REPORT ON THE FEI TSUI ROAD LANDSLIDE OF 13 AUGUST 1995

Volume 2

FINDINGS OF
THE LANDSLIDE INVESTIGATION

Geotechnical Engineering Office
Civil Engineering Department
Hong Kong Government

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First published, February 1996
Reprinted, May 1996

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

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EXECUTIVE SUMMARY

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

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

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

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

CONTENTS

1. INTRODUCTION

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

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

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

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

(b) analysis of the rainfall records,

(c) interviews with witnesses to the landslide,

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

(e) geological mapping,

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

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

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

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

2. DESCRIPTION OF THE SITE

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

The cut slope was registered as No. 11SE-D/C42 by the consultants engaged by Government to prepare the Catalogue of Slopes in 1977. Before the landslide, the cut slope inclined towards the north at an average angle of about 60° to the horizontal, with a maximum height of about 27 m. Rock was exposed at the lower part of the cut slope, and the upper portion was covered with chunam. A photograph of the cut slope taken after its formation in mid-1970s, which shows the condition of the rock face and the chunamed slope surface at the time, is shown in Plate 2. Photographs of the site taken in 1994 show that the cut face was covered extensively by unplanned vegetation (Plate 3).

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

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

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

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

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

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

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

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

3. DESCRIPTION OF THE LANDSLIDE

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

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

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

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

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

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

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

4. HISTORY OF THE SITE

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

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

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

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

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

5. ANALYSIS OF RAINFALL RECORDS

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

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

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

Shown in Figure 8 is a comparison between the pattern of the rainfall prior to the 1995 landslide and those of previous major rainstorms affecting the site since installation of raingauge No. H14 in 1979. It can be seen that the rain that preceded the landslide was the highest recorded by the raingauge for durations in excess of 15 days. For rainfall durations of 7 days or less, the rainfall intensities of this rainstorm were comparable to those experienced previously.

6. SEQUENCE OF FAILURE

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

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

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

7. SUBSURFACE CONDITIONS AT THE SITE

7.1 General

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

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

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

7.2 Geology

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

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

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

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

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

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

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

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

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

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

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

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

7.3 Soil and Rock Properties

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

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

The shear strength properties of altered tuff were assessed by direct shear tests (Head, 1982) and consolidated undrained triaxial compression tests (Head, 1986). The properties of weathered rock joints were assessed by direct shear tests according to the method recommended by Hencher & Richards (1989). The test results and the shear strength parameters of the materials determined from the line of best-fit by the least squares method, are shown in Figures 12 & 13.

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

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

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

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

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

7.4 Groundwater Conditions

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

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

(b) pre-landslide observations made in previous inspections of the slope that water was seeping out of the slope at a weathered seam midway between the toe of the slope and the berm halfway up the cut face (Sections A.2.2, A.2.3 & A.2.4, Appendix A),

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

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

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

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

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

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

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

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

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

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

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

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

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

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

According to WSD's records, the section of the abandoned 6-inch water main that was severed in the landslide was connected to the service reservoir before the slope failure. Only 2.4 m of the water main were recovered by the WSD from the debris and less than 20 m of the water main were identified by the GEO from photographic records of the landslide debris. This is far short of the approximate 55-m length according to WSD's records of the water main alignment (Figure 14). Hence, there are uncertainties about the actual alignment and condition of this water main.

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

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

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

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

9. THEORETICAL STABILITY ANALYSES

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

Information obtained from the post-failure ground investigation field work, laboratory testing, and site observations and measurements was used in the analyses. A representative cross-section of the landslide site and the input parameters adopted in the analyses are shown in Figure 18. A perched water table up to 4 m above the altered tuff layer at the time of the landslide was assumed in the analyses.

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

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

10. DIAGNOSIS OF THE CAUSES OF THE LANDSLIDE

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

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

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

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

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

The possibility of the landslide being triggered by a build up of water pressure in the rock joints at or near the back scarp was also examined. Analyses have shown that, in the absence of a perched water table, the rock joint at the back scarp would have to have been filled with water to a great depth and lateral extent prior to the landslide. This scenario is considered less probable than that of the development of a perched water table. However, the possible presence of some water pressure within the rock joints cannot be excluded, and this might have combined with the perched water table in triggering the landslide.

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

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

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

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

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

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

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

11. CONCLUSIONS

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

12. REFERENCES

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

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

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

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

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

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

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

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

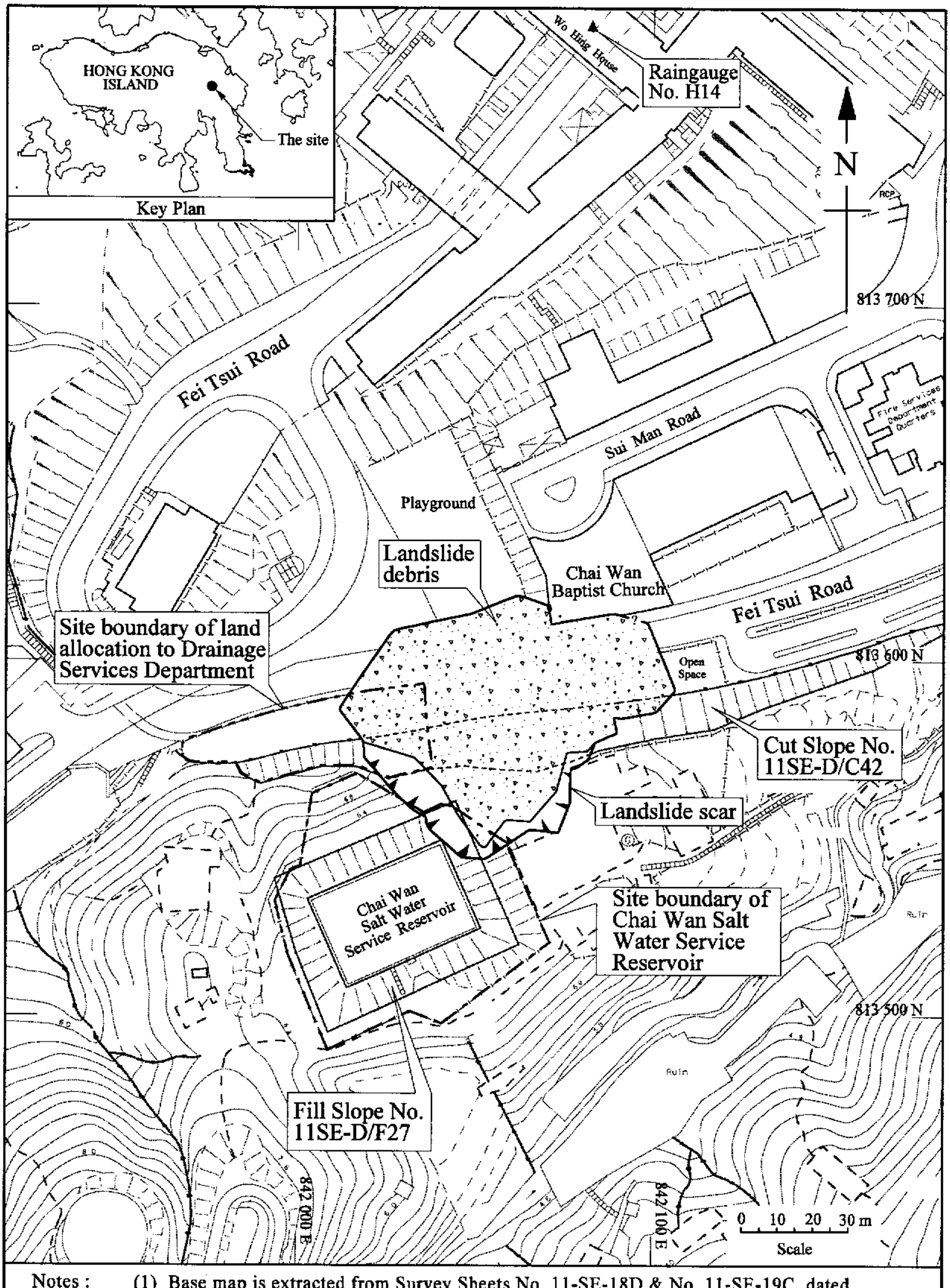
Hencher, S.R. & Richards, L.R. (1989). Laboratory direct shear testing of rock discontinuities. Ground Engineering, vol. 22, no. 2, pp 24-31.

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

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

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

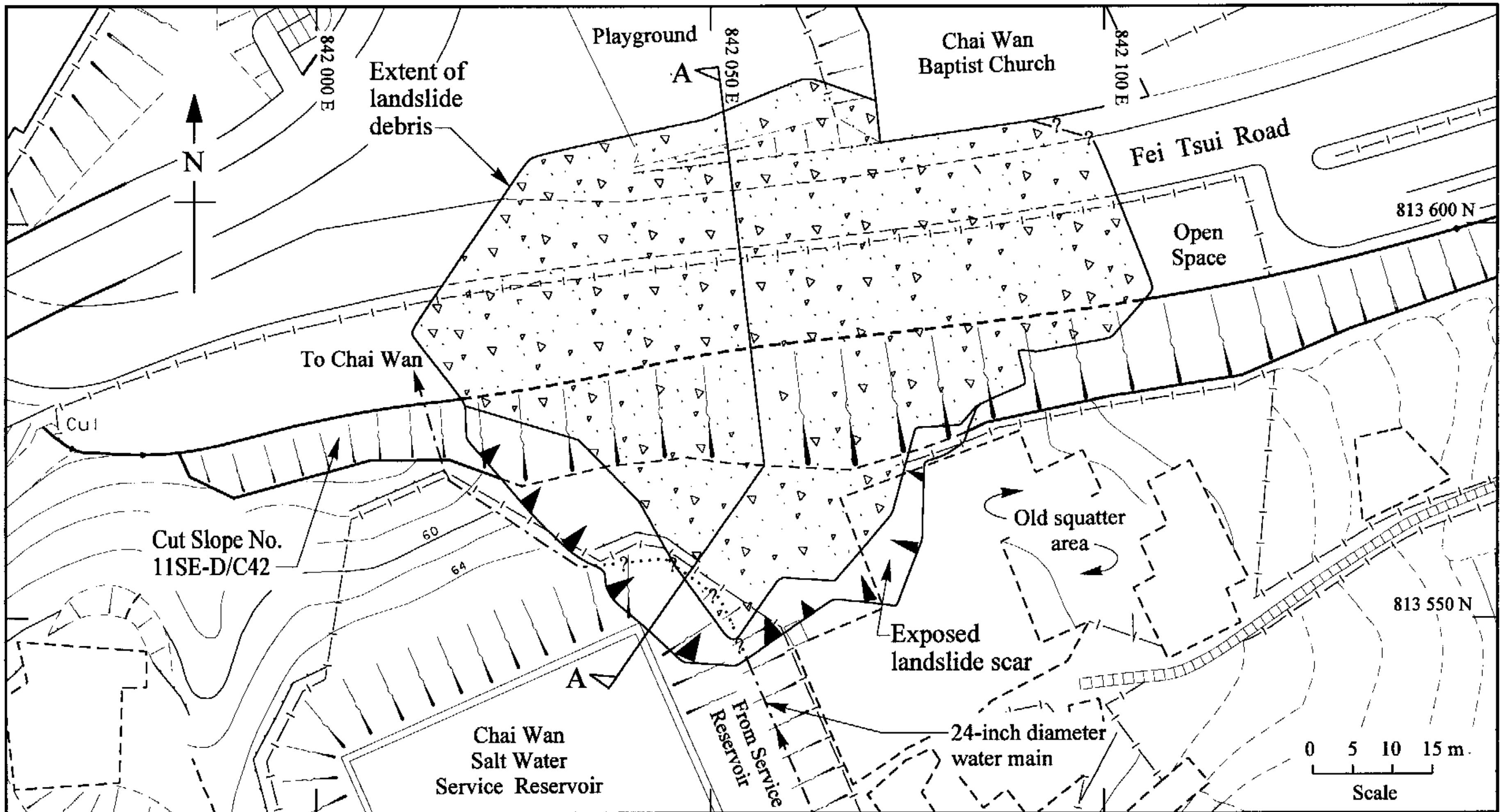
LIST OF FIGURES



Notes : (1) Base map is extracted from Survey Sheets No. 11-SE-18D & No. 11-SE-19C, dated 29 September 1992, scale 1 : 1000.

