REPORT ON THE KWUN LUNG LAU LANDSLIDE OF 23 JULY 1994

GEO REPORT No. 103

N.R. Morgenstern & Geotechnical Engineering Office

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION

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First published, August 2000

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This publication is available from:

Government Publications Centre,
Ground Floor, Low Block,
Queensway Government Offices,
66 Queensway,
Hong Kong.

Overseas orders should be placed with:

Publications Sales Section,
Information Services Department,
Room 402, 4th Floor, Murray Building,
Garden Road, Central,
Hong Kong.

Price in Hong Kong: HK$172
Price overseas: US$30.5 (including surface postage)

An additional bank charge of **HK$50** or **US$6.50** is required per cheque made in currencies other than Hong Kong dollars.

Cheques, bank drafts or money orders must be made payable to
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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. A charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the 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
August 2000

FOREWORD

This GEO Report comprises two volumes printed separately when first published in November 1994. Volume 1 contains the independent findings of Professor N.R. Morgenstern as to the causes of the Kwun Lung Lau landslide 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 were reviewed and agreed by Professor Morgenstern who relied on them in his own assessment given in Volume 1.

Y.C. Chan
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VOLUME 1: CAUSES OF THE LANDSLIDE AND ADEQUACY OF SLOPE SAFETY PRACTICE IN HONG KONG

N.R. Morgenstern

This report was originally produced in November 1994
as Report on the Kwun Lung Lau Landslide of 23 July 1994

CONTENTS

1. INTRODUCTION

At about 8:53 p.m. on 23 July, 1994 a landslide occurred below Block D at Kwun Lung Lau, Kennedy Town, Hong Kong. The landslide resulted in five fatalities, and three other people were injured.

The Geotechnical Engineering Office (GEO) of the Civil Engineering Department (CED) commenced an investigation of the landslide late on the evening of 23 July, 1994. The investigation involved a variety of activities ranging from interviews of witnesses to geological studies and to evaluation of groundwater sources. A Progress Report from the GEO was released to the public on 9 August, 1994 that described the site and the landslide. It also described the investigation that was in hand at the time and noted the possible factors that could have contributed to the landslide. They were multiple.

The interim report emphasized that the preliminary findings and evaluations presented could be invalidated by subsequent studies and that further work was required before more definite conclusions could be reached. The Progress Report promised that the findings of the landslide investigation would be documented in a final report. Volume 2 of the Report on the Kwun Lung Lau Landslide comprises this final document.

On 3 August, 1994 the Writer was approached by the CED to participate in the inquiry into the Kwun Lung Lau landslide and he agreed to do so. An Agreement was drawn up with the Civil Engineering Department, Government of Hong Kong, and signed later in August, 1994.

2. TERMS OF REFERENCE

The following, taken from the Agreement, constitutes the terms of reference for Volume 1 of this Report.

"2. Study Background

A landslide occurred below Block D of Kwun Lung Lau, Kennedy Town on 23 July 1994. The incident resulted in five fatalities and three other people were injured. The Geotechnical Engineering Office (GEO) is carrying out an investigation into the cause of the landslide.

It is proposed to have an independent review of the above investigation.

3. Objective of the Assignment

The objective of this assignment is to review the GEO investigation of the Kwun Lung Lau landslide, which occurred on 23 July, 1994, to report on it independently to the Government of Hong Kong and to advise the Government on the adequacy of its current approach to slope safety in Hong Kong.

4. Deliverables

A study report on the review of the GEO investigation into the Kwun Lung Lau landslide shall be submitted on completion of the investigation. A separate report on the adequacy of Hong Kong Government's current approach to slope safety shall be submitted upon the completion of this assignment. Interim review reports may be submitted during the course of this assignment as agreed with the Director's Representative.

5. Services to be Provided by the Consultants

Services to be provided by the Consultant are to carry out a review of the GEO investigation of the Kwun Lung Lau landslide, to report his findings independently to the Governor of Hong Kong and, in the light of the findings of the investigation of the Kwun Lung Lau landslide, to advise the Government on the adequacy of its current approach to slope safety in Hong Kong.

6. Response to Queries

The Consultant shall respond to queries under Clause 20 of the General Conditions of Employment raised prior to a date six months after the final submission of the Deliverables required under the Agreement. Such date shall be confirmed in writing to the Consultant by the Director's Representative.

7. Programme of Implementation

The due date for commencement of the Agreement is the date when this agreement is signed. The work shall continue until the Consultant has completed the final report for submission to the Governor. Subject to the receipt of information in a timely manner from the Employer, completion is intended on or before the end of November 1994 unless otherwise advised and agreed by the Director's Representative."

3. ORGANIZATION OF THE REPORT

This Report is presented in two volumes. The first volume prepared by the Writer is intended to meet the objectives outlined in Section 2, Terms of Reference. The second volume, prepared by the GEO, and reviewed by the Writer, presents the results of the comprehensive investigation conducted into the factors that have contributed to the Kwun Lung Lau landslide. The Writer is satisfied that this investigation has been conducted to the highest standards practicable and that all issues considered relevant by the Writer or the GEO investigation team have been pursued. Clearly Volume 1 relies on Volume 2 as a resource document, but the evaluation, opinions, and recommendations presented in Volume 1 are attributable to the Writer alone.

While the Terms of Reference call for two reports from the Writer, one dealing with the subject landslide itself and the other with the adequacy of the Hong Kong Government's current approach to slope stability, he has deemed it preferable to combine the two issues in a single report. The Writer has purposefully limited his evaluation of slope management matters to those issues arising from the lessons learnt from the Kwun Lung Lau landslide as opposed to a broadly-based assessment. Therefore the two components of the Writer's assignment are best presented in a single report since they are intimately related and one issue follows on from the other.

In the following section, the Writer's conduct of his study is outlined in detail. This is intended to make transparent all of the work elements that have led to this volume of the Final Report.

The subsequent section discusses the factors that have contributed to the Kwun Lung Lau landslide and the causes of the landslide are presented in the section that follows. The issue of preventability of the Kwun Lung Lau landslide is then raised. The lessons learnt from the tragic event are discussed and this volume of the Report ends with specific recommendations.

4. ORGANIZATION OF THE STUDY

Following the agreement to undertake this study, the GEO provided the Writer with the results of its interim investigation as well as various background documents to slope stability in Hong Kong. The Writer arrived in Hong Kong for his first visit on 19 August, 1994. The purpose of this visit was to :

(a) observe the site and the field investigations then underway,

(b) review in detail the investigation and studies proposed by the GEO,

(c) provide additional emphasis or re-direction to various work elements of the study as needed, and

(d) establish a time frame for completion of the study.

Prior to departure from Hong Kong on 23 August, 1994 the Writer appeared before the Legislative Council of Hong Kong, Panel on Lands and Works, as well as before the Press, to advise that the objectives of this first visit had been met and that he was content with the effort that the GEO were devoting to the field investigation. Some re-direction had been given with regard to field investigation, laboratory testing, and analytical issues.

In the subsequent period, prior to his second visit, the Writer received weekly to biweekly progress reports on the investigation and various draft documents to review as they were prepared. The Writer arrived in Hong Kong for his second visit on 22 October, 1994. The intent of this visit was to review in detail the results of all field, laboratory and documentary investigations and to agree on various theoretical analyses to be conducted. It was anticipated that this would mark the end of the information gathering phase of the study and allow concentration on the evaluation phase. The Writer advised the Legislative Council of Hong Kong, Panel on Planning, Lands and Works, as well as the Press, that for all practical purposes the investigative phase was complete and that causes of the landslide had been determined. He departed from Hong Kong on 27 October, 1994.

During the next few weeks the Writer received draft material for Volume 2 to review and began drafting his own report. He returned to Hong Kong on 25 November in order to complete the Report which was transmitted to His Excellency, the Governor, on 30 November, 1994.

5. FACTORS CONTRIBUTING TO THE KWUN LUNG LAU LANDSLIDE

5.1 Introduction

It is well-known that intense rainfall triggers landslides in Hong Kong and this is the rationale behind the landslip warning system implemented in Hong Kong. The landslip warning was in effect at the time of the Kwun Lung Lau landslide but it is clearly not sufficient to simply conclude that the landslide was caused by high intensity rainfall. The landslide became a tragedy because of the magnitude of the slide, its sudden development and its mobility. Had it been smaller, or provided warning, or moved only slightly, it is likely that it would not have resulted in deaths and personal injury.

Therefore the explanation of the causes should be constrained not only by the time and location of the landslide, but also by its style of movement. In addition, the explanation should also be consistent with observations made at the time. Witnesses accounts are summarized in Appendix F, Volume 2. Among the more significant observations are the following :

(a) Prior to the slide, muddy waters had been observed seeping through the weepholes and joints of the masonry wall supporting the slope. The extent of the seeping zone increased upward with time.

(b) Caretakers of Kwun Lung Lau had inspected the slope above the masonry wall from the slope crest twice on the day before the landslide. There were three inspections on the day of the landslide. The last inspection was about six hours before the landslide. Neither signs of defects nor blockage of drainage channels were detected.

(c) About two and a half hours before the landslide, a witness reported seeing a hole in the chunam near the manhole on the slope at the eastern edge of the landslide area while walking along the footpath. The hole was said to be about 1 m in diameter, and soil was apparently exposed due to the loss of the chunam cover.

(d) Rain was light at the time of the collapse.

(e) Failure was sudden, taking place over a very short period of time.

(f) The masonry wall burst out at about mid-height, followed by the instant collapse of the wall and slope.

A large number of factors need to be considered in order to put forward an explanation that best reconciles these and other observations arising from the comprehensive investigation summarized in Volume 2. These factors are discussed separately below. Volume 2 contains reference figures to assist the reader, in particular the Site Location Plan (Figure 2), the Plan of the Landslide (Figure 3), Sections (Figures 4 & 11), and Layout of Drains (Figures 8 & 9).

5.2 Masonry Wall

The masonry wall supported a cutting made in native ground consisting of decomposed volcanic soils and rock. A review of early maps, Appendix B, Volume 2, reveals that the wall could possibly have been built by 1889. There is an indication from the maps that the wall was in place by 1901. It has been shown explicitly on maps since 1959.

Detailed observations on the wall are provided in Appendix E, Volume 2. It is significant that the wall was only about 750 mm thick. Such thin walls are relatively uncommon. Based on studies of typical old masonry walls in Hong Kong, a base width of 3000 - 3500 mm would be expected, as opposed to the 750 mm measured. It appears that the wall was conceived as a thin facing to protect against shallow instability and erosion of the cutting, as opposed to providing any structural support against deep-seated movement. The high slenderness ratio of the wall (height/base-width) was almost without precedent in Hong Kong, for a wall of its height.

5.3 Site Development

Documents available to record the housing estate development are listed in Appendix A, Volume 2. The statutory checking system at the time of the development of Kwun Lung Lau is described in Appendix D, Volume 2. No evidence of an assessment of the stability of the masonry wall has been found. The Site Formation Plans for Kwun Lung Lau submitted by the Authorized Person and approved by the Building Authority on 16 March, 1965 showed the geometry of the wall in solid lines to be a stepped masonry wall with a base width of about 4 m. This was obviously incorrect.

It was common practice at the time to shape platforms for estate development by cut and fill. Fill was generally placed in thick lifts and poorly compacted. In some instances fill was end-dumped. The consequences of poorly compacted fill in contributing to previous landslide disasters in Hong Kong, such as Sau Mau Ping, are well-known.

At Kwun Lung Lau, several metres of fill were added to the slope above the masonry wall. The re-constructed cross-sections before the landslide, which display this fill, are given in Appendix H, Volume 2, Figures H8 - H11. In addition, considerable fill adjacent to the building was used to establish grade on the south side of Block D, Kwun Lung Lau. An indication is provided by Appendix K, Volume 2, Figure K6. Part of the foul sewer system was buried in the fill on the slope while part of the stormwater system was buried in the fill to the south.

The presence of the poorly-compacted fill had three deleterious effects :

(a) it added additional load to the wall thereby reducing its stability,

(b) because of its comparatively high permeability, it could readily imbibe moisture and transmit it, and

(c) because of poor compaction it would settle with time and wetting, thereby contributing to the cracking and subsequent deterioration of services buried within the fill.

5.4 Groundwater and Water Infiltration

The Kwun Lung Lau landslide was about 3-5 m deep. It has been well established for some time in Hong Kong that the stability of slopes in fill and decomposed volcanics is enhanced by the state of the pore water in the soil which develops a suction as a result of a moisture deficit. As water infiltrates into the slope, this suction is reduced and the propensity for landsliding increases. Much of the slope maintenance work in Hong Kong such as that related to maintaining the integrity of the chunam cover on many slopes is directed at minimizing surface infiltration. Any source of water that would increase the moisture content in the upper few metres of the soil profile would be a factor contributing to the slide that occurred. There are three potential sources that need to be considered. They are :

(a) Groundwater table fluctuation.

(b) Surface infiltration.

(c) Infiltration from buried services.

Groundwater monitoring undertaken as part of this study is summarized in Appendix H, Volume 2. The contours of groundwater level observed indicate that the highest recorded groundwater level was about 3 m below the toe of the landslide scar and about 13 m below the crest of the landslide scar. The groundwater level was well below the landslide scar shortly after the landslide and remained low. Groundwater levels in Hong Kong are known to respond to regional factors as well as local rainfall. Field observations made immediately after the failure on 23 July, 1994 found no trace of significant water seepage at the landslide scar indicative of a groundwater table or perched water table having risen above the slip surface at the time of the landslide. Therefore the Writer concludes that a rise in the groundwater table can be discounted as a factor contributing to the cause of the landslide.

