# The 23 July 1994 Landslide at Kwun Lung Lau, Hong Kong

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Wong, H.N. & Ho, K.K.S., The 23 July 1994 landslide at Kwun Lung Lau, Hong Kong, Canadian Geotechnical Journal, vol. 34, no. 6, pp 825-840 

Abstract: A landslide occurred adjacent to a footpath within the Kwun Lung Lau housing estate in Hong Kong on 23 July 1994. The incident involved the collapse of a masonry wall of more than 100 years old that was likely to have been intended as a facing to a cut slope; it resulted in five fatalities and serious injuries to three other people. The Geotechnical Engineering Office (GEO) of the Hong Kong Government has carried out a comprehensive investigation into the cause and mechanism of the landslide. A number of technical findings are of interest, including the failure mechanism of a slender masonry wall and effects of leaking underground water-carrying services on slope stability. This paper documents the approach taken for the investigation and describes its technical findings. The investigation concluded that the landslide involved the buckling and brittle collapse of the masonry wall. The failure of the masonry wall was triggered by subsurface infiltration from defective buried drainage systems which saturated and weakened the soil mass. The prolonged and heavy rainfall that preceded the landslide played a contributory part in resulting in significant water ingress into the ground.

Key words: landsliding, retaining wall, infiltration.

Résumé: Un glissement de terrain a eu lieu à proximité d'un trottoir dans le lotissement de Kwun Lung Lau à Hong Kong, le 23 juillet 1994. Cet accident a provoqué la rupture d'un mur en maçonnerie vieux de plus de 100 ans qui avait vraisemblablement été conçu comme parement pour une excavation. Le tout a entraîné la perte de cinq vies humaines et de sérieuses blessures à trois autres personnes. Le "Geotechnical Engineering Office" (GEO, Bureau d'Études Géotechniques) du gouvernement de Hong Kong a conduit une étude approfondie des causes et du mécanisme du glissement. Plusieurs des conclusions techniques présentent un certain intérêt, en particulier le mécanisme de rupture d'un mur en maçonnerie mince et l'influence d'installations hydrauliques souterraines qui fuient sur la stabilité de la pente. Cet article fait le point sur I'approche qui a été suivie et décrit les conclusions techniques. L'étude a déterminé que le glissement de terrain a impliqué le flambage et l'effondrement fragile du mur. Cette rupture du mur a été déclenchée par l'infiltration d'eau souterraine causée par des systémes de drains enterrés défectueux ce qui a saturé et affaibli la masse de sol. Les précipitations prolongées et importantes qui ont précédé le glissement ont joué un rôle en provoquant un afflux d'eau important dans le terrain.

Mots clés : glissement de terrain, mur de souténement, infiltration. [Traduit par la rédaction]

## INTRODUCTION

At about 8:53 p.m. on 23 July 1994, a landslide occurred on private land below Block D of the Kwun Lung Lau housing estate, in the western district of Hong Kong Island (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 Hong Kong Government commenced a detailed investigation into the incident. The investigation team comprised six geotechnical engineers, three geologists - engineering geologists, and one surveyor of the GEO, together with supporting technical staff.

The programme for the investigation, which was carried out during the period July-November 1994, is shown in Figure 2.

## DESCRIPTION OF THE LANDSLIDE

The location of the landslide at Kwun Lung Lau is shown in Figure 3. The ground that failed comprised a portion of the masonry wall and slope below Block D (Figure 4). In front of the masonry wall is a 7-10 m wide footpath that provides a pedestrian access to Kwun Lung Lau. There is a playground and a football pitch adjacent to the footpath.

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 masonry 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 was 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 when inspected within 3 h of the landslide.

As shown in Figure 4, the majority of the landslide debris came to rest on the footpath, with some being deposited on the playground. 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. The remains of the masonry wall were displaced well forward, with the front surface of the upper part facing upward. A cross section through the landslide site, showing the ground profiles before and after the failure, is given in Figure 5.

Figure 1. View of the landslide at Kwun Lung Lau

A broken 300 mm diameter underground foul-water sewer was exposed at the crest of the landslide scar. A large volume of water was discharging from the sewer onto the failure scar after the landslide.

## HISTORY OF THE SITE

The site has a complex development history according to review of aerial photographs, old topographic maps and documents, and detailed photogrammetric survey, and an abridged account is given below.

The footpath fronting the 1994 landslide area was formed by cutting into the natural ground and the masonry wall was constructed some time before 1901. A platform above the masonry wall can be observed in the earliest available set of aerial photographs, 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 Kwun Lung Lau housing estate was constructed between 1965 and 1968. Evidence from available aerial photographs indicates that fill was placed onto the platform above the masonry wall some time between 1964 and 1969, and the ground profile has not been further modified subsequently.

In the site formation plans for Kwun Lung Lau prepared in 1965, the facing to the cut slope was shown erroneously as an existing masonry retaining wall with a base width of about 4 m.

There is no history of any previous instability at the masonry wall or the slope above. However, in other areas of the Kwun Lung Lau estate, which extends 130 m x 200 m, landslides that involved failure volumes ranging from tens of cubic metres to several hundred cubic metres have occurred over the years. The largest previous landslide occurred at a slope below Block B about 50 m to the east of the 1994 landslide location (Figure 3) following intense rain in June 1985, at the time of which the site formation works for the adjacent development (Smithfield Terrace) were in progress at the location of the failure.

## DESCRIPTION OF THE MASONRY WALL

The masonry wall that failed was a pointed squared-rubble structure with stone ties and weepholes, which was similar in appearance but thinner than the kind of masonry retaining walls once commonly constructed in Hong Kong. The masonry wall was inclined at about 75° to the horizontal (Figure 5) and had a maximum height of 10.6 m. A view of the masonry wall taken in 1991 is shown in Figure 6. The typical size of the masonry blocks found in the debris was about 300 mm high, 450 mm wide, and 300 mm long.

The surface appearance of the masonry wall, as revealed by inspection of the adjacent unfailed portion of the structure, suggests that the workmanship of the construction was generally very good. The slope above the masonry wall generally varied between 2.5 and 6 m in height at an angle of 20° to 50° to the horizontal and was covered with chunam.

At the failure location, the masonry wall was 700-800 mm thick (Figure 7), and was constructed against a cutting into natural ground. The thickness was substantially less than that shown on the site formation plans referred to earlier. It was also far below what could reasonably have been expected on the basis of previous experience of old masonry retaining walls in Hong Kong. In order to judge this "thinness" in the context of old masonry retaining walls in Hong Kong, reference was made to the available database. The records for 100 old masonry walls were examined, but only 76 of these have sufficient information on dimensions, 57 of which were studied in detail. The slenderness (i.e., height to base width) ratios of the 57 structures are plotted in Figure 8.

The majority of the walls have a slenderness ratio of less than four. Those with a slenderness ratio greater than four generally have some special reasons for their slenderness (e.g., part of the wall retaining slightly to moderately weathered rock). The masonry wall at Kwun Lung Lau had a maximum height of 10.6 m and a base width of 0.8 m (i.e., a slenderness ratio

