INVESTIGATION OF SOME MAJOR SLOPE FAILURES BETWEEN 1992 AND 1995

GEO REPORT No. 52

Y.C. Chan, W.K. Pun,
H.N. Wong, A.C.O. Li & K.C. Yeo

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
HONG KONG

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Geotechnical Engineering Office,
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PREFACE

In keeping with our policy of releasing information of general technical interest, we make available some of our internal reports in a series of publications termed the GEO Report series. The reports in this series, of which this is one, are selected from a wide range of reports produced by the staff of the Office and our consultants. A charge is made to cover the cost of printing.

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

A.W. Malone
Principal Government Geotechnical Engineer
August 1996

EXPLANATORY NOTE

This GEO Report consists of six Advisory Reports on investigation of some major slope failures between 1992 and 1995.

They are presented in six separate sections in this Report. Their titles are as follows:

SECTION 1 : REPORT ON THE INVESTIGATION OF THE 8 MAY 1992 BAGUIO LANDSLIDE

Y.C. Chan & W.K. Pun

This report was originally produced in September 1992
as GEO Advisory Report No.ADR 26/92

FOREWORD

Following the landslide at Baguio Villas on 8 May 1992, the Geotechnical Engineering Office carried out an investigation into the landslide.

The investigation was carried out by a team in the Special Projects Division led by Mr Y.C. Chan and including Messrs J.M. Shen and W.K. Pun. Topographic survey of the landslide area was carried out by the Survey Division, CED. The Photogrammetric Unit of BLD carried out aerial photogrammetry. Mr K.W. Lai of the Planning Division mapped the failure scar, initially with safety measures provided by the CAS. API service was provided by Mr A. Hansen of the Planning Division. Messrs T.C.F. Chan and Y.K. Shiu of the Advisory Division helped to collect background information and Mr T. Blower, also of the Advisory Division, helped to collect information from eye-witnesses.

Y.C. Chan
Chief Geotechnical Engineer/Special Projects

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1. INTRODUCTION

On 8 May 1992, a landslide occurred on the hillside above Baguio Villas, Pokfulam, Hong Kong (Figure 1). The landslide involved the collapse of a masonry retaining wall and part of a fill platform behind the wall. Debris from the collapse travelled from Pokfulam Temporary Housing Area (THA) at an elevation of about 149 mPD (metres above Principal Datum) down a natural gully to the south of Block 26, Upper Baguio Villas to Victoria Road at about 58 mPD. The debris flow crossed Victoria Road and continued down to Blocks 41 to 44 of Lower Baguio Villas, coming to rest on the podium and car parking floors of the building at elevations between 34 mPD and 40 mPD (Figures 2 and 3). A child within an apartment of Block 44 and a government engineer on a duty inspection on the podium of Block 44 were fatally injured in the landslide.

The fill platform that collapsed was part of the sitting-out area of the THA. The age of the fill platform is not known but its existence can be traced back to 1924. It may well date from the development of the site as a farm in the 1880's. The platform remained within the ownership of the Dairy Farm Company Limited until 1988 when the estate was returned to the government. Part of the estate was then allocated to the Housing Department for the construction of a Temporary Housing Area. The masonry retaining wall that supported the fill platform was not registered in the Catalogue of Slopes prepared by the government in 1977 and 1978.

The Geotechnical Engineering Office (GEO) was charged with the investigation of the Baguio landslide. The investigation included site surveys, collection of documentary information, eye-witness interviews, stability analyses and consideration of the probable causes of the landslide. This report summarises the findings of the investigation.

2. POKFULAM TEMPORARY HOUSING AREA

The THA was constructed between 1988 and 1990. At the location of the fill platform which collapsed on 8 May 1992, work for the THA included limited filling (800 mm maximum thickness) to about 2 m back from the crest of the masonry retaining wall, construction of new pavement slabs, fencing and seating on part of the fill platform, building of a latrine and installation of water pipes, sewers and stormwater drains. Details of the sitting-out area prior to the landslide have been reconstructed and are shown in Figures 4 and 5. An elevation of the masonry retaining wall and the fill platform is given in Figure 6. Layouts of the sewerage system and water pipes are presented in Figures 7 and 8. The stormwater drainage system is depicted in Figures 9 and 10.

The drainage system shown in Figure 9 in the area enclosed by the chain link fence was installed by Housing Department. That outside is the old farm drainage system.

3. RAINFALL

The first six days of May 1992 were dry at Baguio. On 7 May, a total of 53.5 mm of rain was recorded by an automatic raingauge (H03) on the roof of Block 44, Baguio Villas.

On 8 May, rain was heavy over the whole of the territory (Figure 11). The 24-hour rainfall was heaviest in the western part of Hong Kong Island where Baguio Villas is located.

The five-minute rainfall intensities recorded by the raingauge H03 on 8 May are shown in Figure 12. On that day, heavy rain started at about 6 a.m. and continued until shortly before 9 a.m. A total of 216 mm of rain was recorded in this period, out of which about 180 mm fell in two hours. The two-hour rainfall corresponds to a 1-in-30-year event according to statistical data from the Royal Observatory. The peak hourly rainfall was 102.5 mm between about 7 and 8 a.m.

Short bursts of rain resumed at about 11 a.m., intensifying at around 1:40 p.m.; 40 mm of rain fell in the following 20 minutes. By 2 p.m., the cumulative rainfall since 6 a.m. amounted to about 295 mm (see Figure 13).

A short spell of very intense rainfall commenced at about 3:10 p.m. and 22.5 mm of rain were recorded in the five-minute period ending at 3:20 p.m. This corresponds to a 1-in-60-year event.

4. SEQUENCE OF EVENTS RELATING TO THE LANDSLIDE

To assist in reconstructing the events of 8 May, the Geotechnical Engineering Office sent out 700 questionnaires to residents of Baguio Villas and the Pokfulam THA on 11 May. In total, 112 questionnaires were returned and 24 persons were contacted through other channels. Subsequently, 58 persons were interviewed for further information. Apart from the questionnaires and witness statements, 96 photographs and 3 video tapes were obtained.

The following sequence of events has been reconstructed from the eye-witness reports and photographs and site surveys carried out after the landslide.

Despite the exceptionally heavy flow of surface water, the natural gully, the masonry retaining wall and the fill platform survived the rainstorm in the morning of 8 May. By about 11 a.m. the flow in the natural gully had abated substantially.

Intense rainfall resumed at around 1:40 p.m. and the central part of the masonry retaining wall (zone (1) in Figure 14) collapsed at about 2 p.m. This released into the natural gully about 250 cubic metres of material comprising stone blocks, soil and some building debris which included pipes and pieces of concrete.

Thereafter, the unsupported fill of the THA platform gradually collapsed. About 450 cubic metres of material are estimated to have been lost from the fill platform in this phase of gradual slumping. The approximate extent of ground affected in this phase is shown as zone (2) in Figure 14.

During the period of very intense rainfall which started at about 3:10 p.m., the northern part of the fill platform and the side slope of the upper farm platform (zone (3) in Figure 14) collapsed and released about 1 000 cubic metres of material into the natural gully. The landslide debris was fluidised by the large volume of surface water entering the natural gully. The fluid debris was carried down the gully, scouring out more material along its path. The debris accumulated on Victoria Road, then overflowed onto the podium of Blocks 41 to 44. The fatal injuries to two persons at the podium level of Blocks 41 to 44 were caused by this debris flow.

Although rain had stopped by about 4 p.m., landslide debris continued to flow intermittently until evening. Another 500 cubic metres of material were lost from the fill platform in this final phase of the landslide. This material accumulated on Victoria Road and on the podium and within the car parking levels of Blocks 41 to 44.

The debris flows in the afternoon of 8 May 1992 eroded a total of about 1 500 cubic metres of material from the natural gully.

5. FAILURE MECHANISMS OF THE RETAINING WALL

The landslide started with the collapse of the masonry retaining wall supporting the fill platform. No drawings of this retaining wall have been found and its nature and dimensions have had to be estimated from such evidence as is now available from site surveys, aerial photographs and interviews with former farm employees. As part of this investigation the stability of this structure has been analysed by conventional theoretical methods, assuming it was in good structural condition prior to failure. The results of the analyses are summarised in Table 1 and illustrated in Figure 15.

The calculations indicate that the masonry retaining wall could in theory have failed through loss of support if a layer of soil greater than 2 m in thickness was lost from the ground in front of the wall. This could be a result of toe erosion (Figure 15a) or slope failure due to transient high internal water pressures. Erosion is considered more likely to be the cause than slope failure and this is corroborated by the evidence of eye-witnesses who reported that at the time of the wall failure, trees and other vegetation in front of the fill platform remained intact.

Water that enters the ground will tend to seep downward. When the water meets a less permeable layer, it will tend to accumulate there to form 'perched water'. From the analyses, the masonry retaining wall could in theory have failed, without loss of ground in front of the wall, if perched water developed on a surface at 143 mPD elevation to a depth greater than 2 m (Figure 15b).

There are also combinations of lesser degrees of toe erosion and perched water that might in theory have caused the masonry retaining wall to fail (Figure 15c). Surcharge pressures from the limited filling carried out for the THA were included in the calculations, but the effect of these was found to be negligible.

The masonry retaining wall collapsed at the end of a spell of intense rain, which lasted about 20 minutes. During that period, runoff from the upper farm platform and the unpaved area behind the wall would have been concentrated onto the crest platform behind the masonry retaining wall (Figure 4). Much of the old drainage system on the crest platform was lost in the landslide. The portion of the drainage system that remained south of the landslide was found to be clogged by soil (soil which pre-dated the landslide). Even if it had not been choked, the old drainage system was probably inadequate in capacity. Overtopping of the masonry retaining wall is likely to have occurred during periods of intense rainfall, and there is evidence from an eye-witness that this occurred on the morning of 8 May, providing a mechanism for toe erosion. It seems very likely that toe erosion began in the morning and continued during the 20-minute period.

It is also possible that the 20-minute storm caused an increase in transient water pressure behind the wall. It takes time for water to percolate through the ground but infiltration can be accelerated by preferential flow paths. It is known that conditions favourable to the formation of perched water existed within the fill behind the retaining wall. A permeable fill layer with pockets of broken tiles was observed on the face of the landslide scar at an elevation of between about 143 mPD and 144 mPD. Seepage was issuing from the fill at this elevation on 9 May and issues continued from the regraded head of the landslide scar. Rapid ingress of water into the fill or into the wall during the 20-minute spell of intense rain at around 2 p.m. might have resulted from the exposure of preferential flow paths by erosion. Permeable fill layers or lenses in the fill could have provided preferential flow paths.

An examination of the remaining southern part of the masonry retaining wall after the landslide revealed that at one section, the front layer of stone blocks was missing from the top 1.5 m portion of the wall. If the part of the wall that failed on 8 May was similarly damaged, this would have reduced its stability, and it could have failed with a less severe combination of perched groundwater conditions and toe erosion than indicated by the theoretical analyses.

6. FACTORS CONTRIBUTING TO THE SCALE OF THE LANDSLIDE

Many factors contributed to the scale of the landslide at Baguio Villas on 8 May 1992, but the more important are the following :

(a) intense rainfall,

(b) large volume of loose fill behind the old masonry retaining wall, and

(c) steep natural gully in front of the old masonry retaining wall which collected runoff and directed debris to Blocks 41 to 44.

The sequence and intensity of rainfall on 7 and 8 May were crucial to the scale of the landslide. The rainfall on 7 May and intense rainfall on the morning of 8 May would probably have saturated and weakened the soil in the area behind and in front of the masonry retaining wall, making it vulnerable to surface erosion, and would probably have caused a perched water table to form in the fill. The spell of intense rain around 2 p.m. gave runoff which triggered the collapse of the masonry retaining wall. The very intense rainfall at around 3:10 p.m. led to a major failure in the unsupported fill releasing a large volume of debris into the head of the gully.

Slumping of soil from the fill platform would probably have remained gradual and limited had it not been for the rainfall at 3:10 p.m. The ground in front of the fill platform has an average gradient of about 33°. Landslide debris consisting of soil deposited on this slope would not travel far from the site of the collapse unless in a fluid condition. However, the ground in front of the fill platform is a natural gully which, although normally dry, collects surface water from adjacent areas during heavy rain. On 8 May, concentration of water in the gully, in combination with the large volume of debris released into the gully, produced a fluid debris flow which travelled down the gully and was directed onto Victoria Road and beyond to Blocks 41 to 44.

7. CONCLUSIONS

The immediate cause of the landslide at Baguio Villas on 8 May 1992 was the high intensity and sequence of rainfall. Surface soil erosion and the rapid development of perched water caused by rainfall were probably contributory factors in the failure of the masonry retaining wall. The structural condition of the wall was a further possible contributory factor. The failure of the wall removed part of the support to a large body of loose fill. Further rainfall led to failure of the unsupported fill. Debris collected in the natural gully in front of the fill platform and was liquified by concentration of surface water running off from adjacent areas. The debris flow travelled a substantial distance from the site of the collapse, coming to rest on Victoria Road and Blocks 41 to 44 of Lower Baguio Villas with fatal consequences.

LIST OF TABLES

Table 1 - Results of Stability Analyses of the Masonry Retaining Wall

Perched Water Level Above 143 mPD (m)	No Erosion of Fill	1 m of Fill Eroded	2 m of Fill Eroded
0	stable	stable	marginally stable
1	stable	marginally stable	marginally stable
2	marginally stable	marginally stable	unstable
3	unstable	unstable	unstable

Notes :
(1) The section analysed is that illustrated on Figure 5 and indicated on Figures 4 and 6.
(2) For the marginally stable condition, the stability of the retaining wall depends on its structural integrity and the shear strength of the retained soil. Critical instability mode is overturning.
(3) Within the fill behind the masonry retaining wall, there is a permeable layer in which perched water can develop. This is illustrated on Figure 15. The base of the permeable layer is at about 143 mPD.