There are numerous examples of landslides in Hong Kong having been triggered by surface infiltration. The presence of chunam and other surface drainage and erosion control features is partly intended to reduce wetting due to surface infiltration. Over the years a number of inspections of the retaining wall and the slope above it were undertaken by Consultants to the Hong Kong Housing Society (HKHS) and by the GEO. Observations were made related to minor surface deterioration, characteristic of normal wear and tear. In the view of the Writer, the HKHS was generally responsive to recommendations made with regard to surface slope maintenance.

The last inspection before the landslide was undertaken by Consultants to the HKHS and reported on 15 June, 1994. They observed : "Subsidence and cracks were noted near the slope crest. Unplanned vegetation on the retaining wall. Unstable tree at the toe of the slope". They commented that the slope was in fair condition and made recommendations with regard to maintenance, including the recommendations that a leaking manhole and associated pipelines be checked and the leaking points be made good. The HKHS proceeded to prepare tender documents for this work but the landslide occurred before any further action could be taken.

During the rainstorm, the caretakers at Kwun Lung Lau appeared to have been observing slope condition in a diligent manner and they commented that there was no apparent surface deterioration of the chunam up to their last inspection made six hours before the landslide.

Hence, the conditions of the surface cover of the slope prior to the landslide are known. While some minor defects existed, the chunam and related surface drainage features appeared to fulfil their intended function and it does not appear possible to attribute the wetting of the ground to abnormal surface infiltration directly from the slope.

Another potential source of surface infiltration that could find its way to the landslide arises in the yard area to the south of Block D. However, this area is paved except for the garden portion. Infiltration tests on the near-surface soils in the garden area indicated relatively low permeability. Therefore the capacity for direct surface infiltration from this source contributing much water to the landslide is limited.

The most important source of water communicating directly to the potential landslide mass is sub-surface infiltration arising from defective foulwater and stormwater drains. The results of a detailed investigation of the drainage system in the Kwun Lung Lau area are given in Appendix J, Volume 2.

The 300 mm diameter foulwater sewer traversed the upper part of the landslide area and it was ruptured by the slide. Segments recovered revealed that the pipe was of a rigid construction and there was black staining on the soil adjacent to the pipe, symptomatic of long term leakage. This was confirmed by chemical analyses of soilwater obtained from samples at depth below the landslide base and downslope of the sewer. The analyses displayed high chloride content characteristic of the seawater used for flushing. A considerable time would be required to achieve the extent of penetration of the contaminated water and therefore it is likely that this sewer had been leaking for some time before the slide.

The investigation also discovered that two stormwater drains in the yard area to the south of Block D were defective in a manner that would allow stormwater to enter the porous subsoil, as opposed to being carried away. The leakage from these sources was likely to be significant and some attempt to quantify it is given in Appendix L, Volume 2.

The chemical analyses of soilwater are also helpful in documenting this additional source of infiltration. Water samples have been extracted from high moisture content specimens located above the elevation of the foulwater line. These samples display a low chloride content confirming that there was an additional source of water entering the ground, one that is not contaminated by chloride. The investigation has also confirmed that the foulwater source and the stormwater source are unconnected.

5.5 Rainfall

Rainfall records have been obtained at Kwun Lung Lau since September, 1978. July, 1994 was the wettest month recorded since the local raingauge was installed. Rainfall data and related statistical information are compiled in Appendix G, Volume 2. There can be little doubt that this exceptional rainfall was an important factor contributing to the cause of the landslide. However the timing of the landslide with respect to rainfall history leads to some interesting observations.

Previous detailed studies by the GEO into the relationship between rainfall and landslide incidence have revealed that the local short-period rainfall intensity was the major cause of severe landslides which led to substantial damage. Correlations indicated that if the 24-hour rainfall is greater than 175 mm, then the occurrence of landslides is almost certain. Most landslide activity occurs almost coincidentally with the maximum hourly rainfall intensity and is generally complete within the subsequent twelve hours.

The maximum 24-hour rainfall was 362 mm at 18:30 on 22 July, 1994 and the maximum hourly rainfall intensity was 101 mm at 03:05 on 22 July, 1994. The landslide occurred at around 21:00 on 23 July, 1994, about 42 hours after the maximum hourly intensity. Moreover, only 29 mm of rain were recorded in the ten hours before the landslide. While the 48-hour rainfall was relatively rare, with an estimated return period of 28 years, about 95% of the 48-hour rainfall fell more than 10 hours before the landslide.

The Kwun Lung Lau landslide does not fit the normal pattern of rainfall triggering of landslides observed in Hong Kong. It is an example of a delayed failure that occurred some prolonged time after the end of the highest rainfall intensity. Some examples of delayed failure have been encountered in Hong Kong but they are not common.

The delay process implies a stronger correlation with antecedent rainfall effects and an indication of the existence of fluid pathways that attenuate the influence of the high intensity rainfall peaks. The more immediate response correlation which was not the case here is consistent with instability triggered by shallow surface infiltration effects.

5.6 Soil Strength and Deformation Properties

Extensive laboratory tests were conducted on soil samples obtained from the site investigation. Details are provided in Appendix I, Volume 2. Essentially two types of soil were encountered, (a) fill, (b) decomposed volcanics.

The fill was shown to be very loose to loose and therefore disposed to considerable volume reduction when wetted and loaded. Strength test results were consistent with previous experience with these materials.

The decomposed volcanics are denser than the fill and dilatant when sheared. The strength tests for these specimens were also generally consistent with past experience with these materials.

Of special interest were the tests designed to simulate the stress and strain response of the decomposed volcanics under conditions that more closely reproduce the behaviour of a slope failing by increased wetting or a gradual increase in pore pressure. These tests displayed much stiffer and more brittle behaviour than the routine tests. Strains to failure were small, generally less than 1%. The implication of these findings is that little deformation in the decomposed volcanics would develop prior to failure of the slope, which minimises the capacity of the slope to display warning signs before ultimate collapse.

6. ANALYSES OF THE KWUN LUNG LAU LANDSLIDE

In addition to an understanding of the factors that have contributed to the cause of the Kwun Lung Lau landslide, various analyses have been undertaken in support of the explanation of cause. Details are provided in Appendix L, Volume 2. Several analyses are particularly noteworthy.

The limit equilibrium analyses are undertaken to demonstrate consistency in the argument that reduced soil strengths as a result of wetting brought the slope and wall into a state of limiting equilibrium where movement would occur. The strength parameters operational at the time of failure cannot be determined with precision because the actual degree of wetting at the time of failure is not known. The analyses are also dependent on the strengths adopted for the mortar joints in the masonry wall which are highly variable and therefore can only be assumed within reasonable bounds.

It is of interest to analyse the 1924 ground profile that had remained stable since its formation. The limit equilibrium analyses indicate that regardless of how wet the 1924 ground profile became, failure would not have occurred. This is compatible with historical experience and it provides a test for the consistency of the analysis. For the 1994 profile, failure occurs when the wetting of the decomposed volcanics has reduced the cohesion of the soil to 6 kPa or less. These results are reasonable and are sensibly constrained by laboratory data.

Analyses were also undertaken to simulate how the ground and wall might move, once movement was initiated. It is important to note that this analysis, a deformation representation, is entirely independent from the analyses described above. The results of the deformation analysis indicate that for significant wall movements to develop, the strength must drop as a result of wetting to the same range as found from the limit equilibrium analyses. However, the presence of the wall inhibits failure from taking place until the soil has been weakened by wetting to an extended depth. That is, the wall promotes the accumulation of a greater mass of potentially unstable material than would have accumulated without the wall. When the wall begins to fail, it fails by buckling at its mid-height. The predicted failure mode and resulting post-failure attitude of the wall mimics closely the observations made in the field, in particular the front face of the wall is predicted to face upward in the debris, as was encountered.

Seepage analyses are presented in Appendix L as well. These analyses are intended to be illustrative of the process of how water might find its way from the defective storm sewer, through the pathways discovered beneath Block D, and into the landslide mass. They also indicate how time is involved in this process thereby accounting for the delayed failure and they show how infiltration from the two buried drainage pipes can account for the extensive depth of wetting that occurred.

7. CAUSES OF THE KWUN LUNG LAU LANDSLIDE

The Kwun Lung Lau landslide occurred as a result of sub-surface infiltration from defective buried drainage systems. Had the buried services been performing without leakage, insufficient moisture would have entered the ground from surface sources during the rainstorms of July, 1994 to have caused instability.

It is likely that the foulwater sewer had been leaking for some time prior to the landslide. The sewer was of a rigid construction and it was buried in loose fill which could have settled differentially in the past resulting in cracking of the earthenware pipes and/or their joints. Considerable organic staining in the soil adjacent to the pipe and deep migration of chloride contaminated water attest to the long duration of foulwater sewer leakage. Leakage from the foulwater sewer alone was inadequate to account for the slide. If one were to attribute most of the wetting to be associated with infiltration from the foulwater sewer, it would be necessary to conclude that any relationship with the rainfall was only coincidental since the foulwater and stormwater systems were not connected.

Leakage from the defective stormwater system was an important factor. Pathways for substantial subsurface flow beneath Block D into the landslide mass have been shown to exist, see Appendix K, Volume 2. Water chemistry tests reveal that there is another source of water contributing to saturation of the landslide mass, other than the chloride contaminated foulwater. In addition, the pathway from the defective stormwater system to the landslide provides a logical explanation for the delayed failure process.

It is likely that the additional infiltration of stormwater brought the landslide mass close to failure. As movements developed, the foulwater sewer ruptured. An increase in foulwater flow would account for the observation by a witness of a hole in the chunam near the manhole on the slope at the eastern edge of the landslide area.

The two sources of water began to saturate the soil mass and increased volumes of water reached the retaining wall. The sequence is illustrated in Figure 12, Volume 2. Initially, when the depth of wetting was relatively low, the wall remained stable and held the weakened soil back. As the depth of wetting increased the forces on the wall increased until failure was initiated. Failure occurred by buckling and brittle collapse. The volume and mobility of the slide mass was increased by the presence of the wall. The failure mode was sudden providing little to no warning to anyone at risk. That is, the presence of the wall aggravated the consequences of the landslide. Had the wall been thicker, it likely would have deformed in a more ductile manner which could have provided some warning of the impending danger.

8. PREVENTION OF THE KWUN LUNG LAU LANDSLIDE

Was the Kwun Lung Lau landslide preventable?

Based on the explanation of the cause of the landslide presented above, it is evident that there were a variety of means available to prevent the landslide. Had the drainage systems not been defective, it is unlikely that sufficient moisture would have entered the ground to cause any instability. Had the retaining wall been made stronger, it could have resisted the slope instability or at least deformed in a ductile manner which would have provided some warning of danger. Even the weak fill and upper decomposed volcanics could have been removed or possibly strengthened although this would not likely be a method of choice.

All of the methods listed here are conventional in nature. There appear to be no exotic phenomena involved in the Kwun Lung Lau landslide and no exotic methods would be required to prevent it. All methods required to prevent the Kwun Lung Lau landslide are available in Hong Kong and have been so for a number of years.

While the technical means to prevent the Kwun Lung Lau landslide were readily available, their adoption and implementation were totally contingent on foreseeing that there was a problem. Therefore prevention relies on foreseeability.

9. FORESEEABILITY OF THE KWUN LUNG LAU LANDSLIDE

9.1 Introduction

The documents viewed in this investigation are listed in Appendix A, Volume 2 and a summary review of these documents, as they relate to the investigation of the landslide, is contained in Appendix C, Volume 2. Various aspects of the Kwun Lung Lau landslide were foreseeable, in theory, at different stages of the evolution of the site. Therefore in assessing whether or not the Kwun Lung Lau landslide was foreseeable or not, for purposes of discussion it is helpful to separate the involvement of the various technical agencies that might have foreseen some aspects of the problem.

9.2 Development Stage, 1964-1968

No evidence has been found that the Authorised Person undertook any stability analysis of the slope and retaining wall either before the proposed development or after it was modified by site formation. This is not surprising since geotechnical practice with respect to housing estate development in Hong Kong at that time was fairly rudimentary. Earthwork design was limited and quality control, particularly with regard to fill, was poor. Therefore, there is no reason to anticipate any activities that might have evaluated subsequent slope instability during the site development stage. However, actions on the part of the Authorised

Person made matters worse.

The site formation plans submitted by the Authorized Person and approved by the Building Authority show the retaining wall as a stepped masonry wall with a base width of about 4 m. This was totally without justification and it ultimately proved to be a factor inhibiting future stability assessments from being valid.

9.3 Geotechnical Engineering Office (1978-1987)

The GEO (formerly Geotechnical Control Office - GCO) was established in 1977 and it initiated its slope management system by conducting an inventory of fill slopes, cut slopes and retaining walls. The masonry wall was registered in the Catalogue of Slopes in 1978 and recommendations for additional study were made, although the priority was not high.

The next level of slope management in Hong Kong involves a Stage 1 Study which is aimed at identifying features which pose the greatest risk to life and determining the need for further study. A Stage 1 Study is a preliminary stability assessment consisting of a detailed field inspection, a desk study, and a geotechnical appraisal based on the available information. New field work involving borings is not undertaken in this screening study.

The Stage 1 Study of the masonry wall was completed by the GEO in August, 1987. It concluded that "the slope and the wall are generally in good condition except trees are growing in the top part of the wall" and it advised, "It is therefore recommended that no further study is required under the present circumstances".

The results of the preliminary assessment of stability were much influenced by the assumed width of the retaining wall. To his credit, the GEO engineer decided to treat the wall in a more conservative manner than shown on the approved drawings. He assumed that the back of the wall was vertical which projected to a base width of 2.4 m. This is less than the 4 m shown on the drawings. All other parameters used in his analyses were reasonable for their intended purpose and the wall was shown to have adequate safety.

