一九九五年八月十三日
深灣道山泥傾瀉事件報告
REPORT ON THE SHUM WAN ROAD LANDSLIDE OF 13 AUGUST 1995

土力工程處報告系列第178號
GEO REPORT No. 178

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

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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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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 last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
February 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 Shum Wan 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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Y.C. Chan
Deputy Head (Planning & Standards)
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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 April 1996
as Report on the Shum Wan Road Landslide of 13 August 1995

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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 Shum Wan Road adjacent to Aberdeen Harbour, severely damaged three shipyards and a factory on the sea front; a fire subsequently occurred in the collapsed structures. There were two fatalities and five other people were 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 (GEO, 1996).

It was decided to have an independent 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 achieved. An advanced draft of the GEO report was reviewed directly with the GEO.

2. DESCRIPTION OF LANDSLIDE

The original hillside between Shum Wan Road and Nam Long Shan Road, some 70 m above, sloped at about 27° and was covered in dense vegetation. The slope failed at about 0400 on 13 August 1995, subsequently carrying away Nam Long Shan Road. The slide debris, which flowed across Shum Wan Road for a distance of about 70 m from the toe of the slope, damaged and then forced the buildings adjacent to Aberdeen Harbour into the sea.

The slide scar is about 140 m in plan length and varies from about 60 m in width at the level of Nam Long Shan Road to 90 m in width near the base of the slope. The form of the backscar, which is on the uphill side of the original location of Nam Long Shan Road, is defined by steep joints in the volcanic rocks. The ground falls away immediately below the backscar into a deep depression in the hillside which is concave in section (the "concave scar"). There were some exposures of partially weathered tuffs and a clay seam within the floor and sides of the depression although these had been mainly obscured soon after the failure by debris falling from the backscar. On the downhill side of the concave scar there was a pile of debris which included the bitumen surface of Nam Long Shan road surface and two lorries which had been parked in a passing bay on the outer side of the road. This parking bay, which had been constructed on fill, was about 5 m wide and immediately downhill from the road there had been three small retaining walls.

Farther downhill, the scar was composed of a sheet of slipped debris and fill resting on top of the original ground surface (the "planar scar").

A rock cliff is present at the toe of the slope and this was overridden by the slide debris. However, much of the debris at the slope toe itself and covering Shum Wan Road was composed of soft fluvial material washed down from the slope after the main failure had taken place and ponded between the cut and a mass of slide debris burying the site of the shipyards. Three prominent erosional channels were eroded through the slipped material

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resting on the slope into the underlying formations. The sheet of slide debris on the reclaimed land occupied by the shipyards overlay, in part, organic material which had been stripped from the original slope surface and trapped below the debris.

The failure was progressive and was observed by an eye witness to be developing at about 0400 on 13 August. The main landslide occurred a few minutes later and Nam Long Shan Road failed at about 0430. Fluid debris was observed to be washed down by surface water after these failures.

The landslide involved about 26,000 m^3 of soil and rock debris. After the Po Shan Road landslide of 1972 this is the largest rapid onset slope failure within a hillside that has been affected by human activities recorded in Hong Kong.

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 investigation carried out. The method and scale of the investigation was wholly appropriate to the nature of the landslide.
>
> 2. The Landslide Site
>
> This section describes the essential features of the form of the site prior to the landslide. Attention is drawn to the drainage measures, and sewer and water services, associated with Nam Long Shan Road.
>
> 3. History of the Site
>
> A factual account of the history of the site is provided, as based primarily on the study of maps and air photographs. Attention is drawn to the limited information available on the construction of the retaining walls and passing bay next to Nam Long Shan Road, and reference is made to past squatter activities on the slope (cleared in 1988) and two small landslides which occurred in 1983.
>
> The area was studied as a part of the GEO Geotechnical Area Studies Programme (GCO, 1987). The area of the Shum Wan Road landslide is specifically delimited as a "Zone of general instability associated with predominantly colluvial

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terrain" on the Physical Constraints map in (GCO, 1987). A new aerial photographic interpretation of the development of the site has been carried out, drawing attention to the presence of relict landslide scars on the slope together with associated mass-wasting deposits. Whereas the more recent landslides on the slope had been small, the overall morphology of the slope was that of a degraded landslide complex associated with past slope movements.

4. Analysis of Rainfall Record

There are two rain gauges near the site, H20 some 1.8 km west of the site and H05 some 2 km north-west of the site; these gauges were installed in 1983 and 1979 respectively. The records from these gauges were used to evaluate 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 for the month of August, the rainfall being particularly heavy in the early part of the month. The rainfall recorded at H05 was the highest 31-day rainfall recorded and the rainfall intensities for periods shorter than 12 hours were comparable to the highest intensities recorded in previous events.

5. Description of the Landslide

5.1 Field Observations and Measurements

The description of the landslide covers the geometrical form of the landslide, the nature and distribution of the different types of debris and the progressive nature of the failure.

Most of the slope is covered by colluvial and decomposed rock debris. There was a considerable amount of fill, construction materials and general rubbish. Much of the fill, presumably derived from the passing bay to Nam Long Shan Road, together with the fragments of the retaining walls, occurs mainly in the lower half of the slope. The original surface of Nam Long Shan Road could be identified together with the two vehicles which had been parked on the passing bay. Disintegration of the rock faces which formed the backscar created bouldery screes at the very top of the landslide which were contained within the concave scar.

An important, and indeed unexpected observation, was the recognition that the debris which crossed Shum Wan Road

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onto the reclaimed land was a relatively intact "slab" of rock about 2 to 3 m in thickness. This "slab" consisted of partially weathered tuff with local infilling of joints by kaolinitic clay. Careful mapping of the "slab" revealed that there was continuity in the geological structures within the rock of the "slab" indicating that it moved from the hillside to the waterfront in essentially one piece. The surface of the "slab" was covered with rafts of vegetation identical to that formerly growing on the lower part of the hillside.

Reference is also made to the dislocation of the sewer and water pipes, and the observation of discharges from both pipes after the failure.

5.2 Witness Accounts

Somewhat unusually for a landslide failure at night, a detailed account of the sequence of events was obtained from eye witnesses which has proved of considerable corroborative value in reconstructing the likely mode and sequence of failure.

6. Subsurface Conditions at the Site

6.2 Geology

The geological conditions have been investigated by means of mapping of the landslide area, trenches, trial pits, GEO probe tests, boreholes and a seismic reflection survey.

The landslide took place in volcanic rocks grading from completely to slightly decomposed tuff; the depth of weathering is somewhat greater at the top of the landslide. There was a thin cover of colluvium which was carried down by the landslide and does not appear to have contributed to the origin of the failure. Thermoluminescence tests on quartz particles suggests that this colluvium is of the order of 40,000 years old. There are uncertainties associated with the validity of such tests but calibration of the testing method against another site in Hong Kong, where C-14 dates are available, suggests that the thermoluminescence data for the Shum Wan Road landslide may be reliable.

The volcanic fabric in the tuffs dips mainly north-east, the strike being normal to the hillside. Within the landslide the dips are steep (70° to 90°), but outside the limits of the landslide the dips are more gentle (10° to 40°). There are a number of joint sets and the sub-vertical joints become closely spaced within a 6 m zone striking north-west within the concave scar.

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A planar layer of white kaolinitic clay, overlain by a buff, slightly laminated clay layer occurs within the floor of the concave scar. This thin clay seam dips downhill and is slickensided in a downslope direction.

6.3 Material Properties

A comprehensive series of classification and strength tests was carried out; in situ density and permeability tests were also completed. Some of the liquid limit values for the clay seam were unusually high, indicating an activity of about unity whereas kaolinite would normally give a value of about 0.4. X Ray diffraction studies of the white and buff clays confirmed that their mineralogy was similar and both contained kaolinite with probably some halloysite; the presence of the halloysite could explain the relatively high activity values.

For the partially weathered tuff the shear strength parameters were determined to be $\phi' = 38°$ and $c' = 5$ kPa which is comparable to the range for similar material at other sites.

For the kaolinitic clay the peak shear strength parameters were determined to be $\phi' = 26°$ and $c' = 8$ kPa, and the strength parameters of the clay with slickensliding (probably close to the residual strength) were determined to be $\phi' = 21°$ and with zero apparent cohesion.

In situ permeability tests on the partially weathered tuff demonstrated that the tuff, where the jointing was very closely spaced, was relatively permeable.

The fill which formed the passing bay was completely removed by the landslide and its original in situ condition cannot be determined. Fill from an adjacent area, outside the limits of the landslide, was in a loose to very loose condition; such materials would be expected to be relatively permeable.

6.4 Groundwater Condition

Seepage was observed from a number of locations in the landslide. At about mid-height of the slope the seepages correspond to a base groundwater level of about 1 to 3 m below the original pre-failure ground surface.

The regional groundwater level, in rock, is about 5 m below the base of the concave scar in the upper part of the landslide. Seepages above clay seams persisted for about a

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week after the landslide and recurred following rainfall, indicating the presence of perched water table conditions.

7. Condition of Drainage and Water-carrying Services at Nam Long Shan Road

Catchpits and drains were observed to be partially blocked after the landslide. During heavy rain on 14 August 1995 the surface of Nam Long Shan Road was observed to be carrying about 350 litres per second which discharged onto the top of the landslide. Comparison with the rainfall intensity on 13 August suggests that Nam Long Shan Road may have been carrying about 470 litres per second at the time of the landslide.

A closed-circuit survey of the existing sewer pipe indicated the presence of cracking and open joints. In situ inspection of an intact section of pipe indicated that only minor leakage was likely. The fresh water main was formed by threaded pipes and no evidence of leakage was observed.

8. Likely Mode and Sequence of Failure

This section integrates the information available from different sources to provide a coherent, consistent account of the landsliding process.

The main landslide was in weathered rock and was in two parts which probably moved together until the final stages of failure. However, it is probable that the failure was initiated by a small landslide in the fill, presumed to be in a loose state, underlying the passing bay. The small slip spread fill, together with fragments of the retaining walls, over the surface of the lower part of the landslide demonstrating that this debris was dislodged and carried well down the slope early in the failure sequence. The distribution of the fill and the retaining wall fragments would suggest that the failure was in the form of a rapid debris flow such as can originate from loose fill slopes. This slip would then have enabled water flowing down Nam Long Shan Road to also discharge directly onto the upper part of the slope.

The upper part of the main landslide mass took place as a rotational (spoon shaped) slip and this was in contact with the lower part which was a translational (planar) slip. The landslide initially moved relatively slowly. On the commencement of more rapid movement, the lower, translational, failure detached itself as the "slab", travelled down the slope, crossed Shum Wan Road and demolished the structures adjacent to Aberdeen Harbour.

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The upper part of the landslide mass remained on the slope. Large volumes of water continued to be discharged onto the top of the slope causing rapid erosion and washing of soft debris onto Shum Wan Road which collected between the rock cut and the eastern side of the "slab". Subsequently the remnants of Nam Long Shan Road collapsed on top of the slide mass followed by rock falls from the backscar.

9. Theoretical Stability and Seepage Analyses

9.1 General

Stability analyses of the two components of the landslide have been carried out.

9.2 Upper Part of Hillside

Stability analysis demonstrates that the upper part of the landslide would be stable under the normal regional groundwater conditions. Failure, however, could occur on the clay seam if perched water table conditions gave rise to water heads ranging from 1 to 5 m depending on whether the strength of the slickensided clay or the peak shear strength is adopted. A rapid rise in the perched water table, and the consequential heads, could have been induced by direct access of the water flowing down Nam Long Shan Road into the top of the landslide after the initial fill slope failure. It is unlikely that direct infiltration of rainfall into the ground, and improbable that any minor leakage from the sewer pipe, could have created the perched water table condition.

9.3 Lower Part of Hillside

In the lower part of the slope the base groundwater surface was close to the ground surface and the analysis demonstrates that the slope would theoretically be unstable under a range of groundwater conditions, and dependent on the proportion of clay-infilled joints within the sliding surface. This conclusion is consistent with the observation that the slope has been subject to movement in the geological past; major instability has probably not occurred for hundreds and possibly thousands of years.

10. Diagnosis of the Causes of the Landslide

This section provides a comprehensive overview of the

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slope failure process as derived from the investigations. The writer is in agreement with this analysis.

The Shum Wan Road landslide was caused by the effect of elevated groundwater conditions, following an exceptionally wet period of days and during an intense rainstorm, on a slope which had moved previously in the geological (rather than historical) past. The landslide was initiated by a small fill failure at the top of the slope which then permitted the discharge of water from Nam Long Shan Road directly onto the slope. Rapid infiltration into the rock mass generated a local perched water table on a clay seam which rapidly reduced the stability of the upper part of the slope. The lower part of the slope was itself at marginal stability, being partially founded on clay-filled joints in rock, and this was made more severe by the enhanced groundwater conditions associated with the heavy antecedent and then-current rainfall. The lower part of the slope could provide no effective toe support to the upper part. Progressive movements then accelerated to carry away the lower part of the failure, in the form of a intact slab of altered rock across Shum Wan Road to demolish the structures on the reclaimed ground.

11. <u>Other Conceivable Factors</u>

This section reviews a number of factors which may have had an influence on the site condition such as previous squatter activities, illegal dumping and heavy vehicular traffic. The writer agrees that these factors can be regarded, in relative terms, as having an insignificant influence on the Shum Wan Road landslide.

12. <u>Conclusions</u>

The conclusions are a summary of the main contributory factors to 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 Shum Wan 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 causes of, the landslide. The writer is in agreement with the report on all essential matters.

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5. LESSONS TO BE LEARNT

The Shum Wan Road landslide has not identified any new features not previously recognised in geological or landslide prevention practice within Hong Kong. However, there are features relevant to the cause of the landslide which deserve to be highlighted.

5.1 Structural and Mineralogical Controls on Landsliding in Volcanic Rocks

The Shum Wan Road landslide was controlled by structures within the volcanic bedrock which were not bedding-related, and not obviously related to local joint and fault patterns. The presence of kaolinitic clay seams and clay-filled joints at shallow depth within the rock mass was a major contributory factor both to the relatively low shear strength as well as to controls on shallow groundwater flow.

5.2 Discharge of Water along Roads at Head of Potentially Unstable Slopes

The discharge of water into the top of a slope can be an important factor in triggering a landslide. Continued discharge into a slope following a failure will weaken and soften materials within a slope and can prolong the downhill movement of debris. There should be awareness as to the role of roads in acting as catchments for collecting and channelling water into the upper part of slopes.

5.3 Natural Slope Failures

The Shum Wan Road landslide was not a natural failure as it was caused by human influences. However, the landslide occurred in a slope which had moved in the past as identified by the GASP studies (GCO, 1987) and confirmed by more recent aerial photographic studies. In the assessment of the stability of a slope the possible role of natural processes should be taken into account.

6. REFERENCES

GEO (1996) Report on the Shum Wan Road Landslide of 13 August 1995. Volume 2: Findings of the Landslide Investigation. Geotechnical Engineering Office, Hong Kong, 51 p.

GCO (1987) Geotechnical Area Studies Programme - Hong Kong and Kowloon (GASP Report 1), Geotechnical Control Office, Hong Kong, 170 p, plus 4 maps.

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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 April 1996
as Report on the Shum Wan Road Landslide of 13 August 1995

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EXECUTIVE SUMMARY

On 13 August 1995, a landslide took place at the hillside above Shum Wan Road, Aberdeen. It caused the collapse of a 30 m long section of Nam Long Shan Road that included a passing bay supported by a fill embankment. The landslide debris crossed Shum Wan Road and damaged three shipyards and a factory near the seafront. The landslide resulted in two fatalities, and five other people were injured.

A comprehensive investigation into the landslide was carried out by the Geotechnical Engineering Office (GEO) during the period August 1995 to March 1996. This detailed study included a desk study, interviews with witnesses, topographic survey, observations and measurements at the landslide site, geological mapping, ground investigation, examination of the condition of drainage systems and water-carrying services, theoretical stability and seepage analyses, and diagnosis of the causes of the failure.

The investigation concluded that the main landslide involved two distinct parts that occurred almost simultaneously. The failure was caused principally by:

(a) the presence of weak layers in the ground, i.e. clay seams and clay-infilled joints,

(b) ingress of water during prolonged heavy rainfall,

(c) a minor failure of the fill embankment below a passing bay on Nam Long Shan Road, and

(d) water flowing along Nam Long Shan Road, because of partial blockage of its drainage system, and discharge of part of this water onto the hillside.

This report presents details of the investigation and its findings.

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1. INTRODUCTION

At about 4:00 a.m. on 13 August 1995, a landslide took place at the hillside above Shum Wan Road, Aberdeen (Plate 1 & Figure 1), causing the collapse of a 30 m long section of Nam Long Shan Road. The landslide debris crossed Shum Wan Road and damaged three shipyards and a factory near the seafront. The landslide resulted in two fatalities, and five other people were injured.

The Geotechnical Engineering Office (GEO) of the Civil Engineering Department commenced an investigation of the landslide on the morning of 13 August 1995. A Progress Report (Geotechnical Engineering Office, 1995) giving an interim account of the landslide was issued on 21 September 1995.

The investigation, which was conducted between August 1995 and March 1996, included the following key tasks:

(a) desk study, including review of relevant documentary records, examination of aerial photographs and old topographic maps of the site, and analysis of rainfall data,

(b) interviews with witnesses to the landslide and with other concerned persons,

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

(d) geological mapping,

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

(f) inspection of the condition of drainage systems and water-carrying services, and

(g) theoretical stability and seepage analyses.

This report presents the findings of the investigation. Full details of the investigation work undertaken and the results obtained are contained in a set of documents which have been placed in the Civil Engineering Library of the Civil Engineering Building, and these are accessible by the public.

2. THE LANDSLIDE SITE

The landslide on 13 August 1995 occurred at a hillside between Shum Wan Road and Nam Long Shan Road (Figure 1). Prior to the landslide, the hillside was densely vegetated and had an overall gradient of about 27°.

There were three concrete retaining walls in the vicinity of the landslide area below

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Nam Long Shan Road (Figure 1). Two of the walls were about 2 m in height and probably supported the Nam Long Shan Road embankment before the construction of a passing bay. The third wall was about 1.2 m high and was likely to have been constructed to form a squatter platform.

Nam Long Shan Road was about 5 m wide at the location of the landslide, and there was a passing bay on the downhill side of the road. The passing bay, estimated from aerial photographs taken in 1994 to be about 5 m wide, was supported by a fill embankment about 10 m high from toe to crest. The passing bay embankment partly concealed one of the 2 m high retaining walls (the southern concrete retaining wall). There was a 4 m high cut slope on the uphill side of the road. None of the above man-made features were registered in the Catalogue of Slopes prepared by consultants for the Hong Kong Government in 1977 and 1978. It is not known why the 10 m high fill embankment was not registered. The other features did not fulfil the criteria for registration.

At part of the toe of the failed hillside along Shum Wan Road is a steep 7 m high rock cliff. Registered cut slope No. 15NW-B/C77, which did not fail in the incident, is situated to the south of the landslide (Figure 1). Shum Wan Road is 7.5 m wide.

The reclaimed land between Shum Wan Road and the seafront at Po Chong Wan is the site of a temporary industrial area containing a number of shipyards and factories (Figure 1).

Surface water from the hillside above Nam Long Shan Road is collected by natural stream courses and man-made channels and discharges into catchpits leading to cross-road drain pipes buried under the road (Figure 2). Water from these pipes flows into stream courses or man-made channels further downhill, as does surface water discharged from Nam Long Shan Road by drainage openings in the concrete upstand along the downhill side of the road. At the southern side of the landslide, a 1.2 m wide stepped-channel conveyed water from Nam Long Shan Road to Shum Wan Road.

A 225 mm diameter sewer and a 100 mm diameter private fresh-water main run along Nam Long Shan Road.

3. HISTORY OF THE SITE

The site development history has been determined from aerial photographs by expert interpreters, from old maps of the site and from a review of other documentary information.

The earliest available aerial photographs of the site were taken in 1945. Nam Long Shan Road can be seen in these photographs, and it appears that the two 2 m high concrete retaining walls below Nam Long Shan Road had been constructed by that time. Photographs taken in the 1970's show that the passing bay involved in the landslide was added to Nam Long Shan Road between 1976 and 1977. The passing bay was constructed on a new fill embankment.

The photographs show that, by 1977, works had commenced on the reclamation into Po Chong Wan from Shum Wan Road. The cut slopes along Shum Wan Road were formed between 1977 and 1978.

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Along with the roadworks at Nam Long Shan Road and Shum Wan Road, squatter activities on the hillside are evident from aerial photographs. A number of squatter huts are shown on Survey Sheet No. 15-NW-4C dated October 1984 (Figure 1). The 1.2 m high concrete wall below Nam Long Shan Road was likely to have been constructed in the 1970's as part of the squatter development. Signs of illegal dumping from Nam Long Shan Road are apparent in photographs taken since 1977.

There is evidence of minor landslips or soil erosion within or close to the collapsed hillside in photographs taken since the 1940's. These were mainly concentrated on the embankments along Nam Long Shan Road. In the 1988 and 1991 photographs, two patches of erosion can be seen on the fill embankment immediately below the passing bay involved in this landslide. The erosion patches are directly above the southern concrete retaining wall, their approximate locations being shown in Figure 1.

A large ground depression in the hillside north of the landslide is apparent in the 1949 aerial photographs. It may indicate an ancient landslide scar (Figure 1).

There are documentary records (Choot, 1993) of two minor landslips in 1983 at the landslide area (Figure 1). The two landslip incidents, which were classified as "erosion of natural slope" and "erosion of fill platform" respectively, occurred on 17 June 1983.

The squatter huts on the hillside were completely cleared in 1988 under Government's Non-development Clearance Programme.

So far as can be established, there have been no previous studies of the unregistered man-made slopes and retaining walls in the vicinity of the landslide area. There is no record of any design documents for these earthworks and retaining walls having been submitted to the GEO for checking.

The GEO carried out a territory-wide Geotechnical Area Studies Programme from 1979 to 1985 to provide a geotechnical basis for the planning and management of land use in Hong Kong. Along with a significant proportion of hillsides elsewhere in the territory, the land in the neighbourhood of the landslide is classified as a "zone of general instability associated with predominantly colluvial terrain" on the 1:20 000 scale Physical Constraints map produced in the Geotechnical Area Studies Programme (Geotechnical Control Office, 1987a) (Figure 3). The areas mapped as "zones of general instability" are those with signs of downslope mass movement at some time in the past.

4. ANALYSIS OF RAINFALL RECORDS

Two GEO automatic raingauges are located close to the landslide (Figure 4). Raingauge No. H20 at Ap Lei Chau Estate is about 1.8 km west of the site, while raingauge No. H05 at Aberdeen Treatment Works is about 2 km to the northwest.

Rainfall records from both raingauges have been analysed. Their rainfall patterns and intensities in the period before the landslide are broadly similar. The record from raingauge No. H05 is presented in Figure 5 to illustrate the probable rainfall pattern and intensity at the landslide area.

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There was heavy rain during the hours before the landslide at 4 a.m. The distribution across the territory of the 4-hour and 24-hour rainfall before the landslide is shown in Figure 4. Between 11 p.m. on 12 August and 3 a.m. on 13 August, 159 mm of rain was recorded, with a peak hourly rainfall of 48 mm between 2 a.m. and 3 a.m. (Figure 5). The recorded 30-hour rainfall at raingauge No. H05 before the landslide was 381 mm.

The rainstorm on 12 August 1995 prior to the landslide was preceded by a heavy rainstorm around 3 August 1995. A total of 846 mm of rain was recorded by raingauge No. H05 in the 13 days before the landslide (Figure 5).

A comparison between the pattern of the rainfall before the 1995 landslide and that of previous major rainstorms affecting the area since the installation of raingauge No. H05 in 1979 is shown in Figure 6. It can be seen that the rainfall preceding the landslide is on the high side, being the highest recorded by the raingauge for durations exceeding 15 days and comparable to the highest experienced previously for rainfall durations of 24 hours or less. The only period with similar rainfall intensities was that of 23 July 1994.

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 uncommon, with a corresponding return period of about 75 years.

It is apparent that intense rainstorms, of very long return periods according to the rainfall data at the Royal Observatory, have occurred every year in Hong Kong since 1992. The GEO is reviewing whether the repeated occurrence of such rare rainfall events represents a change in the regional climate.

5. DESCRIPTION OF THE LANDSLIDE

5.1 Field Observations and Measurements

The extent and profile of the landslide and the debris were determined by topographic surveys carried out by the Survey Division of the Civil Engineering Department before the landslide was disturbed by any remedial works. The extent of the landslide is shown in Figure 7, and a cross-section through it is given in Figure 8.

The landslide resulted in a 70 m high scar, with a width varying from about 50 m just below Nam Long Shan Road to about 90 m above Shum Wan Road. The upper part of the landslide surface (Figure 8) was concave in shape and was up to about 12 m in depth below the pre-failure ground surface. The lower part of the landslide surface was planar and was 2 to 3 m below the pre-failure ground surface.

The rock cliff at the toe of the hillside did not fail during the landslide. At the top of the landslide scar, a 30 m section of Nam Long Shan Road collapsed, and the landslide extended a short distance up the slope above the road.

The landslide released about 26 000 m^3 of soil and rock, about 12 000 m^3 of which remained on the landslide surface (Figure 8). The remaining debris was deposited on Shum Wan Road and the reclaimed land to the west, spreading over an area of about 5 000 m^2 (Figure 7). A large volume of debris was also deposited in the slipway of a shipyard. The

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top surface of the debris on the reclaimed land was almost horizontal.

The landslide at Shum Wan was unusually large in size (26 000 m^3 in volume). A review of the available records shows that this is the largest landslide in Hong Kong over the past twenty years.

The debris can be classified broadly into four major types (Figure 7). On the reclaimed land to the west of Shum Wan Road, the debris was in the form of a relatively intact ‘slab’ of material, generally about 2 m thick but up to 3 m in places (Plate 2). The ‘slab’ consisted of partially weathered tuff with disturbed but recognisable joint structures of the original weathered rock mass. The joints were closely spaced and were locally infilled with white kaolinitic clay up to about 10 mm thick. The ‘slab’ was underlain in places by a thin layer of mainly vegetation and top soil. Clumps of vegetation were common on the top of the ‘slab’, and some were also deposited around the outer edge of the ‘slab’ near the sea front.

The debris in the space between the ‘slab’ debris and the hillside comprised mainly a very soft or loose fluvial deposit of clay, silt, sand, gravel and some cobbles and boulders (Plate 3). The debris was generally about 2 m thick.

The planar part of the landslide surface was generally covered by soil debris up to about 3 m thick. The soil debris was up to about 5 m thick within the concave scar. The debris included fill material and refuse, such as bottles, polystyrene, car tyres and construction waste (Plate 4). Other man-made objects, including concrete retaining wall fragments, broken pieces of bituminous pavement, sections of 225 mm diameter earthenware pipes, 100 mm diameter galvanised iron water pipes and concrete slabs, were also found (Figure 9). It was noted that most of the concrete retaining wall fragments (Plate 5) were deposited in the lower planar scar, whereas the pieces of bituminous pavement (Plate 6) were all within the upper concave scar.

A large amount of rock debris generally about 2 m thick overlaid the soil debris on the concave scar (Plate 7). The largest rock fragment was about 3 m across.

Two trucks parked on Nam Long Shan Road were brought down during the landslide. One remained sitting on a piece of bituminous pavement which dipped at about 14° to the east on top of the debris. The other truck was buried by debris. Another truck and a taxi remained poised on Nam Long Shan Road, overhanging the crest of the landslide scar. A number of cars were buried by debris on Shum Wan Road.

The scouring action of surface water resulted in three prominent erosion channels in the landslide debris (Figure 7). The erosion channels were typically about 1 m deep and 2 m wide. The largest one was up to about 6 m in width over much of its length.

5.2 <u>Witnesses Accounts</u>

GEO officers interviewed eleven persons and reviewed other records which might provide information about the landslide event, such as Police records and the Ocean Park record of the time that water and power supply to the park area were cut off.

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According to eye-witnesses, the landslide occurred at about 4 a.m. At that time, the lower part of the hillside was illuminated by security lighting from a building at Shum Wan Road whereas the upper part of the hillside was fairly dark. One eye-witness observed a small white patch on the hillside near the position of Nam Long Shan Road. The patch grew bigger progressively. Suddenly the lower portion of the hillside bulged out and slid down as a whole piece across the full width of the landslide scar. The event was reported to the Police at 4:06 a.m. by another eye-witness. Nam Long Shan Road did not collapse during this main landslide but, according to eye-witnesses, failed about half an hour later in a subsequent slip.

A large volume of water was seen discharging onto the landslide scar from the broken sewer pipe until the early afternoon of 13 August 1995. The private fresh-water main was turned off by staff of the Ocean Park at about 7:30 a.m. on 13 August.

Some witnesses reported that there had been illegal dumping of refuse and construction waste on the hillside downslope of Nam Long Shan Road, and that the road had been used frequently by heavy construction vehicles in the months before the landslide. Witnesses also said that the drainage channels down the hillside in the landslide area were relatively dry even when it was raining in the past few months, and that muddy water was seen on the surface of the hillside. Another witness who walked part way up the 1.2 m wide stepped-channel (Figure 2) on 6 August 1995 did not see any blockage of the channel.

6. SUBSURFACE CONDITIONS AT THE SITE

6.1 General

The subsurface conditions at the landslide area were determined from information obtained from desk and field studies. The desk study comprised a review of existing geotechnical data. The field studies included a ground investigation consisting of eight boreholes, fourteen trial pits, nine trial trenches, 22 GEO probe tests, a seismic refraction survey and geological mapping (Figure 10). Piezometers were installed in boreholes to monitor the groundwater pressures. Information from the ground investigation carried out on the adjoining hillside in relation to remedial works design was also used in determining the subsurface conditions at the landslide area.

6.2 Geology

The geology at the landslide area comprised a thin mantle of colluvium overlying partially weathered fine-ash to coarse-ash crystal tuff. A typical geological section through the landslide is given in Figure 11. The colluvium, as exposed on the adjoining hillside, is predominantly a silt/clay with gravel and cobble clasts, forming an impersistent layer up to about 1 m thick. The age of the colluvium determined from laboratory dating of three samples, by the Guangdong Institute of Geochemistry using the thermoluminescence technique, is in the range of 35 000 to 48 000 years before present (Guangdong Institute of Geochemistry, 1995).

Rock fabrics in the partially weathered tuff dip mainly northeast. On the adjoining hillside either side of the landslide, they dip at 10° to 40° to the horizontal. However, the dip

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is steep (70° to 90°) within the concave scar area.

Two sub-vertical joint sets and at least two gently dipping (20° to 35°) joint sets were exposed in the landslide and confirmed by measurements in boreholes. The joints were generally closely to widely spaced. However, the sub-vertical joints within the concave scar were very closely spaced within a zone about 6 m wide striking in a northwesterly direction. These sub-vertical joints would have permitted relatively easy downward passage of water through the partially weathered tuff.