LIST OF FIGURES

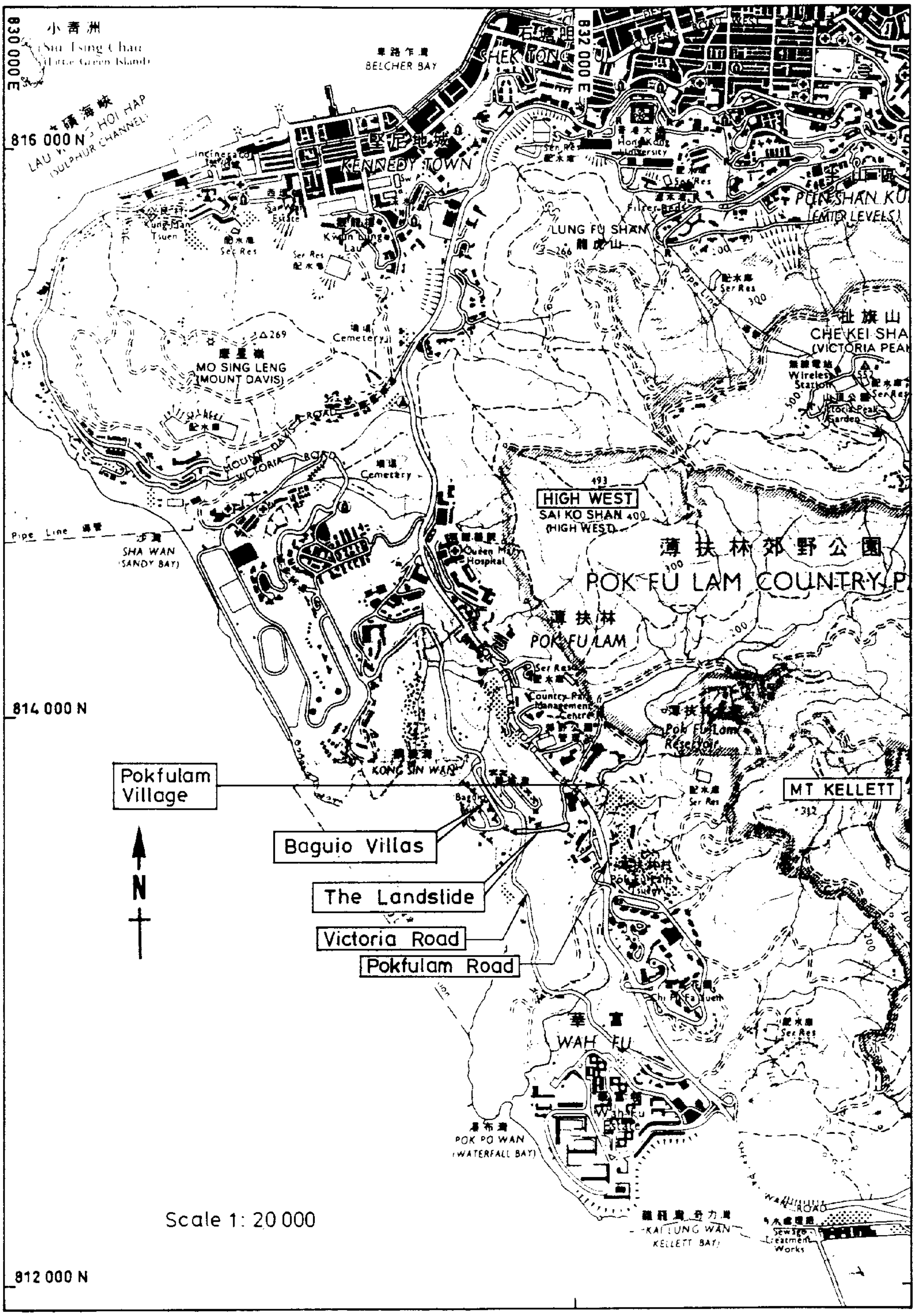


Figure 1 - General Site Location

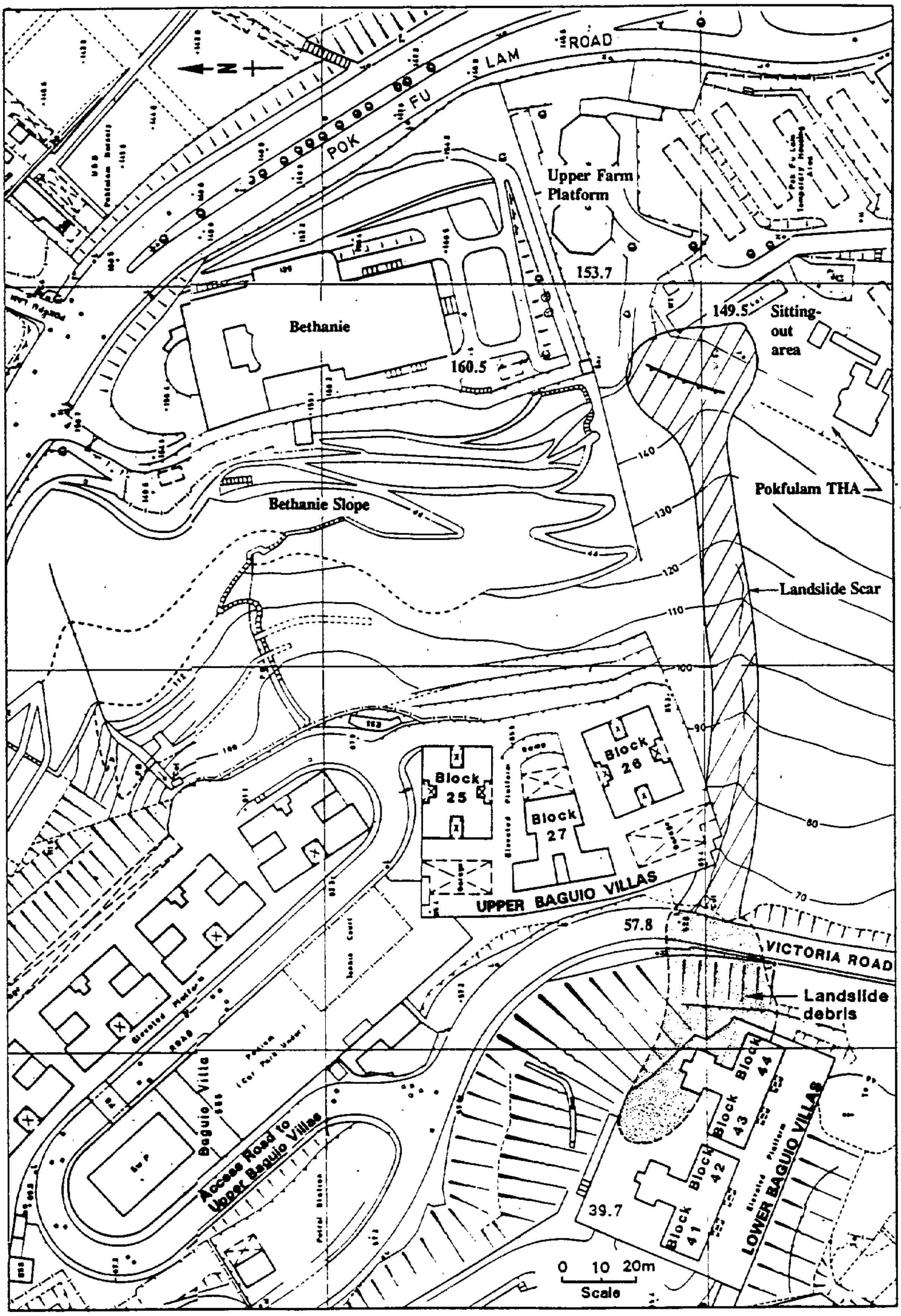


Figure 2 - Topography of Hillside and Extent of the Landslide

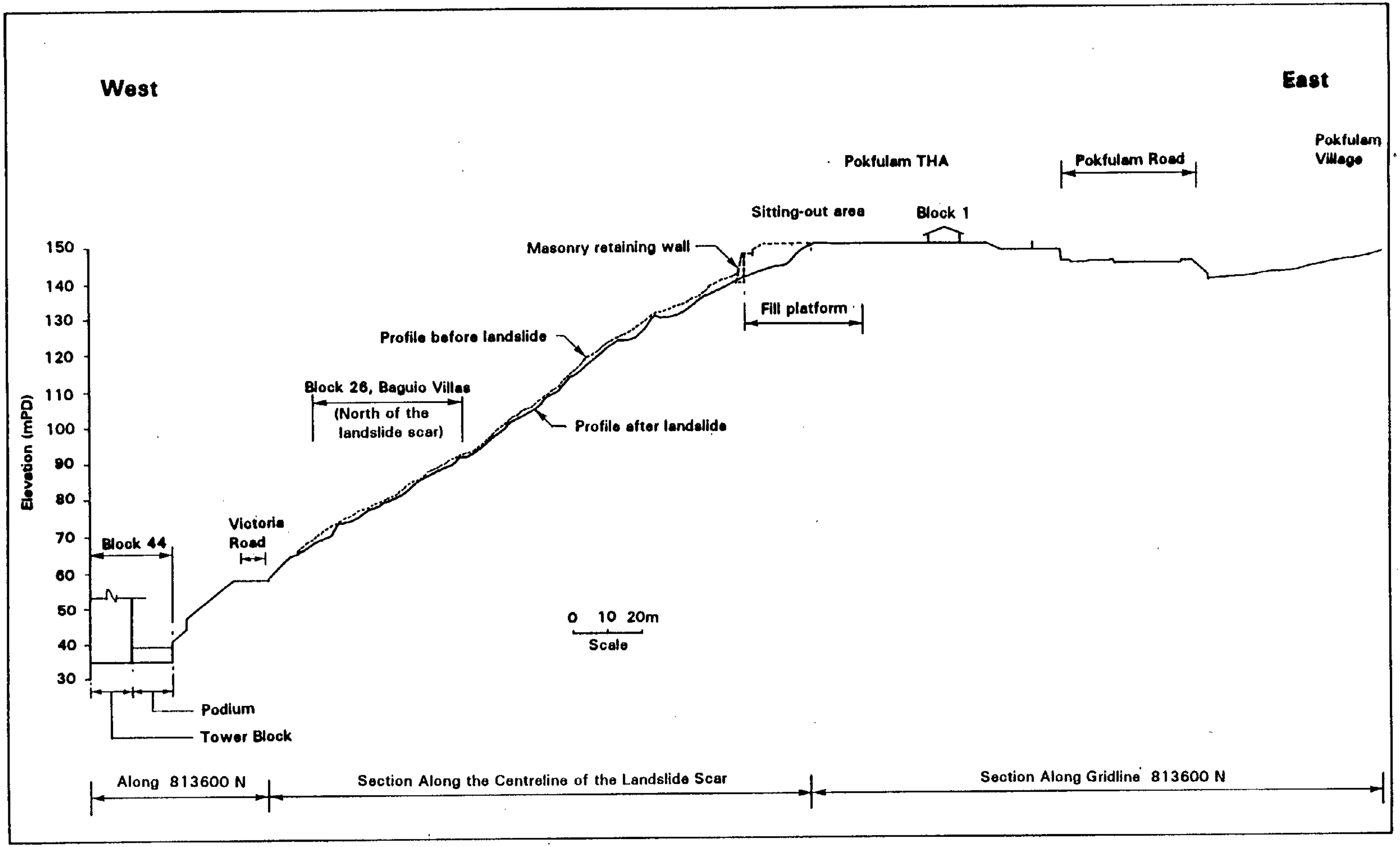


Figure 3 - Cross-section of the Hillside at the Location of the Landslide

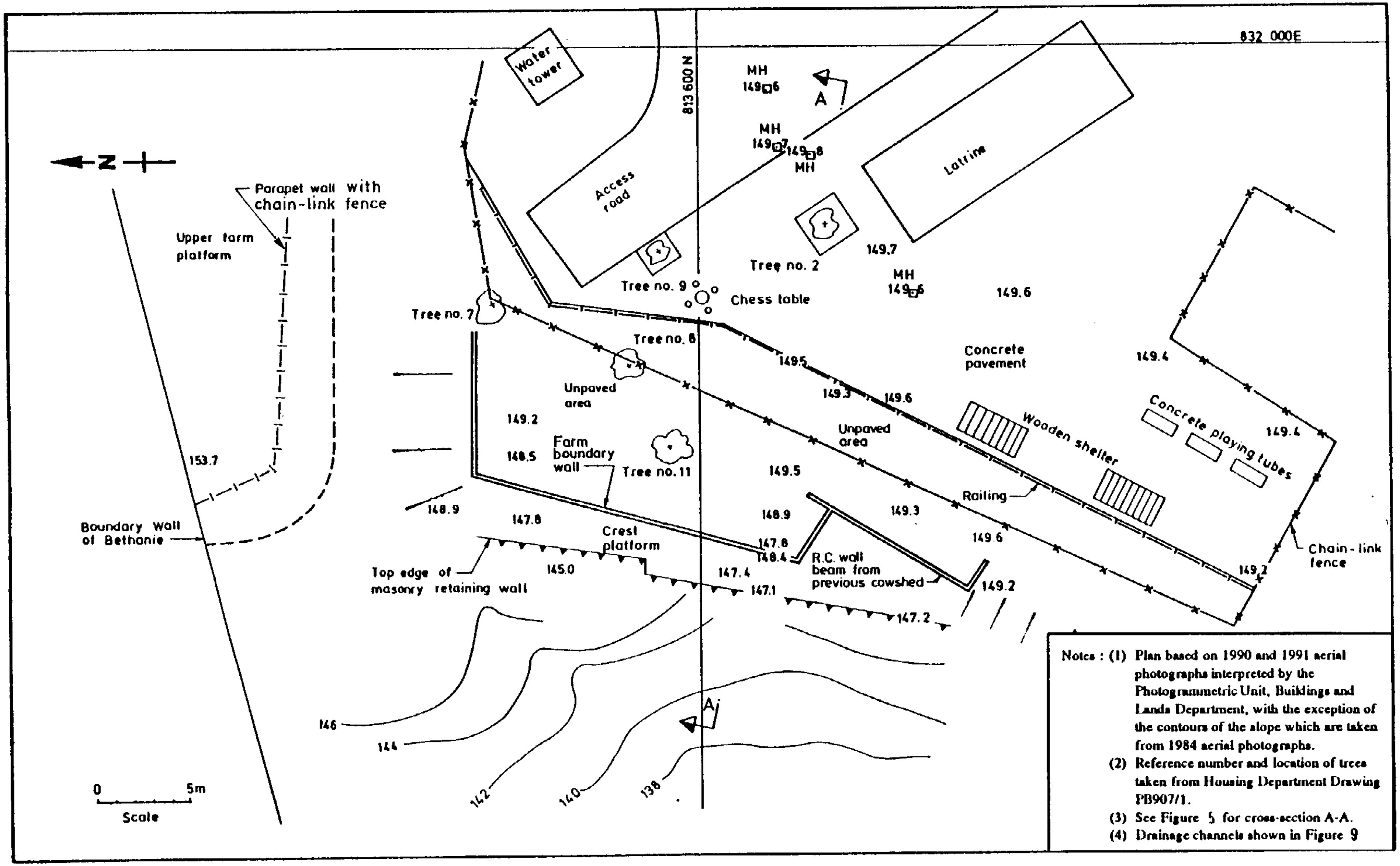


Figure 4 - Site Plan of the Pokfulam THA Sitting-out Area Before the Landslide

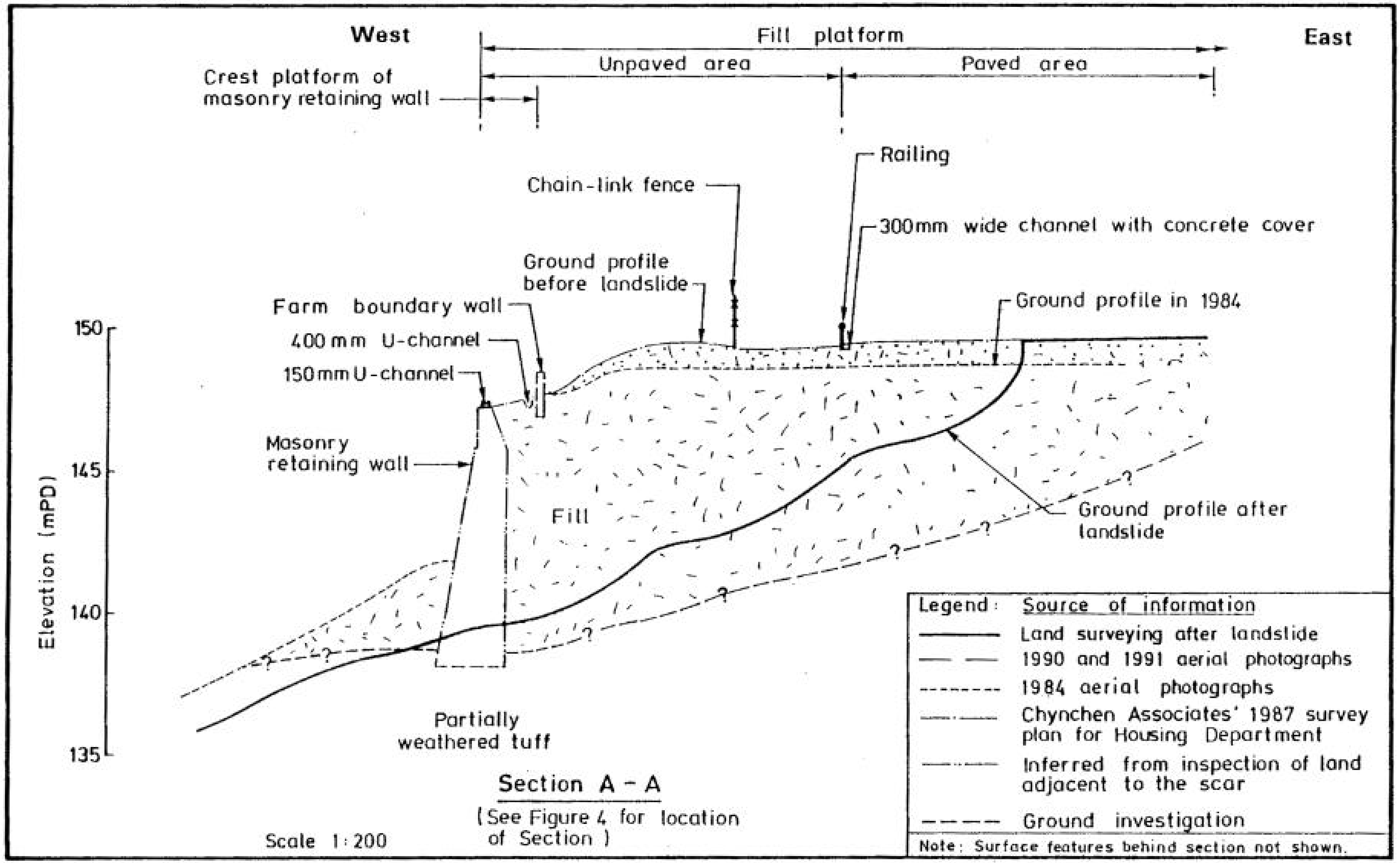
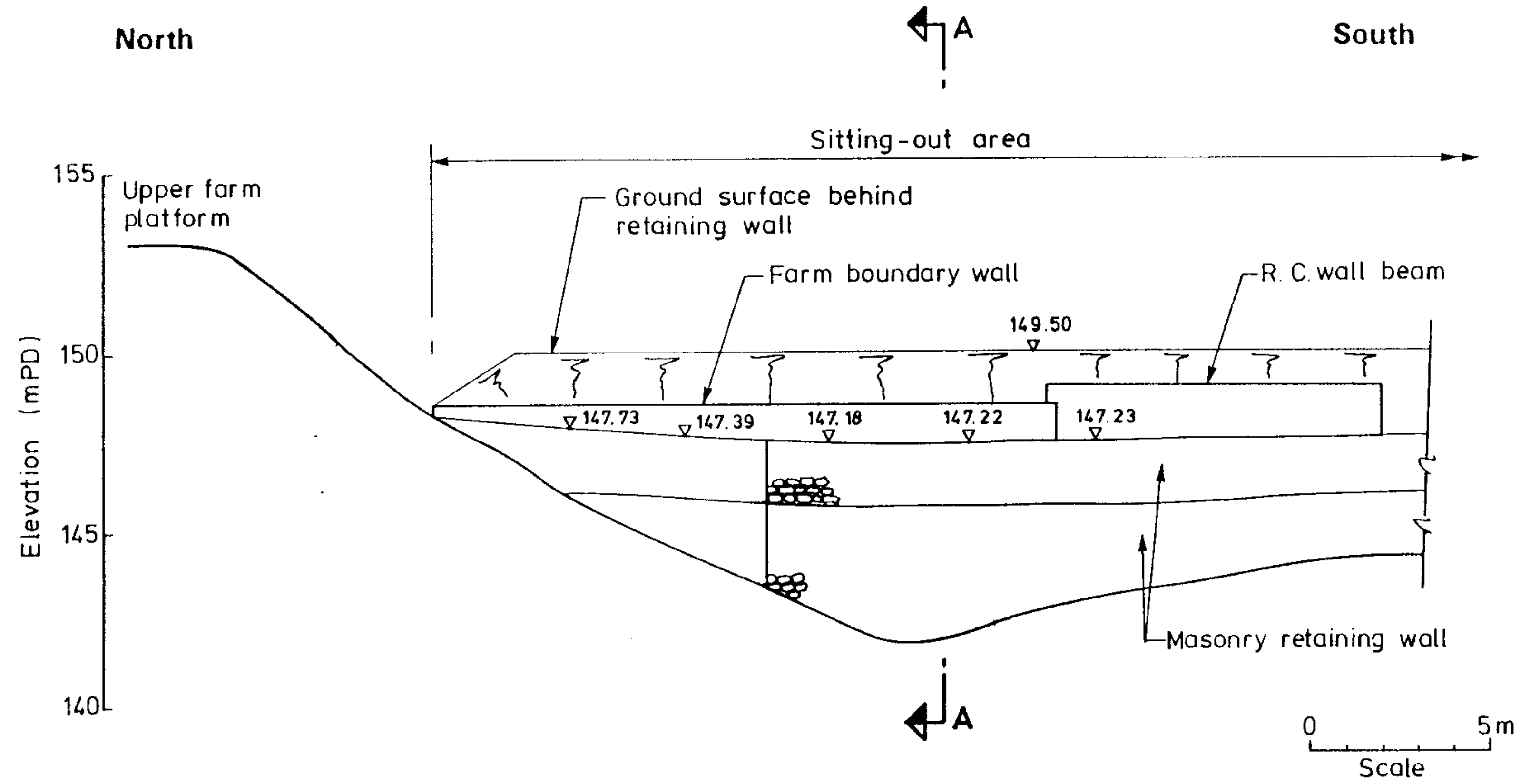
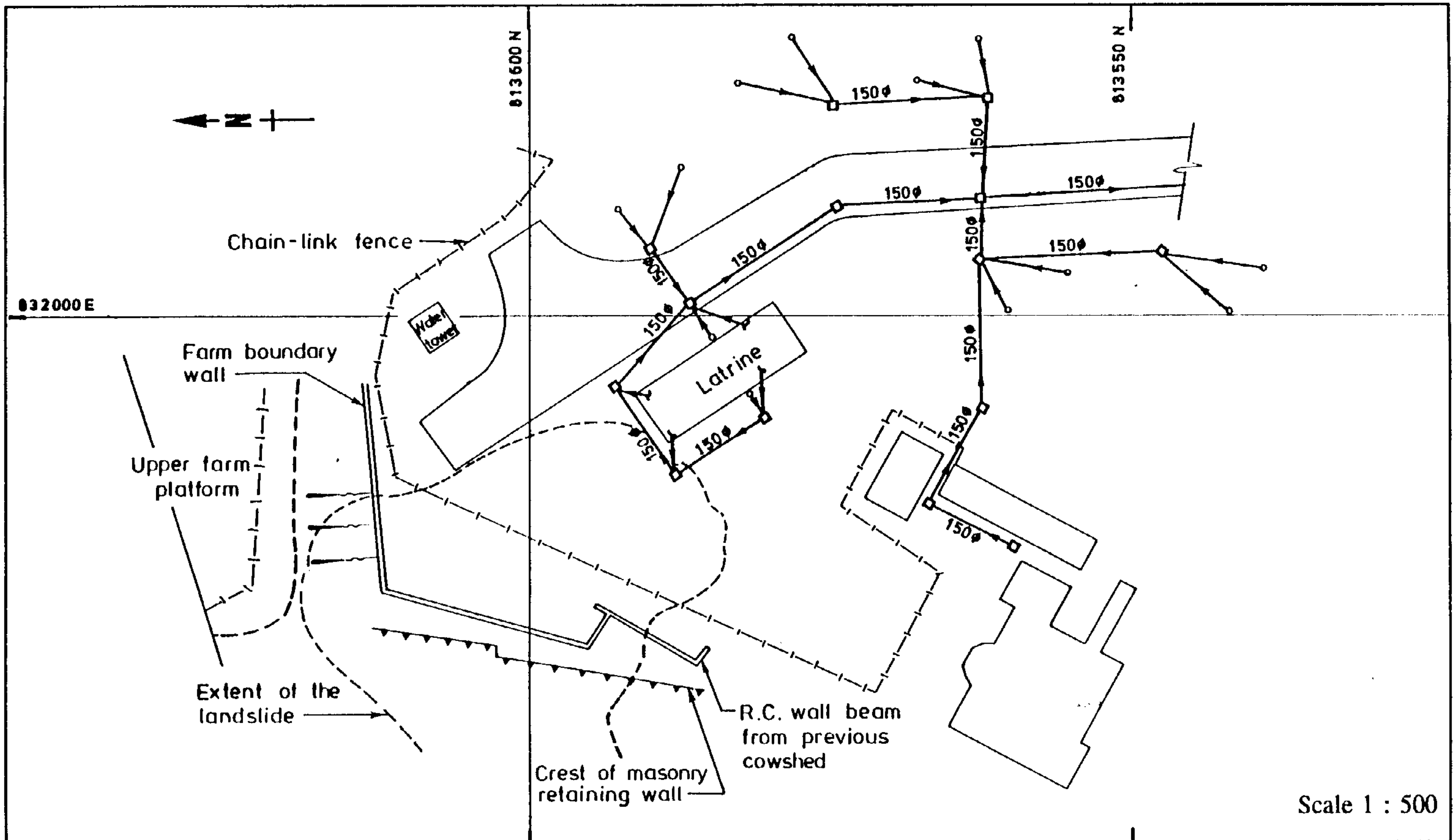


Figure 5 - Cross-section of the Sitting-out Area at Pokfulam THA



Note : 1. Topographic data based on Chynchen Associates' 1987 survey plan for Housing Department (Reference 5715 / 01).
2. Details behind the chain-link fence not shown.
3. Inferred 150mm U-channel at the crest of the retaining wall not shown.
4. See Figure 5 for cross-section A-A.

Figure 6 - Elevation of the Masonry Retaining Wall



Note : Sewerage layout based on Housing Department Drawing No. 20249/52/05H.

Figure 7 - Sewerage Layout at Pokfulam THA Sitting-out Area

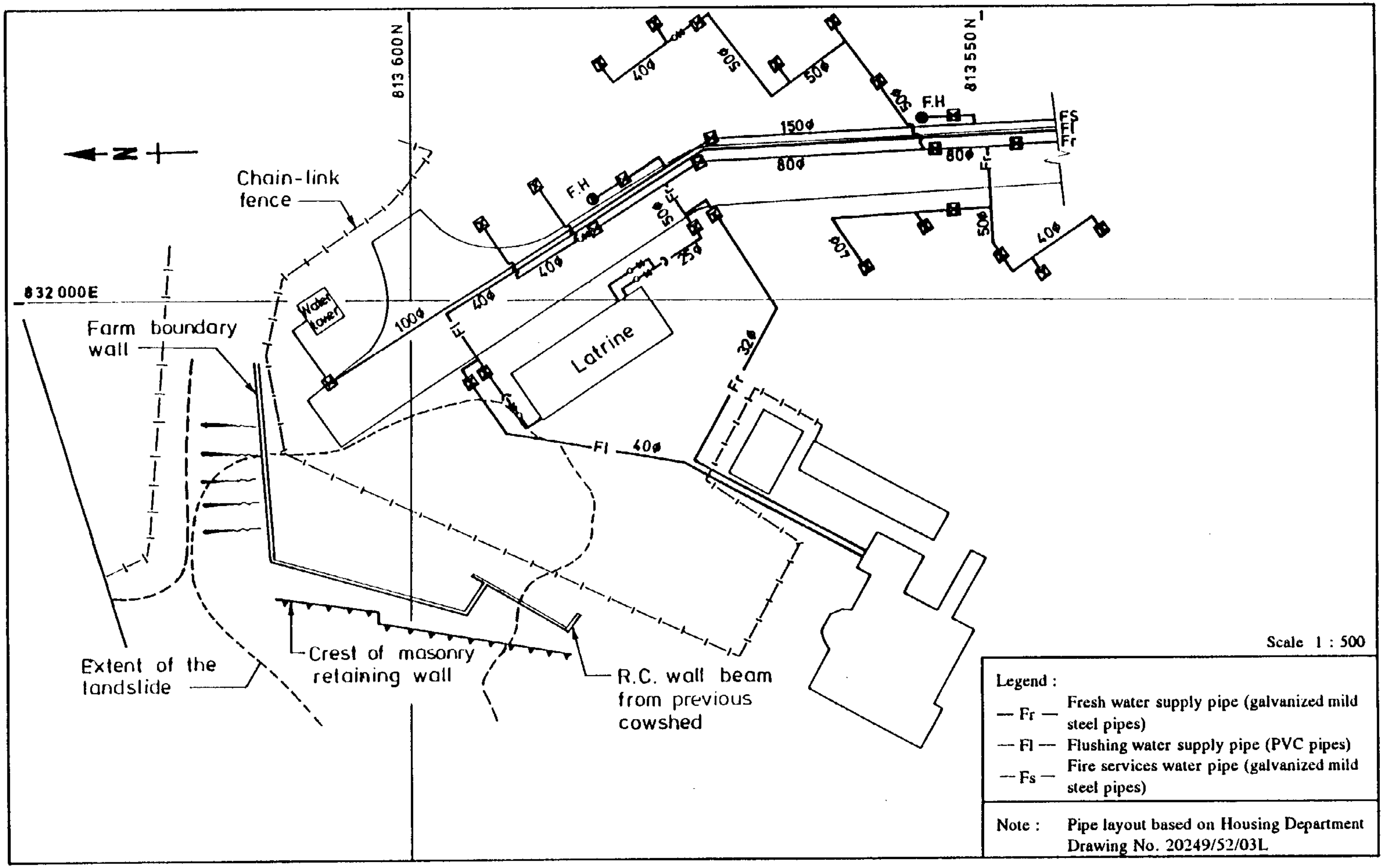


Figure 8 - Water Supply Layout at Pokfulam THA Sitting-out Area

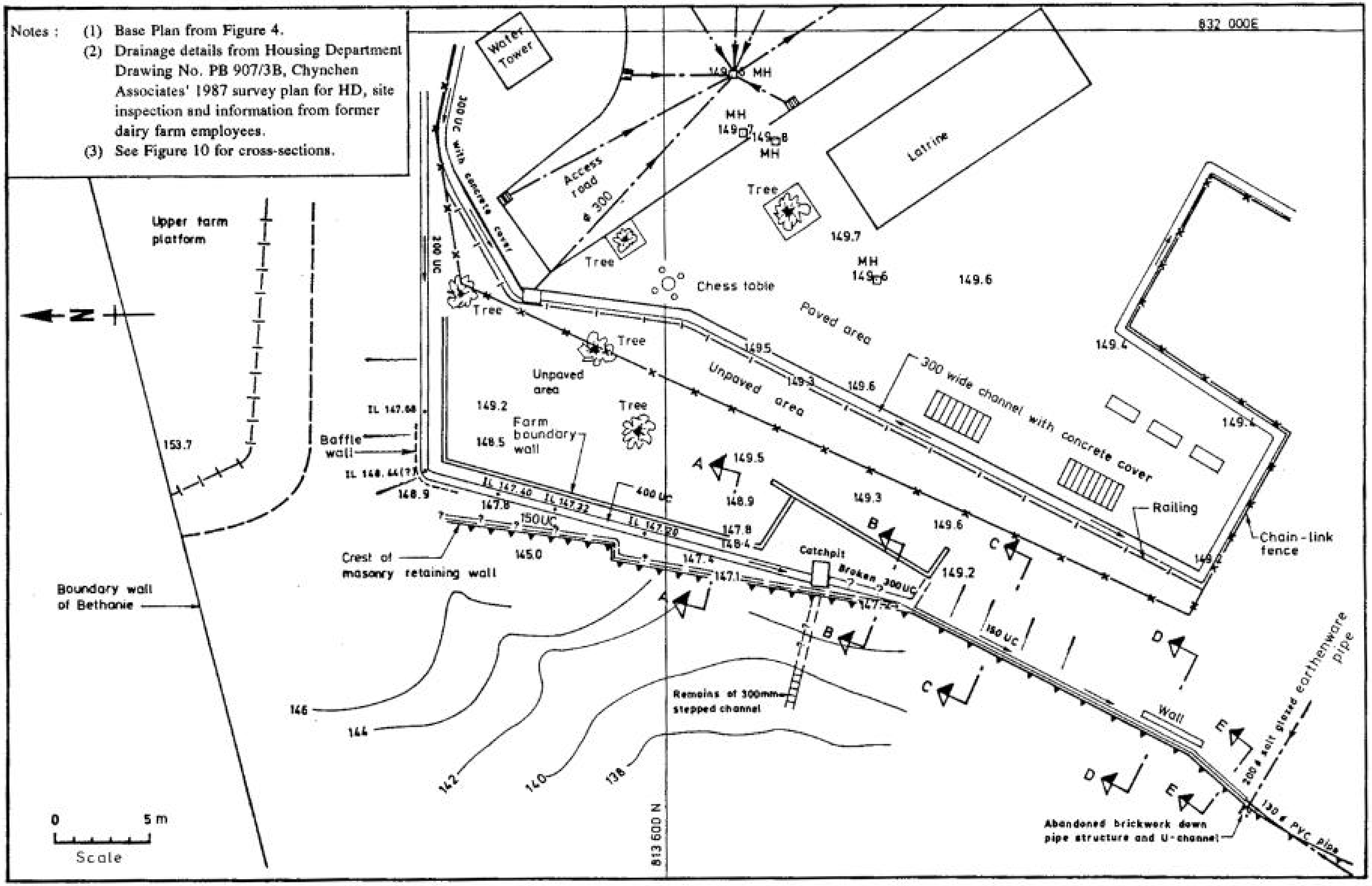
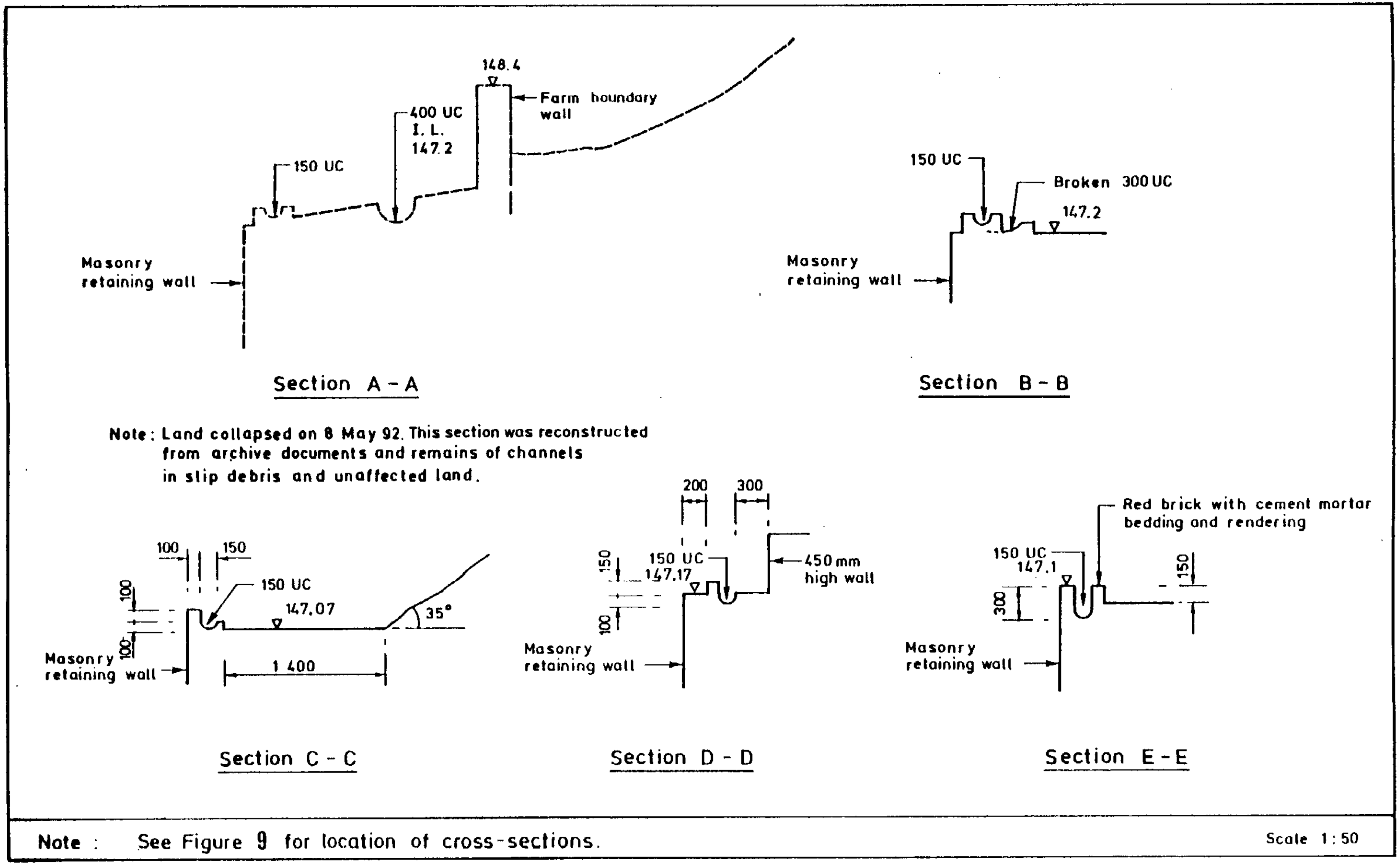


Figure 9 - Stormwater Drainage System of the Fill Platform



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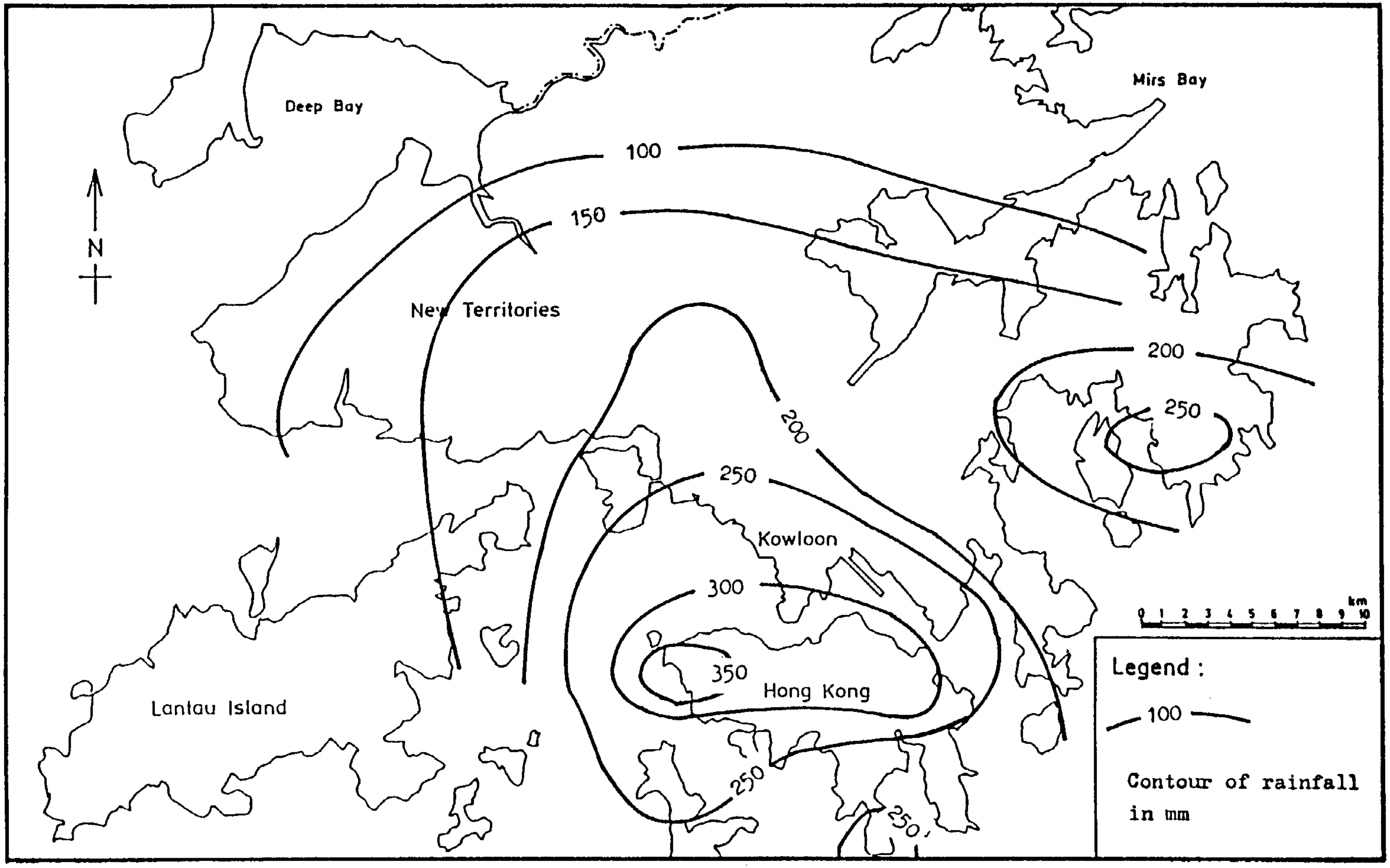