Had the analysis been based on the actual width of the wall, about 750 mm, the wall would likely have been found to be unsafe and the possibility of future instability may have been foreseen. This was thwarted by the incorrect representation of the retaining wall in the approved drawings.

9.4 HKHS and Their Consultants

Over the years the HKHS has relied on a number of Consultants for advice regarding slope stability and maintenance. For simplicity this can be separated into the Fugro phase (1982-1989) and the Mott Connell phase (1990-1994).

Throughout the Fugro phase a number of slides occurred and several observations were made regarding the poor state of the drainage system at Kwun Lung Lau. The approach to slope stability assessment remained dominated by the spatial separation of features imposed by the slope catalogue and the provision of advice for slope maintenance based on visual inspection. This appears to reflect much practice in Hong Kong and it was responsive to the requests of the HKHS. Notwithstanding a number of indications that serious problems existed with regard to the buried services at Kwun Lung Lau and the suspicion that should have been generated by the number of past failures, it would not be possible to foresee the eventuality of a catastrophic slide without a more insightful approach.

Annual slope inspection services were continued during the Mott Connell phase. This included inspection of the retaining wall and the slope above it. Recommendations with regard to matters of maintenance were made. In January, 1993 Mott Connell reported on a Stage 1 general review of the slopes at Kwun Lung Lau based on the following scope :

(a) to search for and to review slope study reports and site investigation reports available from Geotechnical Engineering Office (GEO),

(b) to carry out detailed inspection on the conditions of the slopes immediately outside lot boundary and to report to Hong Kong Housing Society whether instability in such slopes may affect the housing estate,

(c) to carry out a preliminary geotechnical study based on available information and findings of site inspection and to identify the need for supplementary site investigation, and

(d) to alert the attention of Hong Kong Housing Society if adverse conditions are in existence that may affect the stability of the existing building structure within or close to the areas of this study.

These are broad terms of reference and they provided another opportunity to foresee the problems that arose at Kwun Lung Lau.

There is an unresolved dispute between the HKHS and Mott Connell whether this assignment was intended to cover retaining walls or whether they were excluded from it. The report submitted in response to these terms of reference concluded that except for some recommended surface maintenance items, the slopes at Kwun Lung Lau were in fair condition and that it was not necessary to carry out further detailed stability studies. The Writer doubts that if the retaining walls were specifically being considered by the Consultant the conclusions would have been materially different. Just as in the Fugro phase, the approach to slope stability evaluation was excessively influenced by the separation of slopes in the catalogue and the emphasis on surface slope maintenance evaluation. Experience gained at one slope is not used effectively to assess conditions at others. It appears difficult for practice in Hong Kong to develop an alternate way of evaluating slope hazards based on a more integrated perspective.

9.5 Conclusions

(a) The provision of false information on the site formation plan misled subsequent studies undertaken by the GEO to assess stability of the retaining wall and the slope above it.

(b) Practice in Hong Kong with respect to evaluation of slope stability is excessively influenced in a restricted manner by the slope catalogue and is not sufficiently responsive to indications of potential problems on a project or development scale.

10. LESSONS TO BE LEARNT FROM THE KWUN LUNG LAU LANDSLIDE

10.1 Risk Levels

The very considerable concern over the Kwun Lung Lau landslide expressed by both the public and the government indicates to the Writer that Hong Kong is becoming increasingly risk averse. It is important for any community to have some sense of its goals with regard to risk management. One cannot simply say that the goal is to reduce risk to zero. This is both unrealistic and undesirable.

Risk is defined as the combination of the probability of occurrence of an undesired event (such as a landslide) and the possible extent of that event's consequence (such as fatalities, injuries and property damage). Thus defined, risk can be calculated in principle. Although there is an extensive literature concerned with the calculation of risk, it should be noted that this quantitative methodology has limitations when applied to relatively rare events involving geotechnical processes such as landslides.

There are upper and lower limits between which risks and benefits need to be balanced. The upper one, the maximum acceptable level, should not be exceeded, irrespective of the economic or societal benefit that could result from the activity under consideration. The lower one, the negligible level, indicates the level below which it is not sensible to try to further reduce the risk, in view of the fact that man and the environment are already subject to other risks resulting from nature or society.

Between these two levels is a band in which risk reduction is desirable but needs to be reduced by the best practicable means. This is the ALARA (As Low As Reasonably Achievable) range.

Without any attempts at quantification, it is clear that the Kwun Lung Lau episode falls within the ALARA range. One probabilistic factor involved in the landslide was rainfall. While high, the return period was not beyond the bounds that should be safely accommodated to-day in Hong Kong. In any case the rainfall cannot be controlled but other actions can be taken to reduce risk.

As noted, community reaction indicates that the Kwun Lung Lau landslide falls within the ALARA range. There is a desire to reduce the risk of such events and efforts are needed to find practical means for doing so.

10.2 Replication of the Kwun Lung Lau Landslide

Several factors had to combine for the Kwun Lung Lau landslide to occur. The masonry wall had to escape the Stage 1 screening level study imposed by the GEO. Defective services had to exist with specific water pathways directed to or already within the potential slide mass. A statistically uncommon rainfall storm had to take place over the potential landslide and it had to be retained for a while by an exceptionally slender masonry wall.

It is clear that the prospect for replication of these circumstances is remote and therefore it is unlikely that there is much risk elsewhere in Hong Kong for an event directly comparable to the Kwun Lung Lau landslide.

This does not imply any grounds for complacency. The concept of screening implies that some potentially hazardous features may escape undetected. The GEO has encountered other slope failures in the past that were not caught by the screening exercise and is currently updating its slope catalogue in response to that experience. With respect to slope stability, the price of living in Hong Kong, with its topography, geology, climate and population density, is eternal vigilance . Slope management techniques in Hong Kong must continue to benefit from experience and improve based on that experience. For example, this investigation has revealed that estimates of retaining wall thickness based on old drawings cannot be trusted.

10.3 Inspection of Subsurface Drainage Systems

The presence of leaking buried drains was a key factor contributing to the Kwun Lung Lau landslide. Slope management practice in Hong Kong pays considerable attention to potential surface infiltration into slopes but only limited attention to subsurface infiltration. There is no requirement for systematic monitoring of subsurface drains. This is a particularly serious issue in the older housing estates where poorly compacted fill was often used in grade development. Measures are required to improve inspection and maintenance of subsurface drains where it is known that leakage could impact on slope stability.

10.4 Methodology for Evaluating Slope Stability

It is appropriate to develop a Catalogue of Slopes as an inventory for a slope management system. It is also appropriate to utilize this catalogue in the subsequent screening exercise, the Stage 1 Study. However, the Writer has a concern that for slopes that do not pass to Stage 2 evaluation which usually involves site specific investigation, the technical perspective becomes frozen by the catalogue which separates slopes into numerous distinct elements and by the level of technical investigation that had been adopted for screening purposes.

In the context of Kwun Lung Lau, there were a number of clues prior to 1994 that loose fill existed and that there were defects in the buried services. The response of consultants to the requests from the HKHS were conditioned to think in term of slopes as separate and distinct units. An earlier more integrated perspective might have averted the problem that arose and an effort is needed to impose such an integrated perspective into slope stability assessment in Hong Kong.

11. RECOMMENDATIONS

The Writer recommends the following :

(a) The GEO should implement a program of measuring masonry wall thickness in all cases where Stage 1 Studies have relied on the estimate of wall thickness from old drawings.

(b) The Government should develop a program for direct monitoring and repair of buried services at housing estates and other developments in all cases where leakage might impact on slope stability. Priority should be given to older estates where loose fill is known to have been used in site development. Periodic inspection of buried services at hazardous locations should become mandatory. The appropriate period is best established by experience, but a five year interval appears reasonable at this stage.

(c) The GEO should introduce a more integrated approach into the slope stability assessment process. A means of doing this is as follows :

  (i) All landslide occurrences are to be reported to the GEO as an incident report.

  (ii) Based on the incident report, a senior geotechnical engineer confirms that subsequent slope evaluation can either continue in a separated manner as guided by the slope catalogue or a more integrated approach is called for.

  (iii) A more integrated approach will normally be based on the scale of a project or development. It will require identification of what is known and what is assumed; it will require identification whether site specific soil properties and special geological features must be determined; and it will require an accounting of all water flow pathways that might affect the site.

(d) The GEO should undertake and support elsewhere in Hong Kong research into improved means of site characterization focussed on the factors that affect slope instability in Hong Kong. The Writer does not think that the development of slope warning systems for the conditions found in Hong Kong is promising. The critical features are small and numerous and instability often develops in an abrupt manner. However, there are a number of new developments in geophysics such as radar and non-contact resistivity that might be found useful in discovering subsurface defects and enhanced moisture zones.

(e) GEO should consider appointing an external Technical Review Board. It is the experience of the Writer with other geotechnically intensive organizations, that an external review board can be of considerable assistance to management in enhancing technical quality improvement, maintaining knowledge of developments elsewhere, keeping abreast of international standards of risk-taking, as well as other aspects associated with the discharge of due diligence.

VOLUME 2: FINDINGS OF THE LANDSLIDE INVESTIGATION

Geotechnical Engineering Office
Civil Engineering Department
Hong Kong Government

This report was originally produced in November 1994
as Report on the Kwun Lung Lau Landslide of 23 July 1994

CONTENTS

1. INTRODUCTION

At about 8:53 p.m. on 23 July 1994, a landslide occurred below Block D at Kwun Lung Lau, Kennedy Town (Figure 1). A footpath was buried by the landslide debris, resulting in five fatalities and injuries to three people.

Immediately after the fatal landslide, the Geotechnical Engineering Office (GEO) of the Civil Engineering Department commenced a detailed investigation into the incident. A Progress Report, which summarised the preliminary findings of the investigation, was issued on 9 August 1994 (Geotechnical Engineering Office, 1994). This Volume 2 of the Final Report documents the full findings of the investigation, on which the diagnostic report (Volume 1) by Professor N.R. Morgenstern is based.

The investigation reported in this volume was carried out during the period July to November 1994, and comprised :

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

(b) analysis of the rainfall records,

(c) interviews with witnesses to the landslide and with other residents of Kwun Lung Lau,

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

(e) execution of a comprehensive programme of physical site investigation by drilling, insitu testing and laboratory testing,

(f) thorough examination of the stormwater and foulwater drainage systems at Kwun Lung Lau, and

(g) engineering analyses of the stability of the masonry wall that failed and of the subsurface water flow in the vicinity of the wall.

Given below is a brief description of the landslide and the findings of the comprehensive investigation into the failure. Full details of the investigation work undertaken and the results obtained are contained in the appendices in this volume of the report.

The full list of documents viewed during the landslide investigation is given in Appendix A. The detailed findings of the investigation are contained in Appendices B to L. Appendix M comprises photographs of the masonry wall taken before and after the landslide, as well as photographs taken during the post-failure investigation.

2. DESCRIPTION OF THE LANDSLIDE

The location of the landslide at Kwun Lung Lau is shown in Figure 2. The ground that failed comprised a portion of the masonry wall and slope below Block D, as shown in Figure 3. The wall and slope are within the lot boundary of Kwun Lung Lau, which is owned by the Hong Kong Housing Society (HKHS). In front of the masonry wall is a 7 m to 10 m wide footpath that provides the main pedestrian access to Kwun Lung Lau. Next to the footpath is a playground.

The landslide occurred following a period of heavy rain. The full height of the masonry wall above the footpath level collapsed, together with the slope above the wall, resulting in a scar that measured 28 m wide and 14 m high. The average depth of failure was about 3 m, with a maximum depth of 6 m. About 1000 m^3 of debris were released in the landslide, comprising mainly soil, fragments of chunam, blocks of masonry and trees. The soil in the debris was a sandy silt to silty fine sand, which was wet and loose to very loose.

As shown in Figure 3, the majority of the landslide debris came to rest on the footpath, with some being deposited on the playground. The maximum travel distance of the debris amounted to about 13 m, which was about the same as the height of the ground that failed. The maximum depth of the debris was about 6 m. The maximum width of the debris mound was about 38 m.

A large portion of the upper part of the failed masonry wall was found on the surface of the debris, without major disintegration or significant relative displacement of the masonry blocks (Figure 1). The remains of the wall were displaced well forward, with the front surface of the upper part of the wall facing upward.

A broken 300 mm diameter underground foulwater sewer was exposed at the crest of the landslide scar (Figures 1 & 3). A large volume of water was discharging from this onto the failure scar after the landslide.

A cross-section through the landslide site, showing the ground profiles before and after the failure, is given in Figure 4. The ground profile before the landslide was established from survey plans of the site prepared in 1984 and 1986 by an HKHS consultant. The post-failure ground profile was determined by a topographic survey.

3. HISTORY OF THE SITE

It is known from old topographic maps (Appendix B) that the masonry wall was constructed some time before 1901. A platform above the wall can be seen in the earliest available aerial photograph, taken in 1924. Just behind and upslope of this platform, a small cut slope had been excavated in the natural slope at the location of the present landslide. The platform and the crest area of the small cut slope were occupied by squatters between 1945 and 1963.

The history of the development of the landslide site is summarised in Appendix C.

Kwun Lung Lau was constructed between 1965 and 1968. According to observations made from the available aerial photographs, fill was placed onto the platform above the masonry wall some time between 1964 and 1969 (Figure 4), and there is no evidence of the ground profile having subsequently been modified further. In the site formation plans for Kwun Lung Lau approved by the Buildings Ordinance Office (BOO) (reorganised as Buildings Department in 1993) in April 1965, the wall was shown as a stepped-back masonry wall, marked "EX. RETAINING WALL", with a base width of about 4 m. A brief description of the statutory checking system in respect of the engineering aspects of private building developments is given in Appendix D.