Weathering within the rock mass was more pervasive within the area of very closely spaced sub-vertical joints than elsewhere. Over the area of the concave scar, the completely to highly decomposed tuff was up to about 20 m thick prior to the landslide (Figure 11). This compares with the more typical thickness of about 5 m for a similar zone of completely to highly decomposed tuff with wider joint spacings in the area of the planar scar and in the adjoining ground to the north of the landslide.

Joints within the partially weathered tuff were commonly coated with manganese oxide and infilled with white clay up to about 15 mm thick. An extensive clay seam formed part of the base of the concave scar, the approximate extent of which is shown in Figures 7 & 8. It comprised a soft yellowish brown clay layer, typically 100 mm thick (but locally up to about 350 mm) with highly decomposed tuff fragments, underlain in places by a thin soft white clay with manganese coating. The clay seam contained slickensiding and black staining. Another soft yellowish brown clay layer was also encountered in borehole BH3 adjacent to the landslide backscarp at a depth of about 7.7 m below ground surface. The slickensiding indicates possible ancient slope movement, although no surface expression of such movement at the location of the clay seam can be seen from aerial photographs.

The landslide backscarp was structurally controlled by a series of variably clay-infilled joints (Figure 8). In the lower planar part of the scar, the landslide surface was in partially weathered tuff with some clay-infilled joints.

Fourteen samples of the yellowish brown clay and the white clay in the landslide area were sent to the British Geological Survey for mineralogical determination by X-ray diffraction. The results of the examination show that both clays contain kaolinite and probably halloysite as well (Merriman & Kemp, 1995). The white clay and the yellowish brown clay are mineralogically similar.

Before the landslide, an area of fill covered the upper part of the hillside. The fill was estimated from aerial photographs to be up to about 5 m thick.

Rock fill, predominantly between 200 mm and 300 mm in size, was found immediately behind the remaining sections of the 2 m high concrete retaining walls below Nam Long Shan Road . The fill material behind the 1.2 m high concrete wall was a loose yellowish brown sandy silt/clay with some gravel.

6.3 <u>Material Properties</u>

Twelve block samples were collected for laboratory testing to determine the

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engineering properties of the materials at the landslide area. The testing was carried out in the Public Works Central Laboratory. This included classification and index tests in accordance with the methods described by Chen (1994), and consolidated undrained triaxial compression tests with porewater pressure measurements and direct shear tests based on the methods of Head (1986) and Head (1982) respectively. The results of the classification and index tests are summarised in Table 1.

The effective shear strength parameters of the completely decomposed tuff obtained from triaxial compression tests (Figure 12(a)) are within the common range of similar material in Hong Kong (Geotechnical Engineering Office, 1993).

Two series of strength tests were carried out for the white clay and the yellowish brown clay. The peak shear strength of the clay given by the triaxial compression test results is shown in Figure 12(b). These results are consistent with the Atterberg Limits of the clay. The results of direct shear tests on a slickensided surface in clay are shown in Figure 12(c).

None of the fill below the collapsed passing bay remained after the landslide. It is therefore not possible to test the original state of the fill. To infer the possible state, insitu tests were carried out on the embankment supporting the passing bay about 60 m north of the landslide area. The results of the density tests, together with laboratory tests on maximum dry density, are summarised in Table 2. It can be seen that the fill generally had a degree of compaction less than 80% of the Standard Proctor maximum dry density, measured in accordance with the procedures of Chen (1994). The current compaction standard for fill embankments requires a degree of compaction of 95% or greater (Geotechnical Control Office, 1984). Therefore, had the two passing bays been constructed at the same time, the fill at the landslide site could have been loose. However, the state of compaction of the fill cannot now be established with certainty.

Results of permeability tests in boreholes carried out by means of falling head tests and packer tests (Geotechnical Control Office, 1987b) are given in Table 3. The test results show that the coefficient of permeability of the partially weathered tuff with very closely-spaced joints was of the order of 10^{-5} m/s. The coefficient of permeability of moderately to slightly decomposed tuff with widely-spaced joints was significantly lower.

6.4 <u>Groundwater Conditions</u>

During the investigation of the landslide, persistent water seepage from the ground was observed at a number of points on the landslide scar (Figures 9 & 11). The elevations of the seepage points at mid-height of the failed hillside (35 to 50 m above Principal Datum) are compatible with the groundwater levels recorded in boreholes nearby and are considered to be a surface expression of the base groundwater level. The groundwater levels at these locations are about 1 to 3 m below the pre-failure ground surface.

The ground investigation also showed that the base groundwater level within the concave scar was about 5 m below the landslide surface. However, seepage was observed from the ground in the concave scar just above the exposed clay seam (Figures 9 & 11). This seepage persisted for about one week after the landslide and reappeared after subsequent rainstorms. This suggests the presence of a transient perched water condition.

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By December 1995, the base groundwater level recorded in boreholes within the concave scar had fallen by about 2 m on average from the level recorded in October 1995. This shows a trend of falling base groundwater level.

7. CONDITION OF DRAINAGE AND WATER-CARRYING SERVICES

During heavy rainfall on the morning of 14 August 1995, a large volume of surface water was seen flowing out of catchpits along Nam Long Shan Road south of the landslide area. The surface water flowed onto the road pavement, down Nam Long Shan Road and into the landslide scar. Based on measured depth, width and velocity of water flow, the rate of the water flow down Nam Long Shan Road was estimated to be about 350 litres per second. Subsequently, the drainage channels and catchpits at Nam Long Shan Road were examined. It was noted that some of the catchpits and cross-road drains along Nam Long Shan Road were partly blocked by old soil, vegetation and refuse.

The 1.2 m wide stepped-channel carrying water down the hillside between Nam Long Shan Road and Shum Wan Road was partly broken and buried by debris after the landslide (Figure 7). The condition of the broken portion of the stepped-channel before the landslide cannot be ascertained. However, a witness who walked part way up the stepped-channel one week before the landslide did not see any blockage of the channel. Some 20 m portion of the stepped-channel adjacent to the concave scar remained intact and clear after the landslide.

The water-carrying services along Nam Long Shan Road in the vicinity of the landslide were examined after the failure. The condition of the remaining sections of the sewer adjacent to the landslide was inspected by means of a closed-circuit television survey. The sewer was found to have cracked at a number of locations, and some joints were found to be displaced (DSD Survey, 1995). Subsequently, a section of the sewer with cracks and displaced joints was exposed by trenching (No. TT9). The sewer was found to comprise 0.7 m long earthenware pipes connected by rigid socket-and-spigot joints infilled with cement mortar. The pipes were haunched in concrete. A little brown staining was observed in the soil around the displaced joints, and there were no signs of erosion of the soil. The cracks in the pipes were tight. These observations indicated that, if the section of sewer within the landslide area was of similar condition, only minor leakage would have occurred from it prior to the landslide.

The private fresh-water main consisted of thread-coupled galvanised iron pipes. It was laid on the ground surface along the downhill side of the Nam Long Shan Road. No evidence of previous leakage from the remaining part of the water main, e.g. staining or erosion of adjacent ground, was observed in the investigation.

8. LIKELY MODE AND SEQUENCE OF FAILURE

The shape of the landslide surface, which comprises an upper concave scar and a lower planar scar, suggests that the failure consisted of two parts: an approximately spoon-shaped slip in the upper part of the hillside and a planar slip in the lower part of the hillside. Crucial to the reconstruction of the mode and sequence of failure are:

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(a) the eye-witness accounts of the landslide (Section 5),

(b) concrete retaining wall fragments in the debris within the planar scar (Figure 9),

(c) fallen trucks, previously parked at the passing bay before the landslide, and pieces of bituminous pavements at the concave scar (Figure 9), and

(d) the ‘slab’ of debris of relatively intact weathered rock mass on the reclaimed land to the west of Shum Wan Road (Section 5).

Based on the available information, the most likely mode and sequence of the landslide have been reconstructed and are illustrated schematically in Figure 13.

The segregation of concrete retaining wall fragments in the landslide debris from other man-made objects from Nam Long Shan Road, including the fallen trucks and pieces of bituminous pavement and sewer (Figure 9), is best explained by a minor failure of the fill embankment below the passing bay before the main landslide (Figure 13(a)). The minor failure could either have been a shallow slip in the fill or erosion of the fill surface. Although signs of such erosion can be seen on the surface of the fill embankment in aerial photographs taken in 1988 and 1991, a shallow slip might also have been possible, However, sufficient evidence could not be found to establish the nature of the failure. The intense rainfall from 11 p.m. on 12 August to 3 a.m. on 13 August triggered this first failure.

This first failure would have displaced the concrete upstand at the downhill side of the passing bay but probably did not involve the southern concrete retaining wall. However, once the concrete upstand had been displaced, a large amount of surface water would have flowed from Nam Long Shan Road onto the fill embankment, scouring and infiltrating the fill and developing a perched water level in it, particularly in the rock fill (Section 6.2) behind the southern concrete retaining wall. This would have caused the collapse of the wall and the material behind it, probably in the form of a rapid flow. In the process, fragments of the southern concrete retaining wall would have been transported some distance down the hillside.

The main landslide occurred at about 4 a.m. The upper part of the hillside slipped partly on a clay seam in the partially weathered tuff. This failure was approximately spoon-shaped and took with it the passing bay (Figure 13(b)). Movement of the ground associated with this failure was reported by an eye-witness to be not rapid but continuous. It resulted in the deposition of pieces of bituminous pavement and utility pipes on the concave scar. Fragments of the southern concrete retaining wall that were deposited earlier on the upper part of the hillside would have moved with the debris further downhill.

The debris from this spoon-shaped failure loaded the lower part of the hillside and disturbed its equilibrium. The lower part of the hillside failed along a shallow planar surface sub-parallel to the ground surface (Figure 13(c)). This planar failure released a relatively thin layer of partially weathered tuff that slipped largely as an intact unit or ‘slab’ down the hill. The ‘slab’ pushed vegetation and top soil in front of it. Some of the pushed material

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and the front part of the 'slab' were deposited at the toe of the hillside (Figure 13(d)). The remaining portion of the 'slab' then overrode the deposited material and continued its journey towards the sea. The front of this 'slab' finally came to rest at a maximum distance of about 70 m from the toe of the hillside (Figure 13(e)).

A substantial part of the failed ground from the spoon-shaped slip in the upper part of the hillside was deposited on the planar scar in the lower part of the hillside. The southern concrete retaining wall fragments moved with the ground to rest on the planar scar some distance below the concave scar in the lower part of the hillside.

According to eye-witness accounts, Nam Long Shan Road did not collapse in the main landslide. The main landslide was followed by a few small slips cutting retrogressively up the hillside. A notable slip occurred about half an hour after the main event, resulting in the collapse of Nam Long Shan Road and deposition of debris onto the concave scar. The large zone of rock debris that rested on top of the remains of Nam Long Shan Road on the concave scar would have come from a later slip (Figure 13(f)).

After the main landslide at about 4 a.m., water from the broken water-carrying services and Nam Long Shan Road flowed down the landslide area. It eroded channels on the landslide and resulted in deposition of debris on the level ground at the foot of the hillside.

The above sequence of events is consistent with eye-witness accounts, the observed depositional sequence and characteristics of debris, and the estimated volume of material released in the landslide.

Other possible alternatives have also been considered, including initiation of the main landslide by the lower planar slip and occurrence of the upper spoon-shaped slip some time before the main landslide at 4 a.m. These alternatives are discounted because they would not match the eye-witness accounts and physical evidence from the locations of man-made objects.

9. THEORETICAL STABILITY AND SEEPAGE ANALYSES

9.1 General

To check that the mechanism proposed in Section 8 is theoretically admissible, two sets of limit equilibrium slope stability analyses, one for the spoon-shaped slip in the upper part of the hillside and the other for the planar slip in the lower part of the hillside, were carried out. The representative cross-sections for the analyses are shown in Figure 14. Limit equilibrium slope stability analyses were also carried out to assess the stability of the fill embankment below the collapsed passing bay. In addition, a set of seepage analyses were conducted to examine the effects of various water sources on the groundwater conditions at the upper part of the hillside.

9.2 Upper Part of Hillside

Theoretical analysis of the fill embankment by the method of slices, using the rigorous solution given by Morgenstern & Price (1965), confirms that a shallow slip is possible when

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the fill is saturated by water.

In the concave scar area, the post-failure base groundwater level was well below the landslide surface (Section 6.4). Theoretical analyses by the method of slices show that this part of the hillside would be stable under such a deep base groundwater condition. Perched water, however, was observed on the clay seam which formed part of the landslide surface of the concave scar (Section 6.2). The associated porewater pressure could have rendered the ground unstable. The theoretical perched water level needed for limit equilibrium depends on the shear strength of the clay seam. The ground could theoretically have failed under a high perched water level of 4 to 5 m coupled with the peak shear strength for the clay seam, or alternatively under a low perched water level of 1 to 2 m if the clay seam had a shear strength close to that of the clay with slickensiding (Figure 15(a)).

The four sources of perched water that can be postulated are:

(a) direct infiltration of incident rainfall into the ground in the hillside below Nam Long Shan Road,

(b) subsurface water flow from the natural ground above Nam Long Shan Road,

(c) water discharged from Nam Long Shan Road, and

(d) leakage from water-carrying services along Nam Long Shan Road.

About 380 mm of rain had fallen directly on the landslide area during the 30 hours prior to the main landslide. Water could have infiltrated the fill embankment below the passing bay and seep through the underlying partially weathered tuff. Seepage analysis using a finite element computer program (Figure 16) shows that this incident rainwater alone could have caused a low head (about 1 m) of perched water to build up on the clay seam.

Subsurface water flow associated with the rainstorm in early August could have reached the landslide area from the natural ground further uphill by 13 August. Seepage analysis using the finite element program suggests that such recharge would have contributed largely to the base groundwater but little to the perched water level on the clay seam at the site. The level of perched water that could have built up cannot be assessed reliably, but the result suggests that it is likely to be low (about 1 m).

The above theoretical assessments suggest that rainfall infiltration alone would not have resulted in a significant perched water level. Theoretical calculations also show that any minor leakage from the sewer or the water main could not have caused the build up of a significant perched water level.

It is estimated that, upon displacement of the concrete upstand at the passing bay by the minor failure of the fill embankment, hundreds of cubic metres of water would have discharged from Nam Long Shan Road at the landslide location in one hour. This amount of water far exceeded the amount of rain falling directly on the fill embankment. Some of the water would have infiltrated the relatively permeable partially weathered tuff. Seepage

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analyses show that a high perched water could have developed, the height of which depends on the duration of the water flow. For example, a perched water level of about 5 m could theoretically have developed with three hours of water flow from Nam Long Shan Road.

9.3 Lower Part of Hillside

At the location of the planar scar, ground investigation showed that the base groundwater level was closer to the pre-failure ground surface than in the concave scar. Theoretical stability analyses by the limit equilibrium method, using the infinite slope solution, show that the ground could theoretically have become unstable under a range of base groundwater levels between zero and 1 m below ground, depending on the proportion of clay-infilled joints along the landslide surface (Figure 15(b)).

The failure mass from the upper spoon-shaped slip would have loaded the lower part of the hillside and reduced its factor of safety.

10. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

It is known from the location of debris from the retaining wall that the main landslide at about 4 a.m. was preceded by a minor failure at the fill embankment below the passing bay. Results of stability analyses show that a shallow slip could have occurred in the fill embankment when the fill was saturated by water. The minor failure could also be the result of erosion of the fill surface. However, there was insufficient evidence to establish which mode of failure was more likely. In either case, the water that triggered the minor failure could have been rainwater falling directly on the fill or water spilling from Nam Long Shan Road, or a combination of both.

The main landslide comprised two parts, an upper spoon-shaped slip and a lower planar slip. The upper spoon-shaped slip was triggered by perched water pressure on a weak clay seam. At this location, the base groundwater level was well below the landslide surface (Section 6.4). Field monitoring of groundwater pressures and theoretical analyses do not support a hypothesis that the slip was caused by a significant rise (more than 6 m) of the base groundwater level. The clay seam was much weaker than the adjacent partially weathered tuff, and it formed a weak plane for a substantial part of the base of the upper spoon-shaped slip.

From the results of theoretical analyses, the upper spoon-shaped slip could have resulted from a high perched water level (4 to 5 m) coupled with an operational shear strength of the clay without slickensiding, i.e. $c' = 8$ kPa, $\phi' = 26°$. A high perched water level was possible based on seepage analyses (Section 9.2), given the large amount of water that would have discharged from Nam Long Shan Road after the minor failure of the fill embankment (Section 8) and the measured permeability of the partially weathered tuff with closely-spaced joints (Table 3). The high perched water hypothesis is therefore credible.

Slickensiding was observed in a small area in the clay seam (Section 6.2). The shear strength of the slickensided clay was found to be low ($\phi' = 21°$). Had the slickensiding been extensive, a relatively low perched water level could have triggered the landslide (1 to 2 m for

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$\phi' = 21°$). Such a low perched water level could be developed by direct rainfall infiltration and subsurface water recharge from the natural ground uphill. A similar infiltration condition is likely to have occurred regularly in the past, and the low perched water hypothesis could therefore not explain why the hillside did not fail in past rainstorms. This hypothesis also suggests that the fill embankment failure would have been immaterial to the landslide, and that the timing of the fill embankment failure and the main landslide were coincidental. The high perched water hypothesis is much more likely than the low perched water hypothesis.

The base groundwater level at the location of the lower planar slip was high, attributed to the prolonged heavy rainfall in July and early August, and it was about 1 to 3 m below the pre-failure ground surface in October 1995. From the falling trend in the period of October to December 1995 (Section 6.4), the base groundwater level in August 1995 would have been higher, especially during the intense rainfall in the 30 hours before the landslide when the water level could have been very close to the pre-failure ground surface. Theoretical stability analyses show that the ‘slab’ of partially weathered tuff in this area could have failed, partly along clay-filled joints, under such groundwater conditions (Section 9.3). The loading associated with the debris from the spoon-shaped slip above would have triggered the failure.

11. OTHER CONCEIVABLE FACTORS

After the landslide, witnesses reported observations of factors which might have contributed to the landslide. These include previous squatter activities, illegal dumping and passage of heavy construction vehicles. The effects of these factors have been examined in the investigation and were found to be not significant, as discussed below.

Previous squatter activities in the area of the planar slip had modified the landform, e.g. cutting of the hillside for squatter platforms. This could have affected the stability of the ‘slab’ of partially weathered tuff by weakening its toe support in the lower part of the hillside. However, this is insignificant compared to the resistance against sliding provided by the base of the ‘slab’.

Illegal dumping of refuse and construction waste on the hillside downslope of Nam Long Shan Road might have resulted in ponding of water and might have promoted infiltration. However, the rainfall prior to the minor failure of the fill embankment was not heavy enough to exceed the rate of infiltration, and rainwater would therefore have infiltrated the ground without much surface runoff and ponding. After the failure of the fill embankment, the large amount of water from Nam Long Shan Road would have allowed continuous infiltration even without ponding. Illegal dumping would also have imposed an additional surcharge, but this would have been insignificant to the stability of the large hillside and is therefore not considered to have been a contributory factor.

The passage of heavy construction vehicles on Nam Long Shan Road would have imposed a surcharge on the road embankment and might have damaged the concrete upstand at the edge of the passing bay. The surcharge effect was negligible compared to the mass of the soil. It is not known whether the concrete upstand had been damaged by vehicles but, if it had, water from Nam Long Shan Road would readily have discharged onto the fill embankment. It is not possible to obtain evidence on the state of the concrete upstand

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immediately before the landslide to judge whether the passage of heavy construction vehicles contributed to the failure in this manner.

12. CONCLUSIONS

The main landslide involved two distinct parts that occurred almost simultaneously. The failure was caused principally by:

(a) the presence of weak layers in the ground, i.e. clay seams and clay-infilled joints,

(b) ingress of water during prolonged heavy rainfall,

(c) a minor failure of the fill embankment below a passing bay on Nam Long Shan Road, and

(d) water flowing along Nam Long Shan Road, because of partial blockage of its drainage system, and discharge of part of this water onto the hillside.

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Table 1 - Summary of Classification and Index Test Results

Material Type	Sample Location	Depth (m)	Sample Type	Particle Size Distribution				LL (%)	PL (%)	PI (%)	Moisture Content (%)	Specific Gravity
				Gravel (%)	Sand (%)	Silt (%)	Clay (%)					
CDT	S2	Near ground surface	Block	1	17	44	38	67	30	37	29.4 29.2 29.6 30.6	2.64
CDT	S4	Near ground surface	Block	0	8	64	28	48	33	15	30.3 38.6 38.4	2.64
CDT	S5	Near ground surface	Block	1	17	63	19	48	28	20	30.0 35.2 33.4	2.63
CDT	BH3	1.78 - 1.98	Mazier	2	34	42	22	-	-	-	27.4	2.62
Yellowish brown clay	S8	Near ground surface	Block	0	8	28	64	99	39	60	45.6 53.3 52.3	2.63
Yellowish brown clay	S9	Near ground surface	Block	0	12	48	40	123	48	75	63.0	2.62
Fill	TP10	0 - 1 m	Bulk	6	26	54	14	35	21	14	-	-
Fill	TP11	0 - 1 m	Bulk	21	17	42	20	39	22	17	-	-
Fill	TP12	0 - 1 m	Bulk	37	13	38	12	35	21	14	-	-
Fill	TP13	0 - 1 m	Bulk	5	17	49	29	57	24	33	-	-
Fill	TP14	0 - 0.5 m	Bulk	6	29	42	23	41	23	18	-	-

Legend:

CDT	Completely Decomposed Tuff	LL	Liquid Limit	PL	Plastic Limit
PI	Plasticity Index	BH3	Borehole No.3	S2	Block sample No.2
TP10	Trial pit No.10				

Note: Tests were carried out in accordance with Chen (1994).

Table 2 - Results of Density Tests on Fill Material

Trial Pit	Depth (m)	Materials	Insitu Dry Density (Mg/m^3)	Insitu Moisture Content (%)	Laboratory Maximum Dry Density (Mg/m^3)	Optimum Moisture Content (%)	Relative Degree of Compaction (%)
TP10	0	Yellowish brown gravelly sandy silt/clay	1.26	22	1.75	18	72.0
	0.5		1.30	21			74.3
	1.0		1.34	21			76.6
TP11	0	Yellowish brown slightly gravelly sandy silt/clay	1.25	24			71.4
	0.5		1.29	24			73.7
	1.0		1.27	25			72.6
TP12	0	Yellowish brown gravelly sandy silt/clay	1.39	20			79.4
	0.5		1.33	23			76.0
	1.0		1.25	24			71.4
TP14	0.5	Yellowish brown gravelly sandy silt/clay	1.18	15	1.65	21	71.5
	0.5		1.35	14			81.8
Note:	See Table 1 for classification and index properties of fill.						

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Table 3 - Results of Permeability Tests

Material	Type of Test	Coefficient of Permeability (m/s)
Completely to highly decomposed tuff	Falling head test in boreholes	1.3×10^{-5} to 8.0×10^{-5}
Moderately to slightly decomposed tuff with closely spaced joints	Packer test and falling head test in boreholes	1.2×10^{-5} to 6.4×10^{-5}
Slightly decomposed tuff with closely spaced joints	Packer test in boreholes	1.3×10^{-5} to 1.8×10^{-5}
Slightly decomposed tuff	Packer test in boreholes	6.6×10^{-9} to 4.3×10^{-7}

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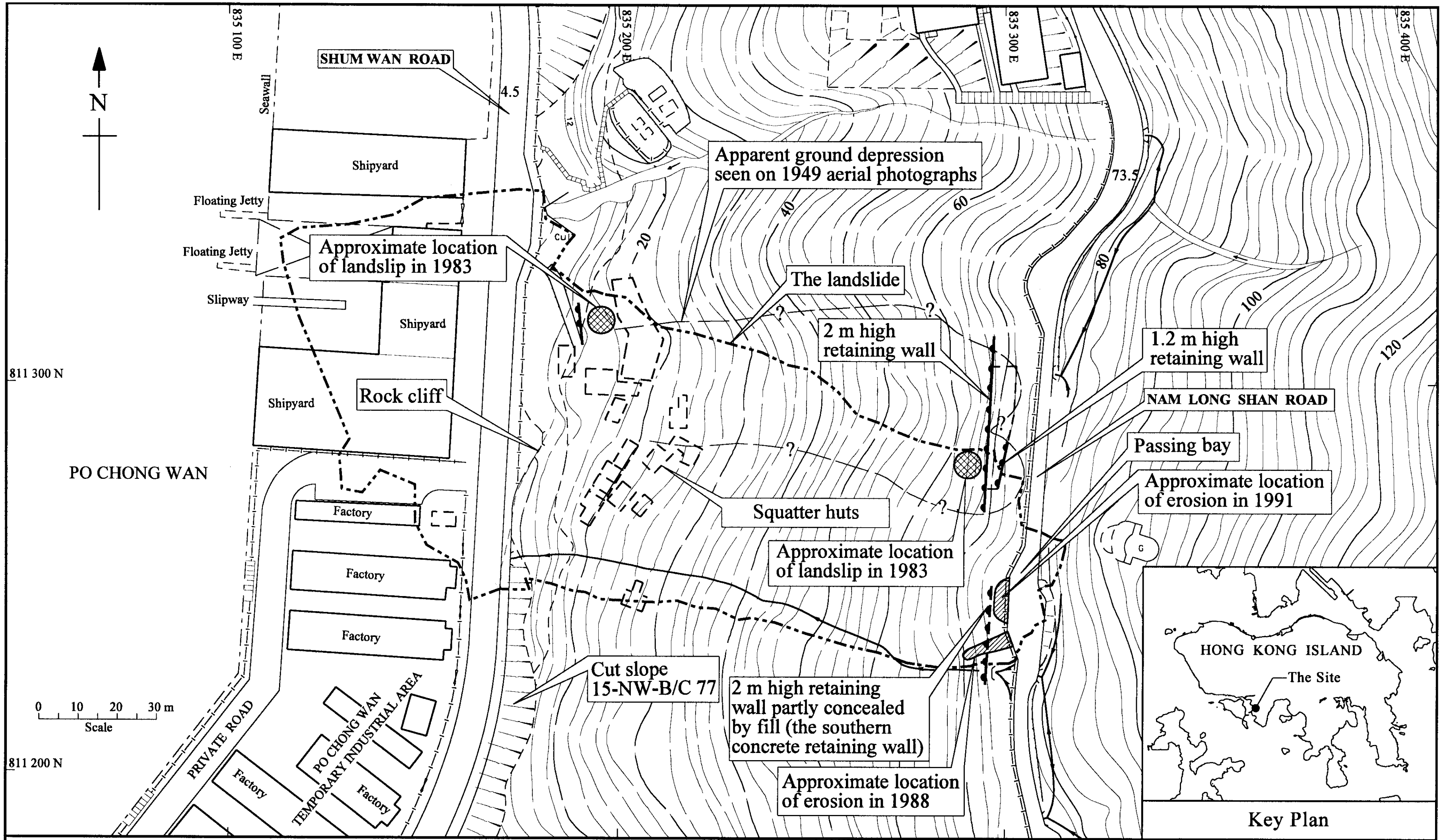

Notes : (1) Base map is extracted from Survey Sheets No. 15-NW-4C & No. 15-NW-4D, dated August 1992 & September 1992 respectively, scale 1 : 1000.
(2) Locations of squatter huts are based on Survey Sheet No. 15-NW-4C, dated October 1984, scale 1 : 1000.