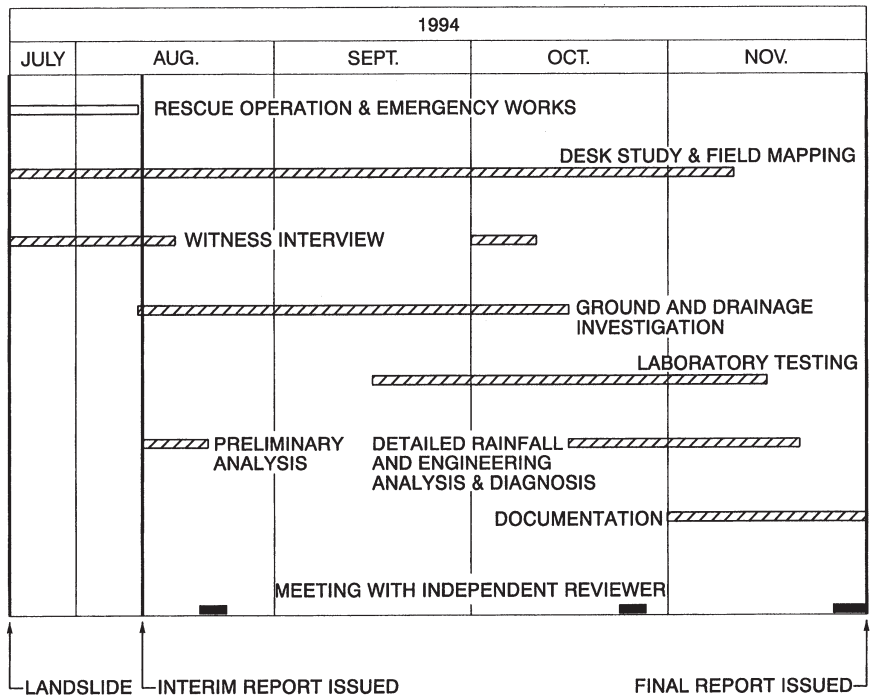



Figure 2. Programme of the Kwun Lung Lau landslide investigation

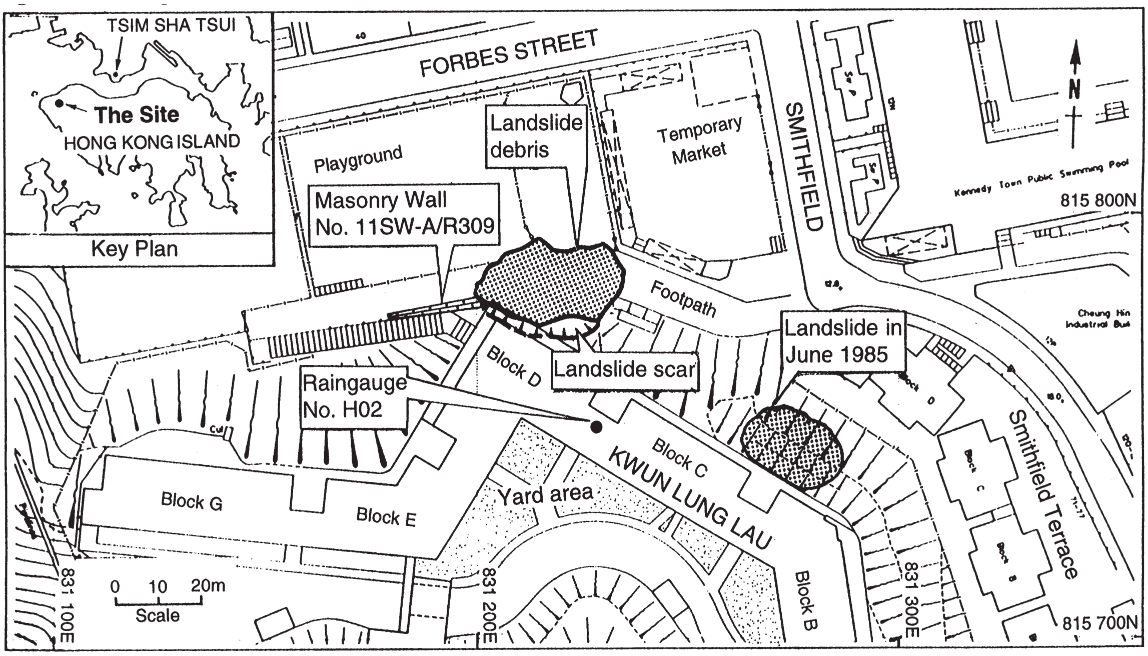



Figure 3. Site location plan. Base map from survey sheet 11-SW-6D dated 29 September 1992. Scale 1 : 1000

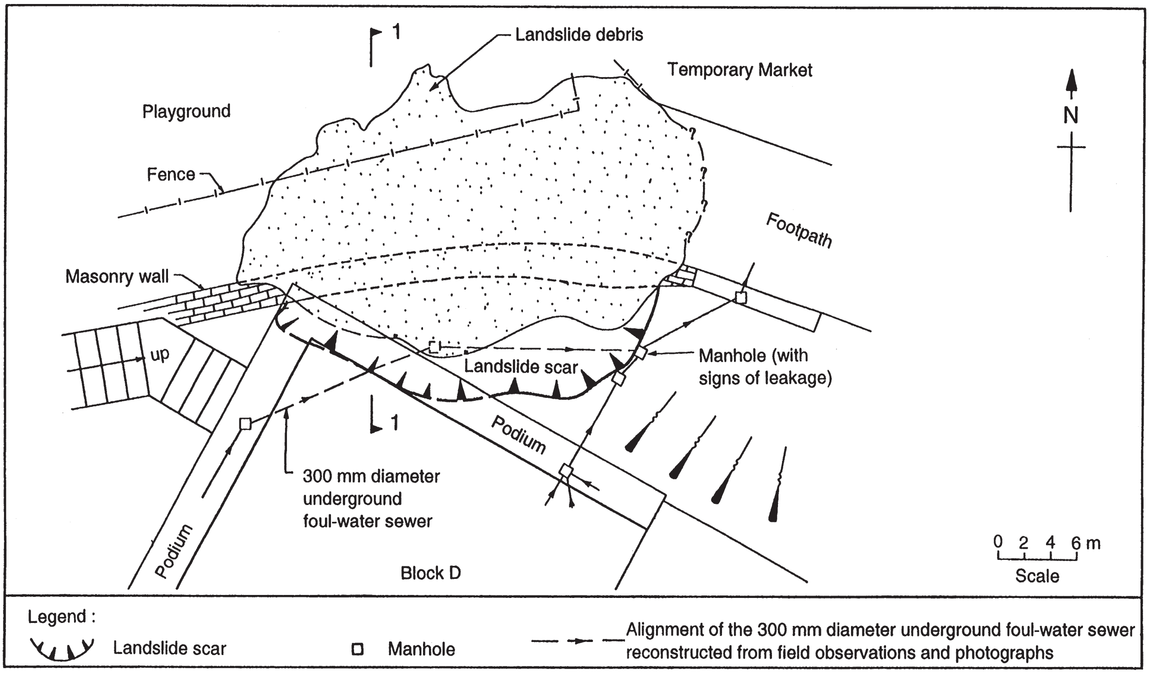
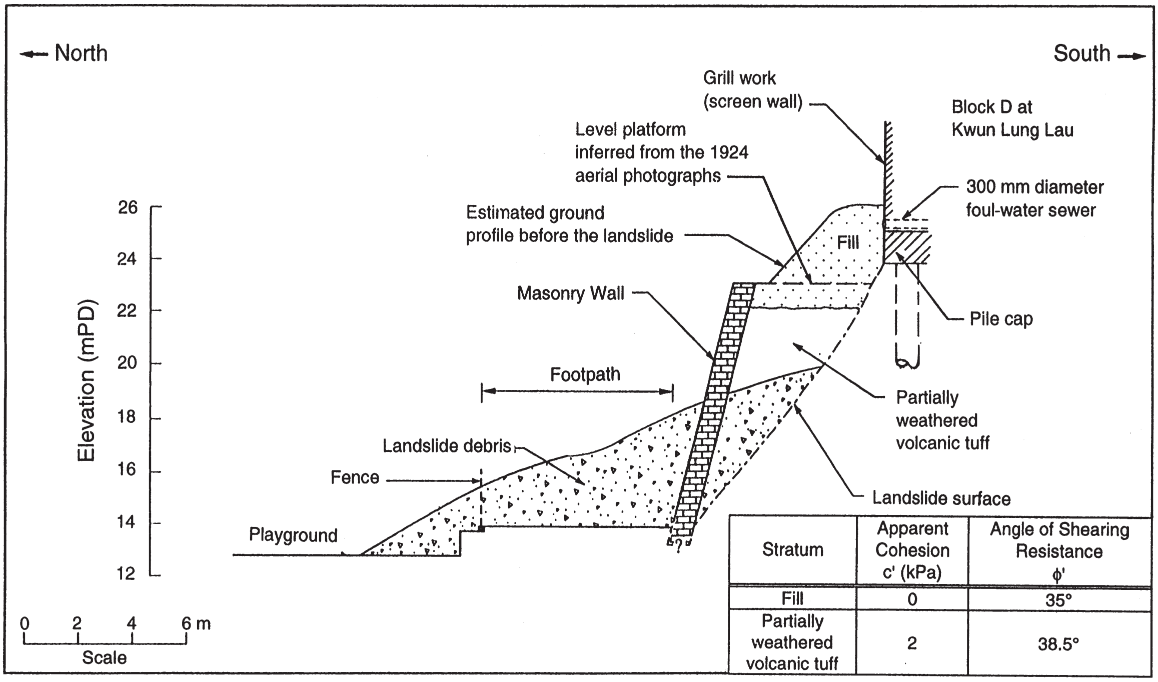



Figure 4. Plan of the landslide. See Figure 5 for details of section 1-1. Information in this figure is based on topographic surveys, engineering geology mapping, and field observations




| Stratum | Apparent Cohesion c' (kPa) | Angle of Shearing Resistance $\phi'$ |
|---|---|---|
| Fill | 0 | 35° |
| Partially weathered volcanic tuff | 2 | 38.5° |

Figure 5. Section 1-1 through the landslide. See Figure 4 for location. For clarity, podium of Block D is not shown. mPD, metres above Hong Kong Principal Datum

of > 13). As can be seen from Figure 8, the wall was exceptionally slender.

