1969 and 1976 photographs

The subject cut slope was built between 1972 and 1976 by the Architectural Office (AO) (reorganised as the Architectural Services Department in 1986), as part of construction for the Hing Wah Estate Phase II development. The 1976 aerial photographs show that construction of Fei Tsui Road was essentially completed and the open space in front of the cut slope had been formed by that time

On the site formation plan entitled "PROPOSED REVISED ROAD LAYOUT OF L.T.R.S. ROUTE No. 81 BETWEEN LIN SHING RD. & ACCESS RD. TO HING WAH R.E. CHAI WAN HK" prepared by the AO in 1971, Fei Tsui Road together with the open space is shown as a dual carriageway. It therefore appears that the open space is a road reserve formed for the future Route No. 81. AO Drawing No. A/45444

The Chai Wan Baptist Church building opposite the landslide location on Fei Tsui Road was built between 1986 and 1987.

The squatter huts over the crest of the cut slope were cleared in 1991.

A.2. PREVIOUS ASSESSMENTS

A.2.1 Slope Registration

In August 1977, the cut slope was registered as Slope No. 11SE D/C42 by the consultants, Binnie & Partners (Hong Kong) Consulting Engineers (B&P), engaged by Government to prepare a catalogue of cut slopes, fill slopes and retaining walls (now commonly known as the 'Catalogue of Slopes'). Binnie & Partners Field Sheet for Slope No. 11SE D/C42

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A.2.2 Landslide Study Phase IIC, Government of Hong Kong

In December 1977, B&P prepared a study report on the stability of the slopes in the Chai Wan area under Landslide Study Phase IIC commissioned by Government. The study, which comprised field inspection, ground investigation and stability analyses, included Cut Slope No. 11SE-D/C42. Binnie (1977)

B&P observed that there was “a 4' wide berm at mid-height” and “beneath the berm the exposed rock face is closely jointed fresh to slightly decomposed volcanic rock with a prominent horizontal weathered seam midway between the road and the berm (plate 8). Beneath the service reservoir we noted seepage along sub-horizontal joints”. Plate 8 in B&P’s report is given as Plate 2 in this landslide investigation report. Para. No. 10.1, Binnie (1977)

Two investigation holes were drilled for the study. B&P noted that “one of which (C8) was between the service reservoir and the section of the slope where we noted the seepage, and the second (C9) within the reservoir grounds about 16 m back from the edge of the highest section of cut slope. Difficult access prevented this hole being sunk at the crest of the cut slope. Both holes penetrated at least 9 m into fresh to slightly decomposed rock”. Para. No. 10.2, Binnie (1977)

B&P described that “Mid way between the road level and the berm halfway up the cut face, there is a prominent horizontal weathered joint approximately 100 mm wide and at least 50 m long”. B&P stated that, for hole C9, “Reduced core recovery between 20.7 m and 21.0 m is at about the same level as the horizontal joint on the cut slope and provides evidence of the possible persistence of this joint”. Paras. No. 10.4 & No. 10.6, Binnie (1977)

B&P reported that “Two piezometers were installed in each hole, one at the base of the hole and one at interface between moderately and slightly decomposed volcanics. The upper piezometer in each hole has remained dry”. As for the lower piezometers, “Both maximum piezometric levels are within slightly decomposed to fresh volcanic rock”. Para. No. 10.7 Binnie (1977)

B&P stated that “There are two possible forms of instability. Beneath the mid-height berm in the cut slope there is the possibility of wedge or plane failure along rock joints or the spalling of loose rocks and overhangs. Above the berm there is a possibility of a soil-type failure in the decomposed volcanics. As the prominent weathered seam (paragraph 10.1 and 10.6) is horizontal we do not consider it to be a potential source of instability”. Para. No. 10.9 Binnie (1977)

The possibility of wedge or plane failures was assessed using a stereogram. It was concluded that “plane failure is unlikely” and “No large scale remedial measures are necessary”. Para. No. 10.11 Binnie (1977)

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In the assessment of soil-type failures, B&P stated that "We have assumed that the descent of a wetting band would result in a perched water table forming above the interface between moderately and slightly decomposed volcanics". It was indicated in Drawing 13 of the report that "Strength parameters used:- a) grade IV/V dv $\phi' = 35°$, $c' = 10kPa$ b) grade III/IV dv $\phi' = 40°$, $c' = 10kPa$". Para. No. 10.7 & Drawing 13, Binnie (1977)

B&P found that "The factors of safety for the 1 in 1000 year rainfall condition are acceptable, however the minimum 1 in 10 year factor of safety is 1.09 which is below the acceptable value of 1.20", and "To increase F to 1.20 an average suction of 4.9 kPa is required along the critical slip surface (surface 3). This is a small value which we expect to exist but we recommend that the existence of suctions of this magnitude is checked using psychrometers". Paras. No. 10.15 & No. 10.16, Binnie (1977)

In March 1978, Scott Wilson Kirkpatrick & Partners (SWKP) commented "on recommendations made by Binnie & Partners in their Phase IIC report on the Chai Wan area concerning measures necessary to improve the stability of the cut slope adjacent to the new access road to the Hing Wah Estate (Stage II)" on behalf of Government. Para. No. 1 in SWKP's memo to Buildings Ordinance Office (BOO) Dated 13.5.1978

SWKP stated that "We agree in general with Binnie & Partners' assessment of the dominant pattern of jointing in the slope and concur with their conclusion that neither plane nor wedge failures bounded by these steeply dipping joints are likely". Para. No. 3iii) in SWKP's memo dated 13.5.78

SWKP commented that "Binnie & Partners have, however, mentioned in their study of the rock joints 'a prominent horizontal weathered joint approximately 100mm wide and at least 50m long' (§ 10.4). Although this joint does strike parallel to the cutting face (and therefore does daylight as a horizontal line) we would suggest that it is one of a family of locally persistent joints dipping at about 25° towards the road. Some of these joints are deeply weathered and show signs of strong seepage. We would recommend that the possibility of instability of blocks above this joint (i.e. local plane failure) should be considered. Depending on the results it may be necessary to design appropriate retaining measures for parts of the face. (There are indications that some blocks above this joint did fall as the cutting was being formed.)". Para. No. 3 iv) in SWKP's memo dated 13.5.78

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SWKP noted that "At the west end of the cutting there is evidence of movement of the residual soil above the rock, causing cracking of the chunam. This has probably occurred as a result of wetting of the soil at the soil/rock interface. It is clear that there is seepage occurring from many points along the cutting and we would strongly recommend: a) that the sources of the seepage in both rock and residual soil are identified and if possible eliminated (the pipeline and filter beds/salt water service reservoir shown at the top of the slope on Binnie and Partners drawings HO74/70/12 and 14 should be checked for leakage); b) that horizontal drains should be installed at the base of the soft material, and in the rock where signs of major seepage are apparent; and c) that the top of the slope should be protected at least as far back as the end of the most critical slip surface to prevent infiltration of water into the slope". Para. No. . 3 v) in SWKP's memo dated 13.5.78

SWKP stated that "An alternative interpretation of the logs of drill holes C8 and C9 would suggest that the soil/rock interface is higher in the boreholes than was assumed and dips towards the cutting. The possibility of failure involving sliding on this wetted soil/rock interface should be considered". Para. No. . 3 vi) in SWKP's memo dated 13.5.78

SWKP noted that "Binnie & Partners have also recommended the installation of two psychrometers in a borehole behind this slope in order that the existence of pore suctions of the order of 5 kN/m² may be confirmed" and that SWKP "are prepared to accept Binnie & Partners' expectation that pore suctions of the required stabilising magnitude will exist at this site". Para. No. . 4) in SWKP's memo dated 13.5.78

There is no record of whether the psychrometers were subsequently installed.

A.2.3 GCO Stage 1 Study

In July 1979, Slope No. 11SE-D/C42 was assessed by the Geotechnical Control Office (GCO) (renamed Geotechnical Engineering Office in 1991) in a Stage 1 Study carried out under the Landslip Preventive Measures (LPM) Programme. This was a preliminary stability assessment to review whether a further detailed stability study was required. The study consisted of field inspection and a geotechnical appraisal based on the available information, without any ground investigation. The Stage 1 Study Report was produced in September 1979. GCO (1979)

"Steady" seepage was observed "From rock joints". On a plan given in the report, a line showing the location of seepage was marked parallel to the toe line of the cut slope near the lower part of the slope. This line covers a 40 m long portion of the cut slope directly opposite the present playground. Another line of seepage, which covers a 15 m long portion of the cut slope to the north of the service reservoir, was also marked on the plan. "Evidence of faulting in places" was also noted. GCO (1979)

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The report stated that "A preliminary joint survey was carried out and potential failure mechanisms assessed." It was found that "The stereographic analysis shows three kinematically possible mechanisms of failure. Planar failure along plane corresponding to pole 5, wedge failure on the line of intersections of planes 2 and 5 and toppling of plane 1. The proximity of the relevant poles to the daylight and toppling envelope indicates that the possibility of failure is only marginal". Items No. 3 & No. 4 of Discussion, (GCO 1979)

The report noted that "The consequence of failure of this slope is relatively small as most of the slide debris would be deposited on the grassed area at the toe and road traffic would be unaffected. A deep-seated failure will be required to affect the squatter huts beyond the slope crest, and this is considered very unlikely". Item No. 5 Of Discussion, (GCO 1979)

The report recommended "No further study" and "Carry out routine maintenance to chunam and drains". Items No. 1 & No. 2 of Recommen-dations, (GCO, 1979)

A.2.4 <u>Architectural Office Programme No. 32H</u>

In August 1979, the cut slope was assessed by Ove Arup & Partners Hong Kong Limited (OAP) employed by the Architectural Office (AO) under AO Programme No. 32H "to examine the cutting slope and to determine measures necessary to ensure its stability". This assessment was in connection with the proposal that "Fei Tsui and Wan Tsui Roads, the new access roads to Hing Wah Estate, be widened. This widening will substantially eliminate the garden area now lying between the road and the adjacent cut slope". OAP submitted their preliminary report of the assessment to AO on 22 August 1979. Introduction (Ove Arup, 1979)

For "Small blocks of rock sliding out of the exposed face along inclined joints", OAP assessed that "Potentially unstable blocks are visible on the exposed face and should be removed" and "It is unlikely that more 10 or 20 blocks will eventually need removal". Item a) of Potential Instability & Item a) of Discussion, (Ove Arup, 1979)

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OAP noted that "Major sliding of rock and soil along the 'prominent horizontal weathered joint'" had been discussed in B&P's Landslide Study Phase IIC Report for Chai Wan area. The consultants commented that "The major 'joint' midway along the cutting and at about 10m above road level is not necessarily a true joint. It appears to thin laterally and finally disappear, rather than being truncated against another joint. It is cut and displaced by numerous near vertical joints. The 'weathered joint' is upto about 250mm thick locally and has a partly laminated structure which may indicate shear. Such shearing could be associated with the major fault along the stream-course to the north. It may alternatively be a decomposed intrusive layer". The consultants stated that "Where the feature is best exposed its inclination is 10° to 25° out of the face. The feature curves laterally as observed on the rock face and may curve within the hill or disappear completely. There is no clear evidence from the two boreholes". OAP noted that "We consider that the stepping of the 'joint' and its generally low inclination make large sliding failures along it very unlikely".

Item b) of Potential Instability & Item b) of Discussion, (Ove Arup 1979)

OAP reported that "Seepages were observed from the 'joint' which may indicate the interception of infiltrating water within the hill. However, other seepages were observed from the rock face and we consider that the majority of the rock mass, at least near the exposed face, to be sufficiently permeable to prevent the build-up of water pressures above any particular joint".

Item b) of Discussion, (Ove Arup, 1979)

OAP noted that "The 'joint' is sub-parallel to a rare set of joints inclined about 25° out of the face. These joints were seen to be generally less than 1m across and were sufficiently rare to not appear on Binnies' stereogram of joint measurements. We consider that this set of joints would only cause slipping of small blocks and this may be avoided by a) above".

Item b) of Discussion, (Ove Arup, 1979)

For "Small circular failures within Grade 5 or 6 material at the top of the face or major circular failures within Grade 3 to 6 material", OAP stated that "We agree with the form of stability analysis employed in the Binnie and Partners report and similarly conclude that soil suction is required to account for the stability of the steepest portions of the soil slope. These suctions appear to be maintained despite direct infiltration of rainwater behind the slope crest. Suctions and hence the safety of the upper soil slopes will be maintained if the ground surface behind the crest is covered by chunam". OAP further noted that "It is essential that water seepages near the reservoir be checked for their salinity. If the water is all saline we must assume that its source is, in part, from leaks in the reservoir or connecting pipes. We must also assume that any leaks may increase and become hazardous. Therefore, the reservoir would require draining and repair of its surface or of its connecting pipes".

Item c) of Potential Instability & Item c) of Discussion, (Ove Arup, 1979)

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For "Slips along the rock/soil interface", the consultants stated that "The weathering profile is expected to have formed approximately parallel to the original ground surface or at a slightly lower inclination. This corresponds to an inclination of about 10° at the eastern end increasing to a maximum of about 30° near the reservoir. We do not believe that the boundaries between Grades 2 and 3 or 3 and 4 will be of such inclination or continuity to provide a sliding surface".

Item d) of Potential Instability & Item d) of Discussion, (Ove Arup, 1979)

For "Sliding along relict joints within Grade 4 material inclined towards the cutting", it was noted that "The joint measurements indicate none with inclinations between about 25° and 70°. The angle of friction along the joints is envisaged to exceed 35°, Thus, such sliding is very unlikely".

Item e) of Potential Instability & Item e) of Discussion, (Ove Arup, 1979)

OAP recommended "i) Remove all potentially unstable blocks of rock from the exposed face", "ii) Prepare ground within ten meters of the drainage ditch at the crest of the chunam slope and remove all vegetation except trees; cover with 50mm. thickness of chunam at such a level to ensure rainwater runoff into the crest drain", and "iii) An area along the base of the slope should be isolated so as to catch any small falling rock fragments. The area should be 2 to 3m wide with a soil cover".

Items No. i), No. ii) & No. iii) of Recommendations, (Ove Arup, 1979)

SWKP reviewed OAP's preliminary report on behalf of Government and provided comments in a memorandum to Government dated 20 September 1979. SWKP stated that "We do not consider that there is sufficient detail in this report for it to be considered as anything other than a preliminary report. However we make our comments on the assumption that working drawings will be submitted in the near future". SWKP discussed that "As a general rule, B&P's Phase IIC Studies should not necessarily be considered as 'design' studies but rather as 'feasibility' studies. In the particular case of slope 11-SE-D/C42 however, we are prepared to accept that a further site investigation is probably unwarranted in view of the low risk to life".

SWKP's memo to BOO, dated 20.9.1979

Paras.. No. 3) & No. 6) of SWKP's memo dated 20.9.1979

SWKP stated that "Seepage was apparent at a number of locations along the slope length" and "the existence of seepage from the 200mm thick horizontal highly weathered based of material at the mid-height of the rock slope". SWKP recommended that "In view of the observed seepage and lack of information on ground water profiles, we recommend that horizontal drains be installed particularly at the soil/rock interface".

Paras. . No. 4 i), No. 4 iv) & No. 15 of SWKP's memo dated 20.9.1979

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SWKP concluded that "We concur generally with items (i), (ii) and (iii) in the recommendations of OAP's report". SWKP noted that "Although slope 11-SE-D/C42 can be stabilized in the immediate sense by the removal of potentially loose blocks and overhangs, it must be borne in mind that weathering is a continuous process. AO must therefore be prepared to carry out minor remedial works from time to time. Such activities will include scaling and clearing debris from drainage channels. Minor falls of loose material will be contained by the rock fall reserve and fence".

Paras.. No. 12 & No. 17 of SWKP's memo dated 20.9.1979

In their letter to the AO dated 17 April 1980, OAP summarised their review of the as-constructed drawings of the Chai Wan Salt Water Service Reservoir and the salinity tests of water samples. OAP stated that "In view of the seepage probably originating from the Reservoir we recommend that horizontal drains be installed in the rockslope to minimise the possible increase in ground water level" and "We recommend that piezometers are installed above the cut slope prior to the installation of the horizontal drains so that the effect of the drains can be monitored both during installation and the long term".

Ove Arup's letter to AO dated 17.4.1980

Sections No.4 & No. 5 of OAP's letter dated 17.4.1980

In response, SWKP stated that "We are in general agreement with the recommendations made by OAP concerning the remedial work to be carried out on the slope" and "Owing to the uncertainty of the jointing in the rock beyond the face we consider that these drains would be better installed wholly within the soil above the interface".

Para.. No. 6 of SWKP's memo to BOO, dated 28.5.1980

Two piezometers were subsequently installed in each of two boreholes in February 1982.

In September 1982, the Highways Office (HO) (reorganised as the Highways Department in 1986) referred OAP's report and SWKP's comments to the GCO. In December 1982, the GCO replied that "At present, there is no visible sign of distress of the slope. Furthermore, the consequence of failure of this slope at present is small as the grassed area and mini-bus park would act as a buffer zone" and "Therefore, I would recommend that no further work be carried out for the present. In the event that the road should be widened in the future, then a formal submission must be made to CGE/NW". Arrangement to carry out the slope works was not made at that time, probably because the works were recommended for the purposes of the future road widening.

HO's memo to GCO, dated 23.9.1982

Paras.. No. 4 & No. 5 of GCO's memo to HO dated 17.12.1982

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A.2.5 Landslip Preventive Measures Programme

In 1986, the cut slope was considered by the GCO for landslip preventive works under the Landslip Preventive Measures (LPM) Programme. One of the criteria adopted at the time was that slopes with previous reports recommending no further study would not be selected. The cut slope was not included for landslip preventive works.

In 1990, the cut slope was inspected by the GCO under the LPM Programme. It was considered that upgrading works would probably be required to bring the slope up to the current engineering standards and the case was placed in Priority Group 5 for further action.

A.3 PREVIOUS LANDSLIDES AFFECTING THE OPEN SPACE IN FRONT OF THE SLOPE

Four previous landslides occurred at the cut slope in 1985, 1986, 1987 and 1993. The approximate locations of these landslides are shown in Figure 6 in this landslide investigation report.

Two of the incidents, viz. the landslides in 1987 (GCO Incident No. HK 87/5/10) and 1993 (GEO Incident No. HK 93/9/11), were reported to the GCO/GEO after the failures. The landslides were of small scale, with a failure volume not more than 50 m^3. The landslide debris came to rest in the open space in front of the cut slope. Some windows of the Chai Wan Baptist Church on the other side of Fei Tsui Road were reported to have been damaged by small pieces of fly rock in the 1993 landslide.

Premchitt (1991) & Chan (1995)

The 1985 landslide was identified from the 1985 aerial photographs. The 1986 landslide was indicated on a plan in a GEO file. No other information about these incidents can be found. The scale of these two landslides was apparently smaller than the 1987 and 1993 incidents.

Following the 1987 landslide, the open space in front of the cut slope was fenced off. Subsequently, the GEO advised Lands Department that "the fenced off area at the toe of the landslip should not be used until the slope is stabilized by future development" and that potential users "should take instability of the slope into account when considering suitability of the site for the intended purposes".

GEO's memo to District Lands Office, Hong Kong East (DLO/HKE) dated 3.9.1987 & 8.6.1994

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In response to the proposed allocation of the eastern part of the open space opposite the playground and Chai Wan Baptist Church to the DSD as a storage compound in October 1994, the GEO advised that "the slope is still potentially unstable" and noted that "the site will be used for open storage purpose ONLY. Habitation at this site will not be permitted". Subsequently, the DSD proposed to relocate the storage compound to the western part of the open space directly below the service reservoir. The western part of the open space and the cut slope directly above it were allocated to the DSD with effect from 27 April 1995.

GEO's memo to DLO/HKE dated 7.10.1994

A.4 REFERENCES

Binnie (1977). Government of Hong Kong Landslide Study Phase IIC Report on the Stability of Slopes in the Chai Wan Area. Binnie & Partners (Hong Kong) Consulting Engineers for Government of Hong Kong, 64 p. plus 12 Plates, 6 Appendixes and 23 Drawings.

Chan, W.L. (1995). Hong Kong Rainfall and Landslides in 1993. Geotechnical Engineering Office, Hong Kong, 214 p. plus 1 Drawing. (GEO Report No. 43).

Geotechnical Control Office (1979). Cut Slope 11SE-D/C42, Sui Man Road. Geotechnical Control Office, Hong Kong, Stage 1 Study Report no. S1 85/79, 3 p. plus attachments. (Unpublished).

Ove Arup (1979). Cutting at Hing Wah Estate, Chai Wan. Preliminary Report submitted to Architectural Office attached with Ove Arup's letter referenced 9165/11/PAR/IK dated 22 August 1979, Ove Arup & Partners Hong Kong Limited for Architectural Office, 6 p.

Premchitt, J. (1991). Hong Kong Rainfall and Landslides in 1987. Geotechnical Engineering Office, Hong Kong, 108 p. plus 1 Drawing. (GEO Report No. 4).

Water Supplies Department (1995). Report on the Landslide at Fei Tsui Road, Chai Wan on 13.8.95. Water Supplies Department, Hong Kong, 7 p. plus 3 Drawings, 6 Appendixes and 11 Plates.