Figure 1 - Site Plan

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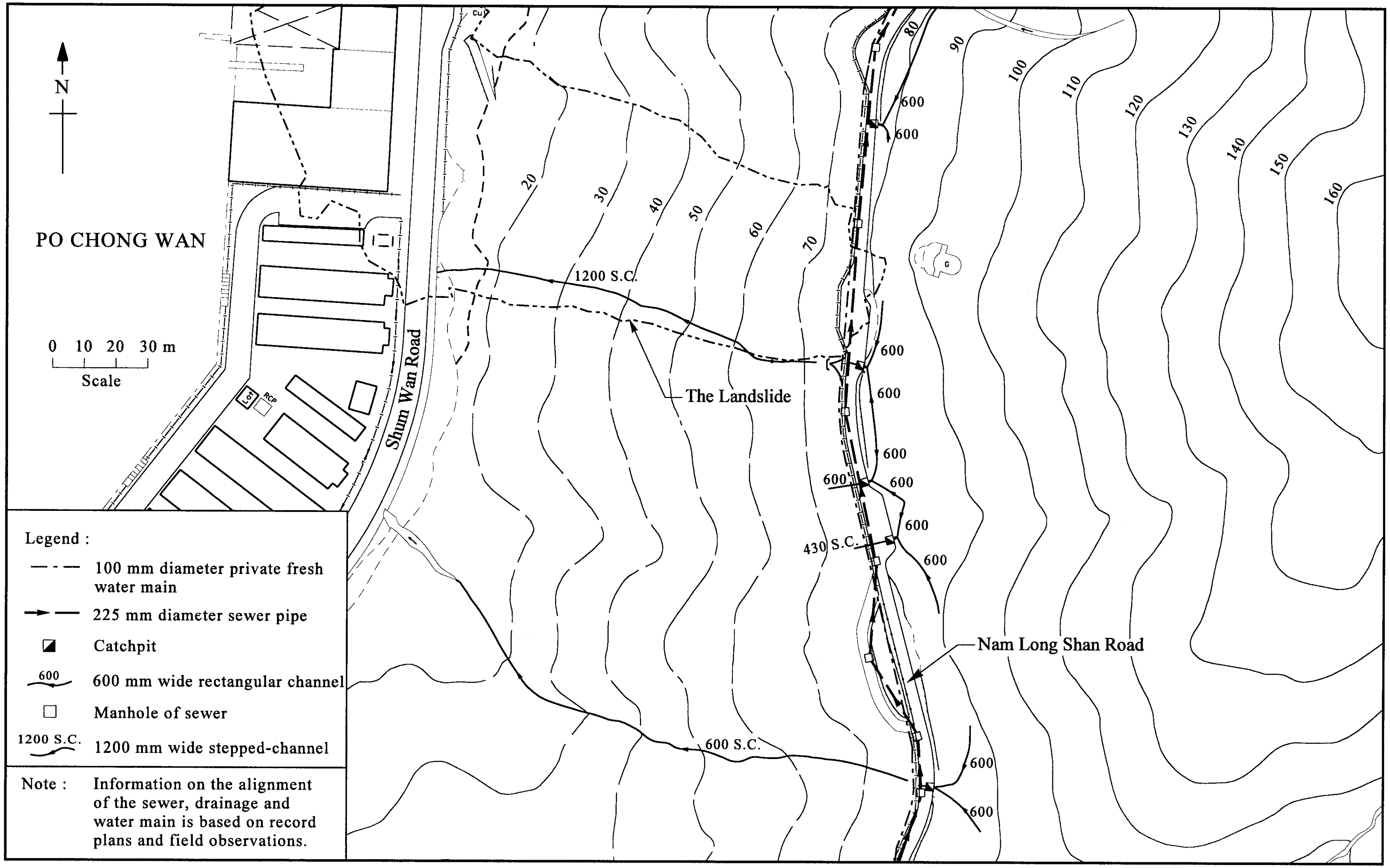

Figure 2 - Layout of Drainage and Water-carrying Services at Nam Long Shan Road

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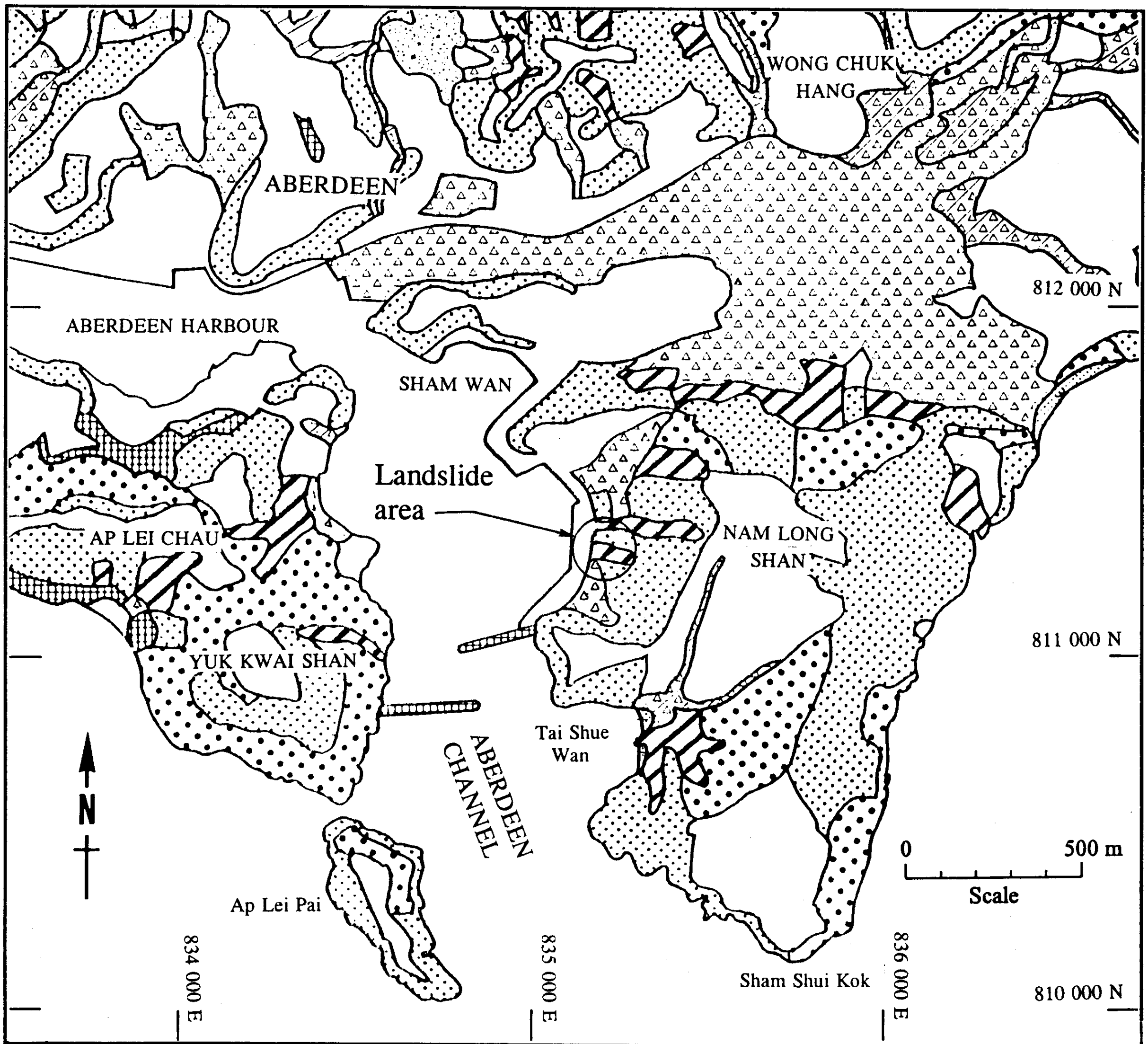

Legend :

- Colluvium
- Zones of colluvium which are subject to overland flow and periodic inundation. Evidence of unusual groundwater regime (delineated as drainage plain on Landform Map)
- Floodplain - subject to overland flow and regular inundation. Evidence of unusual groundwater regime (delineated as floodplain on Landform Map)
- Zones of general instability associated with predominantly colluvial terrain
- Zones of general instability associated with predominantly insitu terrain
- Slopes on insitu terrain which are generally steeper than 30 degrees (other than those delineated as colluvial or unstable)

Notes :

(1) Figure reproduced from map GASP/20/1/6, scale 1 : 20000, produced in the Geotechnical Area Studies Programme (Geotechnical Control Office, 1987a).

(2) The Physical Constraints Map should be used only as a guide to the general nature of terrain-related constraints for regional planning purposes. It is produced at a scale of 1:20 000 and should not be used to evaluate parcels of land smaller than 3 ha in size.

Figure 3 - Physical Constraint Map of the Landslide Area

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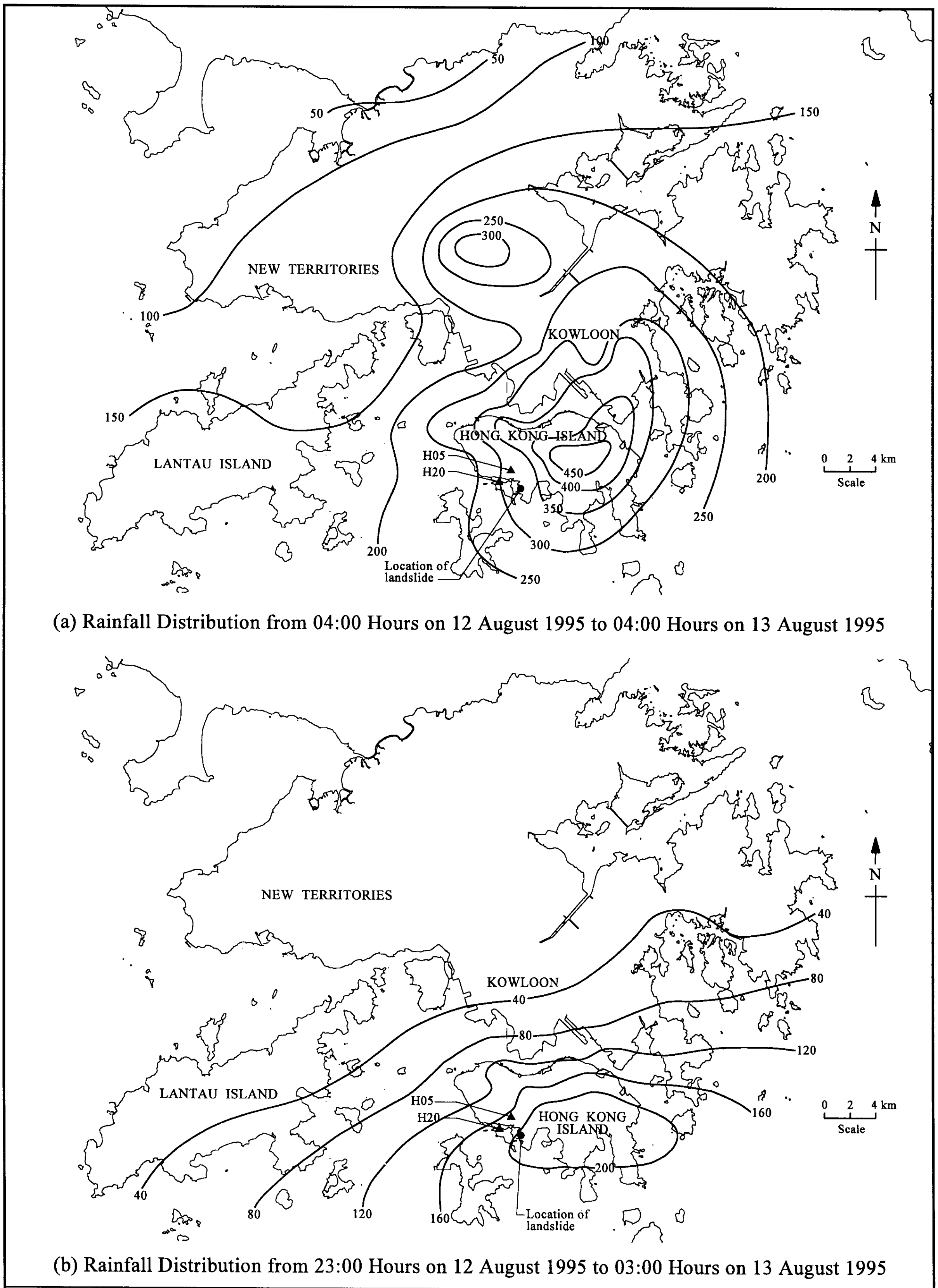

Legend :

–200– Isohyet of rainfall in millimetres

▲ GEO Raingauge

Figure 4 - Rainfall Distribution Prior to the Landslide

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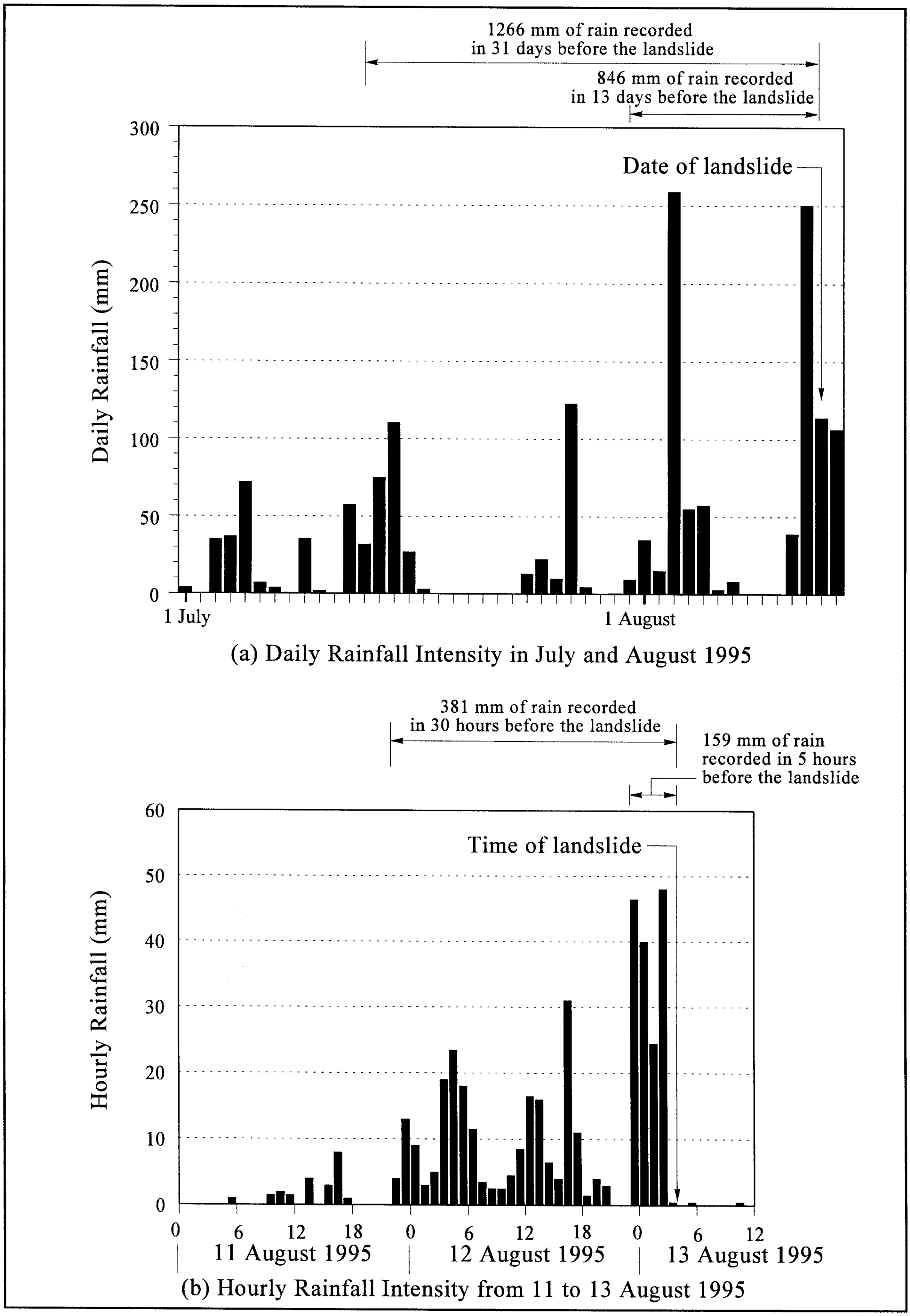

Figure 5 - Rainfall Record of GEO Raingauge No.H05

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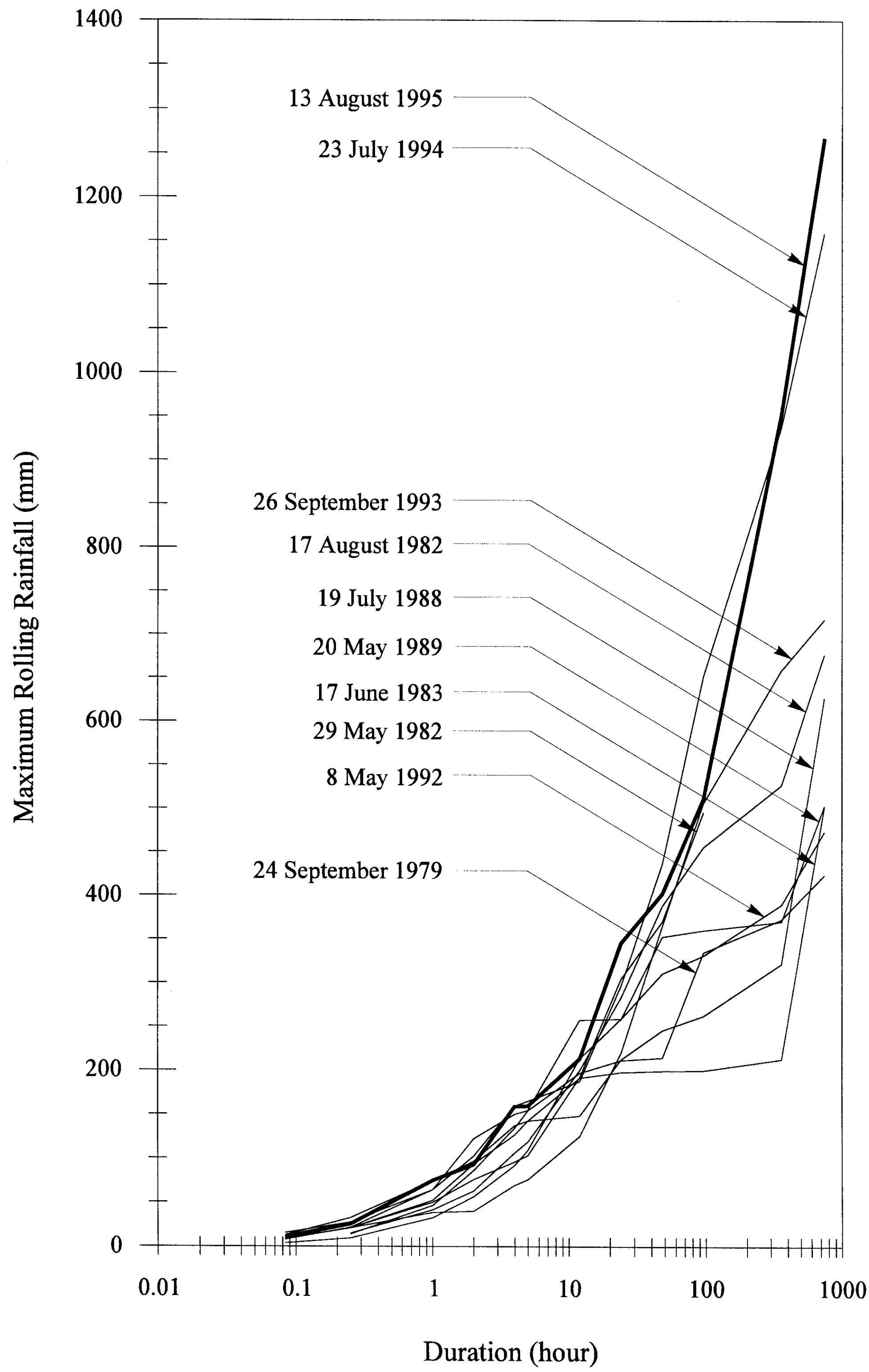

Figure 6 - Maximum Rolling Rainfalls at Raingauge No. H05 for Major Rainstorms

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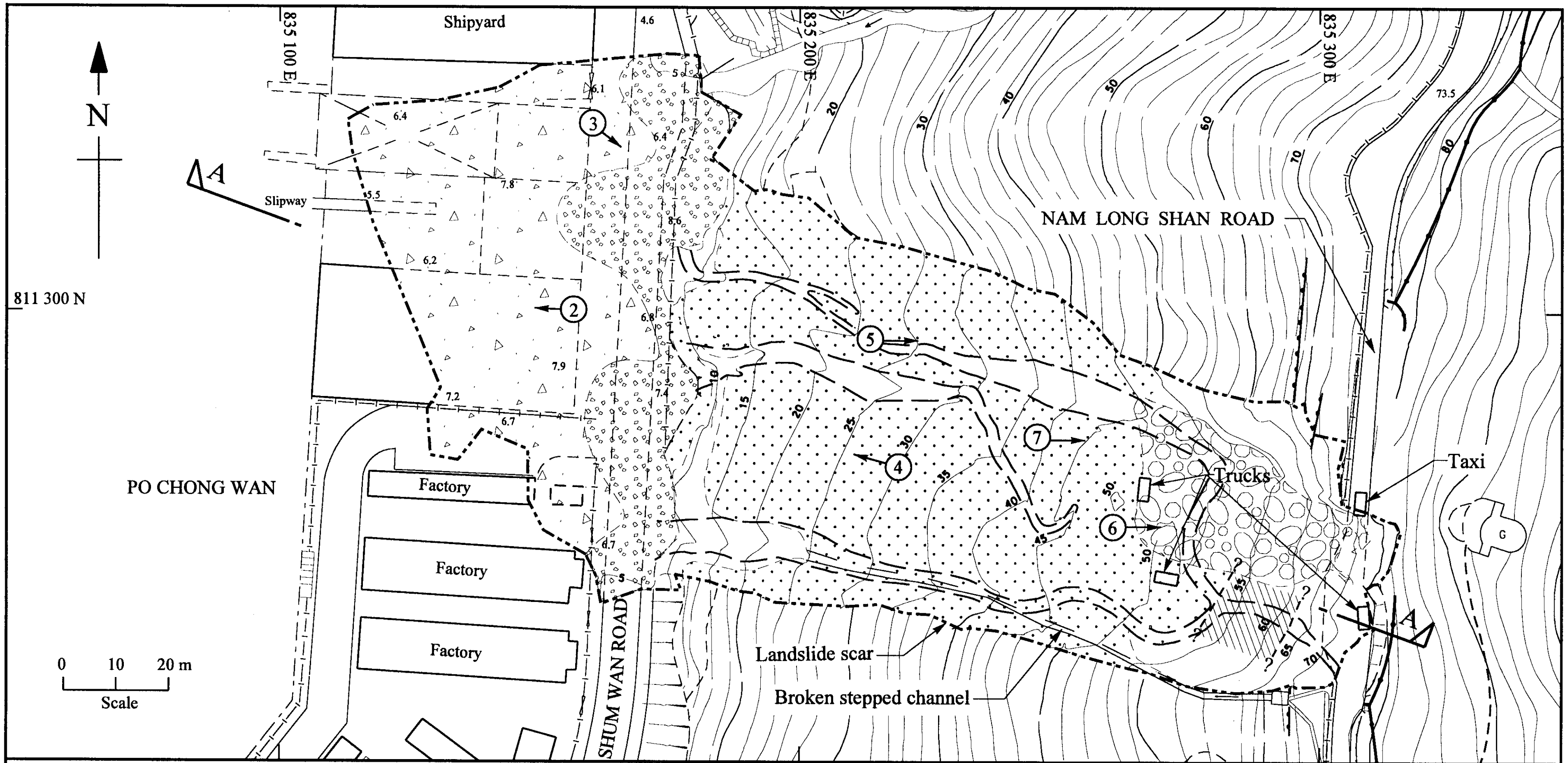
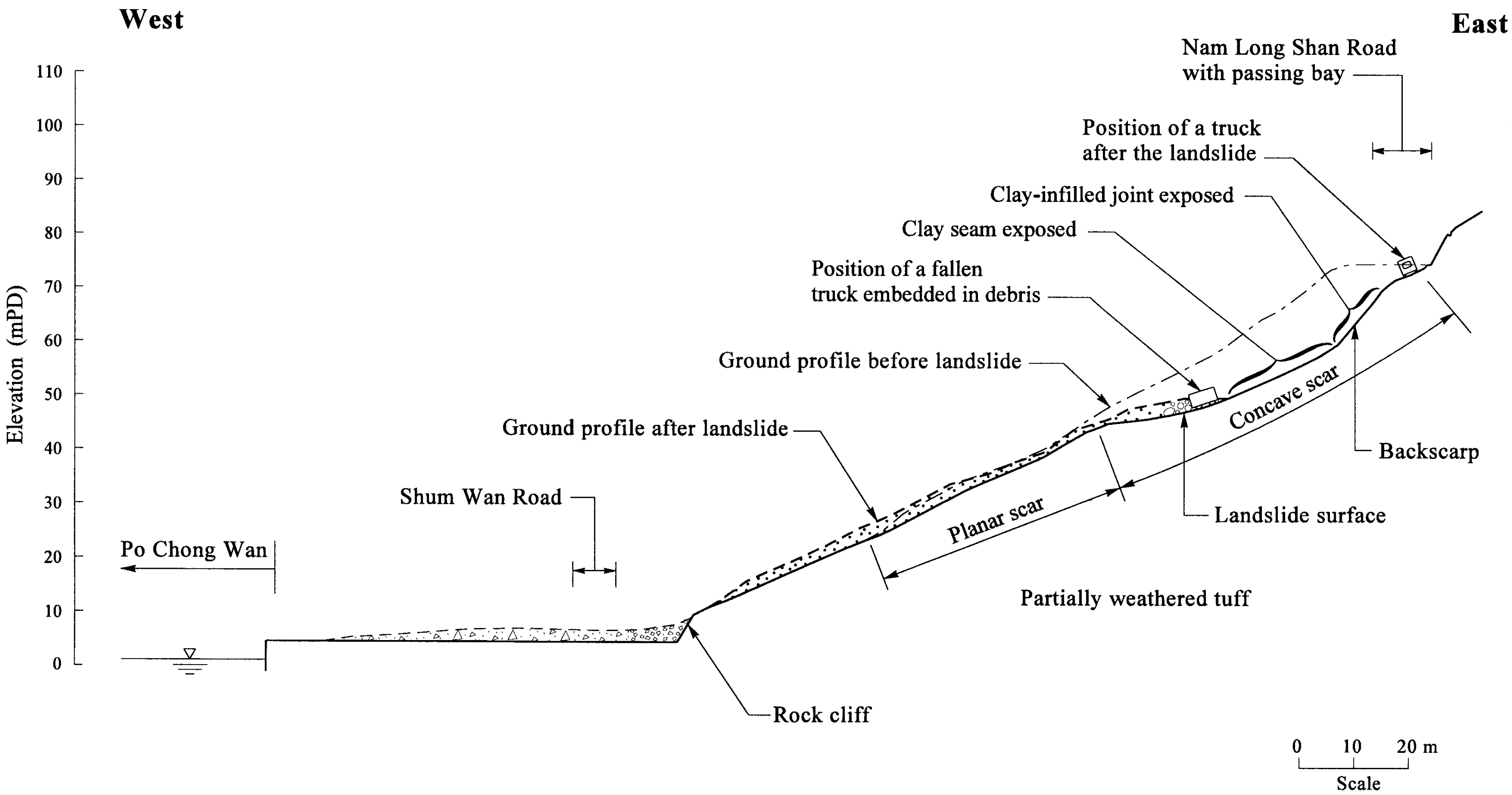


Legend :

35	Contour in mPD	6.7	Spot level in mPD		Retaining wall		Erosion channel
	Debris comprising a 'slab' of soil and rock		Debris containing fluvial deposit		Debris containing fill material		Rock debris
←②	Vantage point of Plate 2		Approximate extent of clay seam exposed				

Notes :
(1) See Figure 8 for Section A-A.
(2) Information shown in this Figure is based on topographic survey, geological mapping and field observations carried out between 13 August and 23 November 1995.

Figure 7 - Plan of the Landslide



Notes : (1) See Figure 7 for location of section and legend of debris.
(2) Information shown in this Figure is based on topographic survey, geological mapping and field observations.

Figure 8 - Section A-A through the Landslide

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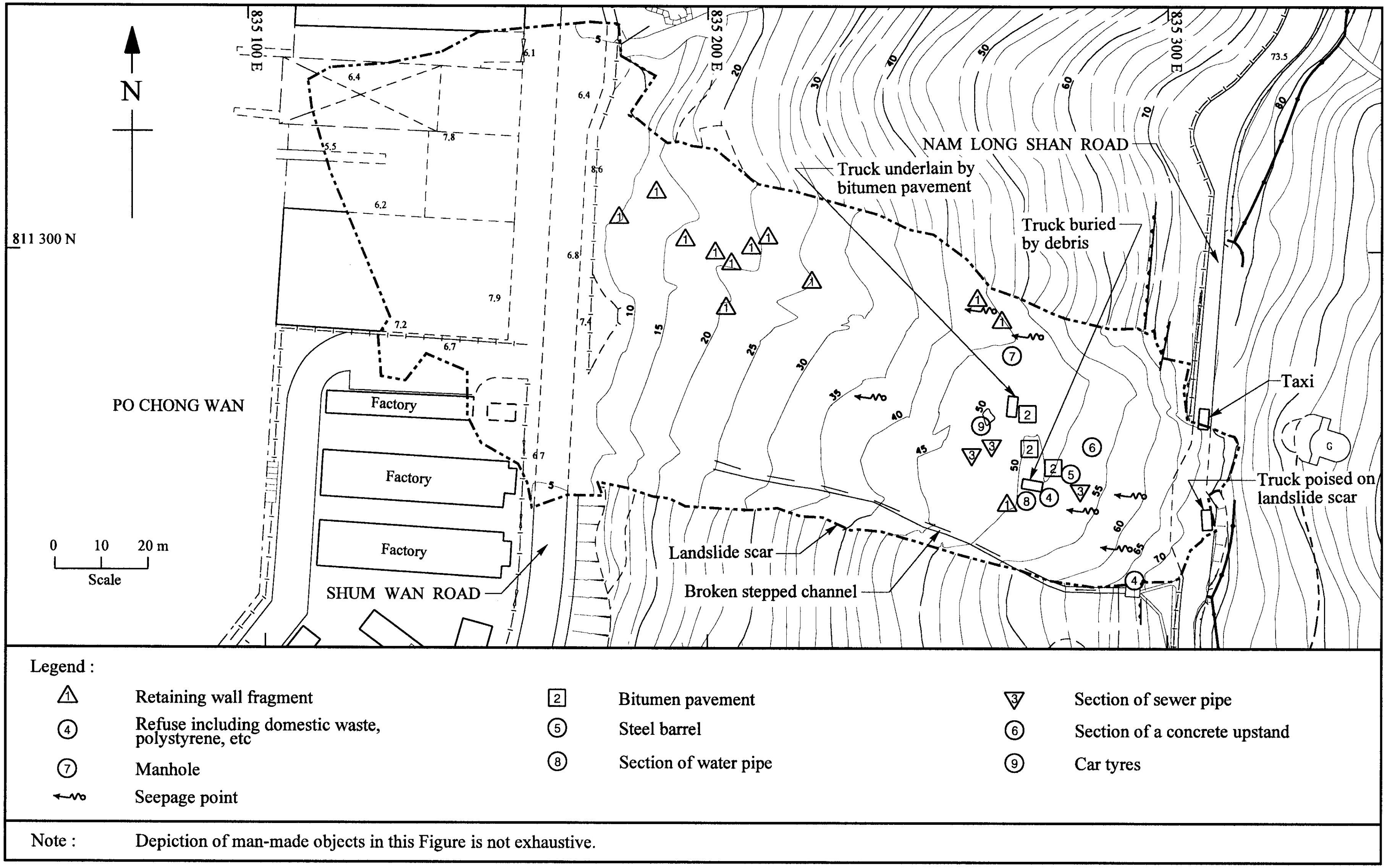

Figure 9 - Locations of Man-made Objects and Seepage Points

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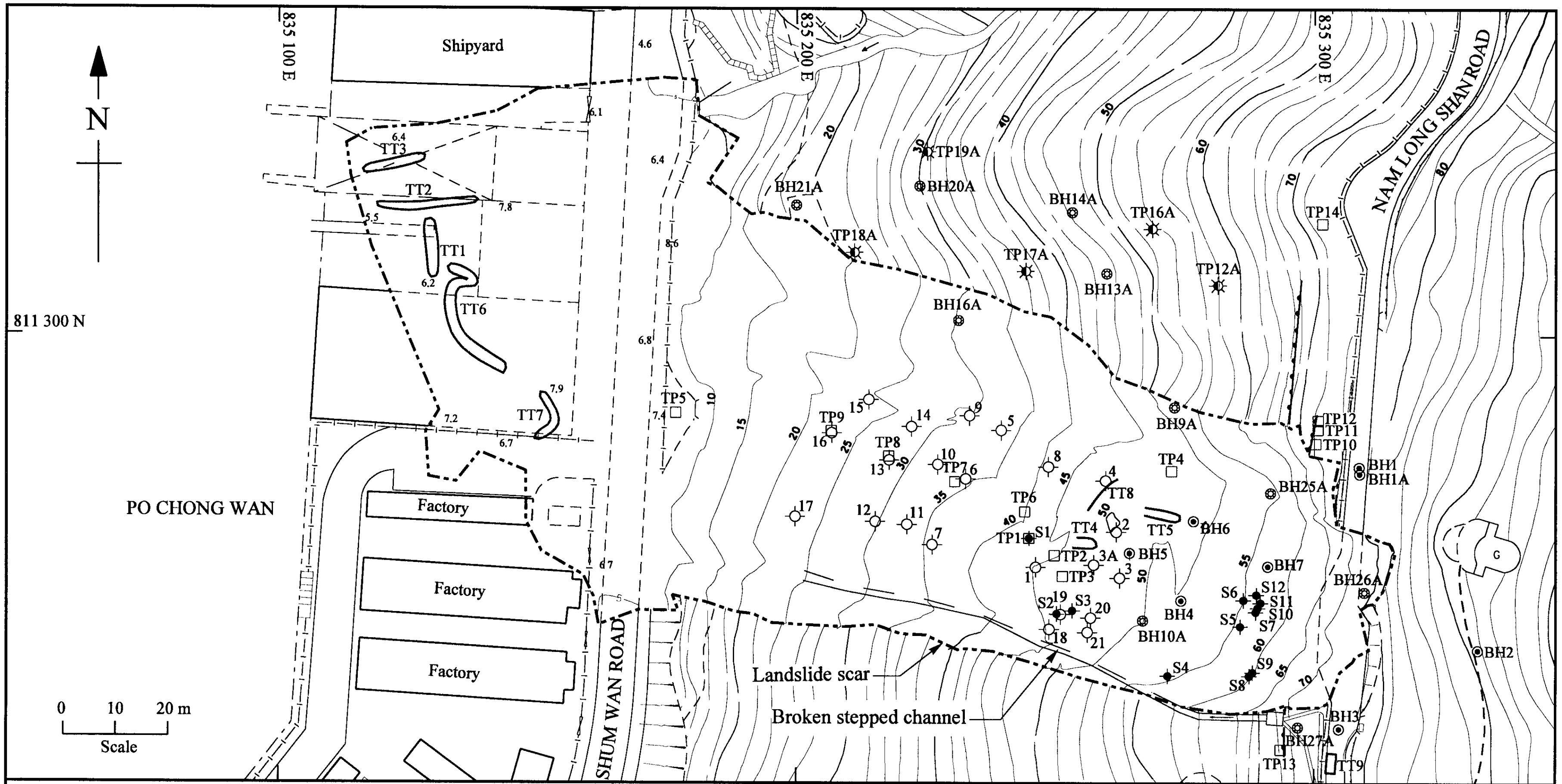


Legend :

35	Contour in mPD	BH1	Borehole	S3	Block sample
TT5	Trial trench	BH16A	Borehole (for remedial work design)	5	GEO probe test
6.7	Spot level	TP12A	Trial pit (for remedial work design)	TP2	Trial pit

Note : Seismic lines have not been shown for clarity.