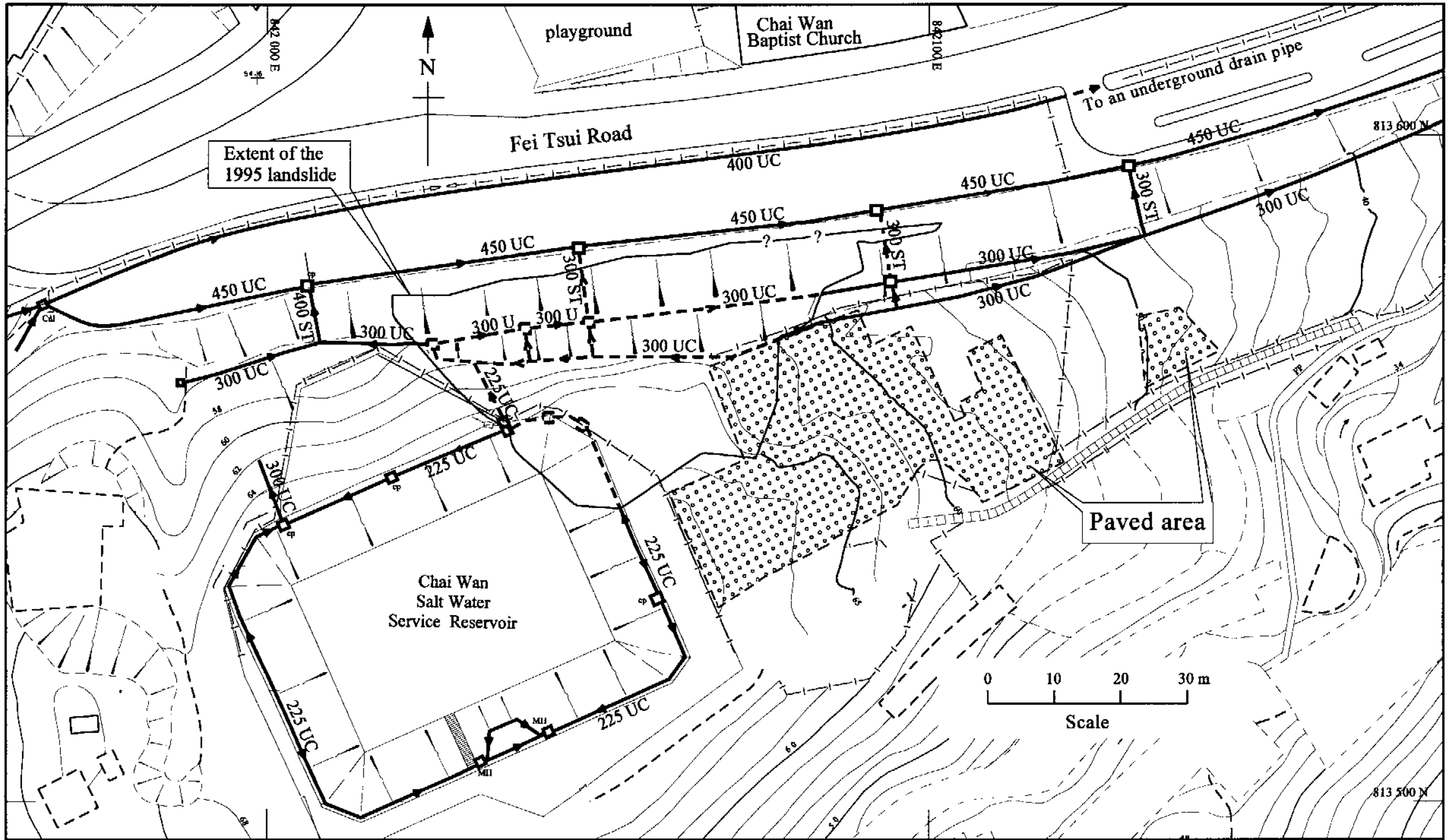
(2) Site boundary of Chai Wan Salt Water Service Reservoir is according to the location of the boundary fence of the service reservoir.

Figure 1 - Site Location Plan



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

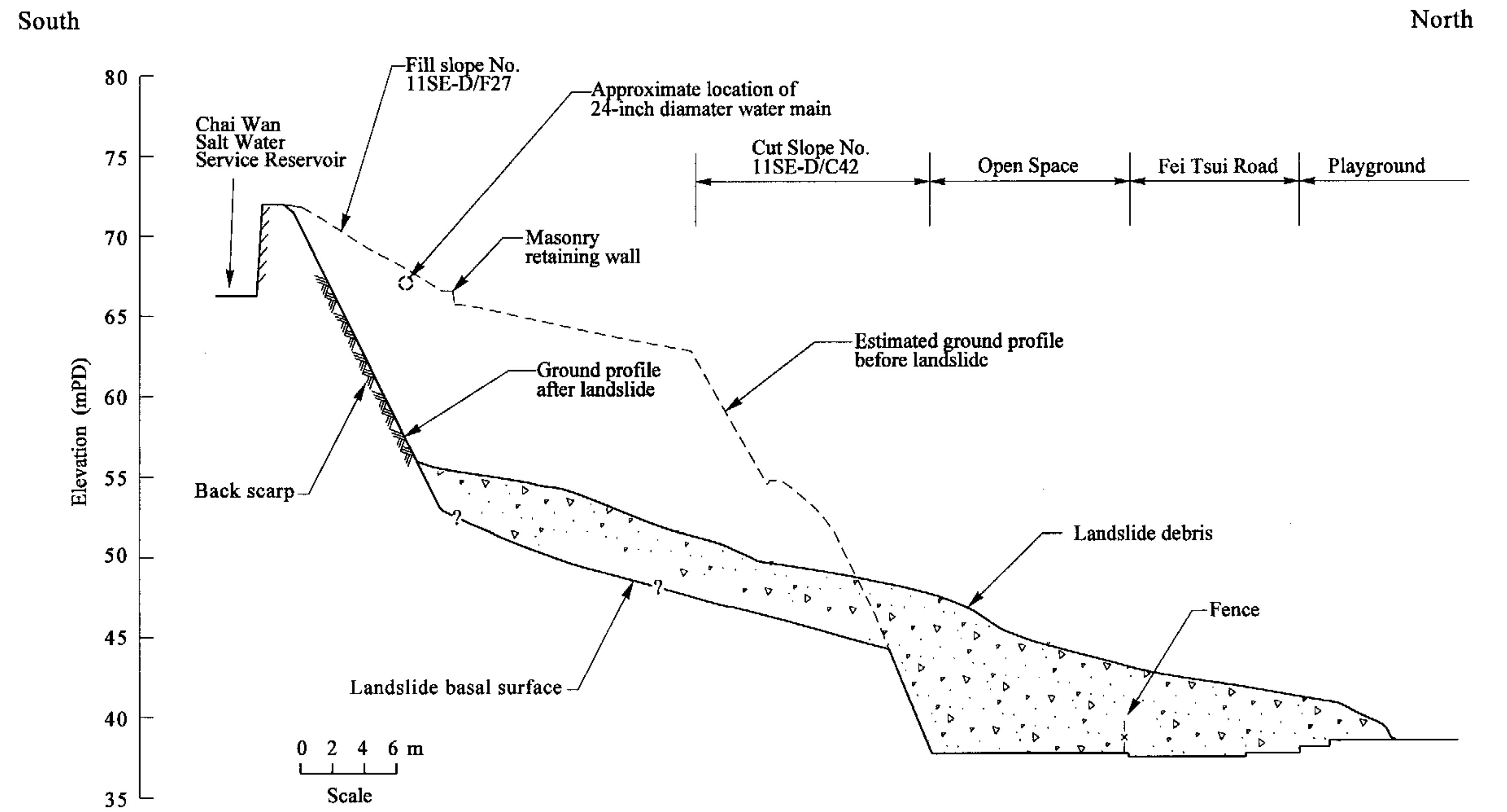
Figure 2 - Plan of the Landslide



Legend :

—➤—	Alignment of surface channel observed on site after the landslide	300 UC	300 mm U-channel
- -➤- -	Alignment of surface channel reconstructed from documentary records and field observations	300 ST	300 mm stepped channel