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一九九五年八月十三日
柴灣翡翠道山泥傾瀉事件報告

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序言

土力工程處的一貫政策，是向公眾及岩土工程業界公開有參考價值的資料。為此，我們選擇部份內部報告，編製為土力工程處報告 (GEO Reports)。 此等報告可於土木工程拓展署網頁 (http://www.cedd.gov.hk) 下載。 我們亦印備部份土力工程處報告，並以印刷成本價發售。

土力工程處又出版其他工程指引刊物系列 (GEO Publications)。 此等刊物系列及有印備的土力工程處報告，均由政府新聞處負責售賣，購買方法詳載於本報告末端第二頁。

土力工程處處長
陳健碩
二零零六年八月

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前言

本報告共分兩冊，第一冊爲黎佐賢爵士的獨立報告，記錄了他對一九九五年八月柴灣翡翠道山泥傾瀉事件及應得教訓的意見。由土木工程署轄下土力工程處所編寫的第二冊，則記述山泥傾瀉調查的詳細結果。黎佐賢爵士檢閱及同意第二冊報告的內容，並以之爲他在第一冊內所作評估的依據。

陳潤祥

土力工程處副處長(規劃及標準)
陳潤祥

第一冊：
就土力工程處的調查所作的
獨立檢討

黎佐賢爵士
英國伯詩亞

本報告源於一九九六年二月土力工程處
一九九五年八月十三日柴灣翡翠道山泥傾瀉事件報告

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1. <u>引言</u>

一九九五年八月十二及十三日，颱風海倫過港後，本港出現連場豪雨，引致超過 120 宗山泥傾瀉。其中在柴灣翡翠道所發生的一宗，瀉下的泥石掩埋路面，導致一人喪生，一人受傷。

土木工程署轄下的土力工程處於一九九五年八月十三日早上，展開對這次山泥傾瀉的調查。調查結果在土力工程處(一九九六)報告中發表。

當局決定就土力工程處的調查，邀請筆者進行獨立的技術檢討，並向香港政府提交報告。筆者於一九九五年九月五日至八日、十月三十一日至十一月三日、及十一月二十七日至十二月一日三段期間專程訪港，並到事發地點兩度視察，及與土力工程處討論調查計劃和各階段取得的調查結果。筆者亦曾直接與土力工程處人員檢討了該處所完成的調查報告初稿。

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2. <u>山泥傾瀉的描述</u>

山泥傾瀉發生的地點，是一個挖建於火山岩山咀的削坡，該山咀自哥連臣山側面朝東北偏東方向伸延而下。事發地點的平整工程於一九七二至一九七六年間進行，是興華邨第 II 期發展工程的一部份。翡翠道建於斜坡北面，最高約 27 米，斜坡的整體角度約 60 度。建於一九五九年的柴灣海水配水庫，座落在山咀頂部鄰接削坡南方。在配水庫東面，位於削坡頂部上方的一帶山咀，是長有植物的平坦土地。直至一九九一年，該處一直是寮屋區。山咀南側是個山谷，有一條小溪，其出水口位於翡翠道東端。

挖掘出來的斜坡在其約三分之二的高度處，有一度窄坡級，斜坡的上部在建築時已鋪上灰泥，其下面部份則露出岩石。山泥傾瀉時斜坡面爲植物所覆蓋。斜坡前面是一條 12 米闊的平地，分隔了斜坡和翡翠道，而該平地被圍起來，部份長有野草，部份則用作臨時貯存渠務工具。

斜坡自挖掘以來，曾分別在一九八五、八六、八七及九三年發生四次小規模崩塌事件，塌下的泥石均落在岩石面前面的平坦空地內。

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這次山泥傾瀉分兩階段發生，其情況由目擊人士詳細記錄。第一次崩塌於八月十三日凌晨約零時五十五分發生，位置在九三年崩塌地點毗鄰，即最後來山泥傾瀉的東端。崩塌涉及數十立方米的岩石。與過往四次小規模崩塌一樣，泥石落在前面的平地範圍。第二次，即主要的崩塌約於 20 分鐘後發生，這次崩塌把斜坡後削約 40 米，造成的殘痕最長達 90 米，平均垂直深度約 15 米。當時約有 14000 立方米的泥石從殘痕位置塌下，超越圍起的平地及翡翠道，瀉入球場，及堆積在柴灣浸信會旁，最高 6 米。泥石堆從坡腳向外瀉的範圍最遠是 33 米，沿翡翠道的覆蓋長度僅逾 90 米，可見並沒有明顯地向橫擴展。

山泥崩塌後的整體外觀顯示可能是一次平移崩塌，岩體沿一個北向的傾斜面向斜坡外滑動。其後的調查證實了這看法。雖然這種滑動是造成岩坡不穩定的一種常見機制，但同類型的大規模崩塌事件在本港並不常見。事實上，翡翠道山泥傾瀉是本港記錄

得最大規模的永久削坡遽然崩塌事件。

3. 土力工程處報告的檢討

現就土力工程處報告的每一節內容順序作出檢討。

「1. 引言

報告的引言說明這次山泥傾瀉的背景，並概述調查工作的主要成份。調查所採用的方法和規模，完全符合這次山泥傾瀉的性質。

2. 事發地點的描述

事發地點的描述連同地圖及照片，提供了位於政府地上事發地點的充份實況。值得注意的是，地面排水系統分佈狀況，是按搜集到的資料，以及觀察到渠管淤塞的情況估計而得。

3. 山泥傾瀉的描述

山泥傾瀉的描述包括了其幾何形態及塌下泥石的性質和分佈。報告亦突出了翡翠道山泥傾瀉的不尋常規模。此外，應特別注意泥石瀉出的距離，較本港山泥傾瀉的預期距離爲遠，顯示了崩塌時滑動物料的流動性較高。如果滑動泥石的表現與一般本港因豪雨引致的山泥傾瀉相同，則主要泥石體只能瀉至翡翠道中間。

報告又指出，據觀察得知，因山泥傾瀉而外露的一截直徑 24 吋地下鹹水管，於崩塌後有水流出。

4. 事發地點的歷史

本節簡述事發地點的發展歷史。對於斜坡下面留下一條平坦地帶的原因，仍未能肯定，但相信在某階段曾考慮擴闊及伸延翡翠道至香港島的南面。因爲這情況，雖然斜坡已登記，過往亦經各種檢討及土力工程處日常處理斜坡的程序，但均沒有與在它旁邊的道路房屋共同考慮。

報告提及該斜坡曾發生四次小規模崩塌事件。其中兩次已正式報知土力工程處。這兩次崩塌的泥石均瀉下斜坡前

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面的平地範圍。一九八七及九三年的事件中，崩塌底部約在路面對上 10 米處，只涉及斜坡的上半部份。檢查當時的記錄及照片後，發現這兩次崩塌的底部，和一九九五年八月十三日山泥傾瀉的底部相同。一九九三年的崩塌事件中，有些飛濺的岩石擊破浸信會的一些窗子。由於這些窗子處於相對較高位置，山泥傾瀉必須有某程度衝力撞擊地面，才可使石塊飛出的軌跡如此陡峭。這情況顯示崩塌是翻倒型的。

5. 雨量記錄分析

離事發現場以北約 220 米的和興樓天台，裝有一個雨量計，從一九七九年八月開始記錄雨量，這些記錄用於評估斜坡於過往事件中所經歷的豪雨。

一九九五年八月本港雨量紀錄，較歷年來於皇家天文台八月份錄得的雨量爲高，該雨量於月初特別強烈。以七天以上的降雨時段計算，雨量計所錄得的雨量是事發地點於過往豪雨中錄得的最高雨量。以十二小時以下的降雨時段計算，這次豪雨的雨量強度與過往豪雨事件中所得的雨量相若。

6. 山泥傾瀉的經過

這次山泥傾瀉有數位目擊人士，包括警務人員，因此能可靠地重組山泥傾瀉兩個階段經過的時間(見第 2 節)。

7. 事發地點地下情況的特徵描繪

7.1 地質

當局於清除大部份泥石後，在崩塌處進行地質勘察，並輔以探溝和鑽孔，以調查該處的地質狀況。崩塌於火山岩內發生，在斜坡頂部主要是完全至高度風化的凝灰岩，而在斜坡較低部份則主要是中度至輕度風化的凝灰岩。岩體由兩組陡峭的節理貫穿。兩組節理分別傾向西北偏西及東北，聯合形成崩塌後的滑坡崖。

該斜坡最引人注意的地質特徵，是該處有一層因凝灰岩蝕變而形成的廣闊高嶺石黏土層,該高嶺石黏土層厚達 0.6 米，並有黏土岩脈。該黏土層起伏伸延，傾向北面，其傾角爲 10 至 25 度，有些起伏部份可能是由小規模斷層斷錯而成。土力工程處特別對這高嶺石層的特徵進行詳細研究。

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該高嶺石黏土層被判斷爲一個剪切區，與凝灰岩的原來層理平行。該土層內的原來物質，可能因九龍花崗岩深成岩體侵入而產生蝕變和變形。該深成岩體的露頭，就在崩塌部份以北約 70 米處。該土層有一組節理傾向北面，傾角爲 10 至 25 度，大約和黏土層平行。崩塌部份的底層表面，即發生位移的部份，並不完全沿該高嶺石層發生。在其上部，底層表面是位於黏土層之上(在含高嶺石岩脈的蝕變凝灰岩內)；而在崩塌部份的下部，雖然有很多黏土曾被滑下的泥石移去，但其底層和該黏土層看來是相同的。

7.2 土壤和岩石的性質

在崩塌範圍內，當局對能影響斜坡的穩定性的物質，包括該高嶺石黏土層和黏土岩脈、蝕變凝灰岩和風化火山岩節理等，進行了一系列全面的分類、強度和固結試驗。

當局根據蝕變凝灰岩內高嶺石的相對比例和相伴的岩脈情況，測定抗剪強度的幅度。當剪切發生於有較高高嶺石含量的土壤時，則ϕ'和 c'的下界值分別設爲 22 度和 0，而$\phi'=29$度及 $c'=0$ 則被採納爲具代表性的數值，適用於構成崩塌底層表面的含豐富高嶺石的蝕變凝灰岩層。

7.3 地下水情況

事發地點發現有兩個地下水體系，一個位於較深的區域地下水位，另一個則位於較淺的上層滯水位。該上層滯水位的形成是由位於較高位置的高嶺石黏土層阻攔水份而引致。

根據山泥傾瀉後所鑽探的鑽孔觀察所得，上述的區域地下水位在該高嶺石黏土層下面 4 米至 8 米處。如本報告較早前(第 2 節)所述，事發地點位於一個山咀的邊旁，該山咀由位於其西南方的高地向前延伸。這樣的地形，會使由高地流下的地下水分散而遠離山咀，因此山咀下面的地下水位，須依賴該處地面水滲入補充。

上層滯水位在高嶺石黏土層的頂部形成，而且有文件證據顯示位於這個黏土層或其上面的岩石表面曾觀察到有輕微滲水情況。在調查期間，在崩塌範圍內的高嶺石黏土層之上亦觀察到滲水情況，而滲水則隨着雨量增加。由於在該高嶺石黏土層之上約 5 米處的崩塌滑坡崖腳，並無明顯的滲水，所以估計該上層滯水位大概位於高嶺石黏土層之上 1 米至 4 米處。

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8. 柴灣海水配水庫及其相連輸水系統的情況

報告扼要闡述海水配水庫的情況，而根據抽水紀錄，可以確定在一九九五年八月十三日凌晨一時十五分前，即是第二次的主要崩塌發生之前，該配水庫並無不正常運作或出現嚴重滲漏的跡象。配水庫的西半部曾進行滲漏試驗(基於安全理由，東半部一直空置)，但無發現有可量度的滲漏。

在發生山泥傾瀉時，直徑 24 吋的輸水管斷落了一段，長 21 米，因而於一段時間內，有水流入崩塌斜坡的泥石中。所有斷落的水管都能檢回，並無發現水管或接口裝置有任何舊或持續的滲漏跡象。

根據這些觀察，証明該海水配水庫的整體滲漏問題，並不是觸發山泥傾瀉的一個因素。

雖然如此，仍進行了有關土壤和水試樣的氯化物含量試驗，以確定是否有海水滲漏和其滲漏程度。在配水庫以北約 20 米的編號 DH9 鑽孔，以及在配水庫東北約 90 米的編號 H16 鑽孔，其區域地下水含有高氯化物含量，從而差不多肯定多年來該配水庫亦發生滲漏。從崩塌西面削坡滲出的水，以及在崩塌底層表面之上滲出的水，也有顯著的氯化物含量。於崩塌極西處的土壤試樣，有較高的氯化物含量，這大概顯示該處曾受折斷水管流出的海水污染。反之，從主要崩塌部份和從滑坡崖東部所收集的土壤試樣，其氯化物含量則較低，這差不多可肯定沒有受到從斷裂的直徑 24 吋輸水管所排出的海水污染。滲水的氯化物含量高於預期的天然水平，從主要崩塌的底層表面所收集的土壤試樣也發現含氯化物，兩者均提供了確實的證據，顯示高嶺石黏土層對上層滯水位的形成，擔當着重要的角色。

9. 工程分析

所進行的工程穩定性分析，是假定在該岩石斜坡沿向外傾斜 20 度的表面發生平移滑動，而泥石脫離由風化火山岩節理構成的滑坡崖。就事發地點的情況來說，這是合理的地質模型。

假定滑動體並無受到水壓影響，則當抗剪角(φ')爲 25 度或以下時，山泥傾瀉就會發生。這個φ'的數值是在量度到的強度之內，但較一般認爲合理的運作數值 29 度爲小。

這項分析假定上層滯水位的情況，當滑動面上的有效水壓上升至 1 米、2 米和 4 米，而滑動面的φ'數值分別爲 26.5

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度、28.0 度和 31.5 度時，則理論上就會發生山泥傾瀉。因此，所得的結論是：根據合理設定的上層滯水位，斜坡理論上可發生平移崩塌。

崩塌的滑坡崖是由兩組陡峭的節理組成，而在崩塌頂部的後面，則有一些表面裂縫，雖然這些裂縫的廣度或深度仍未探究，但其方向和這兩組節理的方向相若。當豪雨時，這些裂縫可以很快注滿水。因此，可以作另一個有關地下水的假定，即當滑動面的ϕ'值爲 29 度，整個滑坡崖內陡峭而開縫的節理，會受水壓影響，引致岩體滑動。在這情況下，節理系統上部須先注滿 9 米至 10 米深的水份，才可以引致理論上的不穩定。這個過程可能是斜坡崩塌過程的其中一個引發因素。

10. 山泥傾瀉的診斷

這節根據調查所得的結果，對斜坡崩塌過程提供一個全面的概述。筆者亦贊同這項分析。

經過爲時多天，異常多雨的天氣後，在一場豪雨中，地下水位上升，影響一層連續而受層理所支配、且內含相對薄弱物質的岩層，引發翡翠道山泥傾瀉。有關這次山泥崩塌的數個特點，例如：極端的雨量及事發前一段時間的高雨量、火山岩中出現一個受層理支配的平直岩層、岩層內大量的高嶺石黏土及這次平移崩塌的規模等，雖不是異常罕見，卻是不常見的。

第一次發生的小型崩塌，可能由翻倒或滑動加翻倒而造成。這初步的移動，可能逐漸啓動了隨後的崩塌過程，引致短時間後發生較大的崩塌。在該段 20 分鐘時間內，可能曾發生一連串漸進但輕微的調整。這些調整導致後來的崩塌機制，以及瀉下相對較遠的泥石。

有證據顯示地下水和土壤受氯化物污染，而這個情況差不多肯定是由於海水配水庫長期滲水所造成，但無證據顯示配水庫的整體滲漏對引發山泥崩塌有任何影響。

11. 結論

所述的結論非常扼要地重述這次山泥傾瀉的成因，而筆者亦贊同所述的成因。」

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4. 檢討土力工程處報告的結論

土力工程處就翡翠道山泥傾瀉所進行的調查，是全面和具專業水平的。報告準確報道了調查的結論，並就山泥傾瀉的引發因素和成因，作出合邏輯的結論。筆者對報告書所述及的主要事項均表贊同。

5. 所汲取的教訓

翡翠道的山泥傾瀉並無發現任何新的、未於香港以前的地質或山泥傾瀉預防工作所認識的特徵。但有些與這次山泥傾瀉成因和事發地點歷史有關的特徵，則值得注意。

5.1 火山岩崩塌如何受地質結構控制

翡翠道山泥傾瀉的崩塌面是受層理支配，香港沒有同類同規模的平移崩塌的報告。香港的火山岩由於曾經褶曲，所以任何存在的層理，其傾角和方向均可能與地形產生不利的組合。和翡翠道同類的狀況，有可能在其他地方出現。

檢閱事發地點的歷史(附錄 A 的 A.2.2 節)，找到有關事發地點過往的文件。在該處進行場地平整工程時，就該斜坡當時情況所撰寫的報告，提到「道路和坡級中間有顯著的水平風化泥層(照片 8)」「約 100 毫米闊及最少 50 米長」。該「照片 8」於一九七六年拍攝，顯示翡翠道山泥傾瀉地點當時的情況。該照片並轉載於土力工程處報告爲照片 2。從該張一九七六年照片所見到非常明顯的平直地質結構，可能就是「顯著的水平風化泥層」。但很明顯，這地質結構並不是水平，而是貫穿岩體並向坡面外傾，其傾角可能引致滑動。雖然其後有其他組織曾視察該斜坡，但他們似乎沒有探討過這個地質結構會引致發生大規模平移崩塌的可能性。

根據這些事後獲得的資料，我們必須認識到，當連貫的不連續面以 20 度(甚或更小)傾向岩石面時，可能導致平移崩塌。

5.2 火山岩崩塌如何受礦物控制

蝕變或風化岩石含有高嶺石的情況，在香港並不罕見。在這次事件中，高嶺石層相對頗厚和延續，則並不常見。用傳統的鑽石鑽探，除非預先懷疑到這類岩層的存在，而在鑽探時採用特別措施，否則可能無法採獲這類岩層，原因是鑽挖過程中的沖水會把岩層物質帶走。

導致翡翠道的蝕變凝灰岩含有高嶺石黏土層的因素，在此階段無法以足夠精確度加以闡述，從而預測這類物質的存在。

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5.3 多次小規模崩塌

自從當初的場地平整工程完成以後，二十年間翡翠道曾發生四次小規模的山泥傾瀉。在其他岩石斜坡不穩定事件中，這些山泥傾瀉曾證實是其後一次大規模崩塌的先兆。

在土力工程處預防山泥傾瀉所用的綜合方法中，應找出一些辦法，識別某些多年來曾發生多次山泥傾瀉的地點，進行適當的鑒証研究。

5.4 配水庫滲漏

雖然在這次事件中，海水配水庫出現的整體滲漏並無導致這次山泥傾瀉的發生，但有證據顯示土壤和地下水因長期的滲漏而受到污染。因此，當局須要留意，配水庫的滲漏有可能影響局部的地下水狀況，而這可能轉而影響斜坡的穩定。

6. 參考書目

土力工程處(一九九六) 一九九五年八月十三日柴灣翡翠道山泥傾瀉事件報告第二冊，山泥傾瀉調查結果。香港土力工程處，64 頁。

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第二冊：
山泥傾瀉調查結果

香港政府
土木工程署
土力工程處

本報告源於一九九六年二月土力工程處
一九九五年八月十三日柴灣翡翠道山泥傾瀉事件報告

撮要

一九九五年八月十三日，翡翠道柴灣浸信會對面的斜坡發生山泥傾瀉，導致一人喪生，一人受傷。登記編號 11SE-D/C42 削坡的一部份，以及削坡頂部毗鄰柴灣海水配水庫的土地，於這次事件中突然崩塌。

土力工程處於一九九五年八月至十二月期間全面調查了這次山泥傾瀉。詳細研究工作包括翻查資料文件、分析雨量記錄、訪問山泥傾瀉目擊人士、進行現場調查、場地勘探、研究配水庫及輸水系統在事件中的角色、以理論方法進行穩定性分析及診斷事件的成因。

調查所得的結論是：山泥傾瀉可能主要是由於崩塌前持續下了極大的雨，使斜坡內廣闊而低強度的黏質土層內的水壓增加所致。

山泥傾瀉調查的詳情及所得的結果載於本報告內。

目錄

1. 引言

一九九五年八月十三日清晨，柴灣翡翠道柴灣浸信會對面的斜坡發生山泥傾瀉(照片 1 及圖 1)。瀉下的泥石掩埋了翡翠道一段 90 米長的路面，導致一人喪生，另外一人受輕傷。

土木工程署轄下的土力工程處於事發後就這次山泥傾瀉進行詳細的調查，並於一九九五年九月二十八日發表了一份進展報告(土力工程處，一九九五年)。

調查工作於一九九五年八月至十二月進行，主要項目如下：

(a) 就事發地點的發展歷史及山泥傾瀉前發生的事，翻查所有已知的有關記錄；

(b) 分析雨量記錄；

(c) 訪問目擊山泥傾瀉的人士；

(d) 於事發地點進行地形測量、詳細的觀察和量度；

(e) 進行地質勘察；

(f) 採用鑽探、現場試驗及室內試驗進行全面的場地勘探；

(g) 研究柴灣海水配水庫及其輸水系統在今次事件中的角色；以及

(h) 以理論方法分析崩塌時斜坡的穩定性。

本報告載列這次調查的結果。調查工作的詳情及所得的結果載於一套文件中，該套文件存放於土木工程署大樓地下一樓土木工程圖書館。

2. 事發地點的描述

翡翠道發生山泥傾瀉的地點見於圖 2。崩塌的地方包括該處一幅削坡其中的一部份及削坡頂部毗鄰柴灣海水配水庫的土地。

政府委聘編製斜坡紀錄冊的顧問於一九七七年登記該削坡，編號 11SE-D/C42。削坡座南向北，山泥傾瀉前，仰角平均約爲 60 度，最高約爲 27 米。削坡的下半部露出岩石，上半部則蓋上灰泥。照片 2 爲一九七零年代中期削坡形成後所拍攝的照片，顯示削坡當時岩石面和灰泥部份的情況。一九九四年拍攝的照片則顯示削坡表面長有大量野生植物(照片 3)。

柴灣海水配水庫爲一座素混凝土儲水結構物，鄰接事發現場西南。配水庫並無上蓋，面積約爲 25 米乘 40 米、深度 5.4 米，被一塡土堤環繞(圖 1)。該塡土堤最高約 6 米，

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仰角約 30 度，於一九七七年被登記入斜坡紀錄冊內，編號爲 11SE-D/F27 塡土坡。

發生山泥傾瀉前，這塡土堤和其與削坡之間的土地，均長有植物。

山泥傾瀉現場東南爲一個走向東北偏東的天然山咀，地面向南下斜至一個山谷。山咀從前是一個寮屋區，現在只殘留一些混凝土地板和舖砌的地面(圖 3)。

於一九九五年八月十三日崩塌的削坡前面，是一條約 12 米闊的平坦空地，毗鄰是翡翠道。除西面部份自一九九五年四月二十七日分配予渠務署作爲臨時貯貨場外，整幅空地均空置(圖 1)。

位於山泥傾瀉現場下面的一段翡翠道，約 7.3 米闊，沿道路北面有一條約 3.3 米闊的行人路。

從可得的文件紀錄和事發後現場觀察結果，可推斷該處的地面排水系統的大概位置，如圖 3 所示。於事發後視察所見，位於削坡未塌下部份的地面排水渠，爲腐植土和有黑漬的粉質土淤塞(照片 4)。位於事發地點以西的削坡頂部的排水渠，則積有約 20 毫米深的落葉及粉質土。

在這次事件中塌下的土地是政府地。削坡西部已分配予渠務署，而削坡上面的土地，有一部份位於柴灣海水配水庫範圍內(圖 1)。

關於斜坡維修方面，渠務署指出「於一九九五年四月二十九日至一九九五年五月三日期間，在渠務署配地範圍內已進行一般場地清理，包括清除地面排水渠上的泥石。並於所屬範圍內定期進行場地檢查，以保證全部地面排水渠均正常運作，未受堵塞」。至於配水庫的維修工作，水務署指出配水庫「已進行定期維修」。在山泥傾瀉前的最近一次「剪草及清理地面排水渠」工作，於一九九四年十二月三十日進行。

3. 山泥傾瀉的描述

一九九五年八月十三日早上所拍攝山泥傾瀉事件的照片載於照片 1，橫貫事發地點的剖面則載於圖 4。

根據目擊證人所作的描述，這次山泥傾瀉涉及兩次崩塌——主要的崩塌於一九九五年八月十三日凌晨一時十五分左右發生，約二十分鐘前則有一次較小型的崩塌。山泥傾瀉的平均深度約 15 米，較本港因豪雨造成的其他斜坡崩塌爲深。這次塌下的泥石，約爲 14000 立方米。從圖 2 可見，塌下的泥石覆蓋了削坡的下半部、削坡前的空地和翡翠道，部份泥石更瀉入道路對面的球場。亦有部份泥石堆在柴灣浸信會西南角，於該處，泥石堆最高約爲 6 米。由崩塌殘痕的頂部開始量度，泥石所移動的平面距離最遠約爲 70 米。泥石堆最闊約 90 米。

塌下的泥石主要爲粗礫至漂石大小的中度及高度風化凝灰岩塊，中間塡塞黏質粉質幼砂至砂質黏質粉土。當中並混有人造物料(圖 5)，包括多段直徑 24 吋和 6 吋的石棉水泥管、混凝土地面排水渠殘段、砌石牆殘塊、混凝土塊及集水井、鐵絲網圍欄、鍍鋅