Figure 11 - Isohyets of 24-hour Rainfall on 8 May 1992

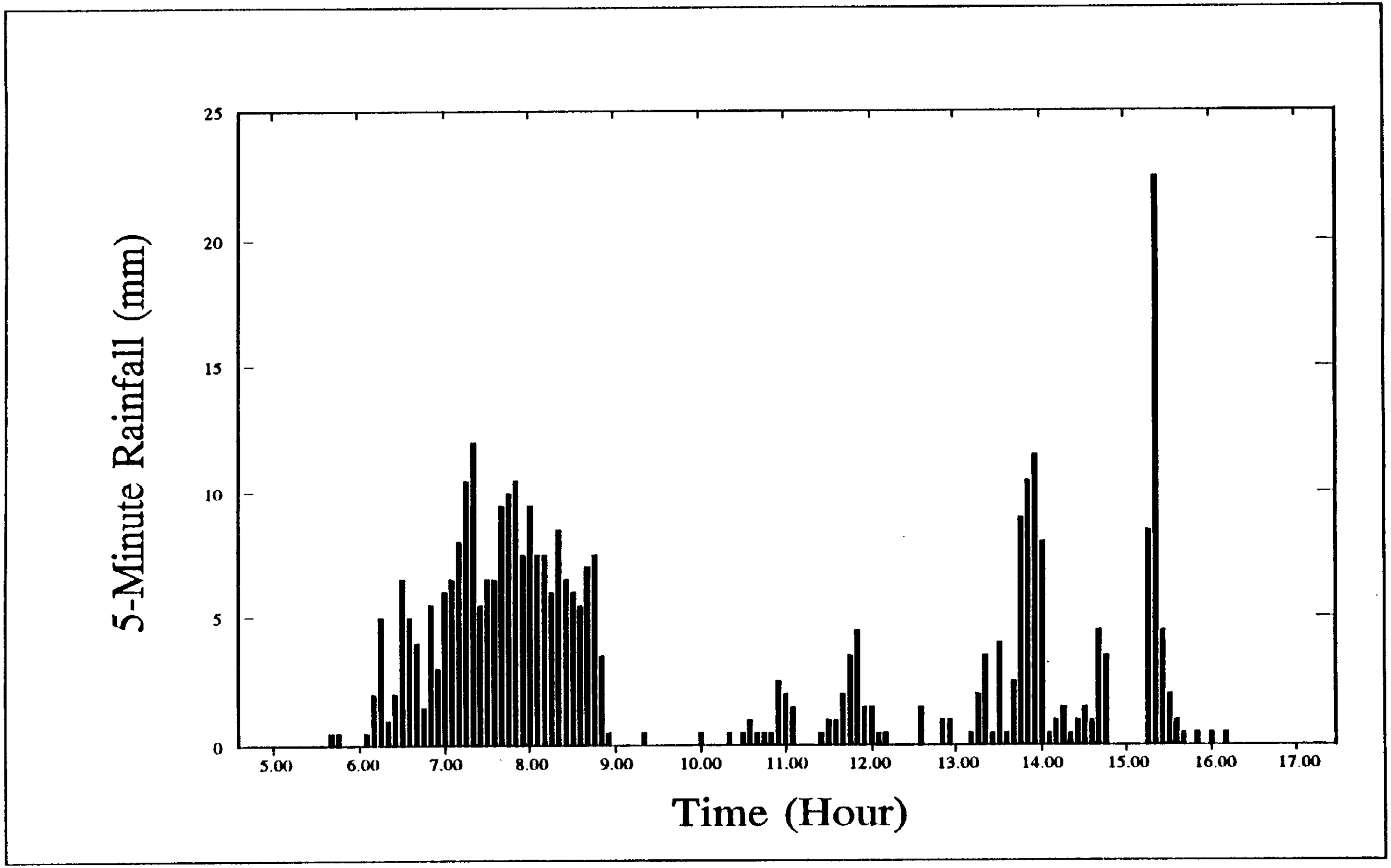


Figure 12 - Rainfall Recorded at Raingauge H03 at 5-minute Intervals on 8 May 1992

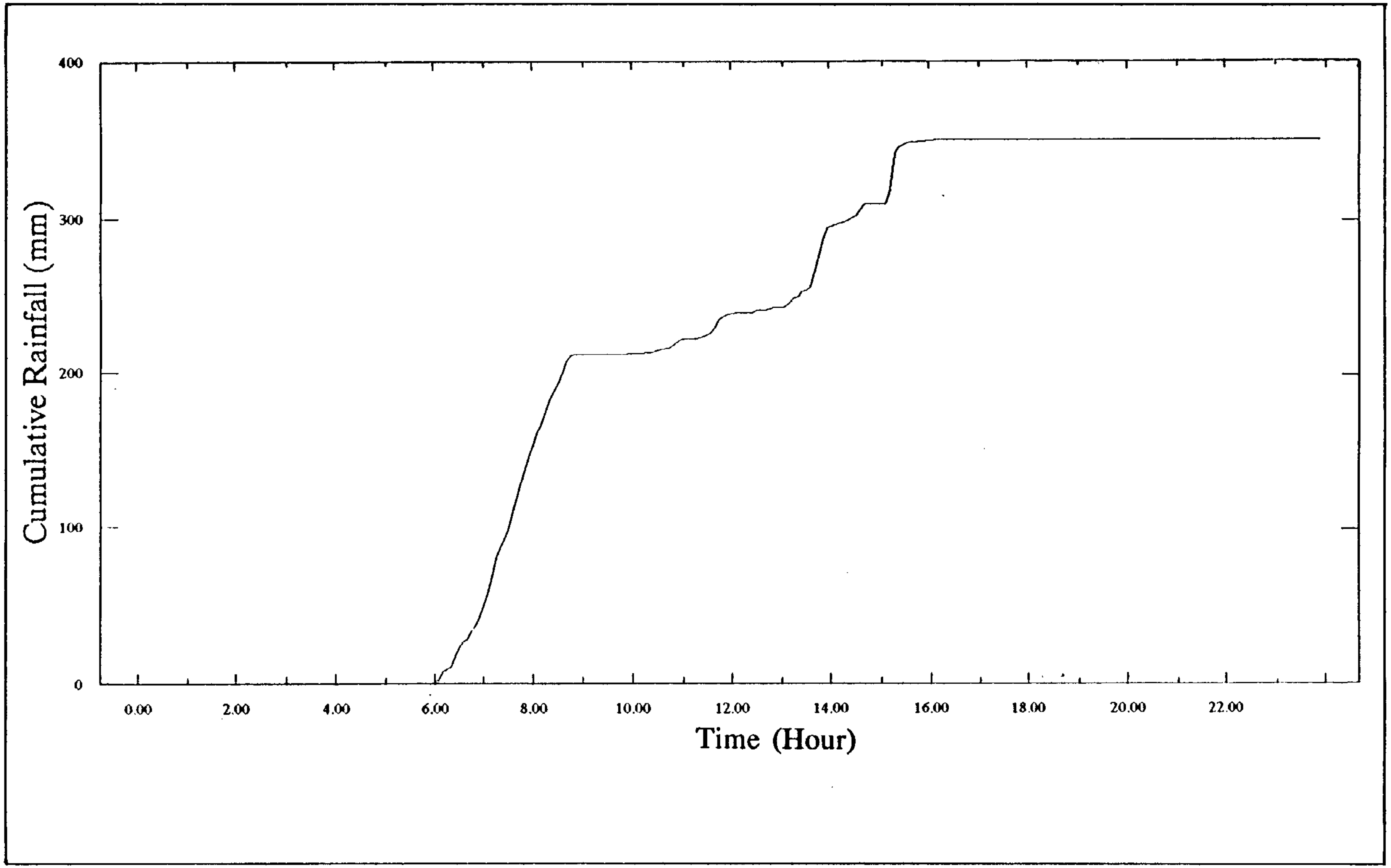
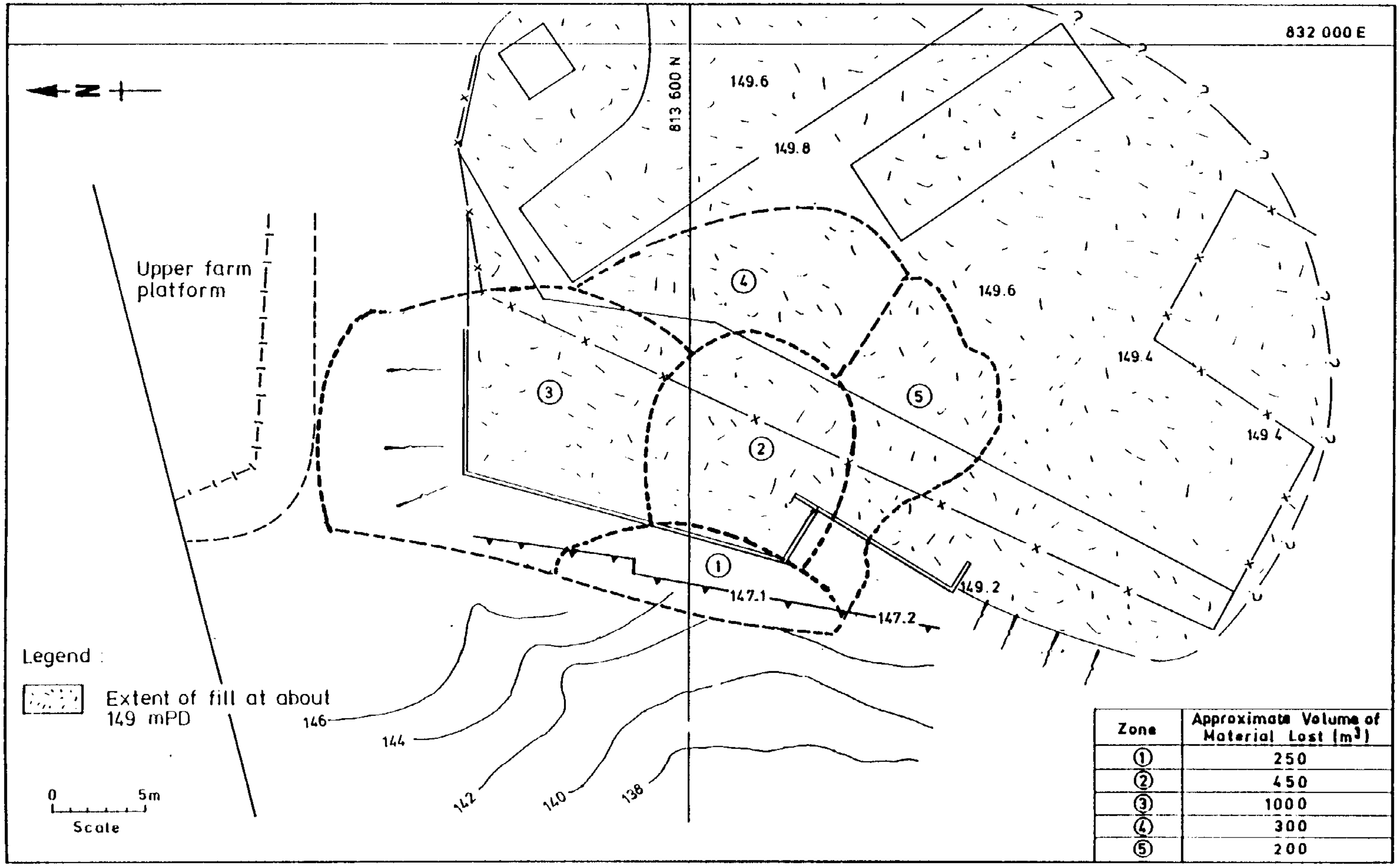


Figure 13 - Cumulative Rainfall Recorded at Raingauge H03 on 8 May 1992



Zone	Approximate Volume of Material Lost (m³)
①	250
②	450
③	1000
④	300
⑤	200

Figure 14 - Zoning of Landslide Scar at the Sitting-out Area

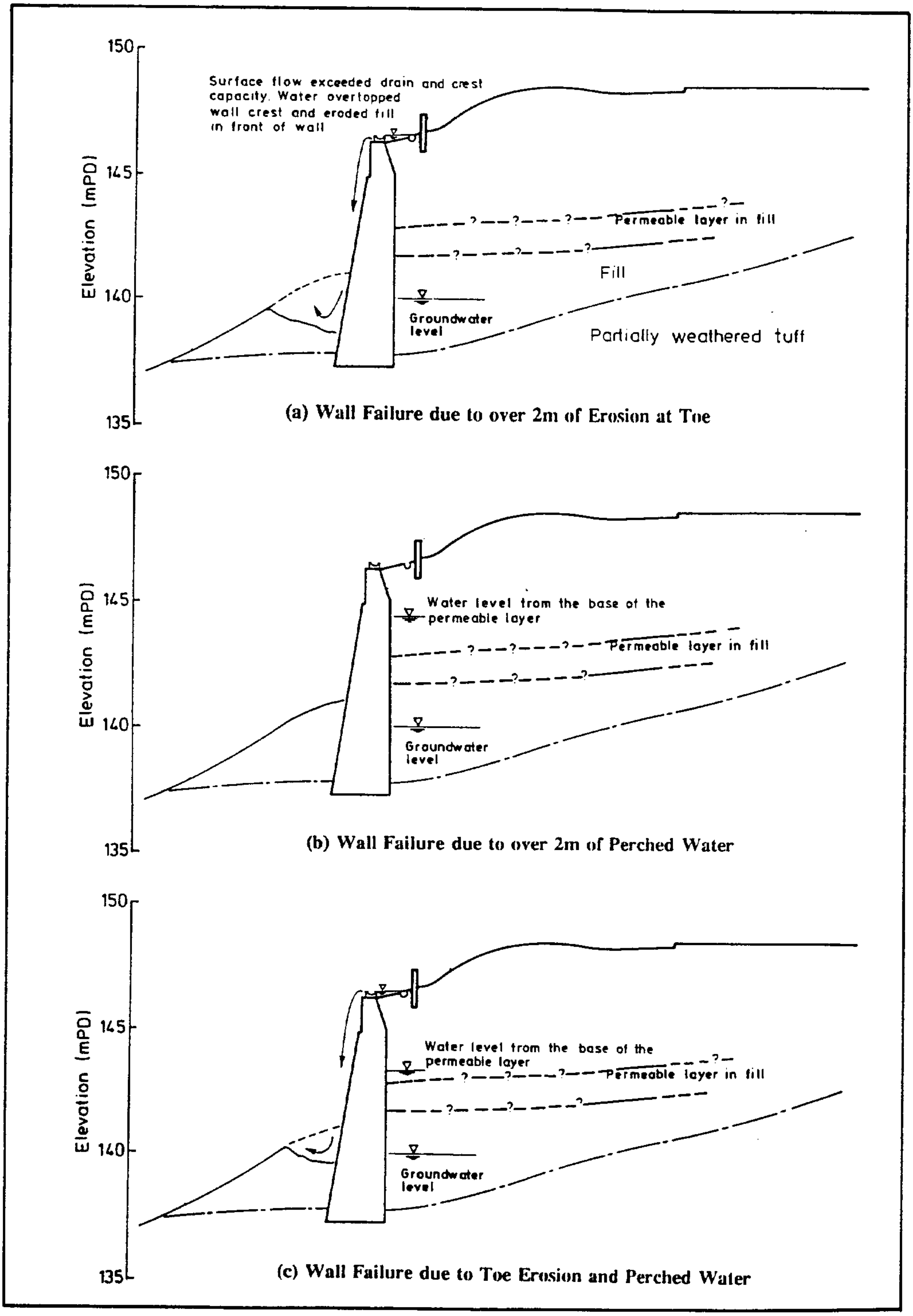


Figure 15 - Possible Conditions for the Failure of the Masonry Retaining Wall

SECTION 2 : REPORT ON THE INVESTIGATION OF THE 8 MAY 1992 KENNEDY ROAD LANDSLIDE

H.N. Wong

This report was originally produced in October 1992
as GEO Advisory Report No.ADR 27/92

FOREWORD

Following the landslide at Kennedy Road, Hong Kong on 8 May 1992, the Geotechnical Engineering Office carried out an investigation into the landslide. This report summaries the findings of the investigation.

The investigation was conducted by Mr H. N. WONG of the Special Projects Division. Topographic survey of the landslide area was carried out by the Survey Division, CED. Dr R. L. Langford of the Planning Division mapped the failure scar and advised on the geological aspect. API service was provided by Mr M. C. CHAN of the Planning Division.

Y. C. CHAN
Chief Geotechnical Engineer/Special Projects

CONTENTS

1. INTRODUCTION

On the morning of 8 May 1992, a landslide occurred below Hong Kong Wah Yan College (Figure 1). A section of Kennedy Road in front of Wan Chai Polyclinic was buried by the debris from the landslide (Figures 2 and 3), and a motorist was fatally injured.

The Geotechnical Engineering Office (GEO) undertook an investigation of the landslide. The investigation included site surveys, collection of documentary information, interviews of witnesses, stability analyses and consideration of the probable causes of the landslide. This report summarises the findings of the investigation.

2. THE SITE

Hong Kong Wah Yan College occupies the flattened top of a small hill in Happy Valley. Immediately to the west of Wah Yan College there are relatively steep, west-facing slopes overlooking Kennedy Road (Figure 1).

The ground that failed on 8 May 1992 was located above the crest of cut slope No. 11SW-D/CR403 (Figure 1). It consisted principally of fill comprising silty-sand-sized material with pieces of brick and stone, and was covered by trees and undergrowth before the landslide. A study of old maps and aerial photographs shows that the fill was placed between 1930 and 1949. The probable extent of fill, as interpreted after the event from aerial photographs taken in 1949, is shown in Figure 4.

Construction of Wah Yan College commenced in 1954. Works comprised site formation and the construction of school buildings, including the West Wing. As part of the site formation work, a masonry retaining wall was built on the fill (Figure 3). The Extension Building of Wah Yan College was added between 1961 and 1963. Cut slope No. 11SW-D/CR403 was formed in 1967 and 1968 to provide space for the realignment and widening of Kennedy Road.

3. RAINFALL

GEO raingauge No. H06 is located on the roof of St. Margaret's College, some 100 m to the south-east of the landslide area. The rainfall recorded by the raingauge on 7 and 8 May 1992 is shown in Figure 5. After twenty-five days of dry weather, 67.5 mm of rainfall were recorded on 7 May. Rainfall on 7 May stopped at about 4 p.m. On 8 May, intense rainfall began at about 6:10 a.m. Two hours later, i.e. by about 8:10 a.m. when the landslide occurred, a total of 163.5 mm of rain had fallen. This corresponds to about a 1-in-20 year event according to historical rainfall records at the Royal Observatory.

4. THE LANDSLIDE

A total of nine persons, including eye-witnesses of the 8 May 1992 landslide and individuals involved in the rescue operation, were interviewed by the GEO. Information about the time and sequence of the landslide and the site conditions thereafter was provided by these persons.

The eye-witnesses reported that it was raining very heavily before and during the landslide. The landslide occurred at about 8:10 a.m. It started with the collapse of a "big" tree and a minor amount of soil onto Kennedy Road. Immediately afterwards, a portion of the slope above cut slope No. 11SW-D/CR403 slipped down. The debris struck a car which was travelling along the south-bound lane of Kennedy Road. The car was pushed across Kennedy Road by the debris, and was buried near the footway in front of the entrance to the carpark of Wan Chai Polyclinic. Muddy water continued to run down the slope during heavy rain subsequent to the landslide. However, the amount of soil brought down was small compared with that of the main landslide.

All eye-witnesses reported that the landslide occurred suddenly and lasted for a very short period of time. The debris comprising soil, stones and trees slipped down as a whole in a matter of seconds. The soil in the debris was soft and wet. A few "big" fallen trees lay across Kennedy Road perpendicular to the direction of the road alignment after the landslide.

A plan showing the failed area and the location of the debris is given in Figure 2. A cross-section of the site after the landslide is shown in Figure 3. About 500 cubic metres of debris slipped down from the failed area and buried the section of Kennedy Road below.

Field mapping was conducted by the GEO within the week following the landslide, before the scar was covered by shotcrete to prevent further failure. Within the constraints of limited time and the threat of further failure, a simple ground investigation including one trial pit, one 'GCO probe' and five hand auger holes was carried out at the locations shown in Figure 2.

It is estimated that the layer of fill which collapsed during the 8 May landslide was about 2 m in thickness. Deposits of fill more than 2.8 m thick at the centre of the crest of the failure scar were found to have remained after the landslide. The fill was wet, and was loose, having a 'GCO probe' resistance of only 1 to 4 blows per 100 mm penetration. At the two sides and the toe of the slip scar, partially weathered granite was exposed. This indicates an old valley into which loose fill had been dumped.

To summarise, the landslide was characterised by:

(a) a shallow slip surface, about 2 m deep (vertical) on average,

(b) sudden and rapid failure taking place in seconds, and

(c) mobile debris travelling across almost the full width of Kennedy Road.

5. DISCUSSION

The 8 May 1992 landslide involved principally failure of loose fill in the surface 2 m of the slope. The presence of loose fill was confirmed by field mapping after the landslide.

Stability analyses indicate that in theory the slope would become marginally unstable when the loose fill was saturated by water infiltrating into the slope during heavy rainfall. Possible sources of water infiltration include:

(a) direct rainfall on the slope, and

(b) stormwater from adjoining catchment areas discharging via surface channels onto the slope.

Three catchment areas in which runoff would have been collected and discharged via surface drainage channels onto the site of the 8 May landslide are shown in Figure 6. From calculations, during the period from about 6:10 a.m. to 8:10 a.m. on 8 May 1992, a maximum of 2000 litres per minute of stormwater would have been discharging from these catchment areas to the surface channels on the slope. In theory, from survey records made before the failure, the design capacity of the surface drainage channels on the slope was sufficient to accommodate stormwater flows during the rainstorm in the morning of 8 May 1992. As such, direct rainfall on the slope would be the primary source of water infiltration. However, if the drainage channels were blocked or broken before the failure, a large amount of stormwater might have overflowed onto the slope.

The heavy rainfall starting from about 6:10 a.m. on 8 May 1992 was capable of saturating the surface 2 m of the loose fill on the slope by infiltration. The slope failure is believed to have been triggered primarily by a reduction of the soil strength due to saturation, followed by liquefaction of the fill.

When the saturated loose fill failed, it slipped rapidly downslope. The landslide debris reached Kennedy Road in seconds, and travelled a distance of about 10 m over the relatively flat surface of the road. The debris came to rest at a surface angle of inclination of about 10°. The mobility and the low angle of repose of the debris indicate that the soil had a low strength during failure. This is typical of loose fill which fails under saturated soil conditions and then liquefies.

The toe of the landslide scar extended into the upper part of cut slope No. 11SW-D/CR403 which comprised residual soil and partially weathered granite. Soil from this part of the cut slope was probably dislodged by debris from the upper part of the landslide.

6. CONCLUSIONS

The landslide of 8 May 1992 occurred in loose fill in the surface 2 m of a slope above the crest of part of cut slope No. 11SW-D/CR403. The loose fill was deposited before 1949.

The cause of the 8 May 1992 landslide was the liquefaction of this loose fill, resulting in the mobile debris flowing rapidly onto Kennedy Road with fatal consequences. Liquefaction would be consistent with surface infiltration of water into the slope, during the two hours of heavy rainfall before the landslide.