Figure 10 - Location Plan of Ground Investigation Work

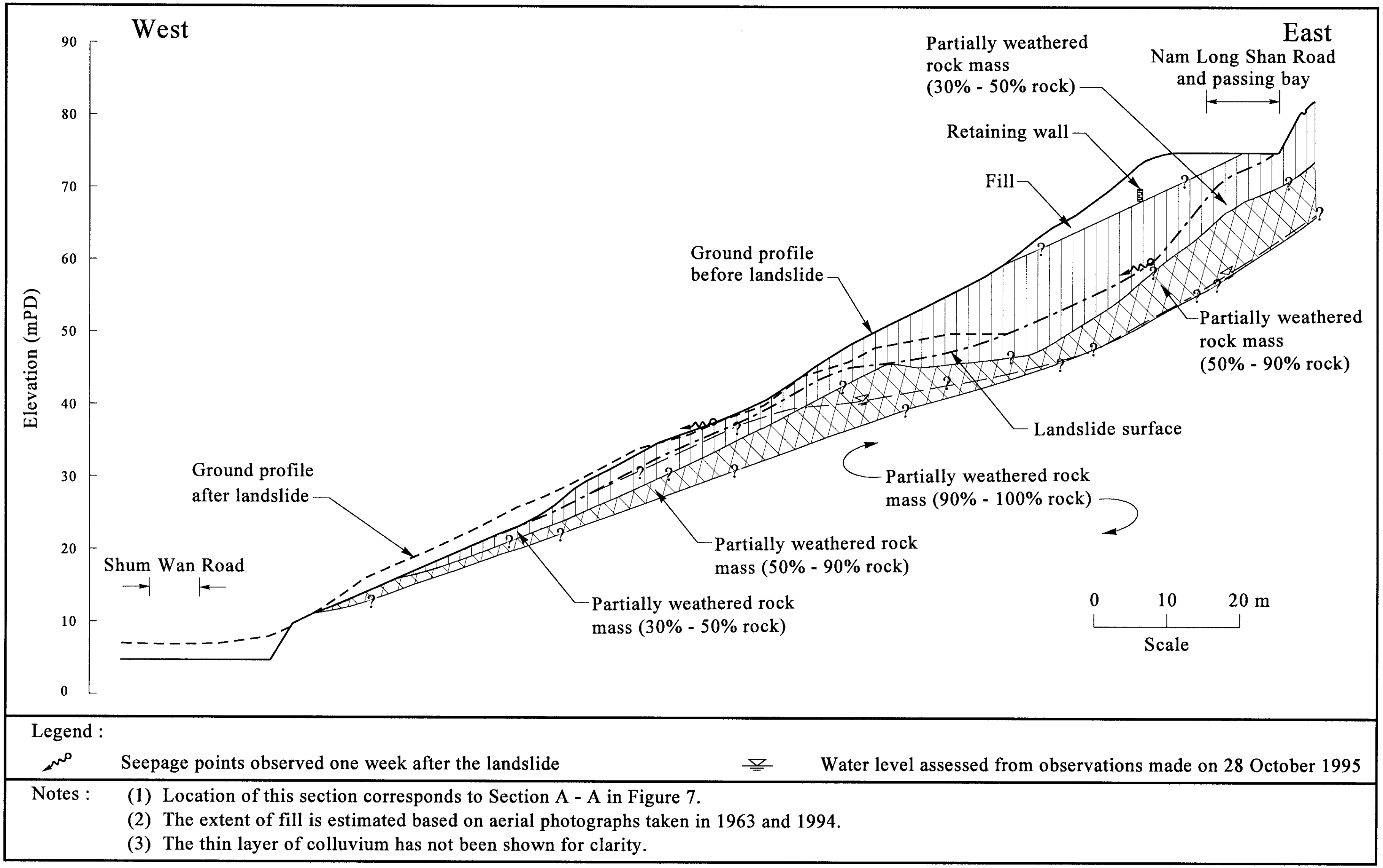
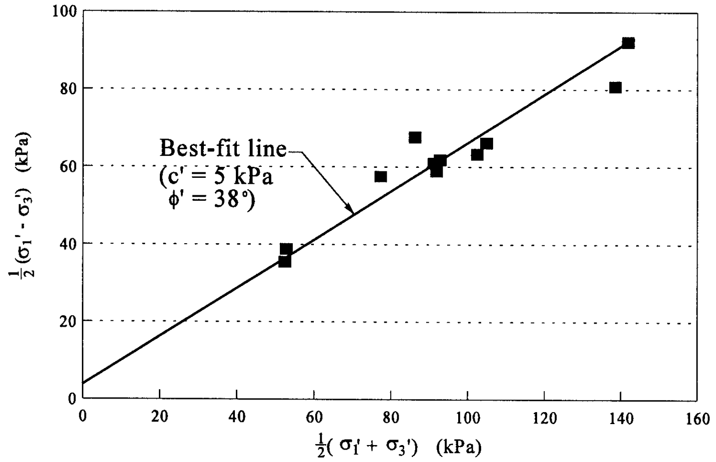
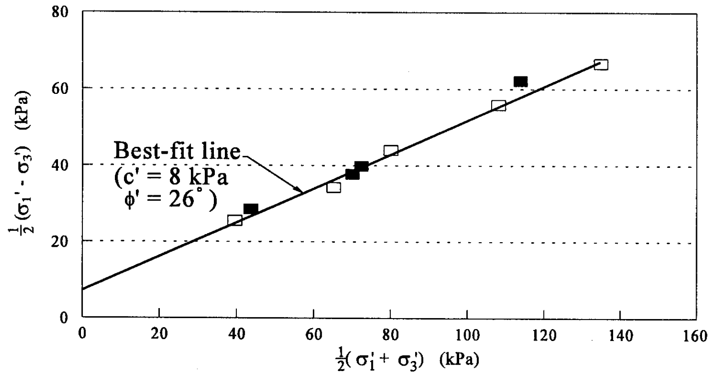


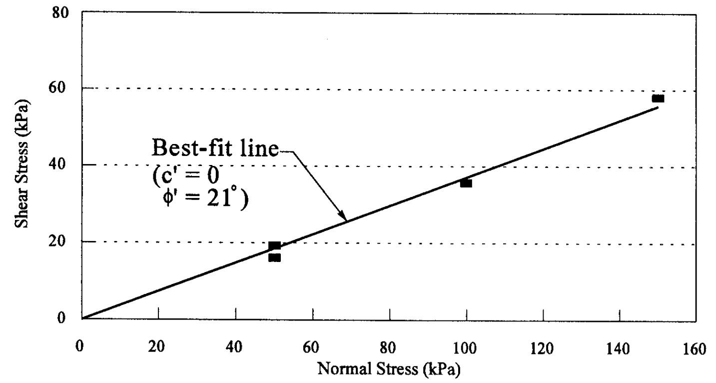
Figure 11 - Geological Section of the Site before the Landslide



(a) Results of Isotropically Consolidated Undrained Triaxial Compression Test with Porewater Pressure Measurement for Completely Decomposed Tuff



(b) Results of Isotropically Consolidated Undrained Triaxial Compression Test with Porewater Pressure Measurement for Yellowish Brown Clay in the Clay Seam



(c) Direct Shear Test Results for Slickensided Surface of Clay in the Clay Seam

Legend :

σ_1'	Major principal effective stress	■	'Undisturbed' block sample
σ_3'	Minor principal effective stress	□	Remoulded sample
c'	Apparent cohesion	ϕ'	Angle of shearing resistance

Note : The data points in Figures (a) & (b) are taken from test results at peak σ_1'/σ_3' ratio.
The data points in Figure (c) are taken from test results at the end of testing.

Figure 12 - Shear Strength of Materials

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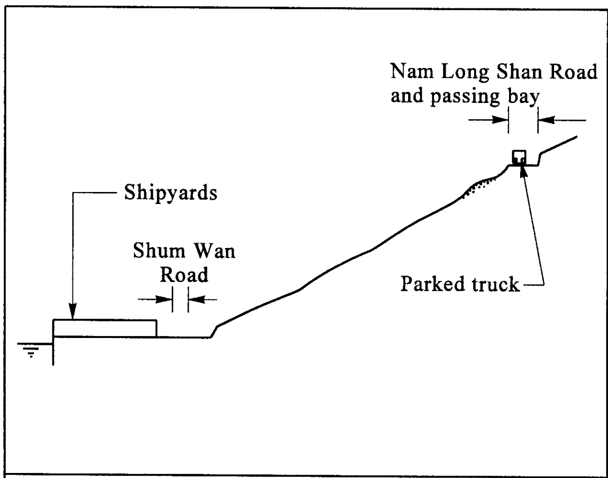
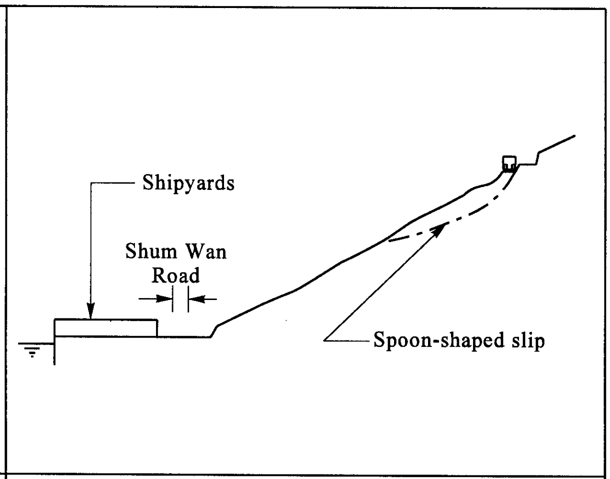
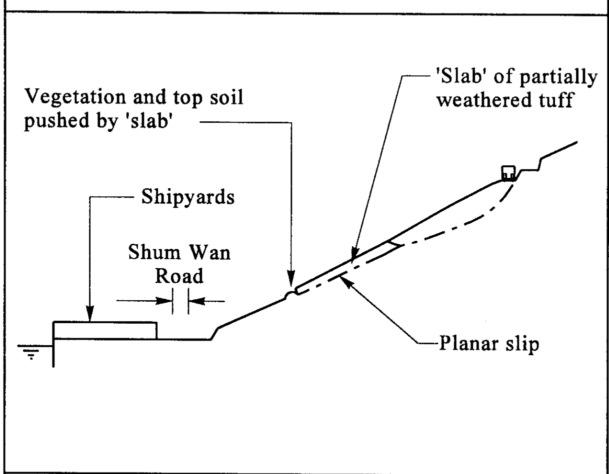


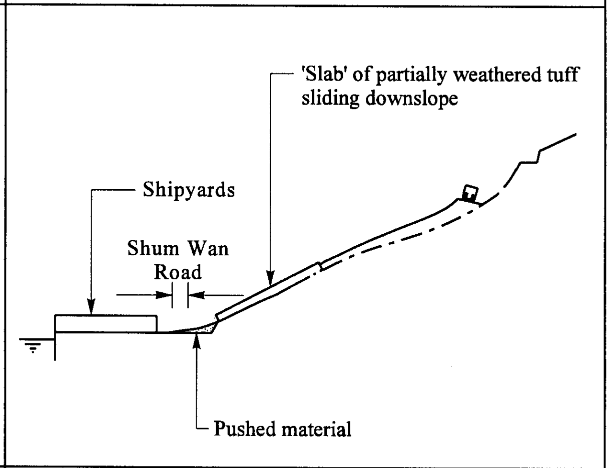
(a) Minor failure occurred at the fill embankment below the passing bay prior to the main landslide (between 11 p.m. and 3 a.m.).



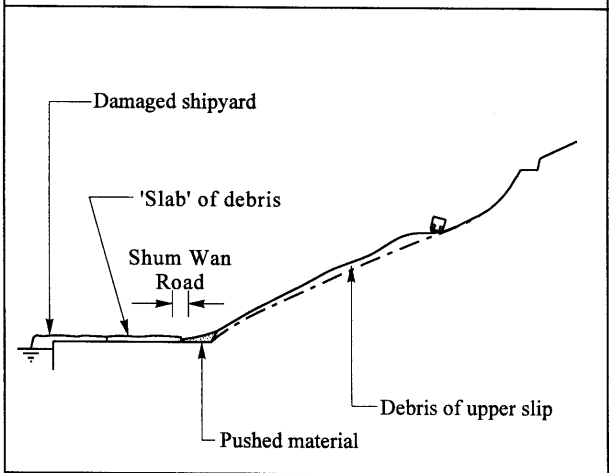
(b) Spoon-shaped slip started at the upper part of the hillside (about 4 a.m.).



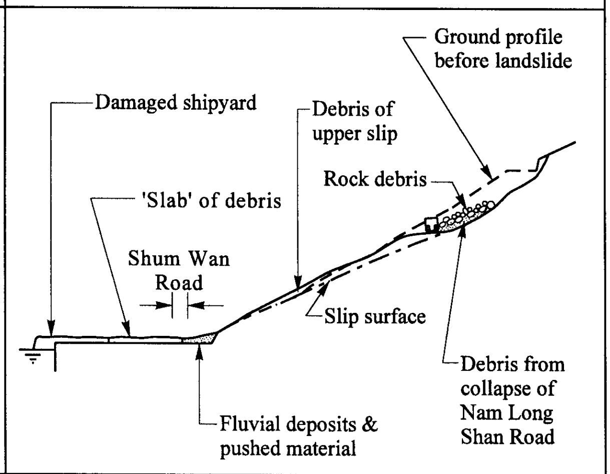
(c) Failed mass from upper slip initiated another slip at the lower part of the hillside (about 4 a.m.).



(d) Pushed material was deposited at toe of hillside (about 4 a.m.).



(e) 'Slab' of debris damaged shipyards and came to rest (about 4 a.m.).



(f) Subsequent slip brought down Nam Long Shan Road, followed by a later slip which released rock debris (about 4:30 a.m.).

Figure 13 - Schematic Representation of Inferred Sequence of Events

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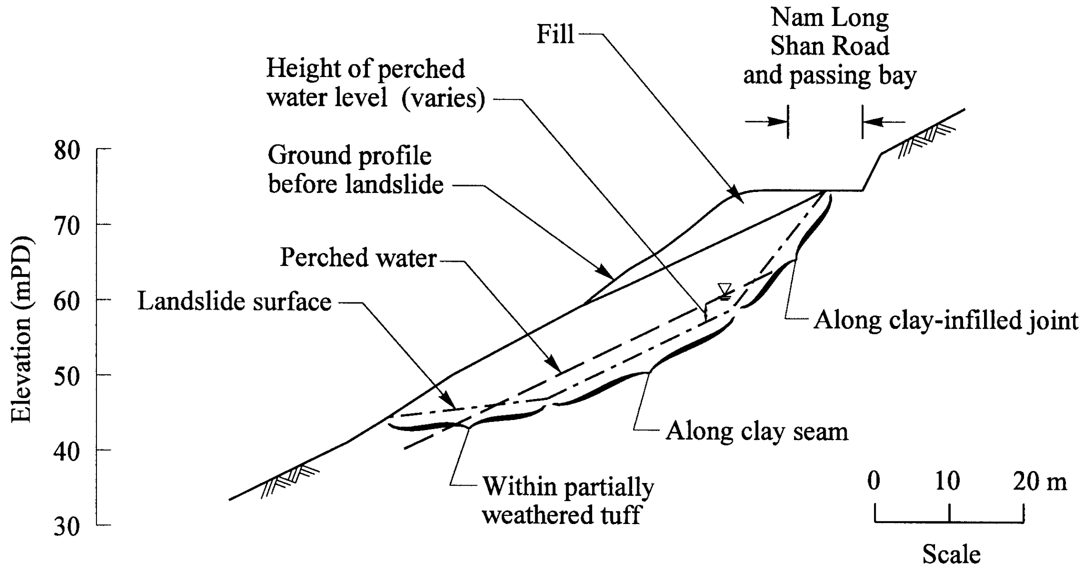
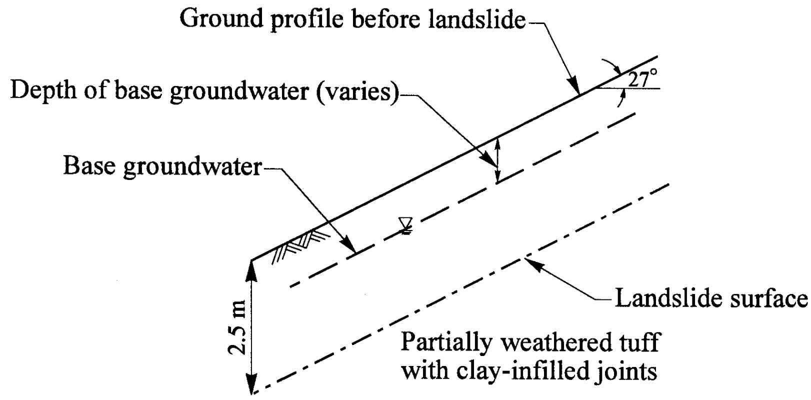


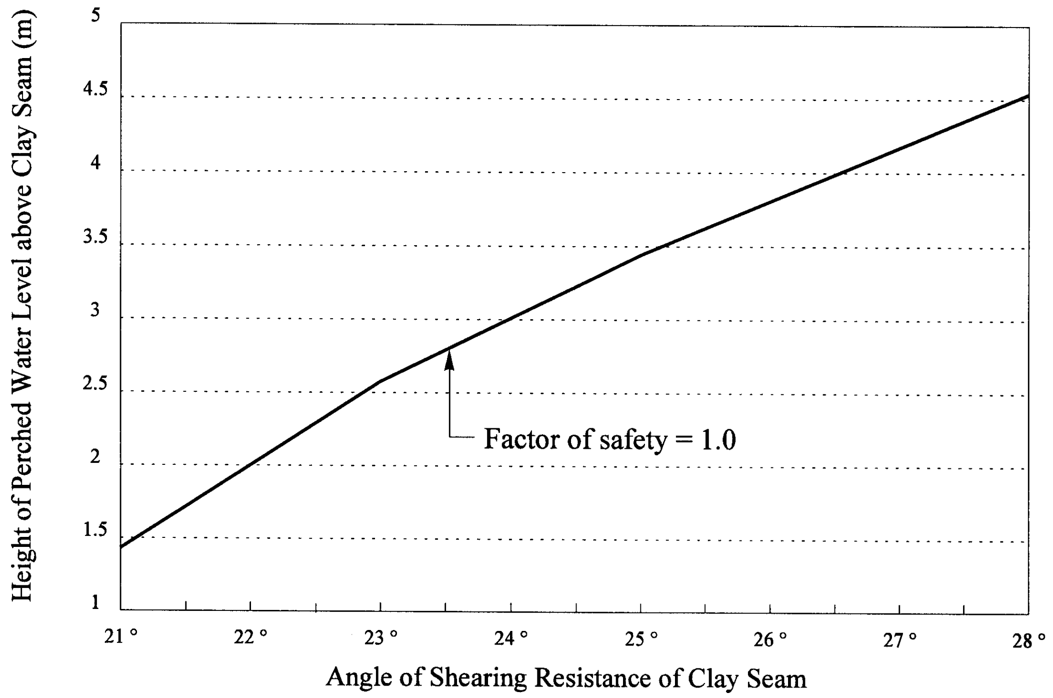
(a) Upper Spoon-shaped Slip



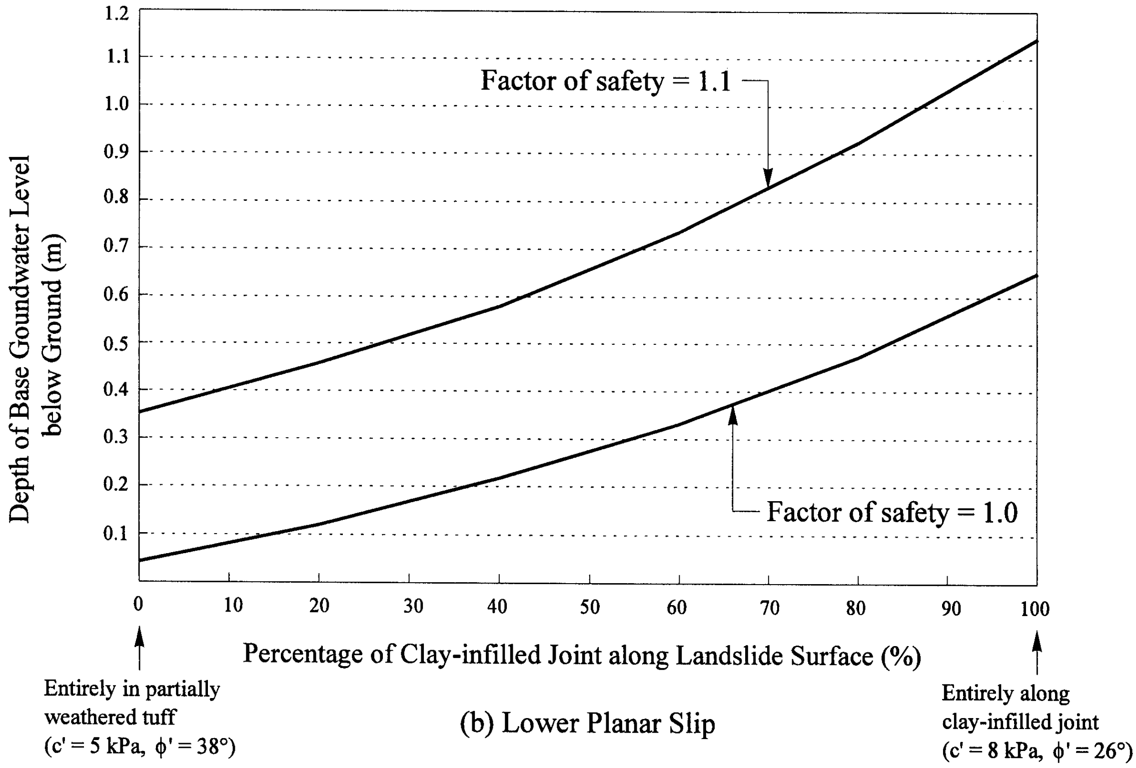
(b) Lower Planar Slip

Material		Apparent Cohesion c' (kPa)	Angle of Shearing Resistance φ '	Unit Weight (kN/m³)
Partially weathered tuff		5	38°	19
Clay seam	Peak strength	8	26°	19
	with slickensiding	0	21°	19
Clay-infilled joint		8	26°	19

Figure 14 - Representative Cross-sections of the Landslide for Slope Stability Analyses



(a) Upper Spoon-shaped Slip



(b) Lower Planar Slip

Note : For ease of comparison, c' = 0 is assumed for the analysis of the spoon-shaped slip. Taking peak strength of clay (c' = 8 kPa, ϕ' = 26°), the height of perched water level for factor of safety = 1.0 is 4.8 m.

Figure 15 - Results of Slope Stability Analyses for Hillside

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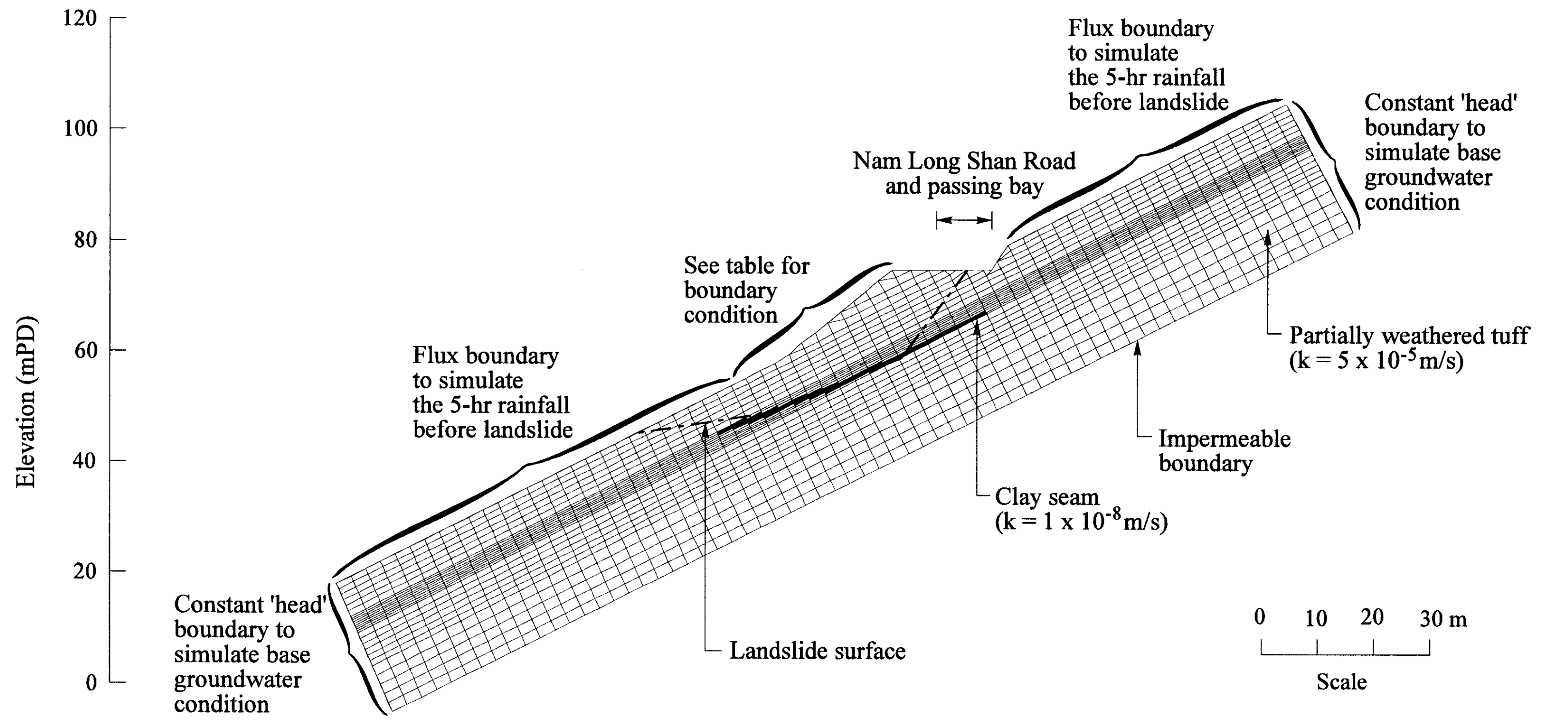

Seepage Analyses	Boundary Condition
To simulate the 5-hr rainfall infiltration before landslide.	Flux boundary
To simulate overflow of water from Nam Long Shan Road.	Constant 'head' boundary

Legend :

k   Coefficient of permeability

Note :   The initial condition for seepage analysis was obtained by assuming the ground surface to be a flux boundary that simulates the 13-day rainfall infiltration before the landslide (from 31 July 95 to 12 August 95).

Figure 16 - Analytical Model for Seepage Analyses

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Plate 1 - The Landslide on 13 August 1995

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Plate 2 - 'Slab' of Debris on the Reclaimed Land

Plate 3 - Deposits at the Toe of the Failed Hillside

Note: See Figure 7 for Locations of Photographs.

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Plate 4 - Debris with Fill Material and Refuse on Landslide Scar

Plate 5 - Retaining Wall Fragment on Lower Part of Landslide Scar

Note: See Figure 7 for Locations of Photographs.

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Plate 6 - Bituminous Pavement (1.5 m x 1 m) on Upper Part of Landslide Scar

Plate 7 - Rock Debris on Landslide Scar

Note: See Figure 7 for Locations of Photographs.