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鋼管、電纜及破毀的電燈柱。部份地面排水渠殘段塞滿腐殖土和有黑漬的泥土(照片 5)，顯示這些殘段在山泥傾瀉前已淤塞一段時期。

崩塌頂部附近已廢棄寮屋區內的舖砌地面發現一些裂縫，最闊約 15 毫米，但這些裂縫究竟是在山泥傾瀉之前或之後形成，則不得而知。

本處曾翻查現存關於本港自一九八四年以來的山泥傾瀉記錄，發現所接報的山泥傾瀉事件中，只有約 0.1%的事件涉及超逾 5000 立方米崩塌泥石。以崩塌規模來說，翡翠道山泥傾瀉塌下約 14000 立方米泥石，極不尋常。事實上，這是本港過去十年所接報的快速削坡崩塌事件中，最大的一宗。

從泥石末梢至崩塌殘痕頂部的仰角，可反映塌下泥石的流動性，角度越小，泥石越容易流動。以本港普遍由豪雨造成的山泥傾瀉而言，角度一般爲 30 度以上(Wong & Ho，一九九六)，但在翡翠道山泥傾瀉事件中，這個角度只有 24 度，顯示泥石的流動程度較一般削坡崩塌爲高。

崩塌殘痕近頂部處露出一截直徑 24 吋已折斷的地下鹹水管(照片 1 及圖 2)。距離這條鹹水管以東約 4 米處，於崩塌殘痕近頂部露出另一條直徑 6 吋已折斷的地下鹹水管(圖 5)。有目擊人士表示於事發後的八月十三日早上，看見有水從這條直徑 24 吋水管流往山泥傾瀉的地點。但土力工程處訪問的目擊人士，則沒有報稱注意到有一條折斷的直徑 6 吋水管。

4. 事發地點的場地歷史

撮述於附錄 A 的場地歷史，是透過研究該處的航空照片和翻查現有資料文件而得。

有關事發地點可得的最早航空照片，拍攝於一九二四年。照片顯示事發地點位於一個走向東北偏東的山咀，當時並未發展。柴灣海水配水庫於一九五九年興建。該處的削坡是在一九七二至七六年間由建築處(一九八六年重組爲建築署)建成，作爲興華邨第二期發展工程的一部份。

削坡以往曾發生多次小規模的山泥傾瀉(圖 6)。

土力工程處曾分別於一九八七年(照片 6)及一九九三年(照片 7)接獲該處發生山泥傾瀉的報告，塌下泥石的體積分別約爲 50 和 30 立方米。在兩次事件中，崩塌均發生於削坡的上半部，崩塌的底部露於翡翠道 10 米之上。除了在一九九三年的山泥傾瀉事件中，浸信會有一些窗子被飛濺的碎石所損毀外，其他塌下的泥石都落在斜坡前的平坦空地上。

一九八五年的航空照片顯示該處的斜坡曾發生崩塌，而土力工程處檔案內有一份平面圖，顯示該處曾在一九八六年發生另一次山泥傾瀉。除此之外，再無其他有關這兩次事件的資料。這兩次山泥傾瀉的規模，明顯較上文提及在一九八七及一九九三年發生的爲小。

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5. 雨量記錄分析

興華邨和興樓天台裝設有自動雨量計(編號 H14)，於事發地點以北約 220 米處(圖 1)。該雨量計在一九九五年七月及八月所錄得的每日雨量，以及由一九九五年八月十一日至十三日所錄得的每小時雨量，均載於圖 7。

由八月十二日至山泥傾瀉發生時均非常大雨。山泥傾瀉前的十二小時及二十四小時期間所錄得的雨量，分別為 231 毫米及 370 毫米。而六十分鐘的最高雨量則為 94.5 毫米，於八月十二日晚上十一時三十分至八月十三日零時三十分所錄得。

在山泥傾瀉前的三十一日期間，該雨量計所錄得的總雨量為 1303 毫米。高於天文台的雨量計自一八八四年以來所錄得的最高月雨量。根據天文台以往的雨量數據，按不同的降雨時段分析這次雨量強度的重現期，發現在三十一日這時段內所錄得的雨量最猛烈，相應重現期約為九十五年一次。

編號 H14 雨量計自一九七九年裝置以來，所錄得過往影響事發地點的嚴重豪雨的雨量變化，以及一九九五年事發前的雨量變化，均載於圖 8, 以作比較。從圖中可見，以十五日以上的降雨時段計算，於山泥傾瀉前錄得的雨量，較前為高。以七天或以下的降雨時段計算，這次豪雨的雨量強度則與過往所錄得的雨量相若。

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6. 山泥傾瀉的經過

根據十二名目擊人士的陳述、警務處和消防處的事件報告，以及本處於事發後在現場進行的觀察，山泥傾瀉的經過可重組如下：

翡翠道的山泥傾瀉，涉及兩次崩塌。一九九五年八月十三日零時五十五分左右，柴灣浸信會對面的削坡東部，發生一次小規模的崩塌。一名警務人員於零時五十六分把事件報告警務處柴灣區控制中心。這次崩塌來得很突然，涉及事發地點東面削坡的上半部，可能於兩個相距數米的位置同時發生。塌下的泥石瀉入空地，崩塌的規模可能為數十立方米。這次崩塌的範圍和規模，可能與一九八七年及一九九三年接報的山泥傾瀉相似。

凌晨一時十五分左右，第二次崩塌突然發生，這是導致傷亡的主要崩塌。這次崩塌涉及整個山泥傾瀉地區，包括削坡和削坡上面的土地。泥石衝下，瞬息間掩蓋了下面的空地和翡翠道。滑坡崖頂部露出一段直徑 24 吋的水管，於事發後的早上有水從這折斷的水管流入山泥傾瀉地區。於凌晨一時十七分，警務處接獲一名市民報告這次崩塌，而消防處則於一時十八分接獲警務處通知。

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7. 事發地點的地下情況

7.1 概述

事發地點的地下情況是根據文件和實地研究所得的資料而確定。文件研究工作包括翻查現存數據，而實地研究工作則包括地質勘察及場地勘探。

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事發地點的地質勘察工作於一九九五年八月十四日展開，並且在清理山泥傾瀉泥石，及在岩土勘探期間，仍繼續進行。

場地勘探工作於一九九五年八月十七日展開，其中大部份是於一九九五年九月中於緊急搶修工程完成後始進行。場地勘探工作包括了 11 個垂直鑽孔、3 個斜向鑽孔、2 個觀測井、9 個探井及 4 個探溝(圖 9)。

7.2 地質

香港地質測量組於事發後，在現場觀察到的地質特徵，載於圖 10。而橫貫山泥傾瀉地點的典型地層剖面則載於圖 11。

崩塌地區的地質由風化的火山岩組成。在斜坡頂部最上層爲約 3 米的填土，覆蓋於完全風化至輕度風化的凝灰岩上。風化岩的厚度，由崩塌範圍東面的 4 米左右增至西面的 11 米左右。

按照該區的 1:20000 比例地質圖(土力工程處，一九八六)，香港地質測量組測繪事發地點的岩石屬於淺水灣火山群的城門組。進行地質調查時，曾採取削坡底部輕度風化岩石的試樣，而在地質報告(Strange & Shaw，一九八六)中則詳細描述手樣本的薄片。該處岩石被形容爲條紋斑雜、含火山礫的凝灰岩。而現時在事發地點進行的地質勘探，亦證實這是事發地點的主要岩性。

風化凝灰岩遍佈節理。主要是兩組傾向西北偏西及東北的陡峭(60 至 85 度)節理，該兩組持續、間距爲密至中等、粗糙而平直的節理普遍是緊閉的。這些節理面形成崩塌的側面及滑坡崖(照片 8)。於一九九五年八月十四日視察所見，該些節理看來是最近才露出的。此外亦發現一組低傾角的節理，主要傾向爲北面 10 至 25 度。

沿著整個削坡下面 5 至 7 米部份，由中度至輕度風化凝灰岩組成。這削坡的下部於山泥傾瀉中並無移動(照片 9)。

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事發地點的一個顯著特色，是有一層橫向持續、並含豐富高嶺石的蝕變凝灰岩，大致傾向北面約 10 至 25 度。據估計，於山泥傾瀉中，這層蝕變凝灰岩爲主要滑動面。橫跨整個事發地點，這層凝灰岩被一連串細小、北向的斷層所斷錯，形成以梯級狀朝西上升。每個斷層的垂直斷錯多爲 1 米以下，但崩塌地點西面有一斷層的垂直斷錯多達 3 米左右。

在削坡連接事發地點東邊沒有崩塌的部份，含豐富高嶺石的蝕變凝灰岩層厚約 0.5 米，上面蓋有中度至輕度風化凝灰岩。這層凝灰岩傾向北面約 20 度，屬高度高嶺石化和完全風化，並含大量高嶺石岩脈(照片 10)。岩脈厚度由 2 毫米至 20 毫米不等，其中一些岩脈與該凝灰岩層的走向近乎平行。

類似的含豐富高嶺石蝕變凝灰岩，亦可以在崩塌底層的東部和中部見到(照片 11)。雖然在一些地方，這次山泥傾瀉已侵蝕該岩層的上部，但剩餘的最厚仍達 0.6 米左右。

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至於崩塌地點西部，探溝露出的崩塌底層滑動面局部較陡，傾向北面約 30 度，蝕變凝灰岩層因這次崩塌而大部份被淘蝕，以致塌下的泥石之下可以直接見到一些中度至輕度風化的凝灰岩。於滑坡崖及崩塌殘痕前緣附近，沿底層滑動面下，可見到該蝕變凝灰岩的殘餘，承托於中度至輕度風化凝灰岩之上(照片 12)。

於西面滑坡崖底部的蝕變凝灰岩層，較爲「分散」。它較厚(約 3 米)，高嶺石化程度較低，雖然大部份屬完全至高度風化，但仍含有一些中度風化的物質。而高嶺石岩脈較少，多在近該層的頂部出現(照片 13)。

該蝕變凝灰岩層在事發地點以東及以西的鑽孔發現，在接近滑坡崖的鑽孔亦找到。但在距離滑坡崖以南 16 米的鑽孔則未有發現。這顯示該凝灰岩層有可能向橫伸延至鄰近山泥傾瀉現場的土地，但蝕變程度和高嶺石岩脈的數量則因地點而異。

關於蝕變凝灰岩層的來源，香港地質測量組指出，含豐富黏土的凝灰岩層是由數個過程造成。該岩層最初是一個與原先凝灰岩層理及/或結構平行的剪切帶。岩石層早期的蝕變和斷層，大概是由於附近的九龍深成岩體(距事發地點西北面 70 米)入侵所致。岩層東面及西面不同程度的蝕變，大概是因原來岩性的差異，引致剪切和熱液效果亦相應有別。最近期，該岩層如岩體的其他部份一樣，受長期近表層風化所影響。

7.3 <u>土壤和岩石的性質</u>

本處在場地勘探時取得土壤和岩石試樣，進行一系列全面的岩土室內試驗，包括粒徑分佈試驗、阿太堡界限試驗、直接剪切試驗、三軸壓縮試驗和壓密試驗，試驗目的是要確定含豐富高嶺石的蝕變凝灰岩和風化火山岩節理的岩土性質，而崩塌底部滑面和滑坡崖，就是分別由上述的凝灰岩和火山岩節理構成。

本處按照 Chen(一九九四)，進行了粒徑分佈及阿太堡界限試驗。不含高嶺石岩脈的蝕變凝灰岩的細粒土(即黏土和粉砂)平均含量爲 71%，而含高嶺石岩脈的蝕變凝灰岩的細粒土平均含量則爲 92%。蝕變凝灰岩及高嶺石岩脈的塑性指數在 9 至 18 之間，液限則在 29 至 50 之間。這顯示物質屬低至中度塑性，與高嶺石的典型性質一致。

本處按照 Head(一九八二)進行直接剪切試驗和按 Head(一九八六)進行固結不排水三軸壓縮試驗評估蝕變凝灰岩的抗剪強度性質。並根據 Hencher & Richard(一九八九)所推薦的方法，進行直接剪切試驗，確定風化岩石節理的性質。剪切試驗所得的結果，以及利用最小平方法求得的最佳擬合線所確定有關物質的抗剪參數，分別載於圖 12 及圖 13。

在不含高嶺石岩脈的蝕變凝灰岩試樣進行直接剪切試驗，所得的抗剪角(ϕ')爲 34 度，黏聚力(c')爲 10kPa。含高嶺石岩脈的蝕變凝灰岩進行直接剪切試驗時，如剪切面平行於岩脈，其ϕ' 的平均數值爲 29 度，c' 的數值爲零。ϕ' 的下界值爲 22 度，這相應於剪切發生在含較高黏土量的土壤。

大部份試驗的樣本都是於山泥傾瀉發生後，從現場殘留的含豐富高嶺石的蝕變凝灰岩收集得來。因此，實際支配山泥傾瀉並於崩塌時滑走的含豐富高嶺石的蝕變凝灰岩

層，可能較所試驗的試樣更弱。基於有大量高嶺石岩脈存在，而其中很多在蝕變凝灰岩層內的走向不利於穩定，故該岩層的抗剪強度，可能主要是由那些高嶺石岩脈所支配。總體來說，含高嶺石岩脈的蝕變凝灰岩試樣，其平均抗剪強度參數(即φ' 爲 29 度和 c' 爲零)，可於以理論方法分析穩定性(第 9 節)時，視作具代表性的數值，適用於構成崩塌底部滑面的含豐富高嶺石的蝕變凝灰岩層。

根據直接剪切試驗的結果，風化火山岩節理的φ' 平均值爲 35 度，而 c' 值爲零。這些數值亦可視作具代表性的數值，適用於構成滑坡崖的持續、粗糙而平直的節理。

從蝕變凝灰岩層所取得的試樣，經壓密試驗和利用直接剪切試驗的固結期的資料，評估其以固結系數所表達的固結性質爲每年 28 平方米至每年 172 平方米不等。由於該岩層的厚度一般少於一米，故此當負載力和地下水情況改變，岩層亦能足夠迅速地作出排水反應，因此在山泥開始崩塌時，應無出現顯著過量的孔隙水壓。

除了上述的土壤和岩石試驗外，調查人員亦進行了土壤和水試樣的標準化學分析，以便找出土地內可能的水源。分析結果於第 8 節闡述。

7.4 <u>地下水情況</u>

調查人員翻查能找到的地下水記錄及滲水觀察資料，評估了事發地點的地下水情況。翻查資料包括以下各項：

(a) 事發前，於一九七六年三月至一九七八年六月期間在兩個鑽孔內(圖 6，編號 C8 及 C9)所得地下水監察資料，及於一九八二年二月至三月間在另兩個鑽孔(圖 6，編號 P1 及 P2)所得地下水監察資料；

(b) 事發前，於過往視察削坡時，觀察到坡腳與削面半腰的坡級之間的一條狹窄風化層，有水滲出(附錄 A 的 A.2.2、A.2.3 及 A.2.4 節)；

(c) 事發後，於一九九五年九月至十二月期間，在 11 個垂直鑽孔(圖 9，編號 DH1 至編號 DH10 及編號 DH4A)及 2 個觀測井(圖 9，編號 DH16 及編號 DH17)內，收集到的地下水監察資料；及

(d) 事發後，探溝內蝕變凝灰岩上的崩塌泥石，有水滲出，滲水的範圍和程度隨降雨量而增加，而未被泥石掩蓋的滑坡崖，並沒有明顯的滲水跡象。

根據上述資料，推想在山泥傾瀉發生時，該處有兩個地下水體系，即蝕變凝灰岩層下面的岩体內的區域地下水位，以及在蝕變凝灰岩層上面的風化火山岩內的上層滯水位。

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根據地下水監察結果所顯示，區域地下水位位於崩塌底層表面以下 4 米至 8 米處。在崩塌發生時，區域地下水位被認爲不可能高於崩塌的底部，因此對這次崩塌不可能有重要的影響。

在蝕變凝灰岩層及其上的岩土出現的上層滯水位體系，可能是造成這次山泥傾瀉的重要因素。崩塌前及其後觀察到的滲水情況，亦支持上層滯水位的構想，這個構想亦與事發地點的地質環境相符：該層蝕變凝灰岩，特別是和其下只能於節理導水的風化岩介面，構成了一低滲透度的界面。豪雨或自其他來源滲入泥土的水，於抵達該低滲透度界面時，水份下流就會受阻，形成上層滯水位。

由於在發生崩塌後外露的滑坡崖並無發現明顯的滲水跡象，這顯示上層滯水位不大可能上升至高於該處泥石約 4 至 5 米的厚度。所以最佳的估算是，在山泥傾瀉發生時，上層滯水位約在蝕變凝灰岩層上面 1 米至 4 米處。

事發前的地表滲透，亦可能引致雨水滲入於後來滑坡崖的位置及其附近的風化岩石節理，令岩石節理內的水壓短暫上升，減低斜坡的穩定性。

8. 柴灣海水配水庫及其相連輸水系統的情況

柴灣海水配水庫及其於崩塌區域附近的輸水系統佈局載於圖 14。配水庫由四幅素混凝土側牆和一塊底板所構成，中間建有一幅混凝土分隔牆，把配水庫分爲東西兩個部份。根據水務署的記錄，在發生山泥傾瀉前，有兩條地下輸水管穿越事發地點的西部。輸水管的直徑爲 24 吋，由多條約 4 米長的石棉水泥管組成，並用石棉水泥套筒和橡膠封環所造成的接合裝置把水管接合起來。在山泥傾瀉前，這條輸水管把配水庫的海水供應給柴灣區。至於直徑 6 吋的輸水管，則自一九八六年已經棄用，並於近削坡腳處(圖 14)蓋封。

海水配水庫於一九九五年八月十三日下午經由水務署及土力工程處檢查，發現配水庫側牆所有外露的接縫均沒有明顯的移動。

一九九五年八月二十一日，土力工程處接到水務署於一九九五年八月十六日就山泥傾瀉撰寫的報告(水務署，一九九五)。報告指出「受影響的 24 吋(600 毫米)及 6 吋(150 毫米)直徑鹹水管，以前並無爆破/滲漏的記錄。根據設計，估計在極接近配水庫出水口的數段喉管內的水壓，在 10 米以下。」

水務署在事發前的水庫水位記錄及所用以抽水到配水庫的抽水機數目載於圖 15。從記錄中看不到主要崩塌前有運作不正常或有整體滲漏的跡象。因爲這些跡象可從水位急劇下降而顯示出來。水務署(一九九五)指出「按過往的視察記錄及柴灣海水配庫、柴灣海水抽水站、及供應與分佈網絡的最新運作記錄，可以斷定配水庫、抽水站及相連的輸水管均在正常運作狀況，而且至事發時仍妥爲運作。」

水務署於一九九五年八月十七日在配水庫的西半部水庫進行滲漏測試，先把西半部水庫注水至 1.3 米深，然後於其後 4 小時監測水位。據水務署指出，監測水位工作的準確度達 0.1 毫米，相當於該受測試西半部水庫的水量約 0.05 立方米。水務署在這次滲

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漏測試中並未察覺有可量度的失水情況。東半部水庫並未進行滲漏測試。基於安全理由，該部份於事發後一直空置。

在發生山泥傾瀉時，直徑 24 吋的輸水管斷落了一段，長 21 米。所有斷落了的輸水管，均從崩塌泥石中檢回。水務署和土力工程處於一九九五年九月七日共同檢查該些輸水管。除了有些新的破損外，輸水管大致上完整無缺。

據水務署的記錄，事發前被截斷的一段已棄用的直徑 6 吋輸水管，於斜坡崩塌前是連接配水庫的。水務署從泥石中只尋回 2.4 米長的輸水管。而土力工程處從山泥傾瀉泥石的照片中，亦只能辨認到少於 20 米長的輸水管。這遠短於水務署的輸水管記錄(圖 14)所示的約 55 米長度。因此，輸水管線有未能確定之處，亦不能從尋回的輸水管，準確地証實水管的情況。

為評估海水可能滲入泥土的程度，本處曾對 37 個土壤試樣和 44 個水試樣進行化學試驗，以確定氯化物的含量。試驗是按照 American Public Health Association (一九九二)和 BSI(一九九零)內的步驟進行。

試驗結果載於圖 16。從配水庫抽取出來的海水，其氯化物含量為每公升 11000 至 19000 毫克。山泥傾瀉東南面山谷的溪澗，受水庫滲水的影響應該甚為微小。在該處抽取的水試樣，進行化驗所得的氯化物含量為每公升 42 毫克。這個數值可作為基準，以評估海水侵入山泥傾瀉附近地下水的程度。

從配水庫以北約 20 米的一個鑽孔內抽取兩個水試樣，其深度位於距離地面約 8 米及 11 米處，試驗結果發現每公升含 4500 毫克的高氯化物含量。這顯示該配水庫可能出現海水滲水。從另一邊翡翠道的兩個鑽孔及從山泥傾瀉地點附近的滲水地點所採取的水試樣，其氯化物含量為每公升 120 毫克至每公升 1300 毫克，這象徵海水存在於地下水內，約佔百分之十的體積。這顯示配水庫可能滲水，流入現場附近的土地內。海水滲水可能有助於使蝕變凝灰岩濕水，但較豪雨期間雨水滲入而可引致的上層滯水位，它應該是較不重要的水源。

土壤的氯化物含量試驗結果載於圖 17。從崩塌發生後在外露的滑坡崖西部坡腳所取得的土壤試樣，發現含 0.07%至 0.17%不等的高氯化物含量。在發生崩塌後，從配水庫經由截斷的直徑 24 吋輸水管所流出的海水，當時是沿該部份滑坡崖流下。但從滑坡崖和崩塌區內其他地點的泥石所收集的土壤試樣，其氯化物含量則頗低，平均含量為 0.03%。因此，並沒有徵象顯示配水庫的滲水，是山泥傾瀉附近地下水的一個主要來源。這與水試樣的氯化物含量試驗所得的結果一致。

9. 理論穩定性分析

本處曾以理論方法進行穩定性分析，以協助判斷這次山泥崩塌的機制和成因。這些分析旨在確定蝕變凝灰岩層在不同上層滯水位的情況下，該岩層於發生崩塌時抗剪強度的可能數值。

這些分析採用的資料來自發生崩塌後所進行的場地勘探工作、室內試驗、現場觀

察和實地量度。具代表性的崩塌地點剖面，以及這些分析所採用的輸入參數，載於圖 18。這些分析假設在發生崩塌時，上層滯水位上升至蝕變凝灰岩層上面 4 米。

分析所得的結果摘要載於圖 19。以安全系數爲 1.0 而言，當上層滯水位爲 1 米、2 米和 4 米的時候，蝕變凝灰岩層的抗剪角(即ϕ')則分別爲 26.5 度、28 度和 31.5 度。上述的ϕ' 數值，與室內試驗所確定的數值一致(圖 13)。隨上層滯水位的形成，斜坡在理論上會變得不穩固，因而引致沿蝕變凝灰層發生剪切，出現平移崩塌。

本處亦進行了敏感度分析，探究假如雨水滲入於滑坡崖位置的節理，其水壓如何影響削坡。於分析時，蝕變凝灰岩和風化火山岩節理的抗剪強度參數，採用了最佳的估算數值。根據分析發現，如無出現上層滯水位，岩石節理內須出現 9 米至 10 米的靜水壓頭，才會引致崩塌。因此，該組岩石節理需注有深闊的積水，才能嚴重影響斜坡穩定，引發山泥傾瀉。