Figure 3 - Probable Drainage Layout of the Site



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

Figure 4 - Section A-A

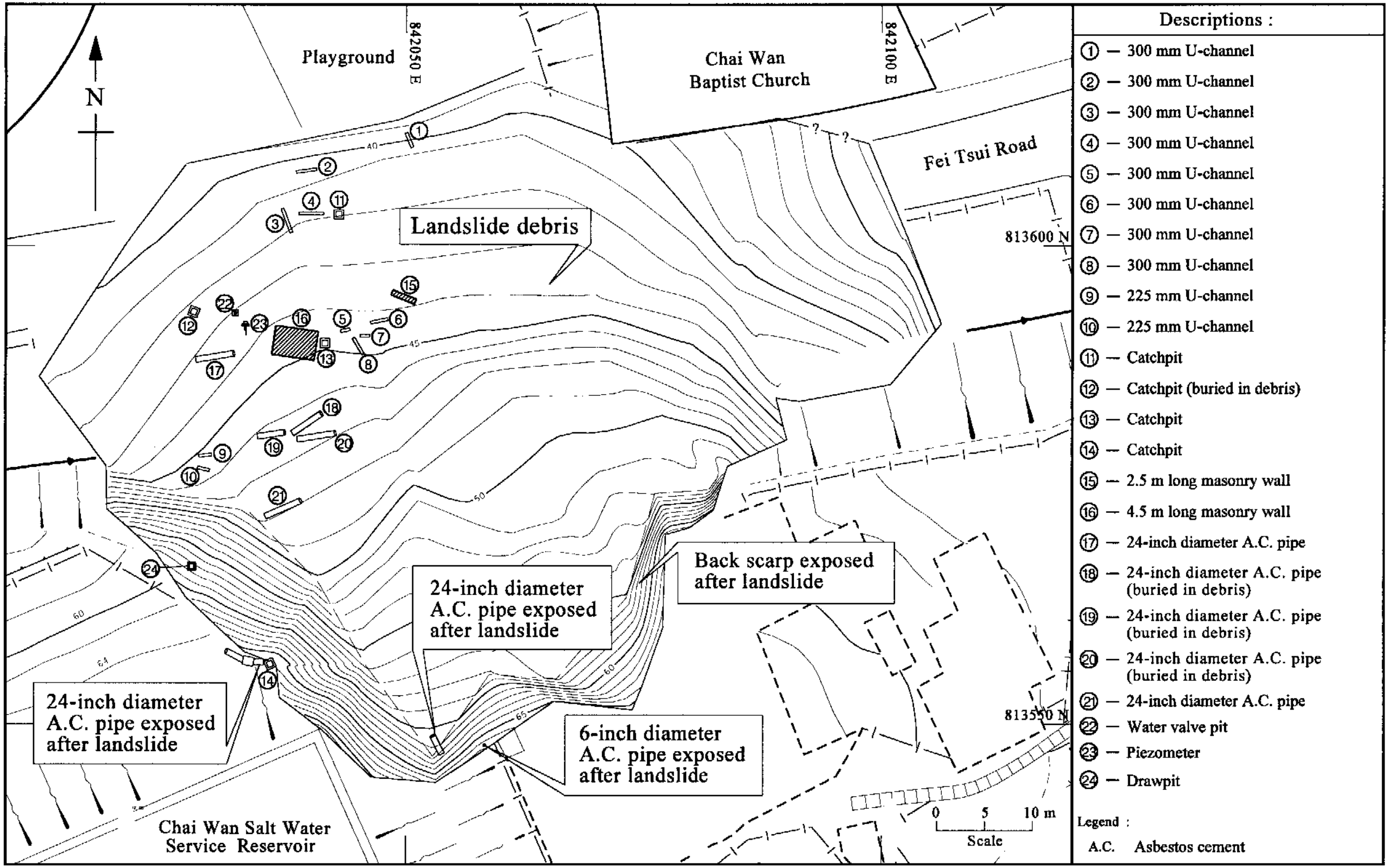


Figure 5 - Locations of Man-made Materials in the Landslide Debris

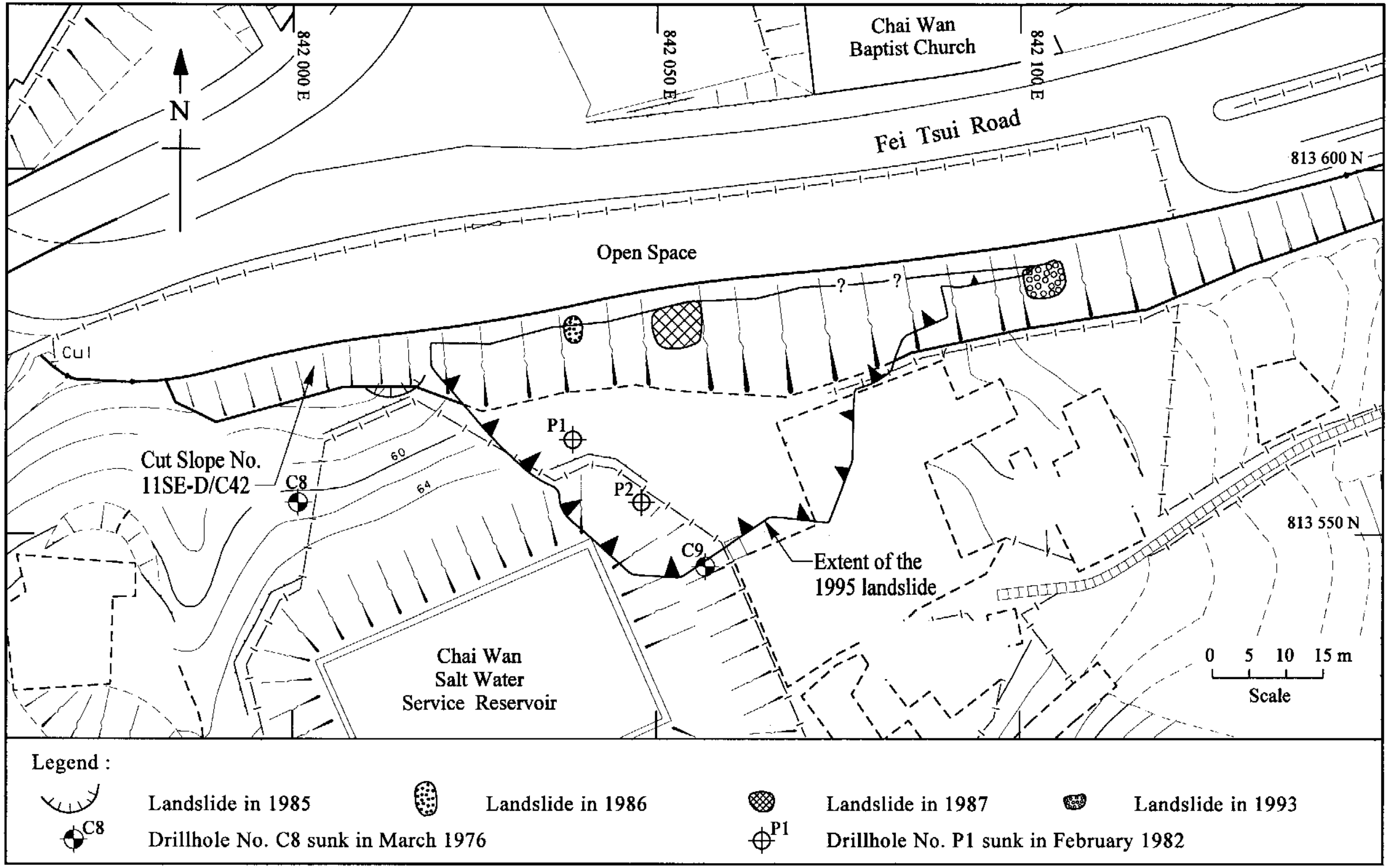
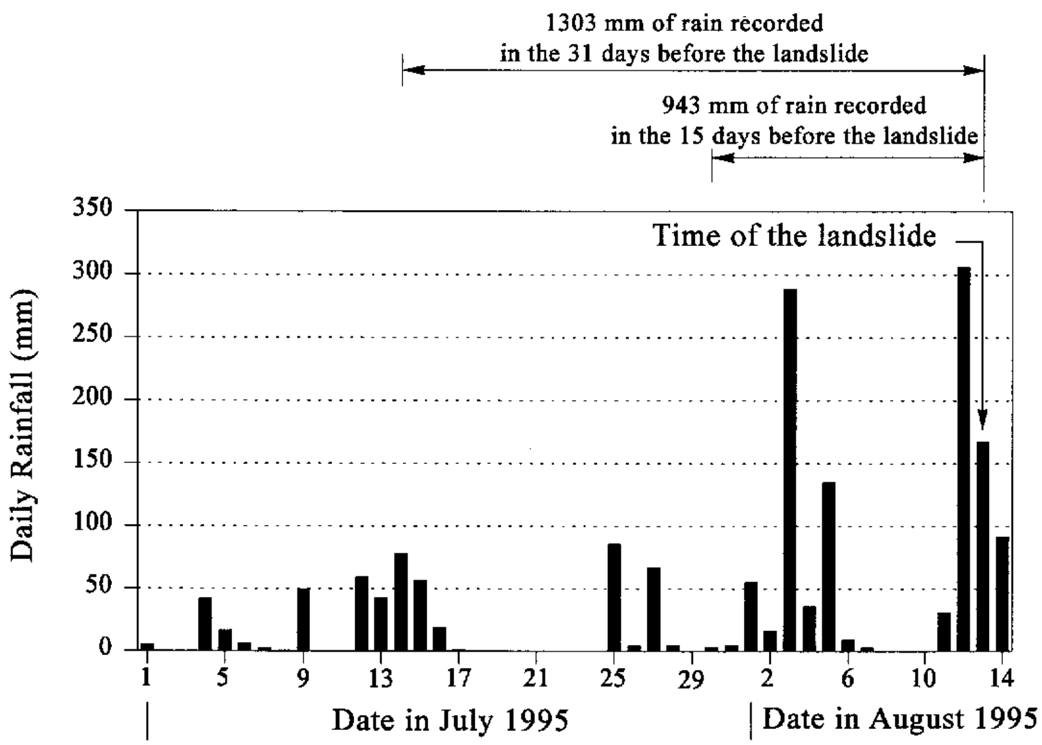
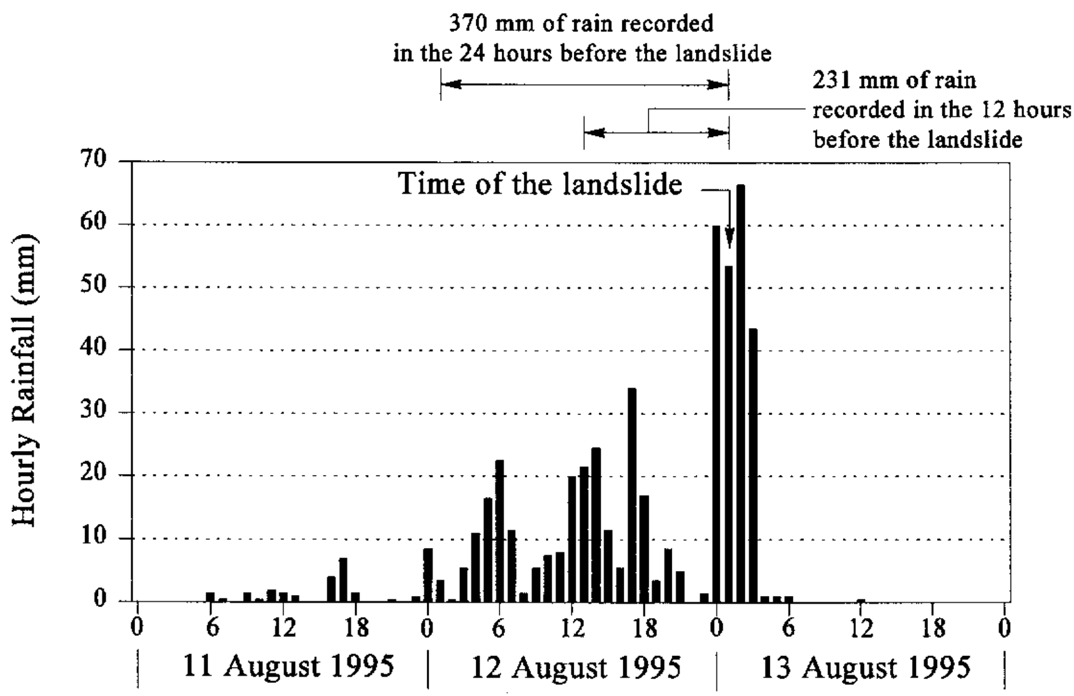


Figure 6 - Locations of Previous Landslides at Slope No. 11SE-D/C42



(a) Daily Rainfall Intensities in July and August 1995



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

Figure 7 - Rainfall Records of GEO Raingauge No. H14

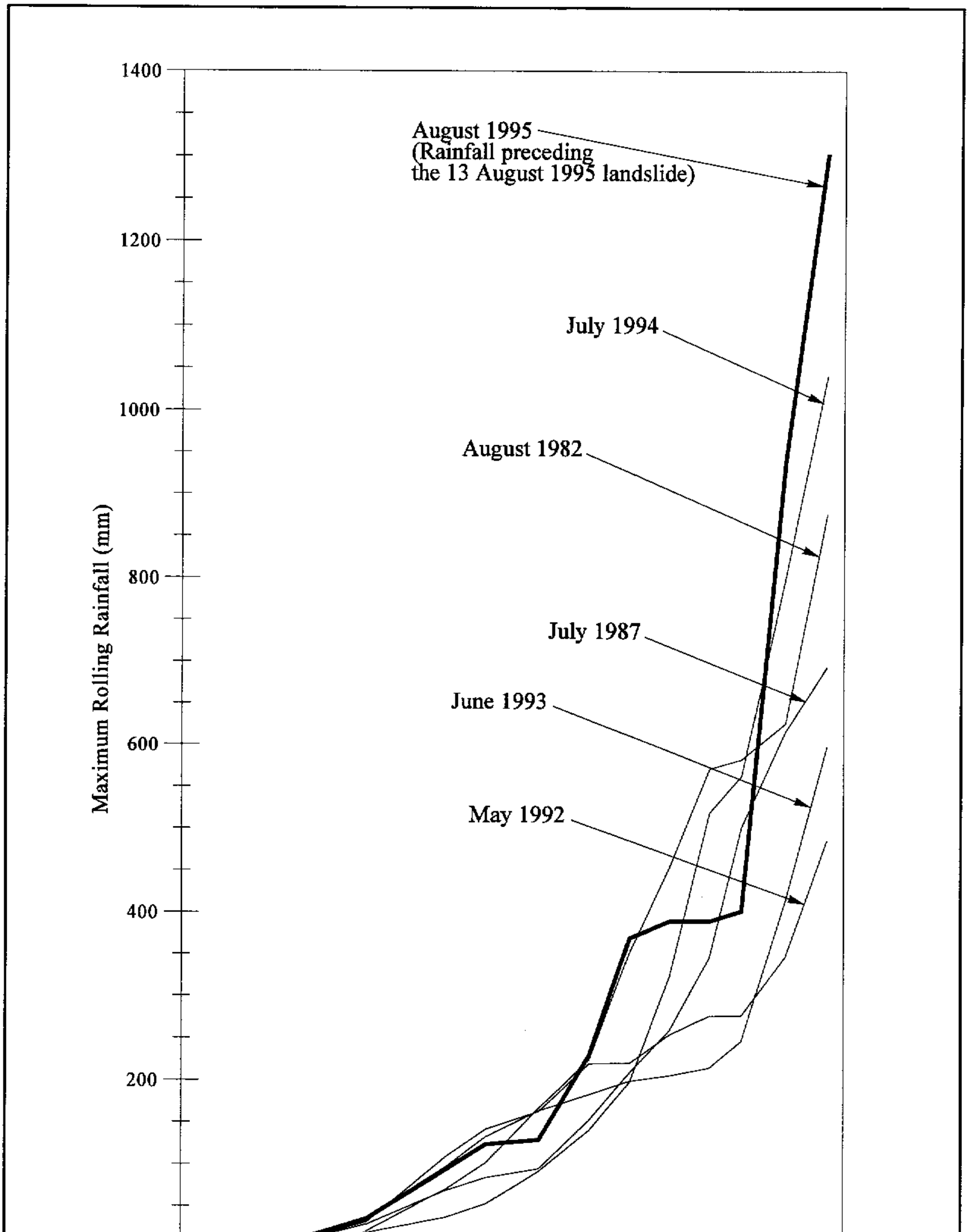


Figure 8 - Maximum Rolling Rainfalls at GEO Raingauge No. H14 for Major Rainstorms