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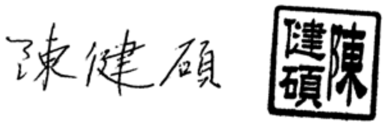
一九九五年八月十三日
深灣道山泥傾瀉事件報告

黎佐賢爵士
英國伯詩亞

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。爲此，我們選擇部份內部報告，編製爲土力工程處報告 (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頁。



土力工程處處長
陳健碩
二零零六年二月

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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深灣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

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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四月土力工程處
一九九五年八月十三日深灣道山泥傾瀉事件報告

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目錄

1. 引言

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，其中在香港仔灣毗鄰的深灣道所發生的一宗，嚴重摧毀了海旁 3 間船廠及 1 間工廠；其後崩塌的建築物發生火警。這次事件導致兩人喪生，五人受傷。

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。

當局邀請筆者對土力工程處的調查，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的初稿。

2. 山泥傾瀉的情況

位於深灣道與南朗山道之間原有的山坡，高約 70 米，斜度約爲 27 度，樹木茂盛。該斜坡約於一九九五年八月十三日凌晨四時崩塌，其後令部分南朗山道塌下。傾瀉的泥石越過深灣道，滑至離山坡腳約 70 米以外，毀壞位於香港仔灣旁的建築物，並將之推進海中。

山泥傾瀉的殘痕平面長度約爲 140 米，闊度在南朗山道約爲 60 米，而在接近山坡腳的位置則爲 90 米。位於原南朗山道的滑坡崖，現出火山岩的陡峭節理。在滑坡崖的下方，地勢陷入山邊形成深窪(凹陷殘痕)。雖然深窪在山泥傾瀉後迅即被從滑坡崖墜下的泥石覆蓋，但深窪的底部及邊緣間，仍顯露出一些部分風化的凝灰岩和一黏土層。在凹陷殘痕的下方有一堆泥石，其中包括南朗山道的瀝青路面和原來停泊在道路外沿讓車處的兩輛貨車。該讓車處建在填土上，約有 5 米闊。道路下方的山坡，有 3 幅擋土牆。

山坡下方的崩塌殘痕(平面殘痕)，是由一層瀉下的泥石和填土，擱在原有的地面上所構成。

傾瀉的泥石跨越位於山坡腳下一岩石峭壁。位於山坡腳蓋著深灣道的大部分泥石，都是軟的沖積物，是在主要的崩塌發生後，由山坡沖下，積在峭壁和掩蓋船廠位置的大量傾瀉泥石之間。山坡上的泥石，有三條顯著的沖蝕溝，露出原有的地層。坐落於填海區船廠位置的傾瀉泥石，覆蓋著一些從原有山坡表面推下的有機物質。

崩塌是逐步發生的。根據一位目擊者所見，是在八月十三日凌晨約四時開始的。主要的山泥傾瀉約在數分鐘之後發生，而南朗山道則在四時三十分左右崩塌。崩塌發生後，曾觀察到地面水把流體的泥石沖向山下。

這次山泥傾瀉的泥石，約有 26 000 立方米。是自本港一九七二年寶珊道的山泥傾瀉以來，於受人類活動影響的山邊，發生的最大規模的遽然崩塌事件。

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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部分。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. 山泥傾瀉地點

本節描述事發地點在山泥傾瀉前地形上的基本特徵，值得注意的是南朗山道上的排水設施、污水渠和食水管道。

3. 事發地點的歷史

本節列出事發地點歷史的事實,資料主要來自地圖及航空照片的研究。值得注意的是，南朗山道旁的擋土牆和讓車處其建造的資料並不多。報告並提及過去山坡上的寮屋活動(一九八八年清拆)，及在一九八三年發生兩宗輕微山泥傾瀉。

土力工程處的岩土地區研究計劃(土力工程處，一九八七)，曾研究該區。深灣道事發地點，在物理性質制肘圖(土力工程處，一九八七)上被特別劃爲「主要爲坡積物地形的一般性不穩定地帶」。當局對事發地點進行航空照片分析，點出山坡上有殘餘的山泥傾瀉痕跡和相關的坡積物。山坡上較近期發生的山泥傾瀉雖屬小規模，但山坡的整體形態，屬於曾移動的已退化的山泥傾瀉地形。

4. 雨量記錄分析

事發現場附近設有兩個雨量計。編號 H20 雨量計位於現場以西約 1.8 公里；編號 H05 雨量計則位於現場西北約 2 公里。這兩個雨量計分別在一九八三年和一九七九年設置。當局利用這些記錄來評估斜坡曾經歷的豪雨。

一九九五年八月本港雨量，是有史以來八月份錄得之最高記錄；尤以八月初的降雨量爲大。以三十一天的降雨時段計算，編號 H05 量雨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，這次豪雨的雨量與過往豪雨中所錄得的雨量相若。

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5. 山泥傾瀉的情況

5.1 實地觀察及量度

山泥傾瀉的描述包括了其形狀、塌下泥石的性質和分佈，以及崩塌的漸進性質。

該山坡大部分被源自坡積物和風化土的泥石所掩蓋，還有相當份量的塡土、建築物料和垃圾。大部分塡土相信是來自南朗山道的讓車處，其中混有擋土牆的碎塊，主要分佈在山坡的下半部分。南朗山道的原有路面及原先停泊在讓車處的兩輛貨車仍可從塌下的山泥中分辨出來。這次山泥傾瀉泥石中最頂層的散石堆，是岩石從滑坡崖崩散滑下而成的，堆積在凹陷殘痕內。

最重要而又預料不到的一項現象，就是越過深灣道及覆蓋著塡海區的泥石竟是一塊近乎完整的「岩塊」，厚度約有 2 至 3 米。這「岩塊」由部分風化的凝灰岩組成，局部節理塡有高嶺土。經詳細勘察該「岩塊」後，發現「岩塊」內岩石的地質結構延續，顯示該「岩塊」基本上是整塊從山邊移到海邊。「岩塊」的表面被植物覆蓋，植物的種類和原先生長在山邊較低部分者相同。

報告內亦提到污水渠和水管的截斷，以及所觀察得該等管道在山坡崩塌後的排水情況。

5.2 目擊人士的闡述

目擊人士所提供的事發經過非常詳盡，就發生在夜間的崩塌來說，是較爲少見，它成爲重組崩塌的可能模式及過程的重要資料。

6. 事發地點的地下情況

6.2 地質

當局在崩塌處進行地質勘察，並輔以探溝、探井、輕型動力觸探測試、鑽孔和地震折射測量等方法，來探測該處的地質情況。

山泥傾瀉發生於完全風化至微風化凝灰岩；崩塌山邊的上部風化較深。一層薄薄的坡積土覆蓋原有山坡，崩塌時隨山坡下滑。坡積土看來不是造成崩塌的肇因。在石英粒子上進行的熱釋光測試，顯示該坡積土有四萬年歷史。現時此

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類測試的可靠程度，尚有不明朗之處，但與本港其他地點的炭十四測年資料比較下，顯示深灣道事發地點的熱釋光測年資料可能是可靠的。

凝灰岩的組構主要傾向東北，走向與山邊成正交。在崩塌殘痕上，組構傾向甚爲陡峭(70 度至 90 度)，但在崩塌殘痕的範圍外，傾向則較爲平緩 (10 度至 40 度)。凝灰岩中有數組節理，近乎垂直的一組，在凹陷殘痕上一走向西北、約 6 米闊的範圍內，間距甚密。

凹陷殘痕的地面有一含高嶺土的黏土層，下面是白色黏土，上面蓋有一層軟而成層狀的暗黃色黏土。這薄黏土層傾向山下，並帶有擦痕面。

6.3 物質特性

當局進行一整系列的分類和強度試驗，及原位密度和滲透試驗。黏土層的部分液限值反常的高，活動值達到 1；而高嶺土的活動值一般約爲 0.4。X 光繞射研究證實白黏土和暗黃黏土的礦物相類似：含高嶺石並可能帶有一些河洛石。河洛石的存在可能是活動值偏高的原因。

部分風化凝灰岩的抗剪強度參數爲 $\phi'=38°$ 及 $c'=5kPa$，與其他地點相類物質的參數幅度相若。

含高嶺土的黏土之最高抗剪強度參數爲 $\phi'=26°$ 及 $c'=8kPa$，而有擦痕面的黏土強度參數(可能近似殘餘強度)則爲 $\phi'=21°$，表面黏聚力爲零。

在部分風化凝灰岩上進行的滲透測試顯示密節理間距的凝灰岩之滲透性較高。

築成讓車處的填土，在這次崩塌中全部移去，其原來的情況無法確定。在崩塌範圍外附近的填土是疏鬆至非常疏鬆，可以預料這樣的物質的滲透性較高。

6.4 地下水情況

在崩塌殘痕若干地點有滲水情況。大約在山腰處，滲水位置顯示地下水位離崩塌前原地面約 1 至 3 米。

在崩塌殘痕上部，岩石中的區域性地下水水位，離凹陷殘痕的底部 約 5 米。黏土層上面的滲水，在山泥傾瀉後持續了約一星期，在大雨後又再出現，顯示該處有上層滯水。

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7. 排水及輸水系統的情況

山泥傾瀉後，當局發現南朗山道的排水井及渠管有部分淤塞。據觀察所得，在一九九五年八月十四日滂沱大雨期間，大量雨水以每秒約 350 公升的流量，從南朗山道流向崩塌殘痕的頂部。和八月十三日的降雨量比較，山泥傾瀉時，雨水在南朗山道的流量可能達到每秒 470 公升。

對現有污水管進行閉路電視檢查後，發現有裂縫和接頭開縫的情況。在現場所檢查的一段完整污水管，顯示滲漏的情況可能甚爲輕微。淡水喉管是由螺紋喉管組成，並無滲漏跡象。

8. 崩塌的可能模式及過程

本節結合從各方面所得的資料，就山泥傾瀉的過程作貫徹及統一的解說。

主要崩塌是在風化岩層發生，並且分爲兩部分。該兩部分相信起初是一起移動的，至崩塌的後期始行分開。不過，崩塌很可能是由讓車處底部的填土堤(假定爲鬆散的)所發生的輕微山泥傾瀉所引發。這輕微崩塌將填土連同擋土牆碎塊帶到崩塌殘痕的下部，這証實在崩塌過程的前期，該些泥石已滑到山坡的下部。填土及擋土牆碎塊的分佈顯示這崩塌的形式是一快速泥石流，例如可發生在鬆散的填土坡那類。在讓車處底部發生的輕微崩塌，亦令南朗山道上的水，直接流至山坡的上部。

主要崩塌發生時，較上部分是沿弧面(匙形)滑動，接合下面平移(平面)部分。崩塌初期，山泥移動得比較慢，到開始加速時，下面平移部分便脫離成爲「岩塊」，向山坡下移動，越過深灣道，然後摧毀香港仔灣旁的建築物。崩塌泥石的上部仍然留在山坡上，當大量的水持續流向山坡的頂部時，便引起快速的侵蝕，將鬆軟的泥石沖到深灣道，積聚在岩石峭壁和「岩塊」的東邊之間。之後，南朗山道的殘餘部分便崩塌到傾瀉了的泥石頂部，接著是石塊從滑坡崖墜下。

9. 理論穩定性及滲流分析

9.1 概述

當局對崩塌的兩部分進行了穩定性分析。

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9.2 山坡上部

穩定性分析顯示，山坡的上部，在正常的區域性水位下，可保持穩定，但卻可因上層滯水影響而沿黏土層滑動，所需滯水位爲 1 至 5 米，決定於黏土抗剪強度，是沿擦痕面滑動的強度，還是剪切未受擾動黏土的最高值。南朗山道填土坡崩塌後，路面的水流入山坡的上部，可以引致該處上層滯水位和水壓急速上升。

9.3 山坡下部

在山坡的下部，地下水的水位接近地面，分析結果顯示，在一些地下水情況下，該山坡在理論上會不穩定。能引致不穩的水位幅度，決定於滑動面有黏土填充的節理的比例。這個結論和觀察所得相符，就是山坡在地質歷史上曾有移動，嚴重的不穩定情況相信已有數百或數千年沒有發生。

10. 山泥傾瀉成因的診斷

根據調查所得的結果，這節全面概述山坡崩塌的過程。筆者亦贊同這項分析。

經過多日連場大雨，地下水位上升，深灣道對上山坡便在一場豪雨中傾瀉。該處山坡在地質史(不是歷史)上曾經移動。山泥傾瀉最初始自山坡頂一個輕微的填土崩塌，因而令南朗山道地面的水直接流到山坡上。水份迅速滲入岩體中，於黏土層上產生上層滯水，令山坡上部的穩定性迅速減低。山坡下部由於部分坐落於填有黏土的節理的岩石，因此本身也並不十分穩定，連場豪雨令地下水提升，使情況更爲不利。崩塌發生時，山坡下部不能爲上部提供有效的支撐，漸進的移動加速，使崩塌泥石下部的一整塊岩塊，越過深灣道，把填海區的建築物摧毀。

11. 其他可構想的因素

本節考慮到一些會對事發地點情況構成影響的因素，如過往的寮屋活動、非法棄置廢物及重型工程車輛。相對來說，這些因素對是次深灣道山泥傾瀉並無顯著影響，筆者贊同它們不是崩塌的誘因。

12. 結論

所述的結論扼要地說明引致這次山泥傾瀉的主要成

因，而筆者亦贊同所述的成因。」

4. 對土力工程處報告所作的結論

土力工程處就深灣道山泥傾瀉事件所進行的調查，是全面和具專業水平的。報告準確地報道了調查的結論，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示贊同。

5. 所汲取的教訓

深灣道山泥傾瀉事件中，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的有關特徵，則值得注意。

5.1 火山岩崩塌如何受地質結構和礦物控制

深灣道山泥傾瀉受火山基岩的結構所支配，該處的岩石沒有層理，且顯然與局部節理和斷層模式無關。導致山泥傾瀉的主要因素，是該處的岩體在輕淺位置，存有黏土層和黏土填充的節理，引致抗剪強度偏低，並控制近地面的淺處地下水活動。

5.2 可能不穩定山坡頂的道路排水

流至一個山坡頂部的水，可以是引發山泥傾瀉的重要因素。在崩塌後，持續有水排進山坡，可將山坡上的物質變弱及軟化，並可延長泥石向山下流動的距離。當局應意識到道路可集水，把水輸到山坡頂部。

5.3 天然山坡崩塌

深灣道山泥傾瀉由於受人爲因素影響，故並非天然的崩塌。然而，發生崩塌的山坡以前曾經移動，一如岩土地區研究計劃(土力工程處，一九八七)所指出，且在後期的航空照片研究中亦加以確定。當局在評估山坡的穩定性時應一併考慮天然因素的影響。

6. 參考書目

土力工程處(一九九六) 八月十三日香港仔深灣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，共 49 頁。

土力工程處(一九八七) Geotechnical Area Studies Programme - Hong Kong and Kowloon (GASP Report 1). 香港土力工程處，170 頁加 4 幅地圖。

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四月土力工程處
一九九五年八月十三日深灣道山泥傾瀉事件報告

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撮要

一九九五年八月十三日，香港仔深灣道對上的山坡發生山泥傾瀉，使南朗山道一段 30 米長的路面，包括以填土堤支撐的一段讓車處，一起崩塌。山泥傾瀉的泥石越過深灣道，摧毀了海旁附近三間船廠及一間工廠，導致兩人喪生，另外五人受傷。

土力工程處於一九九五年八月至一九九六年三月期間就此次山泥傾瀉展開了全面的調查。調查工作包括了資料研究，訪問目擊人士，在事發現場進行地形測量、觀察及量度、地質勘察、場地勘察，視察排水及輸水系統情況，進行理論穩定性及滲流分析，以及判斷崩塌的原因。

調查結果顯示主要的山泥傾瀉涉及幾乎在同一時間發生的兩個明顯部分。崩塌的主因如下：

(a) 土地有薄弱層：即黏土層及有黏土充填的節理；

(b) 連場豪雨後水滲入泥土；

(c) 南朗山道讓車處底下填土堤發生輕微崩塌；

(d) 由於排水系統部分淤塞，水沿南朗山道傾流而下，部 分流至事發的山坡。

本報告載列調查的詳情及調查的結果。

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1. 引言

一九九五年八月十三日凌晨大約四時，香港仔深灣道對上的山坡發生山泥傾瀉(照片 1 及圖 1)，使南朗山道一段 30 米長的路面崩塌。崩塌的泥石越過深灣道，摧毀了海旁附近三間船廠及一間工廠。這次山泥傾瀉導致兩人喪生，五人受傷。

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開始就此次山泥傾瀉展開調查，及後於一九九五年九月二十一日發表一份進展報告(土力工程處，一九九五)，初步描述此次山泥傾瀉事件。

調查工作在一九九五年八月至一九九六年三月進行，主要包括：

(a) 資料研究，包括翻查有關的文件記錄、研究事發地點的航空照片及過往的地形測量圖，以及分析雨量數據；

(b) 訪問山泥傾瀉的目擊人士及其他有關人等；

(c) 在山泥傾瀉地點進行地形測量、詳細觀察及量度；

(d) 地質勘察；

(e) 採用鑽探、現場試驗及室內試驗等方法進行全面的場地勘探；

(f) 視察排水及輸水系統情況；

(g) 以理論方法分析山坡的穩定性和滲流情況。

本報告載列調查工作的結果。調查工作的詳情及所得結果則載於一套文件中。該套文件已存放在土木工程署的土木工程圖書館內，公眾人士可前往該處查閱。

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2. 山泥傾瀉地點

一九九五年八月十三日的山泥傾瀉位於深灣道與南朗山道之間的山坡(圖 1)。在山泥傾瀉前，該山坡樹木茂盛，平均斜度約為 27 度。

南朗山道對下山泥傾瀉範圍的附近有三幅混凝土擋土牆(圖 1)，其中兩幅約有 2 米高，相信是在南朗山道建造讓車處前用來支撐該處路堤的。第三幅牆約為 1.2 米高，看來是在營造寮屋地台時築成的。

在發生山泥傾瀉的地點，南朗山道闊度約為 5 米，在靠近下山的一面有一讓車處。根據一九九四年的航空照片估計，該讓車處約為 5 米闊，由一幅約 10 米高的填土堤支撐。該讓車處的填土堤遮蓋了其中一幅 2 米高的擋土牆(南面混凝土擋土牆)的一部份。在道路靠山的一邊有一幅 4 米高的削土坡。以上各項人造斜坡及擋土牆，在一九七七至

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一九七八年香港政府委託顧問公司所完成的斜坡記錄冊內，均沒有記錄。當局並不清楚沒有記錄該幅 10 米高填土堤的原因，至於其他斜坡及擋土牆，則不符合當時列入記錄冊的準則。

在發生山泥傾瀉的山坡腳下一處，沿深灣道有一個 7 米高陡峭的岩石峭壁。編號 15NW-B/C77 削土坡位於山泥傾瀉位置的南面(圖 1)，在這次事件中沒有塌下。深灣道的闊度爲 7.5 米。

在深灣道與布廠灣濱海區之間的一幅填海土地上有一個臨時工業區，建有多間船廠及工廠(圖 1)。

南朗山道以上山坡的地面水，由天然山溪及人工排水渠收集，經集水井流入埋在道路下的橫向排水管(圖 2)，繼而再經溪澗或人工水渠向山下流，而南朗山道的地面，水則經沿道路下山一邊所築混凝土緣石的排水口流出。同樣地向山下流。在山泥傾瀉位置的南面，有一條 1.2 米寬的梯級渠，將地表水從南朗山道帶至深灣道。

沿南朗山道築有一條直徑 225 毫米的污水管和一條直徑 100 毫米的私用淡水喉管。

3. <u>事發地點的歷史</u>

事發地點的發展歷史已從研究舊地圖，翻查其他文件資料和由專家研究航空照片分析出來。

現存有關事發地點的最早航空照片攝於一九四五年。從這些照片可見到南朗山道，其下方兩幅 2 米高的混凝土擋土牆當時似已建成。七十年代拍攝的照片顯示，山泥傾瀉波及的讓車處，是在一九七六至一九七七年間，在南朗山道加建在一道新的填土堤上的。

照片顯示，在一九七七年，填海工程已開始由深灣道向布廠灣伸延。至於深灣道沿路的削土坡，則是一九七七年至一九七八年間建造的。

除了南朗山道和深灣道的道路工程外，航空照片亦清楚顯示山邊寮屋活動的情況。一九八四年十月編號 15-NW-4C 的地圖顯示該處的一些寮屋的位置(圖 1)。南朗山道下方 1.2 米高的混凝土牆，相信是在七十年代建造寮屋時築成的。一九七七年以後攝得的照片，清楚顯示南朗山道有非法傾倒廢料的情況。

四十年代開始，攝得的照片顯示崩塌山坡範圍內或附近，曾發生輕微山泥傾瀉或沖蝕，主要是集中在沿南朗山道的坡堤上。在一九八八年及一九九一年的照片中可見，此次山泥傾瀉所波及的讓車處，其正下方的填土堤，有兩處沖蝕的痕跡。該兩處沖蝕的位置，剛好在南面混凝土擋土牆的上方，其大約位置見於圖 1。

在一九四九年的航空照片中，可清楚看見事發地點北面山坡上有大塊地面凹陷的跡象，顯示該處可能是古時山泥傾瀉的痕跡(圖 1)。

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在此次山泥傾瀉的範圍內(圖 1)，當局存有兩次輕微山泥傾瀉的記錄(Choot, 一九九三)，該兩次事件均發生於一九八三年六月十七日，被列爲是「天然斜坡沖蝕」和「填土平台沖蝕」。

出事山坡上的寮屋，於一九八八年按照政府的「非發展性清拆計劃」，全部清拆。

從現時能找到的資料看，當局從未研究過此次山泥傾瀉地點附近那些未經列入斜坡記錄冊的人造斜坡和擋土牆，土力工程處亦無該些斜坡和擋土牆的設計或審批記錄。

土力工程處曾於一九七九年至一九八五年間，進行一項全港的岩土地區研究計劃，以便爲當局規劃及管理香港的土地使用，提供有關岩土的資料。此次山泥傾瀉地點一帶的土地和本港其他很多山坡地區，在此研究計劃中製成的 1：20 000 比例的物理性質制肘圖(土力工程處，一九八七 a)(圖 3)上，被列爲「主要爲坡積物地形的一般性不穩定地帶」。被劃爲「一般性不穩定地帶」的地區都是一些過往曾有土體向下移動跡象的地方。

4. <u>雨量記錄分析</u>

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土力工程處在山泥傾瀉現場附近設有兩個自動雨量計(圖 4)。編號 H20 雨量計位於鴨脷洲邨，離現場以西約 1.8 公里；編號 H05 雨量計則位於香港仔水塘濾水站，在現場西北面約 2 公里。

當局分析了該兩個雨量計的雨量記錄。它們在山泥傾瀉前一段時間內所錄得的雨量變化和強度，大致相若。圖 5 顯示編號 H05 雨量計的記錄，以說明事發地點的大概雨量變化和強度。

在發生山泥傾瀉(凌晨四時)前的數小時內，雨勢很大。圖 4 顯示山泥傾瀉前四小時及二十四小時內全港的雨量分佈情況。在八月十二日晚上十一時至八月十三日凌晨三時期間，所錄得的雨量爲 159 毫米，其間凌晨二時至三時的兩量最高，爲每小時 48 毫米(圖 5)。編號 H05 雨量計在山泥傾瀉前三十小時內所錄得的雨量爲 381 毫米。

在一九九五年八月十二日的暴雨之前，於一九九五年八月三日左右，還有另一場豪雨。因爲這兩場豪雨，編號 H05 雨量計在山泥傾瀉前的十三日內，共錄得 846 毫米雨量(圖 5)。

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編號 H05 雨量計自一九七九年裝置以來，所錄得過往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 6，以作比較。從圖中可見，於山泥傾瀉前錄得的雨量偏高。以十五天以上的降雨時段計算，這次豪雨的雨量爲該雨量計有史以來錄得的最高記錄。以二十四小時或較短的時段計算，雨量跟以往錄得的相若。過去的豪雨，雨量跟這次相若的，只有在一九九四年七月二十三日發生的一次。

根據天文台過往的雨量數據，按不同的降雨時段分析這次雨量強度的重現期，發現三十一日時段的雨量最爲罕有，其相應重現期約爲七十五年一次。

根據天文台的雨量數據，本港自一九九二年以來，每年皆發生雨勢大而重現期極長

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的豪雨。土力工程處現正研究，這樣罕有的降雨情況經常出現，是否意味著地區氣候有所轉變。

5. 山泥傾瀉的情況

5.1 實地觀察及量度

土木工程署測量部在當局未展開山泥傾瀉善後工程之前，進行了地形測量，以確定山泥傾瀉和泥石的範圍及地形。山泥傾瀉的範圍載於圖 7，其剖面圖則載於圖 8。

山泥傾瀉所造成的崩塌殘痕，高度爲 70 米，闊度在南朗山道下方約爲 50 米，而在深灣道對上處則約爲 90 米。山泥傾瀉面(圖 8)的上部份形狀是凹陷的(在此稱爲凹陷殘痕)，其最大深度，從崩塌前地面計算，約爲 12 米。山泥傾瀉面的下部是平坦的(平坦殘痕)，離開崩塌前地面 2 至 3 米。

在山泥傾瀉時，位於山坡腳的岩石峭壁沒有崩塌。在崩塌殘痕頂部，一段長 30 米的南朗山道塌下，山泥傾瀉範圍向上伸展至南朗山道對上的斜坡不遠處。

約有 26 000 立方米的泥石在這次山泥傾瀉中塌下，其中約 12 000 立方米停留在山泥傾瀉面(圖 8)。其餘的泥石則散佈於深灣道及其西的填海土地上，泥石散佈的範圍約爲 5 000 平方米(圖 7)。同時也有大量泥石散佈於一間船廠的滑台。填海土地上的泥石表面近乎平坦。

深灣道山泥傾瀉範圍異常龐大(體積爲 26 000 立方米)。資料顯示，這是香港過去二十年來最大宗的山泥傾瀉。

塌下的泥石可大致分爲四個主要類別(圖 7)。在深灣道以西的填海土地上的泥石，是一片近乎完整的「岩塊」，厚度一般約爲 2 米，部份達 3 米(照片 2)。該「岩塊」由部分風化凝灰岩組成，並保留了風化岩的原有節理結構。這些節理結構曾受擾動，但仍可辨認。節理的間距很密，部分節理還蓋了白色的高嶺土，約達 10 毫米厚。該「岩塊」在一些地方覆蓋著植物和表土。「岩塊」的頂部有一堆堆的植物，而其外圍邊緣近海處也散佈了植物。

位於「岩塊」與山坡之間的泥石，主要是十分鬆軟的沖積物，由黏土、粉砂、砂、礫石，以及一些中礫和巨礫組成(照片 3)。泥石一般約 2 米厚。

山泥傾瀉面的平坦部分普遍被厚約 3 米的泥石覆蓋，在凹陷殘痕內，泥石厚達 5 米。泥石主要爲土質，內有填土物料和廢料，例如瓶、發泡膠、車胎及建築廢物(照片 4)。泥石中亦發現其他人造物件，包括混凝土擋土牆碎塊、瀝青路面殘塊、數段直徑 225 毫米的陶管、直徑 100 毫米的鍍鋅鐵水管和混凝土板塊等(圖 9)。此外，大部分混凝土擋土牆碎塊(照片 5)，散佈於較低的平面殘痕上，而瀝青路面殘塊(照片 6)則全部散佈於較高的凹陷殘痕上。

在凹陷殘痕上，有以大量岩石碎塊爲主的泥石覆蓋著土質泥石(照片 7)。岩質泥石

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一般約 2 米厚，最大的岩石碎塊寬約 3 米。

兩輛泊在南朗山道的貨車隨山泥傾瀉墜下，一輛仍停留在一塊瀝青路面上，該塊路面坐在泥石上，並向東傾斜約 14 度，另一輛貨車則被泥石掩埋。在南朗山道，有一輛貨車及一輛的士懸垂於崩塌殘痕的頂部。此外，亦有多輛汽車埋於深灣道的泥石中。

地面水的沖刷作用，在山泥傾瀉泥石中形成了三條顯著的沖蝕溝(圖 7)。沖蝕溝一般約 1 米深及 2 米闊，最大的一條，其闊度大部分約爲 6 米。

5.2 <u>目擊人士的闡述</u>

土力工程處人員訪問了十一位人士，並翻查了其他可能會提供山泥傾瀉事件資料的記錄，例如警方的記錄，以及海洋公園有關其水電供應遭截斷的時間記錄等。

據目擊人士稱，山泥傾瀉在大約凌晨四時發生。當時，山坡下部受深灣道一幢大廈的保安燈光照明，但山坡的上部則頗暗。一位目擊人士觀察到山坡上近南朗山道處有一細小白塊，該白塊逐漸變大。突然，該山坡的下部鼓起，橫跨崩塌殘痕整個闊度的山坡隨之整幅滑下。另一位目擊人士於凌晨四時零六分向警方報告山泥傾瀉。在這次主要崩塌中，南朗山道沒有塌下。據目擊人士所述，南朗山道是於約半小時後發生的一次山泥傾瀉中塌下。

直至一九九五年八月十三日下午早段時間，有人看見大量的水從斷裂的污水管排往崩塌殘痕。海洋公園的職員於上午七時三十分將公園的私用淡水管道關閉。

根據一些目擊人士稱，在南朗山道對下的山坡，曾有人非法傾倒垃圾和建築廢物。而在山泥傾瀉發生前數月，經常有重型工程車駛經該路。目擊人士亦聲稱，在事發前數個月內，在山泥傾瀉範圍內沿山坡向下伸延的排水渠，即使在下雨時水流量也頗低。他們亦提到看見有泥水在山坡表面。另一位目擊人士表示曾在一九九五年八月六日沿一條 1.2 米闊的梯級渠(圖 2)向上行了一段渠道，但沒有看見該渠淤塞。

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6. <u>事發地點的地下情況</u>

6.1 <u>概述</u>

事發地點的地下情況，是根據文件及實地研究所得的資料而確定。文件研究包括翻查現有的岩土資料。實地研究則包括場地勘探，共鑽了八個鑽孔，挖了十四個探井、九個探溝，並進行了二十二次輕型動力觸探測試、一次地震折射測量及地質勘察(圖 10)。鑽孔內安裝了水壓計，以監察地下水的壓力。爲設計善後工程而在毗鄰山坡進行場地勘探所得的資料，亦用來判定崩塌地點的地下情況。

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6.2 <u>地質</u>

崩塌地點的地質，是由部分風化的細粒至粗粒火山灰晶屑凝灰岩而組成，上面覆

蓋著一薄層坡積物，崩塌地點的地質剖面圖見圖 11。該層坡積物，按在毗鄰山坡所見，主要是粉砂／黏土夾雜著一些礫石及中礫碎屑，形成一層不持續的地層，厚約 1 米。經廣州地球化學研究所使用熱釋光技術，在實驗室測定三個坡積物樣本的年代，確定坡積物的年代介乎三萬五千年至四萬八千年之間(廣州地球化學研究所，一九九五)。

部分風化凝灰岩的岩石組構，主要傾向東北面，而在崩塌地點兩旁的毗鄰山坡，岩石組構的傾向和水平線成 10 度至 40 度。然而，在凹陷殘痕內，岩石組構傾向則頗陡峭(70 度至 90 度)。

山泥傾瀉處露出兩個近垂直節理組，以及最少兩個平緩(20 度至 35 度)的節理組，鑽孔內量度得的數據亦證實此點。節理的間距一般有密有疏，但在凹陷殘痕一個約 6 米闊的地帶內，走向為西北方的近垂直節理，其間距卻非常密。這些近垂直的節理，應可讓水份頗容易地穿過部分風化的凝灰岩而向下流。

在間距非常密的近垂直節理的地帶內，岩體的風化程度較其他地方的為大。在山泥傾瀉發生前，在凹陷殘痕範圍內，完全至高度風化的凝灰岩，其厚度約達 20 米(圖 11)。而在平面殘痕範圍內和崩塌地點北面毗鄰山坡，節理間距較疏，完全至高度風化的凝灰岩的厚度，一般約為 5 米。