10. 山泥傾瀉成因的診斷

根據這次調查所得的資料，推想翡翠道山泥傾瀉是由下述兩個主要因素造成：

(i) 斜坡本身含有廣闊的薄弱物質；及

(ii) 在持續豪雨後地下水壓增加。

崩塌殘痕底部的滑面是一層持續且含豐富高嶺石的蝕變凝灰岩層，而殘痕兩側的釋放面和滑坡崖，則是風化凝灰岩內的陡峭和走向不利的平直節理。在發生山泥傾瀉後進行的室內試驗和地質勘察，證實該組平直節理和蝕變凝灰岩層，特別是後者，遠較斜坡的大部份構成物質爲弱。這些因素令岩體沿著輕微傾斜的蝕變凝灰岩層剪切滑動，引致深層平移的崩塌機制。

山泥傾瀉發生於一段異常持續的豪雨之後，這場豪雨的雨量，是崩塌地點附近編號 H14 雨量計自一九七九年安裝以來，所錄得的最高雨量記錄。豪雨可能導致蝕變凝灰岩層上面形成上層滯水位。這個設想和現場觀察所見相符，而且與事發地點的水文地質環境一致。上層滯水位上升會增加蝕變凝灰岩層的水壓，減弱物質抗剪強度，最終促致山泥傾瀉。

本處亦曾研究由於滑坡崖和其附近的岩石節理水壓上升而引發山泥傾瀉的可能性。根據分析顯示，在沒有上層滯水位的情況下，位於滑坡崖的岩石節理在發生山泥傾瀉之前，需於很深和廣闊的面積塡滿水，才可有足夠水壓推動岩體。這設想與上層滯水位形成的設想比較，其發生的機會較低。但不能排除岩石節理內存在一些水壓，結合上層滯水位，而促致山泥傾瀉。

這宗山泥傾瀉事件涉及兩次不連續的崩塌。雖然估計這宗山泥傾瀉的主要成因是基於上述的兩個因素，但下文仍討論了在某些程度上能促致山泥傾瀉的其他可設想因素。

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第一次的崩塌於一九九五年八月十三日約零時五十五分發生。崩塌範圍只局限於削坡東面的上部，所塌下的泥石體積約爲數十立方米。斜坡缺乏足夠維修，使斜坡情況衰退，令地面排水渠淤塞而引致雨水集中滲入泥土，可能有助於引發第一次局部崩塌。

於凌晨一時十五分左右發生的第二次山泥傾瀉，是主要的崩塌。第一次崩塌，令削坡失去部份支承力和旁邊阻力，可能有助於引發這次主要的崩塌。雖然如此，當斜坡發生局部崩塌時，整個山泥傾瀉區域也有可能已達至全面崩塌邊緣。

在主要崩塌發生前，並無資料顯示，配水庫出現不正常運作，亦無證據顯示配水庫和其相連的輸水系統出現嚴重滲漏。

導致山泥傾瀉的有關事件經重組後的發生次序，相信如圖 20 所示。

翡翠道的山泥傾瀉，其崩塌規模之大和泥石移動距離之遠，實屬罕見。這主要與該處有一範圍廣闊並支配這次山泥傾瀉的蝕變凝灰岩層有關，這個情況在本港的其他山泥傾瀉事件中，並不常見。持續而弱的蝕變凝灰岩層，位於削坡頂部下面約 15 米處，使滑動體可以既深且大。在山泥崩塌發生前，極大而持續的雨，引致地下深處大面積的地下水壓上升，因而觸發這次大規模的山泥崩塌。

泥石堆末端與崩塌頂部的連接線，其傾斜度反映出泥石的流動性，以及支配泥石移動的摩擦角。就本港常見的斜坡崩塌而言，這個角度一般在 30 度至 40 度之間。這與本港典型土壤和岩石的φ'數值幅度相符(Wong & Ho，一九九六)。在翡翠道山泥傾瀉事件中，這角度的數值只爲 24 度，這與室內試驗所確定的蝕變凝灰岩強度相若。其他可能導致泥石有較高流動性的因素，包括崩塌規模龐大及輕微傾斜的滑動面。後者令泥石以較平緩角度衝擊地面，損失較少的能量。

11. 結論

所得的結論：一九九五年翡翠道山泥傾瀉，主要是由於事發前持續的極大豪雨，使斜坡內廣闊而低強度的、含豐富高嶺石的蝕變凝灰岩層內水壓上升所引致。

12. 參考書目

土力工程處 (一九八六) Hong Kong and Kowloon： solid and superficial geology. Hong Kong Geological Survey. Map Series HGM 20, 第 11 張，比例 1：20 000。香港土力工程處。

土力工程處 (一九九五) 一九九五年八月十三日柴灣翡翠道山泥傾瀉事件調查進展報告。香港土力工程處，16 頁。

水務署 (一九九五) Report on the Landslide at Fei Tsui Road, Chai Wan on 13.8.95. 香港水務署，7 頁加 3 幅圖，6 份附錄及 11 幅照片。

賓尼 (一九七七) Government of Hong Kong Landslide Study Phase IIC - Report on the Stability of Slopes in the Chai Wan Area. 賓尼組合(香港)顧問工程師爲香港政府撰寫，64 頁加 12 幅照片，6 份附錄及 23 幅圖。

American Public Health Association (一九九二) Standard Methods for the Examination of Water and Wastewater. (18th edition). Section 4 : Inorganic non-metallic constituents. American Public Health Association, 135 頁。

BSI (一九九零) Methods of Test for Soils for Civil Engineering Purposes. (BS1377：1990). Part 3：Chemical and Electro-chemical Tests. British Standards Institution, London, 44 頁。

Chen, P.Y.M. (一九九四) Methods of Tests for Soils in Hong Kong for Civil Engineering Purposes (Phase 1 Tests). 香港土力工程處，91 頁。(土力工程處報告第 36 號)。

Head, K.H. (一九八二) Manual of Soil Laboratory Testing. Volume 2： Permeability, Shear Strength and Compressibility Tests. Pentech Press, New York, 747 頁。

Head, K.H.(一九八六) Manual of Soil Laboratory Testing. Volume 3： Effective Stress Tests. Pentech Press, 1238 頁。

Hencher, S.R. & Richards, L.R. (一九八九) Laboratory direct shear testing of rock discontinuities. Ground Engineering, vol.22, no.2, 第 24-31 頁。

Strange, P.J. & Shaw, R. (一九八六) Geology of Hong Kong Island and Kowloon. 香港土力工程處，134 頁。(香港地質測繪報告第二號)。

Wong, H.N. & Ho K.K.S.(一九九六) Travel distance of landslide debris. Proceedings of the Seventh International Symposium on Landslides, Trondheim, Norway. 印刷中。

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附圖

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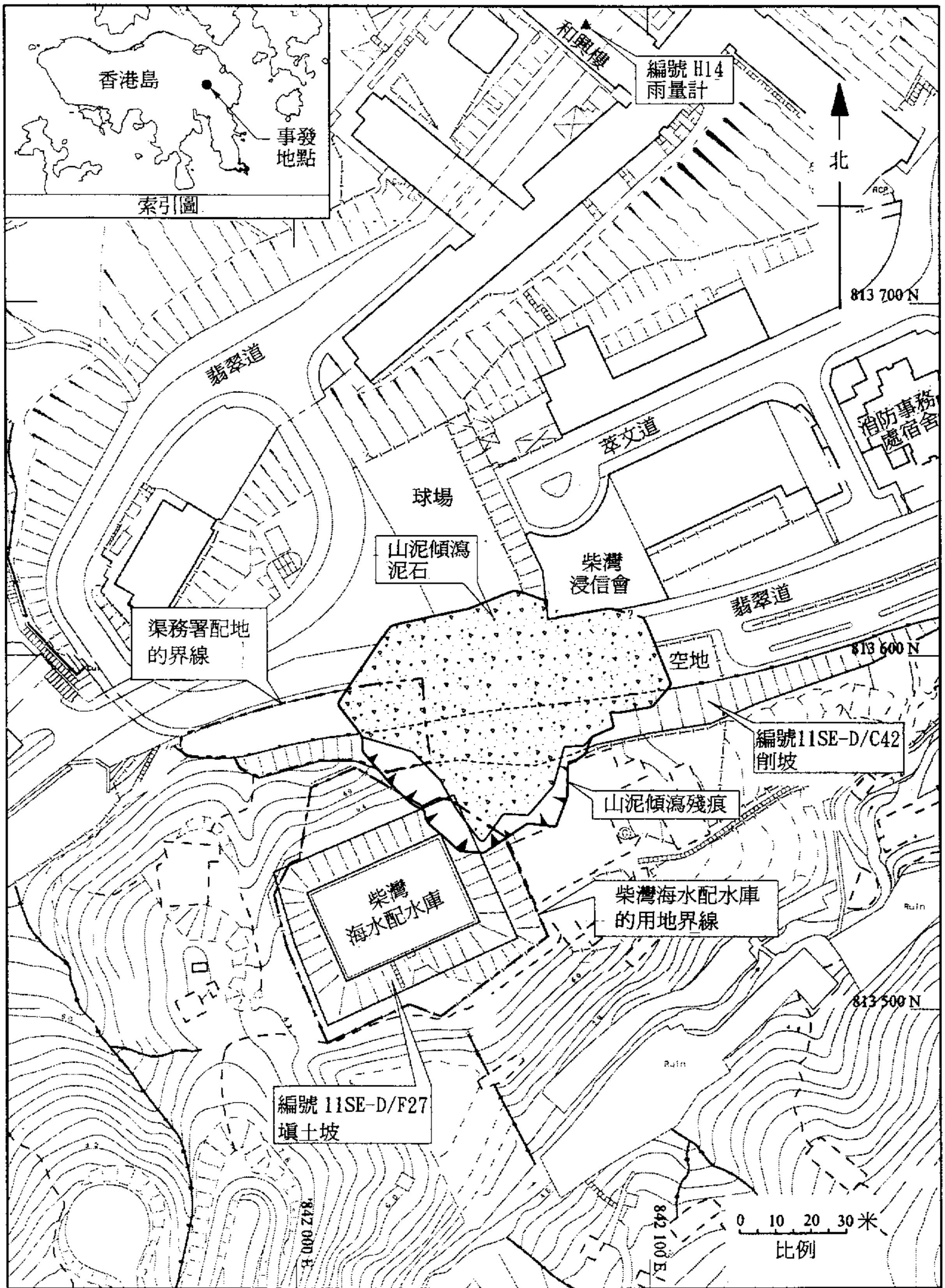

註： 1. 取自一九九二年九月二十九日編號 11-SE-18D 及編號 11-SE-19C 測量圖(比例 1:1000)
2. 柴灣海水配水庫的用地界線是根據配水庫的界線圍欄位置

圖 1 – 山泥傾瀉位置圖

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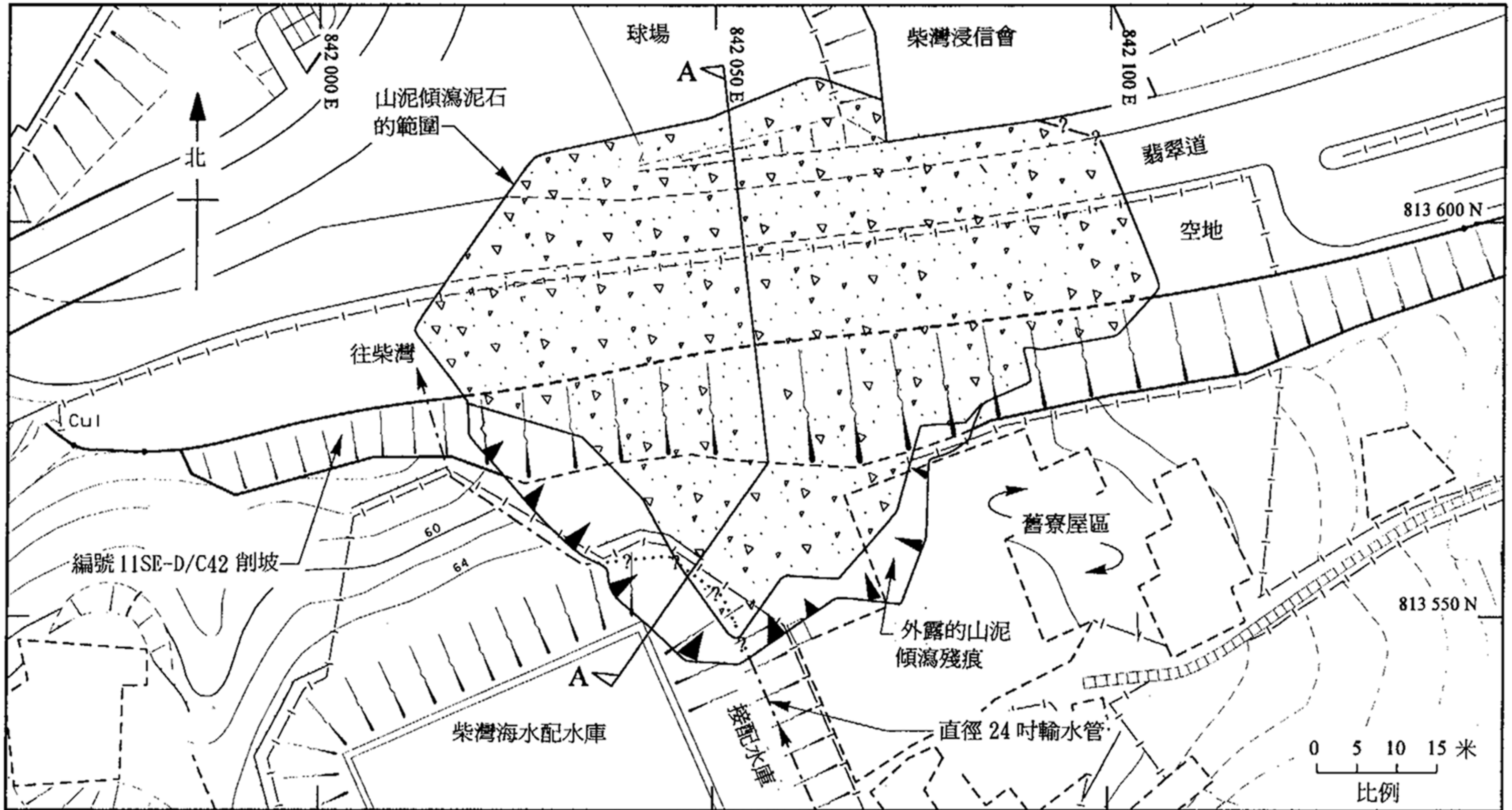

註：　(1) 剖面 A-A 見圖 4 。
(2) 圖中所示資料以地形測量、地質勘察、現場觀察及文件紀錄爲根據。
(3) 其他輸水管的分佈載於圖 14 。爲求清晰，該些資料並未繪在本圖上。

圖 2 － 山泥傾瀉平面圖

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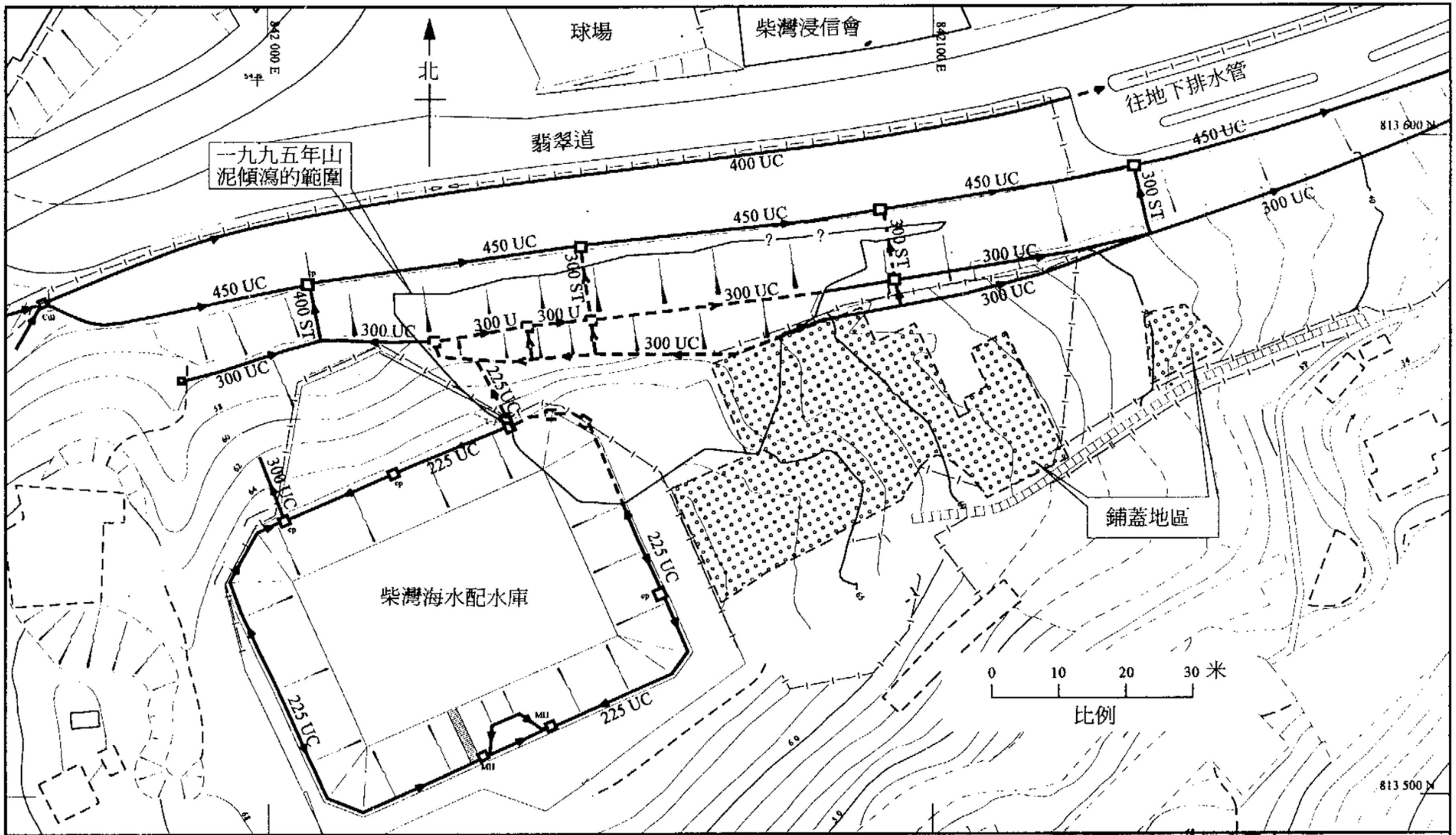

圖例:

———→	山泥傾瀉後在現場觀察到的地面排水渠線	300 UC	300 毫米 U 形渠
- - →- - -	按文件紀錄及實地觀察推斷的地面排水渠線	300 ST	300 毫米梯級渠

圖 3 － 事發地點排水系統的大概分佈

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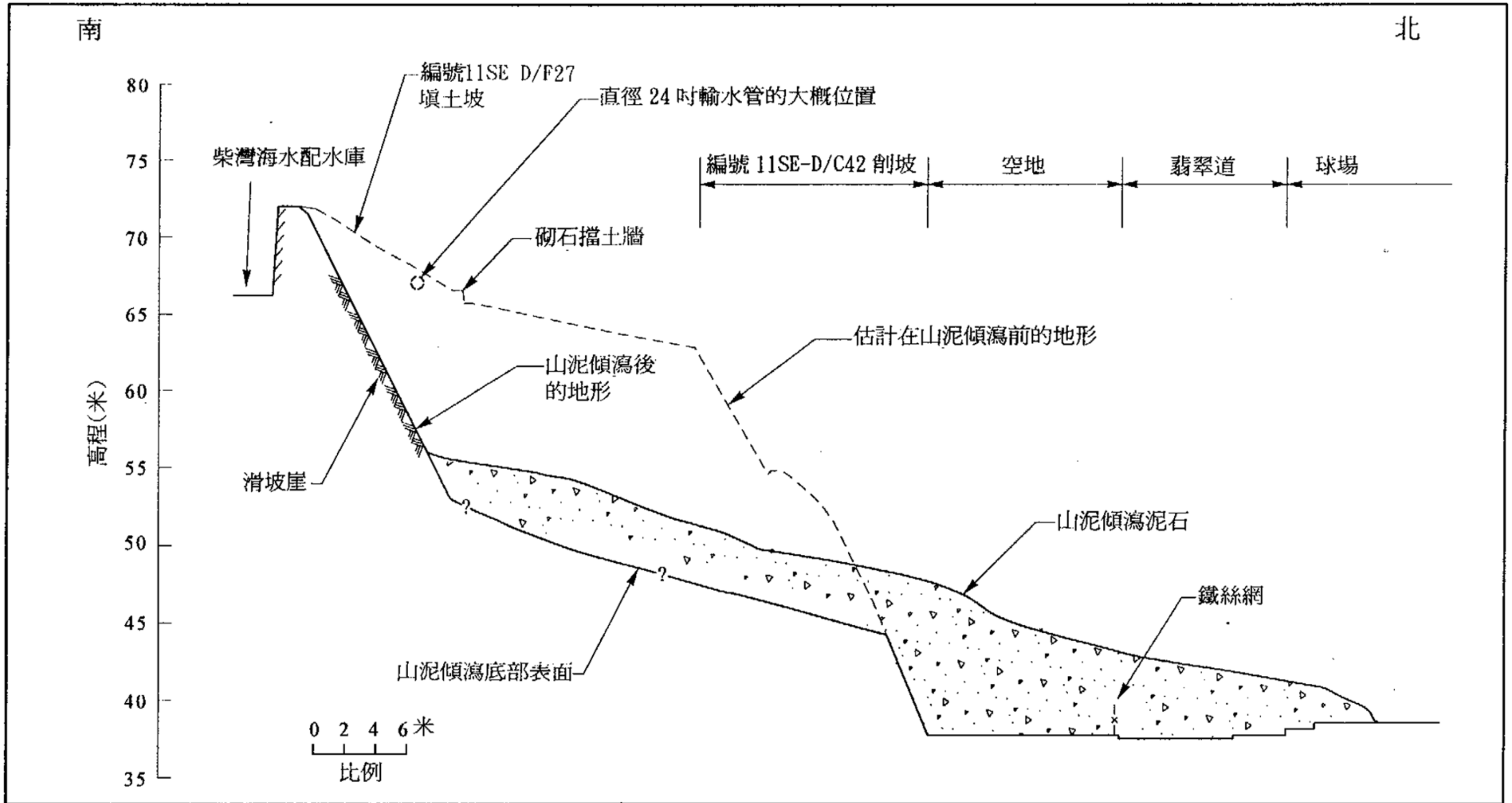

註： (1) 剖面的位置見圖 2 。
(2) 本圖所示資料以地形測量、地質勘察、現場觀察及文件紀錄爲根據。
(3) 棄用的直徑 6 吋水管未能確定位置，故並未繪在本圖上。