ANALYSIS OF RAINFALL RECORDS

One of the 48 automatic rain gauges (No. H02) of the GEO is located on the roof between blocks C and D

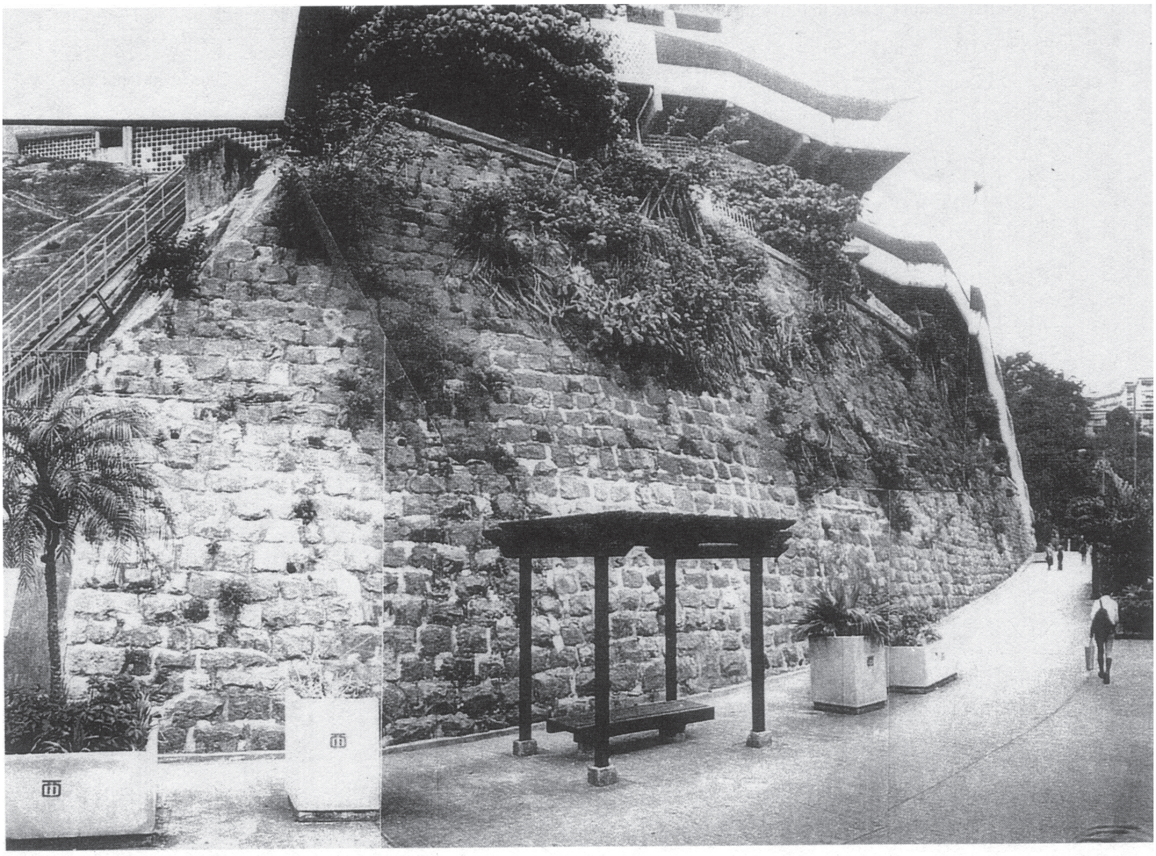


Figure 6. Masonry wall 11SW-A/R309 in January 1991

Figure 7. Unfailed part of the masonry wall immediately to the west of the landslide area (photograph taken on 25 July 1994)

at Kwun Lung Lau. The daily rainfalls recorded by the rain gauge in July 1994, together with the hourly rainfalls from 21 to 23 July 1994, are shown in Figure 9. In the month of July 1994, a total of 915 mm of rain had been recorded by the rain gauge up to the time of the landslide on 23 July 1994. This amount had already exceeded the highest rainfall ever registered in any July at the Royal Observatory's station in Tsim Sha Tsui (Figure 3) since records began in 1884.

There was very heavy rain on 22 and 23 July 1994. In the 48 h before the landslide, a total of 547 mm was recorded by rain gauge H02. 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 h 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 and 3:05 a.m. on 22 July 1994, about 42 h prior to the landslide. Only 29 mm of rain was recorded by the rain gauge between 11:00 a.m. on 23 July 1994 and the time of the landslide, i.e., approximately 95% of the 48 h rainfall fell more than 10 h 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.

It is noteworthy that the July 1994 rainstorm contained the heaviest long-duration rainfall (i.e., 2 days or more) recorded at Kwun Lung Lau since the installation of rain gauge H02 in September 1978 (Figure 10). The long-duration rainfall intensities were significantly higher than those of the previous rainstorms experienced at Kwun Lung Lau since September 1978. For example, the 48 h rainfall before the landslide exceeded the previous maximum,

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recorded in the September 1993 rainstorm, by about one third.

## OBSERVATIONS MADE PRIOR TO THE LANDSLIDE

The masonry wall and the slope above were inspected by a number of professional parties between 1980 and 1994. The masonry wall was found to be in fair or good condition during the visual inspections, and no signs of distress were ever recorded. Recommendations made as a result of routine maintenance inspections were only for relatively minor maintenance works, such as removal of unplanned vegetation and clearance of weepholes.

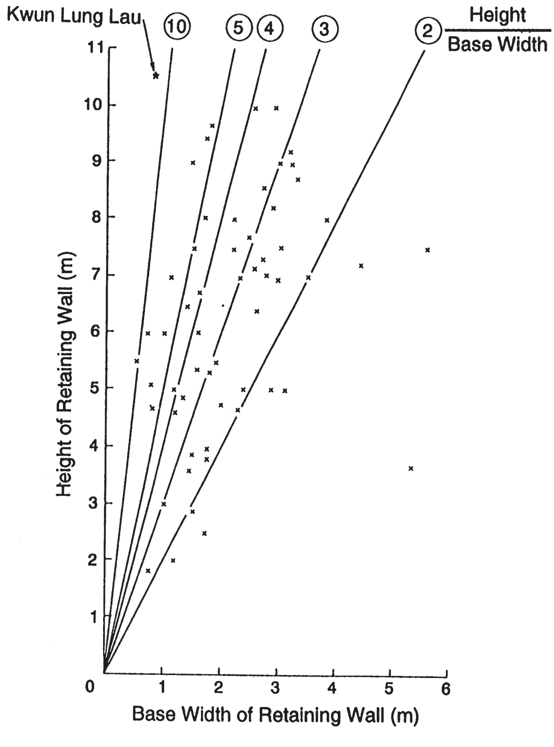


Figure 8. Dimensions of masonry retaining walls examined in previous GEO studies. The circled numbers refer to the slenderness ratio

In the annual maintenance inspection carried out in June 1994 by the owner's consultants, some signs of distress and inadequate maintenance were noted, including localized settlement near the slope crest, cracking of the chunam cover, and presence of unplanned vegetation. In addition, leakage from a manhole located immediately east of the slope above the masonry wall was also observed.

The sequence of events prior to the failure on 23 July has been reconstructed from the accounts of witnesses.

In the 2 days before the landslide, muddy water was observed coming out of weepholes near the toe of the masonry wall.

During the period of about $2^1/_2$ hours before the landslide, a number of pertinent observations were made. 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 allegedly 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 "burst" at about mid-height and collapsed almost instantaneously.

To summarize, the landslide was characterized by (i) sudden and rapid failure taking place almost instantaneously; (ii) delayed, rain-induced failure; (iii) extensive seepage, which is indicative of saturation of the ground mass shortly before failure; and (iv) brittle failure involving buckling at mid-height of the masonry wall with little prior warning.

## SUBSURFACE CONDITIONS AT THE SITE

A detailed engineering geological study was conducted to establish the representative geological model for the site. This included field mapping of limited exposures of the landslide scar and the adjacent areas (the mapping was impeded by the rescue operation, debris removal, difficult accessibility, and emergency repair works), review of published geological maps and documents, search of previous ground investigation data, and interpretation of aerial photographs.

The ground investigation, carried out in accordance with the standard of practice presented by the Geotechnical Control Office (GCO 1987), comprised 18 drill holes, 15 trial pits, three surface strippings, and 10 coreholes through the unfailed part of the masonry wall (Figure 11). Disturbed and undisturbed soil and rock samples were retrieved for visual inspection and laboratory testing. Field tests, including standard penetration tests (SPT), falling-head permeability tests, sand replacement tests, GCO-probing tests, and index tests were carried out to determine the in situ properties of the soil. In addition, 24 pits were dug underneath Block D and in the yard area to the south of the block for inspection of the building substructure and drainage systems and observations of water seepages.

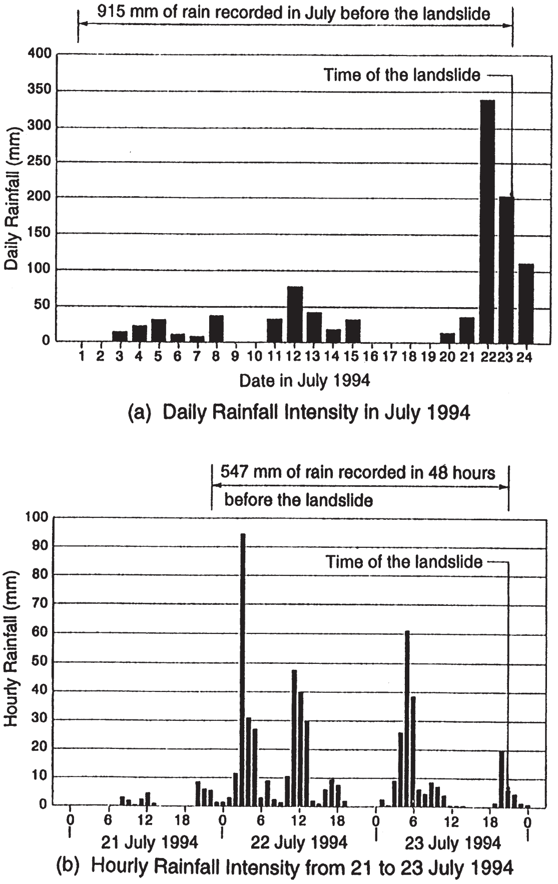


Figure 9. Rainfall records of GEO rain gauge H02

Full-length, triple-tube, core-barrel samples were retrieved from selected drill holes for logging purposes and specifically to identify the presence of any unusual geological material or features.