部分風化凝灰岩內的節理，其表面通常有一層氧化錳，並填有白色黏土，厚度約達 15 毫米。凹陷殘痕的底部，部分是由一個廣闊的黏土層構成，其大概範圍見圖 7 及圖 8。該黏土層包括一層軟的黃褐色黏土，其厚度一般為 100 毫米(但局部地方則約達 350 毫米)，內有高度風化的凝灰岩碎塊，該黏土層底部有些地方覆蓋著一層薄而軟的白色黏土，而白黏土表面又有一層錳。該黏土層帶有擦痕和黑色污痕。在滑坡崖旁的編號 BH3 鑽孔內，亦在地面以下約 7.7 米深處發現另一層軟的黃褐色黏土層。黏土層內的擦痕顯示出該山坡可能在古時曾經移動，但從航空照片卻看不出黏土層所在位置的山坡地面有移動的現象。

滑坡崖在結構上受到一系列在不同程度上填塞黏土的節理所支配(圖 8)。在崩塌地點下部的平坦殘痕部分，山泥傾瀉面是部分風化凝灰岩，內有一些填塞黏土的節理。

當局從山泥傾瀉範圍內收集了十四個黃褐色黏土和白色黏土樣本，送交英國地質調查所，利用 X 光繞射來測定黏土礦物。根據檢驗結果顯示，該兩種顏色黏土均含有高嶺石，並可能含有河洛石(Merriman & Kemp, 一九九五)。白色黏土和黃褐色黏土的礦物成分相類似。

在發生山泥傾瀉前，一幅填土覆蓋該山坡的上部。根據航空照片估算，該幅填土的厚度約達 5 米。

位於南朗山道下方的 2 米高混凝土擋土牆的殘餘部分，其背後的填土為石塊，大部分的尺寸在 200 毫米至 300 毫米之間。在該幅 1.2 米高混凝土牆背後的填土，則是鬆散的黃褐色砂質粉砂／黏土，並夾著一些礫石。

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6.3 物料性質

當局收集了十二個塊狀樣本進行岩土室內試驗，以測定崩塌地點物質的工程性質。試驗於工務中央試驗所進行，包括依照 Chen (一九九四)所述的方法進行分類和指標試驗，以及分別根據 Head (一九八六)及 Head (一九八二)的方法進行固結不排水三軸壓縮(並量度孔隙水壓)試驗和進行直接剪切試驗。分類和指標試驗的結果摘要載於表 1。

從三軸壓縮試驗所得的完全風化凝灰岩的有效抗剪強度參數(圖 12(a))，是在香港類似物質的常見數值幅度之內(土力工程處，一九九三)。

此外，當局亦對白色黏土和黃褐色黏土進行了兩系列的強度測試。從三軸壓縮試驗的結果而獲得黏土的最高抗剪強度數據，載於圖 12(b)。這些試驗結果與黏土的阿太堡限度相符。在黏土擦痕表面進行的直接剪切試驗結果，載於圖 12(c)。

塌下的讓車處下方的填土，在崩塌後一點不留，故此不能測試填土的原本狀況。當局為了推斷這填土之可能狀況，在位於崩塌地點以北約六十米的讓車處的路堤，進行了現場密度試驗。試驗結果連同在室內試驗所得的填料最高乾密度的資料，扼要載於表 2。填土的壓實程度一般低於依照 Chen (一九九四)的步驟量度所得的標準普氏最高乾密度的 80%。根據現行的標準規定，填土路堤的壓實程度須在 95%或以上(土力工程處，一九八四)。故此，若兩讓車處是同一時期興建，在崩塌地點的填土可能是鬆散的。可是，這填土的壓實狀況現已不能確定了。

利用變水頭滲透試驗和壓水試驗(土力工程處，一九八七 b)在鑽孔進行滲透試驗的結果，載於表 3。試驗結果顯示，節理間距極密的部分風化凝灰岩，其滲透系數大約為 10^{-5}m/s。節理間距疏的中度至微風化凝灰岩，其滲透系數明顯較低。

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6.4 地下水情況

當局就這次山泥傾瀉進行調查期間，觀察到崩塌殘痕有多處地點的地面出現持續滲水情況(圖 9 及圖 11)。在崩塌殘痕中部(高程 35 至 50 米)的滲水位置，與附近鑽孔內所錄得的地下水水位相符，滲水可視為是地下水流出地面的現象。這些地點的地下水水位，位於山泥傾瀉前地面以下約 1 至 3 米處。

在凹陷殘痕內場地勘探的結果，顯示該地方的地下水位，位於山泥傾瀉面以下約 5 米處。不過，在凹陷殘痕內露出的黏土層上，亦觀察到滲水情況(圖 9 及圖 11)。上述的滲水情況，於發生山泥傾瀉後持續出現了約一個星期，又於後來大雨後再次出現。這顯示該處出現了短暫的上層滯水。

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一九九五年十二月，在凹陷殘痕內的鑽孔所錄得的地下水位，與一九九五年十月所錄得的水位比較，已經平均下降了約 2 米。這顯示了地下水位下降的趨勢。

7. 排水及輸水設施的情況

一九九五年八月十四日早上下大雨時，位於崩塌地點南面沿南朗山道的排水井有大量地面水流出。這些地面水流出路面，沿南朗山道而下，流入崩塌殘痕。根據所量度得到的水流深度、闊度和速度，流下南朗山道的水流量估計爲大約每秒 350 公升。其後，當局檢查南朗山道的排水渠和排水井，發現南朗山道有些排水井和橫向道路渠管，部分被舊有泥土、植物和垃圾堵塞。

在崩塌地點南面，位於南朗山道與深灣道之間的 1.2 米闊梯級渠，有部分於發生山泥傾瀉後破損，並埋於泥石之中(圖 7)。該條梯級渠破損部分在發生山泥傾瀉前的情況，無法予以確定。不過，有一名目擊人士在發生山泥傾瀉前的一個星期曾沿該梯級渠向上步行了一段渠道，他沒有看見該段渠道出現淤塞。在凹陷殘痕旁的一段 20 米長的梯級渠於山泥傾瀉發生後完整無損，亦無淤塞。

當局於發生山泥傾瀉後檢查了南朗山道在崩塌地點附近的帶水設施。在崩塌地點旁的餘下污水管利用閉路電視檢查後，發現該污水管有多處地方出現裂縫，並有部分接頭開縫(DSD Survey, 一九九五)。其後，當局挖了一探溝(編號 TT9)，露出一段有裂縫和接頭開縫的污水管，發現該段污水管是由 0.7 米長的陶管建造而成，陶管由承插式接頭連接在一起，接頭內填滿水泥砂漿。該等陶管以混凝土加拱支承。在接頭開縫四周的土壤，有小量褐色污痕，但無發現土壤沖蝕的痕跡。陶管的裂縫緊密。根據這些觀察所得的資料顯示，如該段位於山泥傾瀉範圍內的污水渠也是類似情況，則在發生山泥傾瀉之前，該段污水管只會出現輕微滲漏。

該條私用淡水管由螺紋接合的鍍鋅鐵水管組成，沿南朗山道靠下山的一邊地面上鋪設。在進行調查時，當局並無觀察到有任何漏水跡象，例如污痕或毗鄰土地受到沖蝕等，顯示該水管的餘下部分以前曾出現漏水情況。

8. 崩塌的可能模式及過程

山泥傾瀉面的形狀，由上部一個凹陷殘痕及下部一個平面殘痕組成，顯示山泥傾瀉包括兩個部分：在山坡上部一個約爲匙形的滑動面及山坡下部一個平面滑動面。以下爲重組崩塌的可能模式及過程的主要資料：

(a) 山泥傾瀉目擊人士的敘述(第 5 節)；

(b) 平面殘痕上泥石中的混凝土擋土牆碎塊(圖 9)；

(c) 山泥傾瀉前停泊在讓車處及其後墜在凹陷殘痕的貨車，及瀝青路面殘塊(圖 9)；

(d) 散落在深灣道以西填海區上頗爲完整的「岩塊」(第 5 節)。

利用於所得的資料，可重組崩塌的最可能模式及過程，以圖解方式載於圖 13。

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崩塌泥石中的混凝土擋土牆碎塊，遠遠離開了由南朗山道塌下的其他人造物件，包括墜下的貨車、瀝青路面碎塊及污水管的位置(圖 9)，最合理的解釋是在發生主要山泥傾瀉前，位於讓車處下方的填土堤已發生輕微崩塌(圖 13(a))，這可能是填土上發生淺崩塌或填土表面沖蝕。雖然從一九八八年及一九九一年拍攝的航空照片中，可以看見填土堤表面沖蝕的痕跡，淺崩塌亦有可能，可是當局沒有充份証據來確定崩塌的性質。八月十二日晚上十一時至翌日凌晨三時的連場大雨觸發了這最初的填土崩塌。

讓車處下坡山邊的混凝土緣石，因這輕微崩塌而移位，但可能未波及到南面混凝土擋土牆。不過混凝土緣石一旦位移，大量地面水即可從南朗山道流往填土堤，沖蝕填土，並滲入填土，尤其在南面混凝土擋土牆後的填石內，產生上層滯水(第 6.2 節)。這會導致擋土牆及牆後物質下塌並可能高速流動，在這過程中，南面混凝土擋土牆的碎塊會被帶往山坡下一段距離。

主要的山泥傾瀉發生在凌晨四時。在山坡上部，部分地方沿局部風化凝灰岩裡的黏土層滑動，滑動面呈匙形，並帶同讓車處一起塌下(圖 13(b))。一名目擊人士形容，滑動時地面移動並不快速，但卻是持續的，致令瀝青路面及公共設施喉管的碎塊散落在凹陷殘痕上。初期散佈在山坡上部的南面混凝土擋土牆碎塊會隨泥石一並移往山下去。

這匙形滑動造成的泥石對山坡下部構成負荷，影響其平衡狀況，令山坡下部沿一近乎與地面平行的淺平面滑動(圖 13(c))。這平面滑動引致一層相對地薄的部分風化凝灰岩，以近乎完整的塊狀或「岩塊」形式，向山下滑動。此「岩塊」將其前面的植物及表土向下推；部分受推動的植物和表土，以及「岩塊」的前部散落在山坡腳(圖 13(d))。「岩塊」的餘下部分其後掃過堆積的物質，繼續滑向大海方向。此「岩塊」前部最終在距離山坡腳約 70 米處停下來(圖 13(e))。

大部分從山坡上部匙形滑動面塌下的泥石，都堆積在山坡下部的平面殘痕上。南面混凝土擋土牆的碎塊隨著泥石，停留在山坡下部的平面殘痕上，離凹陷殘痕頗遠。

根據目擊人士的敘述，南朗山道在這主要山泥傾瀉中並無崩塌。在主要山泥傾瀉發生後，接著還發生數次輕微崩塌，逐漸向上山方向削去。主要山泥傾瀉後半小時，發生一次較顯著的崩塌，令南朗山道墜下，其泥石堆積在凹陷殘痕上。停留在凹陷殘痕而覆蓋著南朗山道泥石的大幅岩質泥石，相信是從後來一次崩塌中滑下來的(圖 13(f))。

在凌晨約四時發生主要山泥傾瀉後，水由破裂的帶水管道及南朗山道流下崩塌範圍內，沖蝕泥石而形成沖蝕溝，及令泥石堆積在山坡腳的平地上。

上述山泥傾瀉過程與目擊人士敘述、觀察得的泥石堆積次序及特徵，以及從山泥傾瀉中滑下物質的估計體積相符。

其他可能程序亦曾予以考慮，包括主要山泥傾瀉由下部的平面滑動觸發，或上部的匙形滑動發生在凌晨四時主要山泥傾瀉之前等。因為與目擊人士的敘述及人造物件的分佈情況不配合，以上可能情況全不成立。

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9. 理論穩定性及滲流分析

9.1 概述

爲查核第 8 節所提議的機制在理論上是否有可能，當局進行了兩套極限平衡斜坡穩定性分析，一套爲山坡上部的匙形滑動面進行，而另一套則爲山坡下部的平面滑動面進行。分析的具代表性剖面載於圖 14。同時，當局亦以極限平衡斜坡穩定性分析，評估位於已崩塌讓車處底下塡土堤的穩定性。此外，當局更進行滲流分析，以研究各種水源對山坡上部地下水狀況的影響。

9.2 山坡上部

調查人員以條分法分析塡土堤的穩定性，運算時採用了 Morgenstern & Price (一九六五)的精解法。結果證實當塡土受水份飽和時，可能發生淺滑動。

在凹陷殘痕範圍內，山泥傾瀉後地下水的水位，比山泥傾瀉面要低得多(第 6.4 節)。以條分法進行的理論分析顯示，在地下水位如此深的情形下，山坡此部分是穩固的。不過，在凹陷殘痕內的山泥傾瀉面，部分爲黏土層，其上發現有上層滯水(第 6.2 節)，其相關的孔隙水壓可引致地面不穩固。使山坡達到極限平衡狀態的上層滯水位，決定於黏土層的抗剪強度。若以黏土的最高抗剪強度來作分析，理論上，當上層滯水達到 4 至 5 米時，山坡便會崩塌。如黏土層的強度與有擦痕黏土的抗剪強度相近，則崩塌時的上層滯水可以是 1 至 2 米(圖 15(a))。

引致上層滯水的水源可以有以下四個：

(a) 雨水直接滲入南朗山道下方山坡的地面；

(b) 由南朗山道對上山坡流下的地下水；

(c) 南朗山道排放下來的水；

(d) 沿南朗山道鋪設的帶水設施滲漏的水。

在發生主要山泥傾瀉前三十小時，共有約 380 毫米雨量直接落在山泥傾瀉的範圍。雨水可以滲入讓車處底下的塡土堤及滲過下層的部分風化凝灰岩。用有限元電腦程式進行滲流分析(圖 16)，顯示單是上述降雨量，只會在黏土層上形成低水頭(約 1 米)的上層滯水。

八月初豪雨所引致的在南朗山道對上山坡的地下水，在八月十三日可能已流抵崩塌地點。用有限元電腦程式進行的滲流分析的結果，顯示上述水源大部分會灌入地下水，只有少量流至山泥傾瀉現場的黏土層上，產生上層滯水。至於可積聚的上層滯水水位則無法作確實評估，但結果顯示水位偏低(約 1 米)的可能性較高。

上述理論評估顯示單是雨水滲透，並不能造成一個顯著的上層滯水位。理論計算亦顯示污水管或供水管輕微滲漏，亦不可能產生一個顯著的上層滯水位。

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調查人員估計，當填土堤出現輕微崩塌而令讓車處的混凝土緣石移位後，在一小時內會有數以百計立方米水從南朗山道排放至崩塌地點，水量大大超過直接降落在填土堤的雨量，其中部分水流可能滲入透水度較高的部分風化凝灰岩內。滲流分析顯示，這可以產生高水位的上層滯水，水位視乎水流的持續時間而定。舉例來說，理論上，由南朗山道流下的水流在三小時內，可將上層滯水的水平推高至約 5 米。

9.3 山坡下部

在平面殘痕位置進行的場地勘探，顯示這處地下水的水位，比在凹陷殘痕更為接近山泥傾瀉前的地面。以極限平衡穩定性分析，利用無限坡解法，顯示當地下水的水位升至離地面 0 至 1 米之間，山坡理論上可以變得不穩定，導致山坡不穩的水位，視乎沿滑動面有黏土填塞的節理的比例而定(圖 15(b))。

山坡上面匙形滑動相關的泥石，會對山坡下部構成負荷，因而降低其安全系數。

10. 山泥傾瀉成因的診斷

從擋土牆散落的泥石位置，可知在約凌晨四時發生主要山泥傾瀉之前，讓車處底下的填土堤，曾發生輕微崩塌。斜坡穩定性分析結果顯示，當填土水份飽和時，填土堤可能會作淺度滑動。這輕微崩塌亦可能是由於填土表面受侵蝕所引致。可是，當局沒有充份証據來確定那一個崩塌模式可能性較高。在上述任何一個情況下，觸發輕微崩塌的水可能來自直接降落在填土上的雨水，或是由南朗山道流下的水，亦可由上述兩個來源組合而成。

凌晨四時發生的主要山泥傾瀉可分為兩部分，分別是上部一個匙形滑動和下部一個平面滑動。上部的匙形滑動是由黏土層的上層滯水壓力所觸發。在這位置，地下水的水位遠低於山泥傾瀉面(第 6.4 節)。實地監察地下水壓力及理論分析顯示，地下水的水位顯著升高(超過 6 米)而導致滑動的假設，並不成立。黏土層比毗鄰的部分風化凝灰岩薄弱得多，在上部匙形滑動面的底層的大部分地方，形成一個薄弱的平面。

理論分析結果顯示，在崩塌時，上部的匙形滑動大有可能是由高的上層滯水位(4 至 5 米)所引致，而黏土層的強度接近無擦痕黏土的抗剪強度，即 $c'=8kPa$，$\phi'=26°$。在填土堤發生輕微崩塌後，有大量水自南朗山道排下(第 8 節)，及在密間距節理的部分風化凝灰岩，量度得的高滲透性(表 3)，根據滲水分析(第 9.2 節)，高上層滯水位是有可能的。故此，高的上層滯水位導致山泥傾瀉的假設是可以成立的。

調查發現，黏土層一小部分地方有擦痕(第 6.2 節)。有擦痕黏土的抗剪強度頗低($\phi'=21°$)。如擦痕面範圍廣闊，則較低的上層滯水位便可觸發山泥傾瀉(如 $\phi'=21°$，需 1 至 2 米水頭)。如此低的上層滯水位，可以由雨水直接滲入崩塌地點及來自山坡上的地下水所造成。類似的滲入情況在過往可能亦曾定期出現，故低上層滯水的假設，無法解釋山坡為何沒有在過往的豪雨中崩塌。這個假設亦表示填土堤的崩塌與這次山泥傾瀉並無直接關連，而主要山泥傾瀉發生在填土堤崩塌後，純屬巧合而已。以上討論可見，高上層滯水假設實較低上層滯水假設更有可能。

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在下部平面滑動處的地下水水位頗高，主要歸因於七月及八月初的連場大雨。在一九九五年十月，錄得的地下水約位於山泥傾瀉前地面以下 1 至 3 米。一九九五年十月至十二月期間的地下水水位下降趨勢(第 6.4 節)，顯示一九九五年八月的地下水水位應更高，特別在發生山泥傾瀉前三十小時連場豪雨期間，水位應升至十分接近未崩塌前的地面。理論穩定性分析顯示，在上述地下水情況下，此處的部分風化凝灰岩可以崩塌，並以一整「岩塊」方式，部分沿著有黏土填充的理節滑下(第 9.3 節)。上部匙形滑動產生的負荷觸發此平面滑動。

11. 其他可構想的因素

崩塌後，目擊人士提供了他們觀察到可導致這次崩塌的因素，包括寮屋、非法棄置廢料及重形工程車輛。當局在這次調查中曾審查這些因素，發現它們對崩塌並無顯著影響，其論據如下。

平面滑動處在以前曾建有寮屋，因而改變了該處的地形，例如削平山坡以建寮屋。此舉削弱了部分風化凝灰岩「岩塊」在山坡下部的坡腳支撐力，影響了「岩塊」的穩定性。不過，坡腳支撐力的減少若與「岩塊」底部的抗滑動能力比較，實在微不足道。

在南朗山道斜坡處非法棄置垃圾及建築廢物可引致積水，加多了滲水情況。不過，在填土堤輕微崩塌前的降雨量，並未高過泥土的滲透率，因而大部份雨水可滲入地下而沒有流走或產生積水。填土堤崩塌後，大量水由南朗山道傾流而下，因此不須要出現積水情況亦不斷有水滲入地下。非法拋棄的廢料亦可能造成額外負荷，但對大幅山坡的穩定性並無顯著影響。因此廢料不構成山泥傾瀉的誘因。

往來南朗山道的重型工程車輛可加添路堤的荷載，亦可能毀壞讓車處側的混凝土緣石。不過如與崩塌山坡泥土的重量比較，額外負荷所引致的影響頗微。當局無法確定車輛有否損毀混凝土路堤，若是有的話，由南朗山道傾流而下的水便可隨時流入填土堤。當局無法找到資料來鑑定混凝土緣石在崩塌前的情況，以判斷來往重型工程車輛是否因此而成為山泥傾瀉原因之一。

12. 結論

主要的山泥傾瀉涉及幾乎在同一時間發生的兩個明顯部分，崩塌的主因如下：

(a) 土地有薄弱層：即黏土層及有黏土充填的節理；

(b) 連場豪雨後水滲入泥土；

(c) 南朗山道讓車處底下填土堤發生輕微崩塌；

(d) 由於排水系統部分淤塞，水沿南朗山道傾流而下，部分隨後流至事發的山坡。

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附表

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表 1 － 分類及指標試驗摘要

物料種類	樣本位置	深度(米)	樣本種類	粒徑分佈				LL (%)	PL (%)	PI (%)	含水量 (%)	比重
				礫石 (%)	砂 (%)	粉砂 (%)	黏土 (%)					
CDT	S2	近地面	塊狀	1	17	44	38	67	30	37	29.4 29.2 29.6 30.6	2.64
CDT	S4	近地面	塊狀	0	8	64	28	48	33	15	30.3 38.6 38.4	2.64
CDT	S5	近地面	塊狀	1	17	63	19	48	28	20	30.0 35.2 33.4	2.63
CDT	BH3	1.78 - 1.98	Mazier	2	34	42	22	-	-	-	27.4	2.62
黃褐色黏土	S8	近地面	塊狀	0	8	28	64	99	39	60	45.6 53.3 52.3	2.63
黃褐色黏土	S9	近地面	塊狀	0	12	48	40	123	48	75	63.0	2.62
填土	TP10	0 - 1 米	散樣	6	26	54	14	35	21	14	-	-
填土	TP11	0 - 1 米	散樣	21	17	42	20	39	22	17	-	-
填土	TP12	0 - 1 米	散樣	37	13	38	12	35	21	14	-	-
填土	TP13	0 - 1 米	散樣	5	17	49	29	57	24	33	-	-
填土	TP14	0 - 0.5 米	散樣	6	29	42	23	41	23	18	-	-

圖例：

CDT	完全風化凝灰岩	LL	液限	PL	塑限
PI	可塑性指數	BH3	3 號鑽孔	S2	塊狀樣本 2 號
TP10	10 號探井				

註：依 Chen（一九九四）所述的方法進行各項測試。

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表 2 － 填土物料密度試驗的結果

探井	深度(米)	物料	實地乾密度 (Mg/m³)	實地含水量 (%)	試驗所最高乾密度 (Mg/m³)	最適度含水量 (%)	壓實程度 (%)
TP10	0	黃褐色含礫石、砂的粉砂/黏土	1.26	22	1.75	18	72.0
	0.5		1.30	21			74.3
	1.0		1.34	21			76.6
TP11	0	黃褐色含微量礫石、砂的粉砂/黏土	1.25	24			71.4
	0.5		1.29	24			73.7
	1.0		1.27	25			72.6
TP12	0	黃褐色含礫石、砂的粉砂/黏土	1.39	20			79.4
	0.5		1.33	23			76.0
	1.0		1.25	24			71.4
TP14	0.5	黃褐色含礫石、砂的粉砂/黏土	1.18	15	1.65	21	71.5
	0.5		1.35	14			81.8

註：填土的分類和特性指標，見表 1。

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表 3 － 滲透試驗的結果

物料	試驗種類	滲透系數 (m/s)
完全至高度風化凝灰岩	鑽孔內的變水頭測試	1.3×10^{-5} to 8.0×10^{-5}
有極密間距節理的中度至微風化凝灰岩	鑽孔內的壓水及變水頭測試	1.2×10^{-5} to 6.4×10^{-5}
有極密間距節理的微風化凝灰岩	鑽孔內的壓水測試	1.3×10^{-5} to 1.8×10^{-5}
微風化凝灰岩	鑽孔內的壓水測試	6.6×10^{-9} to 4.3×10^{-7}

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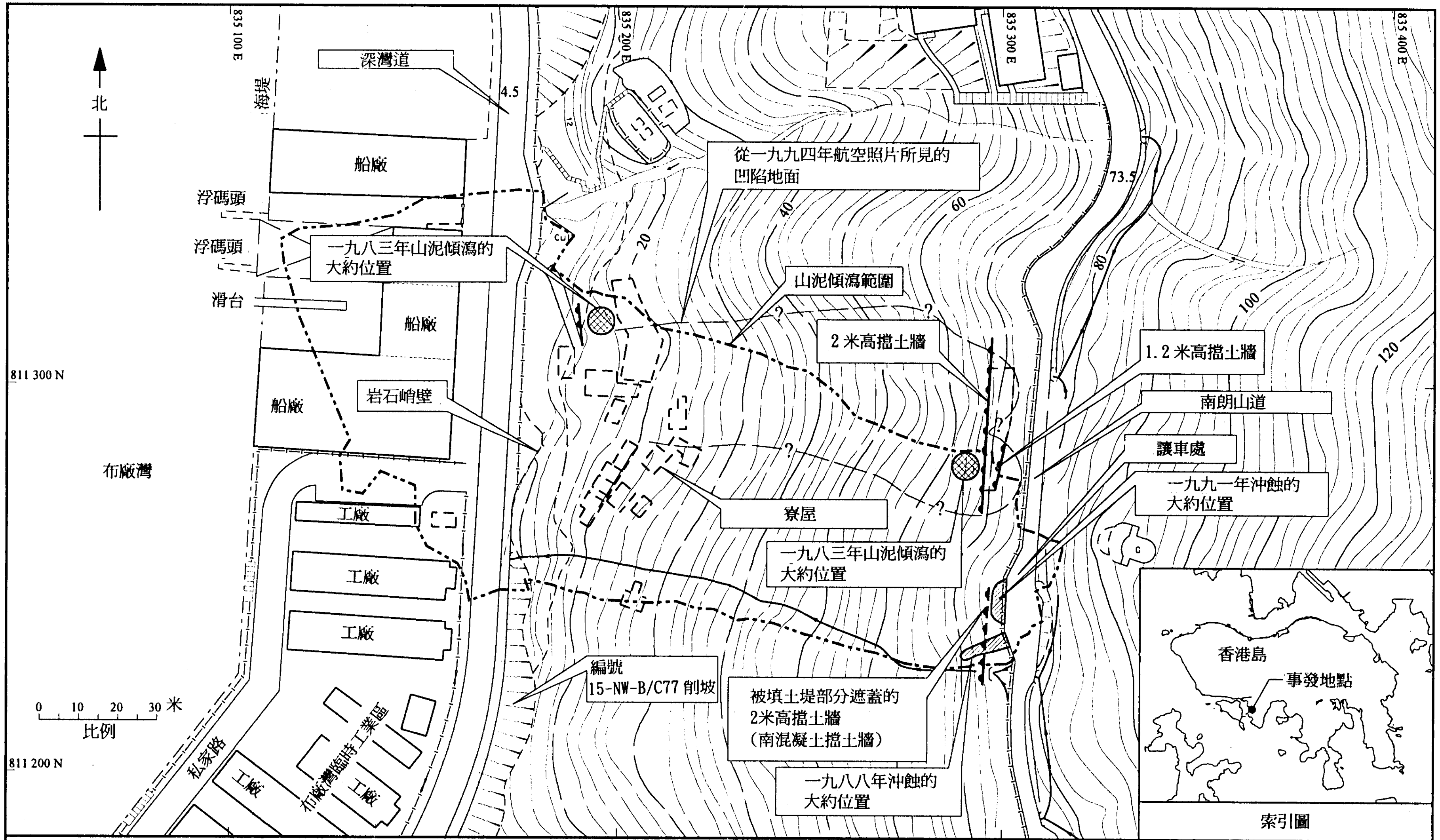

註：（1）底圖取自一九九二年八月編號 15-NW-4C 及一九九二年九月編號 15-NW-4D 地圖(比例：1:1000)。
（2）寮屋位置資料取自一九八四年十月編號 15-NW-4C 地圖(比例:1:1000)。