圖 4 – 剖面 A-A

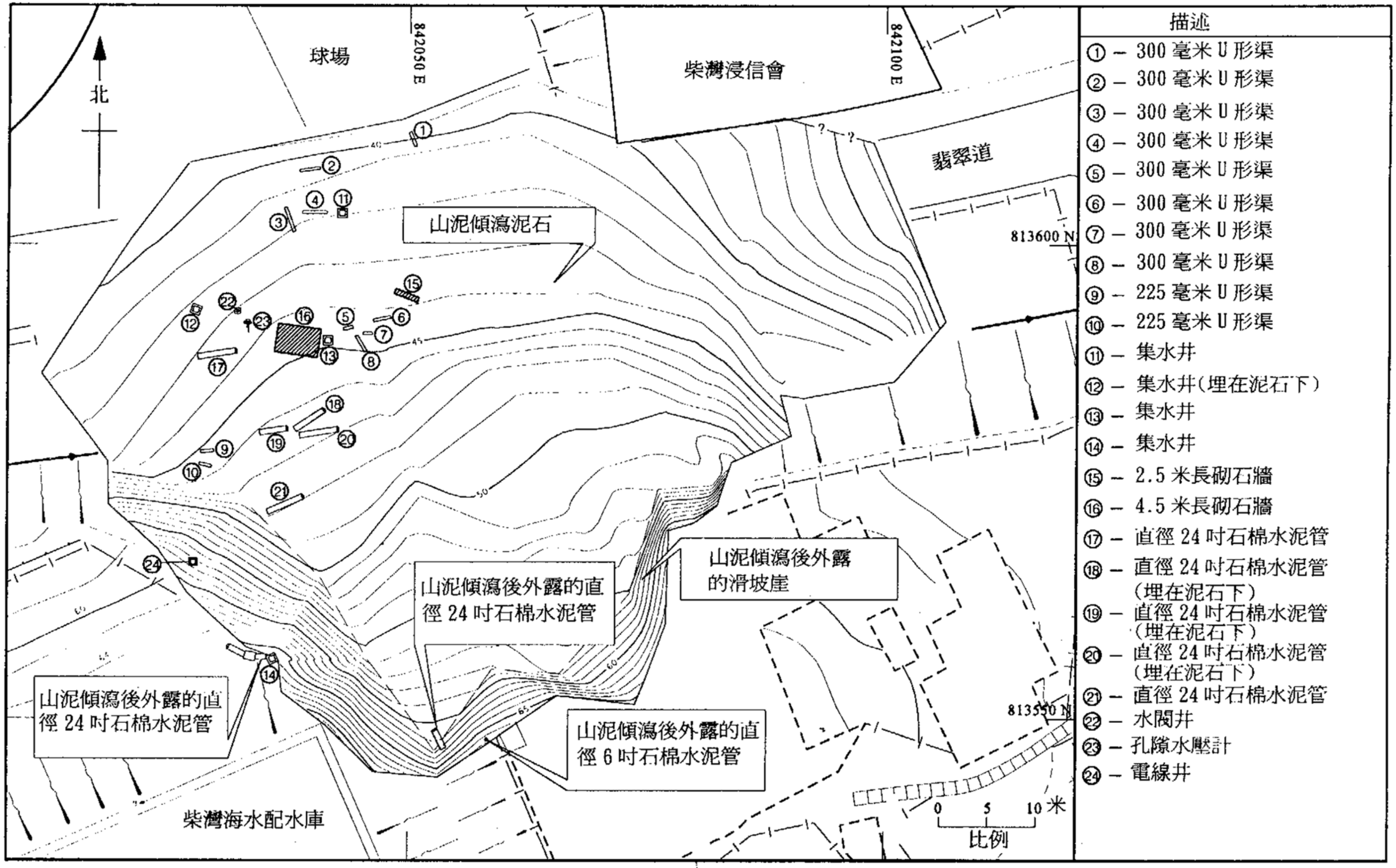

圖 5 – 山泥傾瀉泥石中人造物料的位置

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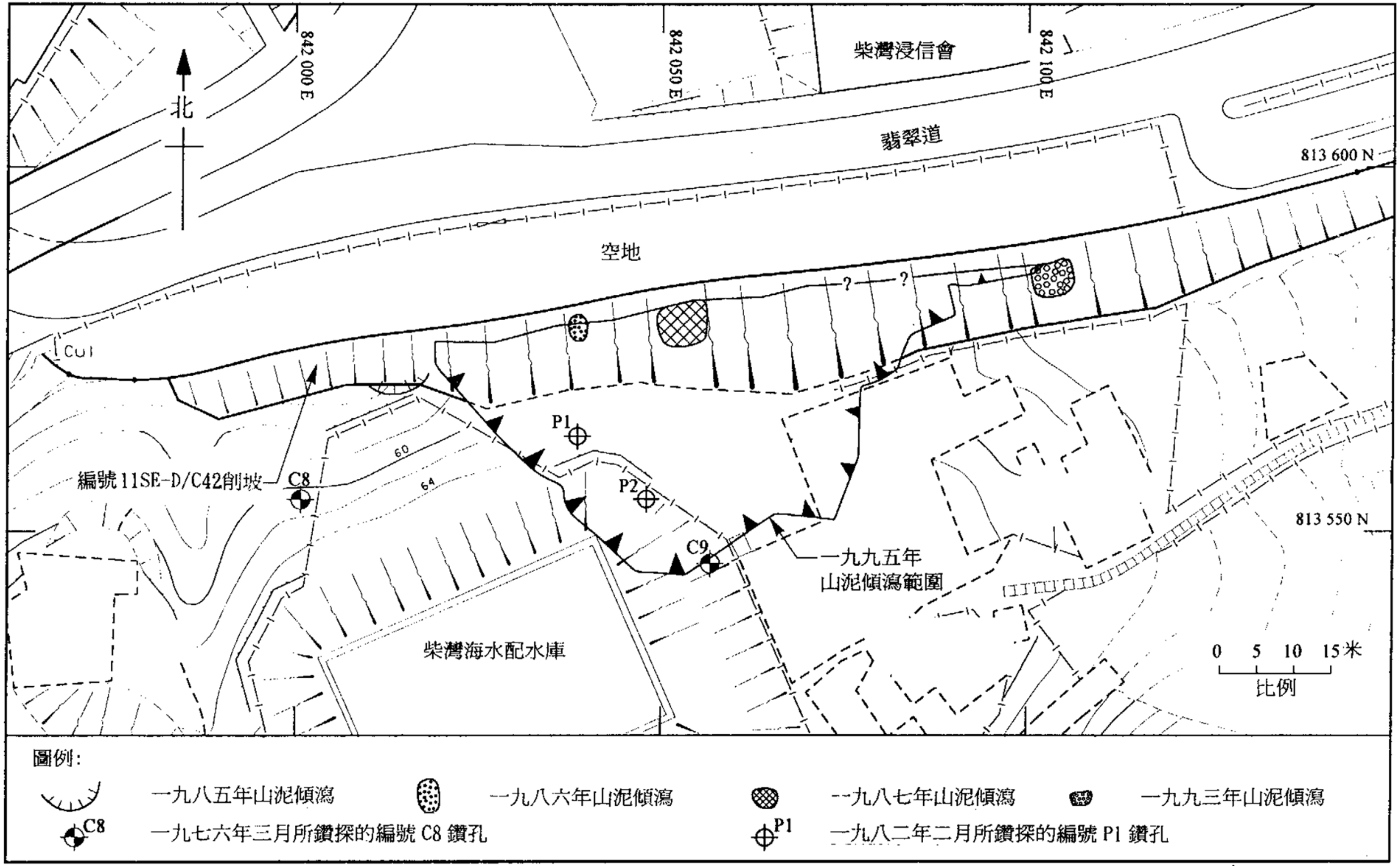
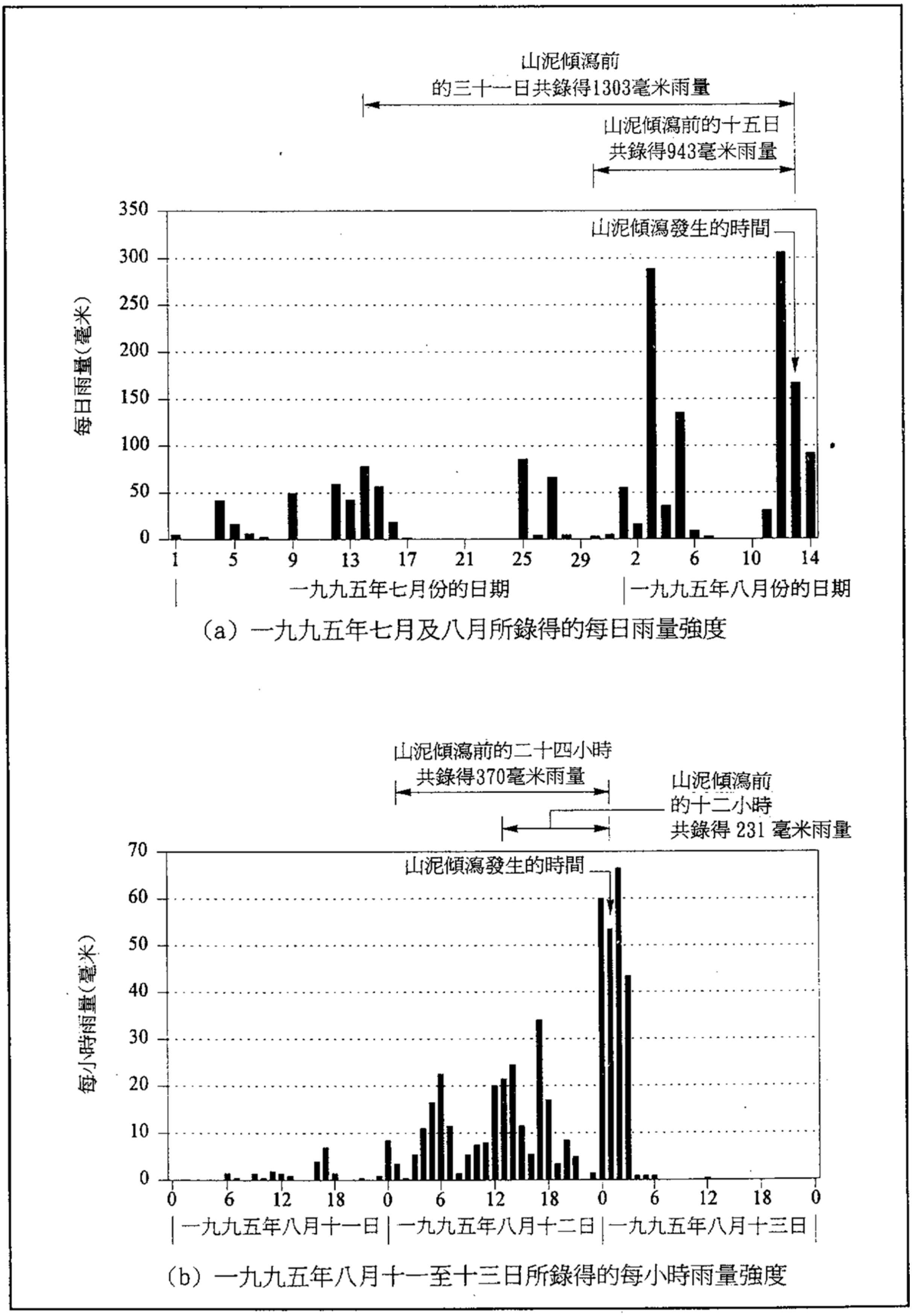


圖 6 - 編號11SE-D/C42削坡以往幾次山泥傾瀉的位置



(a) 一九九五年七月及八月所錄得的每日雨量強度

(b) 一九九五年八月十一至十三日所錄得的每小時雨量強度

圖 7 - 土力工程處編號 H14 雨量計的雨量記錄

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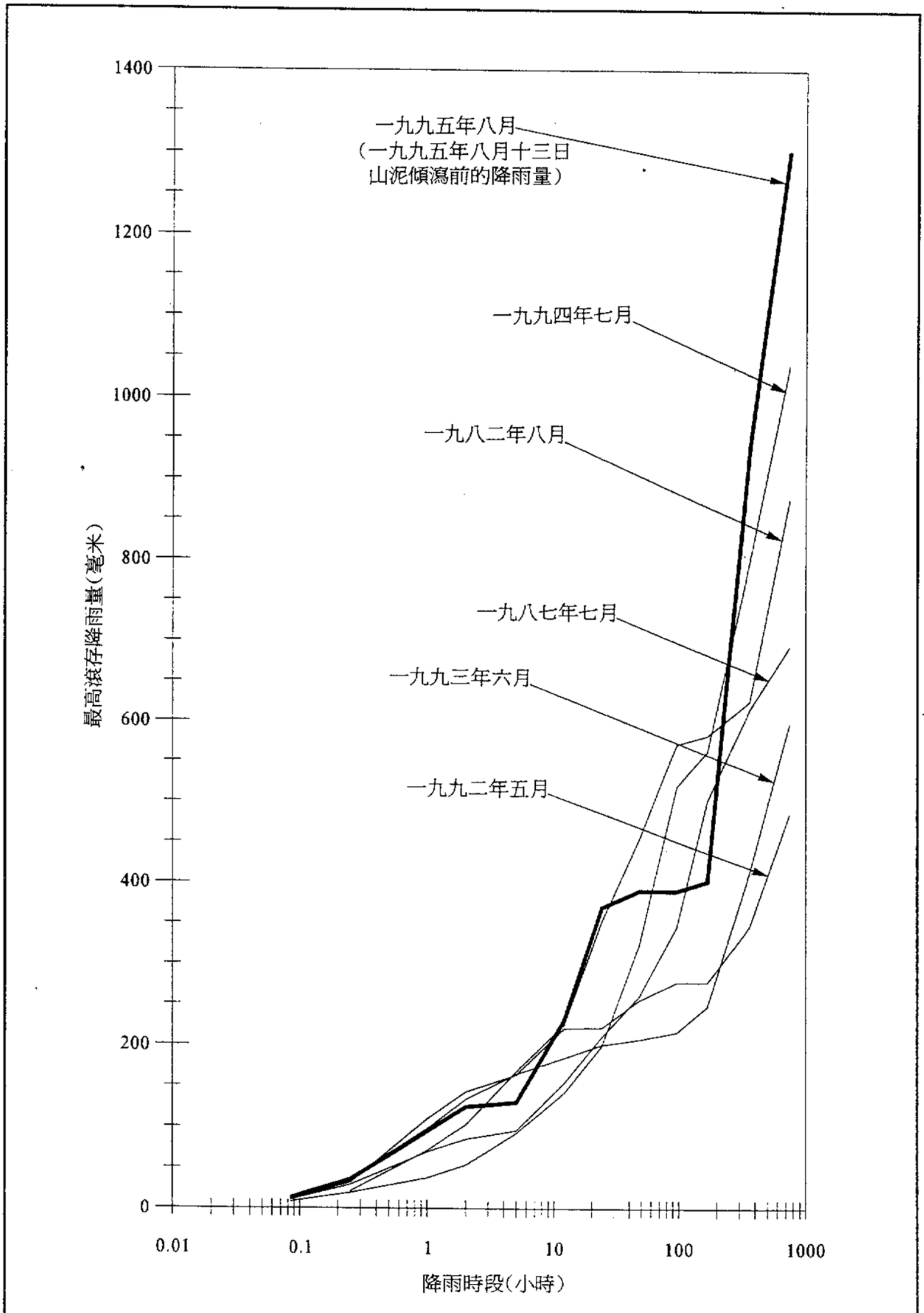

圖 8 – 土力工程處編號H14雨量計於歷次豪雨中錄得的最高滾存降雨量

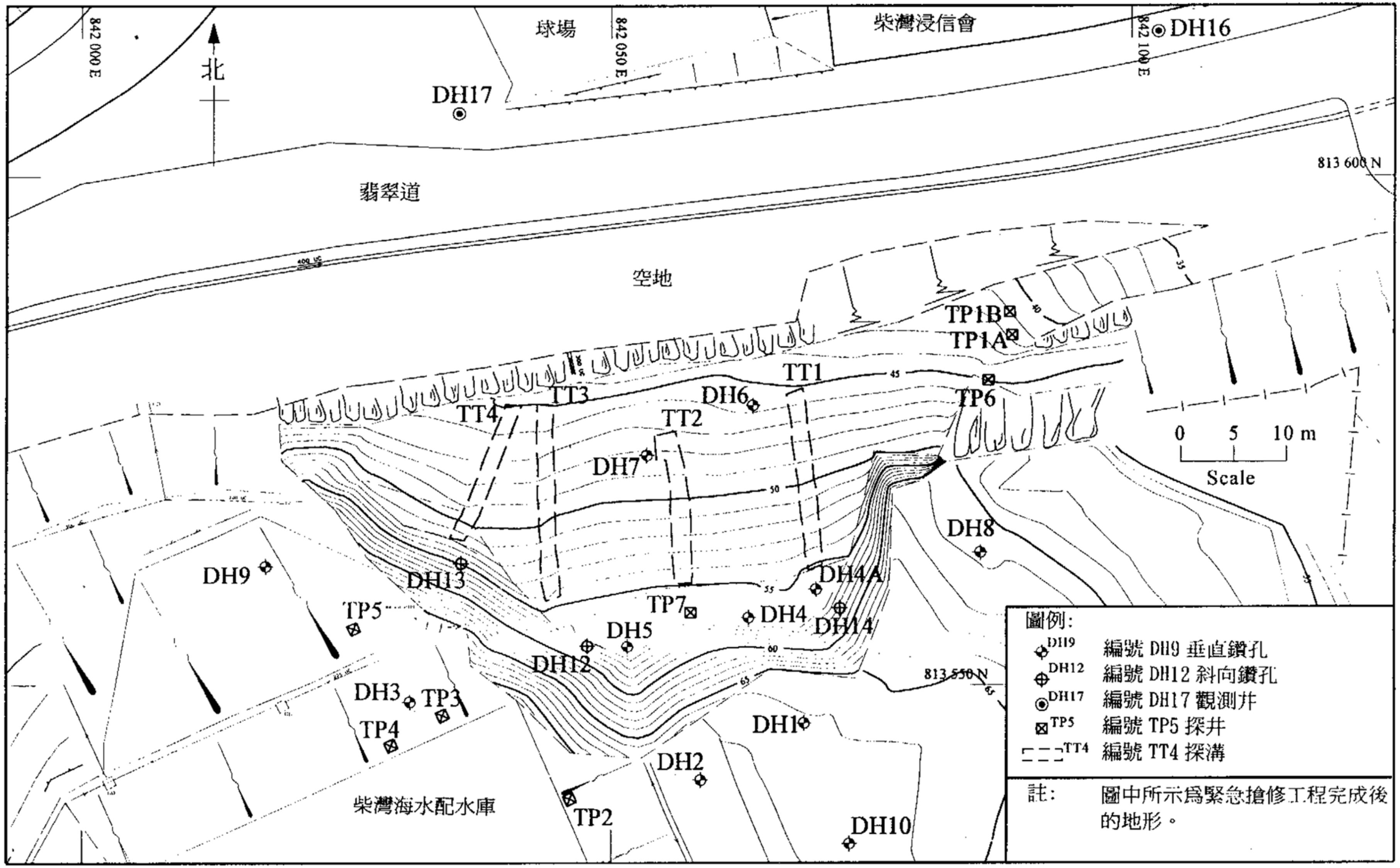

圖 9 － 場地勘探工程的位置圖

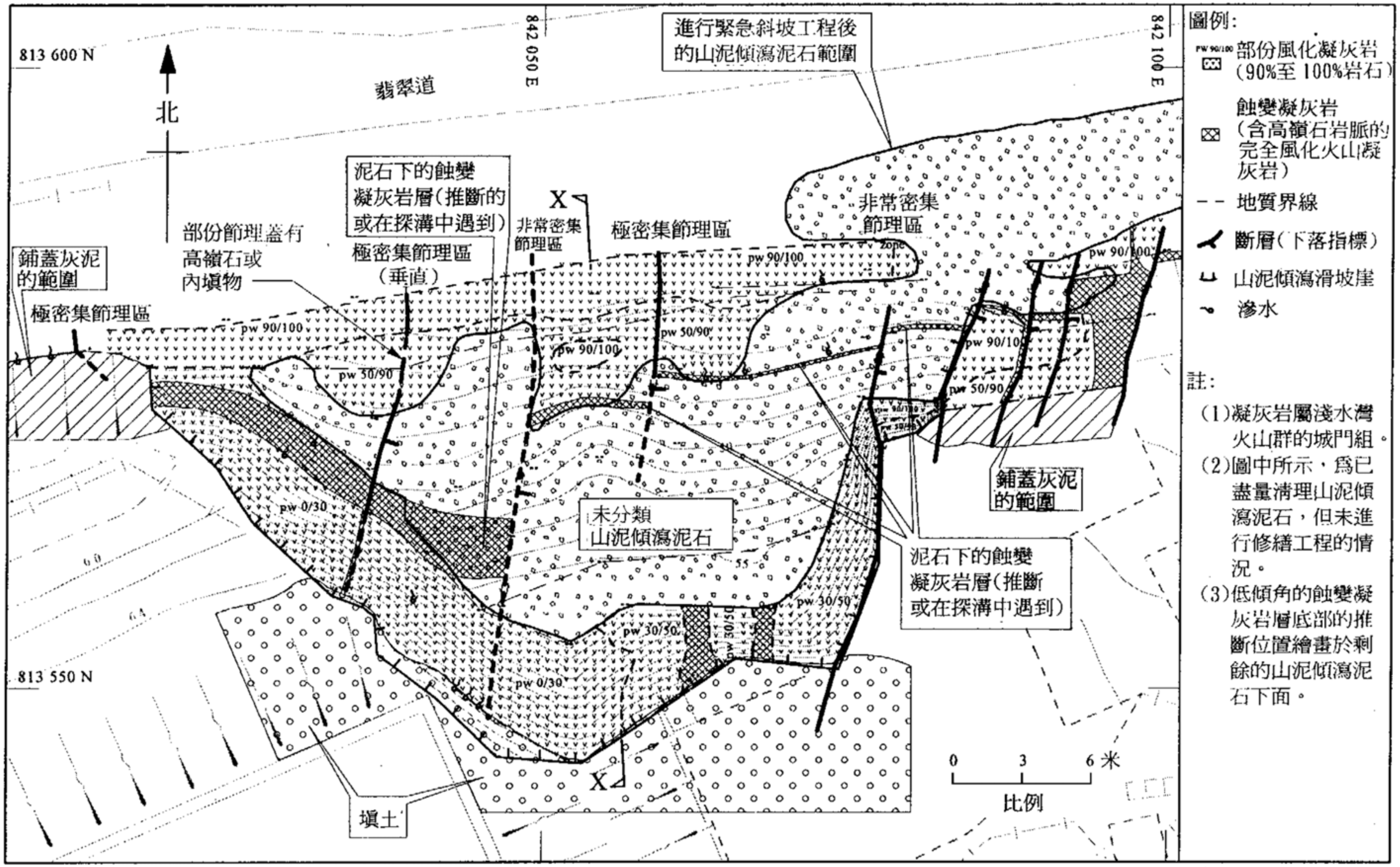

圖 10 – 山泥傾瀉的地質圖

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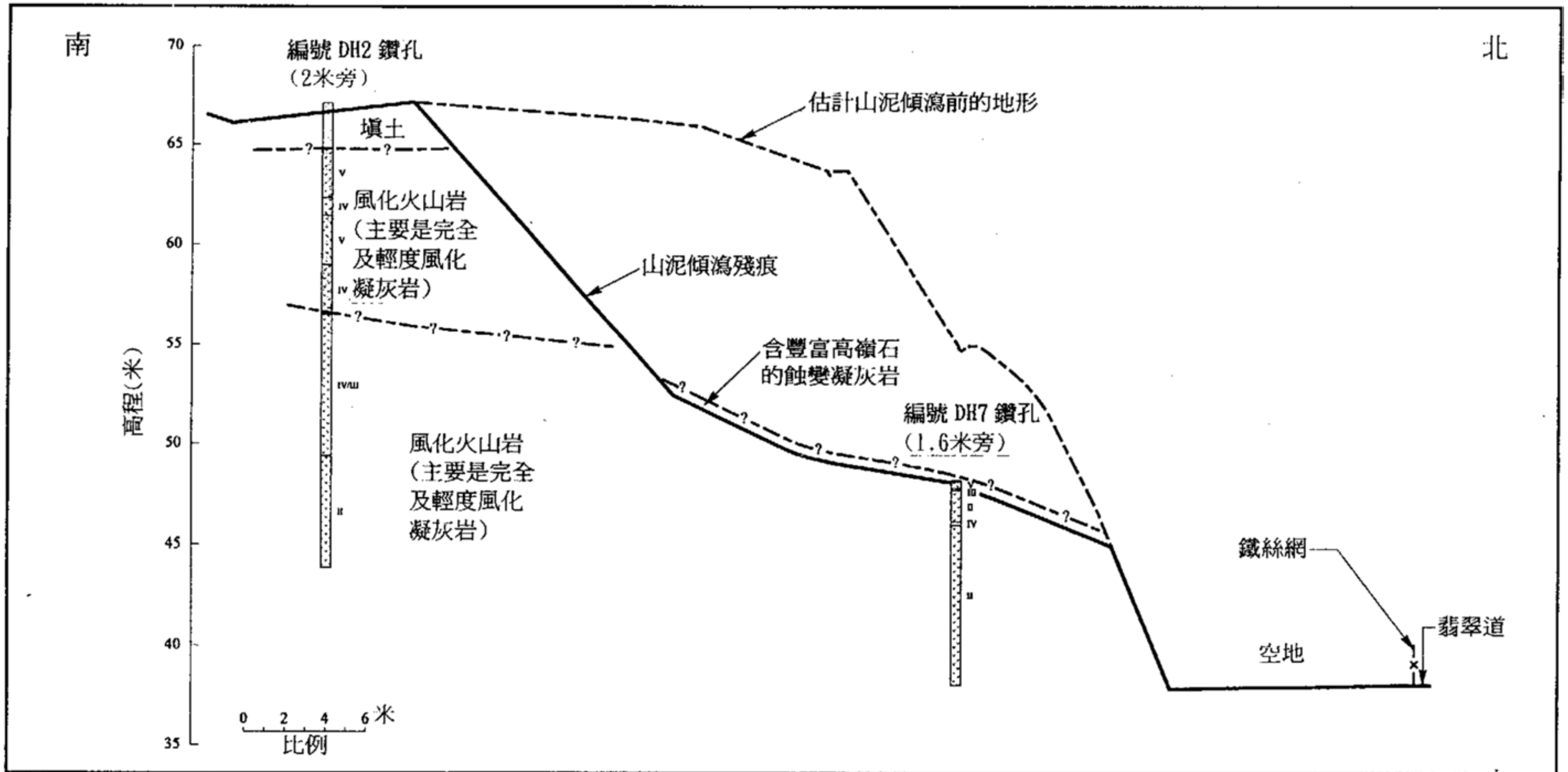