The geology at the landslide location comprised fill overlying partially weathered volcanic tuff. The interface between the fill and the underlying natural ground was interpreted to have been about 1 m below the crest of the masonry wall.

The fill was generally derived from weathered volcanic tuff. It comprised very loose to loose silty gravelly fine sand and soft to firm sandy clay and silt with some cobbles and fragments of brick, concrete, plastic sheet, and other man-made materials. Hand penetrometer readings show that the fill was variable and could be locally very loose or very soft. The partially weathered volcanic tuff consisted of completely decomposed and highly decomposed coarse ash tuff. No adversely orientated and persistent relict joints within the partially weathered volcanic tuff could be identified.

The results of water-content determinations indicated that the ground beneath the landslide scar was very wet, with the soils having a high degree of saturation.

The results of the dry or water-flush coreholes through the adjacent unfailed portion of the masonry wall were inconclusive because of the difficulties in properly retrieving rubble set in soil or mortar matrix. Better results were achieved by a horizontal drill hole which was sunk into the section of the masonry wall to the west of the landslide area using a double-tube core barrel with foam as flushing medium. The horizontal drill hole showed that the masonry wall thickness was about 0.8-1 m at that location.

## GROUNDWATER MONITORING AND FIELD PERMEABILITY TESTS

Twenty-one piezometers and a standpipe were installed in boreholes, and the highest groundwater level recorded after the landslide was about 3 m below the lowest point of the portion of the masonry wall and slope that failed. This is consistent with the field observations that 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 at the time of the landslide. Localized seepages were observed, however, in the fill stratum during the course of the ground investigations.

A total of 15 jet-fill tensiometers were installed after the landslide to measure the soil suction in the fill and partially weathered volcanic tuff. Results indicate that the soil suction below the landslide scar and in the ground behind the masonry wall generally ranged from

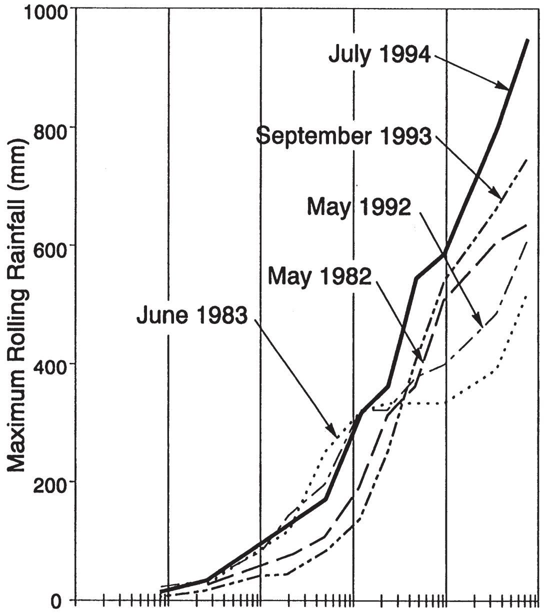


Figure 10. Maximum rolling rainfalls at rain gauge H02 for major rainstorms

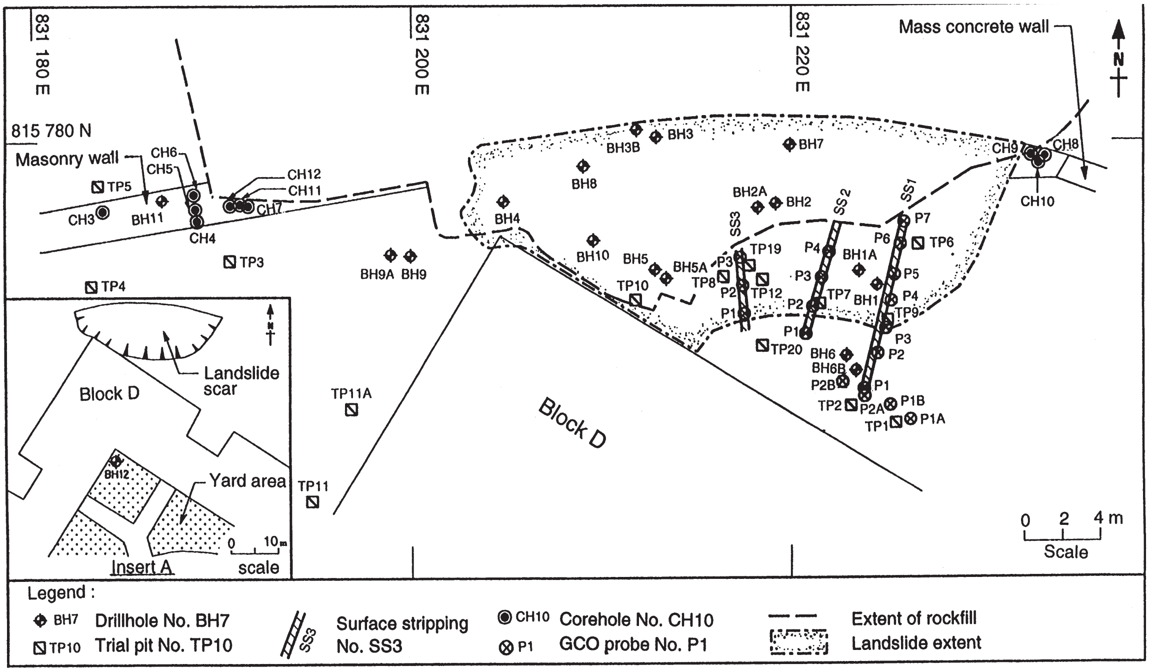


Figure 11. Location plan of the ground investigation work

15 to 30 kPa.

The permeability of the ground was assessed by tests in drill holes, tests in trial pits dug in the ground underneath Block D, and infiltration tests in the yard area to the south of Block D.

Borehole tests generally indicate that the permeability of fill is about $10^{-4}$ m/s and the permeability of partially weathered volcanic tuff is of the order of $10^{-6}$ m/s.

The lack of response in all the tensiometers, except for one, during the water tests underneath Block D indicated that the fill material largely remained unsaturated and that water mainly flowed in the fill along some preferential flow channels rather than through the soil mass.

The permeability of the lower part of the fill stratum, where there were no noticeable preferential flow channels, was estimated to be about $10^{-5}$ m/s, However, the upper part of the fill stratum, with preferential flow channels, was much more permeable. The permeability of the flow channels was of the order of 5 x $10^{-2}$ m/s, and the average mass permeability of the fill stratum was calculated to be about 2 x $10^{-4}$ m/s.

Double-ring, constant-head field infiltration tests in the unpaved ground in the yard area to the south of Block D show steady-state infiltration capacities ranging between 1.1 x $10^{-6}$ and 2.8 x $10^{-6}$ m/s.

## LABORATORY INVESTIGATION OF MATERIAL CHARACTERISTICS

A range of classification and index tests, such as Atterberg limits, specific gravity, particle-size determination, and compaction, was carried out.

The shear strengths of the fill and partially weathered volcanic tuff in the vicinity of the failure were measured by consolidated drained and undrained triaxial compression tests, shear box tests, and special tests such as dead-load tests, stress-path tests (involving increasing pore-water pressure at constant deviator stress), and unsaturated triaxial tests.

The best estimate strength parameters for partially weathered volcanic tuff correspond to apparent cohesion $c'$ of 2 kPa and angle of shearing resistance $\phi'$ of 38.5° and those for fill correspond to $c'$ of zero and $\phi'$ of 35°. No exceptionally weak material was found in the trial pits, boreholes, or exposures at the landslide site.

The results indicate that the stress-strain relationship obtained from the imposed stress path (Figure 12), which better simulates the process of gradual increase in pore-water pressure in a slope, exhibits stiffer and more brittle behaviour than that obtained from conventional triaxial compression tests. The soil strengths given by the special tests are generally higher than that determined in conventional triaxial compression tests, whereas the postpeak strengths are comparable.

Results of unsaturated triaxial tests are consistent with those from previous tests on similar materials.