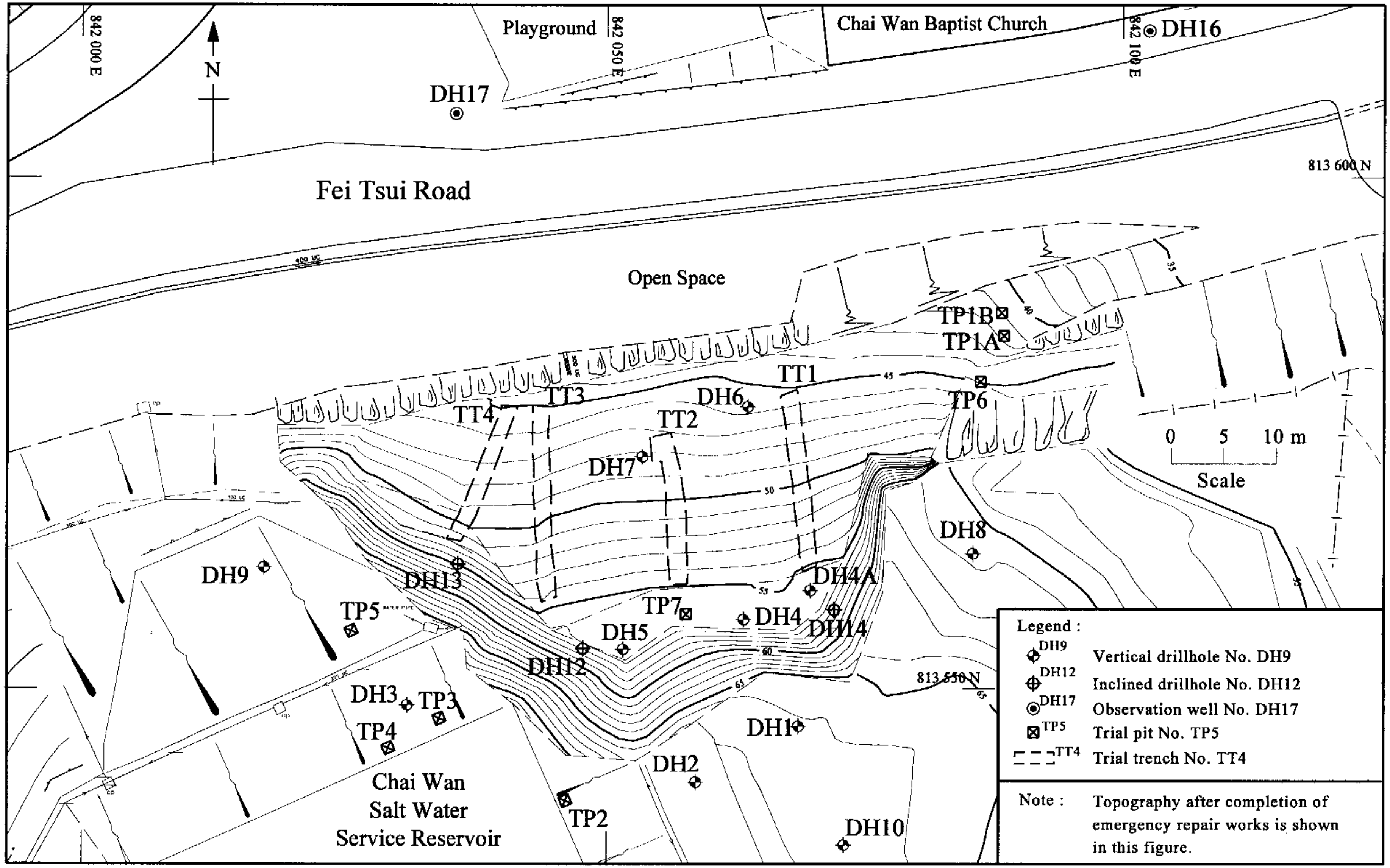


Figure 9 - Location Plan of Ground Investigation Works

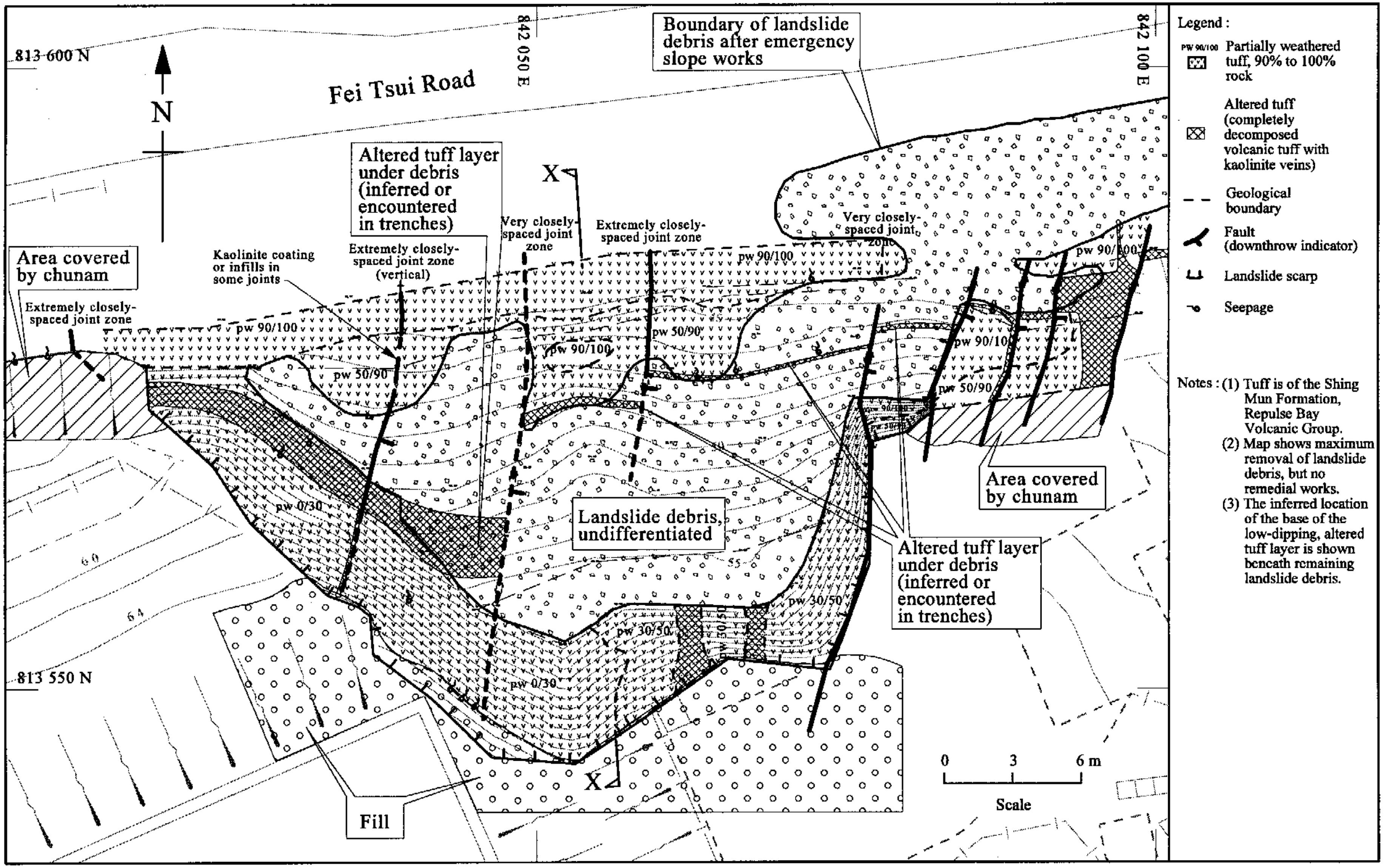
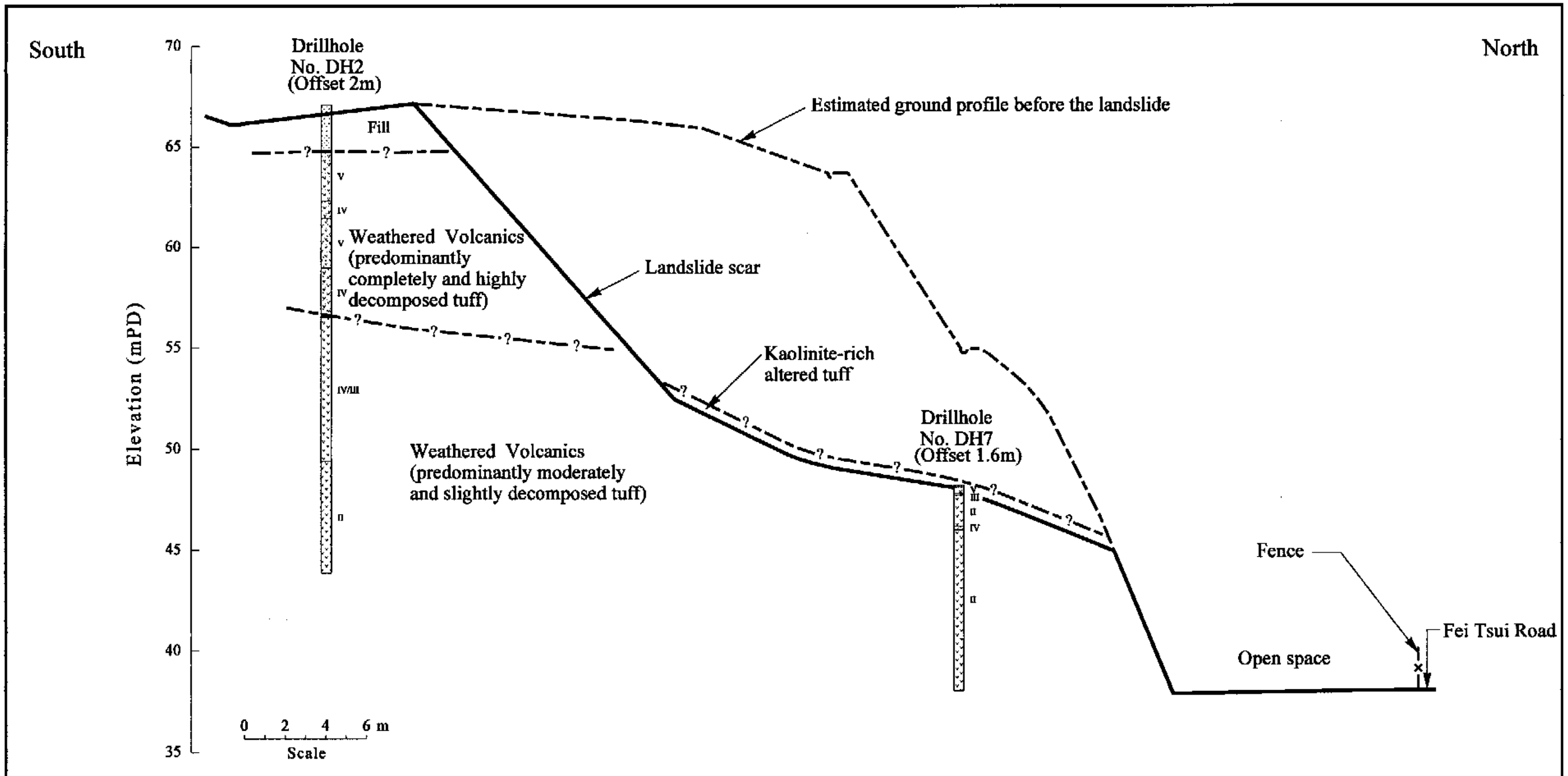