圖例：

V 完全風化凝灰岩	IV 高度風化凝灰岩	III 中度風化凝灰岩
II 輕度風化凝灰岩	填土	

註：（1）剖面位置見圖 10。
（2）圖中所示的資料以地形測量、場地勘探、地質勘察及文件紀錄爲根據。

圖 11 － 山泥傾瀉地點的典型地層剖面 X-X

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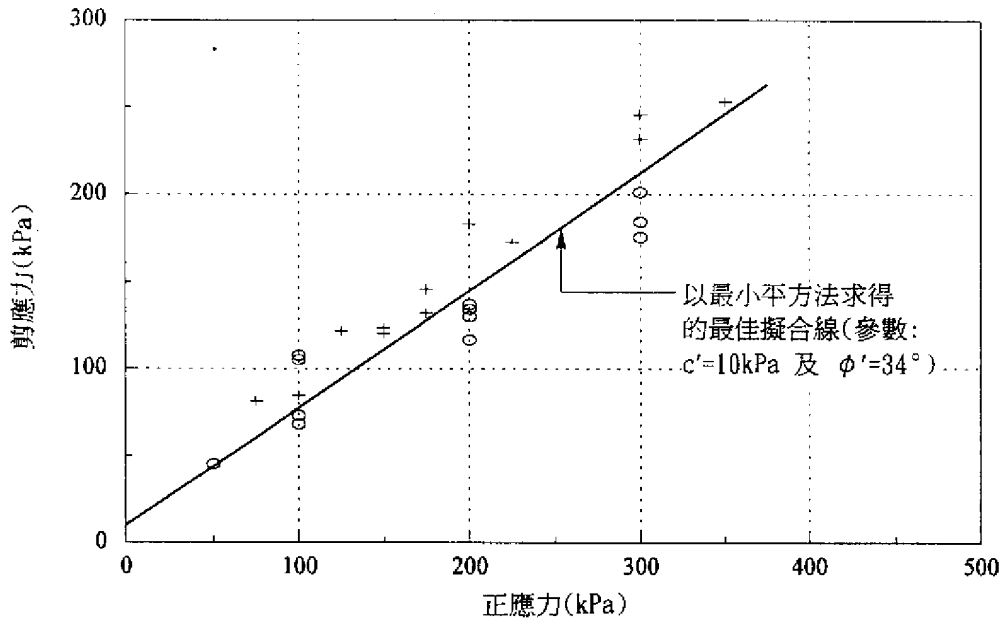
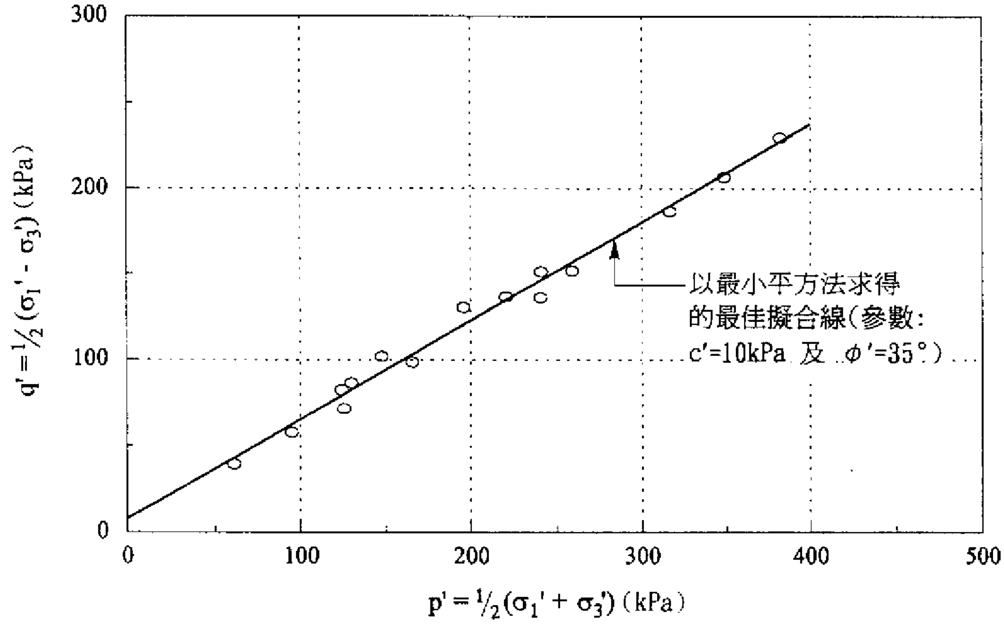


(a) 直接剪切試驗的結果



(b) 包括量度孔隙水壓的固結不排水三軸壓縮試驗結果

圖例:

+	單階段試驗	σ_1'	最大有效主應力	c'	黏聚力
○	多階段試驗	σ_3'	最小有效主應力	φ'	抗剪角

註: 本圖所示的直接抗剪試驗結果是相應於高峰抗剪應力的結果，三軸壓縮試驗的結果則相應於最大應力比率的結果，即最大 σ_1'/σ_3'比率。

圖 12 - 不含高嶺石岩脈的蝕變凝灰岩直接剪切及三軸壓縮試驗所得結果

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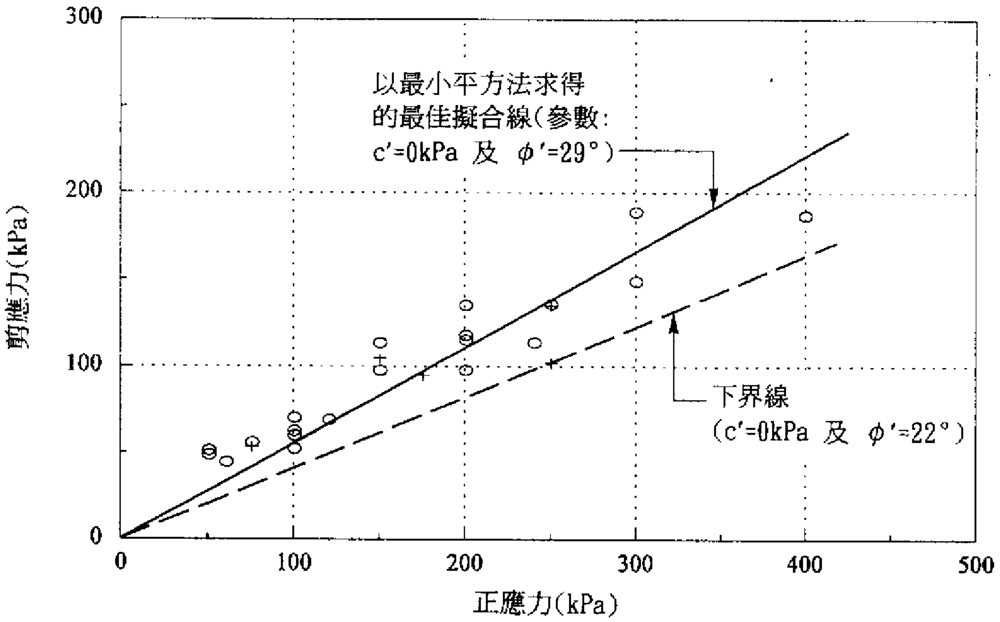
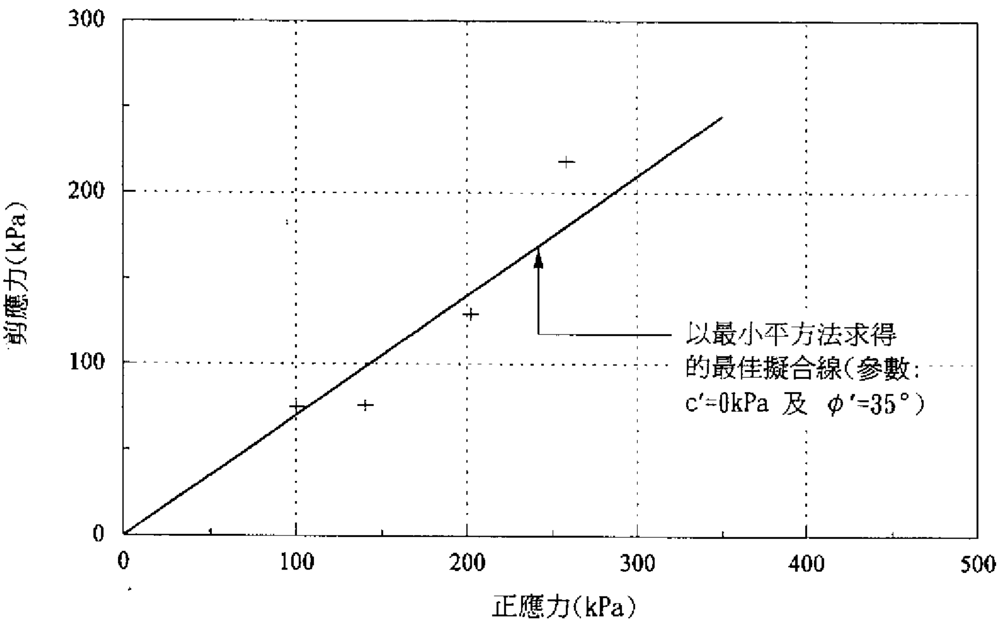


(a) 於含高嶺石岩脈的蝕變凝灰岩進行試驗的結果



(b) 於風化火山岩節理進行試驗的結果

圖例:

+	單階段試驗	c'	黏聚力
○	多階段試驗	φ'	抗剪角

註:
(1) 穿過數據點的最佳擬合線，是假設凝聚力爲零，並以最小平方法求得。
(2) 圖(a)所採用的試樣，按高嶺石岩脈平行剪切方向而固定。
(3) 本圖所示的直接剪切試驗結果，相應於高峰抗剪應力的結果。

圖 13 - 含高嶺石岩脈的蝕變凝灰岩及風化火山岩節理直接剪切試驗結果

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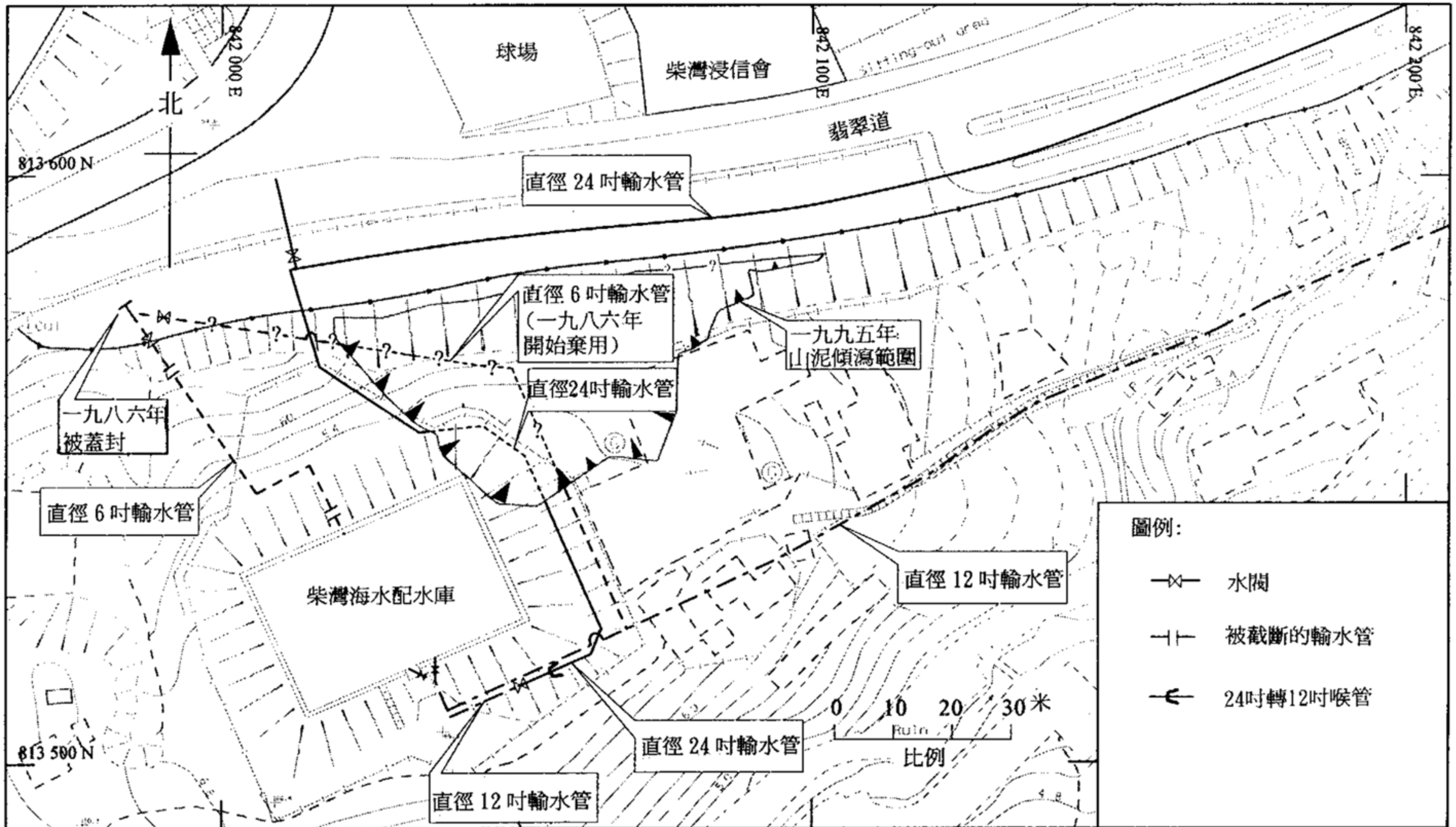

註： 輸水管線根據水務署資料。爲山泥傾瀉截斷的一段直徑24吋輸水管線，曾據現場觀察所得，稍作修正。

圖 14 － 山泥傾瀉前輸水管的分佈

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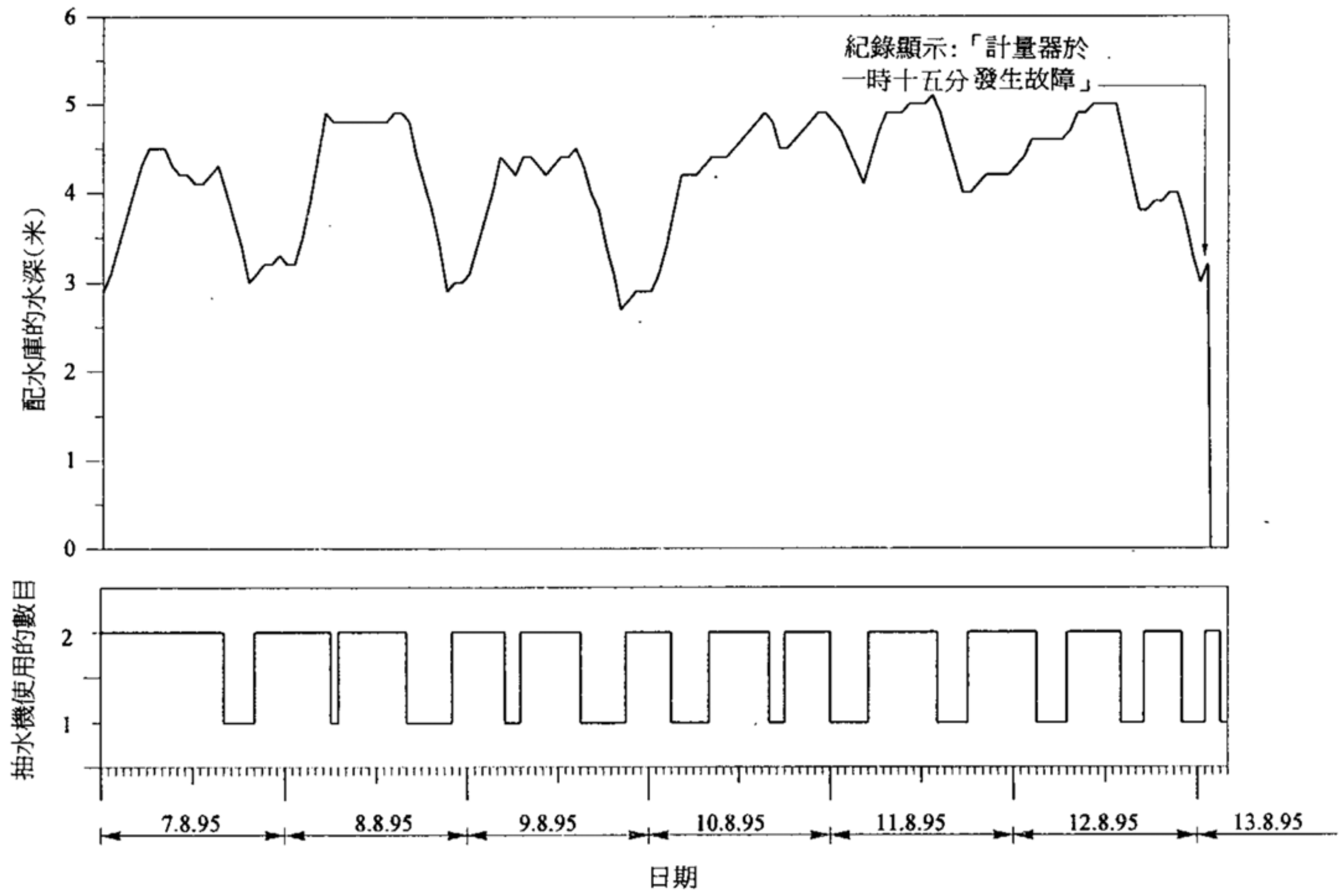