In 1978, the masonry wall was registered as No. 11SW-A/R309 by a consultant engaged by the Geotechnical Control Office (GCO) (renamed Geotechnical Engineering Office in 1991), to prepare a catalogue of cut slopes, fill slopes and retaining walls (now termed 'Catalogue of Slopes') in Hong Kong. The wall was classified as 'high risk' (i.e. the consequence of a failure would be high), in accordance with the risk categorisation system established in 1978 by the GCO consultant.

The masonry wall and the slope above were inspected by a number of professional parties between 1980 and 1994. These included the GCO and two consultants appointed by the HKHS. Detailed inspection of the masonry wall was not possible before March 1987 because of the presence of squatters on the footpath in front of the wall.

The masonry wall was assessed in July 1987 by the GCO in a Stage 1 Study carried out under the Landslip Preventive Measures Programme. This was a preliminary stability assessment to determine whether a detailed stability study was required. It consisted of a detailed field inspection, a desk study and a geotechnical appraisal based on the available information, without any physical site investigation. In the preliminary stability assessment, the width of the wall base was taken to be 2.4 m, an assumption believed to be conservative at the time given the wall dimensions shown in the 1965 approved site formation plans. It was concluded that "No further study is required under the present circumstances". The risk (i.e. consequence) category of the masonry wall was also regraded from 'high' to 'low to moderate' to take account of the clearance of the squatters on the footpath in March 1987, in accordance with the risk categorisation system established in 1978.

In 1992, a consultant was engaged by the HKHS to carry out a "Study of Existing Slopes - Stage 1 General Review", part of the requirements of which was "to alert the attention of Hong Kong Housing Society if adverse conditions are in existence that may affect the stability of the existing building structures within or close to the areas of the study". This study was undertaken in December 1992, and the consultant reported that "we consider that it is not necessary to carry out further detailed stability study for these slopes".

The masonry wall was found to be in fair or good condition during the inspections carried out by the Geotechnical Control Branch (merged with the GCO in 1983) of the BOO in 1980 and by the GCO in 1987, and this continued to be so during the inspection in 1983 and the annual inspections between 1987 and 1994 undertaken by HKHS consultants. No signs of distress of the masonry wall were ever recorded. Recommendations made as a result of the inspections were only for relatively minor maintenance works, such as removal of unplanned vegetation and clearance of weepholes.

In the most recent annual maintenance inspection carried out in June 1994, the HKHS consultant noted subsidence near the crest of the slope above the masonry wall, cracking and dislodgement of the chunam cover of the slope, an unsealed tree ring, an unstable tree at the toe of the slope, and unplanned vegetation on the wall. Leakage from a manhole located to the immediate east of the slope above the masonry wall was also observed. Although the consultant did not record any blocked weepholes in the masonry wall, the weepholes in the chunam cover to the slope above the wall were recorded as being blocked. The range of maintenance works recommended by the HKHS consultant for the masonry wall and the slope above is summarised in Appendix C. The consultant also recommended that "the manhole and associated pipelines should be checked and the leaking points should be made good".

With respect to the implementation of the works recommended in June 1994 by its consultant, the HKHS advised the GEO subsequent to the failure that "As no urgent works were required and as a normal procedure, tender documents were completed on 19 July 1994 by Hong Kong Housing Society for the recommended remedial works to be carried out accordingly".

There is no history of any previous instability at the masonry wall or the slope above. However, in other areas of Kwun Lung Lau, landslides that involved failure volumes ranging from tens of cubic metres to several hundred cubic metres have occurred over the years (Figure 5). The largest previous landslide occurred at the slope to the north of Block B following intense rain in June 1985, at the time of which the site formation works for the development of Smithfield Terrace (Figure 2) were in progress at the location of the failure. A report prepared by the Authorised Person responsible for the works suggested that a leaking sewer near the crest of the slope contributed to the failure. Prior to the 1985 landslide, a report by a GCO consultant engaged to investigate the disused tunnel beneath the slope to the north of Block C (Figure 2) made reference to "rupture of sewer" and "leaking sewer".

After the 1985 landslide, the HKHS consultant carried out a visual inspection of the drainage system in the vicinity of the crest of the adjoining slope to the north of Block C upon the advice of the BOO. It was reported that "neither major cracks on the surface drainage channels nor signs of leakage of subsurface drains can be observed".

4. DESCRIPTION OF THE MASONRY WALL

The masonry wall was a pointed squared-rubble wall with stone ties and weepholes, of the kind once commonly constructed in Hong Kong. The face of the wall was inclined at about 75° to the horizontal (Figure 4) and had a maximum height of 10.6 m. At the failure location, the wall was 700 mm to 800 mm thick, and was constructed against a cutting into natural ground. The base of the wall was below the level of the footpath.

The surface appearance of the wall, as revealed by inspection of the adjacent unfailed portion of the wall, suggests that the workmanship of the construction was generally very good. The actual 700 to 800 mm thickness of the wall, when revealed by the landslide, was far below what could reasonably have been expected on the basis of previous experience of old masonry retaining walls in Hong Kong (Appendix E).

The masonry wall was designated as No. 11SW-A/R309 in the Catalogue of Slopes compiled by the GCO consultant in 1977/78, and it was ranked as No. 758 in the Master Ranking List established by the GCO in 1979 for the purpose of prioritizing Stage 1 Studies of slopes and walls under the Landslip Preventive Measures Programme.

The slope above the wall generally varied between 2.5 m and 6 m in height at an angle of 20° to 50° to the horizontal (Figure 4), and was covered with chunam. Some unplanned vegetation on the slope and a tree at the toe of the slope above the wall are visible in photographs of the site taken in June 1994. The slope was recorded as a "fill slope" on the field sheet prepared for the masonry wall in 1978 as part of the Catalogue of Slopes.

5. OBSERVATIONS MADE PRIOR TO THE LANDSLIDE

The sequence of events prior to the failure on 23 July has been re-constructed from the accounts of witnesses (Appendix F).

In the two days before the landslide, muddy water was observed coming out of weepholes near the toe of the masonry wall.

During the period of about two-and-a-half hours before the landslide, a number of pertinent observations were made (Figure 6). These included the presence of a hole of approximately 1 m in diameter on the chunam surface, and the seepage of muddy water through weepholes and joints between the masonry blocks, the extent of which apparently increased with time. According to observations made about half an hour before the landslide, the area of the masonry wall affected by the seepage of muddy water increased in size to cover approximately the whole of the subsequent failure location.

Shortly before the landslide, the metal railing at the top of the masonry wall was observed to be bent and broken, and pieces of the railing were reported to have fallen onto the footpath.

The landslide was immediately preceded by the dislodgement of some "stones". The masonry wall apparently "burst" at about mid-height and collapsed almost instantaneously.

6. ANALYSIS OF RAINFALL RECORDS

An automatic raingauge (No. H02) is located on the roof between Blocks C & D at Kwun Lung Lau. The daily rainfalls recorded by the raingauge in July 1994, together with the hourly rainfalls from 21 to 23 July 1994, are shown in Figure 7.

The rainfall records have been analysed in detail (Appendix G). In the month of July 1994, a total of 915 mm of rain had been recorded by the raingauge up to the time of the landslide on 23 July, and this amount had already exceeded the highest rainfall ever registered in any July at the Royal Observatory since records began in 1884.

There was very heavy rain on 22 and 23 July 1994. In the 48 hours before the landslide, a total of 547 mm was recorded by the raingauge. An assessment of the return periods of the rainfall intensities of this rainstorm for different durations based on historical rainfall records at the Royal Observatory shows that the 48-hour rainfall was the most severe, with a corresponding return period of about 28 years.

The maximum hourly rainfall was 101 mm, which occurred between 2:05 a.m. and 3:05 a.m. on 22 July 1994, about 42 hours prior to the landslide. Only 29 mm of rain were recorded by the raingauge between 11:00 a.m. on 23 July 1994 and the time of the landslide, i.e. approximately 95% of the 48-hour rainfall fell more than ten hours before the landslide. The Kwun Lung Lau landslide was therefore a 'delayed' failure, in that it occurred several hours after cessation of the intense rain.

The rainfall preceding the landslide was the highest recorded for durations exceeding 24 hours by raingauge No. H02 since its installation in September 1978. For example, the 48-hour rainfall before the landslide exceeded the previous maximum, recorded in the September 1993 rainstorm, by about one third. On the other hand, the one-hour to 12-hour intensities were comparable to the maximum figures experienced in the past, whilst the very short-term (e.g. five-minute) intensity was only about two-thirds of the previous maximum recorded in the May 1992 rainstorm.

7. SUBSURFACE CONDITIONS AT THE SITE

The geology at the landslide location comprised fill overlying partially weathered volcanics (Appendix H). The fill behind and above the wall was loose and generally very permeable. The partially weathered volcanics consisted of completely decomposed and highly decomposed coarse ash tuff.

The interface between the fill and the underlying natural ground is interpreted to have been about 1 m below the crest of the masonry wall. The typical stratigraphy through the landslide site is shown in Figure 4.

During the investigation of the landslide, there was no trace of significant flow or seepage of water from the landslide scar indicative of a high groundwater table or perched water table. The ground investigations carried out as part of the landslide investigation confirmed that the main groundwater table was well below the landslide surface. Localised seepages were observed, however, in the fill stratum during the course of the ground investigations. Also, the results of laboratory water content determinations (Appendix I) indicated that the ground beneath the landslide scar was very wet, with the soils having high degrees of saturation.

The shear strengths of the fill and partially weathered volcanics in the vicinity of the failure were measured by a variety of laboratory test methods (Appendix I). These included shear box tests, conventional triaxial compression tests and special triaxial tests. The results of these tests enabled the best estimates of the strength parameters of the soils shown in Figure 4 to be established. These material strengths are consistent with those measured previously on similar materials in Hong Kong, and no exceptionally weak material was found to be present at the landslide site.

Point-load tests were carried out on samples of the mortar from between the masonry blocks of the wall recovered from the landslide debris (Appendix I). The test results showed that the mortar was of good quality, which was consistent with the field observations, and no reduction in the strength of the mortar was observed after soaking the samples in water for seven days prior to testing.

8. FOULWATER AND STORMWATER DRAINAGE SYSTEMS

A detailed drainage investigation of the Kwun Lung Lau area was carried out jointly by the Buildings Department (BD) and the GEO after the landslide (Appendix J). The layouts of the underground foulwater and stormwater sewers in the vicinity of the landslide are shown in Figure 8 and Figure 9 respectively. These drainage systems were constructed as part of Kwun Lung Lau in the mid- to late-1960s.

The portion of the 300 mm diameter foulwater sewer that was severed by the landslide was assessed to have run across the upper part of the landslide area before the failure (Figure 3), within 2 m of the ground surface. Sections of the severed pipe recovered from the debris showed that the sewer was constructed of about 1 m long earthenware pipes connected by rigid socket-and-spigot joints infilled with cement mortar. Some of the pipes were partly encased in concrete. Such rigidly-jointed earthenware pipes are brittle and are susceptible to breakage as a result of differential settlement.

There were black stains on the soil that adhered to the outside surfaces of the pipe junctions and concrete encasement of the pipe sections recovered from the debris. Chemical analyses of the black-stained soil showed that the material had probably been affected by foulwater, suggesting that leakage of foulwater had been occurring for some time before the landslide.

A careful examination of the Kwun Lung Lau foulwater and stormwater systems in the vicinity of the landslide revealed defects such as local settlement, cracked pipes, and dislocated or open joints. In particular, two sections of the underground stormwater pipes in the yard area to the south of Block D (Figure 9) were found to be suffering from major leakages.

The broken sewer exposed in the landslide scar carried foulwater from Blocks D, E, F & G. The rate of discharge from this sewer was measured over a period of two weeks after the failure, and this was found to range generally from 0.1 m^3/min to 0.7 m^3/min, with the peak rate occurring between 6 p.m. and 10 p.m.

A very thorough investigation of the sewer systems in the vicinity, comprising dye tests, manhole inspections, closed-circuit television surveys and flow monitoring, provided no evidence of any connection of the stormwater system to the foulwater sewer.

Chemical analyses of water samples obtained from the soil beyond the landslide scar downslope of the foulwater sewer showed that the chloride content was high, and was comparable to that of the foulwater. However, soil above the elevation of the foulwater sewer had a low chloride content despite a high moisture content. This evidence suggests that a large amount of foulwater had entered the ground before the landslide, but that there was a source of water other than the foulwater sewer that had also contributed to the wetting of the ground.

9. SUBSURFACE SEEPAGE FLOW

The layout of the pile caps and structural walls of the substructure of Block D is shown in Figure 10. Block D is underlain by two large pile caps founded on partially weathered volcanics, with fill placed between the caps. The fill extends to the landslide site and the yard area to the south of Block D, as shown in Figure 11.

The fill was observed to be loose and to contain voids through which subsurface seepage flow could take place preferentially. Water tests (Appendix K) carried out after the landslide confirmed that the fill was permeable. During the tests, a large amount of water came out within the fill near the crest of the landslide area shortly after water had been introduced into the ground below Block D.

Except for the garden areas, the yard area to the south of Block D is paved. The fill beneath the yard area, within which the underground stormwater pipes are located, was also found to be loose and permeable. Within the unpaved garden areas, the near-surface soil is less permeable, as confirmed by infiltration tests carried out after the landslide.