Figure 10 - Geological Map of the Landslide



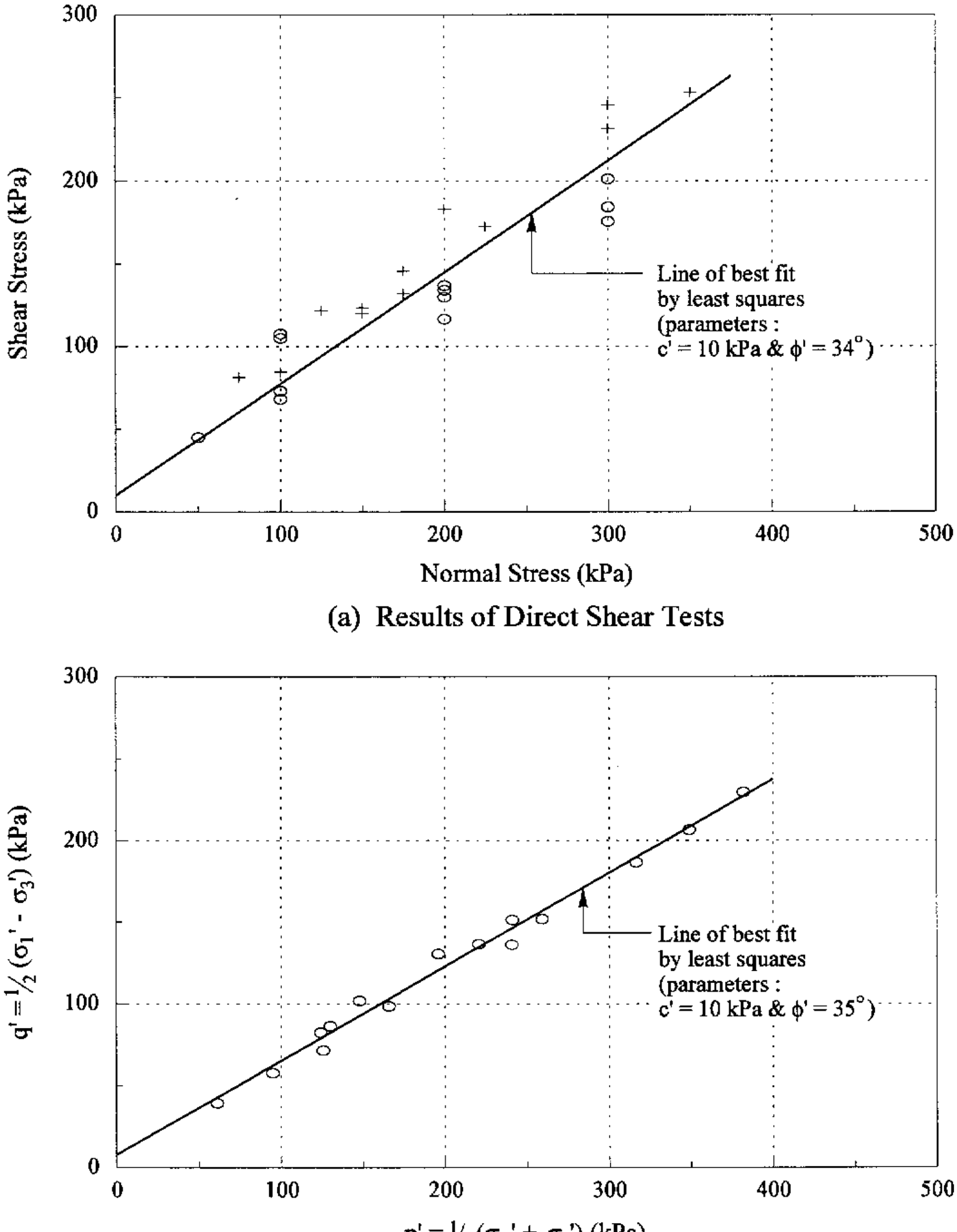
Legend :

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

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

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

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



(a) Results of Direct Shear Tests

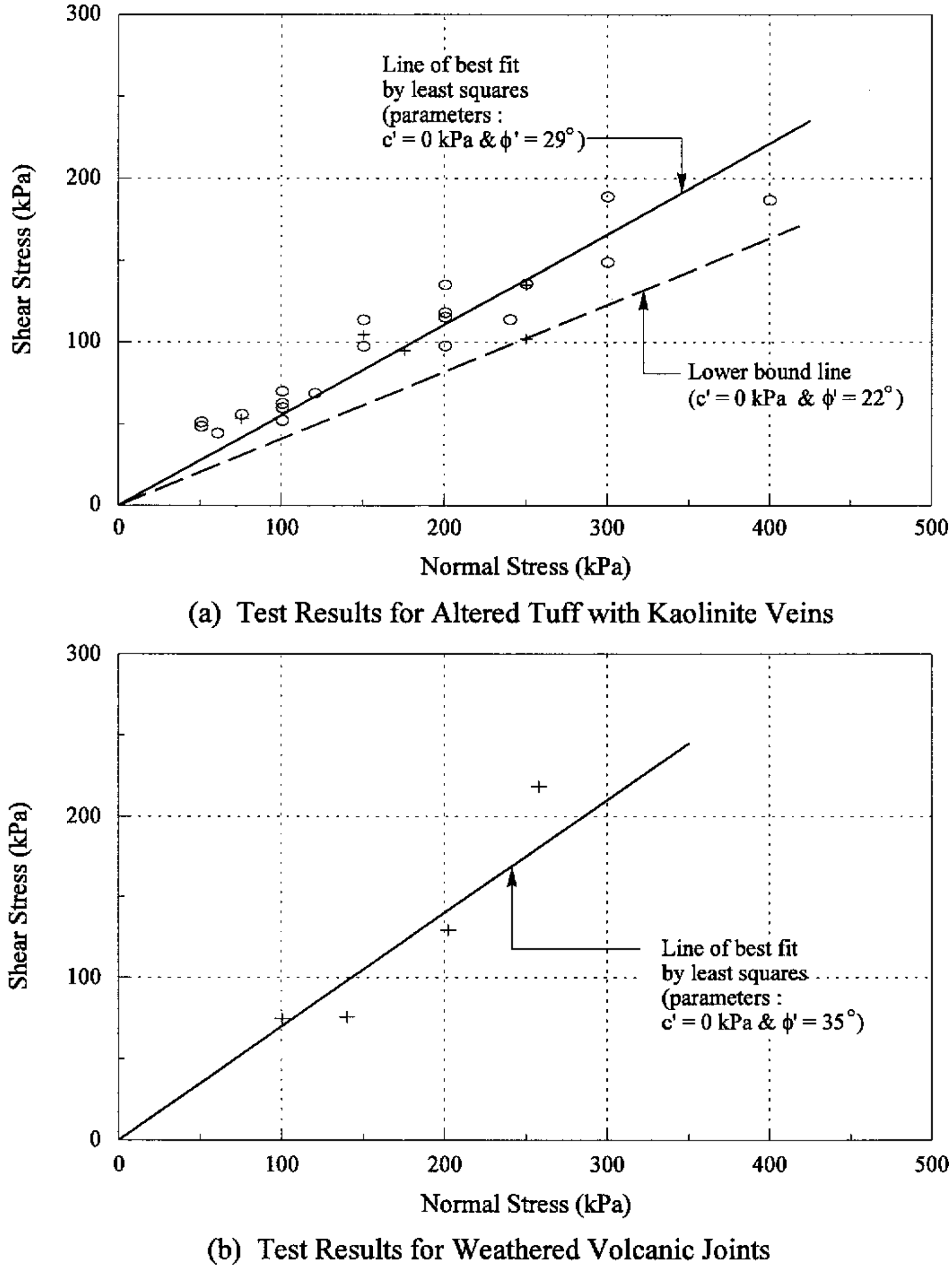
(b) Results of Consolidated Undrained Triaxial Compression Tests with Pore Water Pressure Measurement

Legend :

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

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

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



Legend :