LIST OF FIGURES

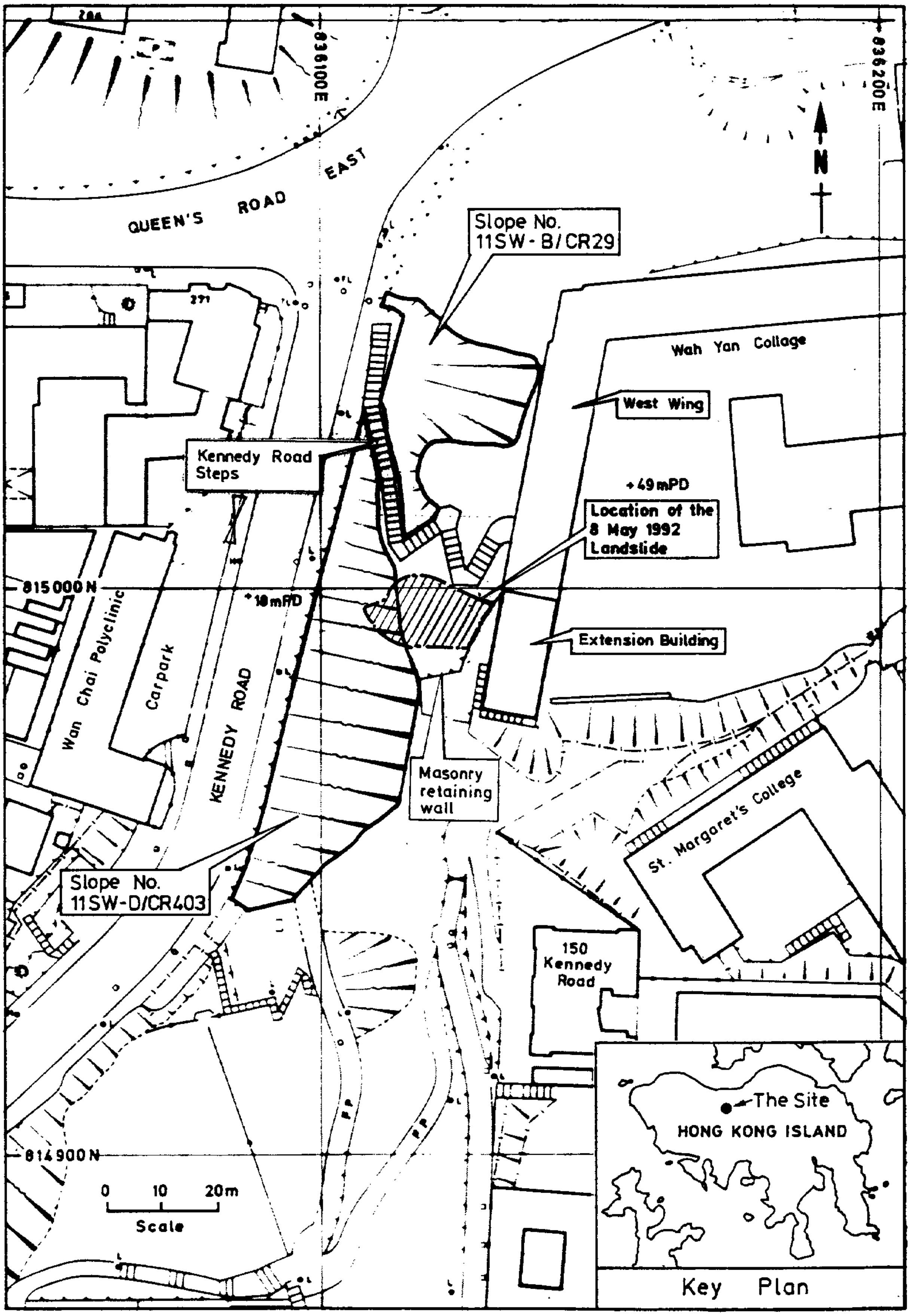


Figure 1 - Site Location Plan

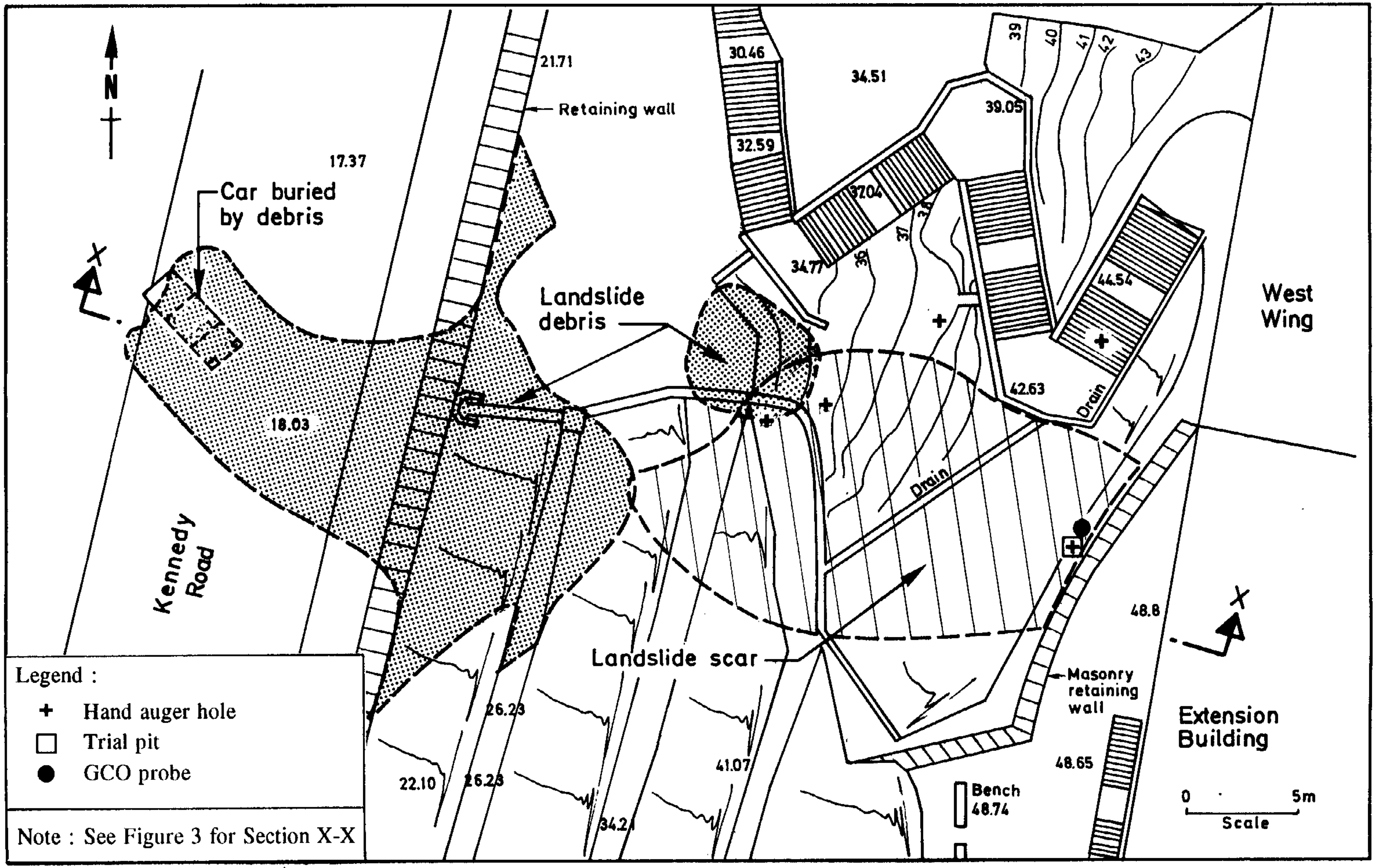


Figure 2 - Plan of the 8 May 1992 Landslide

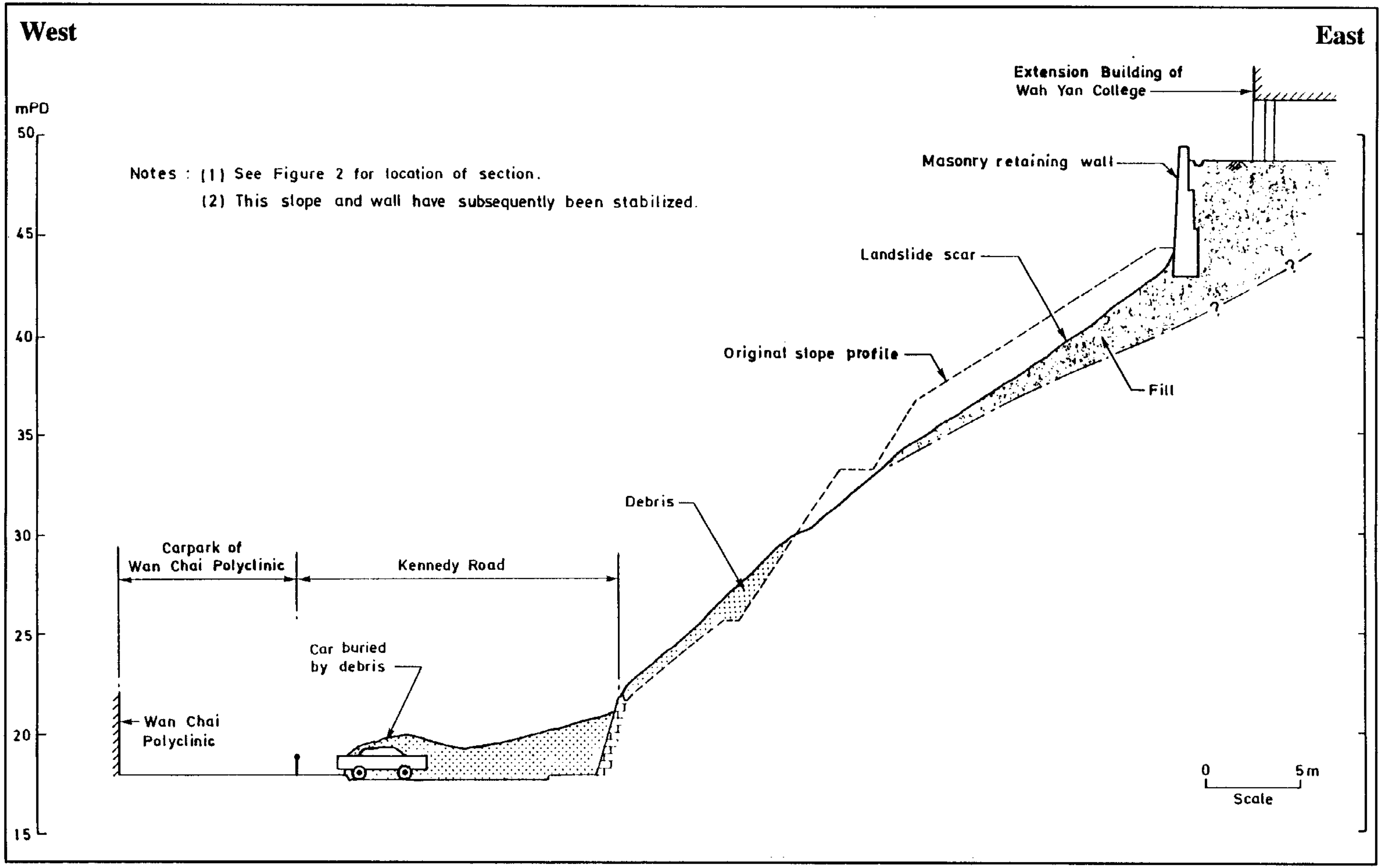


Figure 3 - Section X-X of the 8 May 1992 Landslide

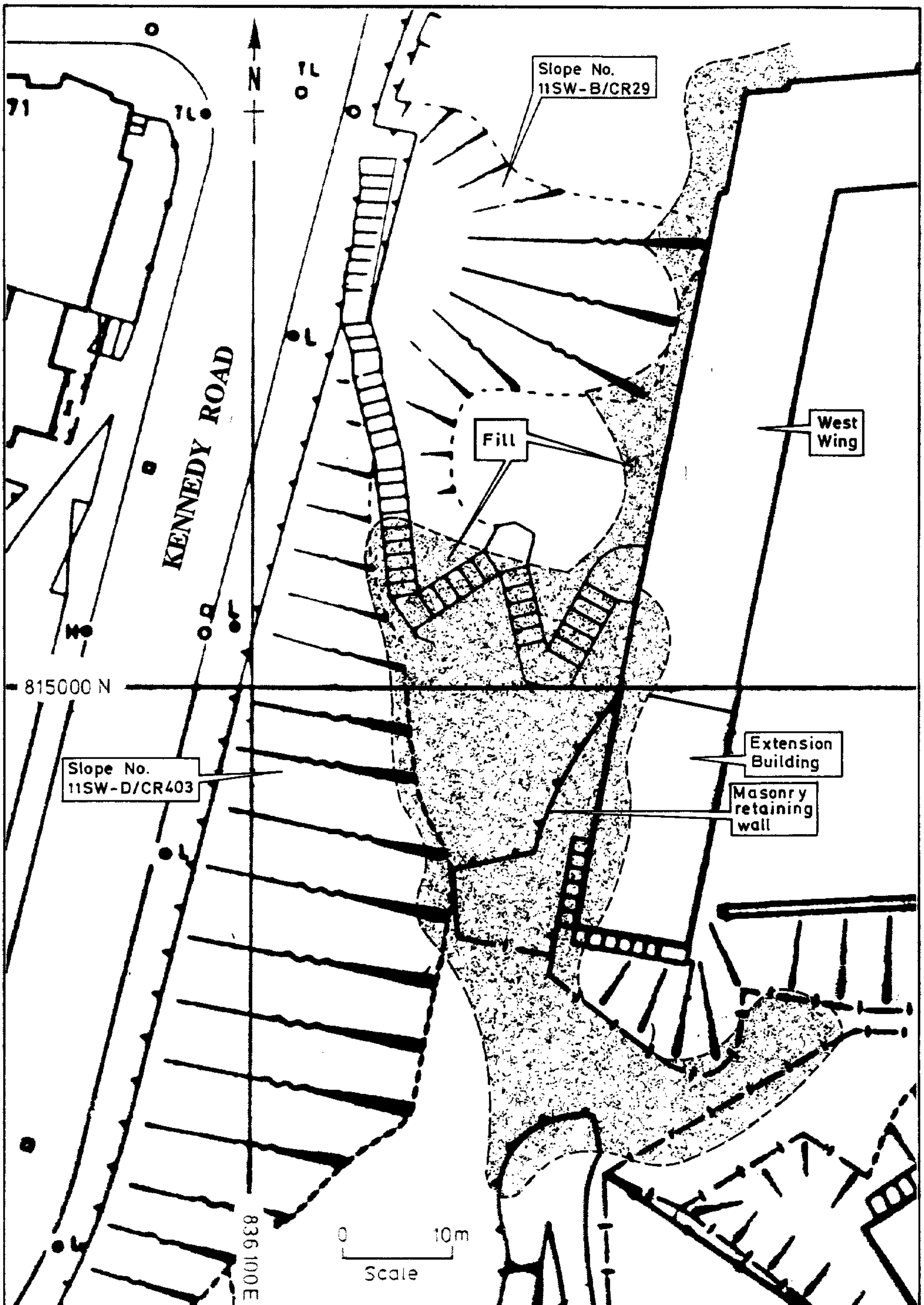
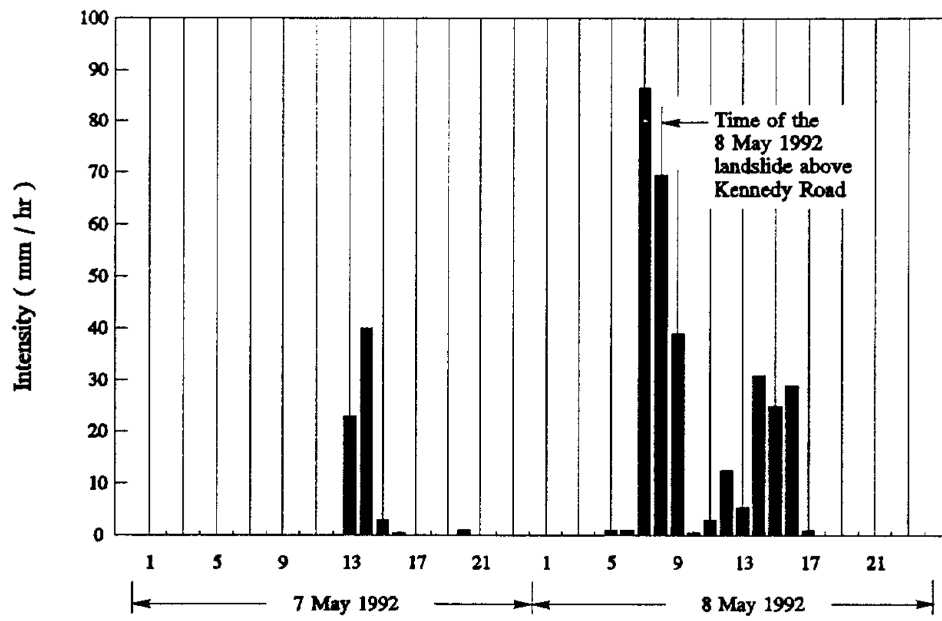
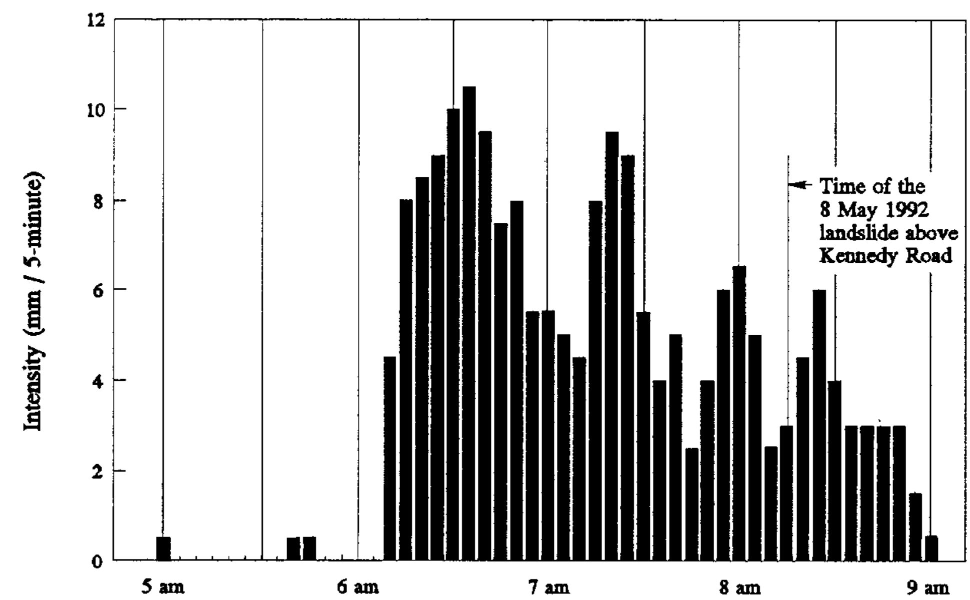


Figure 4 - Probable Extent of Fill

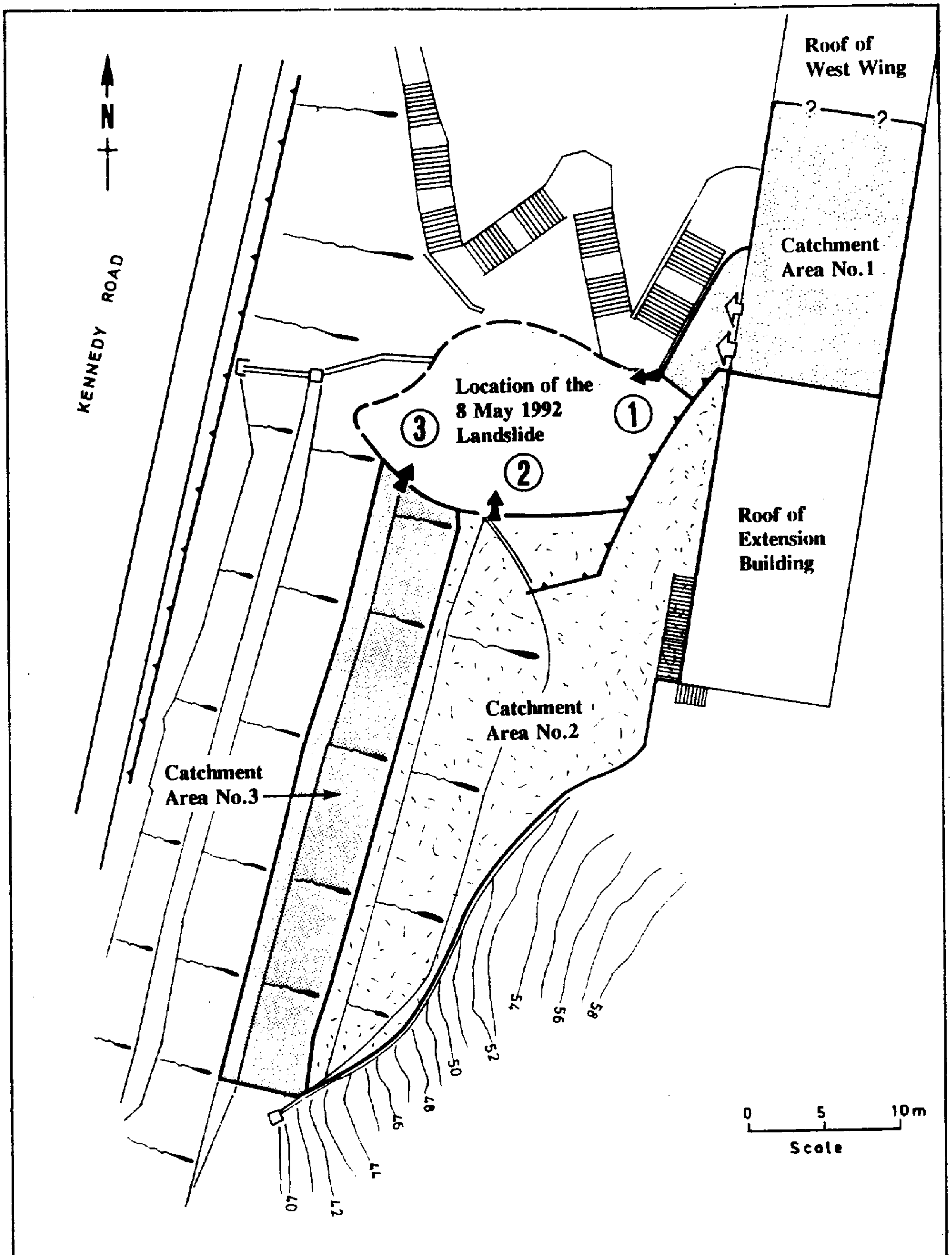


(a) Hourly Rainfall Intensity on 7 and 8 May 1992



(b) 5-minute Rainfall Intensity on 8 May 1992

Figure 5 - Rainfall Record of GEO Raingauge H06



Legend :

↑②	Discharge point of Catchment Area No. 2	⇦	Locations where stormwater flowed from the roof of West Wing onto the slope below

Figure 6 - Catchment Areas Adjoining the Location of the 8 May 1992 Landslide

SECTION 3 : REPORT ON THE INVESTIGATION OF THE 16 JUNE 1993 LANDSLIP AT CHEUNG SHAN ESTATE

W.K. Pun & A.C.O. Li

This report was originally produced in September 1993
as GEO Advisory Report No.ADR 10/93

FOREWORD

Following the landslip at Cheung Shan Estate on 16 June 1993, the Geotechnical Engineering Office carried out an investigation into the landslip.

The investigation was carried out by Mr W.K. Pun and Dr A.C.O. Li of the Special Projects Division under the supervision of Mr Y.C. Chan. Topographic survey of the landslide area was carried out by the Survey Division, CED. The Photogrammetric Unit of BLD carried out aerial photogrammetry. Dr R.L. Langford of the Planning Division mapped the failure scar. API service was provided by Mr M.C. Chan of the Planning Division. Ground investigation and laboratory testing were conducted by the Materials Division.

Y.C. Chan
Chief Geotechnical Engineer/Special Projects Division

CONTENTS

1. INTRODUCTION

On 16 June 1993, a landslip occurred at the crest of a cut slope adjacent to Cheung Shan Estate bus terminus, Kwai Chung, New Territories (Figures 1 and 2). At the time of the landslip, a double-decker bus was parked at a bus shelter in front of the cut slope. The landslip debris first hit a temporary shed at the foot of the slope and then continued towards the bus (Figures 1 and 6). A woman at the bus shelter was fatally injured in the landslip and five other people were also injured.

The Geotechnical Engineering Office (GEO) undertook an investigation of the landslip. The investigation included site surveys, collection of documentary information, interview of eye-witnesses, ground investigation, field tests, laboratory tests and consideration of probable causes of the landslip. This report summarises the findings of the investigation.

2. THE SITE

The ground that failed on 16 June 1993 was the crest of a 15 m high cut slope which forms the southern portion of cut slope No. 7SW-C/C128 (Figure 1). The cut slope was initially formed along the former Shing Mun Road some time before 1924. The exact date of formation of the slope is not known. In 1976, the slope was regraded to the present configuration for the construction of Cheung Shan Estate. During the construction of the road network of Route 5 (Shing Mun Tunnel) in the period of 1987 to 1990, the area in front of the cut slope was turned into a bus terminus.

Based on an examination of aerial photographs, it appears that chunam was applied to part of the cut slope in 1972 and 1982. The apparent chunam works in 1972 and 1982 were observed on part of the cut slope below the 16 June 1993 landslip scar and on the slope at a distance about 10 m to the southwest respectively. However, no documentary information on the chunam works is available.

3. RAINFALL

According to the rainfall records, the first 15 days of June 1993 were wet. Up to 16 June, a total of about 383 mm of rain was recorded by an automatic raingauge (number N03) at a distance of about 0.5 km southwest of the Cheung Shan Estate bus terminus. Rain was heavy over the whole of the Territory on 16 June and a total of about 200 mm of rain was recorded by raingauge N03 on that day (Figure 3).

The five-minute rainfall intensities recorded by raingauge N03 on 16 June are shown in Figure 4. On that day, heavy rain started at about 11 a.m. and continued until 1 p.m. A total of about 136 mm of rain was recorded during this period. The peak hourly rainfall was about 79 mm between 11.00 a.m. and noon. This corresponds to a 1-in-10-year event according to statistical data from the Royal Observatory. The peak one-hour and two-hour rainfall intensities on the day of the landslip were the highest recorded by raingauge N03 since it was installed in June 1983.

4. THE LANDSLIP

After the landslip, 12 persons including eye-witnesses and people involved in the rescue operation were interviewed by GEO officers to assist in reconstructing the sequence of events. The following information was obtained.

The landslip occurred suddenly at about 12.25 p.m. with no prior signs. Accompanied by a loud thunder-like noise, debris comprising soil, rock and sections of a concrete U-channel was released from the crest of the cut slope. It was reported that the debris flowed down the slope "very quickly". At the time of the landslip, a double-decker bus was parked at the bus shelter in front of the cut slope (Figures 1 and 6). The debris first hit a temporary shed at the foot of the slope behind the bus shelter and then continued towards the bus. Debris with thicknesses ranging from 0.5 m to 2 m was trapped in the space between the cut slope and the bus, being retained to the southwest of the bus shelter by the bus regulator's kiosk. Six persons were partially or wholly buried by the debris. A woman was fatally injured and five other people were injured.

According to the eye-witnesses, muddy water continued to flow down the slope after the landslip during the heavy rainfall (for a period of approximately 30 minutes, based on the rainfall record). Small amounts of soil also slipped down from the slope for a short period of time.

It is estimated that about 35 cubic metres of earth were released from the slope in the landslip. The slipped materials bulked to about 50 cubic metres of debris.

5. GROUND CONDITIONS AND DRAINAGE

Field mapping and ground investigation were conducted by the GEO shortly after the landslip. Field work including four trial pits and six 'GEO probes' was carried out at the locations shown in Figure 5. Based on the information obtained from the field mapping and the ground investigation, a cross-section of the failed cut slope was constructed as shown in Figure 6. The ground at the location of the landslip comprised colluvium about 1 m thick over partially weathered granodiorite. Colluvium consists of deposits formed by the downslope movement of soil and rock over a long period of time (several thousands to hundreds of thousands of years) under the action of gravity, commonly found on the lower reaches of natural slopes in Hong Kong. The slope failure appears to have taken place entirely within the colluvium, exposing the underlying partially weathered granodiorite. Field tests showed that the colluvium adjacent to the landslip scar was very loose, having a 'GEO probe' resistance of less than 5, and was more permeable than the partially weathered granodiorite underneath. Laboratory tests on samples collected from the site confirmed that the colluvium was very loose.

Site inspections after the landslip revealed that the cut slope did not have a surface protection except for a chunamed portion of the slope at a distance about 10 m to the southwest. The cut slope was overgrown with vegetation and there was a 300 mm U-channel along the crest of the slope at the location shown in Figure 5. The remaining portions of the U-channel on both sides of the landslip scar were found to be partially silted. Evidence of soil erosion was also observed in the vicinity of the remaining portions of the U-channel (for example, see section A - A in Figure 5).

6. PROBABLE CAUSES OF THE LANDSLIP

Many factors could have contributed to the landslip at Cheung Shan Estate on 16 June 1993. The more important ones are the presence of a thin layer of very loose colluvium in the slope and the intense rainfall.

The colluvium at the crest of the cut slope was very loose although it might have been slightly reinforced by plant roots in the ground. The cut slope was stable when it was dry. However, the rainfall in early June would probably have saturated the colluvium in the slope. Upon saturation, the colluvium layer was only marginally stable.

During the heavy rainfall on 16 June, rain water could infiltrate the ground which was covered by natural vegetation. Water that entered the ground would tend to seep vertically downward under the action of gravity. When the water met the less permeable partially weathered granodiorite, its direction of movement would become parallel to the surface of the partially weathered granodiorite and a 'perched water' table would develop. Pressure generated by 'perched water' would adversely affect the stability of the slope. As infiltration continued, the 'perched water' level would gradually rise (Figure 7(a)). The stability of the thin layer (1 m) of very loose colluvium would have been sensitive to rise in 'perched water' level.

The base of the 300 mm U-channel along the crest of the cut slope was only about 0.3 m above the top of the partially weathered granodiorite (Section A - A in Figure 5). When 'perched water' rose to a level higher than the base of the U-channel, the flow of water within and down the slope would be dammed and the water behind the U-channel would rise to a higher level (Figure 7(b)).

Analyses by conventional theoretical methods showed that the cut slope could in theory have failed if the 'perched water' on the downslope side of the U-channel developed to a depth greater than about 0.5 m from the base of the colluvium. Analyses also showed that higher water level behind the U-channel due to damming had little effect on the stability of the slope but could have enlarged the failure zone up the slope.

Because of the partial silting of the 300 mm U-channel, surface water from the upper part of the natural slope could not be effectively carried away by the U-channel. Water therefore might have overflowed onto the cut slope during heavy rainfall, which would increase the amount of water infiltrating the cut slope. This could have contributed to the slope failure.

7. CONCLUSION

The upper part of the southern portion of cut slope no. 7SW-C/C128 comprised a thin (1 m) layer of very loose colluvium which was more permeable than the underlying partially weathered granodiorite. The landslip was likely to be caused by the development of 'perched water' in the colluvium due to the intense rainfall on 16 June 1993. The partial silting of the U-channel along the crest of the cut slope would have resulted in overflow of water, which could have contributed to the landslip.

LIST OF FIGURES

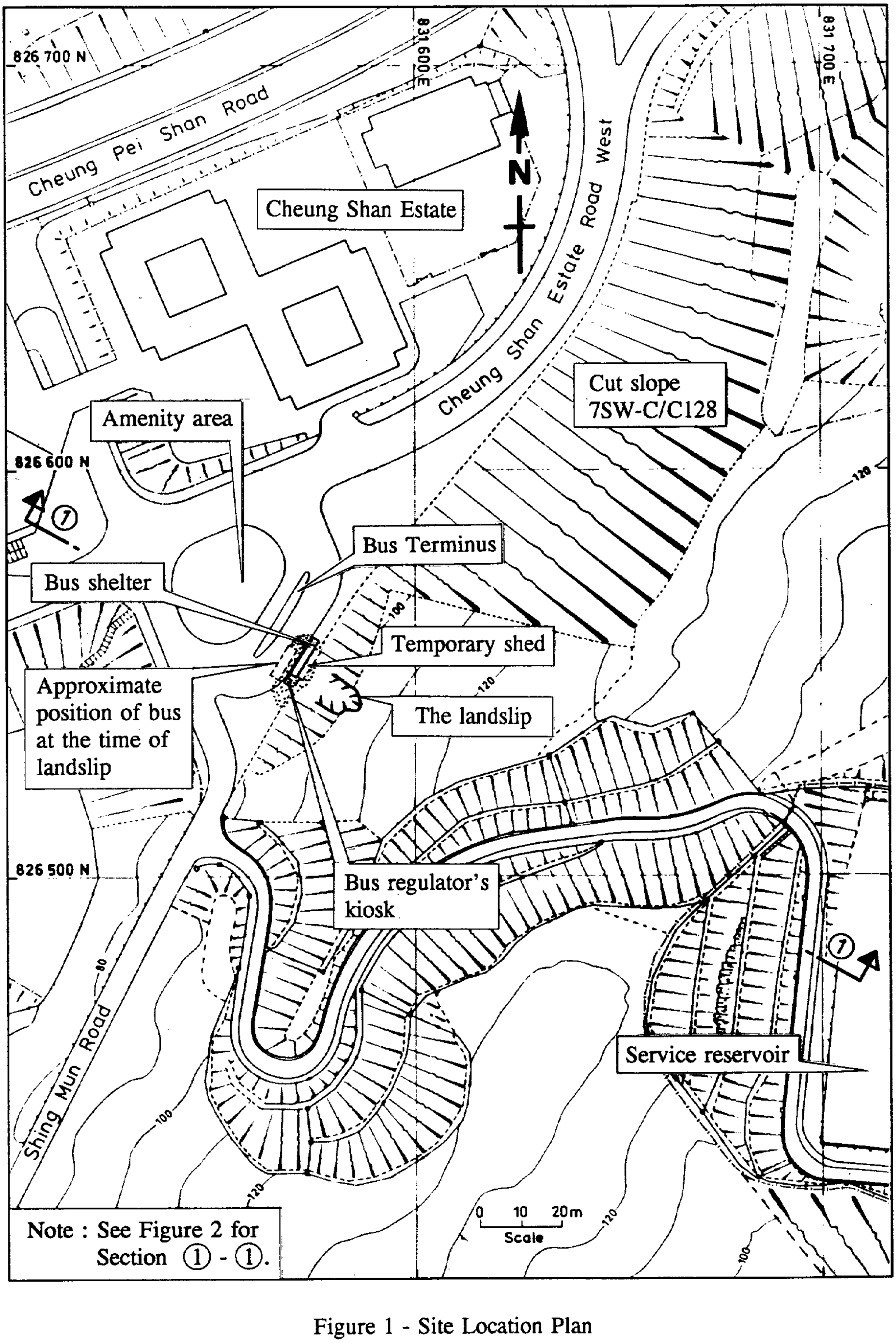
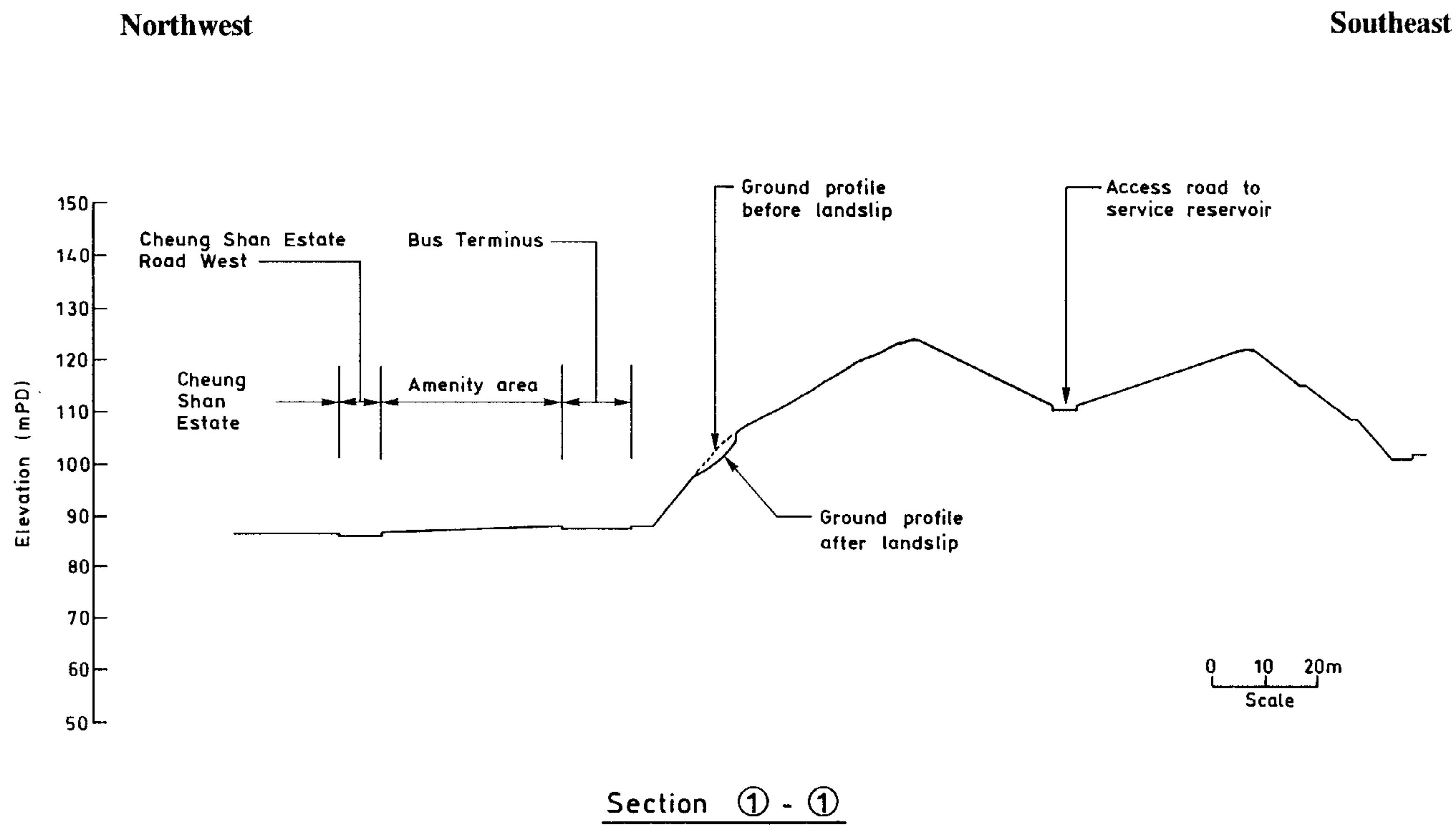


Figure 1 - Site Location Plan



Note : See Figure 1 for location of section.