Point-load tests in accordance with the procedures recommended by the International Society for Rock Mechanics (ISRM 1985) were carried out on mortar

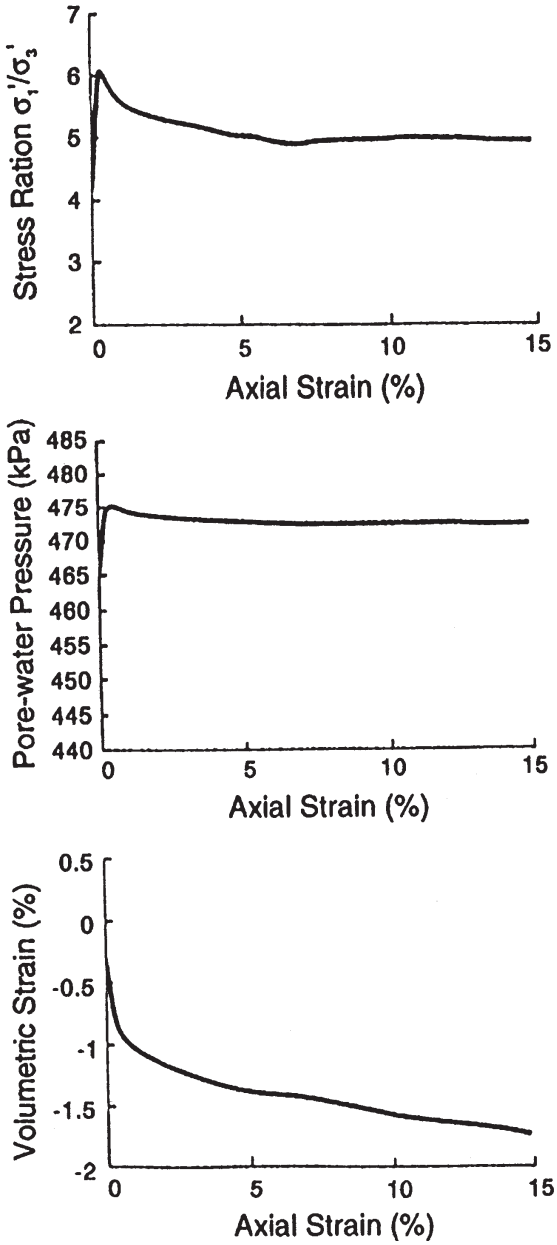


Figure 12. Typical behaviour of CDV in a stress-path triaxial test. $\sigma_1'$ and $\sigma_3'$, major and minor principal effective stress, respectively

samples taken from between the masonry blocks recovered from the landslide debris.

The tests were conducted on samples in their natural state and on samples soaked in water for 7 days. Results show that there is some scatter in the results, with the point-load indices varying between 0.08 and 0.26 MPa for samples in their natural state, with a mean of 0.18 MPa. The corresponding indices for soaked samples varied between 0.07 and 0.48 MPa, with a mean of 0.22 MPa. Overall, there is no significant reduction in the mortar strength upon soaking.

## CHEMICAL TESTS

Chemical analyses of both soil and water samples were carried out in accordance with the procedures recommended by the British Standards Institution (BSI 1990), American Public Health Association (1992), and American Society for Testing and Materials (1993). Water samples were either taken directly from the landslide site or prepared by squeezing the soil samples with the use of an oedometer.

For the soil samples, the analyses comprised the determination of organic matter content to assess if the black-stained soil in the close vicinity of the foul-water sewer was affected by leakage of foul water. Some samples provided a control in that they were known not to have been affected by foul water (organic matter content of 0.1 - 0.3% by mass).

The sample of deposit recovered from the inside surface of the foul-water sewer had a much higher organic matter content than the control samples (>20% by mass), whereas samples of black-stained soil recovered from the outside surface of the foul-water sewer or from manholes with signs of leakage had an organic matter content of about 3-5% by mass. It can therefore be concluded that the black-stained soil taken from the outside of the foul-water sewer or from manholes was likely to have been affected by leakage from the foul-water sewer.

As sea water is used for toilet flushing at Kwun Lung Lau, chloride contents (about 18 000 mg/L) can be used as a reasonable basis for delineating the source of the water in the soil. Samples taken underneath Block D which are away from the influence of the defective foul-water sewer provided a control on the chloride content (<150 mg/L). Samples of the foam used for advancing the drill holes were also tested, and the results confirmed that the chloride content of the soil would not have been affected by the foam.

Chemical analyses of water seepages from the ground and water samples obtained from the soil beyond the landslide scar downslope of the foul-water sewer showed that the chloride content was high (generally in the range 500-2000 mg/L), and was comparable to that of the foul-water samples in the sewer (4000-7000 mg/L). However, soil above the elevation of the foul-water sewer had a low chloride content (75-150 mg/L) despite a high moisture content. Although foul water was discharged from the severed sewer onto the landslide scar, the point of discharge was far from the drill-hole locations. Therefore, the high chloride content measured cannot be due to the discharge of foul water subsequent to the landslide. Thus, the evidence suggests that a large amount of foul water had entered the ground prior to the landslide, but

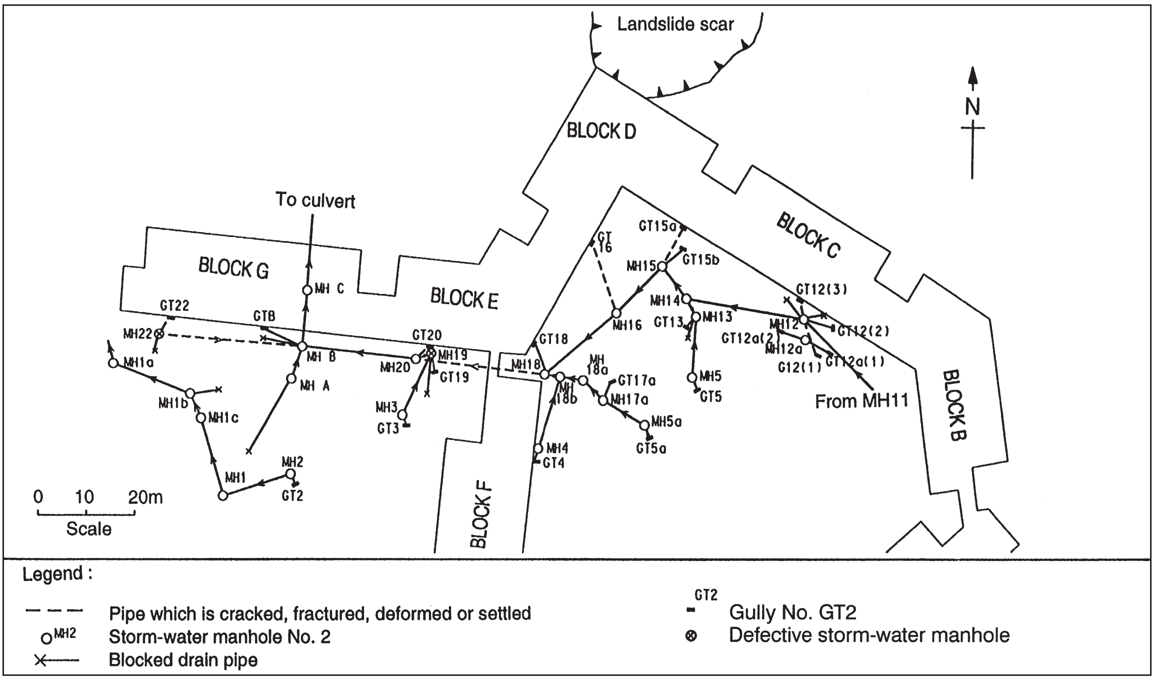


Figure 13. Layout and condition of the storm-water pipes at the time of the postfailure investigation. The drains between gully GT15a and manhole 15 and between gully GT16 and manhole 16 were found to suffer from major leakages

that there was a source of water other than the foul-water sewer which had also contributed to the wetting of the ground.

## FOUL-WATER AND STORM-WATER DRAINAGE SYSTEMS

A detailed drainage investigation of the area around blocks B-G of the Kwun Lung Lau estate was carried out after the landslide. The layout of the underground storm-water sewers in the vicinity of the landslide is shown in Figure 13. These drainage systems were constructed as part of Kwun Lung Lau in the mid- to late 1960s.

The portion of the 300 mm diameter foul-water sewer that was severed by the landslide was assessed to have run across the upper part of the landslide area before the failure (Figure 4), 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

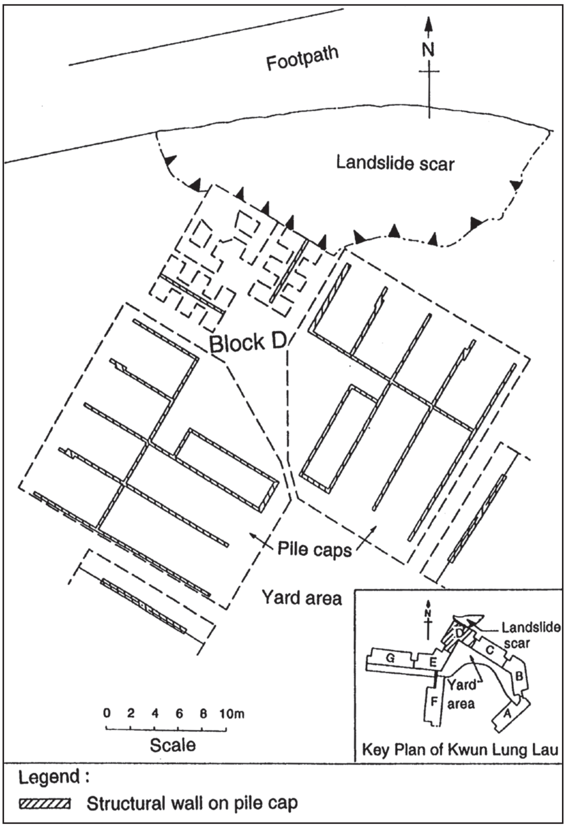


Figure 14. Layout of pile caps under Block D

debris. Chemical analyses of the black-stained soil showed that the material had probably been affected by foul water, suggesting that leakage of foul water had been occurring for some time before the landslide.