註: (1) 抽水機設置在柴灣抽水站。所示水深爲配水庫東西兩部份的水深。
(2) 本圖根據水務署(一九九五)資料。

圖 15 - 配水庫水位紀錄及抽水機的使用數目

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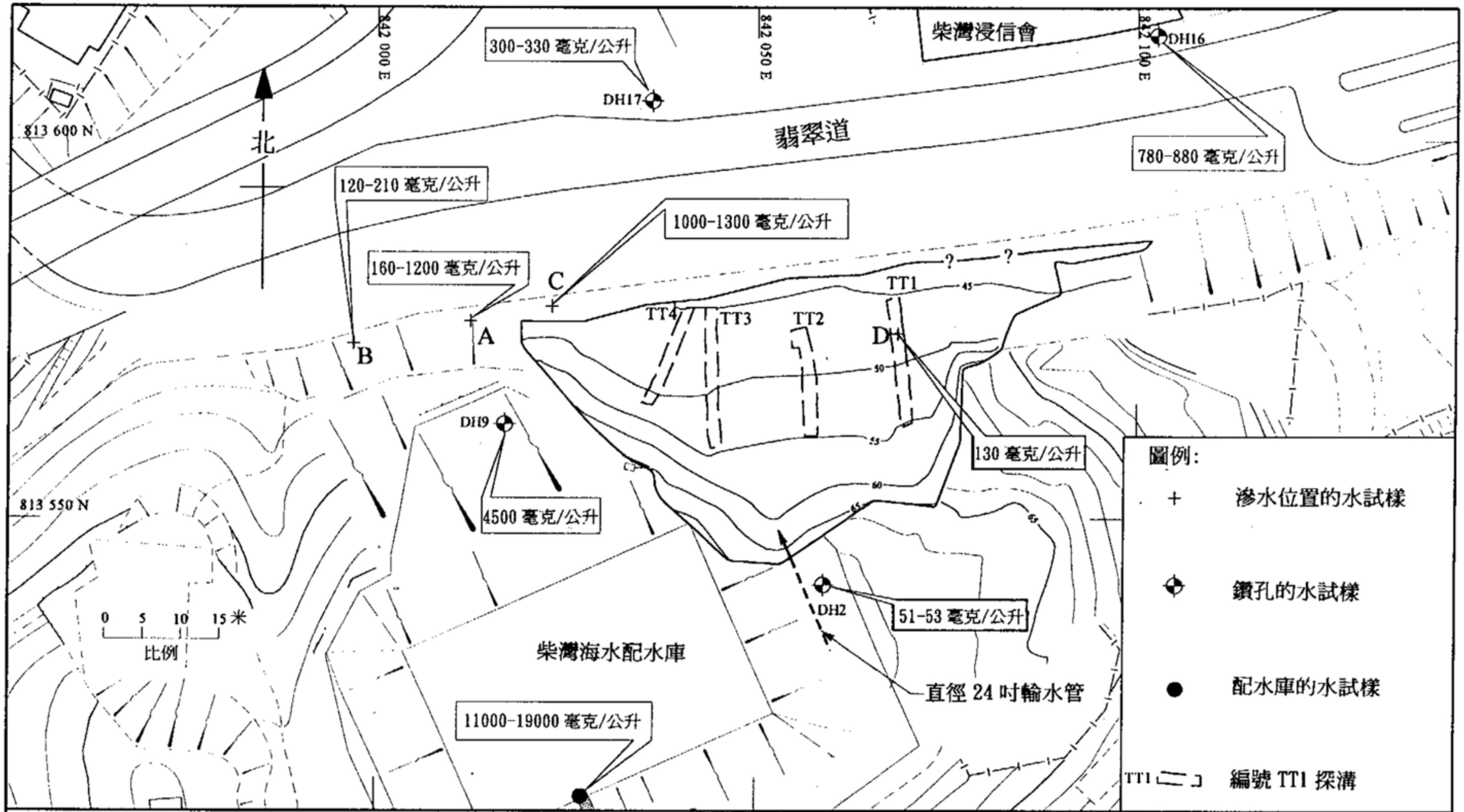
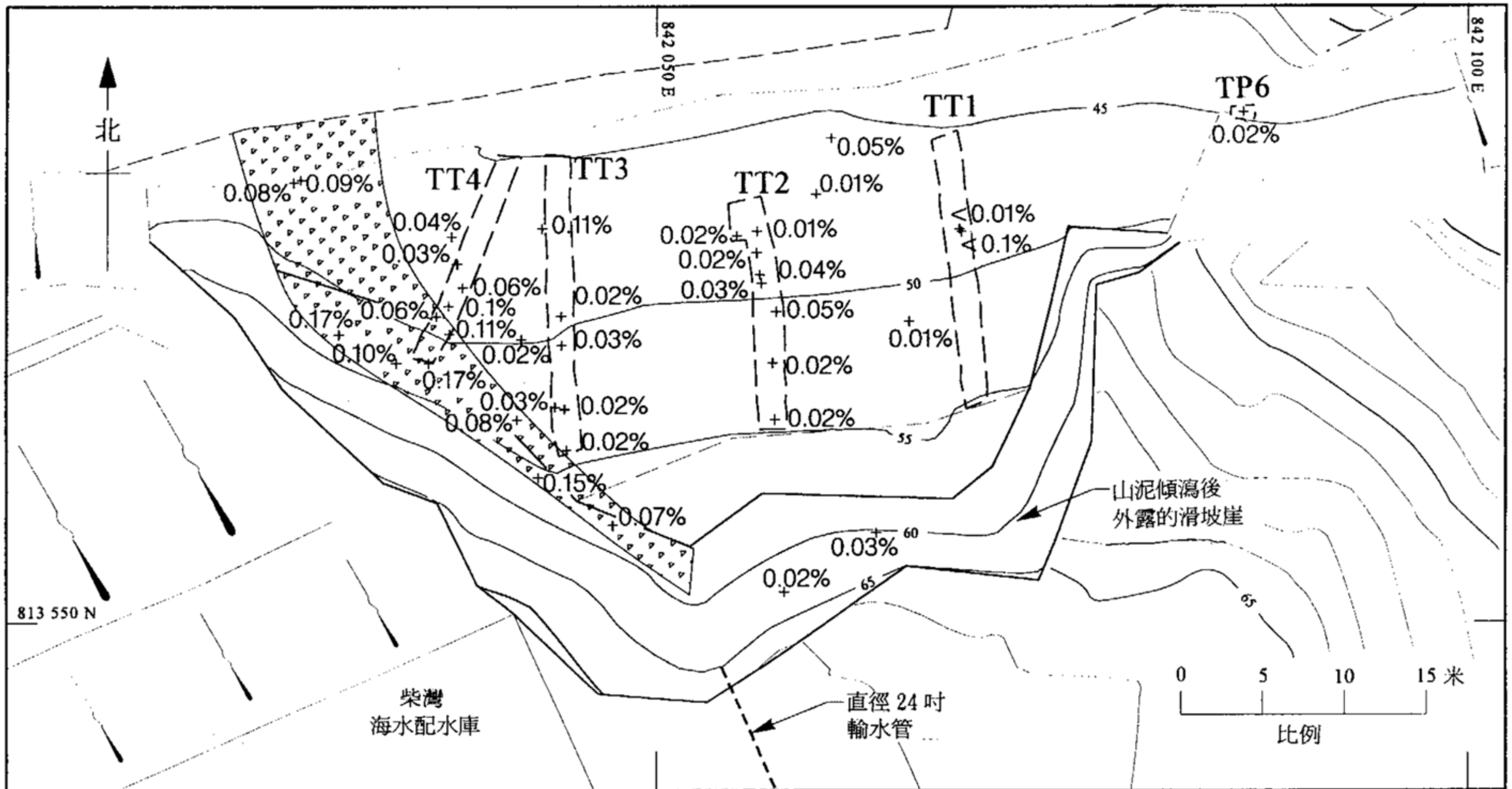


註:
(1) 水試樣的氯化物含量以毫克/公升顯示。
(2) A、B及C是斜坡面的滲水位置。D是在含豐富高嶺石的蝕變凝灰岩層上的滲水。
(3) 於山泥傾瀉東南面約200米的山谷溪澗，取得水試樣的氯化物含量爲42毫克/公升。
(4) 圖中所示爲緊急搶修工程完成後的地形。

圖 16 － 水試樣的氯化物含量試驗結果

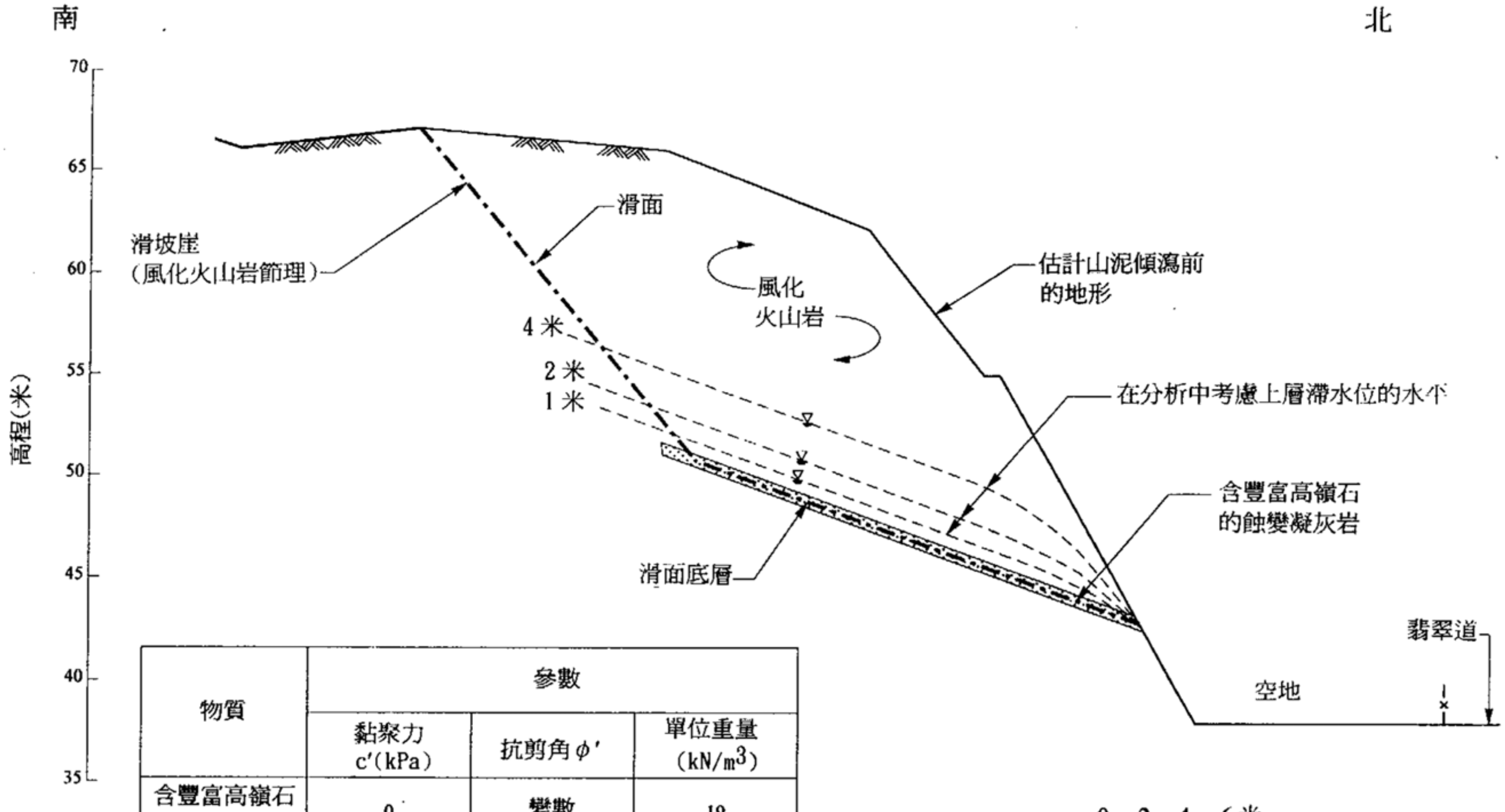


圖例：

+0.15%	採取泥土試樣進行氯化物含量試驗的地點，試驗結果以重量百分比顯示。
TT1	編號 TT1 探溝
	山泥傾瀉後，海水從直徑 24 吋輸水管流出，其可能影響的範圍。

註：　圖中所示爲緊急搶修工程完成後的地形。

圖 17 － 土壤試樣的氯化物含量試驗結果



物質	參數		
	黏聚力 c'(kPa)	抗剪角 ϕ'	單位重量 (kN/m^3)
含豐富高嶺石的蝕變凝灰岩	0	變數	19
風化火山岩節理	0	35°	19 (風化凝灰岩)

註: 沒有上層滯水位的情況亦有分析。

圖 18 – 理論穩定性分析所採用具代表性的山泥傾瀉剖面

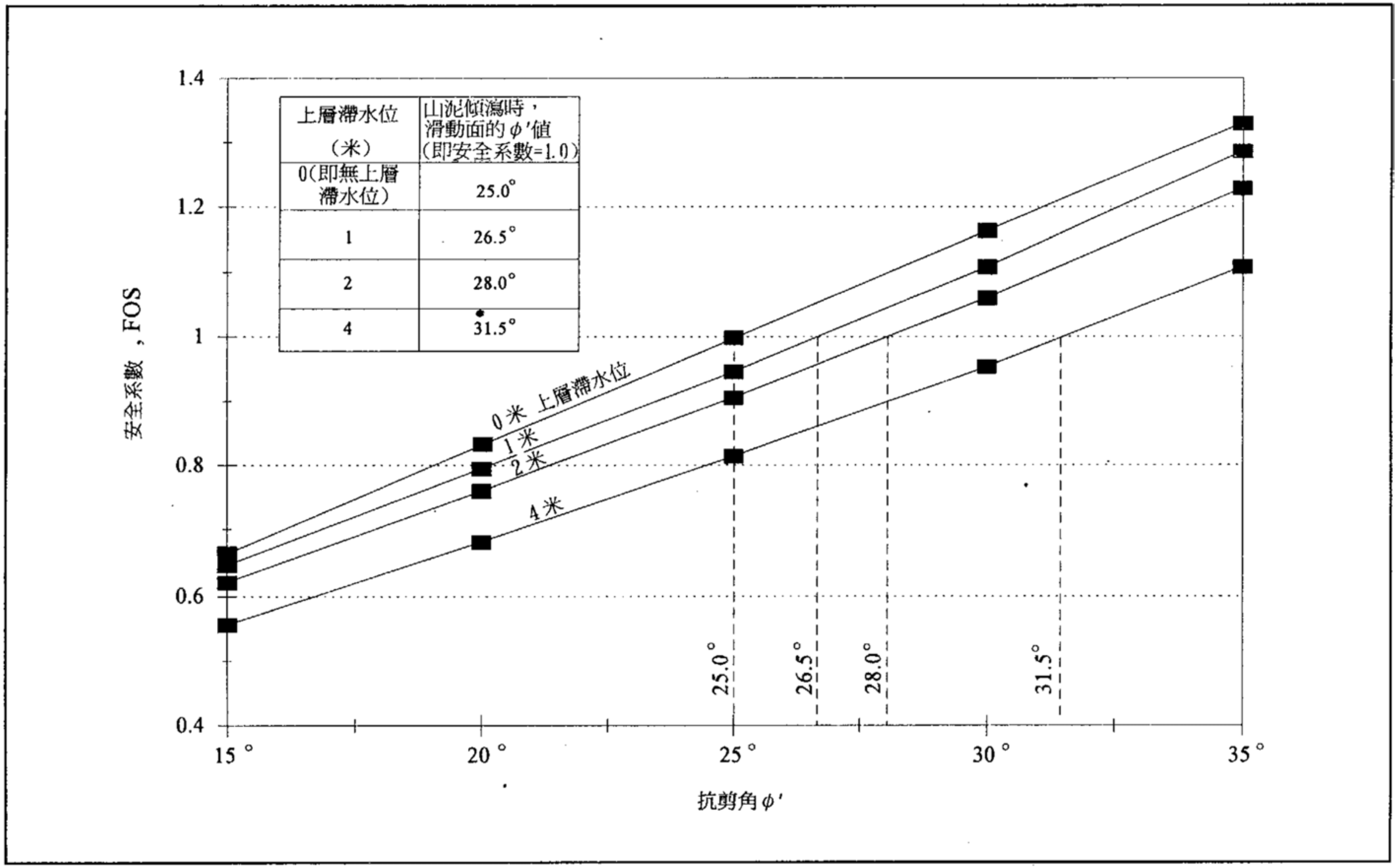


圖 19 － 理論穩定性分析的結果

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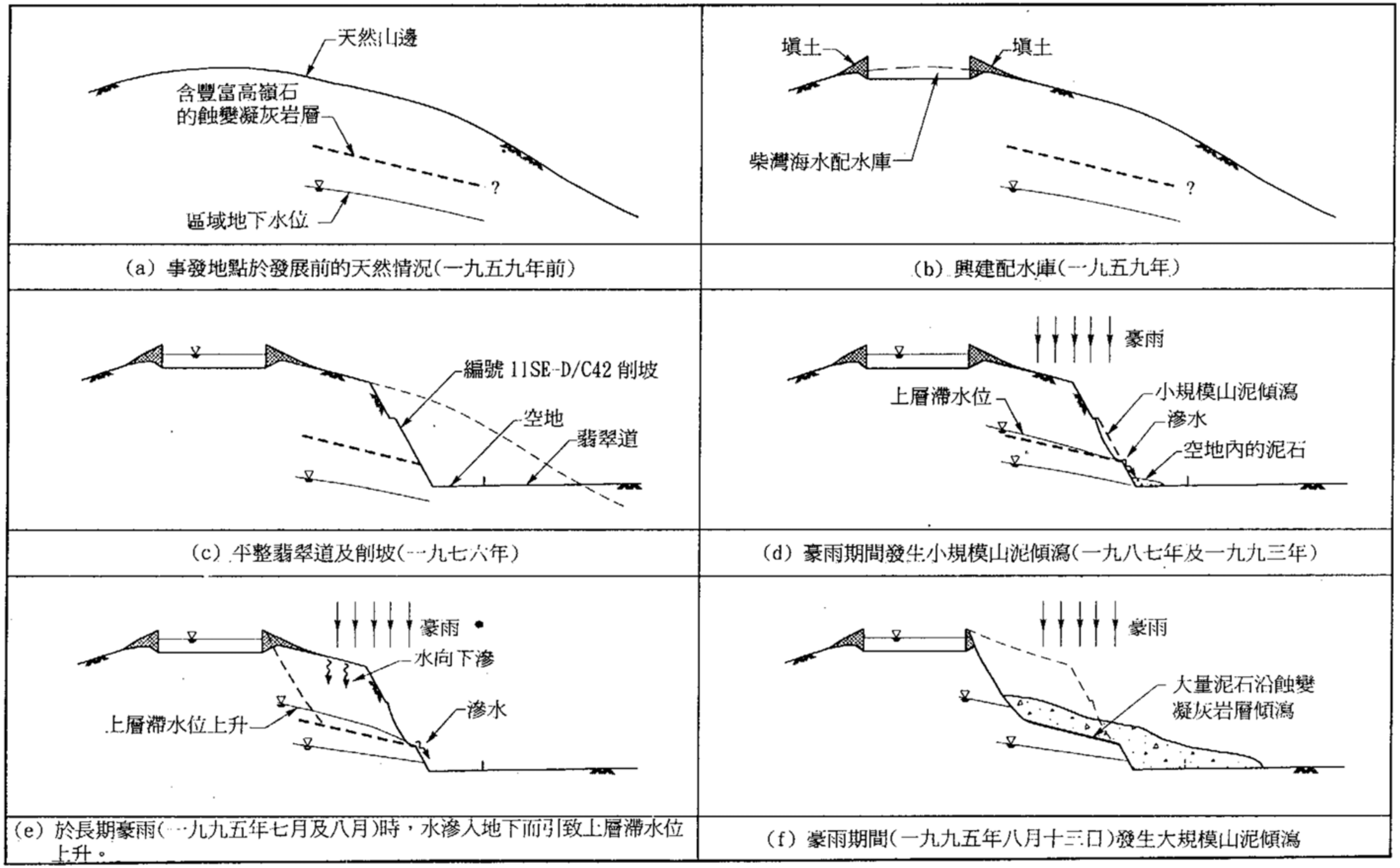

圖 20 – 大概的事發過程

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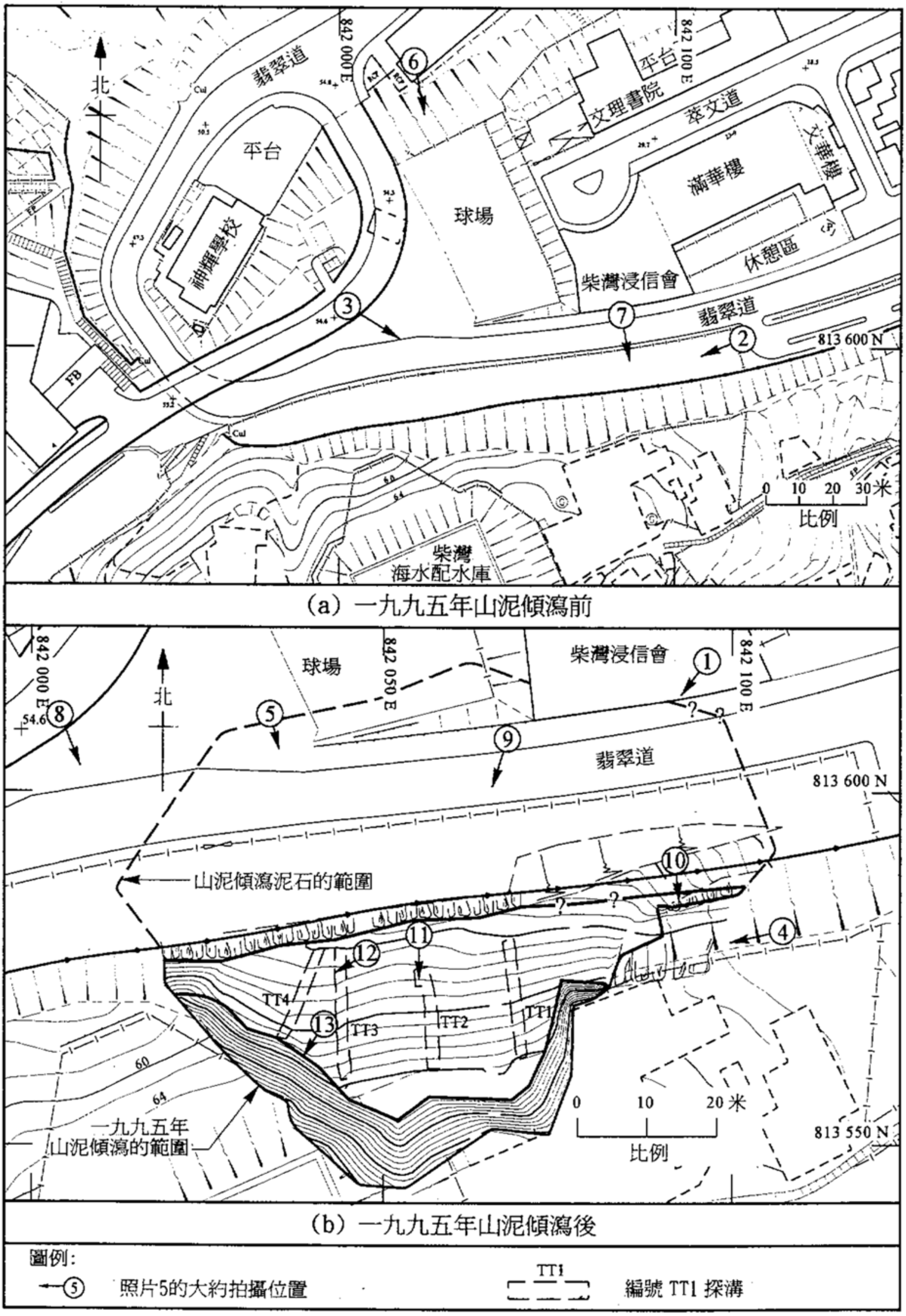

(a) 一九九五年山泥傾瀉前

(b) 一九九五年山泥傾瀉後

圖 21 – 照片位置圖

照片

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照片1 - 一九九五年八月十三日早上攝得的山泥傾瀉照片(其位置見圖21)

照片2 - 於斜坡平整工程後，攝得的編號11SE-D/C42削坡照片
(翻印賓尼(一九七七)的照片 8。其位置見圖21)

照片3 - 一九九四年十一月九日攝得的編號11SE-D/C42削坡照片
(其位置見圖21)

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照片4 - 削坡未崩塌部份
的淤塞地面排水渠
(一九九五年十月十日
拍攝。其位置見圖21)

照片5 - 山泥傾瀉泥石中的淤塞地面排水渠
(一九九五年八月十三日拍攝。其位置見圖21)

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照片6 - 一九八七年山泥傾瀉的景象
(一九九五年八月十七日拍攝。其位置見圖21)

照片7 - 一九九三年山泥傾瀉的景象
(一九九三年九月二十七日拍攝。其位置見圖21)

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的風化岩石節理
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照片9 - 山泥傾瀉底部下面的岩石面
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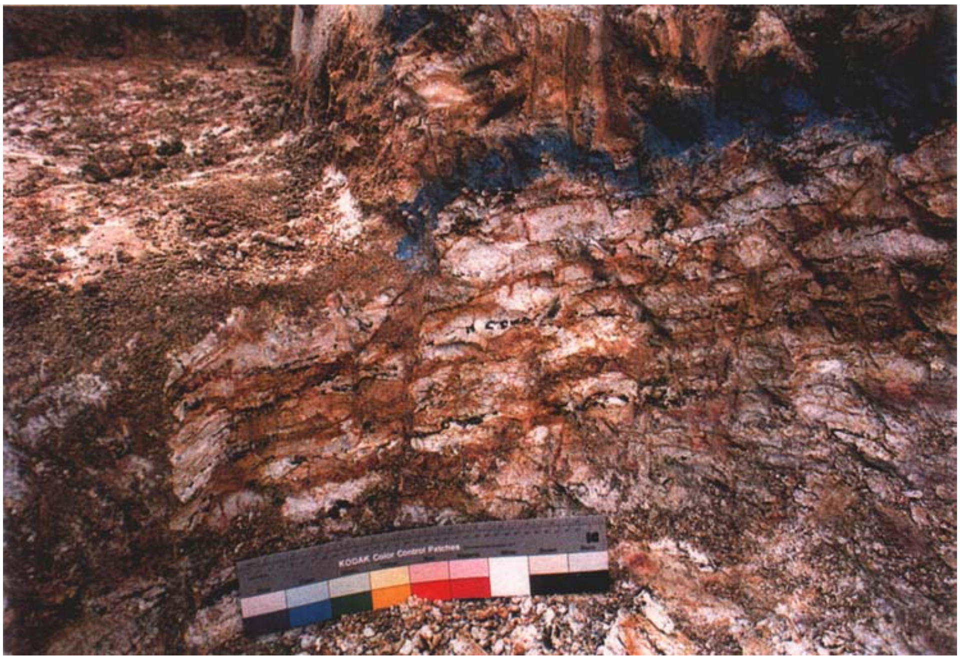
(一九九五年十月八日拍攝。其位置見圖21)