The above field observations and tests confirmed that any water that might have entered the ground in the yard area could have been directed towards the location where the landslide occurred by subsurface flow through the continuous permeable fill layer.

10. THE MODE OF THE LANDSLIDE

The relatively thin masonry wall, which supported a steep cutting and the slope above, had remained stable for many years without any signs of distress. Engineering analyses (Appendix L) carried out on the basis of the post-failure ground investigation data and laboratory test results have shown that, prior to the placement of the fill behind the wall between 1964 and 1969, the wall would not become unstable even if the retained soil became fully saturated. After placement of the fill, the stability of the wall relied on suction forces that developed in the soil beneath the surface cover, and it would become unstable if the soil became saturated.

The large areal extent of seepage through the wall as observed shortly before the landslide suggests that water had entered the ground behind the wall, so increasing the moisture content and reducing the suction. As the soil strength decreased, the earth pressure acting on the wall would have increased until failure occurred. . A conventional stability analysis of the wall section, on the assumption of a rigid wall, has shown that the wall would be unstable in overturning if the soil behind were saturated. A slope stability analysis has also shown that the factor of safety against overall shear failure through the wall and the retained ground would be less than unity if the ground was substantially saturated and the cohesive strength of the mortar between the masonry blocks was below about 25 kPa.

According to the accounts of witnesses, the masonry wall failed initially by bursting at about its mid-height. This failure mode had also been observed in some previous failures of old masonry walls in Hong Kong (Chan, 1982, 1994; Shelton, 1990).

The numerical analysis involving assessment of deformation used as part of this investigation has confirmed that the failure is likely to have involved a complex mode. The analysis predicts a bulging of the wall at about mid-height, accompanied by overturning of the portion of the wall below mid-height. These modes combine to cause a tensile failure of the mortar between the masonry blocks, with an accompanying reduction in the shear strength of the mortar joints. The bulging and overturning modes continue, resulting in brittle fracture of the wall at about mid-height, followed by the forward sliding of the failed ground mass.

The numerical analysis further predicts that the upper portion of the masonry wall is displaced by the sliding mass and comes to rest on the surface of the debris, with the front face of the wall facing upwards. The lower portion of the wall is predicted to be buried by the debris.

The above deduced failure mode is consistent with the field observations, including the disposition of the masonry wall after the landslide (Section 2).

11. DIAGNOSIS OF THE LANDSLIDE

The stability analyses have confirmed that the wall would have become unstable if the ground became substantially saturated. Calculations suggest that the amount of water required to saturate the failed mass would have been considerable, being in the order of 100 m^3.

The four possible causes of water ingress, either separately or in combination, that have been identified are :

(a) a rising groundwater table,

(b) direct surface infiltration,

(c) subsurface seepage flow, and

(d) leakage from the stormwater and/or foulwater systems.

The main groundwater table was found to be well below the failure surface, and there is no evidence to support a hypothesis that the landslide was caused by a significant rise of the groundwater table.

Although cracks in the chunam cover and an unsealed tree ring were observed about one month before the landslide, the extensive chunam cover would have prevented significant direct surface infiltration at the landslide location. Inspections by staff of the HKHS indicated that there had been no drastic deterioration in the condition of the surface cover and drainage channels during the period between the inspection made by the HKHS consultant in June 1994 and the day of the landslide. It is therefore considered most unlikely that direct surface infiltration at the landslide location was a primary source of the water ingress.

There is corroborative evidence to suggest that subsurface seepage flow through the permeable fill layer, at an elevation higher than that at the landslide location, had taken place before the landslide, with the water originating from the ground to the south of Block D

(Sections 8 & 9). The source of the water was either from the defective stormwater drains in the yard area, from direct surface infiltration through the unpaved garden areas in the yard or through the slopes further uphill to the south.

It is considered that leakage from the defective stormwater drains beneath the yard area is likely to have been the principal source of subsurface seepage flow towards the landslide location.

Seepage analysis suggests that surface infiltration through the unpaved areas in the yard was likely to have been of secondary importance relative to the potential leakage from the defective stormwater drains, and at best it might have constituted only a contributory source of water into the ground.

No signs of seepage have been noted from the slopes to the south of the yard area. On the basis of this observation and the topography of the area in relation to the landslide location, it is considered unlikely that direct surface infiltration through the slopes further to the south would have given rise to significant recharge of the groundwater at the landslide location.

The observed signs of prolonged leakage from the foulwater sewer (Section 8) suggest that this must have contributed to the water ingress into the area where the landslide occurred. Measurements of the flow rate in the foulwater sewer after the landslide indicate that the sewer would have provided a sufficient quantity of water to substantially saturate the failure mass if significant leakage had been initiated some hours before the landslide.

Other conceivable factors that could have contributed to triggering the landslide are :

(a) deterioration in the structural condition of the masonry wall,

(b) application of surcharge loading at the back of the wall or excavation works in front of the wall shortly before the landslide, and

(c) removal of trees from the slope above the masonry wall.

The available evidence suggests that these three factors can be discounted.

Up to 3 m of fill was placed on the ground surface behind the masonry wall between 1964 and 1969. This would have reduced the margin of stability of the wall, but it could not have been a direct trigger of the landslide because the filling was undertaken some 25 years ago.

12. LIKELY MECHANISM OF THE LANDSLIDE

The landslide was most likely to have been caused by the ingress of a large volume of water as a result of leakage from defective underground services in the vicinity of the slope. The water would have led to a loss of soil suction with accompanying reduction in soil strength, with the consequent increase in earth pressure causing brittle failure of the unusually thin masonry wall. Direct surface infiltration through the unpaved areas in the yard to the south of Block D and through minor defects on the surface cover to the slope above the masonry wall could have been a contributory factor, but not the major cause.

Two sections of the underground stormwater pipes in the yard area were found to have suffered from major leakages, and there is evidence to suggest that the foulwater sewer had been leaking for some time before the landslide. However, it is not easy to establish the relative contributions to the failure of leakage from the foulwater sewer and the stormwater pipes.

The landslide could have been triggered by substantial saturation of the ground under one of the three following scenarios :

I. Subsurface seepage flow through the fill layer underneath Block D promoted downward infiltration into the ground behind the masonry wall, with minor leakage of the foulwater sewer being a secondary factor.

II. Major leakage from the foulwater sewer resulted from gradual deterioration of the sewer, with subsurface seepage flow being only a contributory factor.

III. Major leakage from the foulwater sewer resulted from deformation of the fill material caused initially by subsurface seepage flow.

Scenario I implies that the foulwater sewer did not play a significant role in triggering the failure. However, the results of the post-failure chemical analyses indicate that foulwater had spread to the ground beyond the landslide scar. This is evidence to suggest that a large volume of sewage must have been discharged into the ground within the vicinity of the failure some time prior to the landslide. It is therefore considered that Scenario I is highly unlikely.

Scenario II implies that the landslide was caused primarily by a major leakage of foulwater shortly before the landslide following continued deterioration of the sewer that eventually resulted in significant defects. This would mean that the collapse was unrelated to the rainfall pattern, and the timing of the failure after a period of heavy rain was coincidental. Although this possibility cannot be totally discounted, the availability of a credible alternative explanation of the probable course of events (see Scenario III) suggests that, on the balance of probabilities, pure coincidence alone is less likely to have prevailed under the circumstances of this landslide.

Scenario III implies that both leakage of the foulwater sewer and subsurface seepage flow were together responsible for substantially saturating the ground mass. If this were so, the loose fill behind the wall would have been wetted by subsurface seepage arising primarily from the leakage of defective stormwater drains during times of prolonged and high intensity rainfall, perhaps supplemented by some minor and localised leakage from the foulwater sewer. This would have caused deformation in the loose fill, which would have led to distress or rupture of the rigidly-jointed sewer, resulting in major leakage of foulwater. This, together with subsurface seepage flow, would have provided a sufficient quantity of water to saturate the soil behind the wall. Seepage analyses have shown that it was possible for the subsurface seepage flow to reach the fill in the slope above the wall in about one day after heavy rain, and that it would take another day for the water (both stormwater and foulwater) in the fill to substantially saturate the partially weathered volcanics. This is considered to be the most likely scenario leading up to the landslide. The probable mechanism of the landslide is illustrated in Figure 12.

13. CONCLUSIONS

It is concluded that the sudden failure of the old masonry wall at Kwun Lung Lau was triggered by the substantial saturation of the soil behind the wall, which was appreciably thinner than was usual for walls of this type. Leakages from defective underground water-carrying services (both the stormwater drains and the foulwater sewer) were the principal sources of the water.

The prolonged and heavy rainfall preceding the landslide played a contributory part in resulting in a significant quantity of water having entered the ground, initially through the defective stormwater drainage system.

14. REFERENCES

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Chan, Y.C. (1994). Study of Old Masonry Retaining Walls in Hong Kong. Geotechnical Engineering Office, Hong Kong. (GEO Report No. 31). (Under preparation).

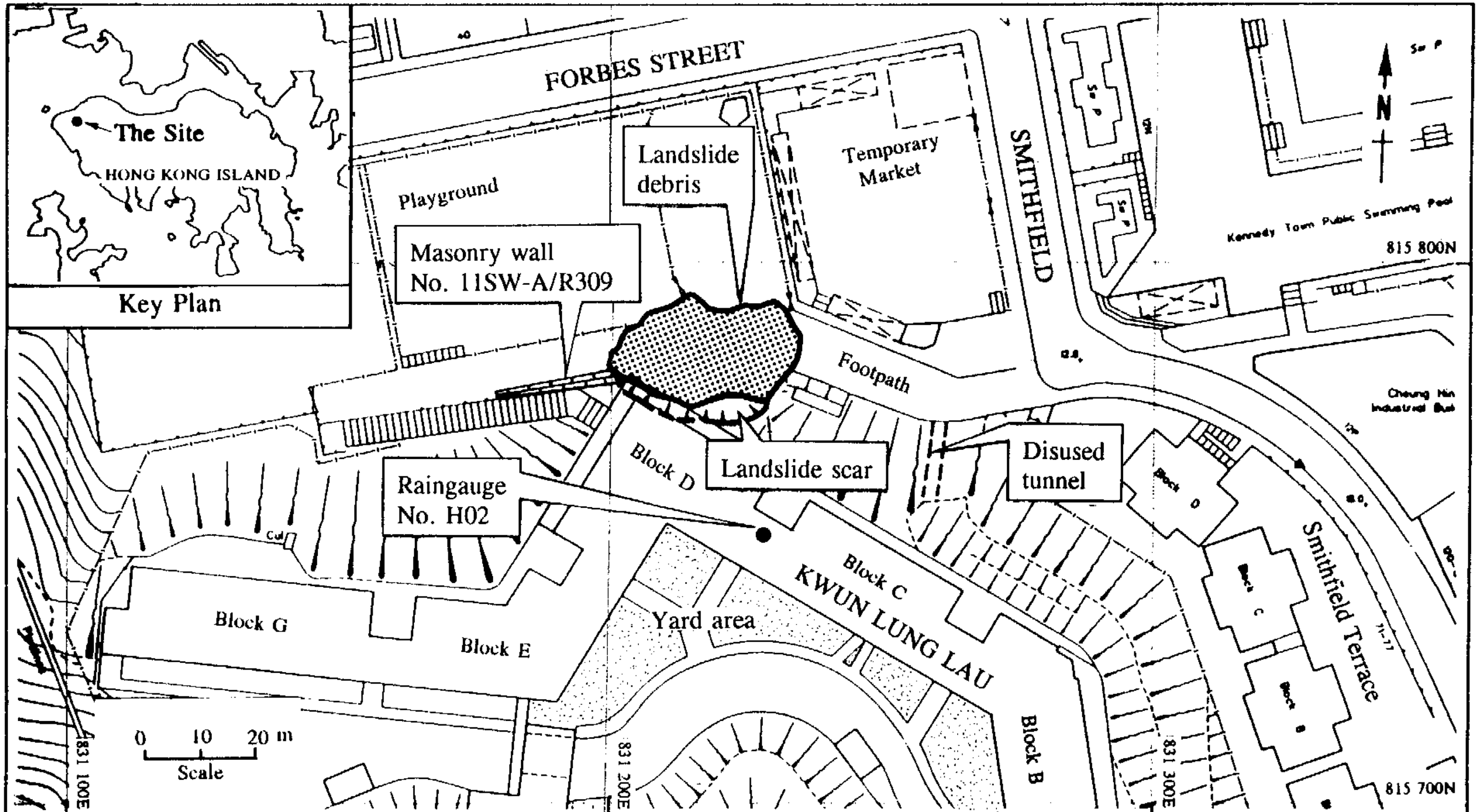
Geotechnical Control Office (1984). Geotechnical Manual for Slopes. (Second edition). Geotechnical Control Office, Hong Kong, 295 p.

Geotechnical Engineering Office (1994). Progress Report on the Investigation of the Kwun Lung Lau Landslide of 23 July 1994. Geotechnical Engineering Office, Hong Kong, 16 p.

Shelton, J.C. (1990). A masonry retaining wall failure at Square Street, Hong Kong. Proceedings of the HKIE Seminar on Failures in Geotechnical Engineering, Hong Kong, pp 37-60.