Figure 2 - Cross-section of the Hillside at the Location of the Landslip

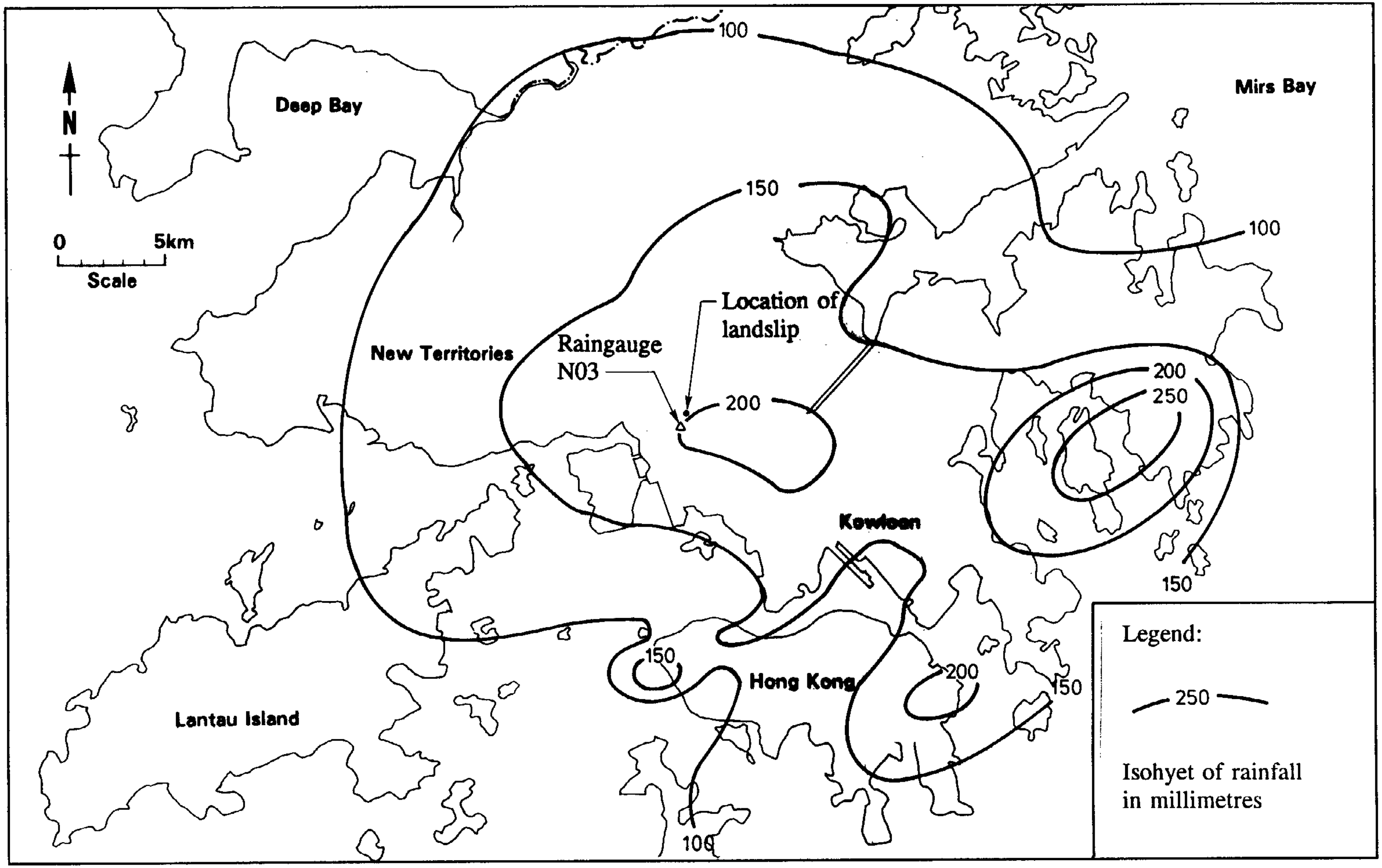


Figure 3 - Isohyets of 24-hour Rainfall on 16 June 1993

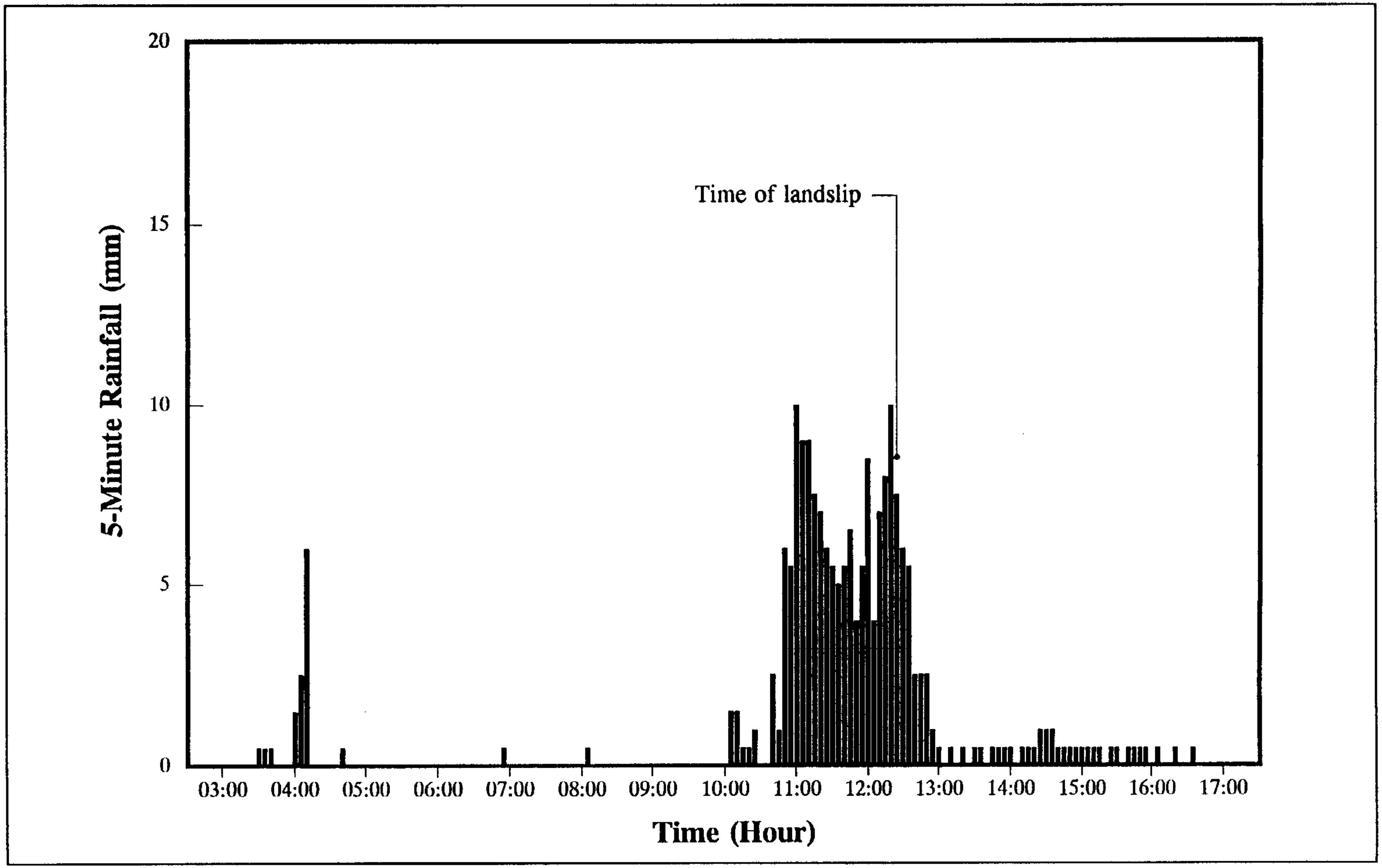


Figure 4 - Rainfall Recorded at Raingauge N03 at 5-minute Intervals on 16 June 1993

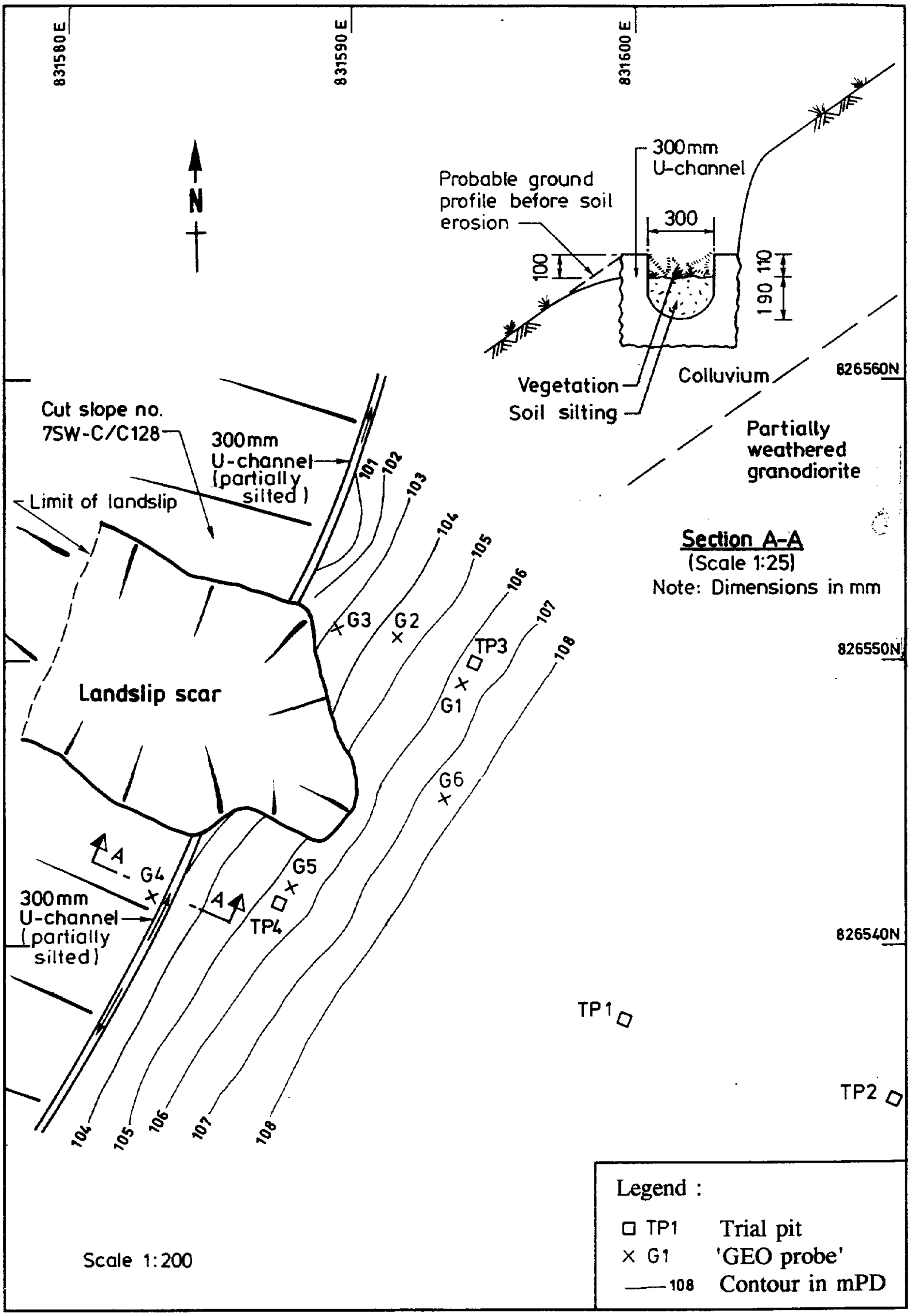


Figure 5 - Plan of the Landslip Scar

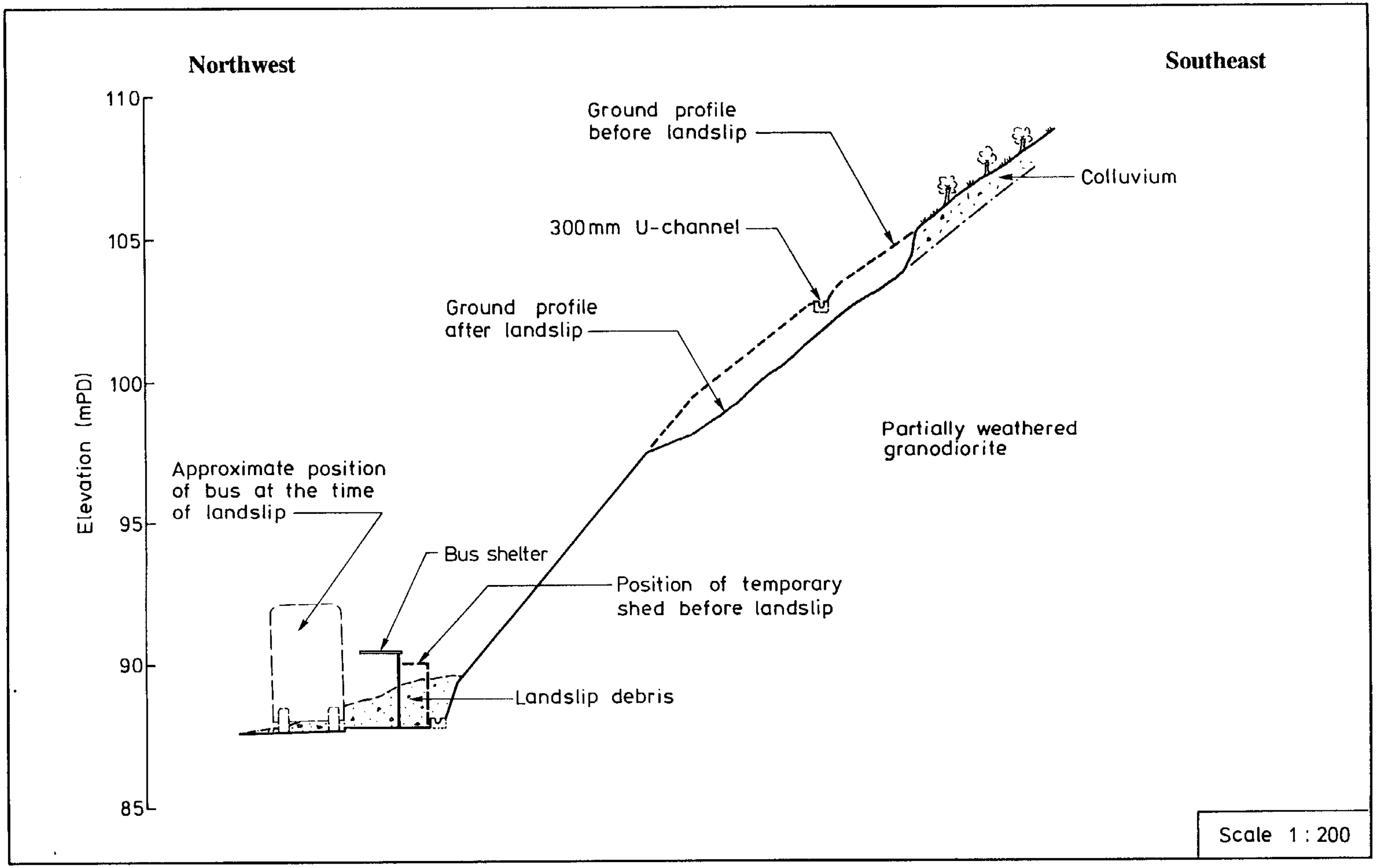


Figure 6 - Cross-section of the Failed Slope

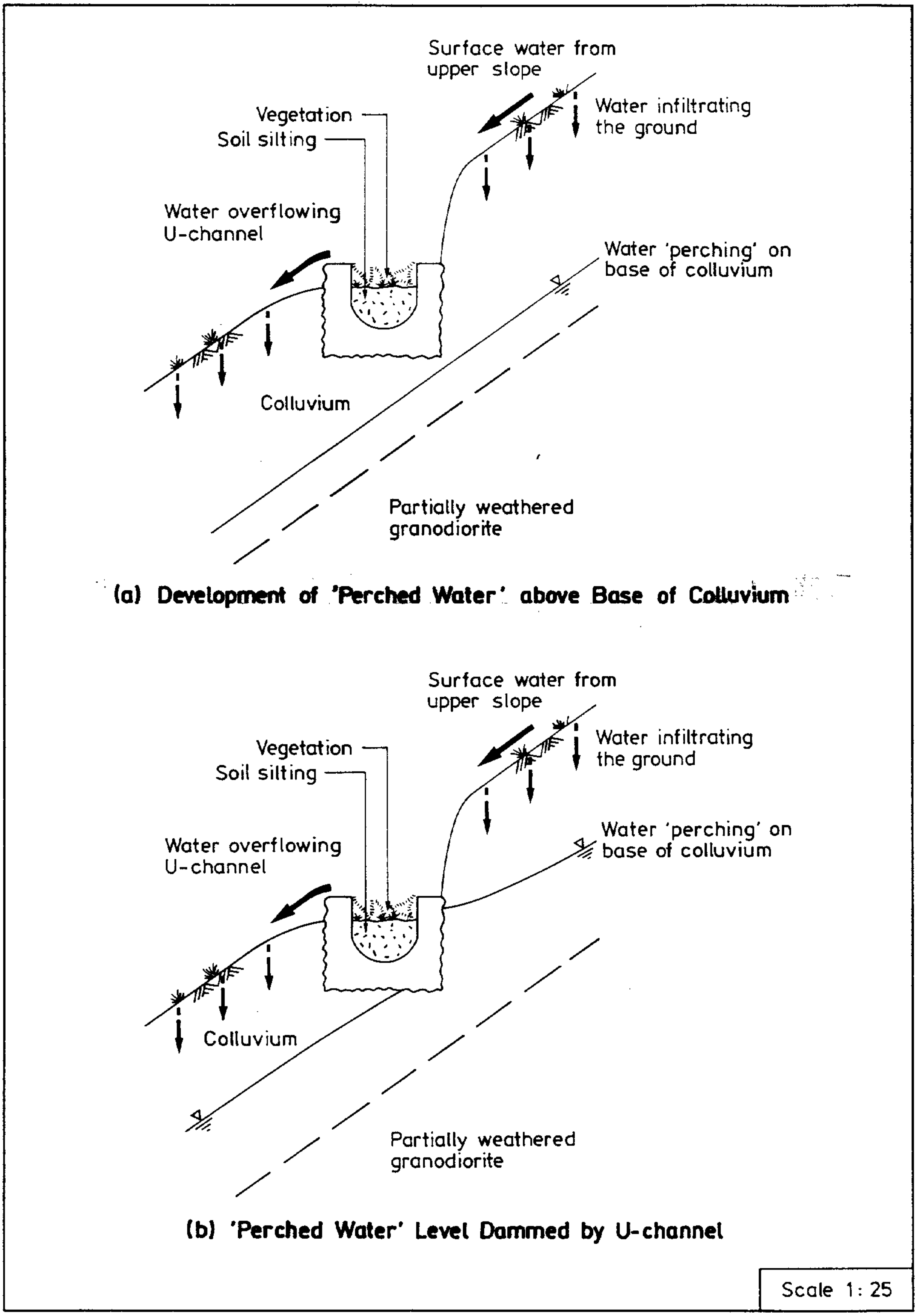


Figure 7 - Possible Conditions for the Failure of the Slope

SECTION 4 : FINAL REPORT ON THE ALPINE GARDEN FLOODING INCIDENT OF 5 NOVEMBER 1993

W.K. Pun & A.C.O. Li

This report was originally produced in January 1994
as GEO Advisory Report No.ADR 2/94

FOREWORD

Following the flooding incident at Alpine Garden, Tuen Mun on 5 November 1993, the Geotechnical Engineering Office carried out an investigation into the incident.

The investigation was carried out by Mr W.K. Pun and Dr A.C.O. Li of the Special Projects Division under the supervision of Mr Y.C. Chan. Topographic survey of the affected area was undertaken by the Survey Division, CED. Mr K.W. Lai of the Planning Division mapped the affected area. API service was provided by Dr K. Emery, a consultant of the Planning Division.

The draft of the report was reviewed by the Drainage Services Department, the Territory Development Department and the Water Supplies Department.

Y.C. Chan
Chief Geotechnical Engineer/Special Projects Division

CONTENTS

1. INTRODUCTION

On 5 November 1993, serious flooding occurred at Alpine Garden, Tuen Mun (Figure 1). The flooding was accompanied by erosion of the slopes to the east of Alpine Garden. On the day of the flooding, a landslip occurred at a cut slope above the Tai Lam Chung catchwater at a location (marked 'III' in Figure 1) southeast of the area, which caused some soil to fall into the catchwater and to block it partially.

The Special Projects Division of the Geotechnical Engineering Office of the Civil Engineering Department undertakes investigations of failures and was tasked to investigate the causes of the flooding incident. An Interim Report, which documented the factual information collected in the first stage of the investigation, was prepared in early December 1993. This Final Report presents the full findings of the investigation.

2. THE SITE

The cut slopes east of Alpine Garden were formed in two stages. Those around the Home for the Elderly (Figure 1) were formed between 1980 and 1981 as a borrow area. During the site formation for Alpine Garden between 1984 and 1987, the cut slope to its immediate east was formed. The superstructure of Alpine Garden was constructed between 1989 and 1990.

There is no record of any previous landslips on the cut slopes since their formation. However, in May 1989 during Typhoon Brenda, some slopes above the Tai Lam Chung catchwater failed and the landslip debris partially blocked the catchwater. Water overflowed from the catchwater and flooded the area below it. The slopes to the east of Alpine Garden were eroded at two locations close to the locations where the erosion occurred on 5 November 1993. After Typhoon Brenda, landslip preventive measures were carried out by Government on the slopes above the catchwater, and all the improved slopes remained stable on 5 November 1993.

3. RAINFALL

Heavy rainfall was recorded on 4 and 5 November 1993. The 24-hour rainfall distribution in the territory is shown in Figure 2. A total of about 380 mm of rain was recorded on the two days by an automatic raingauge (Number N07) located about 2 km northwest of Alpine Garden. This rainfall intensity corresponds to a return period of about 50 years. The hourly rainfall recorded by the raingauge is presented in Figure 3. The peak hourly rainfall occurred at about 8 a.m. on 5 November 1993.

4. THE FLOODING INCIDENT

According to eyewitness accounts, at about 6:00 a.m. on 5 November 1993, a large volume of muddy water was seen gushing into Alpine Garden from the adjacent slopes, causing flooding of the area. The incoming flow was observed to be heaviest around 8:00 a.m. and gradually subsided thereafter. The heavy water flow also caused erosion of the slopes. It is estimated that about 300 cubic metres of soil was washed from the slopes onto the building platform of Alpine Garden.

Eyewitnesses also reported that, at the time of flooding, a large volume of water was seen overflowing from the catchwater at an overflow weir (marked 'I' in Figure 1) to the northeast of Alpine Garden.

5. DRAINAGE LAYOUT AND CONDITION

The approximate layout of the drainage system in the affected area is given in Figure 4. The system generally consists of the Tai Lam Chung catchwater on the hillside above Alpine Garden, a trapezoidal channel (marked 'A' in Figure 4) just to the north, and networks of U-shaped concrete channels on the cut slopes to the east of Alpine Garden. The catchwater was constructed about thirty years ago, long before the site formation for Alpine Garden.

Field observations made immediately after the flooding incident revealed that the U-shaped concrete channels were blocked in places, partly as a result of squatter activities on the slope. The approximate locations of blockage are shown in Figure 4.

6. SOURCES OF WATER

Based on field observations recorded after the incident, the flow paths of water entering Alpine Garden and the extent of the severely eroded areas have been approximately determined to be as shown in Figure 1. Most of the water appeared to have come from an overflow weir (marked 'I' in Figure 1) of the Tai Lam Chung catchwater. Overflow weirs are designed to discharge water at pre-determined points when the water level in the catchwater is too high. This is a safety measure to prevent uncontrolled overtopping.

Other sources of water on 5 November 1993 were rain water falling directly on the ground between the catchwater and Alpine Garden, and surface water flowing from the northern end of a site where the Geotechnical Engineering Office was undertaking landslip preventive works to the cut slopes above the catchwater (marked 'II' in Figure 1).

7. CAUSES OF THE FLOODING

On 5 November 1993, the water level in the Tai Lam Chung catchwater east of Alpine Garden was high because of heavy rainfall. The partial blockage of the catchwater by landslip debris (marked 'III' in Figure 1) would have further raised the water level in the catchwater. Water discharged from the overflow weirs as intended.