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

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

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

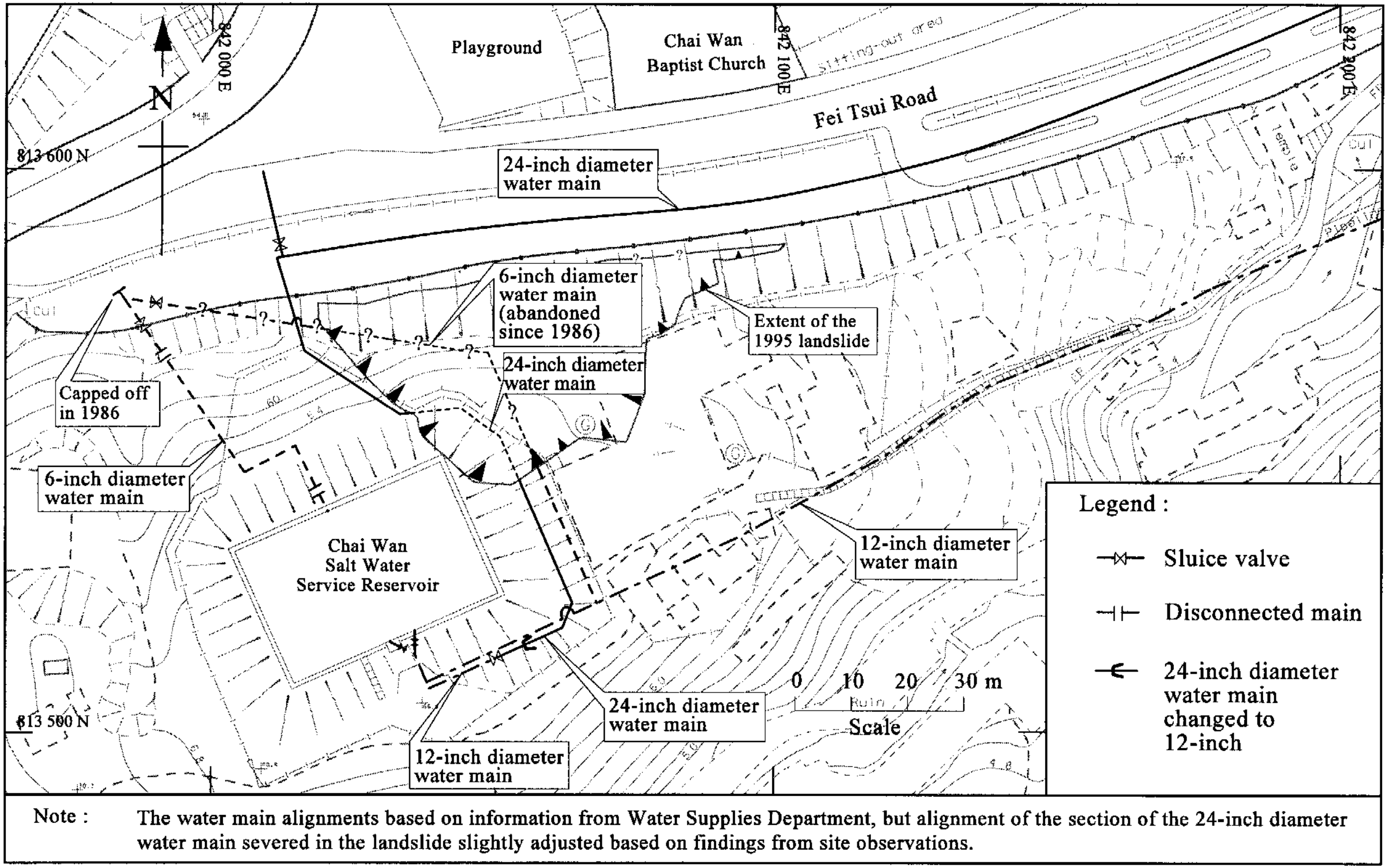
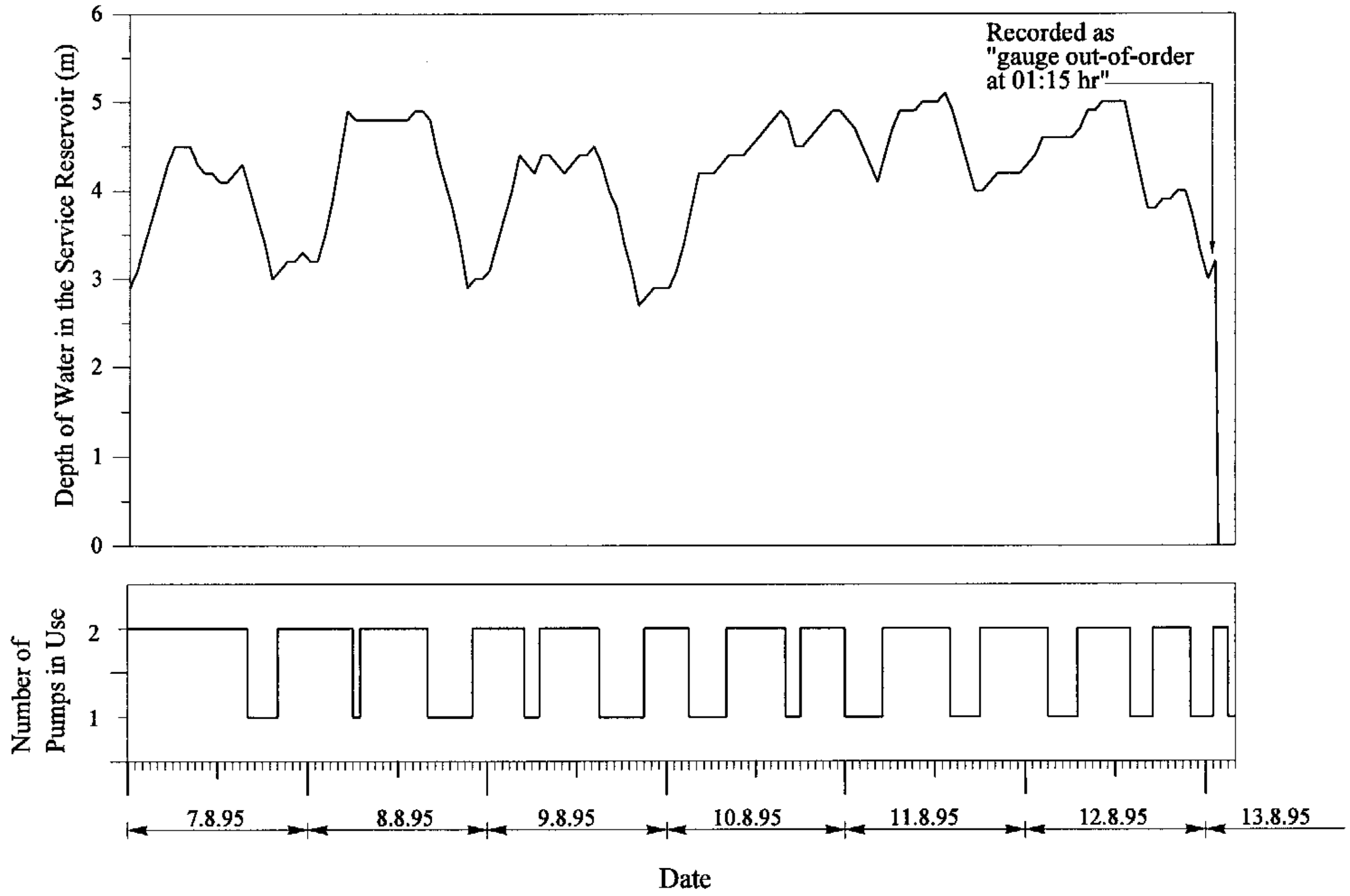


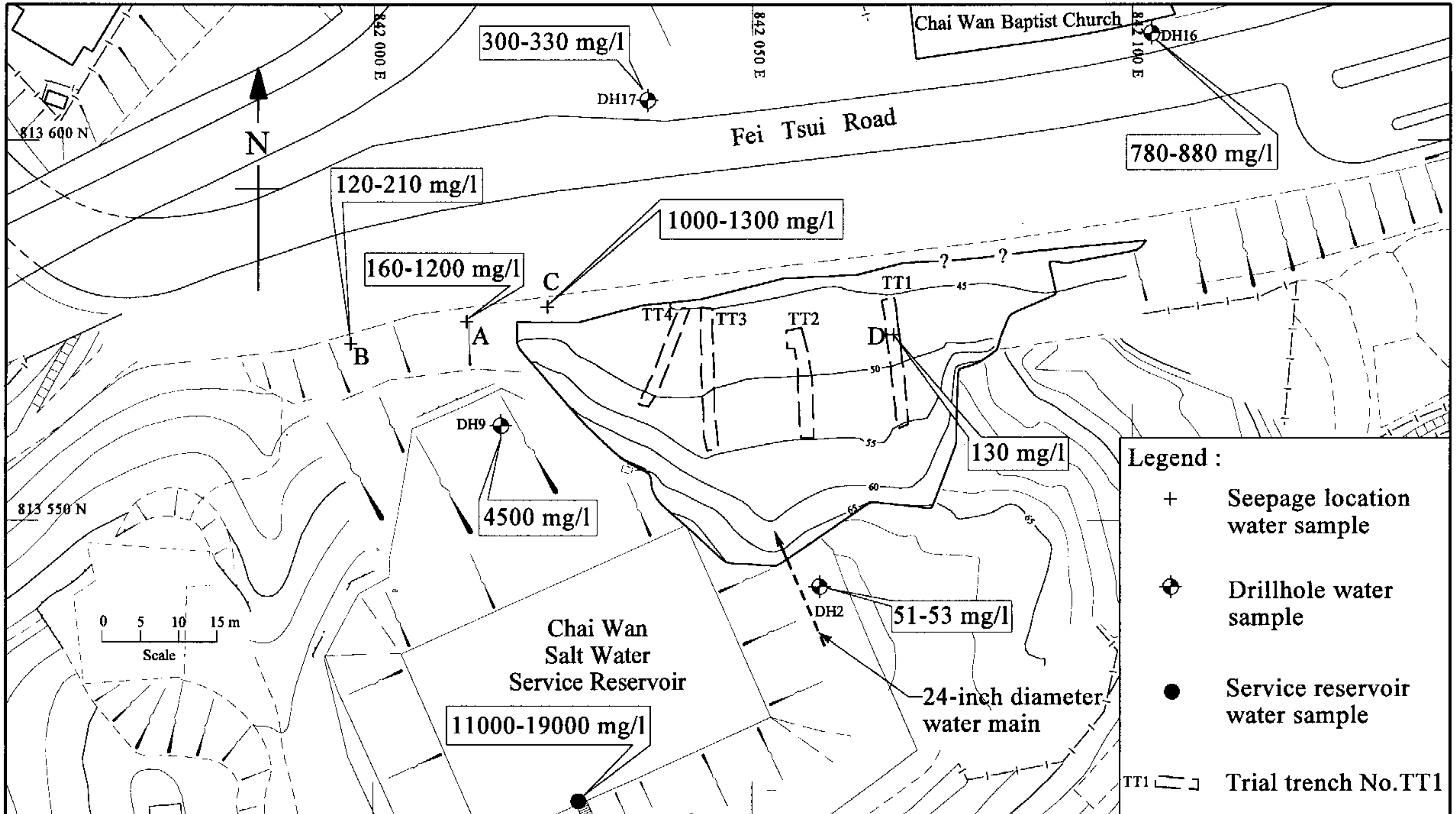
Figure 14 - Layout of Water Mains before the Landslide



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

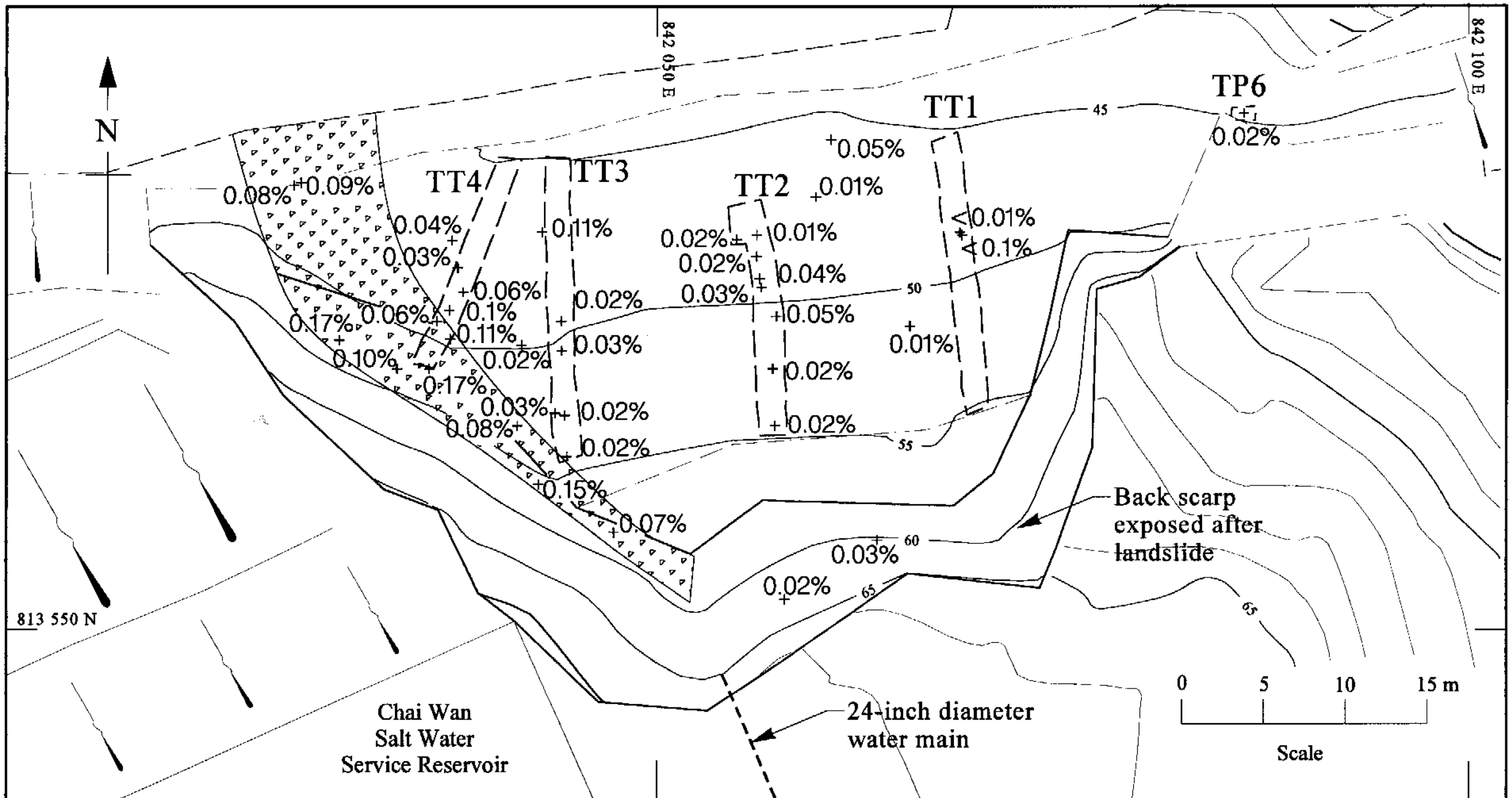
(2) This figure was based on information obtained from Water Supplies Department (1995).