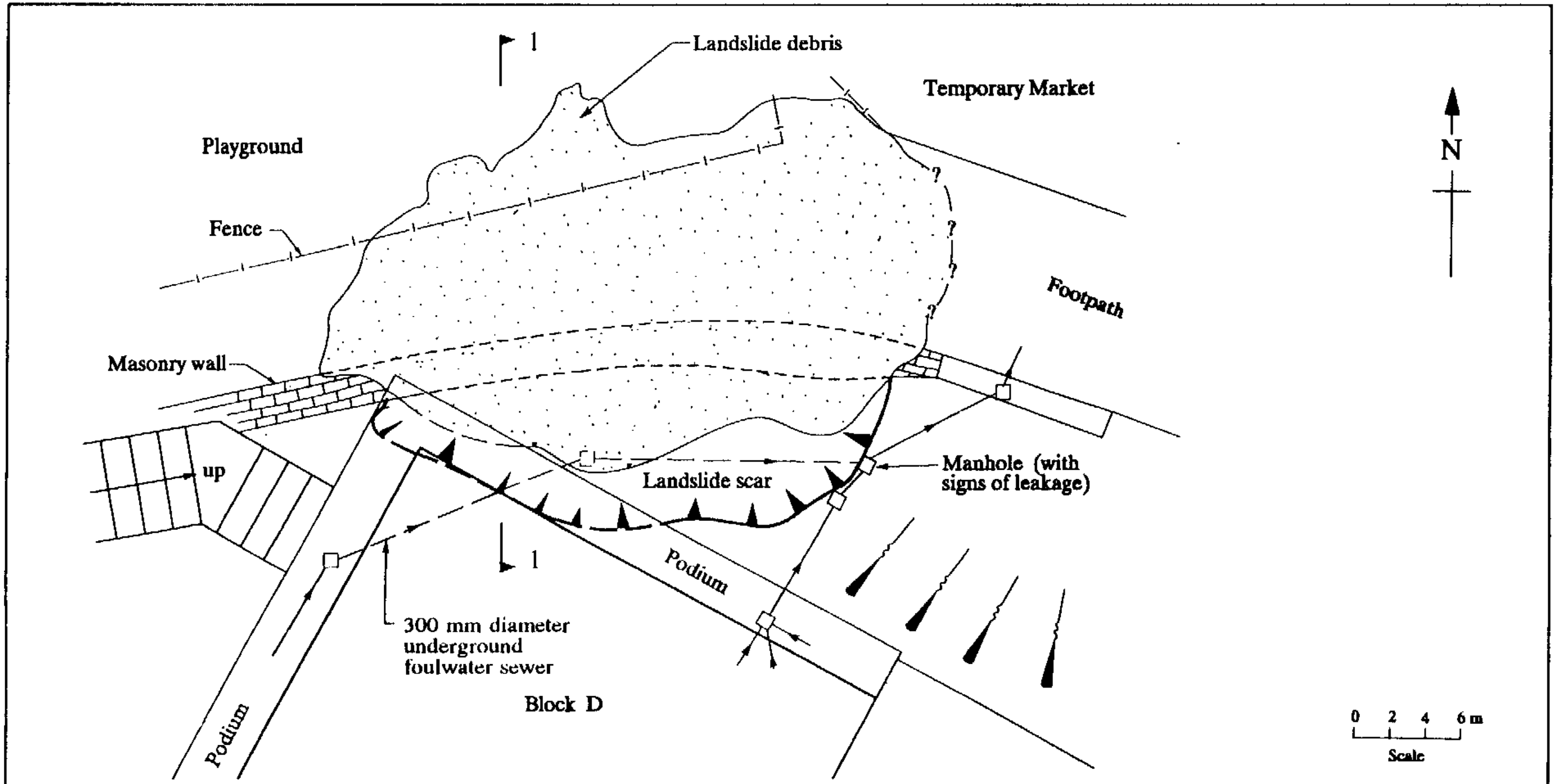
LIST OF FIGURES

Figure 1 - Photograph of the Landslide Taken on 24 July 1994



Note : Base map is extracted from Survey Sheet No. 11-SW-6D (scale 1:1 000) dated 29 September 1992.

Figure 2 - Site Location Plan

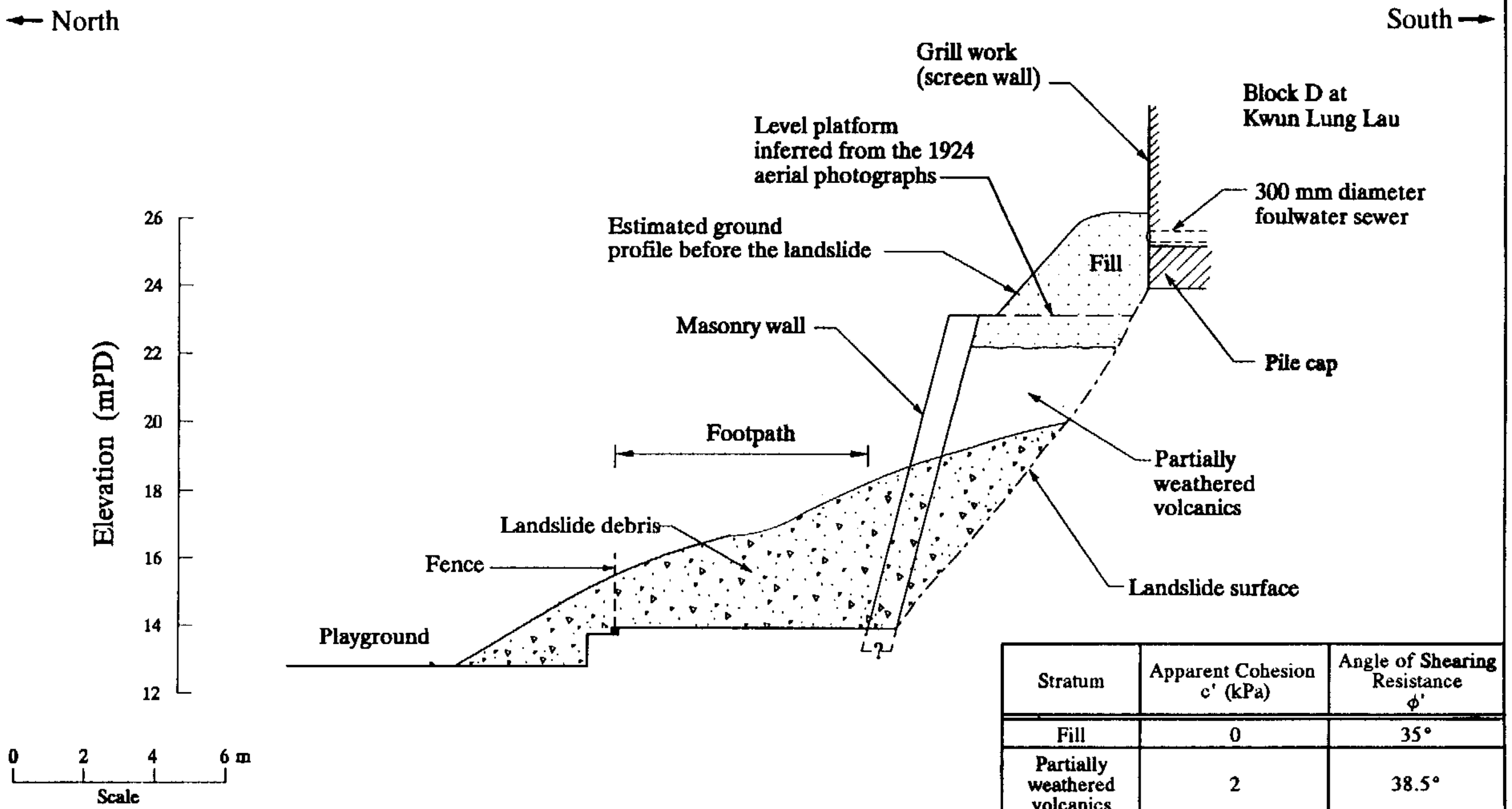


Legend :

Landslide scar   □ Manhole   — ➝ — Alignment of the 300 mm diameter underground foulwater sewer reconstructed from field observations and photographs

Notes : (1) See Figure 4 for Section 1 - 1.
(2) Information shown in this Figure is based on topographic surveys, engineering geological mapping and field observations.

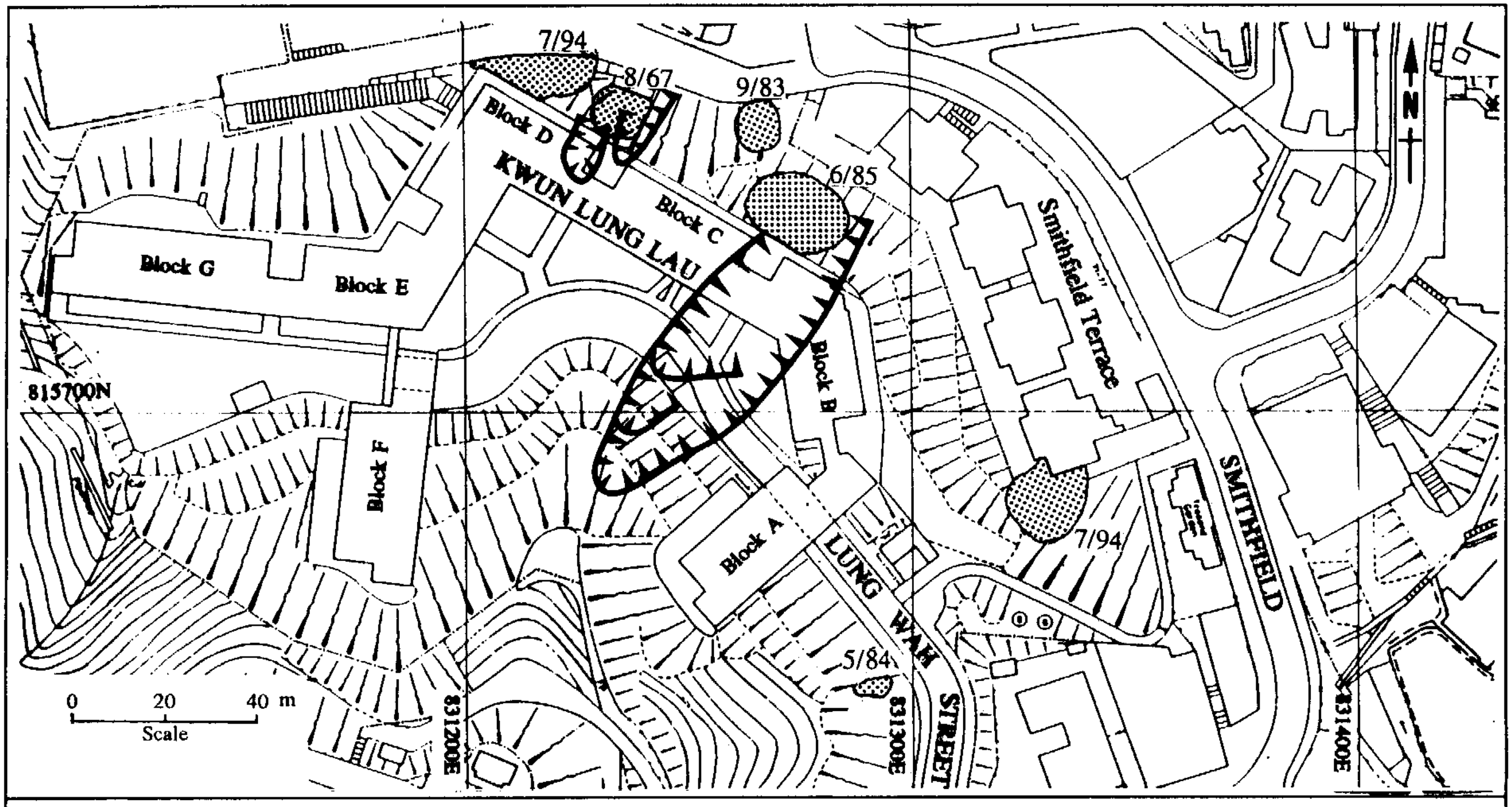
Figure 3 - Plan of the Landslide



Stratum	Apparent Cohesion c' (kPa)	Angle of Shearing Resistance ϕ'
Fill	0	35°
Partially weathered volcanics	2	38.5°

Notes : (1) See Figure 3 for location of section.
(2) Podium of Block D is not shown for clarity.