照片11 - 於山泥傾瀉底部中間部份，含豐富高嶺石的蝕變凝灰岩
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附錄A

事發地點歷史摘要

目錄

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A.1 事發地點的發展

現存最早有關事發地點的航空照片，攝於一九二四年。事發地點所在，走向東北偏東的山咀，當時尚未發展。

根據水務署的紀錄，柴灣海水配水庫建於一九五九年。 水務署(一九九五)

一九六一年的航空照片中，配水庫東北面下端斜坡有一些寮屋。從一九六九及一九七六年的照片可見，其後數年間，寮屋數目增加。

事發的削坡於一九七二至一九七六年間，由建築處(一九八六年重組爲建築署)建成，作爲興華邨第二期發展工程的一部份。一九七六年的航空照片顯示翡翠道已大致上建成，而削坡前的空地當時已平整。

建築處於一九七一年繪製的場地平整圖則，標題爲「長期道路研究第 81 號幹線，香港柴灣連成道與往興華邨通路間的建議修改路線圖」，圖中顯示翡翠道連同空地爲雙程車路。因此，該空地看似是爲未來第 81 號幹線而平整的道路備用地。 建築處圖則編號 A/45444

翡翠道山泥傾瀉位置對面的柴灣浸信會，建於一九八六至一九八七年間。

削坡頂部的寮屋於一九九一年已全部清拆。

A.2. 過往的評估

A.2.1 斜坡登記

政府委聘以製備削坡、塡土坡及擋土牆目錄(現通稱爲斜坡紀錄冊)的顧問：賓尼組合(香港)顧問工程師(賓尼組合)於一九七七年八月登記削坡爲編號 11SE-D/C42。 賓尼編號 11SE-D/C42 斜坡的現場記錄表

A.2.2 香港政府山泥傾瀉研究第 IIC 期

一九七七年十二月，賓尼組合受聘於香港政府進行第 IIC 期山泥傾瀉研究，製備了一份柴灣區斜坡穩定性研究報告。研究工作包括現場視察、場地勘探及穩定性分析，編號 11SE-D/C42 削坡亦包括在該次研究之內。 賓尼(一九七七)

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賓尼組合指出「於半高度處有 4 呎闊的坡級」，及「坡級下面露出的岩石面為未受風化至輕度風化火山岩，其節理間距極密，而道路與坡級中間有顯著的水平風化泥層(照片 8)。我們留意到配水庫下面近乎水平的節理有滲水現象。」賓尼組合報告內的照片 8，轉載於本報告為照片 2。

賓尼(一九七七)
10.1 段

該次研究鑽了兩個探孔。賓尼組合指出「其一(C8)位於配水庫與我們發現滲水的一段斜坡之間，另一(C9)則位在配水庫土地內，於削坡最高段邊緣後退約 16 米。由於通行不便，該鑽孔未能在削坡頂部進行鑽探。兩個鑽孔均深入未受風化至輕度風化火山岩內最少 9 米」。

賓尼(一九七七)
10.2 段

賓尼組合形容「位於路面與削面中段的坡級中間，有一條約 100 毫米闊及最少 50 米長的顯著水平風化節理」。賓尼組合指出，有關 C9 鑽孔「於 20.7 米至 21.0 米之間，岩芯回收率減少，這位置與削坡的水平節理約在同一水平。這提供了該節理可能是持續的證據。」

賓尼(一九七七)10.4
及 10.6 段

賓尼組合報稱「每個鑽孔分別裝上兩個孔隙水壓計，其一在鑽孔的底部，另一在中度及輕度風化火山岩的介面。每個鑽孔內上面的孔隙水壓計均依然乾涸」。至於下面的孔隙水壓計，「兩個的最高水位均在輕度至未受風化火山岩之中」。

賓尼(一九七七)
10.7 段

賓尼組合指出「削坡有兩種不穩定形式。在削坡半高度的坡級下面，可能沿岩石節理有楔體或塊體崩塌，或者鬆脫及懸垂岩石剝落。坡級之上，風化火山岩可能出現土壤形式的崩塌。由於顯著的風化泥層(第 10.1 及 10.6 段)是水平的，我們不認為它是引致不穩定的潛在原因」。

賓尼(一九七七)
10.9 段

利用立體投影圖對楔體或塊體崩塌的可能性作出評估，獲得的結論是「不大可能出現塊體崩塌」及「毋須進行大規模的修繕措施」。.

賓尼(一九七七)
10.11 段

評估土壤形式的崩塌時，賓尼組合指出「我們已假設濕水帶的下降會引致在中度與輕度風化火山岩介面之上出現上層滯水位」。報告的圖 13 顯示「所使用的強度參數：a」風化火山岩級別 IV/V $\phi'=35^{\circ}$，$c'=10kPa$；b) 風化火山岩級別 III/IV $\phi'=40^{\circ}$，$c'=10kPa$」。

賓尼(一九七七)10.7
段及圖 13

賓尼組合發現「按每 1000 年 1 遇的降雨情況所得的安全系數是可以接受的，但每 10 年 1 遇的最低安全系數為 1.09，較可接受的 1.20 值為低」，及「要增加 F 至 1.20，沿最重要的滑面(滑面 3)須有 4.9kPa 的平均毛管吸力。我們預料這一小數值應該存在，但我們建議使用毛管吸力計檢查這毛管吸力確實存在」。

賓尼(一九七七)
10.15 及 10.16 段

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一九七八年三月，史偉高顧問工程師(史偉高)代表政府，評論「賓尼組合在其柴灣區第 IIC 階段報告內建議的提高通往興華邨(第 II 期)新路毗鄰削坡穩定性的必需措施」。

史偉高於一九七八年五月十三日發給建築物條例執行處的備忘錄第 1 段

史偉高指出「我們大致上同意賓尼組合對斜坡主要節理的評估，亦同意其結論，認爲不可能在該些傾角陡峭的節理範圍內發生塊體或楔體崩塌。」

史偉高於一九七八年五月十三日發出的備忘錄的第 3iii)段。

史偉高作出評論「但是，賓尼組合在研究岩石節理時提及『一條約 100 毫米闊及最少 50 米長的顯著水平風化節理』(§10.4)。雖然這節理走向和削面平行(因此露出坡面時成水平線狀)，但我們認爲它是屬於一組向路面傾斜，約 25 度傾角的局部持續節理。該組的其中一些節理已深度風化，並有嚴重滲水跡象。我們建議應考慮該節理以上石塊不穩(即局部塊體崩塌)的可能性。視乎結果如何，有可能須在削面部份設計適當的擋護措施。(有跡象顯示該節理以上的一些石塊曾於開整削面時塌下。)」。

史偉高於一九七八年五月十三日發出的備忘錄的第 3iv)段。

史偉高指出「在削面的西端岩石上面有跡象顯示殘積土移動，引致灰泥產生裂縫。這種情況大概是因土壤/岩石介面的泥土濕水而致。沿削面多處均有明顯的滲水，我們強烈建議：a)找尋岩石及殘積土中的滲水來源，及如有可能則除去該些來源(應檢查賓尼組合的 H074/70/12 及 14 兩幅圖則內所示，斜坡頂部的水管及過濾池/海水配水庫是否有滲漏)；b)應於鬆軟物質底部及顯然有嚴重滲水跡象的岩石內，裝設排水斜管；及 c)斜坡頂部應作出保護，最低限度應到達最危險的滑面末端，以防有水滲入斜坡」。

史偉高於一九七八年五月十三日發出的備忘錄的第 3v)段。

史偉高指出「對鑽孔 C8 及 C9 記錄的另一個解釋，是認爲鑽孔內土壤/岩石的介面較假設的爲高，並且傾向削面。應考慮於濕水的土壤/岩石介面上出現滑動崩塌的可能性」。

史偉高於一九七八年五月十三日發出的備忘錄的第 3vi)段。

史偉高指出「賓尼組合亦建議於斜坡後面一個鑽孔內裝置兩個毛管吸力計，以便能証實每平方米約 5kN 的毛管吸力是否存在」，而史偉高「願意接納賓尼組合的預計，指該地點會有所需用作穩定的毛管吸力」。

史偉高於一九七八年五月十三日發出的備忘錄的第 4 段。

並無紀錄顯示其後是否已裝置毛管吸力計。

A.2.3 土力工程處的第 1 階段調查

土力工程處就防止山泥傾瀉計劃，於一九七九年七月對編號 11SE-D/C42 斜坡作出第 1 階段調查的評估。此乃初步的穩定性評估，以評定是否需要進一步展開詳細的穩定性研究。調查包括現場視察、及於沒有進行場地勘探下，根據可得資料作岩土考核。第 1 階段調查報告於一九七九年九月製成。

土力工程處(一九七九)

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「從岩石節理」觀察到「穩定的」滲水。報告內的一幅圖上繪劃了一條顯示滲水位置的線，該線與近斜坡下部份的削坡腳線平行。這條線橫貫一段 40 米長的削坡，正好在現時球場的對面。另一條滲水線橫過配水庫北面一段 15 米長的削坡，亦繪劃在圖上。又注意到「有些地方有斷層跡象」。

土力工程處(一九七九)

報告指出「已進行初步的節理測量及對潛在崩塌機制作出評估」，發現「立體測繪分析顯示三種動能上可能的崩塌機制。沿對應 5 號極的平面所發生的塊體崩塌、在平面 2 及 5 的交界線所發生的楔體崩塌、以及平面 1 所發生的翻倒。有關的極，與出露和翻倒包絡線接近，顯示僅可能發生崩塌。」

討論　項目 3 及 4(土力工程處，一九七九)

報告指出「這些斜坡崩塌引起的後果相對較小，因為大部份滑動的泥石會積聚在山腳的草地上，不會影響道路交通。只有深層崩塌才會影響斜坡頂部的寮屋，而這被認為極不可能。」

討論　項目 5(土力工程處，一九七九)

報告建議「毋須作進一步研究」及「進行灰泥及渠管的例行維修」。

建議　項目 1 及 2(土力工程處，一九七九)

A.2.4 <u>建築處第 32H 號計劃</u>

一九七九年八月，建築署就第 32H 號計劃委聘奧雅維工程顧問(奧雅納)評估削坡，「以檢查削坡，及決定確保其穩定性所須採取的措施」。這項評估是有關於提議「擴闊通往興華邨的兩條新路，翡翠道及環翠道。於擴闊工程後，現時位於道路與毗連削坡間的花園區將所餘無幾」。奧雅納於一九七九年八月二十二日向建築處提交初步評估報告。

引言 (奧雅納，一九七九)

對於「沿傾斜節理滑出外露面的小塊岩石」，奧雅納評估為「外露面可見到潛在不穩定的石塊，應予清除」，及「所有須清除的石塊不大可能超過 10 或 20 塊」。

潛在不穩定　項目 a)及討論項目 a) (奧雅納，一九七九)

奧雅納留意到賓尼組合的柴灣區山泥傾瀉研究第 IIC 期報告已討論及「沿『顯著水平風化節理』的大型岩石和土壤滑動」。顧問評論說「沿削坡中段並在路面之上約 10 米的主要『節理』，不一定是真正的節理。該節理似乎橫向地變薄並最後消失，而不是被另一組節理截斷。該節理遭無數近乎垂直的節理錯動。該『風化節理』局部厚至約 250 毫米，並有部份薄層狀構造，這可能顯示剪切。這剪切情況可能與沿北面溪流的主要斷層有關，亦可能是風化的侵入岩層」。顧問指出「這節理在其最佳外露的地方，以 10 至 25 度傾向山坡外。從岩石面可見這節理橫向地彎曲，並可能在山坡內彎曲或完全消失。從兩個鑽孔未能獲得明顯的證據」。奧雅納指出「我們認為『節理』成梯級狀及其大致上低傾斜度，使沿著其發生的大型滑動崩塌不大可能」。

潛在不穩定　項目 b)及討論項目 b) (奧雅納，一九七九)

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奧雅納報告說「可觀察到『節理』有滲水情況，這可能顯示它阻截了滲入山坡的水。但是岩石面亦見到其他滲水，而我們認爲大部份岩體，最低限度近外露面處，其透水性，足以防止任何一個節理之上的水壓上升」。

討論　項目 b) (奧雅納，一九七九)

奧雅納指「『節理』與一組罕有斜出表面約 25 度的節理近乎平行。可見的這些節理大致上橫向不闊於 1 米，而且因不常見而至未在賓尼的量度節理立體投影圖上出現。我們認爲這組節理只會引起小石塊滑動，而且可以利用上述 a)避免」。

討論　項目 b) (奧雅納，一九七九)

關於「面上級別 5 或 6 物質的小型弧型崩塌或級別 3 至 6 物質的大型弧型崩塌」，奧雅納指出「我們同意賓尼組合報告所採用的穩定性分析形式，而且同樣地論斷須有土壤的毛管吸力，以支持土坡最陡峭部份的穩定。於斜坡頂部後面，雖然雨水直接滲入，這些吸力似仍能維持。如果頂部後面的地面爲灰泥所蓋，毛管吸力和上段土坡的安全則可維持」。奧雅納進一步指出「需檢查配水庫附近滲水的含鹽量。如果所有的水都含鹽，我們必須假設其來源(最少有部份)是從配水庫或相連喉管漏出。我們亦須假設滲漏可能增加並變成危險。因此，配水庫可能須排水，以修補其表面或其相連的喉管」。

潛在不穩定　項目 c) 及討論項目 c)(奧雅納，一九七九)

有關「沿岩石/土壤介面的滑動」，顧問指出「預料風化層大致平行或稍較平緩於原來地面。這相應於東面末端約 10 度的傾斜增至近配水庫最斜約 30 度。我們不相信級別 2 及 3 或 3 及 4 之間的界線會有如此的傾斜度或持續情況，而引致出現滑動面」。

潛在不穩定　項目 d) 及討論項目 d) (奧雅納，一九七九)

有關「傾向削土的級別 4 物質內，沿傾向削坡的殘餘節理滑動」方面，注意到「量度結果顯示沒有傾斜 25 至 70 度的節理。節理的摩擦角預料超過 35 度。因此，這種滑動是極不可能的」。

潛在不穩定　項目 e) 及討論項目 e) (奧雅納，一九七九)

奧雅納建議「i)清除外露面全部有可能不穩定的岩石塊」；「ii)整理灰泥斜坡頂部排水渠 10 米以內的地面，及清除樹木以外的全部植物；在可保證雨水能流入頂部水渠的高程，蓋上 50 毫米厚的灰泥」；及「iii)應該隔離沿斜坡底部的範圍，用以攔接任何跌下的細小岩石碎片。該範圍應闊 2 至 3 米並以土壤覆蓋。」

建議　項目 i)、ii) 及 iii) (奧雅納，一九七九)

史偉高代表政府檢討了奧雅納的初步報告，於一九七九年九月二十日以備忘錄向政府提出評論。史偉高指出「我們不認爲報告載有足夠詳情，故只能作爲初步報告。但是我們按不久將來會提交工作圖則的假設，作出評論」。史偉高討論說「作爲一般準則，賓尼組合的第 IIC 期研究不一定須視作『設計』研究，而應視作『可行性』研究。但是關於斜坡 11SE-D/C42，由於對生命構成的風險低，我們準備接受可能毋須進行進一步的場地勘探。」

史偉高於一九七九年九月二十日發給建築物條例執行處的備忘錄

史偉高於一九七九年九月二十日發出的備忘錄的第 3 及 6 段。

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史偉高指出「沿斜坡長度有數處地點出現明顯的滲水」，以及「位於岩坡半高度處的 200 毫米厚的水平高度風化物質底部有滲水情況」。史偉高建議「鑑於觀察到的滲水及缺乏地下水剖面的資料，我們提議裝置排水斜管，特別是裝在土壤／岩石的介面」。

史偉高於一九七九年九月二十日發出的備忘錄的第 4i)、4iv) 及 15 段

史偉高作出的結論是「我們大致上同意奧雅納報告所建議的項目(i)、(ii)及(iii)」。史偉高指出「雖然可透過清除可能鬆脫的石塊及懸垂部份，立即穩固編號 11SE-D/C42 斜坡，我們必須謹記風化是個持續的過程。因此，建築處必須準備不時進行小規模的修繕工程，包括剝除鬆石及清除排水渠內泥石。小量鬆脫物體將掉下岩崩保留區及圍柵的範圍。」

史偉高於一九七九年九月二十日發出的備忘錄的第 12 及 17 段

奧雅納於一九八零年四月十七日發給建築處的信件中，撮要提出了對柴灣海水配水庫建築記錄圖則及對水試樣進行含鹽試驗的檢討結果。奧雅納指出「鑑於滲水可能源自配水庫，我們建議在岩坡裝置排水斜管，以使地下水位上升的可能減至最低」。此外，「我們建議在裝置排水斜管前，在削坡之上安裝孔隙水壓計，以便監察斜管在安裝期間及長期的效果。」

奧雅納於一九八零年四月十七日發信給建築處

奧雅納於一九八零年四月十七日發出信件的第 4 及 5 節

史偉高作出回應，指「我們大致上同意奧雅納就斜坡進行修繕工程所作的建議」，及「由於岩石面以下的節理不能確定，我們認爲在介面以上土壤裡完全裝置斜管會較佳」。

史偉高於一九八零年五月二十八日發給建築物條例執行處的備忘錄的第 6 段

其後，於一九八二年二月在兩個鑽孔內分別安裝兩個孔隙水壓計。

一九八二年九月，路政處(於一九八六年改組爲路政署)把奧雅納的報告及史偉高的評論交給土力工程處。一九八二年十二月，土力工程處回覆說「目前，斜坡沒有可見的危險跡象。而且，由於有草被區及小巴泊車處可作緩衝，目前這斜坡的崩塌的後果很小」及「因此，我建議目前不進行進一步的工程。日後如擴闊道路，則須向西北區的總岩土工程師正式提交申請」。可能由於斜坡工程的建議是因應日後擴闊道路而作出的，因此當時沒有安排進行斜坡工程。

路政處於一九八二年九月二十三日發給土力工程處的備忘錄

土力工程處於一九八二年十二月十七日發給路政處的備忘錄的第 4 及 5 段

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A.2.5 防止山泥傾瀉計劃

一九八六年，土力工程處根據防止山泥傾瀉計劃，曾考慮爲削坡進行防止山泥傾瀉工程。當時採用的準則之一，是不會選擇在過往報告中建議毋須採取進一步行動的斜坡。因此，該削坡並未包括在防止山泥傾瀉工程內。

一九九零年，土力工程處就防止山泥傾瀉計劃而對該削坡進行視察。土力工程處認爲可能需進行改善工程，以令該削坡能達致現行的工程標準。這被列入優先類別五以作進一步行動。

A.3 過往幾次的山泥傾瀉

削坡曾於一九八五、八六、八七及九三年發生共四次山泥傾瀉，其大約位置載於本調查報告圖 6。

土力工程處接報其中兩宗崩塌事件，即一九八七年(土力工程處編號 HK87/5/10)及一九九三年(土力工程處編號 HK93/9/11)的事件。該兩宗屬小規模事件，崩塌量在 50 立方米以內。山泥傾瀉的泥石落在削坡前面的空地。位於翡翠道另一面的柴灣浸信會，其中一些窗子於一九九三年的山泥傾瀉事件中，被飛來的小石塊撞破。

Premchitt(一九九一)及 Chan(一九九五)

一九八五年的山泥傾瀉可從一九八五年的航空照片中辨認出來。一九八六年的山泥傾瀉則在土力工程處檔案中的圖上顯示。此外，沒有找到有關這些事件的其他資料。上述兩宗山泥傾瀉的規模顯然較一九八七及一九九三年的事件爲小。

一九八七年的岩石滑坡發生後，削坡前的空地被欄開。此後，土力工程處建議地政署「在該斜坡未隨將來發展工程穩定前，不應使用山泥傾瀉坡腳爲欄杆分隔的地方」；以及可能使用土地者「在考慮該處是否適合某用途時，亦應考慮斜坡的不穩定性」。

土力工程處於一九八七年九月三日及一九九四年六月八日發給地政處/香港東的備忘錄

就一九九四年十月建議將球場及柴灣浸信會對面空地的東部分配予渠務署作貯貨場之用，土力工程處作出回應並認爲「該處仍潛在不穩定性」及指出「該處只可用作露天貯貨之用，將不批准有人居其內」。其後，渠務署建議把貯貨用地遷到配水庫下面空地的西面部份。空地西面部份及直接在其上的削坡於一九九五年四月二十七日起分配予渠務署。

土力工程處於一九九四年十月七日發給地政處/香港東的備忘錄

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A.4 參考書目

土力工程處 (一九七九) Cut Slope 11SE-D/C42, Sui Man Road。香港土力工程處，第一期階段調查報告編號 S1 85/79，3 頁加附件(沒有公開發行)。

水務署 (一九九五) Report on the Landslide at Fei Tsui, Chai Wan on 13.8.95。香港水務署，7 頁加 3 幅圖、6 份附錄及 11 幅照片。

賓尼 (一九七七) Government of Hong Kong Landslide Study Phase IIC - Report on the Stability of Slopes in the Chai Wan Area。賓尼組合(香港)顧問工程師爲香港政府撰寫，64 頁加 12 幅照片、6 份附錄及 23 幅圖。

奧雅納 (一九七九) Cutting at Hing Wah Estate, Chai Wan。提交建築設計處的初步報告，連同奧雅納於一九七九年八月二十二日的信件編號 9165/11/PAR/IK，奧雅納工程顧問香港有限公司爲建築處撰寫，6 頁。

Chan, W. L. (一九九五) Hong Kong Rainfall and Landslides in 1993。香港土力工程處，214 頁加 1 幅圖。(土力工程處報告第 43 號)

Premchitt, J. (一九九一) Hong Kong Rainfall and Landslides in 1987。香港土力工程處，108 頁加 1 幅圖。(土力工程處報告第 4 號)

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

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

部份土力工程處的主要刊物目錄刊載於下頁。而詳盡及最新的土力工程處刊物目錄，則登載於土木工程拓展署的互聯網網頁http://www.cedd.gov.hk 的"刊物"版面之內。刊物的摘要及更新刊物內容的工程技術指引，亦可在這個網址找到。

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

GEOTECHNICAL MANUALS

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

斜坡岩土工程手冊(1998)，308頁(1984年英文版的中文譯本)。

Highway Slope Manual (2000), 114 p.

GEOGUIDES

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

GEOSPECS

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

GEO PUBLICATIONS

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

GEOLOGICAL PUBLICATIONS

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

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

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TGN 1	Technical Guidance Documents