An examination of the Kwun Lung Lau sewer and stormwater sewer systems in the vicinity of the landslide, including extensive closed-circuit television (CCTV) surveys, revealed defects such as local settlement, cracked pipes, and dislocated or open joints. In particular, two sections of the underground storm-water pipes in the yard area to the south of Block D (Figure 13) were found to be suffering from major leakages.

The broken sewer exposed in the landslide scar carried foul water from blocks D, E, F, and G. The rate of discharge from this sewer was measured over a period of 2 weeks after the failure, and this was found to range generally from 0.1 to 0.7 $m^3$/min, with the peak rate occurring between 6:00 and 10:00 p.m.

A thorough investigation of the sewer systems in the vicinity, comprising dye tests, manhole inspections, CCTV surveys, and flow monitoring, provided no evidence of any connection of the storm-water system to the foul-water sewer.

## PATHWAYS FOR SUBSURFACE SEEPAGE FLOW

The layout of the pile caps and structural walls of the substructure of Block D is shown in Figure 14. Block D is underlain by two large pile caps founded on weathered volcanic tuff, with fill placed within the gap between the caps. The fill extends to the landslide site and the yard area to the south of Block D, as shown in Figure 15.

The fill material was observed to be loose and to contain voids through which subsurface seepage flow could take place preferentially. Water tests carried out after the landslide confirmed that the fill was permeable. During the water 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.

The yard area to the south of Block D is paved, except for the garden areas. The fill beneath the yard area, within which the underground storm-water 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.

## NUMERICAL ANALYSES

### *Conventional slope and retaining wall stability analyses*

Limit equilibrium slope and retaining-wall analyses were carried out. Retaining-wall analyses were undertaken because the masonry wall would have provided a measure of structural support to the cut face by way of its retaining action. The analyses indicate conventional slope and retaining-wall failure due to soil saturation is possible given the probable range of strength parameters of the soil and the masonry wall. However, the stability is particularly sensitive to the strength (viz. $c'$) of the mortar joint between the masonry blocks. It is noteworthy that the actual failure mechanism was different to those assumed in the conventional slope and retaining-wall analyses. Hence, the results of the conventional analyses may not be representative.

### *Distinct-element analyses*

Advanced numerical analyses were carried out to assist in the diagnosis of the mechanism and cause of the landslide. The analyses were aimed at modelling the response of the masonry wall and the retained ground when the soil's shear strength was reduced upon saturation. No prior assumptions were made about the mode of the failure.

The analyses were carried out using the distinct-element computer program UDEC supplied by ITASCA. In the analyses, the masonry wall was modelled as a discontinuum, and was represented as an assembly of discrete blocks with mortar in between. Large displacements along the mortar joints and rotation of the blocks were permitted in the program. The assumed constitutive model simulated the effect of damage to the mortar joints due to tensile or shear failure, with $c'$ of the failed portion of the joints being set to zero. The retained ground was modelled as a continuum. Details of the modelling procedures and assumptions on material properties are given in Geotechnical Engineering Office (1994).

The analyses predict that the masonry structure would fail in a complex mode. The mode of masonry wall deformation and slope movement during the landslide is depicted in Figure 16. The masonry wall is found to bulge initially at about mid-height, accompanied by overturning of the portion of the masonry wall below this level. These deformation modes combine to lead to tensile failure and consequential reduction of the shear strength of the mortar joints. Bulging and overturning continue, resulting in brittle fracture of the masonry wall at about mid-height. The ground behind the masonry wall

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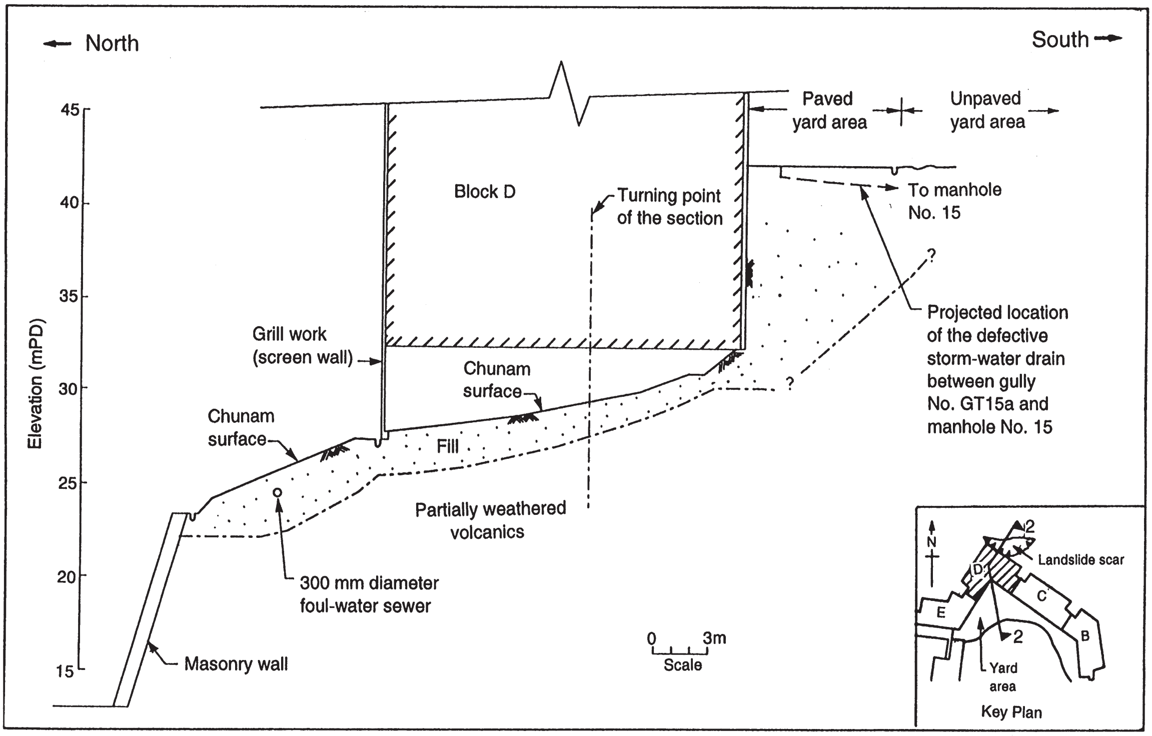


Figure 15. Section 2-2 through the landslide location, Block D, and the yard area

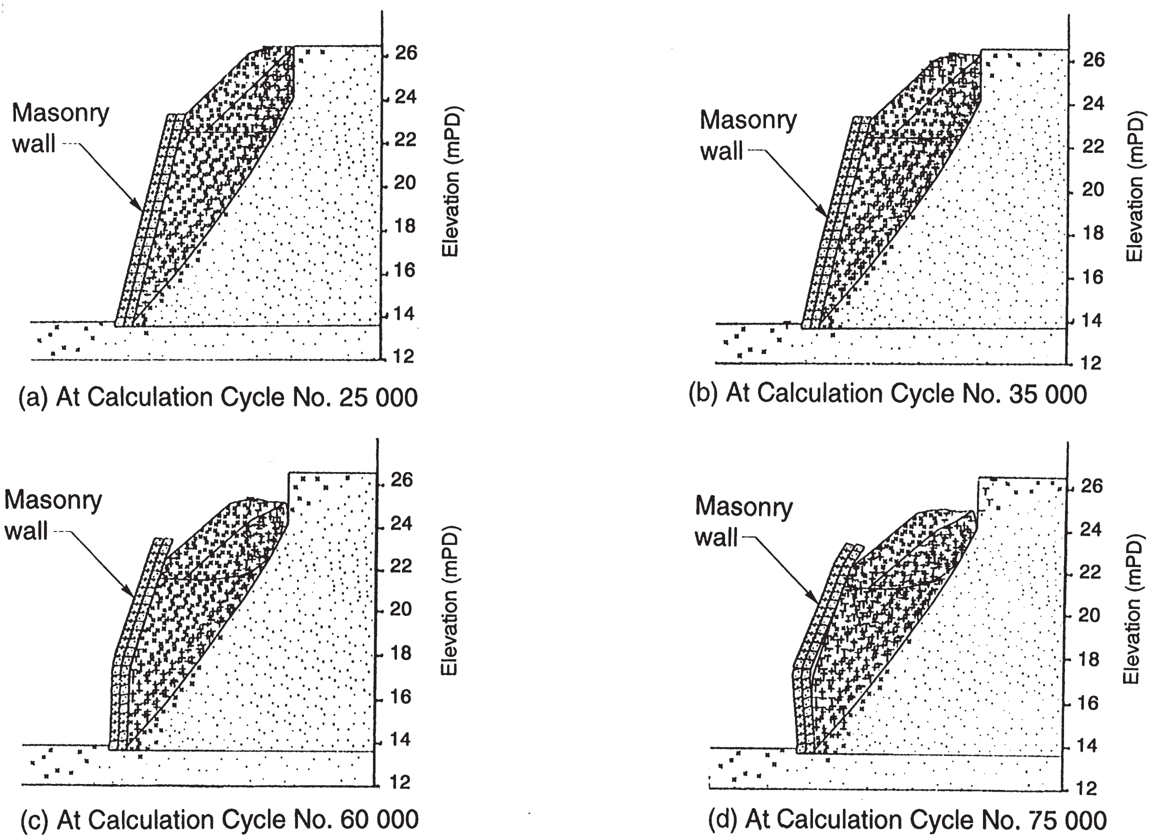


Figure 16. Results of UDEC analyses (wall condition and stress state of case KC-1 during bulging failure)