Figure 4 - Section 1-1 through the Landslide



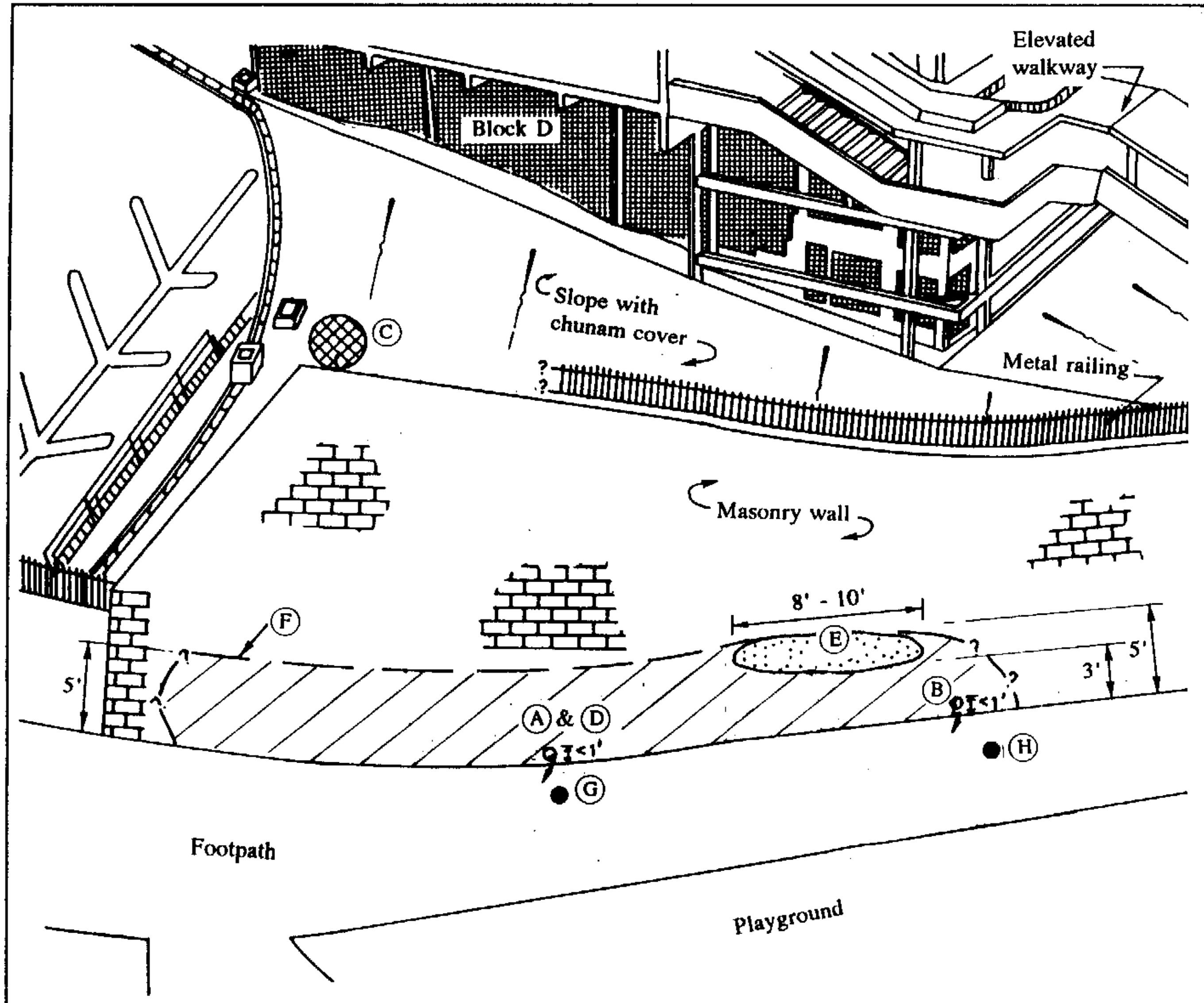
Legend :

6/85 Location of recorded landslide dated June 1985

Landslide scars observed in 1945 aerial photographs

Note : Base map is extracted from Survey Sheet No. 11-SW-6D (scale 1:1000) dated 29 September 1992.

Figure 5 - Locations of Previous Landslides at Kwun Lung Lau



Observation	Time of Observation before the Landslide
(A) - Muddy water seeping from a weephole	2 days
(B) - Muddy water seeping from a weephole	2 days
(C) - 1 m diameter hole in the slope	2½ hours
(D) - Muddy water seeping from a weephole	2 hours
(E) - Muddy water seeping through joints of masonry blocks	1½ hours
(F) - Muddy water seeping from weepholes	½ hours
(G) - Fallen strips of metal railing and pieces of concrete	10 minutes
(H) - Fallen strips of metal railing and pieces of 'stone'	5 minutes

Figure 6 - Summary of Key Observations Reported by Witnesses

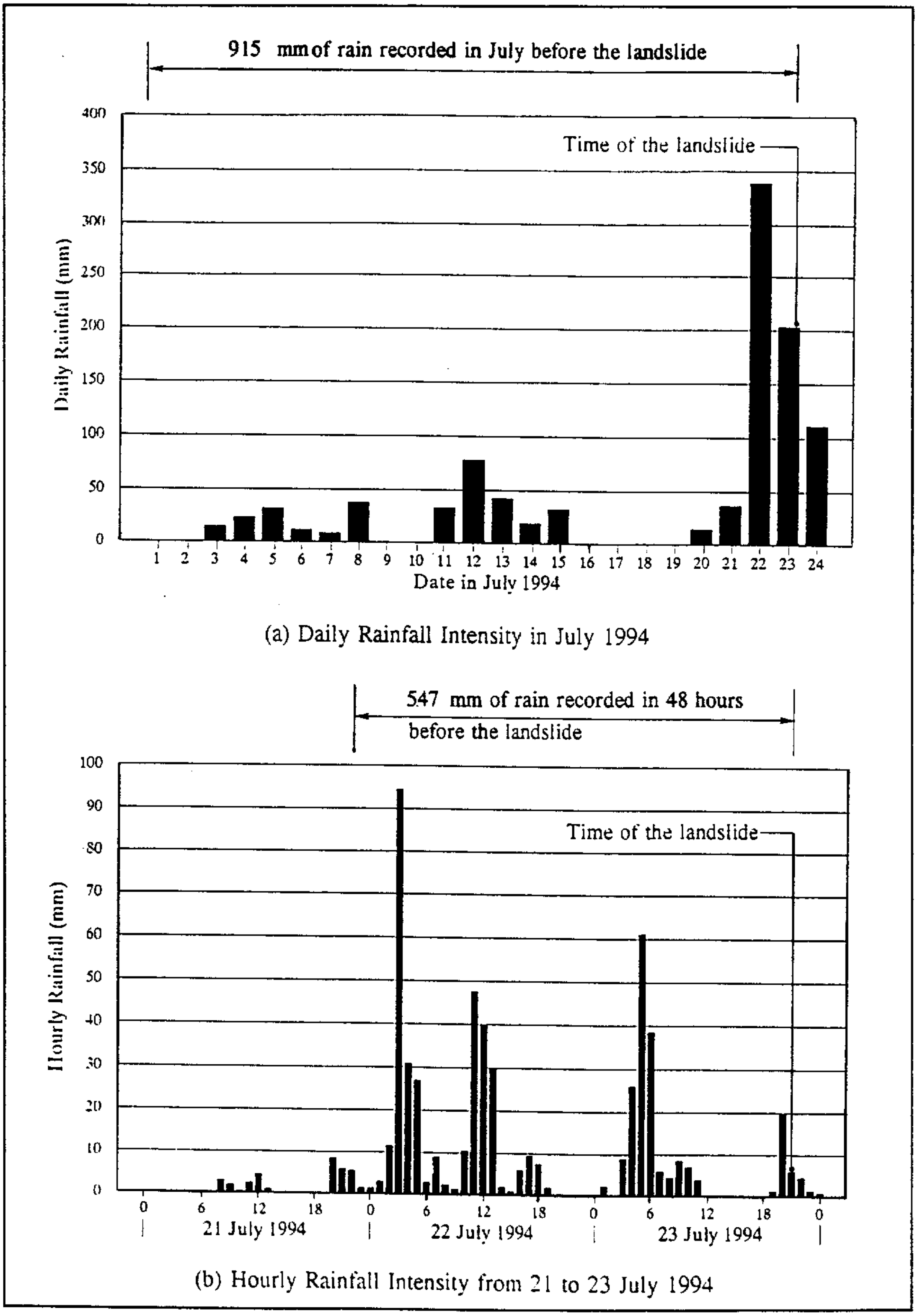
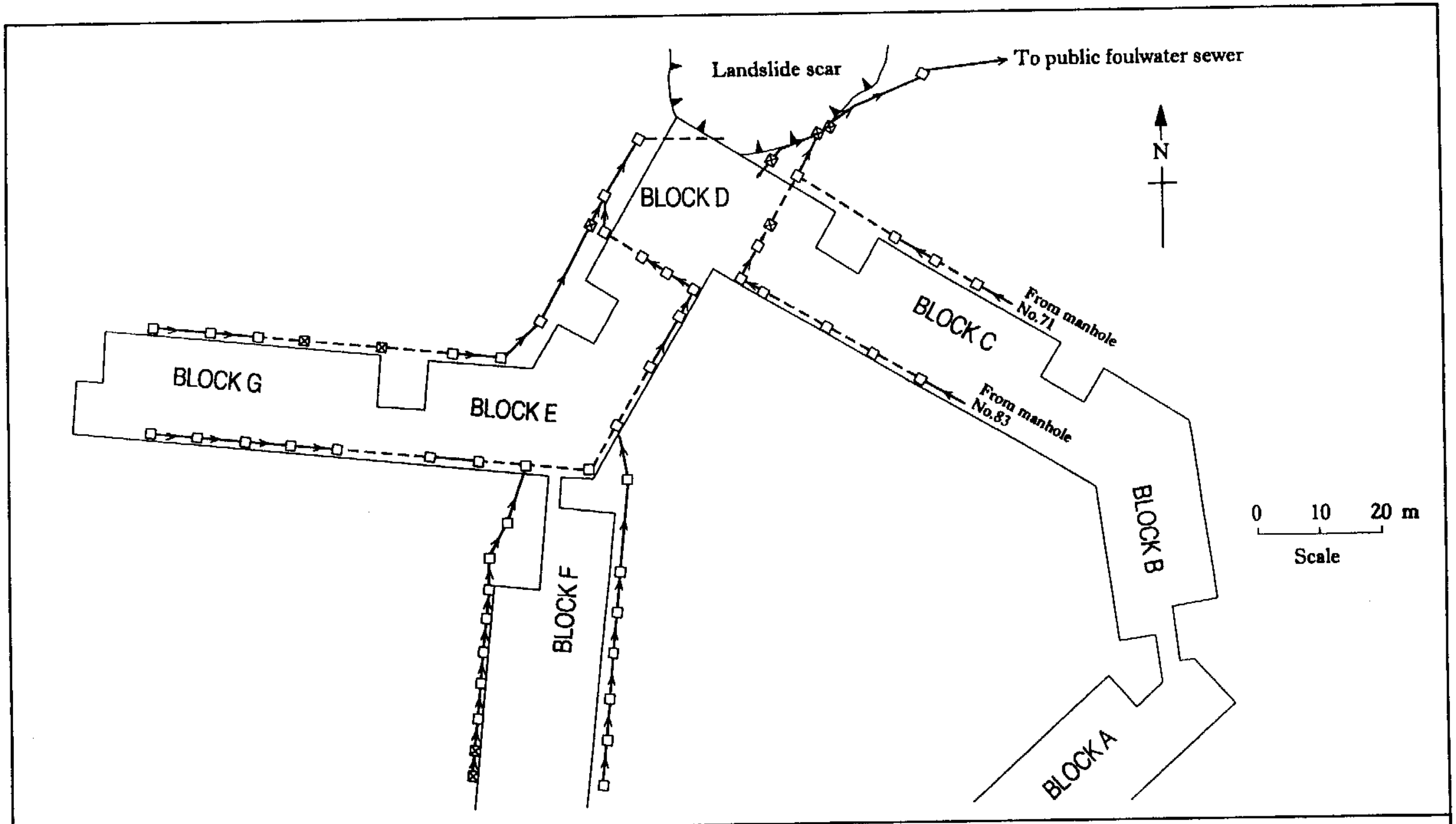


Figure 7 - Rainfall Records of Raingauge No. H02



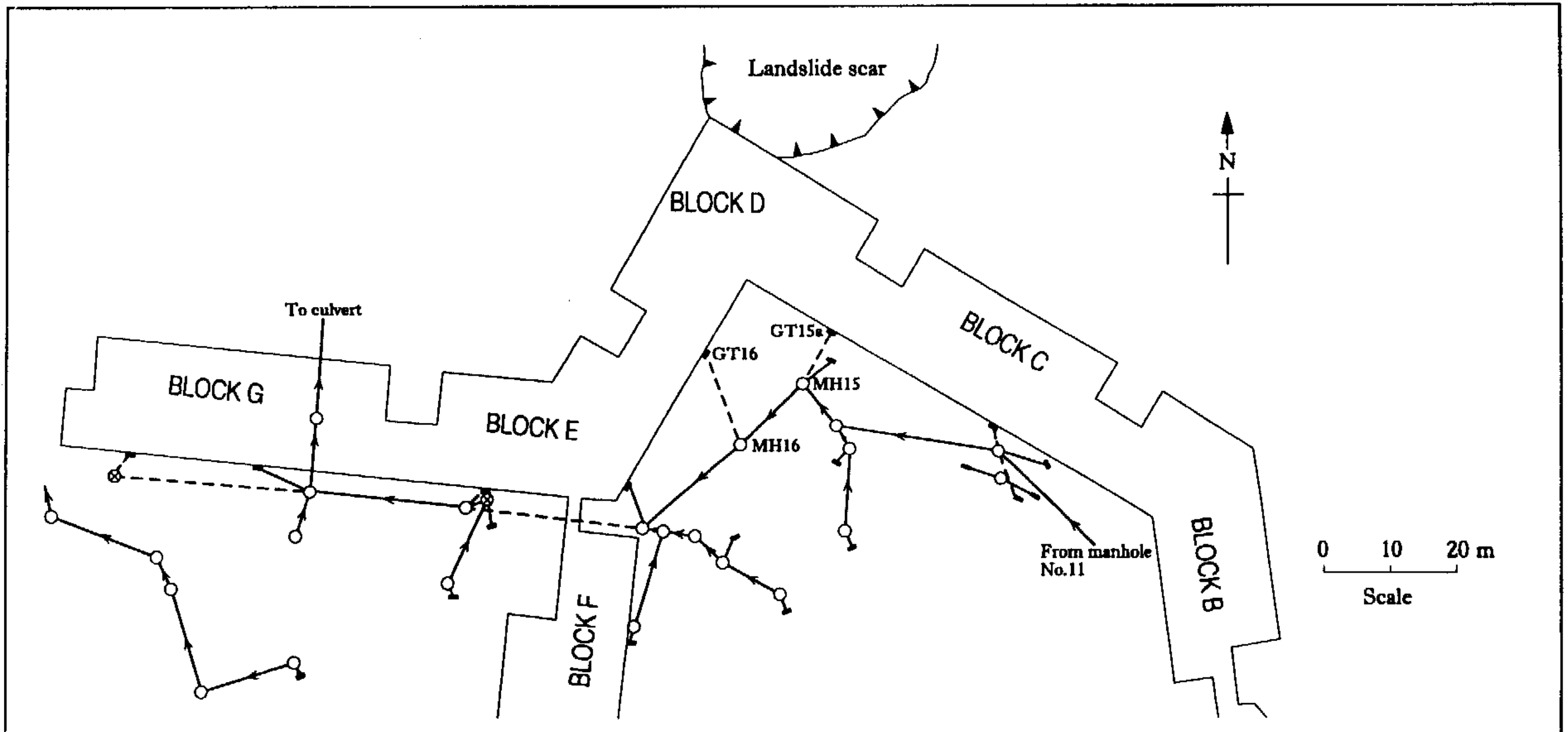
Legend :

– – – – Pipe cracked, fractured, deformed or settled

□ Foulwater manhole

⊠ Defective foulwater manhole

Figure 8 - Layout and Condition of the Foulwater Sewers as Revealed by the Post-failure Investigation



Legend :

– – – –	Pipe cracked, fractured, deformed or settled	- GT15a	Gully No. GT15a
MH15 ○	Stormwater manhole No. 15	⊗	Defective stormwater manhole

Notes :
(1) Blocked stormwater drains are not shown for clarity (see Figure J2 for their locations).
(2) The drains between gully No. GT15a & manhole No. 15, and between gully No. GT16 & manhole No. 16 were found to suffer from major leakages.