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



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

Figure 16 - Results of Chloride Content Tests on Water Samples

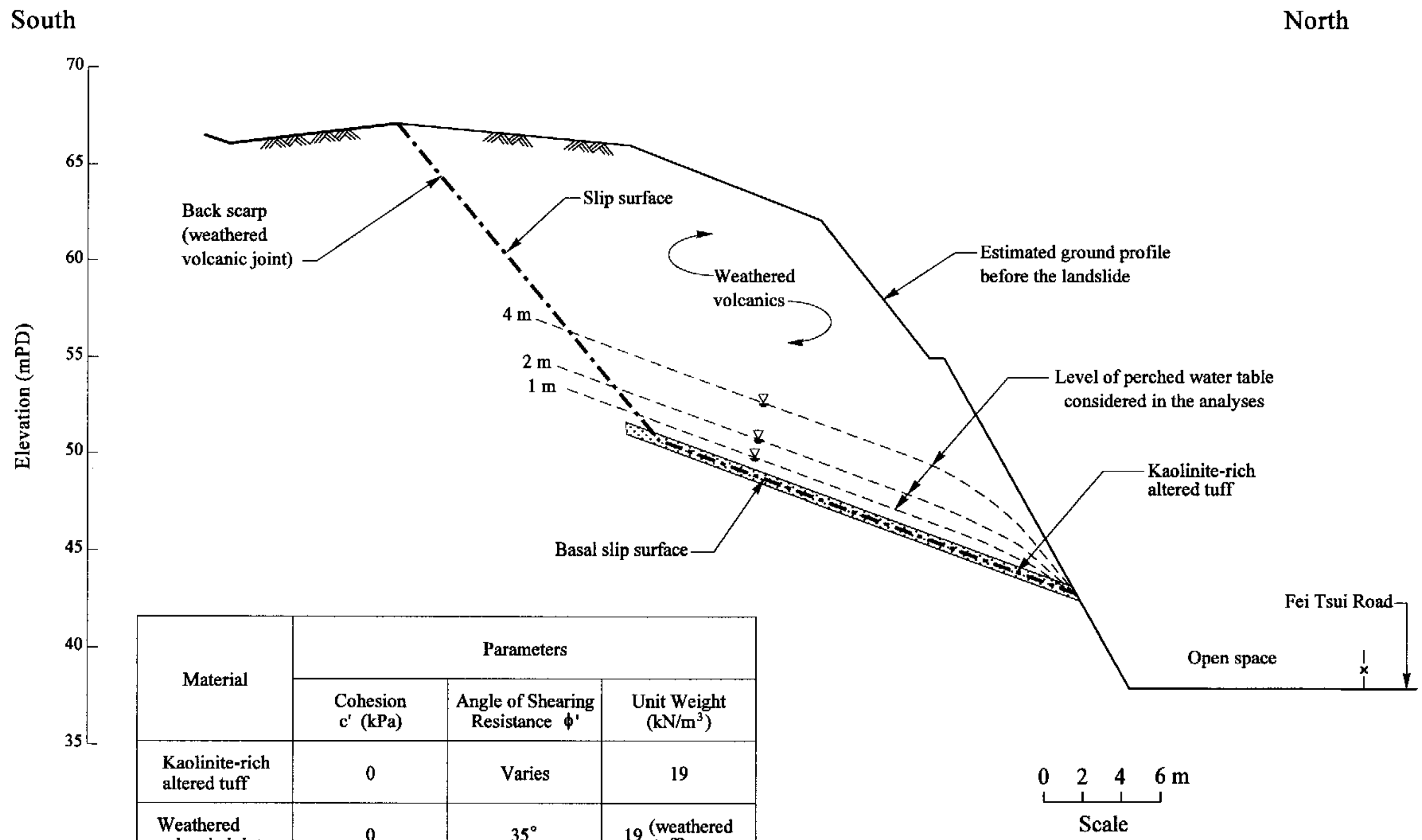


Legend :

+0.15%	Location of soil sample for chloride content test and the result shown in percentage by weight
TT1	Trial trench No.TT1
	Possible extent of area affected by salt water discharged from the 24-inch diameter water main after the landslide

Note : Topography after completion of emergency repair works is shown in this figure.

Figure 17 - Results of Chloride Content Tests on Soil Samples



Material	Parameters		
	Cohesion c' (kPa)	Angle of Shearing Resistance ϕ'	Unit Weight (kN/m^3)
Kaolinite-rich altered tuff	0	Varies	19
Weathered volcanic joint	0	35°	19 (weathered tuff)

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

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

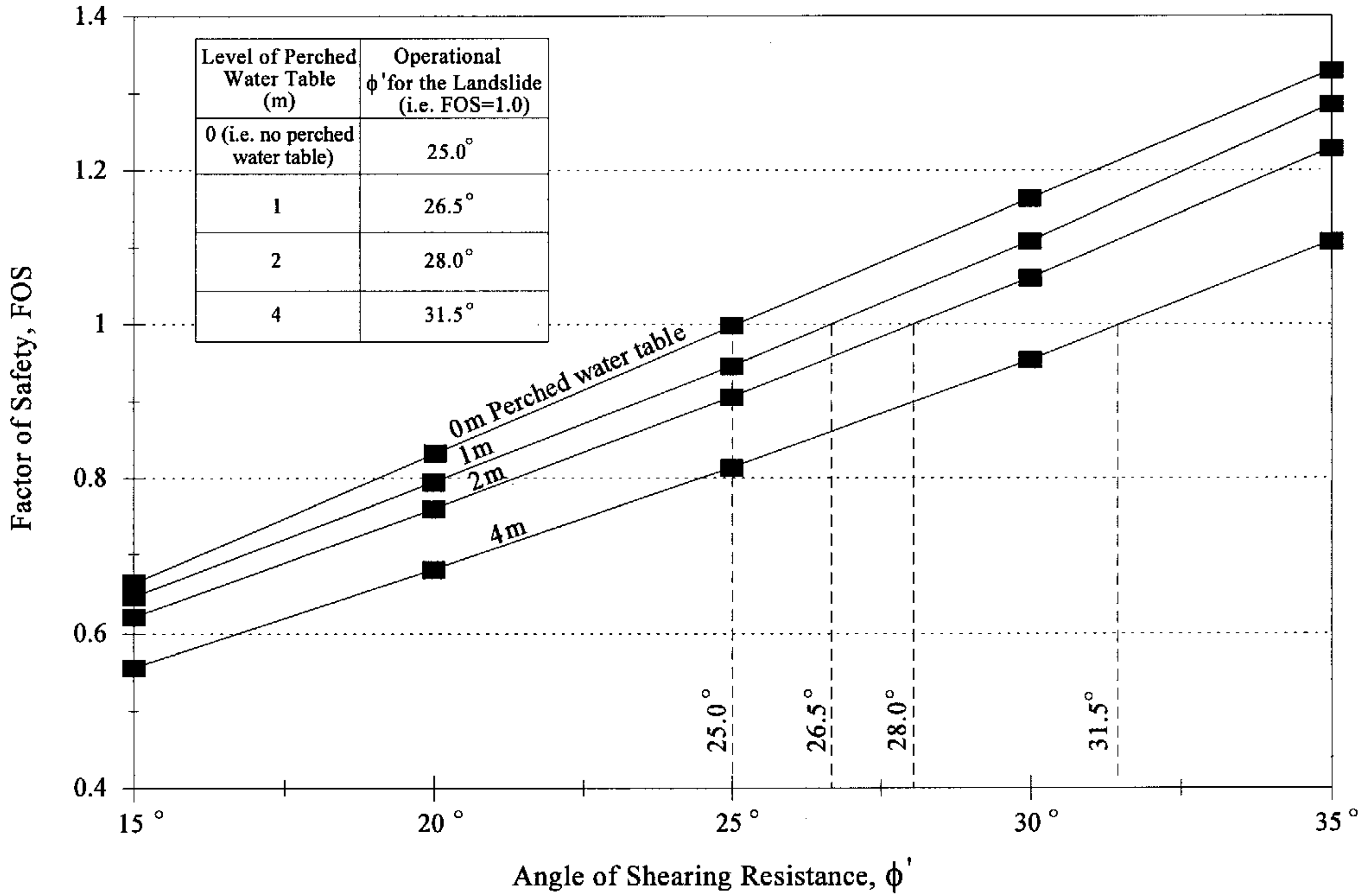
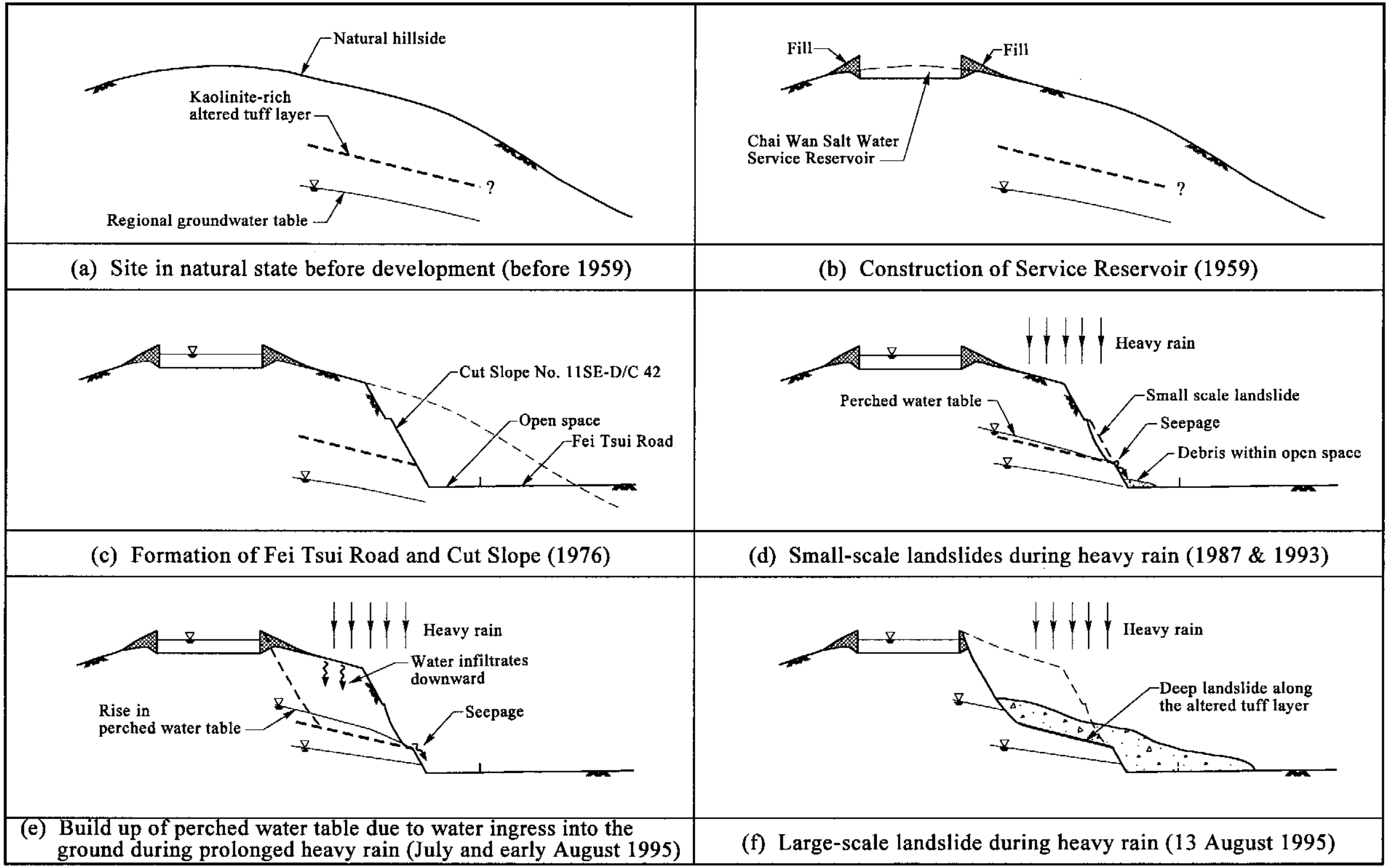


Figure 19 - Results of Theoretical Stability Analyses



(a) Site in natural state before development (before 1959)

(b) Construction of Service Reservoir (1959)

(c) Formation of Fei Tsui Road and Cut Slope (1976)

(d) Small-scale landslides during heavy rain (1987 & 1993)