圖 1- 山泥傾瀉位置圖

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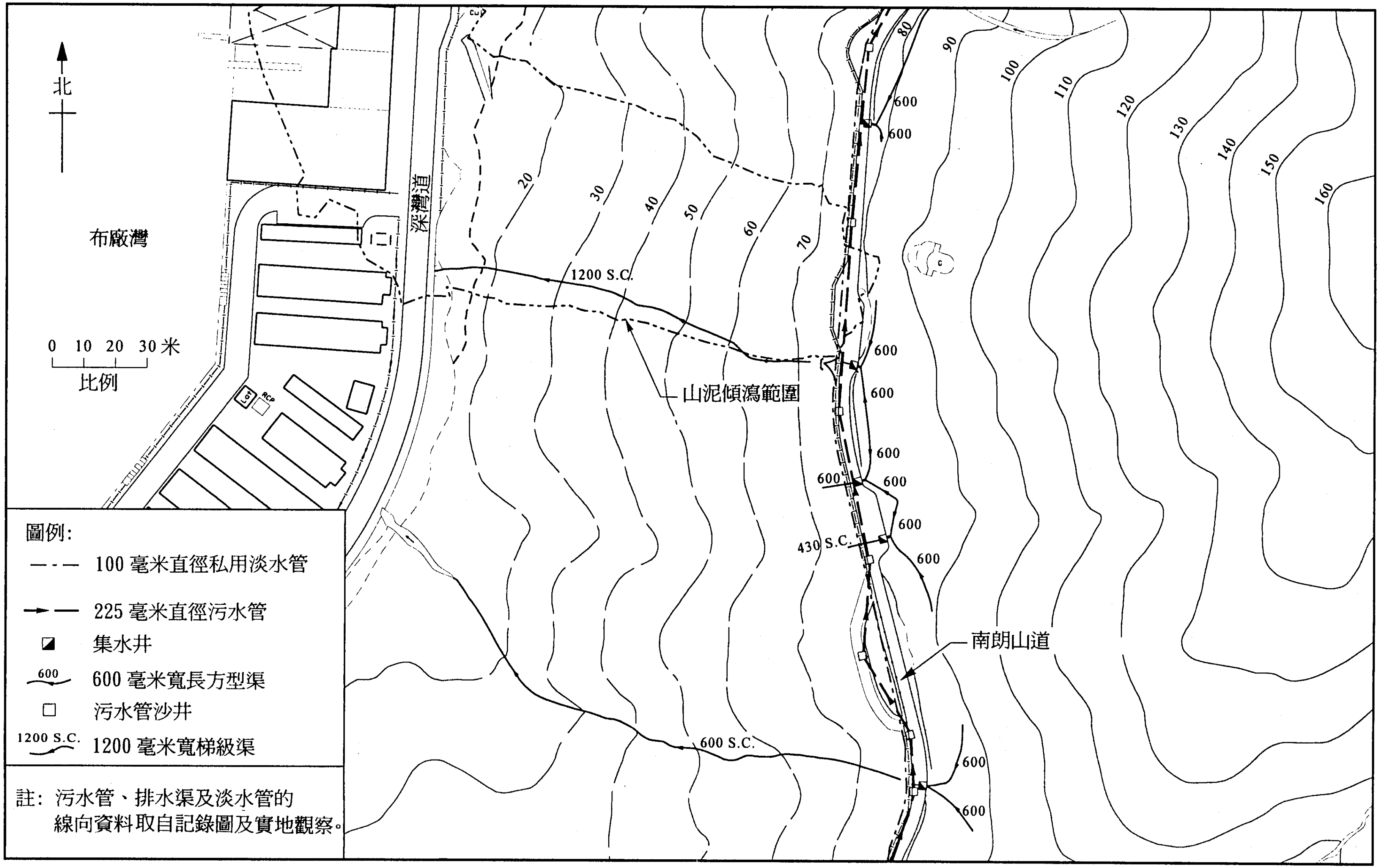
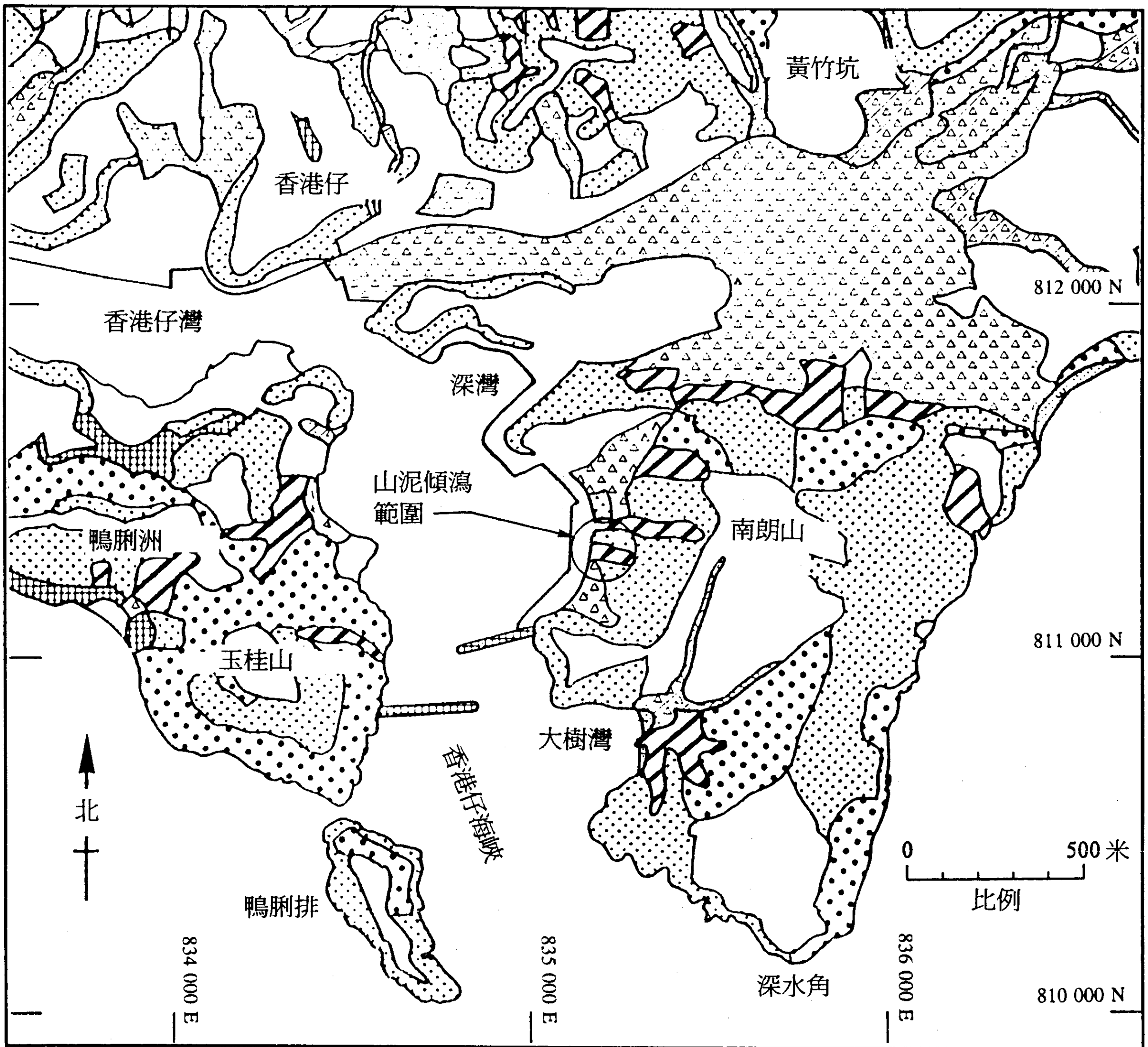


圖 2 － 南朗山道排水管及輸水系統的分佈圖



圖例:

坡積物

受地面徑流及周期性淹沒的坡積物層。有不尋常地下水體系的跡象(地形圖中劃爲排水平原)

洪泛區 - 受地面徑流及定期淹沒影響。有不尋常地下水體系的跡象(地形圖中劃爲洪泛區)

主要爲坡積物地形的一般性不穩定地帶

主要爲原有岩層地形的一般性不穩定地帶

原有岩層地形傾斜度一般超過30度的斜坡(列爲坡積土地形或不穩定者除外)

註:(1)本圖取自岩土地區研究計劃(土力工程處,一九八七a)中GASP/20/1/6 地圖(比例爲1:20000)。

(2)物理性質制肘圖只可用來作地區規劃時考慮限制的一般性質的指引。地圖比例爲1:20000,不可用來評估面積小於3公頃的土地。

圖3 - 山泥傾瀉地點的物理性質制肘圖

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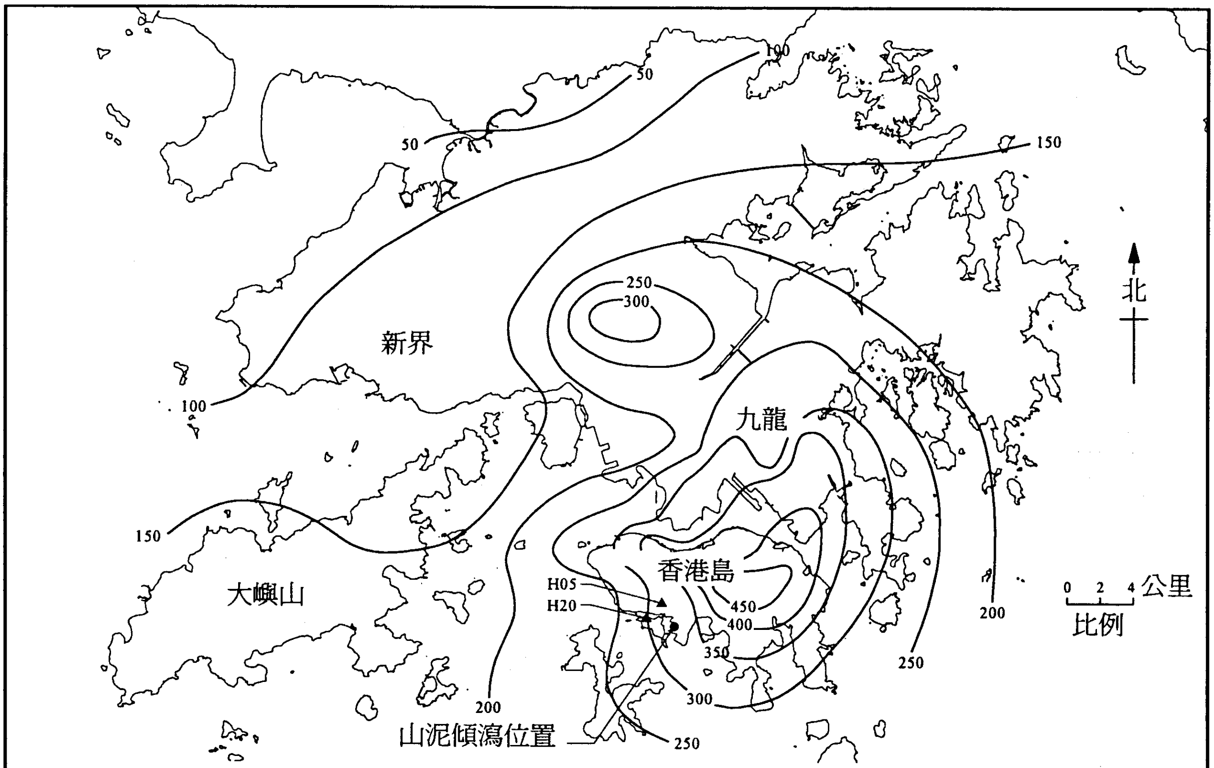
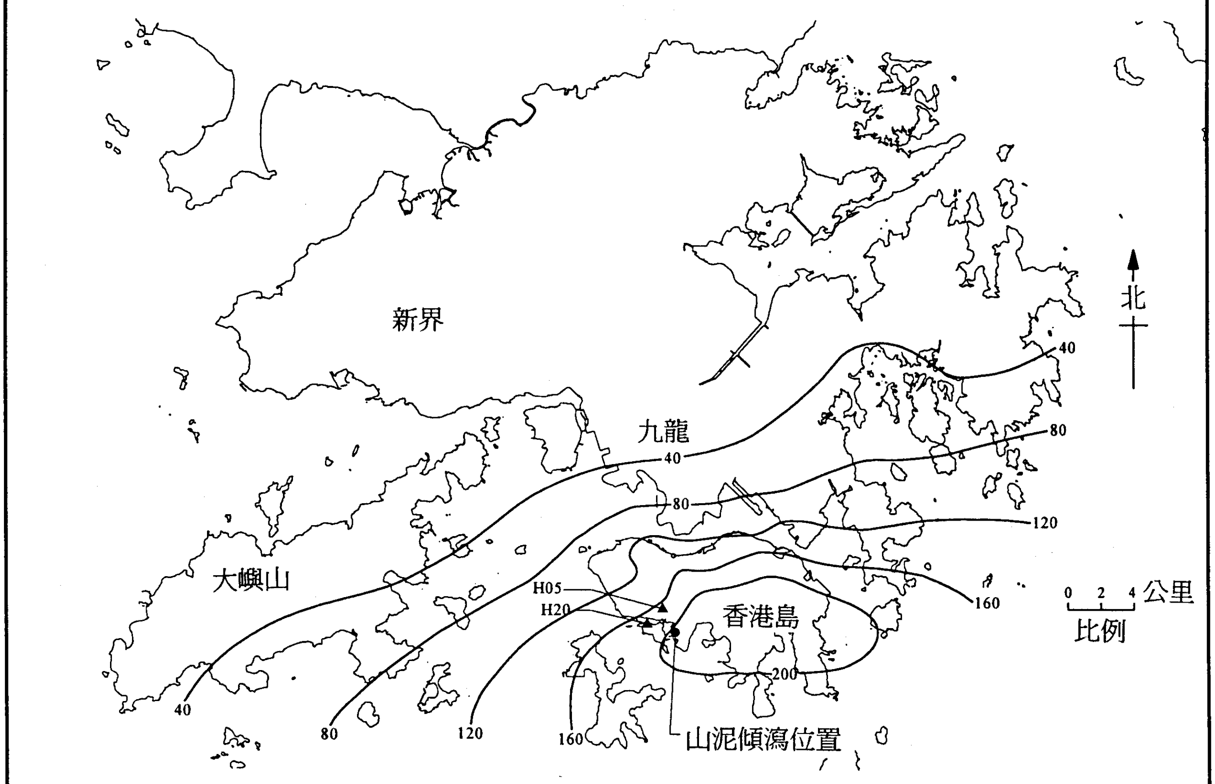


(a) 一九九五年八月十二日凌晨四時至一九九五年八月十三日凌晨四時的雨量分佈



(b) 一九九五年八月十二日晚上十一時至一九九五年八月十三日凌晨三時的雨量分佈

圖例:

—200— 等雨量線(毫米計)　　▲ 土力工程處雨量計

圖 4 - 山泥傾瀉前的雨量分佈

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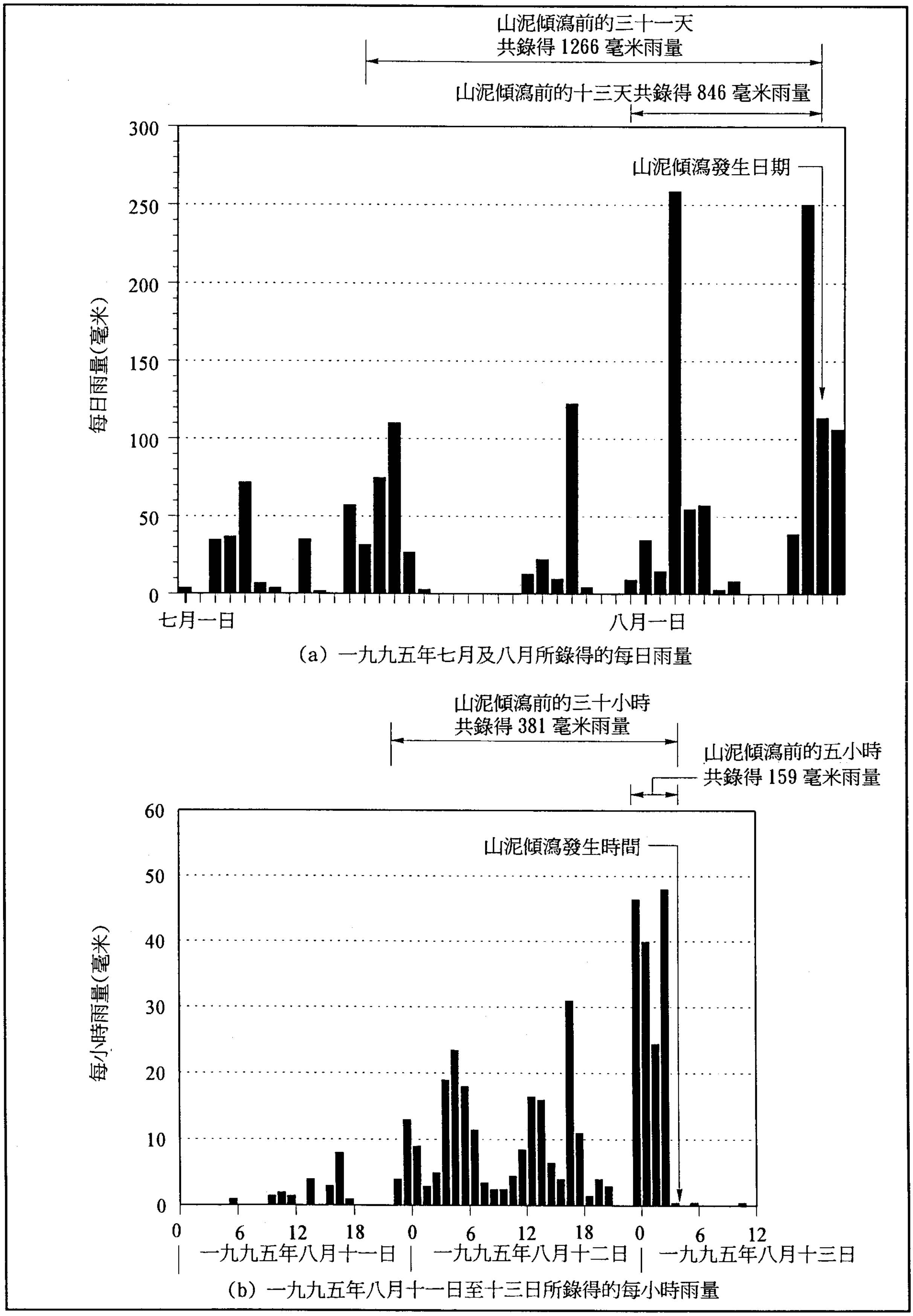

圖 5 – 土力工程處編號 H05 雨量計的雨量記錄

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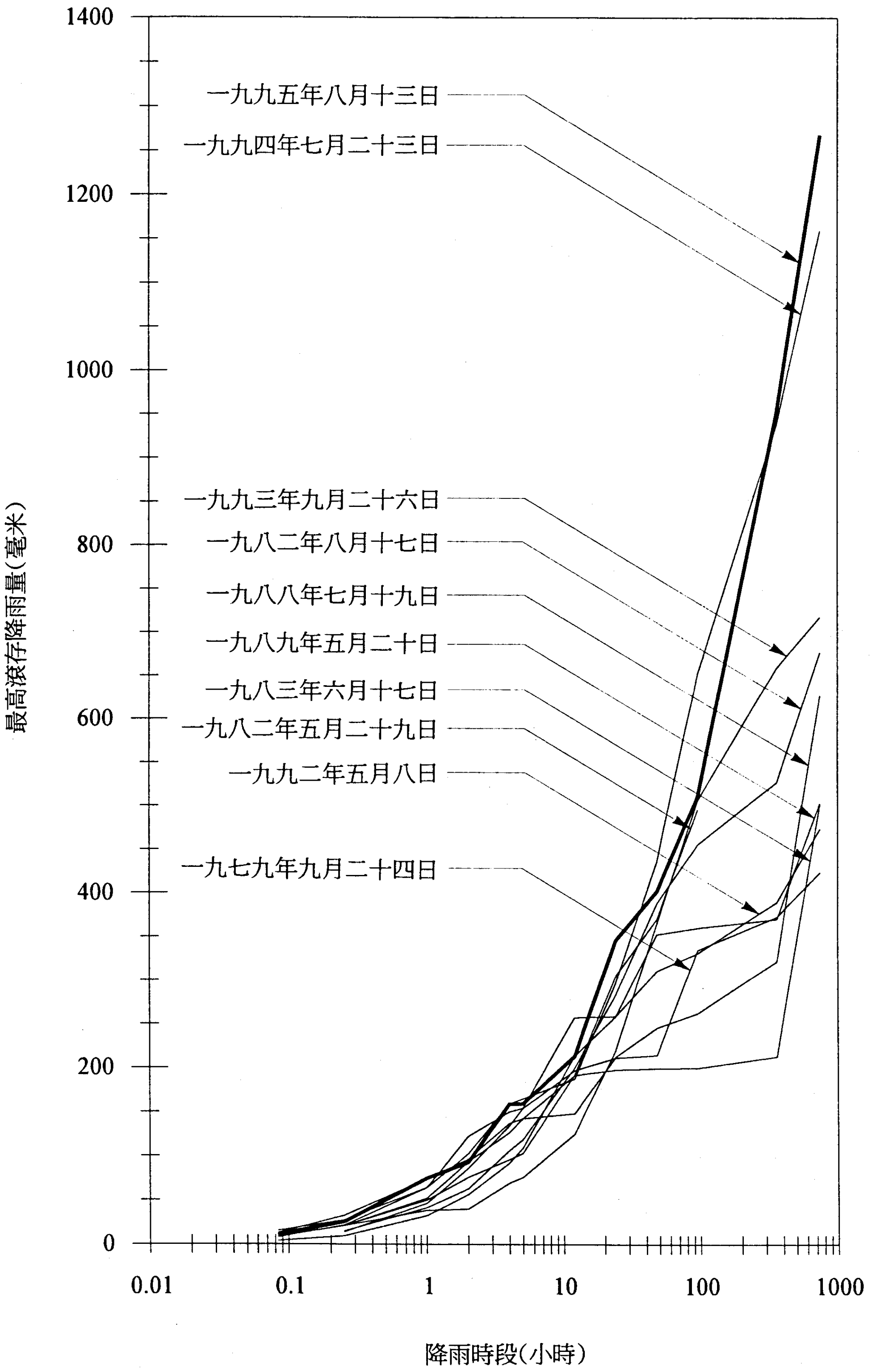

圖 6 － 編號 H05 雨量計於歷次豪雨中錄得的最高滾存降雨量

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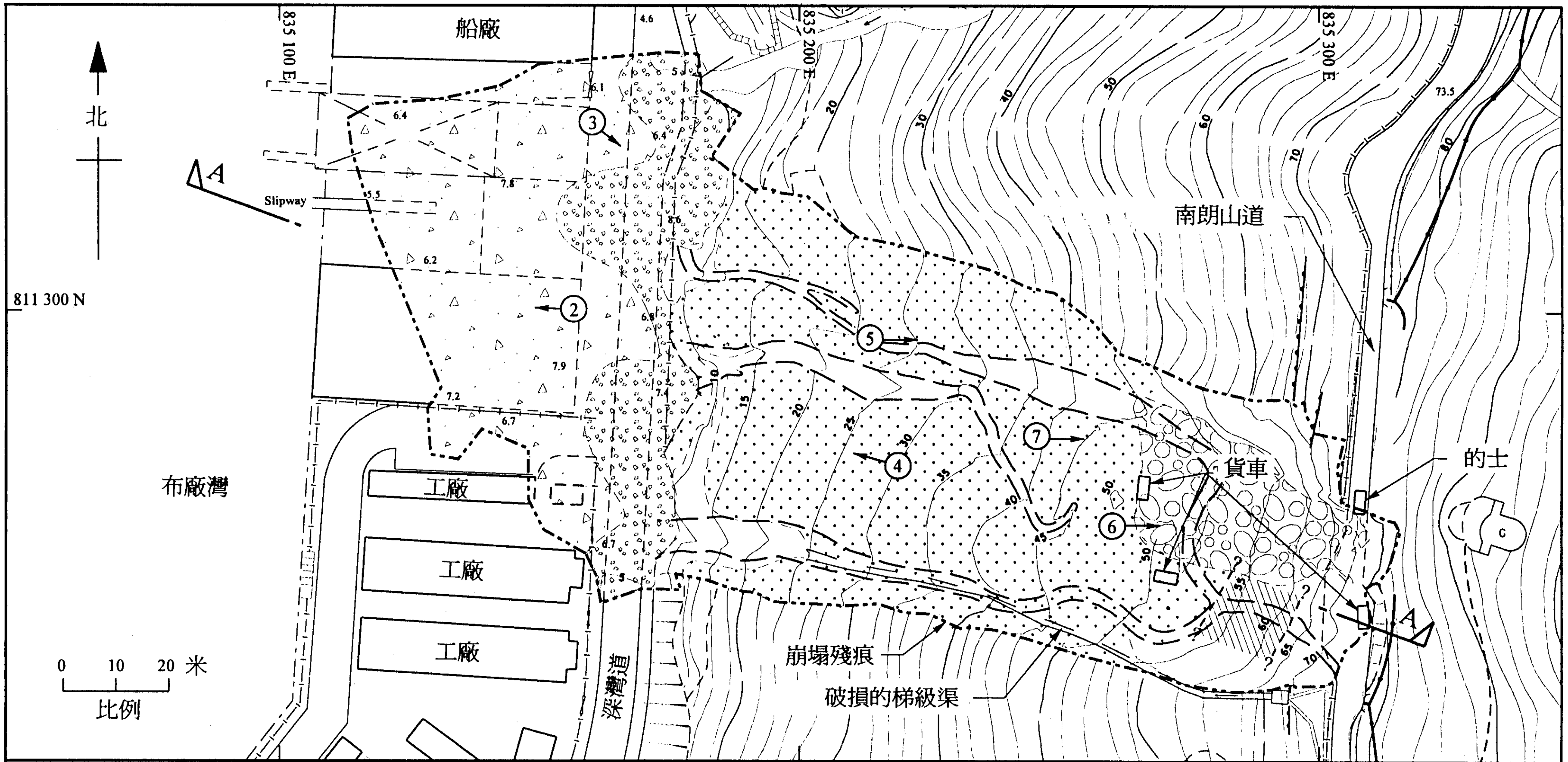

圖例：

35	以米計算的等高線	6.7	以米計算的高程點		擋土牆		沖蝕溝
	由一「岩塊」組成的泥石		主要為沖積物的泥石		帶有填土的泥石		岩石泥石
←②	照片 2 的觀景點		露出黏土層的大約範圍				

註：（1）剖面 A – A 見圖 8。

（2）圖中資料來自於一九九五年八月十三日至十一月二十三日期間進行的地形測量、地質勘察和現場觀察。

圖 7 – 山泥傾瀉平面圖

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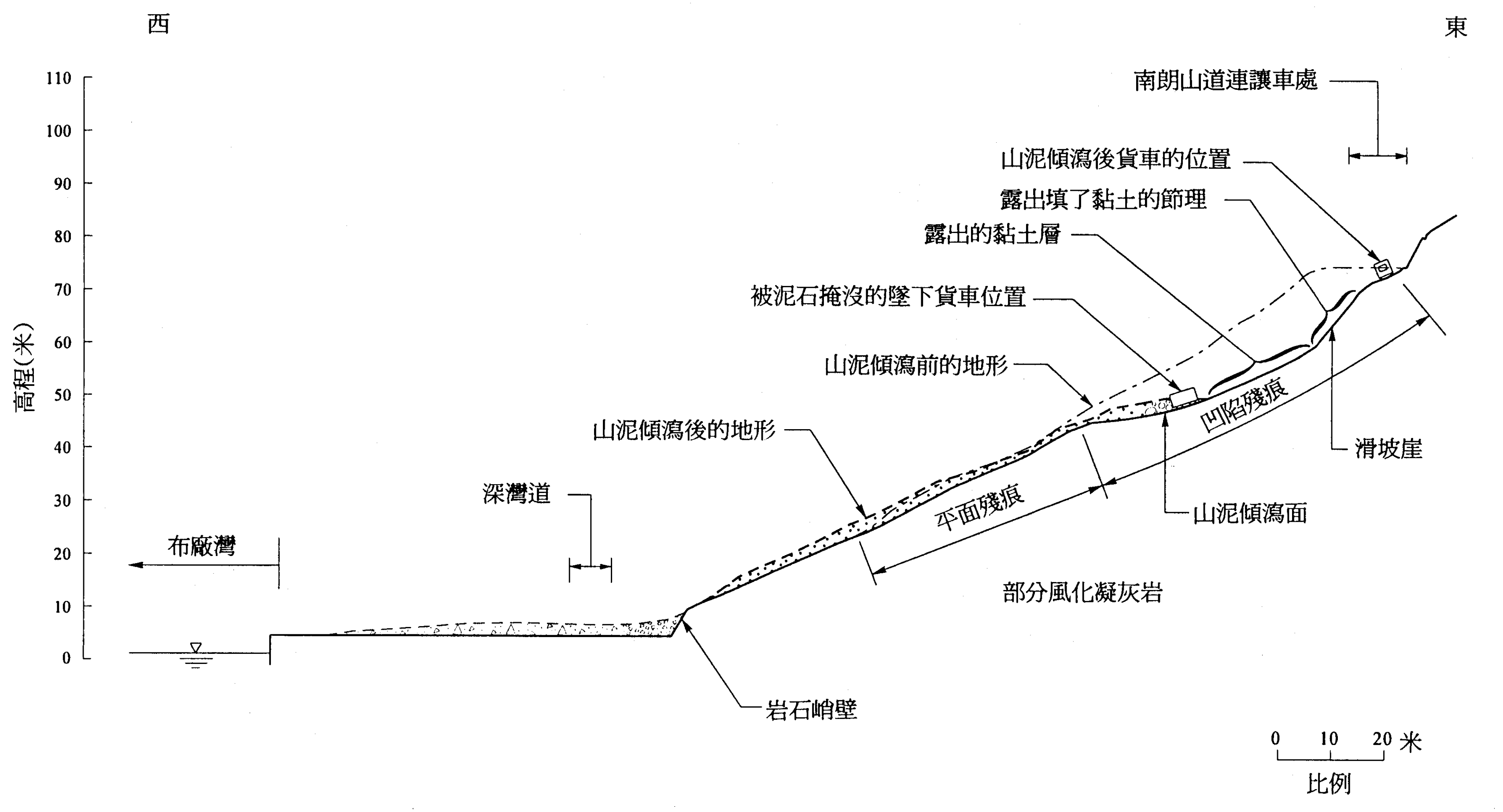
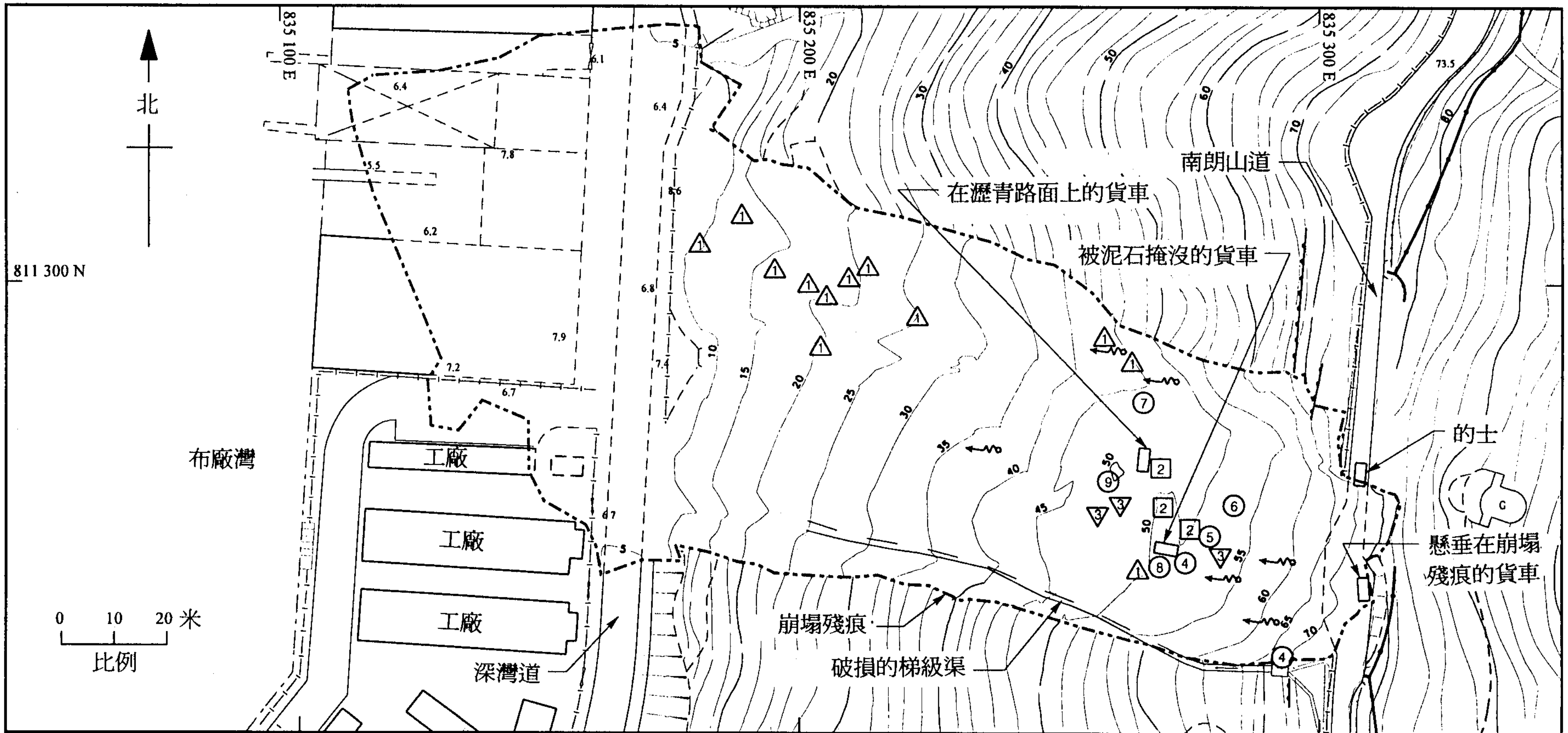