Figure 9 - Layout and Condition of the Stormwater Drains as Revealed by the Post-failure Investigation

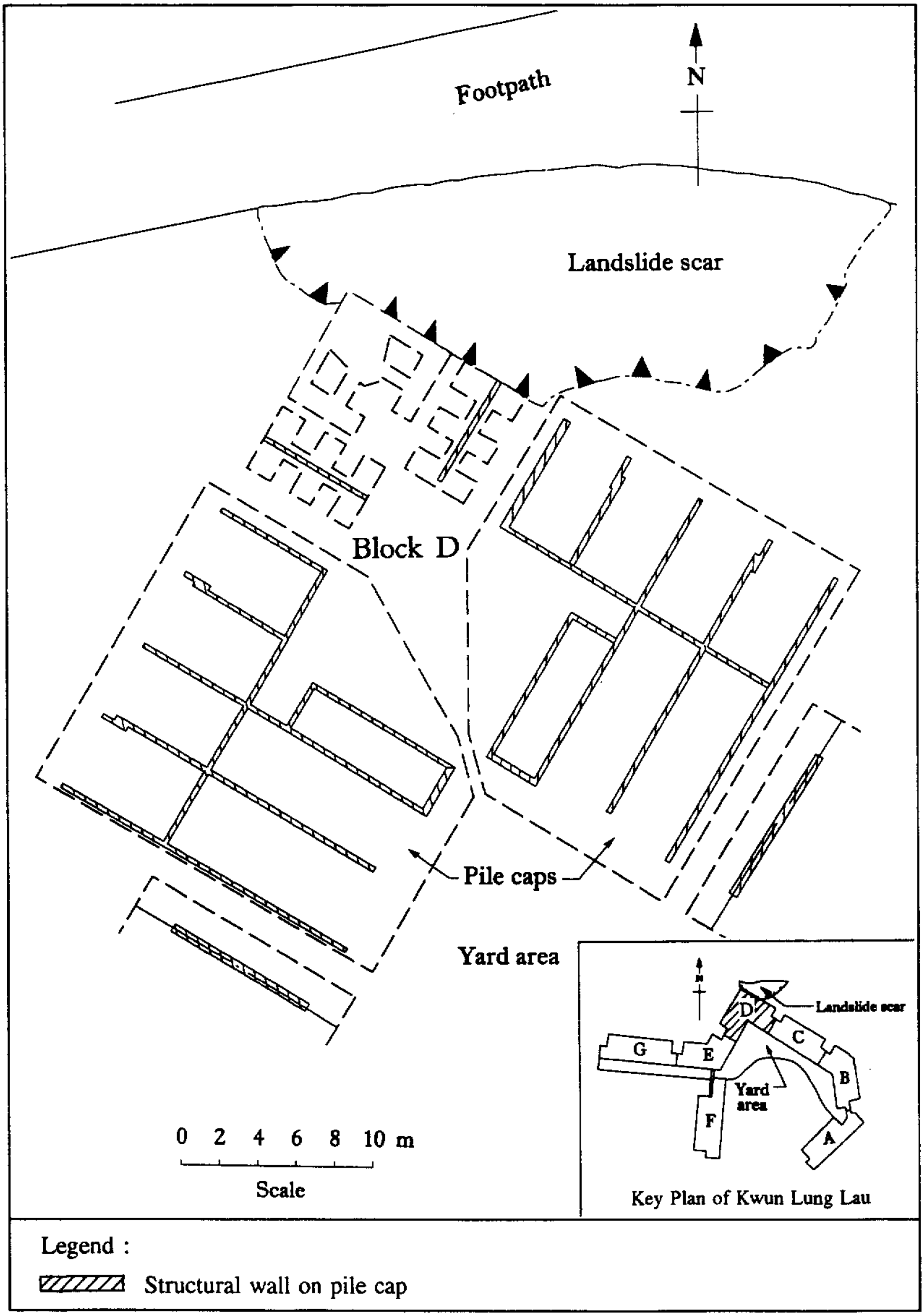


Figure 10 - Layout of Pile Caps under Block D

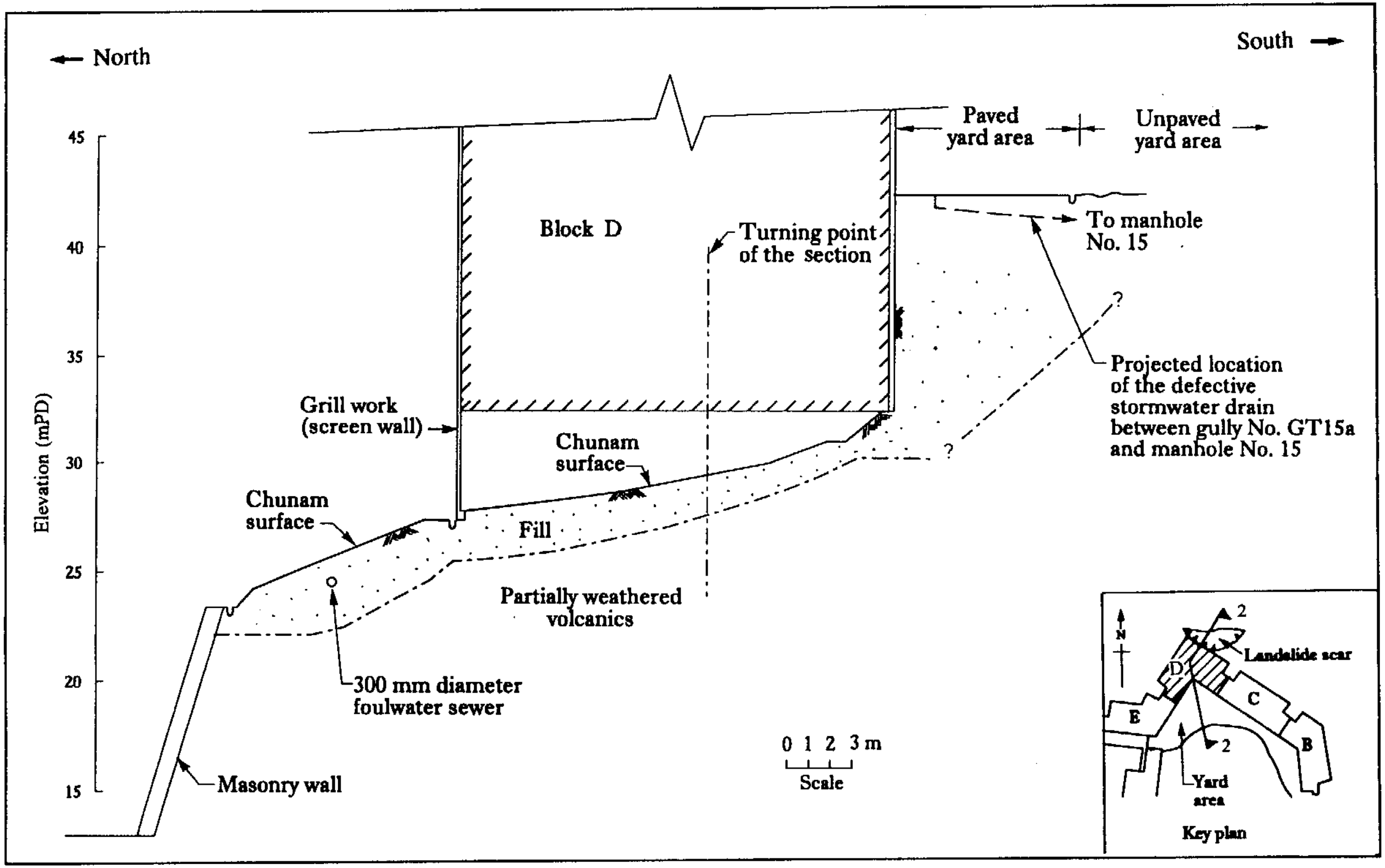
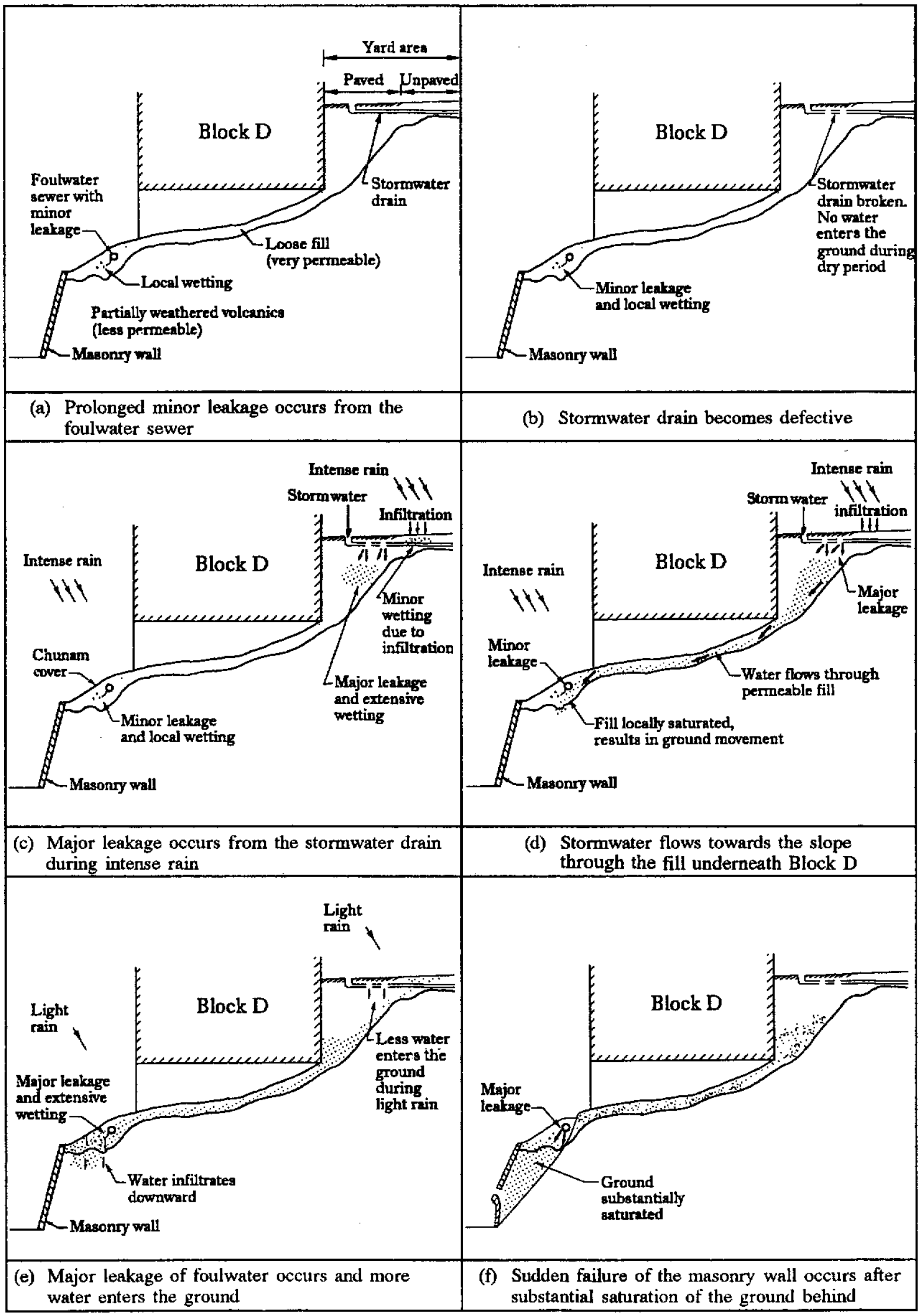


Figure 11 - Section 2-2 through the Landslide Location, Block D and the Yard Area



(a) Prolonged minor leakage occurs from the foulwater sewer

(b) Stormwater drain becomes defective

(c) Major leakage occurs from the stormwater drain during intense rain

(d) Stormwater flows towards the slope through the fill underneath Block D

(e) Major leakage of foulwater occurs and more water enters the ground

(f) Sudden failure of the masonry wall occurs after substantial saturation of the ground behind

Figure 12 - Probable Mechanism of the Landslide