Water from the overflow weir marked 'I' in Figure 1 discharged into a natural stream course that is intercepted by a network of drainage channels (see Figure 4). Field observations of erosion that occurred on 5 November suggested that the channels could not cope with the water flow because they were blocked in places. However, the drainage capacity of the 300 mm channel 'B' (Figure 4) could not take the large volume of flow on

5 November 1993 even without blockage.

A small unquantifiable portion of the water from the overflow weir flowed southwestwards (from the location marked 'IV' in Figure 1) to another network of drainage channels (Figure 4), which also receives water from the slope to the east of the Home for the Elderly, including the surface water flowing from the northern end of the landslip preventive measures site. Theoretical calculations indicate that all the channels in this network should have adequate design capacity to carry the water flow. In particular, channel D (Figure 4) should be adequate in size, although on 5 November it received some water that was not allowed for in the drainage design. However, the channels were blocked at a number of locations, the more important blockages being those at channel 'C' and catchpit 'E' (Figure 4). Blockage of channel 'C' caused water to spill to channel 'D' and to erode the nearby cut slopes. Partial blockage of catchpit 'E', which is situated at the crest of a 6.5 m high cut slope, resulted in the overflow of water and erosion of the cut slope. Most of the soil that was washed into Alpine Garden came from the erosion of this slope.

8. STABILITY OF THE SLOPES

Along with the investigation of the flooding incident, the cut slopes behind Alpine Garden have been fully inspected and the designs reviewed. It has been confirmed that these slopes meet the current standards of stability and safety, and they pose no danger to life even during exceptionally heavy rainfall.

9. CONCLUSION

On 5 November 1993, water flowed out of the drainage channels on the slopes to the east of Alpine Garden. The water eroded the slopes and flooded Alpine Garden. Many factors could have contributed to the flooding incident. The more important ones are the heavy rainfall and the blockage of the channels.

10. REMEDIAL WORKS

Government will rectify the deficiencies in the drainage channels. Eroded parts of the slopes will be reinstated. The 300 mm channel 'B' and selected sections of other channels will be upgraded to improve drainage capacity. The drainage channels will also be kept in good working condition by regular maintenance.

The design of remedial works is in progress. The works are scheduled to begin in February and will be completed before the onset of the next wet season.

LIST OF FIGURES

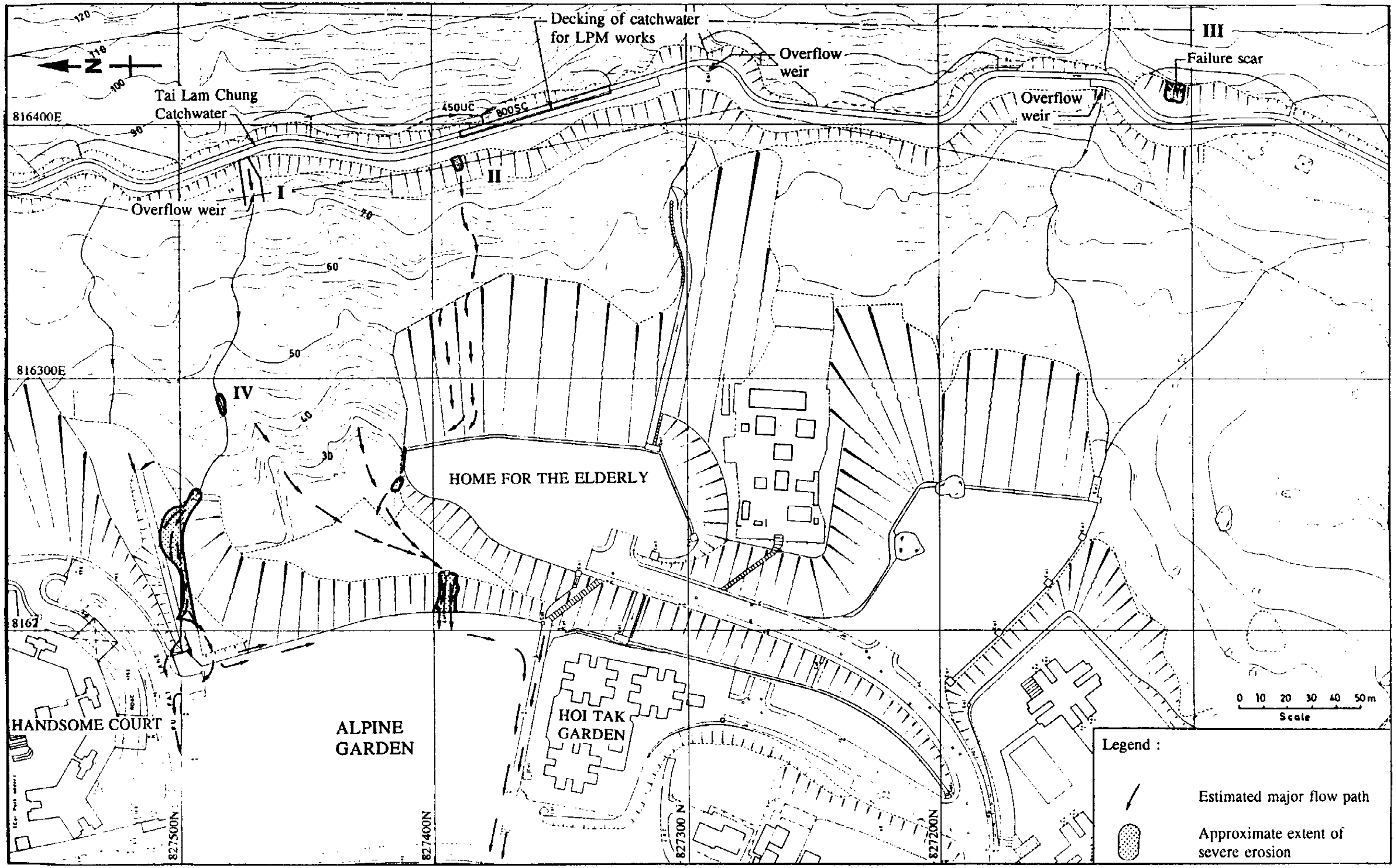


Figure 1 - Site Location Plan

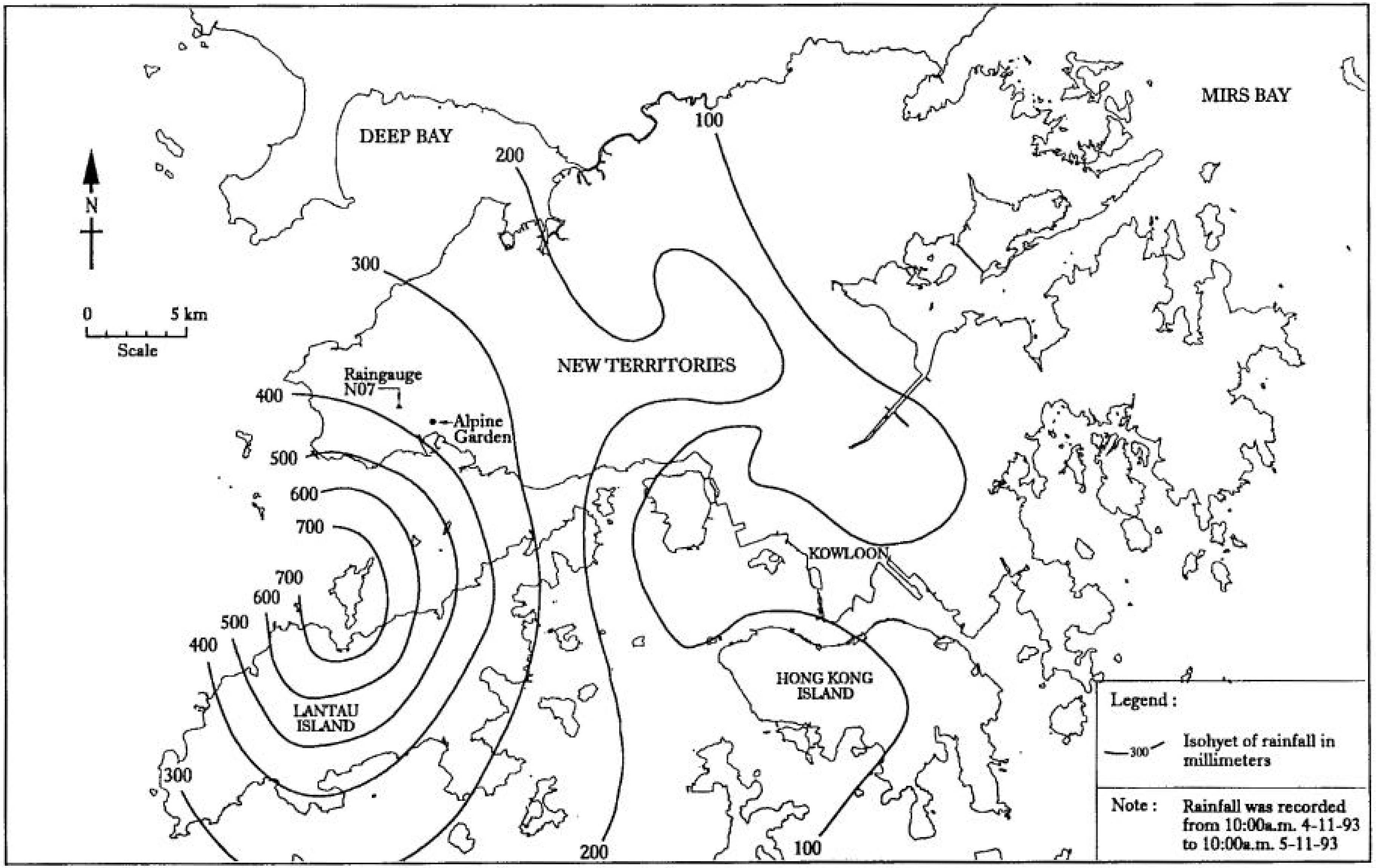


Figure 2 - 24-hour Rainfall Distribution during the Rainstorm of 5 November 1993

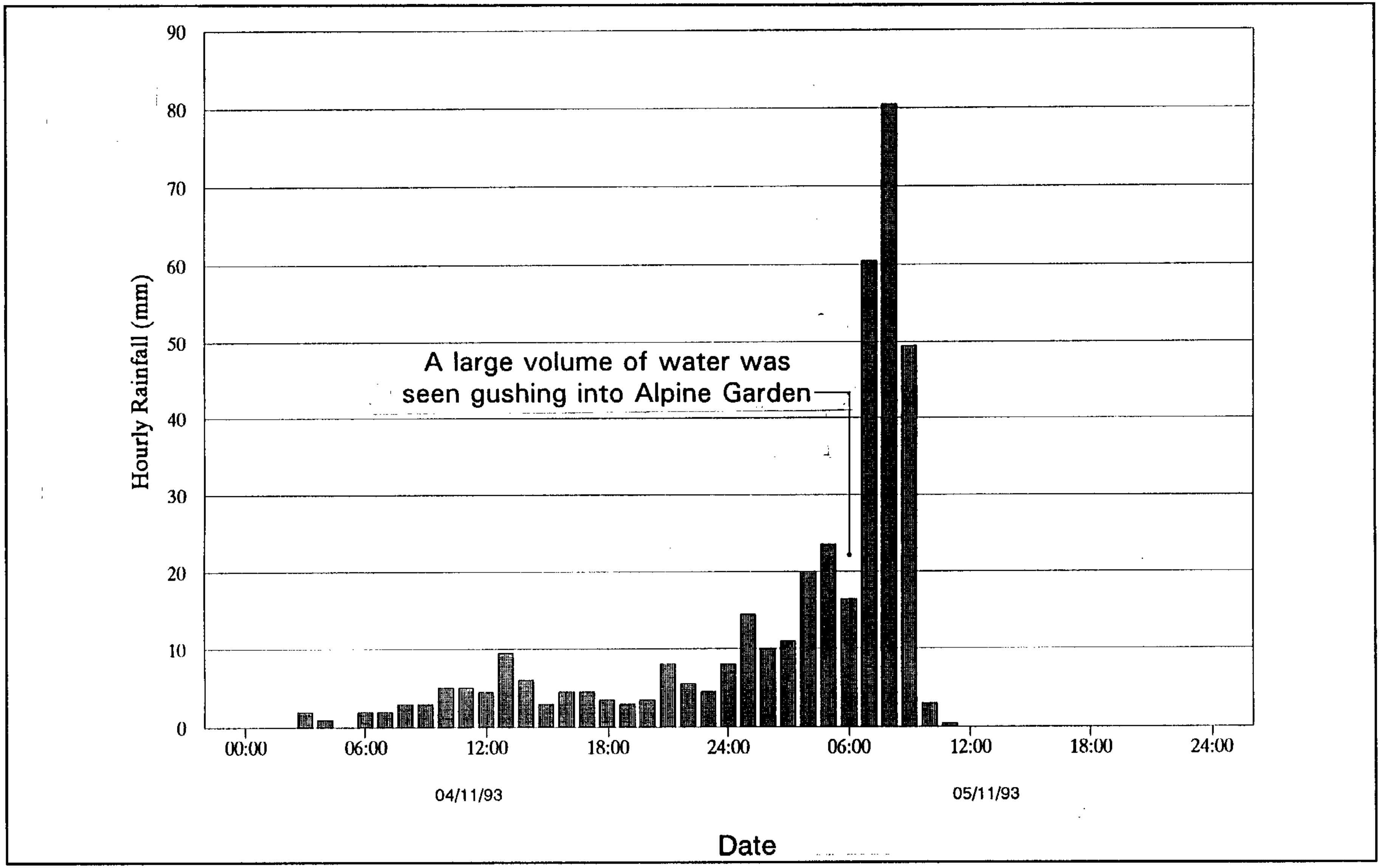


Figure 3 - Hourly Rainfall Recorded at Raingauge N07 on 4 and 5 November 1993

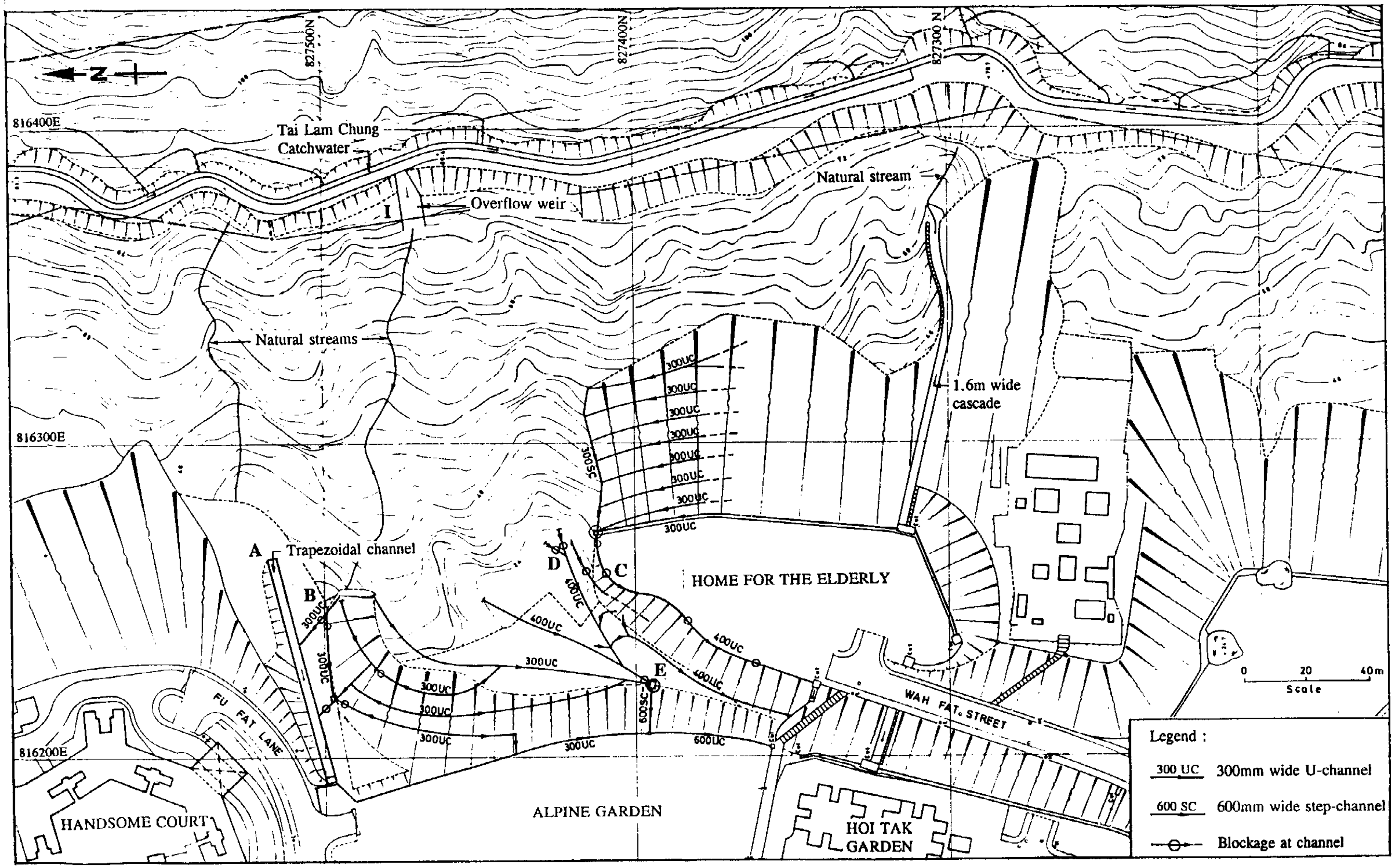


Figure 4 - Approximate Drainage Layout

SECTION 5 : REPORT ON THE INVESTIGATION OF THE 27 SEPTEMBER 1993 LANDSLIP AT ALLWAY GARDENS

A.C.O. Li & W.K. Pun

This report was originally produced in March 1994
as GEO Advisory Report No.ADR 5/94

FOREWORD

Following the landslip at Allway Gardens, Tsuen Wan, on 27 September 1993, the Geotechnical Engineering Office carried out an investigation into the landslip.

The investigation was carried out by Dr. A.C.O. Li and Mr. W.K. Pun of the Special Projects Division under the supervision of Mr. Y.C. Chan. Topographic survey of the landslip area was undertaken by the Survey Division, CED. Mr. K.W. Lai of the Planning Division mapped the failure area. API service was provided by Mr. W.L. Shum of the Planning Division.

Y. C. CHAN
Chief Geotechnical Engineer/
Special Projects

CONTENTS

1. INTRODUCTION

On 27 September 1993 a landslip occurred at a 45 m high cut slope (No. 6SE-D/CR87) adjacent to Block F of Allway Gardens, Tsuen Wan (Figure 1). Some of the debris went into the bedrooms of Flats 801 and 808 of Block F, Allway Gardens. No one was injured in the incident.

The Geotechnical Engineering Office (GEO) undertook an investigation of the landslip. The investigation included site surveys, collection of documented information, interview of eyewitnesses, laboratory testing and consideration of probable causes of the landslip. This report summarises the findings of the investigation.

2. THE SITE

The ground that failed on 27 September 1993 was about 30 m above the toe of the southern portion of the cut slope. The cut slope was covered with chunam and was formed as part of the site formation for the construction of Blocks F to R, Allway Gardens. The site formation work commenced in early 1976 and was essentially completed by 1978.

Based on an examination of aerial photographs, it appears that new chunam was placed on the slope in 1984 at approximately the location of the 27 September 1993 landslip. However, no documented information on the new chunam works is available. In addition, evidence of surface seepage was observed on the slope in 1986 and 1987 at a similar location.

3. RAINFALL

According to the rainfall records, the first 22 days of September 1993 were generally dry, except for the period between 13 and 18 September 1993, during which a total of about 125 mm of rain was recorded by an automatic raingauge (number N03) located about 2.5 km to the southeast of the landslip (Figure 2). Intermittent rainfall was recorded between 23 and 27 September 1993, and a total of about 390 mm of rain was recorded by raingauge N03 over this period.

Prolonged rainfall occurred over the period between 25 and 26 September 1993. There was little rainfall recorded by N03 in the early hours of 27 September 1993 prior to the landslip. Thereafter the rainfall intensity increased again. Based on the 5-day rainfall distribution in the territory between 23 and 27 September 1993 (Figure 3), the total rainfall at the site of the landslip over this period is estimated to range between 450 mm and 500 mm. This corresponds to a return period of about 5 years according to statistical data from the Royal Observatory. The 5-day rainfall intensities between 23 and 27 September 1993 was the highest recorded by the raingauge N03 since it was installed in June 1983.

4. THE LANDSLIP

After the landslip, several persons including residents and staff of the management office of Allway Gardens were interviewed by GEO officers to assist in reconstructing the sequence of events. The following information was obtained.

The landslip occurred at about 5:45 a.m. on 27 September 1993, without any reported prior signs of failure. Accompanied by a loud noise, debris comprising soil and rock was released from the slope. Some of the debris went into the bedrooms of adjacent Flats 801 and 808 of Block F, Allway Gardens.

The landslip surface was shallow, measuring up to about 3 m in depth beneath the slope surface. It was estimated that about 200 to 250 cubic metres of soil and rock were released from the slope in the landslip. About 3 to 5 cubic metres of the debris went into the bedrooms of Flats 801 and 808 of Block F.

5. GROUND CONDITIONS

Field mapping was carried out by the GEO after the landslip, and disturbed soil samples were collected for laboratory tests. Based on the information obtained from a site survey and field mapping, a cross-section of the cut slope at the location of the landslip was determined and is shown in Figure 4. The ground generally comprised a layer of residual soil up to about 1.5 m thick overlying partially weathered volcanic rock. Relict joints, coated with a thin layer of soft to firm clay up to about 10 mm thick, were observed on the landslip scar. It appears that a considerable part of the slip surface was along some smooth relict joints within the weathered rock.

Field inspections carried out after the landslip revealed the presence of pre-existing tension cracks on the upper part of the failure scar (Figure 5). The tension cracks measured up to about 2 m in depth below the slope surface and were coated with black organic materials on the surface. The exact cause for the formation of the tension cracks is not known. However, the existence of tension cracks may indicate past deformation of the slope.

Surface seepage was observed a few hours after the failure on 27 September 1993 on the upper part of the landslip scar (Figure 4). When the site was visited again on 29 September 1993, surface seepage was only observed on the lower part of the scar as shown in Figure 4. These indicated that groundwater was very close to the slope surface at the location of the landslip at the time of the failure. The subsequent rapid fall in groundwater level suggested that the high groundwater in the area was mainly caused by a transient supply of water which was likely to be related to rainfall infiltration uphill of the failure.

6. SURFACE DRAINAGE

The approximate location of the surface drainage system in the affected area is shown in Figures 4 and 5. The system consists of concrete U-channels on the berms of the cut slope, and a catchwater and a trapezoidal channel on the hillside above.

The catchwater, which is located about 20 m above the top of the landslip scar, collects surface water from a natural slope which extends to about 250 m uphill of the scar. A few cracks generally up to about 50 mm wide and 2.5 m in length were observed on the floor of the catchwater, and many of them were found near the bends of the catchwater.

However, the exact cause of the cracks is not known.

The trapezoidal channel, which runs northwards to a nearby culvert (Figure 1), is situated about 10 m above the top of the landslip scar.

The portions of the U-channel that remained on both sides of the landslip scar were found to be partially silted with soil and vegetation.

The cut slope was covered by chunam. The part of the cut slope between the trapezoidal channel and the landslip scar was generally covered by weeds and grass growing from weep holes and cracks up to about 10 mm wide in the chunam (Figure 5). Spacings between the cracks generally ranged from about 1.5 m to 3 m. The area further uphill of the trapezoidal channel, including the natural slope above the catchwater, was covered with grass and shrubs.

7. PROBABLE CAUSES OF THE LANDSLIP

Many factors could have contributed to the landslip at the cut slope adjacent to Allway Gardens on 27 September 1993. The more important ones are the prolonged rainfall, and the presence of pre-existing tension cracks and relict joints.

The cut slope was generally covered by chunam which, if properly maintained, could have prevented significant direct infiltration into the ground. However, surface water could have infiltrated the natural ground above the cut slope, and could have seeped through cracks in the chunam surface. The presence of tension cracks would have also promoted infiltration to the soil at depth.

The large area of infiltration, the rainfall in mid September and the prolonged rainfall between 23 and 27 September 1993 prior to the landslip would have resulted in a rise of the water level in the ground. The resulting water pressure would have adversely affected the stability of the slope. Moreover, water in the tension cracks would have exerted an additional lateral pressure on the slope, further reducing the stability of the slope. However, the exact distribution of the water pressure within the slope is difficult to establish due to the complex hydrogeology and ground conditions in the area.

The strength of the ground was probably governed by the relict joints. These joints would have acted as weak planes along which the landslip would take place, and could have also served as preferential flow paths admitting water into the ground.

Analyses by conventional theoretical methods showed that the stability of the slope was very dependent on the water level in the ground, and the slope would have failed if the water level in the ground rose to about 1 m below the slope surface.

Possible leakage from the cracked catchwater overlooking Allway Gardens might have contributed to the rise in groundwater level in the area. However, based on field observations of the surface seepage on the landslip scar, it appears that the rise in groundwater level in the area of the landslip was mainly caused by the transient supply of water uphill and the contribution by the steady leakage from the cracked catchwater should be insignificant. This is consistent with the findings of theoretical calculations.

8. CONCLUSIONS

Many factors could have contributed to the landslip on 27 September 1993. The more important ones are the prolonged rainfall, and the presence of pre-existing tension cracks and relict joints. The tension cracks would have promoted infiltration into the ground. The relict joints could have facilitated movement of water into the ground and would have served as weak planes along which the landslip took place.