註：(1) 剖面位置及泥石圖例見圖 7 。
(2) 圖中資料來自地形測量、地質勘察和現場觀察。

圖 8 － 崩塌地點剖面 A － A



圖例:

⚠1	擋土牆碎塊	2	瀝青路面	▽3	污水管段
④	垃圾，包括家庭廢物、發泡膠等	⑤	鐵桶	⑥	混凝土緣石段
⑦	沙井	⑧	水管段	⑨	車胎
←∿	滲水點				

註：本圖並未將所有人造物件顯示。

圖 9 – 人造物件和滲水點位置圖

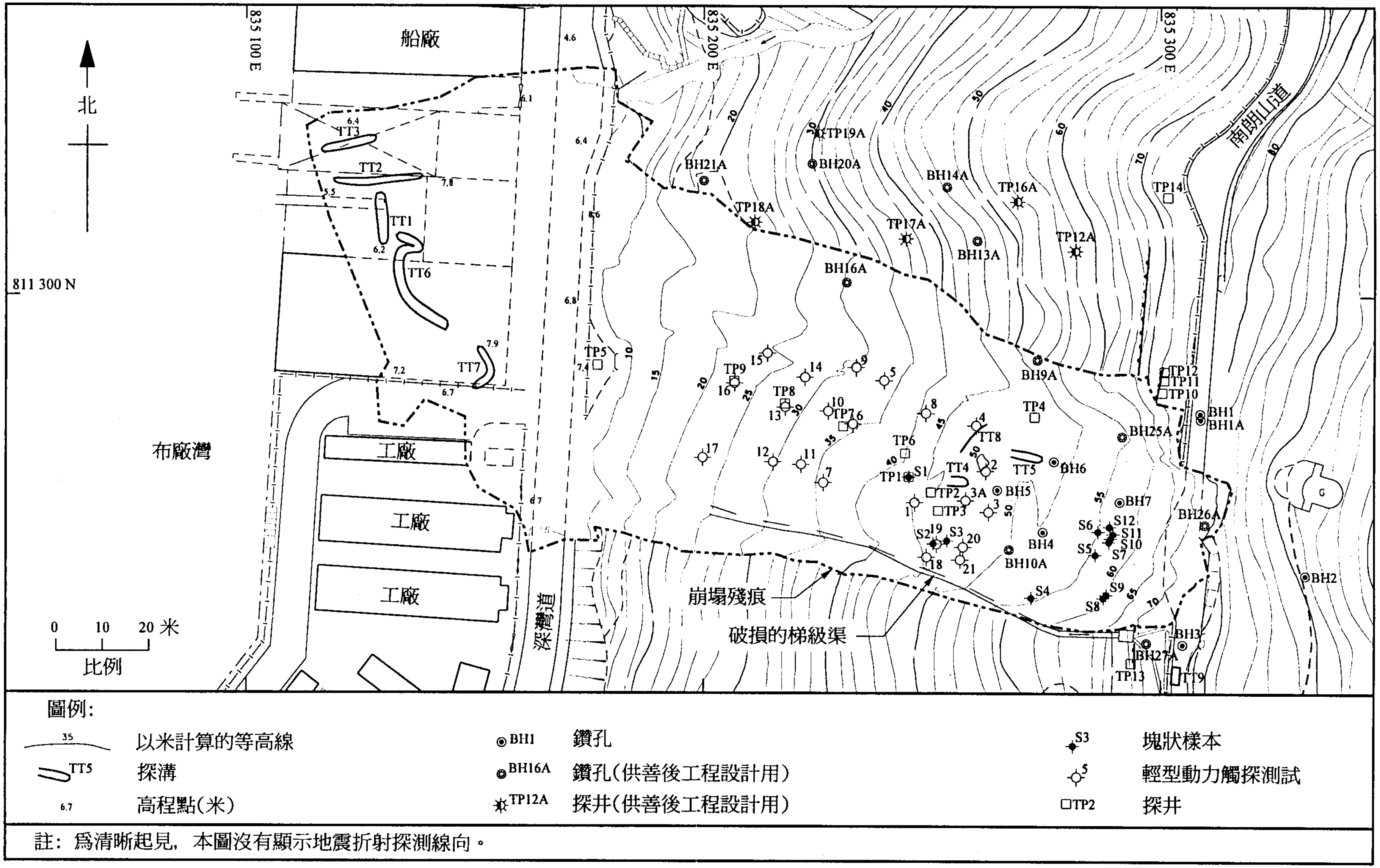
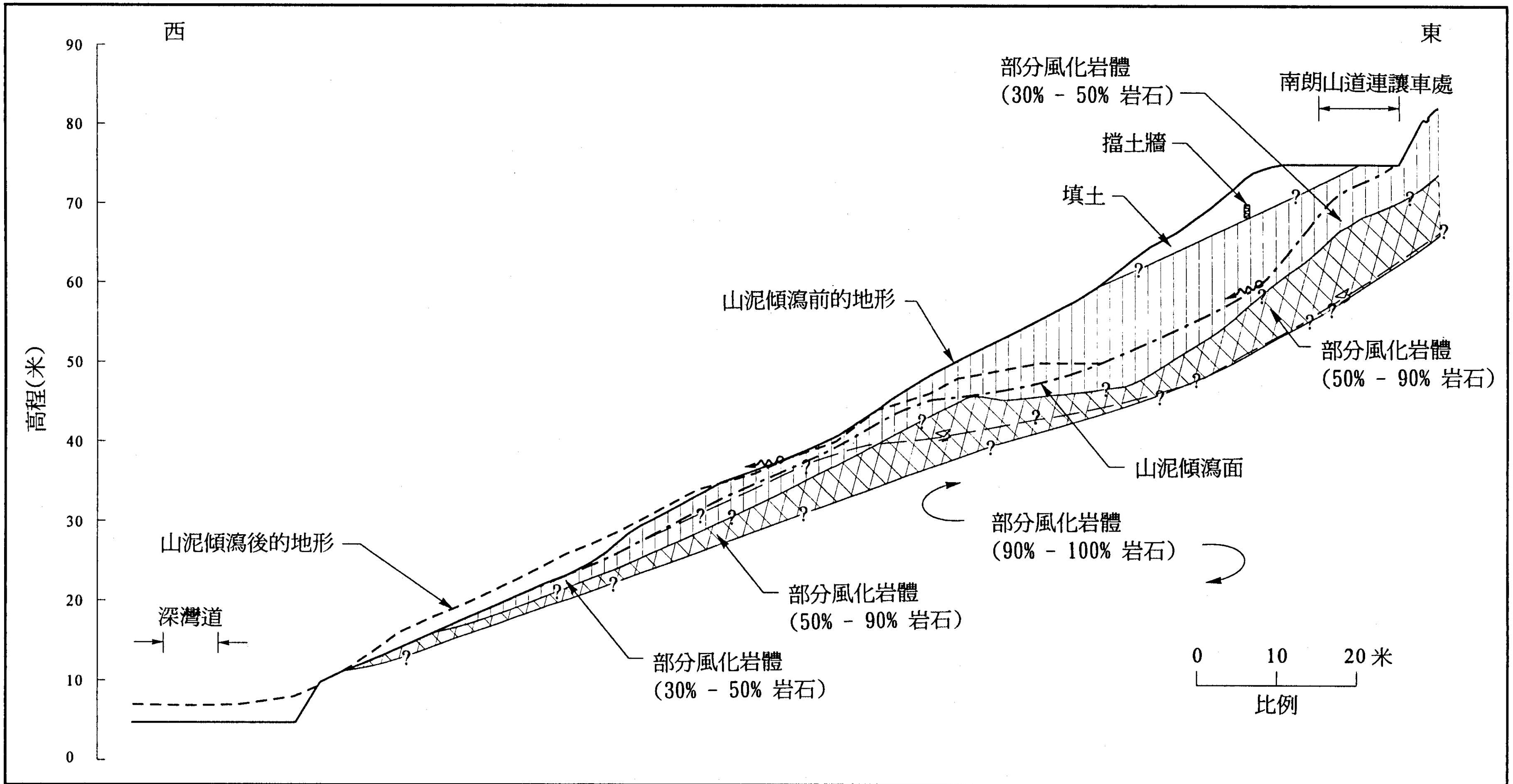


圖 10 － 場地勘探工程的位置圖



圖例:

山泥傾瀉後一星期觀察得的滲水點

在一九九五年十月二十八日觀察所得的水位

註:(1) 這剖面的位置相等於圖 7 的剖面 A - A。

(2) 填土的範圍是根據在一九六三年及一九九四年拍攝的航空照片而估定。

(3) 爲清晰起見,本圖沒有顯示坡積物的薄層。

圖 11 - 山泥傾瀉前現場的地層剖面圖

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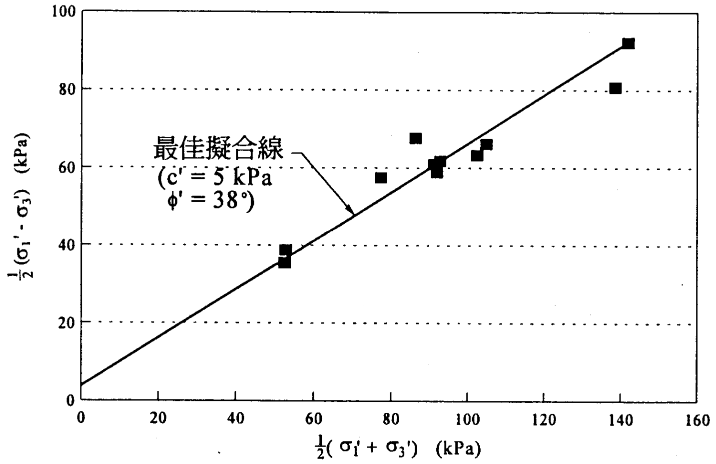
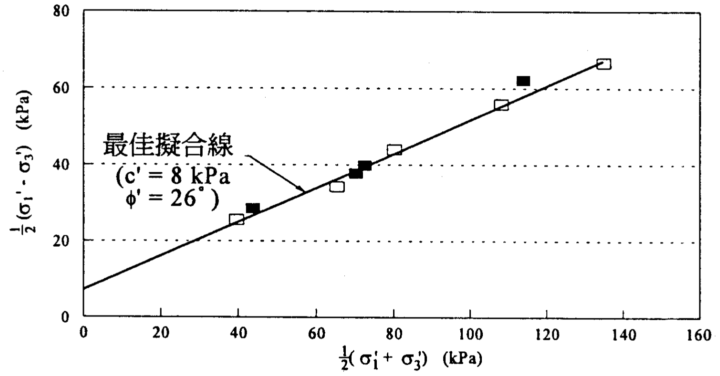
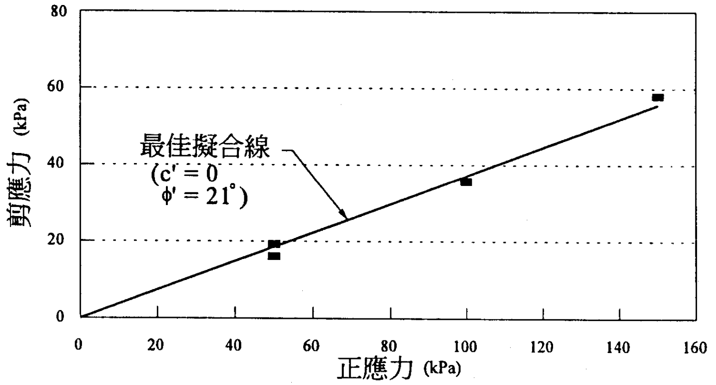


(a) 於完全風化凝灰岩進行包括量度孔隙水壓的各向同性固結不排水三軸壓縮試驗結果



(b) 於黏土層內黃褐色黏土進行包括量度孔隙水壓的各向同性固結不排水三軸壓縮試驗的結果



(c) 於黏土層內白黏土擦痕表面進行直接剪切試驗的結果

圖例:

σ_1'	最大有效主應力	■	未受擾動的塊狀樣本
σ_3'	最小有效主應力	□	重新塑造的樣本
c'	表面黏聚力	ϕ'	抗剪角

註: 圖(a)及圖(b)的數據點取自測試結果中最高 σ_1' / σ_3' 比率。
圖(c)的數據點取自測試完畢時的測試結果。

圖 12 – 物質的抗剪強度

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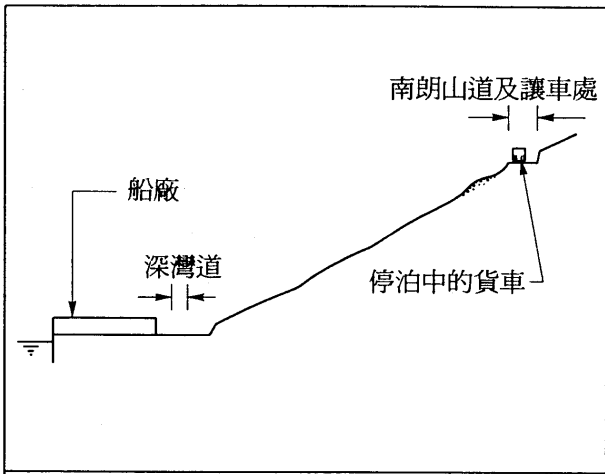
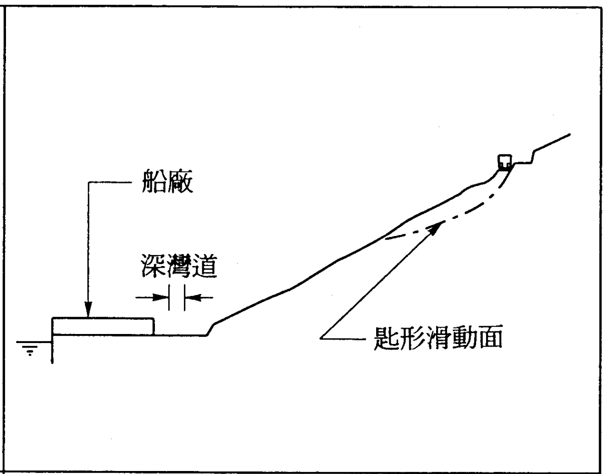
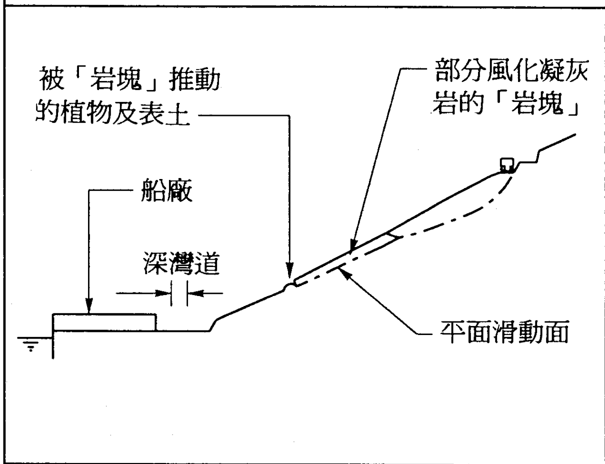


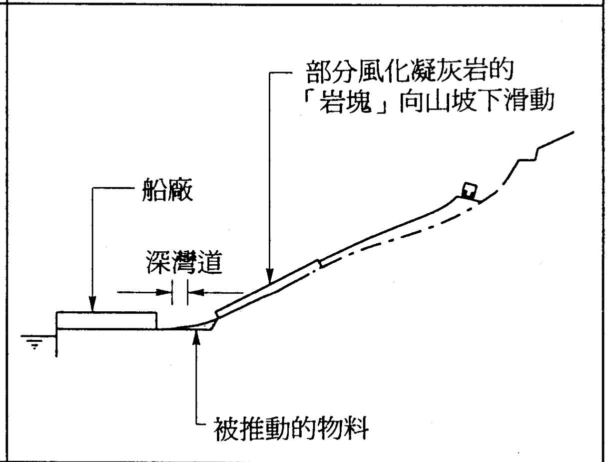
(a) 在發生主要山泥傾瀉前，讓車處底下填土堤發生輕微崩塌(晚上十一時至凌晨三時之間)



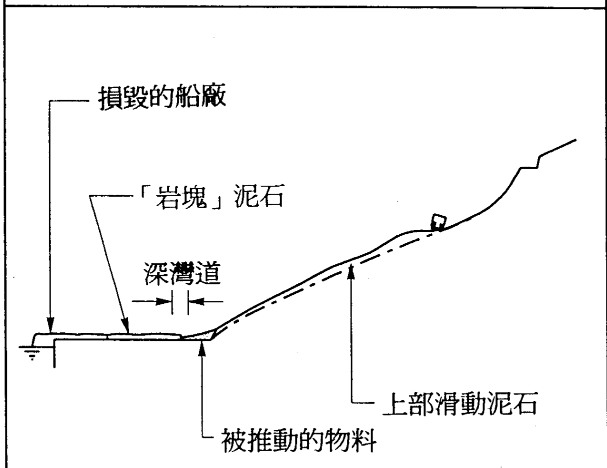
(b) 匙形滑動由山坡上部開始(約凌晨四時)



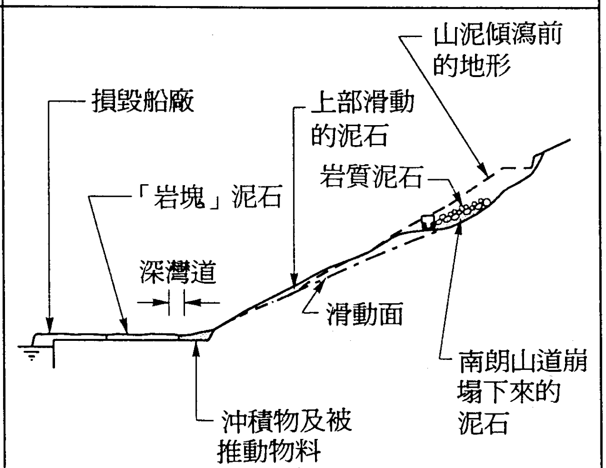
(c) 上部滑動的泥石觸發山坡下部另一次滑動(約凌晨四時)



(d) 被推動的物料散落在山腳(約凌晨四時)



(e)「岩塊」泥石摧毀船廠後停下來(約凌晨四時)



(f) 其後一次崩塌推倒南朗山道，之後再發生一次崩塌，岩石崩塌下來(約凌晨四時三十分)

圖 13 - 山泥傾瀉過程的圖解說明

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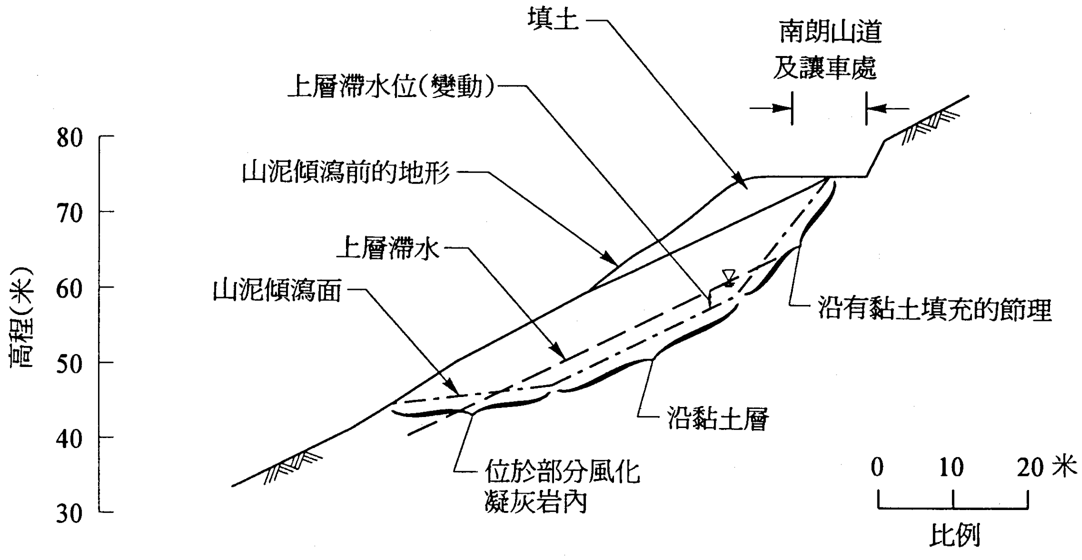
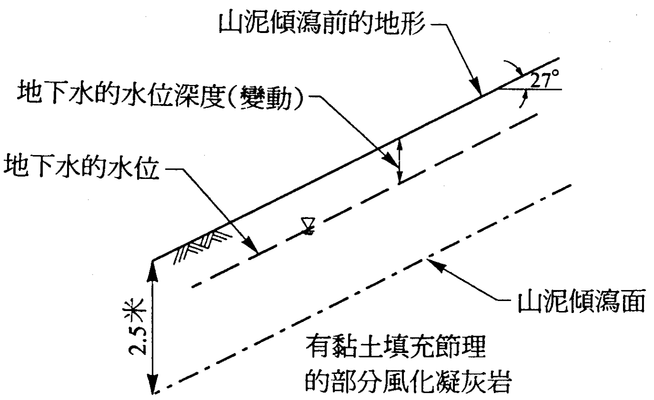


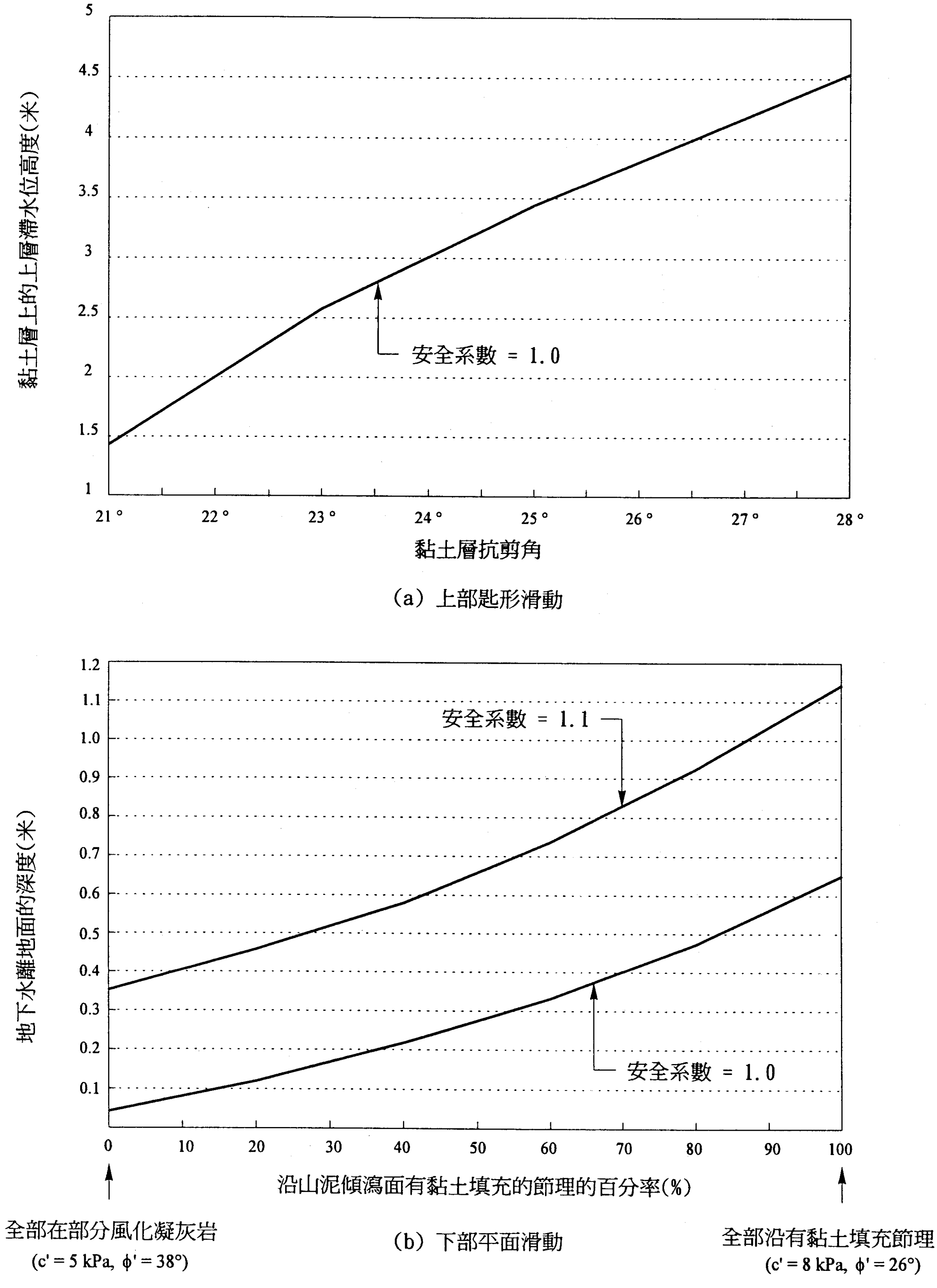
(a) 上部匙形滑動



(b) 下部平面滑動

物料		表面黏聚力 c' (kPa)	抗剪角 φ'	單元重量 (kN/m^3)
部分風化凝灰岩		5	38°	19
黏土層	最高強度	8	26°	19
	有擦痕	0	21°	19
有黏土填充節理		8	26°	19

圖14 – 用作斜坡穩定性分析的事發地點代表性剖面圖



註： 爲方便比較起見，分析匙形滑動時假設c'= 0 。
以黏土最高強度(c'= 8kPa, ϕ'= 26°)作分析，安全系數爲1.0時，上層滯水位高度爲4.8米。

圖 15 - 山坡穩定性分析結果

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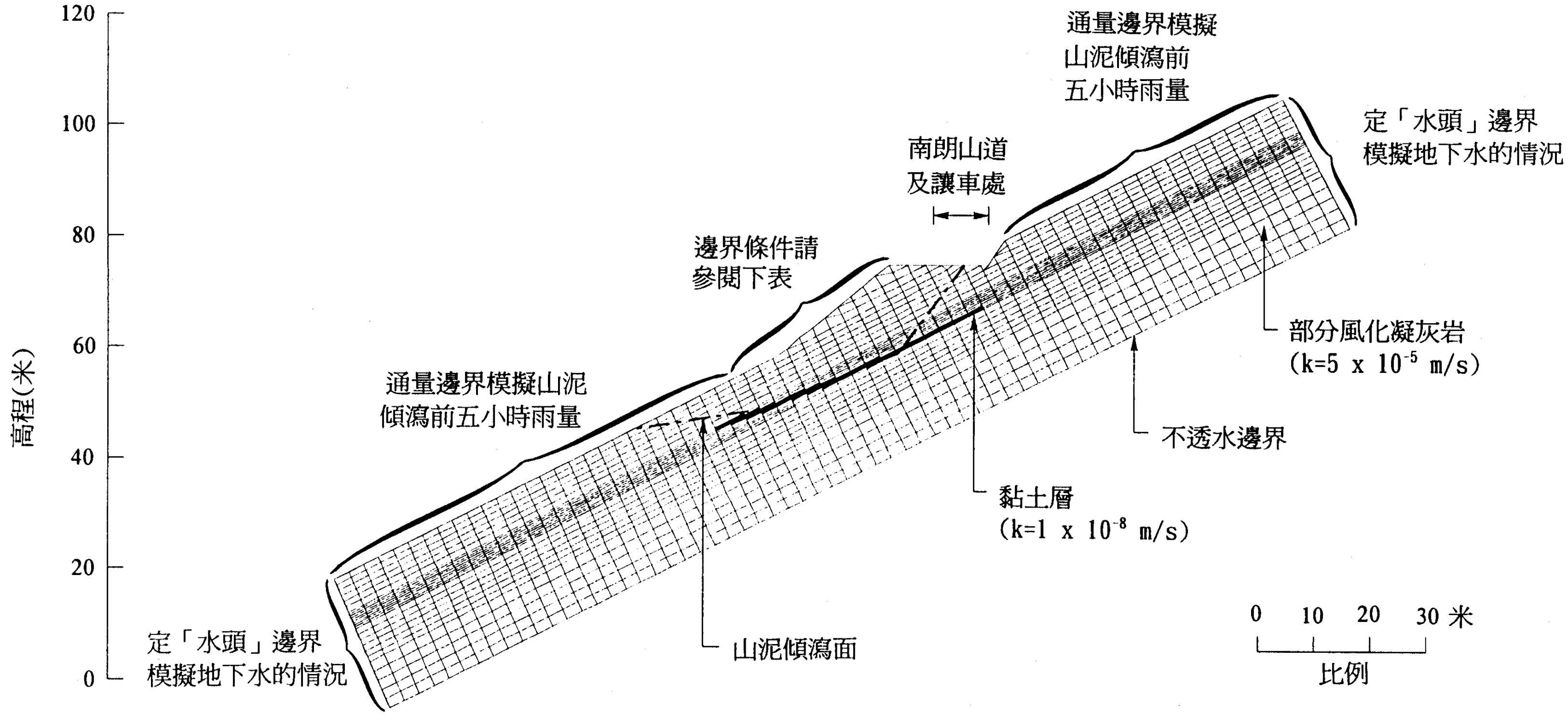

滲水分析	邊界條件
模擬在山泥傾瀉前五小時雨水滲透	通量邊界
模擬水從南朗山道向下奔流	定「水頭」邊界

圖例:

k　　滲透系數

註: 為取得滲流分析的初期情況, 先假定山坡面為通量界線,
模擬山泥傾瀉前十三日雨水滲透(由一九九五年七月三十一日至八月十二日)。

圖16 － 滲流分析的解析模型

<u>照片</u>

照片編號		頁數

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照片1 - 一九九五年八月十三日的山泥傾瀉照片

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照片2 - 填海區上的「岩塊」泥石

照片3 - 崩塌山坡腳的泥石

註：照片位置見圖7。

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照片4 - 在山泥傾瀉殘痕上帶有填料及廢料的泥石

照片5 - 山泥傾瀉殘痕下部的擋土牆碎塊

註：照片位置見圖7。

照片6 - 山泥傾瀉殘痕上部的瀝青路面(1.5米 x 1米)

照片7 - 山泥傾瀉殘痕上的
岩質泥石

註：照片位置見圖7。

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

斜坡岩土工程手冊(1998)，308頁(1984年英文版的中文譯本)。

Highway Slope Manual (2000), 114 p.

GEOGUIDES

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

GEOSPECS

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

GEO PUBLICATIONS

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

GEOLOGICAL PUBLICATIONS

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

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

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