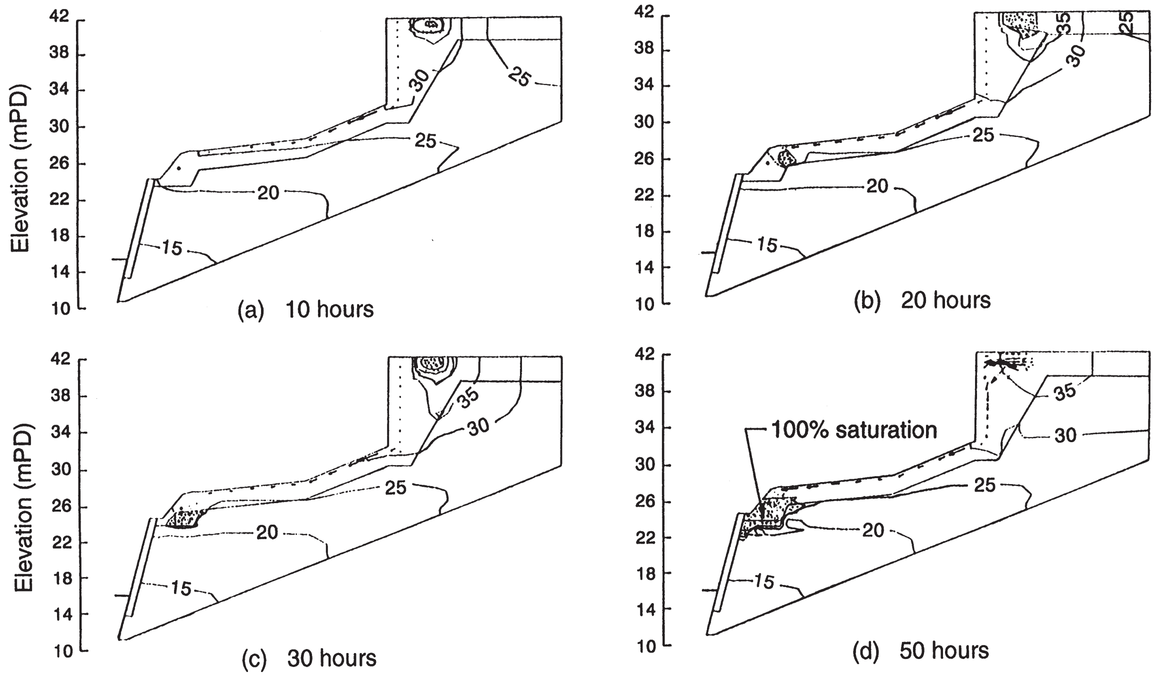


Figure 17. Extent of wetted zone at different times due to storm-water leakage. Contours give total head in metres

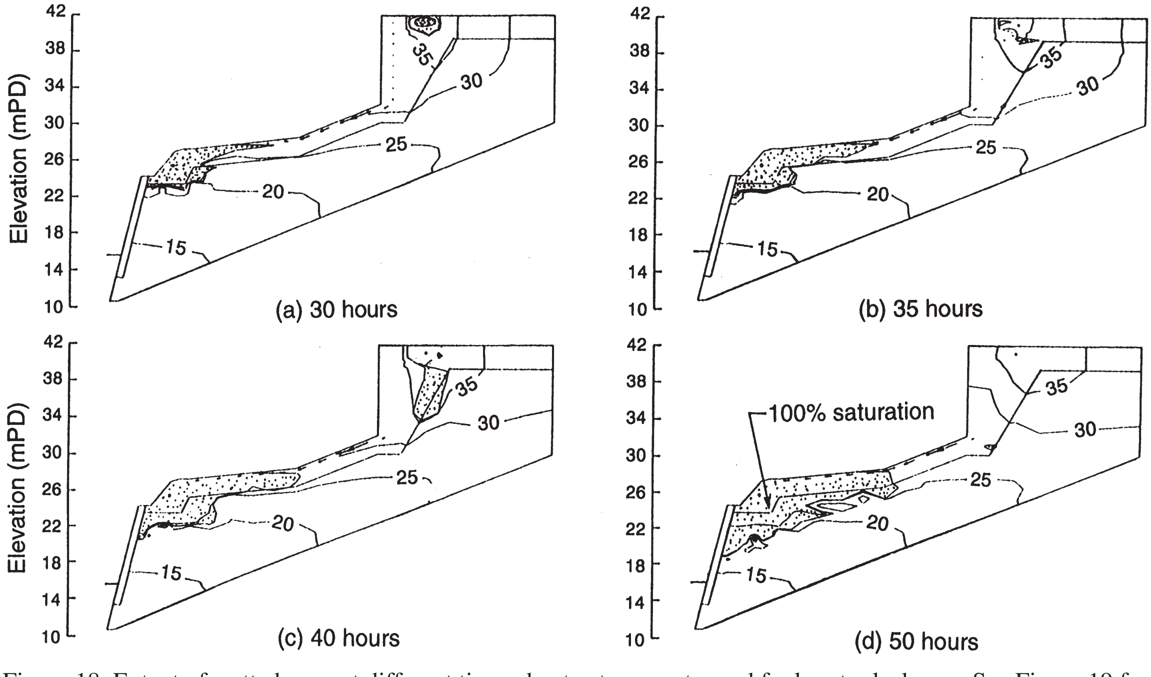


Figure 18. Extent of wetted zone at different times due to storm-water and foul-water leakages. See Figure 19 for extent of the wetted zone before 30 h. Contours give total head in metres

loses support and slides forward. The analyses suggest that the upper part of the masonry wall would rotate backward as a result of the displacement of the sliding mass and is predicted to come to rest on the surface of the debris, with the front surface facing upward. The lower portion of the masonry wall is predicted to disintegrate and become buried by the debris.

An important finding was that once failure of the mortar joints was initiated, the masonry wall would deform rapidly and instability would develop in an uncontrolled manner.

For the 1994 ground profile (i.e., after the construction of Kwun Lung Lau), the analysis showed that the masonry wall would remain stable with relatively small deformations if saturation of the retained material was not extensive. This is consistent with the fact that no significant signs of distress were observed over the years.

Laboratory test results indicated that the saturated $c'$ value of the completely weathered volcanic tuff and completely to highly weathered volcanic tuff is about 2 kPa, and the analyses show that failure occurs if the ground above the observed landslide surface is substantially saturated. Failure still occurs even if the $c'$ value is a few kilopascals higher, which might be the case for the in situ weathered volcanic, because of the presence of less weathered material and bonding which may not be measured by conventional triaxial and shear box tests.

### *Finite-element seepage analyses*

The purposes of the seepage analyses were to assess the contribution of the different sources of water to soil saturation and to examine the delayed nature of the landslide. Finite-element seepage analyses were carried out.

Transient seepage analyses were undertaken to consider the 50 h period starting from the beginning of the rainstorm through to the time of the landslide. The measured rainfall profile was modelled along with the suction measurements given by the tensiometers.

The analyses aim to provide a basis for assessing the relative importance of the various sources of water in triggering the landslide, rather than to give a precise prediction of the pattern of water flow.

Analyses were carried out for both plan and vertical sections. Two-dimensional analyses were considered, but the three-dimensional nature of the problem was taken into account approximately by adjusting the thickness of each element.

The key findings of the analyses may be summarized as follows:

(1) Surface infiltration via the unpaved yard area is of secondary importance in comparison with leakages from the storm-water drains and foul-water sewer.
(2) The fill behind the masonry wall could become substantially saturated due to leakage from the storm-water drains, but only a relatively small part of the weathered volcanic tuff would have been saturated in 50 h (Figure 17).
(3) Substantial saturation of the fill would occur about 1 day before the landslide due to leakage from storm-water drains, and given major leakage from the foul-water sewer following wetting of the fill stratum, a large part of the retained weathered volcanic tuff could become substantially saturated by the time of the landslide (Figure 18).

## DIAGNOSIS OF THE LANDSLIDE

The stability analyses indicate that the masonry wall would have become unstable if the ground became substantially saturated.

The four possible causes of water ingress, either separately or in combination, which have been identified are (i) a rising groundwater table, (ii) direct surface infiltration, (iii) subsurface seepage flow, and (iv) leakage from the storm-water and (or) foul-water 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 1 month before the landslide, the extensive chunam cover would have prevented significant direct surface infiltration at the landslide location. Inspections by staff of the property owner 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 owner's 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.

The forensic investigation has established 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. The source of the water was from the defective storm-water drains in the yard area, direct surface infiltration through the unpaved garden areas in the yard, or through the slopes farther uphill to the south.