LIST OF FIGURES

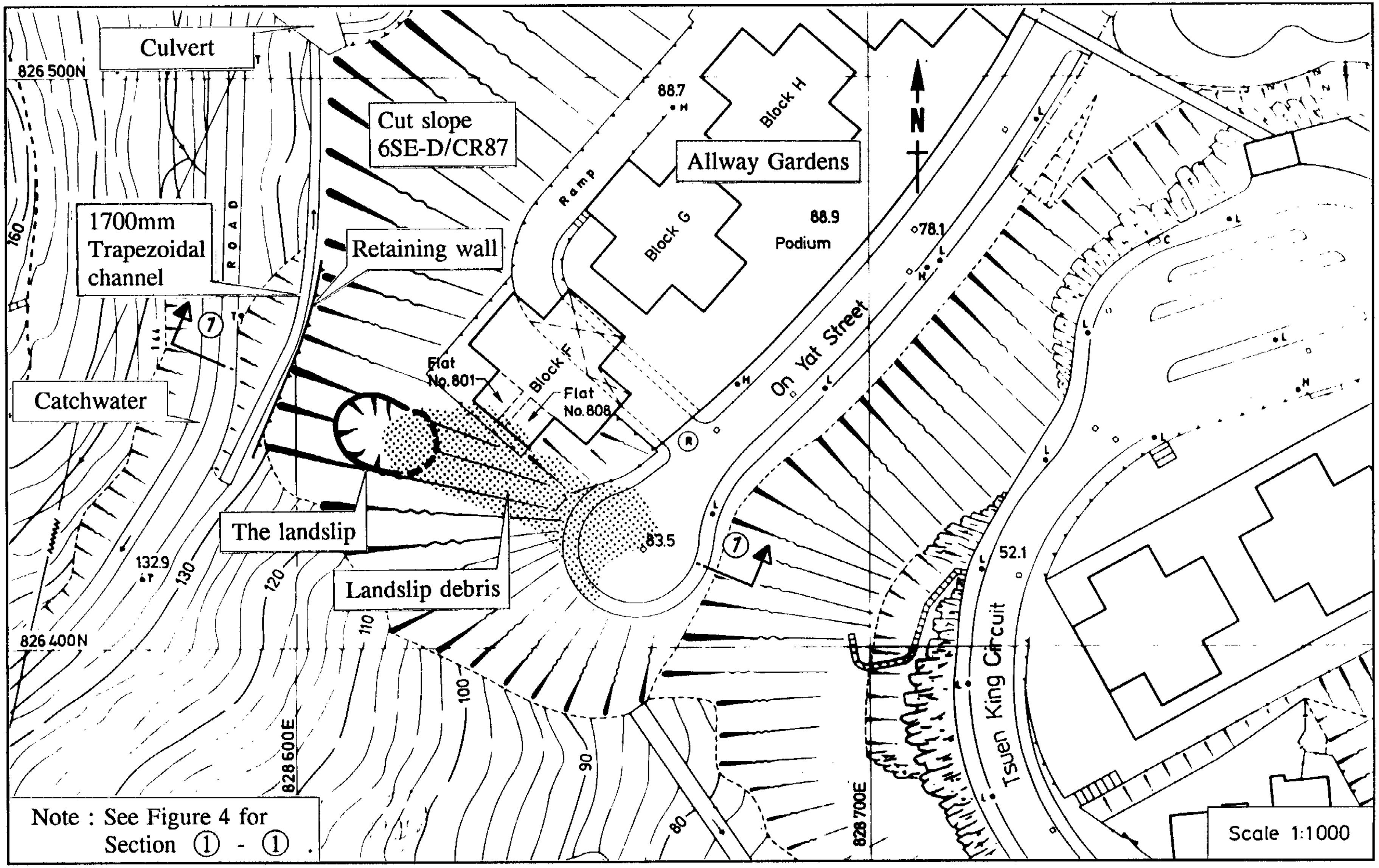


Figure 1 - Site Location Plan

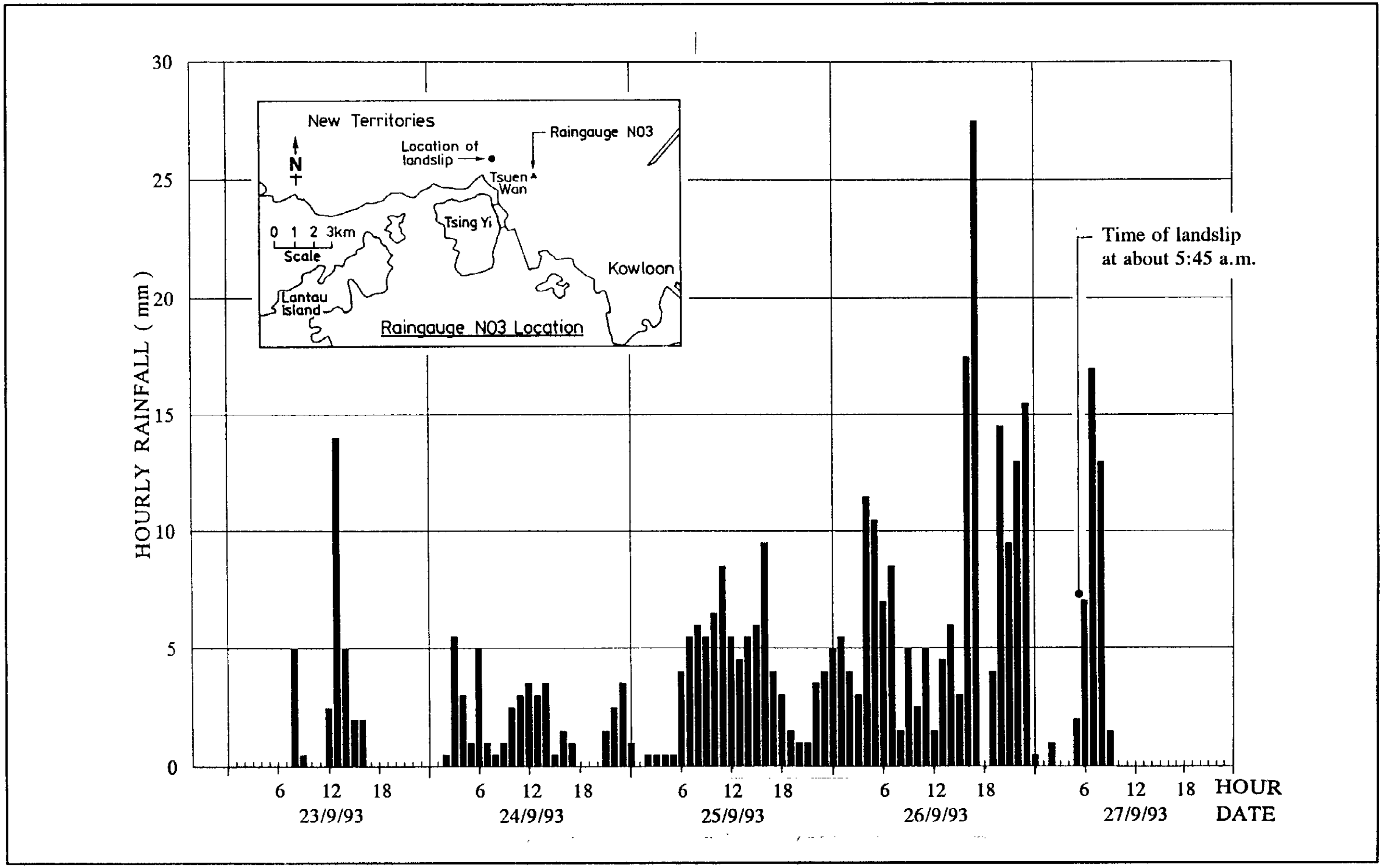


Figure 2 - Rainfall Recorded at Raingauge N03 at Hourly Intervals between 23 and 27 September 1993

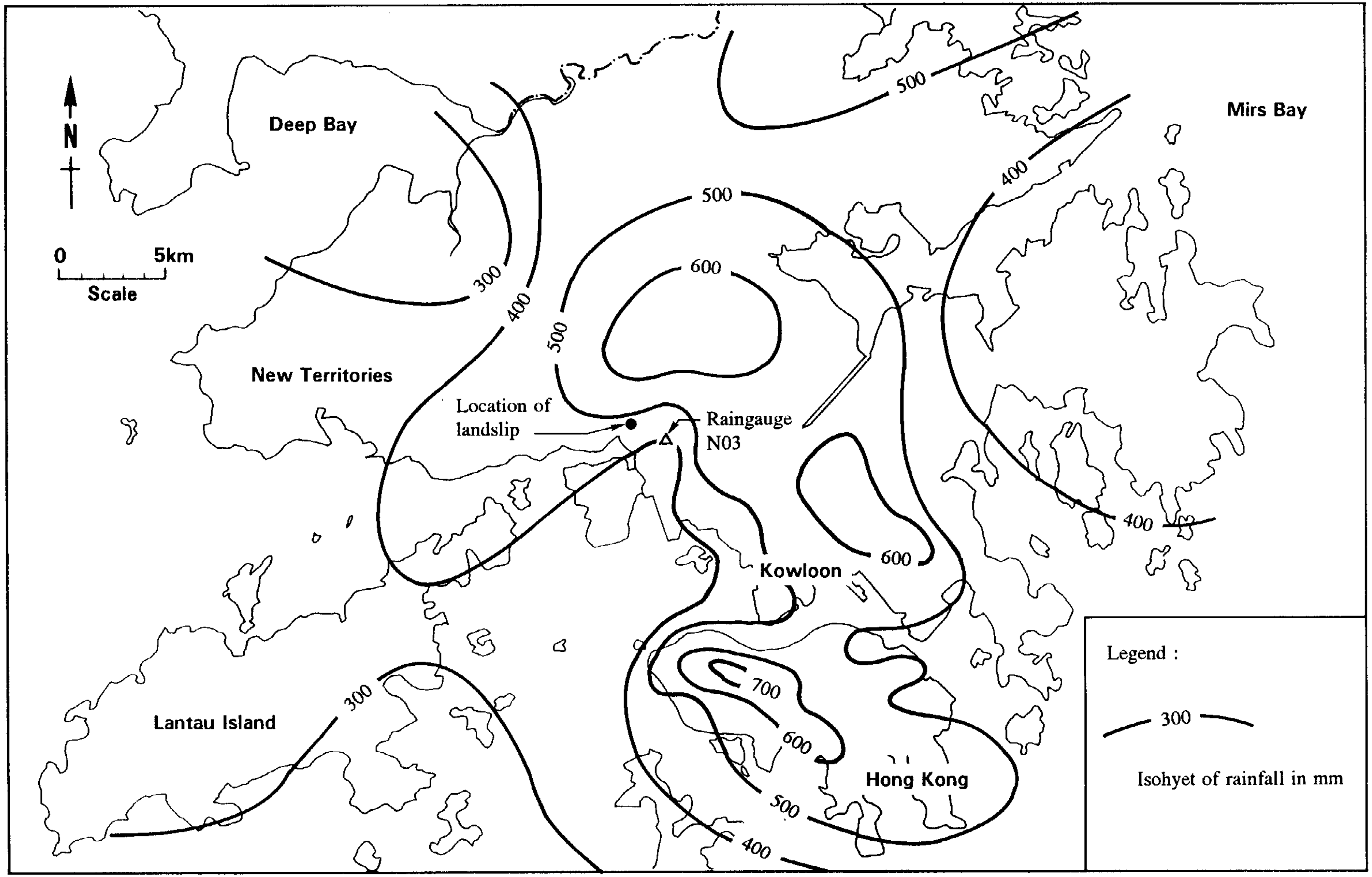
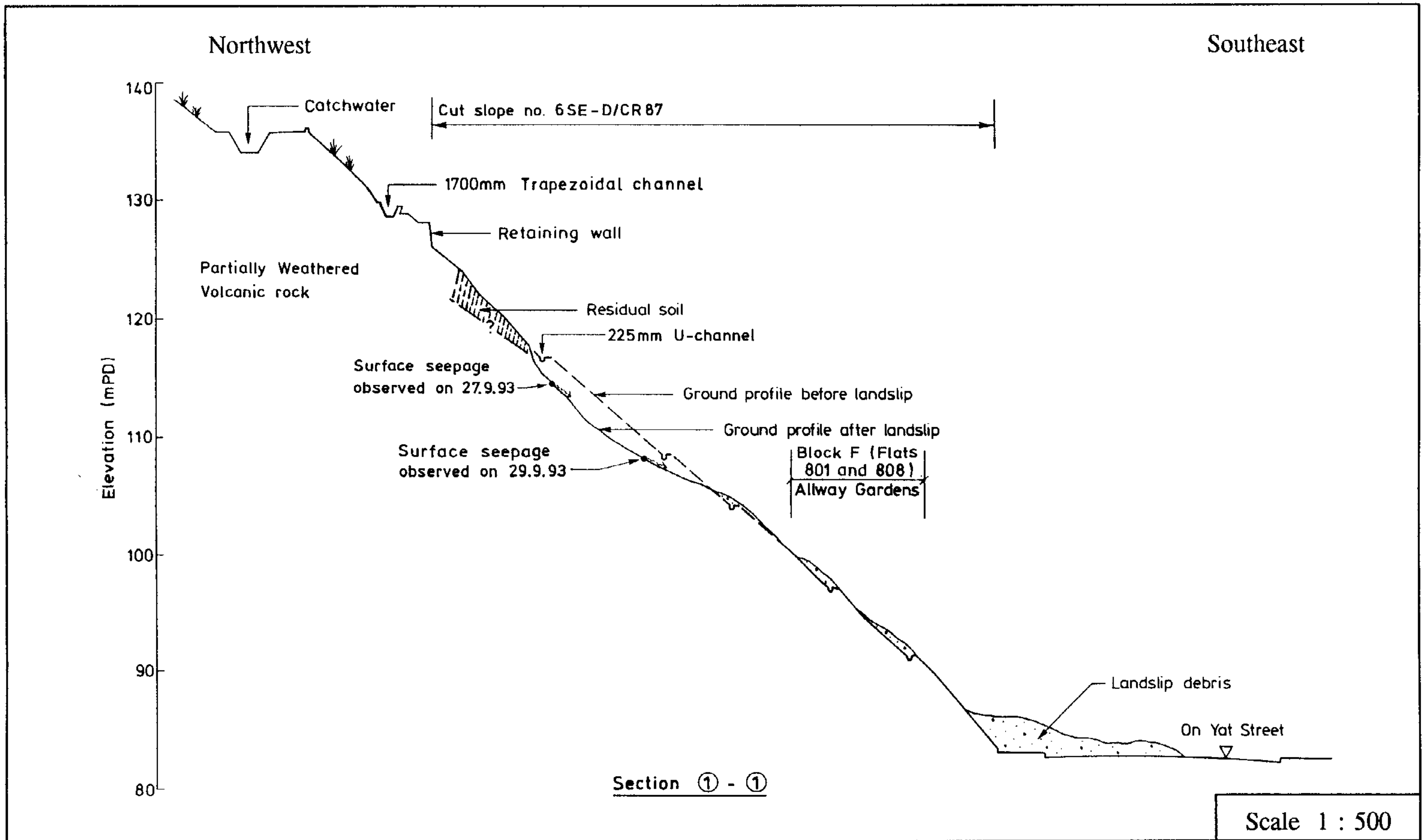


Figure 3 - Isohyets of 5-day Rainfall between 23 and 27 September 1993



Note : See Figure 1 for location of section.

Figure 4 - Cross-section of the Failed Cut Slope

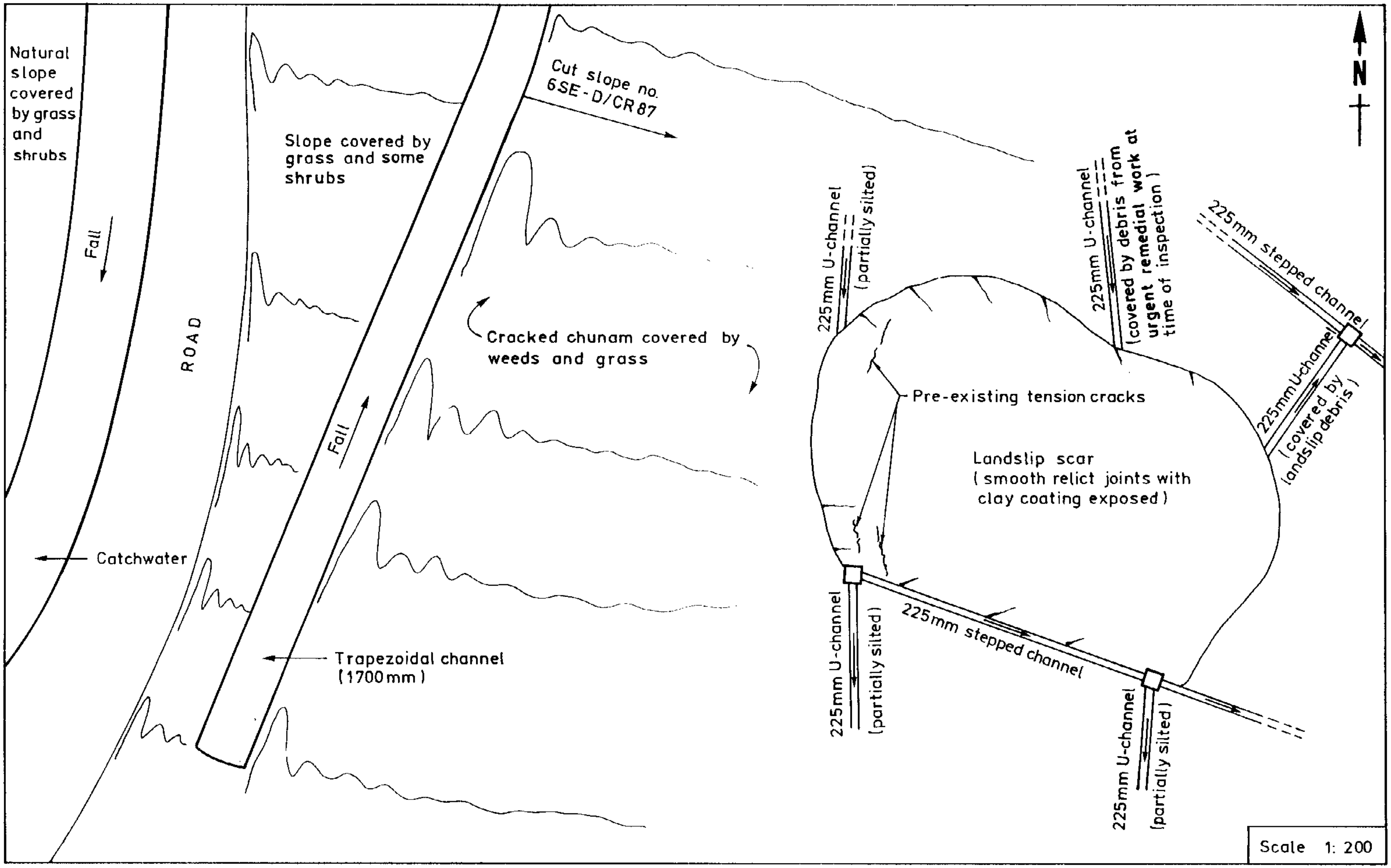


Figure 5 - Plan of Landslip Scar

SECTION 6 : REPORT ON THE INVESTIGATION OF THE 23 JULY AND 7 AUGUST 1994 LANDSLIDES AT MILESTONE 14½ CASTLE PEAK ROAD

W.K. Pun & K.C. Yeo

This report was originally produced in February 1995
as GEO Advisory Report No.ADR 1/95

FOREWORD

A series of three landslides occurred on a slope at Milestone 14½, Castle Peak Road on 23 July and 7 August, 1994. The landslide on 7 August resulted in one fatality. Immediately following the fatal landslide, the Geotechnical Engineering Office started an investigation into the landslides.

The investigation was carried out by Mr W.K. Pun and Dr K.C. Yeo of the Special Projects Division under the supervision of Mr Y.C. Chan. Topographic survey of the landslide area was carried out by the Survey Division, CED. The Photogrammetric Unit of Lands Department provided information on site topography before failure based on aerial photogrammetry.

Dr C.A.M. Franks, Dr R.J. Sewells, Mr W.L. Shum and Mr A. Hansen of the Planning Division assisted in geological aspects of the investigation. In particular, Dr Franks mapped the failure scar and assisted in the planing of the ground investigation. Mr Shum supervised the ground investigation and provided Aerial Photograph Interpretation service under the supervision of Mr Hansen.

Mr W.K. Lai and Ms C.F. Fu of the Mainland West Division provided background information about the site. Ground investigation work and laboratory testing were conducted by the Materials Division.

Y.C. Chan
Chief Geotechnical Engineer/Special Projects Division

CONTENTS

1. INTRODUCTION

In the morning of 23 July 1994, a minor landslide occurred at a cut slope at milestone 14½, Castle Peak Road near Grand Bay Villa, New Territories (Figure 1). In the afternoon on the same day, a second landslide occurred. The slope failure extended to the natural ground above the cut slope. The landslide debris pushed a grab truck off the road at the bottom of the cut slope and down an embankment onto the beach below. The driver of the truck was injured.

On 7 August 1994, a further landslide occurred at the same slope. Emergency repair works had been carried out to it after the landslides on 23 July. A public light bus travelling in the direction of Tuen Mun was hit by the debris and pushed down the embankment onto the beach below the road. A man on board was killed. Seventeen other people were taken to hospital and were subsequently released on the same day.

Immediately after the fatal landslide, the Geotechnical Engineering Office (GEO) undertook an investigation into the incident. The investigation included site surveys, collection of documentary information, examination of aerial photographs, interview of eye-witnesses, ground investigation, field and laboratory testing, analysis of the failure mechanism and consideration of the probable causes of the three related landslides. This report summarises the findings of the investigation.

2. THE LANDSLIDE SITE

The landslides involved an 8 m high, 60° cut slope, which was covered by chunam, in a steep natural slope bordering Castle Peak Road. From an examination of all available aerial photographs and old topographic maps, the cut slope appears to have been formed sometime before 1954. The exact date of the formation of the slope, however, is not known. It was not registered in the Catalogue of Slopes prepared by the consultants for the Hong Kong Government in 1977 and 1978.

Examination of aerial photographs indicated that a landslide had occurred in the cut slope in 1982 or shortly before at approximately the same location as the landslides of 23 July 1994. According to the information provided by an eye-witness and from photographs of the site taken on the morning of 23 July, the bottom of the cut slope was supported by a no-fines concrete wall with a stone-pitching facing. This wall could have been part of the repair works to the 1982 landslide, but no documentary information on that landslide or any wall construction is available.

3. RAINFALL

According to the rainfall records, the month of July and the first week of August were generally wet (Figure 2(a)). Up to 22 July, a total of about 412 mm of rain was recorded by an automatic raingauge (number N10) located at a distance of about 2.5 km northeast of the landslide area (Figure 3). Rain was heavy over the whole of the Territory on 22 July (Figure 3(a)), and about 317 mm of rain was recorded by raingauge N10 on that day. Thereafter, the rainfall intensity decreased, but heavy rain resumed again in early August.

On 6 August, rain was heavy in central New Territories, including the area along Castle Peak Road (Figure 3(b)), a total of about 287 mm of rain being recorded by raingauge N10. The 24-hour rainfalls on 22 July and 6 August were the highest and second highest respectively recorded by this raingauge since it was installed in August 1984.

The hourly rainfall recorded by raingauge N10 on the days of the three landslides are shown in Figures 2(b) and (c). On 23 July, heavy rain started at about 3.00 a.m. and continued until about 6.00 a.m., with a total of about 43 mm of rain being recorded by raingauge N10 during this period. There was little rain falling at the time of the first landslide in the morning, and there was virtually no rain at the time of the second landslide in the afternoon. Prior to the third landslide, on 7 August, the rainfall was very heavy, with the peak hourly rainfall intensity recorded by raingauge N10 between 10.00 a.m. and 11.00 a.m. being about 42 mm.

4. SEQUENCE OF EVENTS

After the landslide on 7 August 1994, the available documentary information was reviewed, and 16 persons, including eye-witnesses and people involved in the rescue operation, were interviewed by GEO officers to assist in reconstructing the sequence of events. The following information was obtained.

The minor landslide occurred shortly before 7.00 a.m. on 23 July 1994, between a cut slope in rock at the west and a no-fines concrete wall with a stone-pitching facing at the east (Figure 4). The no-fines concrete wall moved forward slightly but did not collapse. One lane of Castle Peak Road was blocked by the landslide debris. According to one eye-witness, the volume of debris was approximately 20 m^3.

At roughly 3.00 p.m. on the same day, the lower part of the cut slope to the east of the no-fines concrete wall bulged and broke a 50 mm diameter water pipe running along the toe of the slope (Figure 4). Subsequently, a larger landslide took place at about 3.30 p.m. The debris came down very quickly and pushed a grab truck, which was clearing debris from the previous landslide, down the 7 m high embankment onto the beach. The truck driver was injured, and the road was closed to traffic. The volume of debris from this second failure was estimated to be approximately 700 m^3.

Following the second landslide, the site was inspected by staff of the Highways Department and the GEO. The GEO recommended emergency repair works, before the completion of which both lanes of the road had to be kept closed. The works included removal of loose materials from the landslide scar, trimming of the landslide scar and covering of the landslide scar with tarpaulins. Emergency repair works of this type has been adopted as the general practice for many slope failure cases, and were considered adequately safe at that time for the road to be partially opened to traffic upon their completion.

One lane of the road was re-opened for two-way traffic at 5.16 p.m. on 1 August 1994. The other lane of the road remained closed for the execution of more repair works, which included the provision of a hard surface cover to the trimmed slope. A temporary barrier was erected along the edge of the road to cordon off the works area.

In the morning of 7 August 1994, the rain was heavy, and the third landslide took place at the trimmed slope at about 11.45 a.m. (Figure 4). Just before the occurrence of the landslide, a public light bus travelling in the direction of Tuen Mun stopped behind a private car in front of the temporary traffic signal that controlled the two-way traffic on the single lane of the road. As the signal turned green, the car moved forward and the bus followed. When the two vehicles arrived at the landslide location, drivers and passengers of both vehicles saw debris moving swiftly down the slope. The car passed, but the bus was hit by the debris and tumbled down the embankment to the beach below. About one third of the bus was filled with debris, and three people were trapped in the bus. They were subsequently rescued by firemen. One man was killed, and 17 other people were taken to hospital.

According to eye-witnesses, five to six subsequent minor slips occurred from the landslide scar over the next one to two hours. It was estimated that a total of about 300 m^3 of soil and rock were released from the slope in the third landslide.

The landslide debris generally comprised loose gravelly sand with cobbles and boulders, typically 100 mm to 400 mm in size, but it also contained a large boulder measuring about 2.3 m by 3.5 m by 4 m. Lumps of clayey silt material were also found in the debris.

5. GROUND CONDITIONS

Field mapping and ground investigation were conducted by the GEO shortly after the fatal landslide on 7 August 1994. Four boreholes (three vertical and one inclined at 45 degrees) and one trial pit were sunk at the locations shown in Figure 5, and soil samples were collected for laboratory tests. Based on the information obtained from the ground investigation, field mapping and a site topographic survey, the plan and cross-section of the failed slope were as shown in Figures 5 and 6 respectively.

The ground at the location of the landslides generally comprises partially weathered fine-grained and medium-grained granite, which is a soil of silty sand. Rock of medium-grained granite (slightly to moderately decomposed) was exposed in the cut slope at the western edge of the landslide scar. The partially weathered granite exhibits a well-developed, black-stained relict joint structure. The joint spacing varies from 25 mm to 200 mm. The principal relict joint sets are approximately orthogonal, with two of them being sub-vertical.

Results of laboratory tests on 'undisturbed' samples of the weathered granite have shown that the shear strength of soil at this site is akin to that of similar material found in other parts of Hong Kong.

The granite at the site is intruded by a number of sub-vertical basalt dykes, which run in a northeast direction. (A dyke is a sheet-like body of intrusive igneous rock that cuts across the original rock.) Two completely decomposed basalt dykes approximately 800 mm thick, and some thinner ones about 100 mm thick, were exposed within the landslide scar (Figures 5 and 6). Some other dykes were also found in the natural ground above the scar. The dykes were relatively thin features, generally concealed by landslide debris, and this prevented their detection at the time of the inspection on 23 July 1994.

Field assessment and laboratory tests have revealed that the basalt dykes, when completely decomposed, are rich in clay and silt, and are therefore much less permeable than the partially weathered granite. Hence, the dykes act as barriers to water. Water seepage was observed in the landslide scar on the uphill side of the two decomposed basalt dykes for a period of at least one week after the landslide on 7 August, indicating that the groundwater level was high behind the dykes. Monitoring by piezometers, which were installed in the three vertical boreholes, showed that the groundwater level was about 5 m below the surface of the natural ground for most of the time in late August and early September 1994.

The field inspection carried out a few hours after the landslide on 7 August 1994 revealed the presence of erosion gullies on the unfailed portion of the slope (Figure 5). These gullies were at 0.5 m to 2 m spacings. The widths of the gullies generally ranged from 50 mm to 200 mm, up to a maximum of 360 mm. Their depths were also in the range of 50 mm to 200 mm, up to 400 mm maximum.

During an inspection of the access road on the hillside above the landslide scar, a drainage channel along the edge of the road was observed to be completely blocked by silty sand at one location. At a short distance further downstream, the channel was blocked by construction waste (Figure 5). Because of the blockage, water was seen overflowing from the drainage channel to the natural ground above the landslide scar.

6. FAILURE MECHANISMS AND FACTORS CONTRIBUTING TO THE LANDSLIDES

The minor landslide in the morning of 23 July 1994 was likely to have been caused by a rise in the base groundwater level resulting from water infiltration into the large catchment area of natural ground above the slope during the prolonged rainfall in early to mid-July and the heavy rainfall on 22 July. Theoretical analyses have shown that the road-side cut slope was marginally stable (calculated factor of safety about 1.05) when the groundwater level was below the toe of the slope. However, it could in theory have failed (i.e. calculated factor of safety less than 1.0) if the base groundwater level rose to a level higher than about 1 m above the toe of the cut slope.

Although there was little rain after the minor landslide in the morning of 23 July, the groundwater level in the slope could have continued to rise because of the time lag between water infiltration into the natural ground and the groundwater response at the cut slope. The minor landslide was also too small to release sufficient water to change the groundwater regime in the vicinity of the cut slope.

The groundwater regime in the landslide area was controlled by a number of decomposed basalt dykes and by the rock that outcrops on the western side of the landslide scar (Figure 5). The dykes are relatively impermeable to water and could have dammed the groundwater flow and raised the groundwater level (Figure 7). Furthermore, the orientation of the dykes and the position of the rock outcrop would have allowed groundwater to converge to the cut slope in question. Surface water overflowing from the blocked drainage channel at the access road on the hillside above the failed slope could have contributed to the water infiltration into the natural ground. Theoretical analyses have shown that the second landslide could have occurred as a result of the high groundwater level.

Under most circumstances, the stability of the slope would have been restored after the emergency repair works, because the groundwater level would generally have been lowered and the gradient of the slope reduced. However, because of the presence of the decomposed basalt dykes in the slope, the groundwater level behind the dykes probably remained high. Theoretical calculations have shown that the trimmed slope between the two dykes would have failed when the groundwater level rose to about 4 m below the ground surface.