(e) Build up of perched water table due to water ingress into the ground during prolonged heavy rain (July and early August 1995)

(f) Large-scale landslide during heavy rain (13 August 1995)

Figure 20 - Probable Sequence of Events

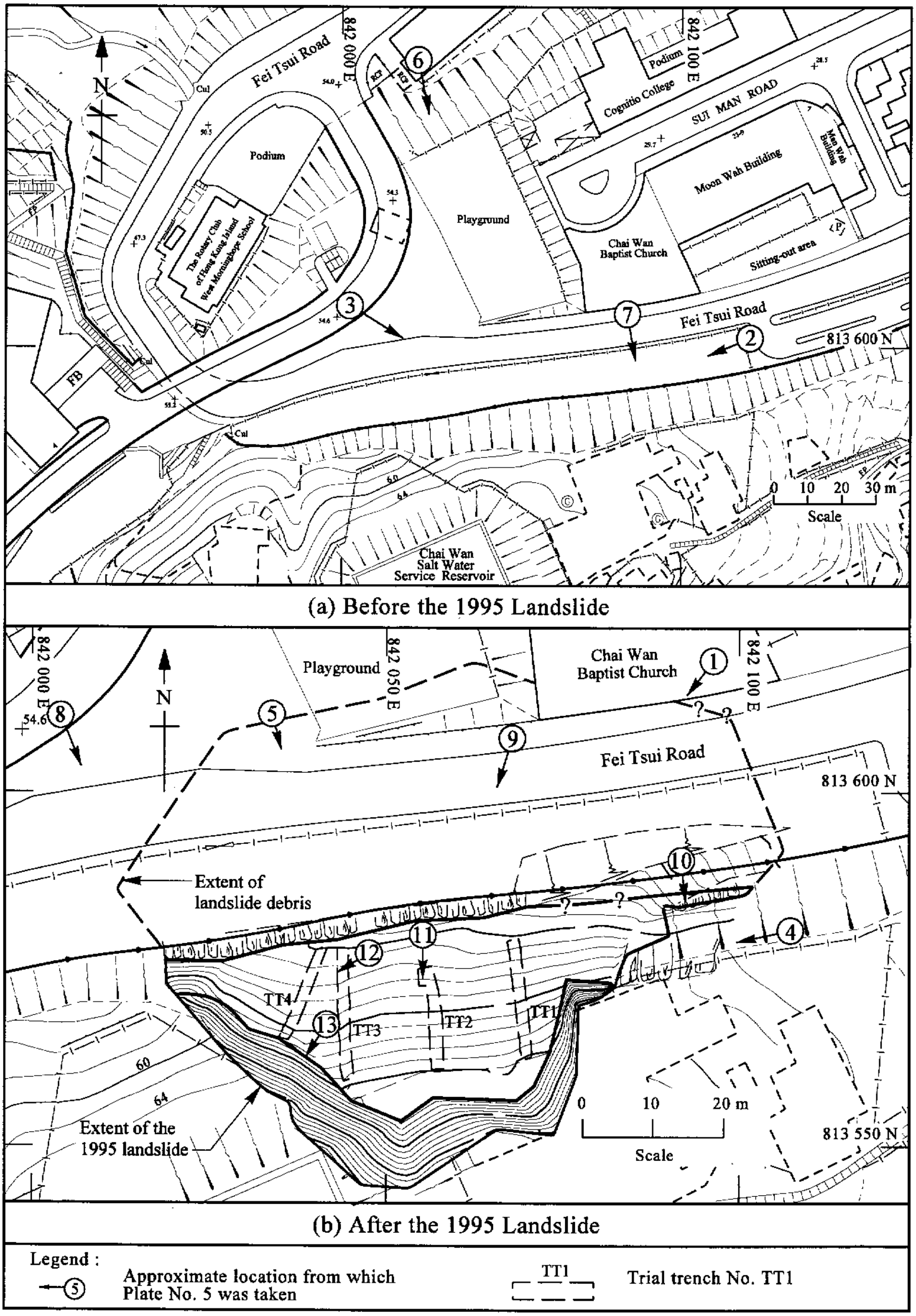


Figure 21 - Location Plan of Photographs Taken

LIST OF PLATES

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

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

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

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

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

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

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

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

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

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

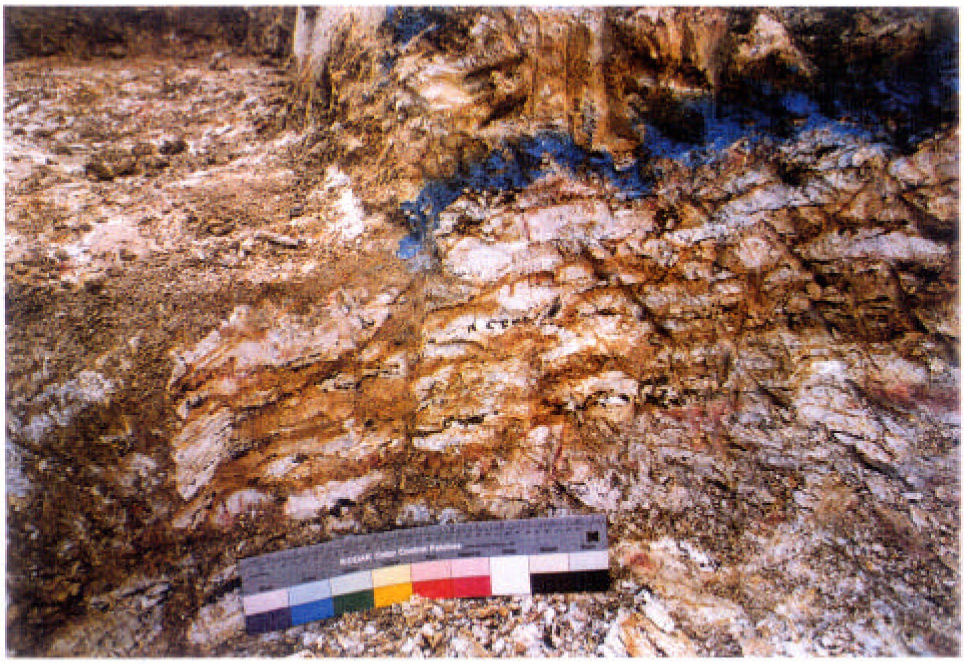

Plate 11 - Kaolinite-rich Altered Tuff at the Central Part of the Base of the Landslide (Photograph Taken on 20 October 1995. See Figure 21 for Location)

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

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

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APPENDIX A

SUMMARY OF SITE HISTORY

CONTENTS

A.1 SITE DEVELOPMENT

References

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

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

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

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

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

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

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

A.2 PREVIOUS ASSESSMENTS

A.2.1 Slope Registration

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

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

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

Binnie (1977)

B&P observed that there was "a 4' wide berm at mid-height" and "beneath the berm the exposed rock face is closely jointed fresh to slightly decomposed volcanic rock with a prominent horizontal weathered seam midway between the road and the berm (plate 8). Beneath the service reservoir we noted seepage along sub-horizontal joints". Plate 8 in B&P's report is given as Plate 2 in this landslide investigation report.

Para. No. 10.1, Binnie (1977)

Two investigation holes were drilled for the study. B&P noted that "one of which (C8) was between the service reservoir and the section of the slope where we noted the seepage, and the second (C9) within the reservoir grounds about 16 m back from the edge of the highest section of cut slope. Difficult access prevented this hole being sunk at the crest of the cut slope. Both holes penetrated at least 9 m into fresh to slightly decomposed rock".

Para. No. 10.2, Binnie (1977)

B&P described that "Mid way between the road level and the berm halfway up the cut face, there is a prominent horizontal weathered joint approximately 100 mm wide and at least 50 m long". B&P stated that, for hole C9, "Reduced core recovery between 20.7 m and 21.0 m is at about the same level as the horizontal joint on the cut slope and provides evidence of the possible persistence of this joint".

Paras. No. 10.4 & No. 10.6, Binnie (1977)

B&P reported that "Two piezometers were installed in each hole, one at the base of the hole and one at interface between moderately and slightly decomposed volcanics. The upper piezometer in each hole has remained dry". As for the lower piezometers, "Both maximum piezometric levels are within slightly decomposed to fresh volcanic rock".

Para. No. 10.7, Binnie (1977)

B&P stated that "There are two possible forms of instability. Beneath the mid-height berm in the cut slope there is the possibility of wedge or plane failure along rock joints or the spalling of loose rocks and overhangs. Above the berm there is a possibility of a soil-type failure in the decomposed volcanics. As the prominent weathered seam (paragraph 10.1 and 10.6) is horizontal we do not consider it to be a potential source of instability".

Para. No. 10.9, Binnie (1977)

The possibility of wedge or plane failures was assessed using a stereogram. It was concluded that "plane failure is unlikely" and "No large scale remedial measures are necessary".

Para. No. 10.11, Binnie (1977)

In the assessment of soil-type failures, B&P stated that "We have assumed that the descent of a wetting band would result in a perched water table forming above the interface between moderately and slightly decomposed volcanics". It was indicated in Drawing 13 of the report that "Strength parameters used:- a) grade IV/V dv $\phi' = 35°$, $c' = 10$kPa b) grade III/IV dv $\phi' = 40°$, $c' = 10$kPa".

Para. No. 10.7 & Drawing 13, Binnie (1977)

B&P found that "The factors of safety for the 1 in 1000 year rainfall condition are acceptable, however the minimum 1 in 10 year factor of safety is 1.09 which is below the acceptable value of 1.20", and "To increase F to 1.20 an average suction of 4.9 kPa is required along the critical slip surface (surface 3). This is a small value which we expect to exist but we recommend that the existence of suctions of this magnitude is checked using psychrometers".

Paras. No. 10.15 & No. 10.16, Binnie (1977)

In March 1978, Scott Wilson Kirkpatrick & Partners (SWKP) commented "on recommendations made by Binnie & Partners in their Phase IIC report on the Chai Wan area concerning measures necessary to improve the stability of the cut slope adjacent to the new access road to the Hing Wah Estate (Stage II)" on behalf of Government.

Para. No. 1 in SWKP's memo to Buildings Ordinance Office (BOO) dated 13.5.1978

SWKP stated that "We agree in general with Binnie & Partners' assessment of the dominant pattern of jointing in the slope and concur with their conclusion that neither plane nor wedge failures bounded by these steeply dipping joints are likely".

Para. No. 3iii) in SWKP's memo dated 13.5.78

SWKP commented that "Binnie & Partners have, however, mentioned in their study of the rock joints 'a prominent horizontal weathered joint approximately 100mm wide and at least 50m long' (§ 10.4). Although this joint does strike parallel to the cutting face (and therefore does daylight as a horizontal line) we would suggest that it is one of a family of locally persistent joints dipping at about 25° towards the road. Some of these joints are deeply weathered and show signs of strong seepage. We would recommend that the possibility of instability of blocks above this joint (i.e. local plane failure) should be considered. Depending on the results it may be necessary to design appropriate retaining measures for parts of the face. (There are indications that some blocks above this joint did fall as the cutting was being formed.)".

Para. No. 3iv) in SWKP's memo dated 13.5.78

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

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

SWKP noted that "Binnie & Partners have also recommended the installation of two psychrometers in a borehole behind this slope in order that the existence of pore suctions of the order of 5 kN/m^2 may be confirmed" and that SWKP "are prepared to accept Binnie & Partners' expectation that pore suctions of the required stabilising magnitude will exist at this site". Para. No. 4) in SWKP's memo dated 13.5.78

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

A.2.3 GCO Stage 1 Study

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

SWKP's memo to BOO dated 20.9.1979

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

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

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

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

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

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

Ove Arup's letter to AO dated 17.4.1980

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

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

Para. 6 of SWKP's memo to BOO dated 28.5.1980

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

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

HO's memo to GCO dated 23.9.1982

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

A.2.5 Landslip Preventive Measures Programme

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

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

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

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

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

Premchitt (1991) & Chan (1995)

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

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

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

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

GEO's memo to DLO/HKE dated 7.10.1994

A.4 REFERENCES

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

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

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

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

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

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