Based on the seepage analyses, leakage from the defective storm-water drains beneath the yard area is likely to have been the principal source of subsurface seepage flow towards the landslide location.

The observed signs of prolonged leakage from the foul-water sewer suggest that this must have contributed to the water ingress into the area where the landslide occurred. Measurements of the flow rate in the foul-water sewer after the landslide indicate that the sewer would have provided a sufficient quantity

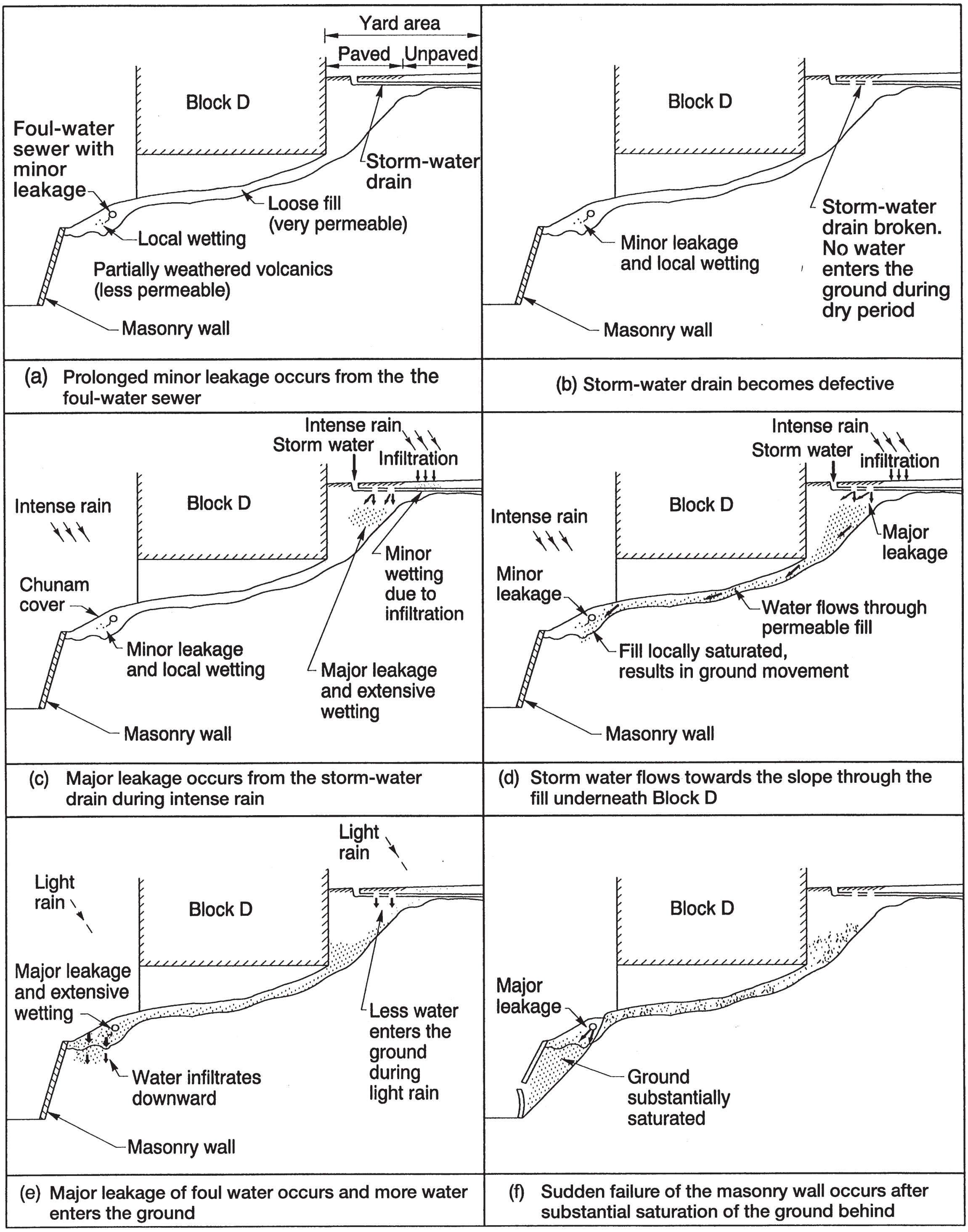


Figure 19. Probable mechanism of the landslide

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 (i) deterioration in the structural condition of the masonry wall, (ii) application of surcharge loading at the back of the masonry wall or excavation works in front of the masonry wall shortly before the landslide, and (iii) 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 masonry wall and the slope behind, but it could not have been a direct trigger of the landslide because the filling was undertaken some 25 years ago.

## LIKELY MECHANISM OF THE LANDSLIDE

Based on the investigation, it is considered that 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 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 storm-water pipes in the yard area were found to have suffered from major leakages, and there is evidence to suggest that the foul-water 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 foul-water sewer and the storm-water pipes.

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

(1) 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 foul-water sewer being a secondary factor.

(2) Major leakage from the foul-water sewer resulted from gradual deterioration of the sewer, with subsurface seepage flow being only a contributory factor.

(3) Major leakage from the foul-water sewer resulted from deformation of the fill material caused initially by subsurface seepage flow.

Scenario 1 implies that the foul-water sewer did not play a significant role in triggering the failure. However, the results of the postfailure chemical analyses indicate that foul water 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 1 is highly unlikely.

Scenario 2 implies that the landslide was caused primarily by a major leakage of foul water 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 3) suggests that, on the balance of probabilities, pure coincidence alone is less likely to have prevailed under the circumstances of this landslide.

Scenario 3 implies that both leakage of the foul-water sewer and subsurface seepage flow were together responsible for substantially saturating the ground mass. If this were so, the loose fill behind the masonry wall would have been wetted by subsurface seepage arising primarily from the leakage of defective storm-water drains during times of prolonged and high-intensity rainfall, perhaps supplemented by some minor and localized leakage from the foul-water 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 foul water. This, together with subsurface seepage flow, would have provided a sufficient quantity of water to saturate the soil behind the masonry wall. Seepage analyses have shown that it was possible for the subsurface seepage flow to reach the fill in the slope above the masonry wall in about 1 day after heavy rain, and that it would take another day for the water (both storm water and foul water) in the fill to substantially saturate the weathered volcanic tuff. This is considered to be the most likely scenario leading up to the landslide. The probable mechanism of the landslide is illustrated in Figure 19.

## DISCUSSION AND CONCLUSIONS

The forensic investigation provided information for the Death Inquest conducted by the Coroner's Court and the subsequent Select Committee on the landslide conducted by the Legislative Council of Hong Kong.

The investigation established that the sudden failure of the 100 year old masonry wall at Kwun Lung Lau was triggered by the substantial saturation of the soil behind the masonry wall, which was appreciably thinner than was usual for masonry retaining walls of a similar type of construction. Leakages from defective underground water-carrying services (both the storm-water drains and the foul-water 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 storm-water drainage system.

Numerical analyses have predicted that the masonry wall would fail with a delayed response to intense rain and with a complex mechanism, involving a combination of bulging and overturning modes followed by sudden total collapse of the ground. The failure has been predicted to be brittle and, once initiated, to develop in an uncontrolled manner. The results of the analyses are consistent with the observed mode of failure and the disposition of the collapsed masonry wall within the debris.

The thin masonry wall provided support to the steep cut by effectively acting as a retaining element and also by preserving soil suction. It has served its stabilizing function and prevented localized failures involving instability of the retained fill. The presence of the masonry wall permitted gradual advance of the wetting front in the soil without significant distress until a time when displacement of the masonry wall ensued and the mortar joint started to break up in tension and shear modes, and the whole structure failed abruptly resulting in a large-scale landslide with grave consequences. In addition, the masonry wall rendered it difficult for inspection engineers to observe telltale signs of distress of the retained material.

The complex failure mechanism cannot be readily discerned from simple slope or retaining-wall analyses, and it would be unconservative to rely on factors of safety calculated by these conventional methods. This study shows that, unlike well-proportioned masonry retaining walls, thin masonry walls are liable to fail in a brittle manner without appreciable prior warning. In practice, it would be prudent to assume such slender elements to be substandard as a retaining wall and upgrading works should be initiated to bring them up to current geotechnical standards.

The vital importance of environmental effects associated with possible leakage of subsurface water-carrying services on slope stability is emphasized by this fatal landslide. Changes in loading and boundary conditions, as a result of deterioration and lack of maintenance of subsurface water-carrying services, can lead to failure of a wall or a slope that has otherwise remained stable for a long time. The range of possible water pathways, together with details of past failures in the vicinity of a given site which might provide clues on possible generic destabilizing factors, should be carefully considered in any stability assessments.



## ACKNOWLEDGEMENTS

The forensic investigation was undertaken by a team of professional and technical staff of the GEO, without whose teamwork the investigation would not have been successfully completed. The Buildings Department and Drainage Services Department assisted with the investigation of the subsurface drainage systems. The laboratory tests were carried out in the Government's Public Works Central Laboratory, University of Hong Kong, Hong Kong University of Science & Technology, and Nanyang University in Singapore. Professor N.R. Morgenstern was appointed as an independent reviewer of the GEO investigation and was to report independently to the Hong Kong Government.

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