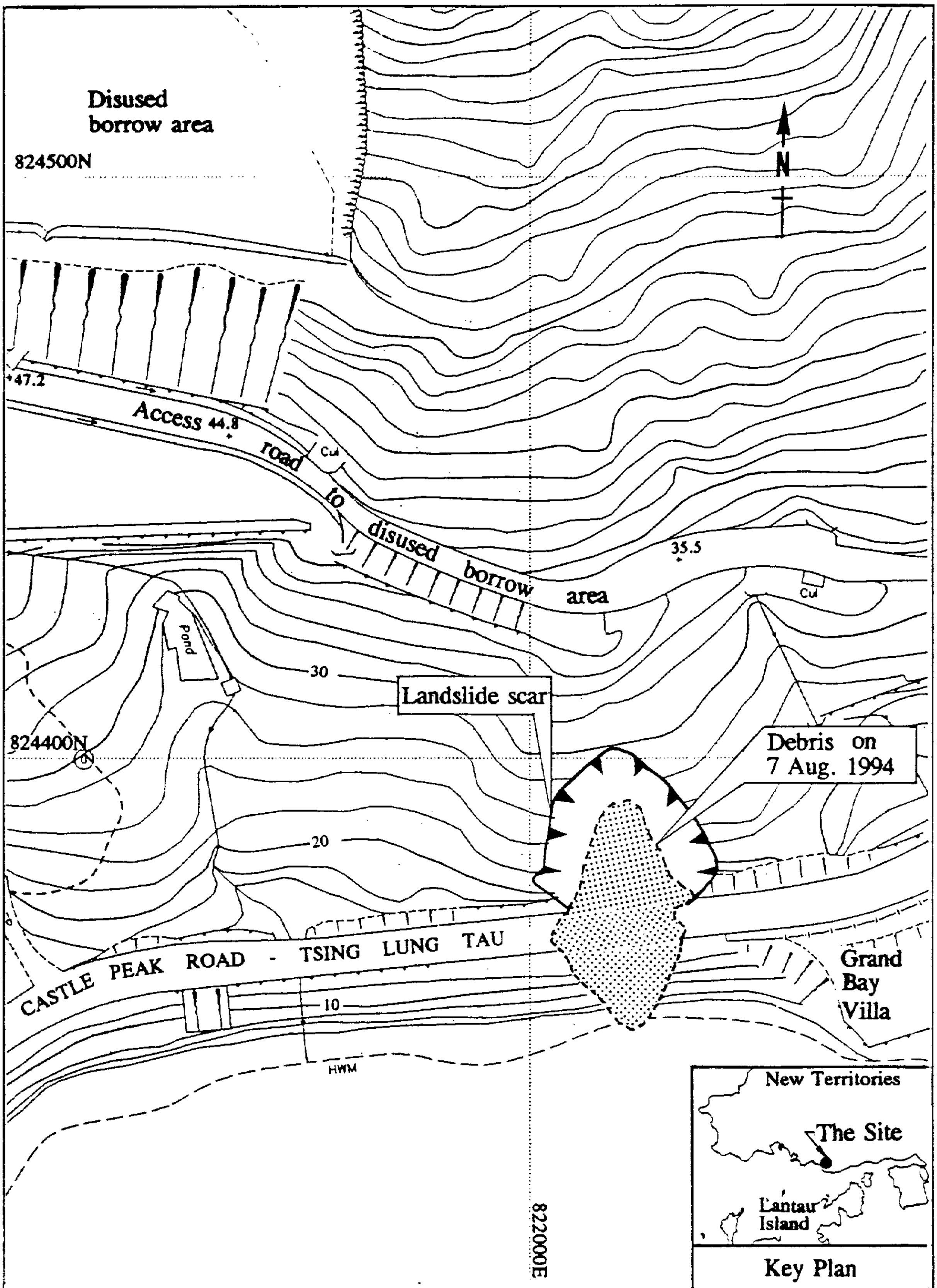
The trimming of the failure scar, as part of the emergency repair works, exposed a large corestone at the crest of the eastern part of the trimmed slope, somewhere between the two decomposed basalt dykes (Figure 4). The corestone caused a local steepening of the slope. If account is taken of the local steepening, the slope was theoretically unstable for a groundwater level of 5 m below the ground surface. Based on the findings of the theoretical analyses, the landslide on 7 August could have been triggered by a local failure under the combined effect of the high groundwater level and the presence of the corestone.

After the landslide on 7 August, erosion gullies were observed on the unfailed portion of the trimmed slope surface, indicating that the material in the slope was susceptible to erosion. It is therefore also probable that the trimmed slope had been weakened by erosion, which would have rendered a slope failure possible with a less severe groundwater condition.

7. CONCLUSIONS

Detailed field mapping and ground investigation revealed that the slope that failed had a complex geology, which was not detected immediately following the landslides because the relevant features were relatively small and were generally concealed by landslide debris. The most important contributory factors to the landslides were the prolonged rainfall and the specific geological conditions in the form of the decomposed basalt dykes and the rock outcrop. Other factors that could have contributed to the landslide on 7 August 1994 are surface water flow and consequent infiltration into the natural ground, local steepening of the slope, and surface erosion.

LIST OF FIGURES



Note : Extracted from Survey Sheet No. 6-SW-25D, date : 21-4-93, scale 1:1000.

Figure 1 - Location of the Landslide Site

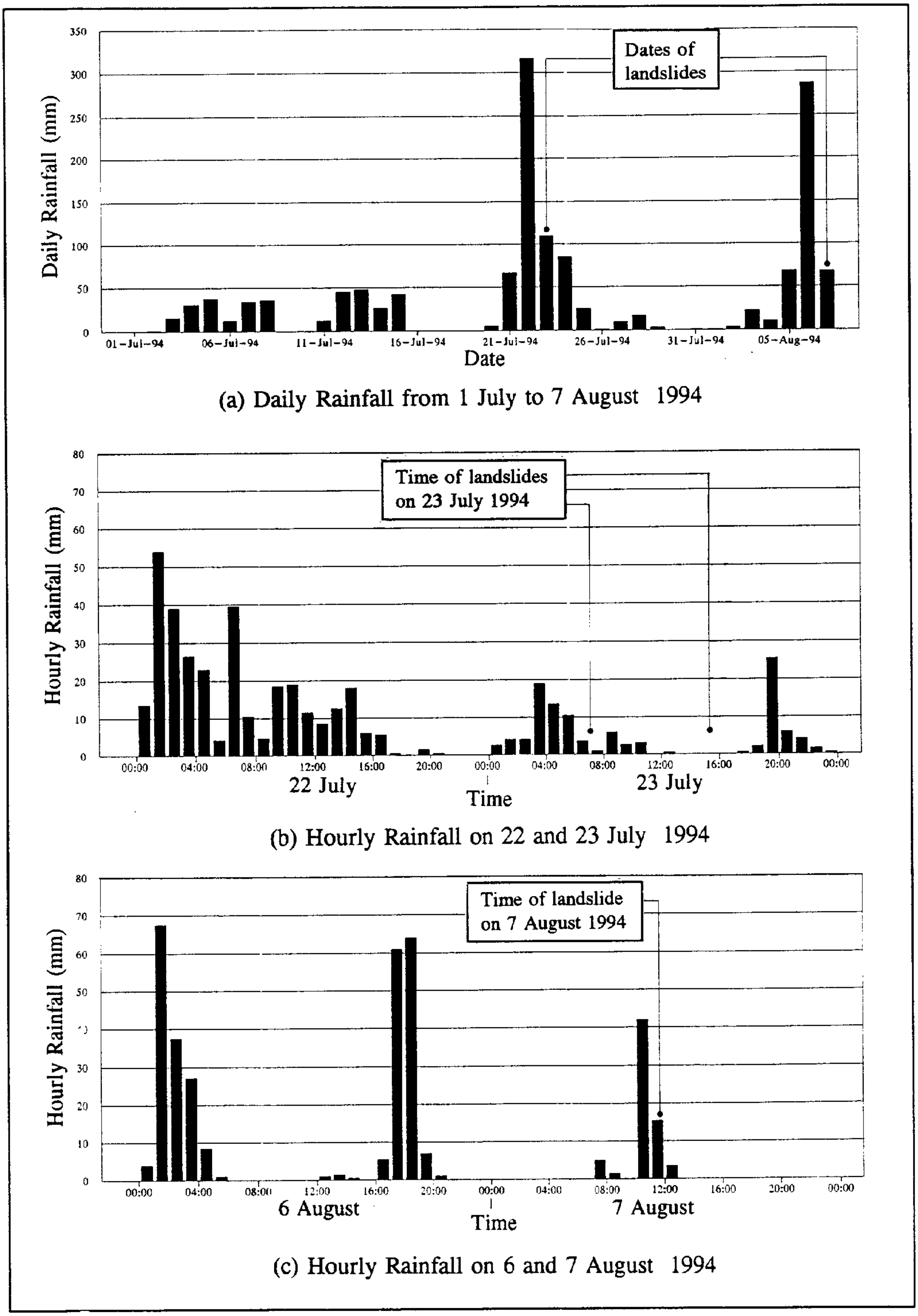
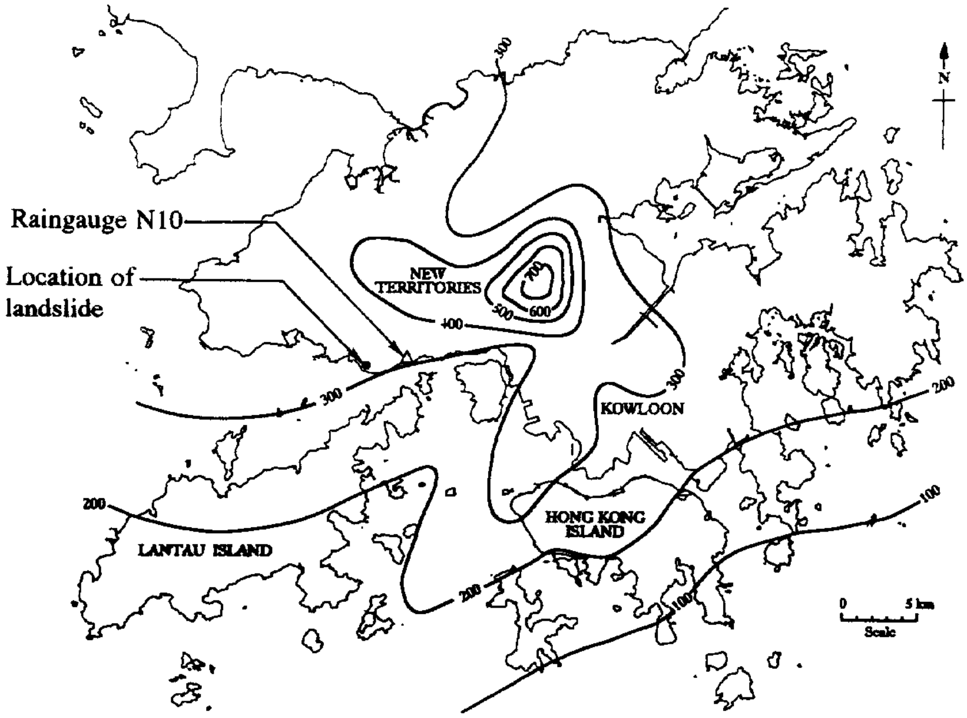
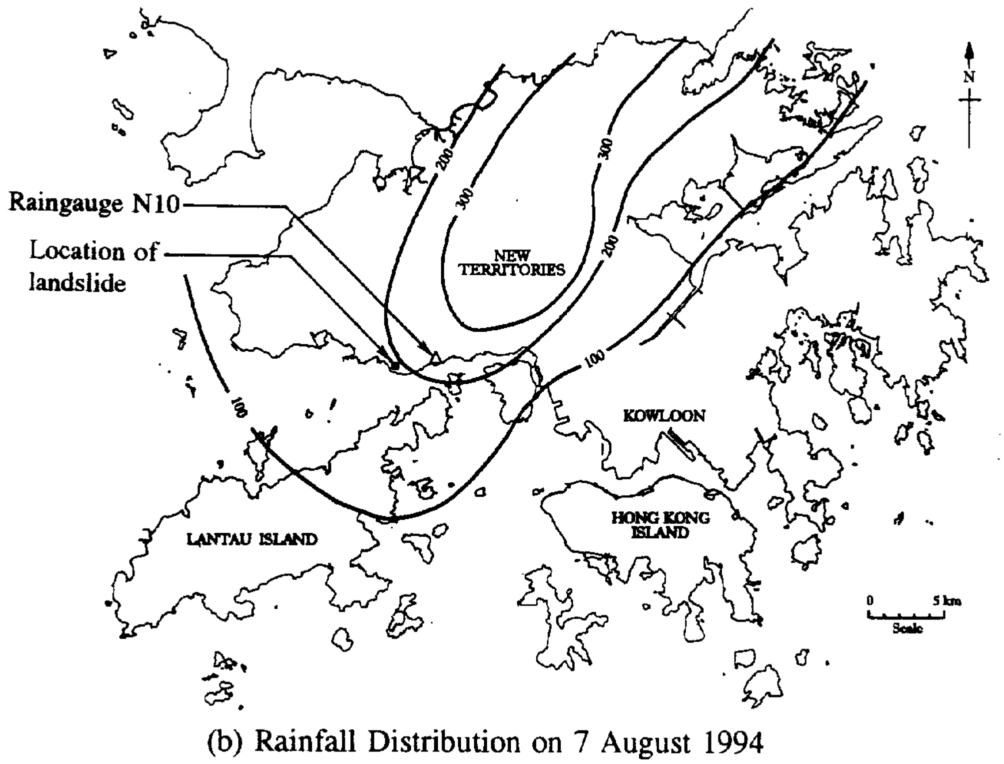


Figure 2 - Rainfall Record for Raingauge N10



(a) Rainfall Distribution on 23 July 1994

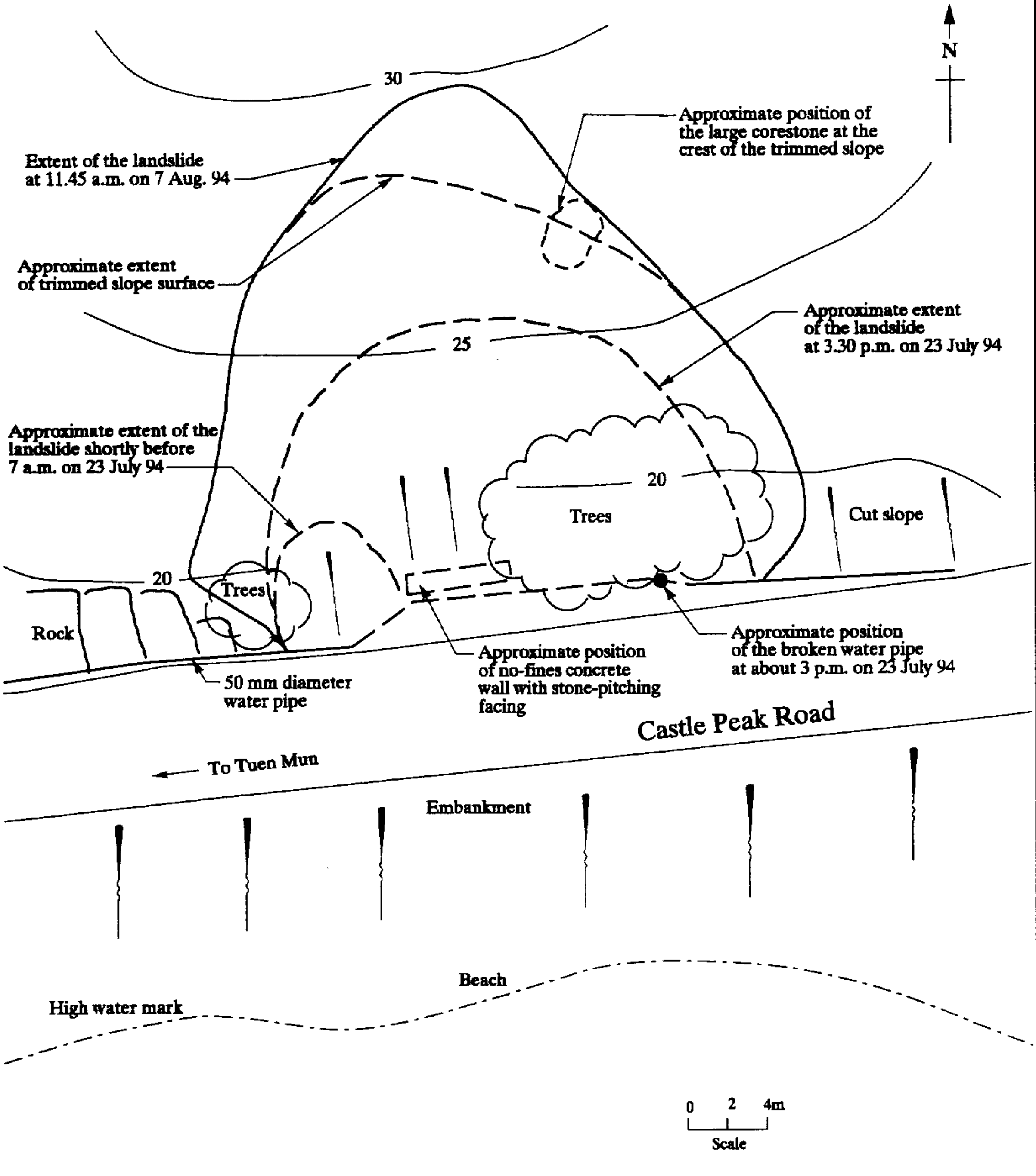


(b) Rainfall Distribution on 7 August 1994

Legend :

—100— Isohyet of rainfall in millimetres

Figure 3 - Isohyets of the 24-hour Rainfall on 23 July and 7 August 1994

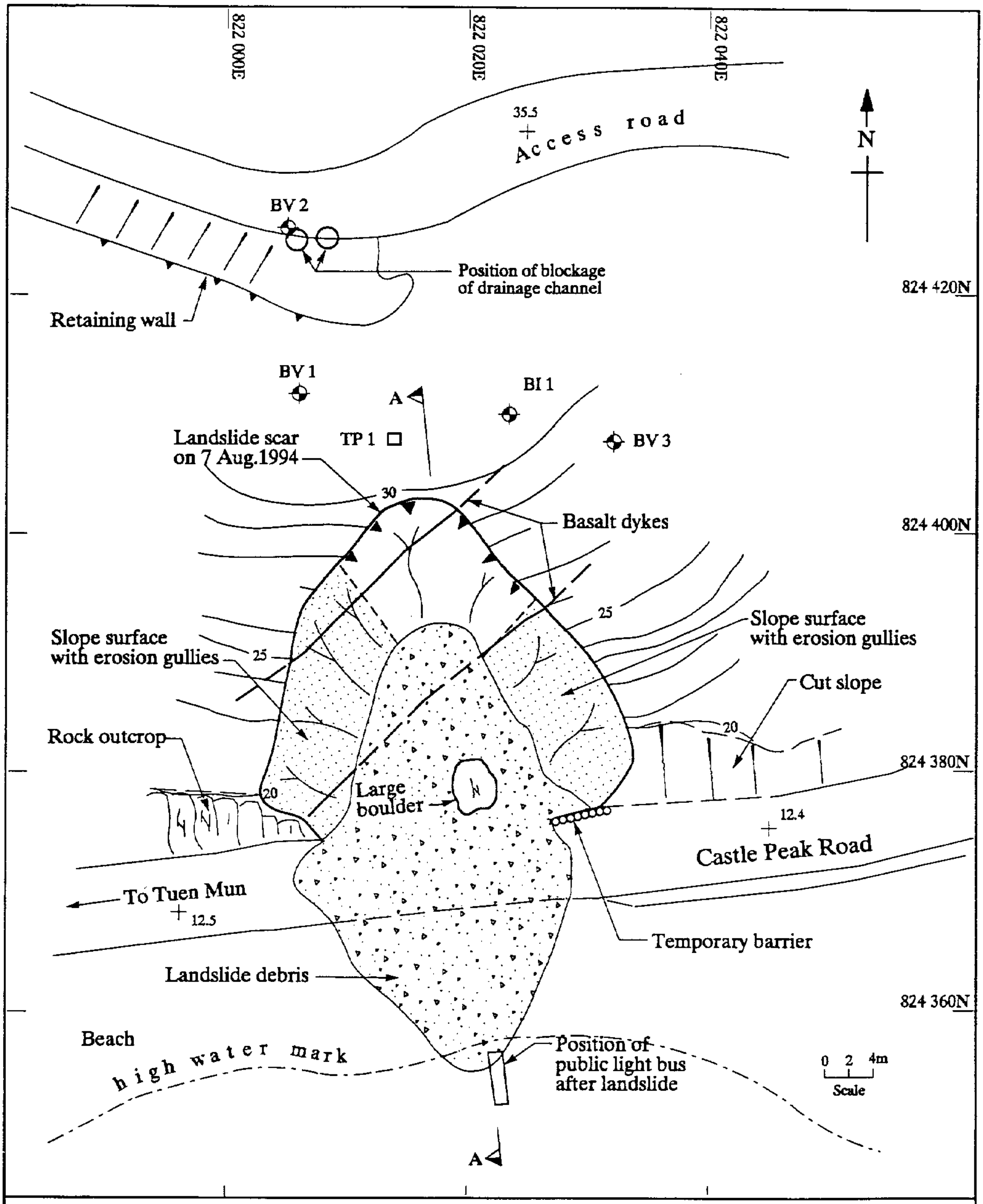


Legend :

— 30 — Contour level in metres above Principal Datum (mPD)

Note : Base map is produced from 1982 aerial photographs.

Figure 4 - Diagrammatic Sketch Showing the Three Landslide Events

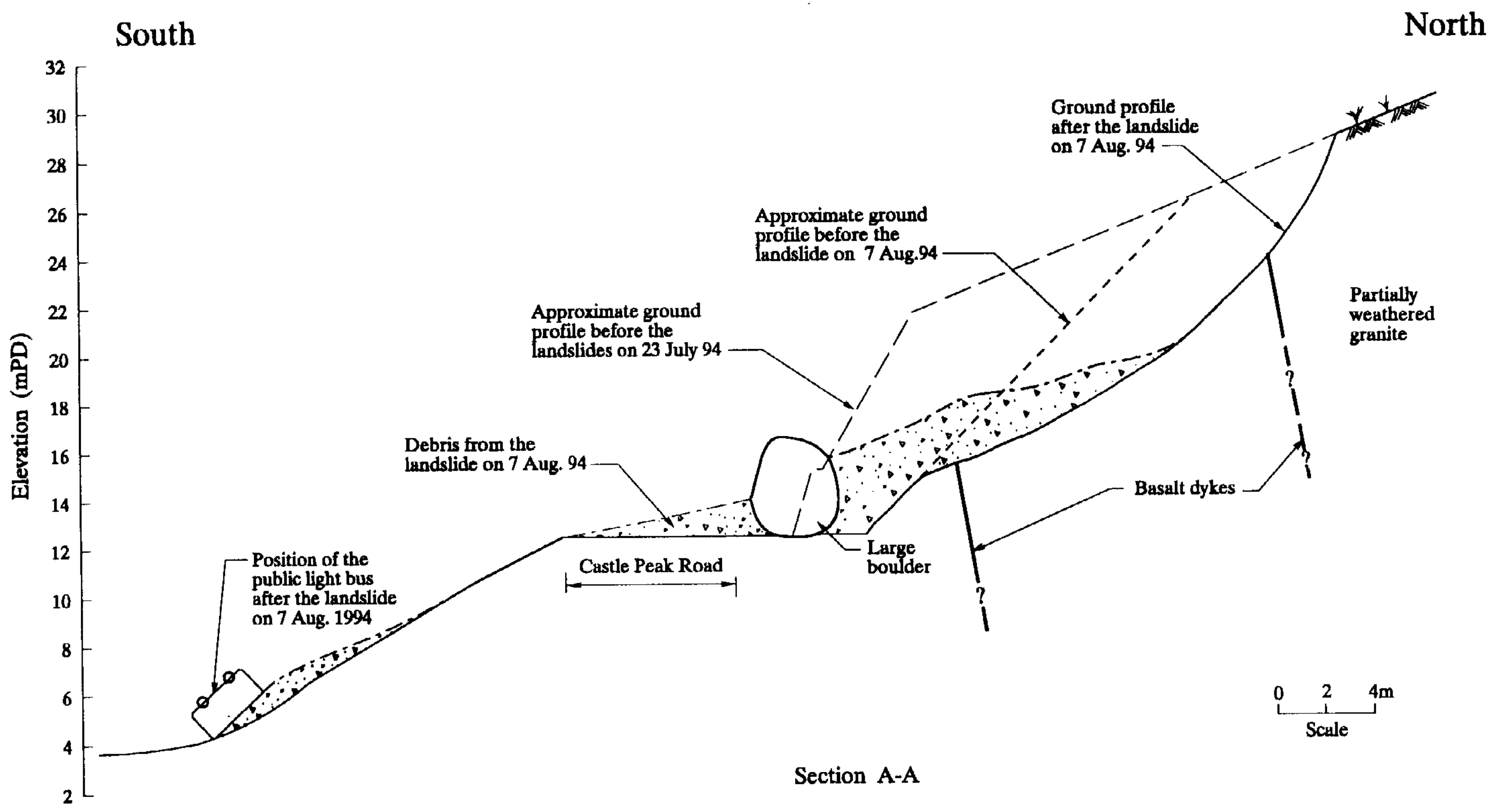


Legend :

⯐	Borehole	□	Trial pit
—30—	Contour in mPD	+ 35.5	Spot level in mPD

Notes : (1) Thinner dykes and dykes outside the landslide scar are not shown.
(2) See Figure 6 for Section A-A.

Figure 5 - Plan of the Landslide Scar on 7 August 1994



Note : See Figure 5 for location of Section A-A.

Figure 6 - Cross-section of the Hillside at the Position of the Landslides

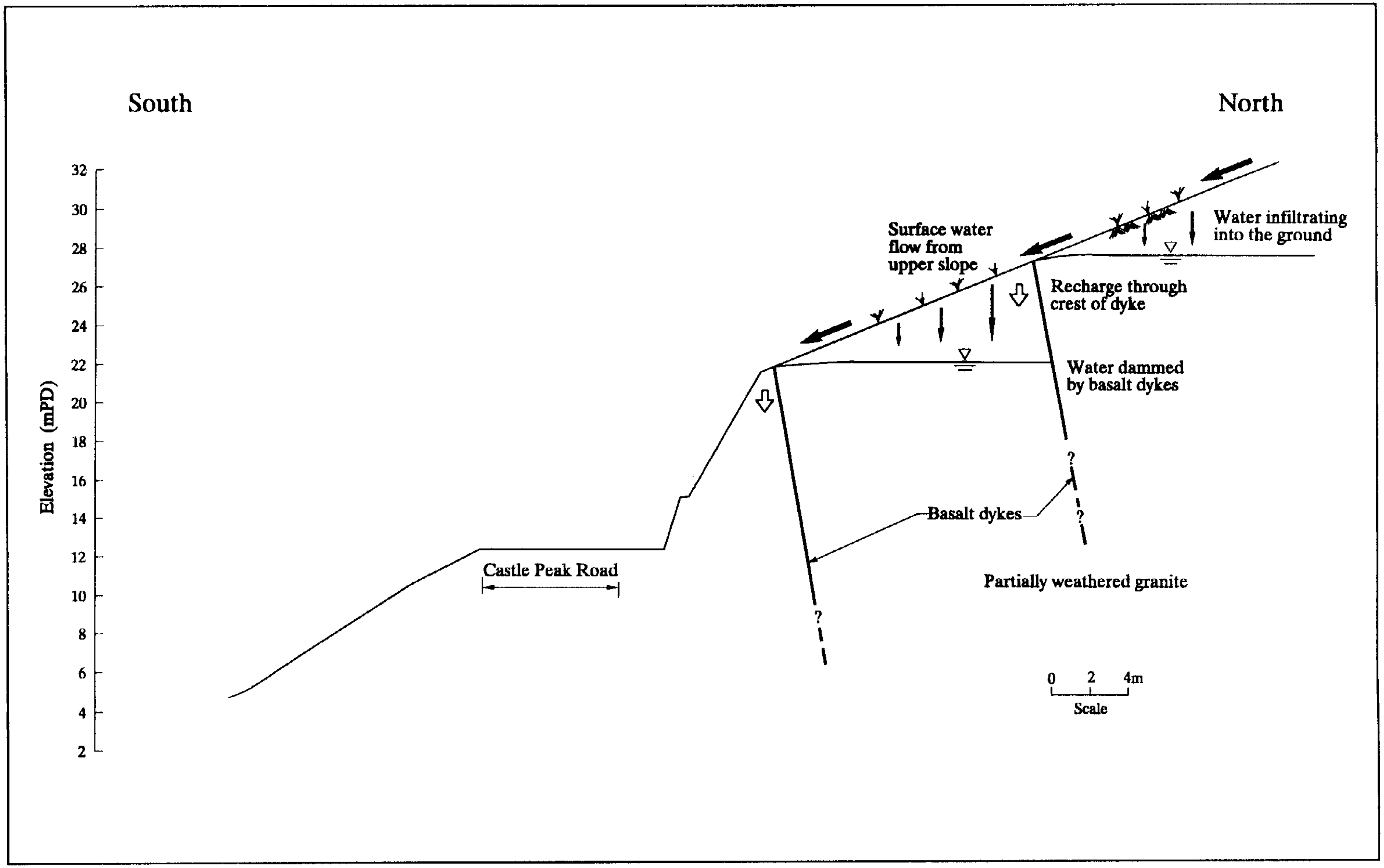


Figure 7 - Schematic Representation of the Damming of Water by the Basalt Dykes