SECTION 2 : DETAILED STUDY OF THE LANDSLIDE AT TAI PO ROAD NEAR CHAK ON ESTATE ON 9 JUNE 1998

Fugro Scott Wilson Joint Venture

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FOREWORD

This report presents the findings of a detailed study of a landslide that occurred during heavy rain within an active construction site on 9 June 1998 at a newly-formed cut slope above Tai Po Road, near Chak On Estate. The total volume of the landslide is of the order of 1,400 m^3. The slope moved approximately 1 m before coming to rest whilst a large amount of material was eroded from the margins of the failed area and onto the slip road under construction below. Tai Po Road, immediately adjacent to the construction works, was closed to traffic as a precaution until temporary stabilisation works had been implemented. No one was injured in the landslide.

The key objectives of the detailed study were to document the facts about the landslide, present relevant background information and establish the probable causes of the failure. The scope of the study was generally limited to site reconnaissance, ground investigation, desk study and stability analysis. Recommendations for follow-up actions are reported separately.

The report was prepared as part of the 1998 Landslide Investigation Consultancy (LIC), for the Geotechncial Engineering Office (GEO), Civil Engineering Department (CED) under Agreement No. CE 74/97. It is one of a series of reports produced during the consultancy by Fugro Scott Wilson Joint Venture (FSW). The report was written by Mr J Hall and reviewed by Mr Y C Koo. The assistance of the GEO in the preparation of the report is gratefully acknowledged.

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1. INTRODUCTION

A landslide (GEO Incident Report No. MW98/6/11) occurred at registered slope No. 11NW-B/C60 within an active construction site in the late afternoon of 9 June near Chak On Estate. The section of Tai Po Road immediately below the slope was temporarily closed to traffic. No one was injured in the landslide.

Following the landslide, Fugro Scott Wilson Joint Venture (FSW), the 1998 Landslide Investigation Consultants, carried out a detailed study of the incident for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED) under Agreement CE 74/97. This is one of a series of reports produced during the consultancy by FSW.

The key objectives of the detailed study were to document the facts about the landslide, present relevant background information and establish the probable causes of the failure. Recommendations for follow-up actions are reported separately.

This report presents the findings of the detailed study which comprised the following key tasks:

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

(b) analysis of rainfall records,

(c) observations and measurements made at the landslide site,

(d) laboratory testing of soil and water samples collected from the landslide site, and

(e) theoretical analyses.

2. THE SITE

2.1 Site Description

The location of the landslide is shown in Figure 1 and photographs of the landslide area, taken on 27 December 1997 and the day before the failure, are shown in Plates 1 and 2 respectively. The failure affected two cut slopes, separated by a strip of the former (now disused) Tai Woh Ping Road. The lower vegetated cut slope has recently been substantially modified by cutting back the former slope No. 11NW-B/C60 in December 1997, to facilitate road widening works at its toe. The newly-formed slope comprises a 14 m-high, 40 degree slope, with a 1.5 m wide berm at about mid-height. A plan and a cross-section of the landslide are shown in Figures 2 and 3 respectively.

The upper cut slope is an older cutting formed for the original Tai Woh Ping Road (Figure 1), constructed in the late 1940's to early 1950's, and comprises an approximately 9 m high, 60° shotcreted cutting (Slope No. 11NW-B/C69). Behind the crest of the upper cutting is a 7 m deep mass concrete salt water reservoir, which was built in the late 1950's to early 1960's.

New surface water drainage channels have been installed as part of the recent earthworks, which include U-channels at the crest, berm and toe of the new cutting, linked by stepped channels via catchpits.

The new slope works were designed by Pypun/Arup Consultants (PAC), the consultants appointed by the Highways Department (HyD), and were constructed by Gammon Construction Limited between October 1994 and December 1997. The works are part of the improvements on Route 4, between Butterfly Valley and Nam Cheong Street, under Phase I of the Lung Cheung Road and Ching Cheung Road Improvements Project.

2.2 Underground Water-carrying Services

Based on the information provided by Water Supplies Department (WSD), Drainage Services Department (DSD) and HyD, together with FSW's field observations made after the landslide, the following underground water-carrying services were identified in the vicinity of the landslide site:

(a) Pressurised and Gravity Salt Water Mains

A 27-inch diameter and an 18-inch diameter water main from the Tai Woh Ping Salt Water Service Reservoir cross the old road some 40 m to the east of the landslide, which then descend the valley side, above ground level, to the level of the existing Tai Woh Ping Road (Figure 7). The 27-inch diameter main and an 18-inch diameter spur off the other 18-inch diameter main cross beneath Tai Po Road, some 40 m south of the landslide. A 10-inch diameter spur off the 18-inch main runs underground to the east, towards Beacon Hill S.W. Pumping Station, located approximately 50 m south-east of the site.

(b) Highway Drains

A 150 mm diameter concrete pipe was observed approximately 700 mm beneath the northern verge of the old road, within a gravel-filled trench. The pipe, which had been dislocated by the landslide, was exposed in the tension crack and lateral margin of the mass movement. This is described in more detail in Sections 4.3 and 8.

According to HyD's records, the 150 mm diameter pipe extends for a considerable distance (over 100 m) up-gradient from the site and its alignment is shown in Figure 7. Observations of the pipe run indicate that it was blocked with black-stained debris in the vicinity of the landslide.

Observations of the pipe run indicate that whilst the pipe exposed in the landslide was of a butt-jointed spun-concrete type, it changed to an older clayware type approximately 10 m upslope from the landslide.

2.3 Maintenance Responsibility

According to the consultant of the Lands Department for the "Systematic Identification of Maintenance Responsibility of Registered Slopes in the Territory" (SIMAR) project, HyD is responsible for the maintenance of the newly-formed cutting (slope No. 11NW-B/C60), whilst WSD is responsible for the maintenance of the older cutting (slope No. 11NW-B/C69) that borders the service reservoir.

3. SITE HISTORY AND PREVIOUS ASSESSMENTS

3.1 Site Development History and Past Landslides

The site development history has been determined from a review of the available aerial photographs and relevant documentation. Details of the aerial photograph interpretation (API) are given in Appendix A and only the salient points are described below.

The earliest photographs taken in 1949 show that Tai Po Road was the only completed highway in the area at this time, and its proposed junction with the original (now former) Tai Woh Ping Road to the southwest of the landslide site was under construction. The cutting for Tai Po Road was, at that time, some way south of the present cutting, bordering the then single carriageway of Tai Po Road. The former cutting was relatively steep and unsurfaced, with a berm at about mid-height. There is a photolineation traversing the eastern half of the cutting, which can be traced across the natural terrain in a northwesterly direction (Figure 4). The section of cut slope to the east of the photolineation, and above the berm, appears to have failed, and a distinct scar is visible in this area. Smaller, similar-shaped, scars are also present at the cutting crest to the west, and in a small cutting for a cut-off drain above the main scar. These latter scars may however represent locations of corestone removal rather than failures. A second photolineation can be distinguished running on an east–west axis, intercepting the cutting at its western extremity. Much rill erosion is apparent beneath this lineation at the western end of the cutting. The location of these features relative to the recent topography is shown in Figure 4.

By 1963, the original Tai Woh Ping Road had been completed, as was the service reservoir above. Tai Po Road had been duelled, and the lower cutting excavated back, with the construction of two equidistantly-spaced drainage channels across its face. No signs of failures were apparent, although erosion was ongoing at the toe of the cut. In 1969 the crest of the lower cutting was being developed as a sitting-out area. In the 1988 photographs, vague lineations are visible in the reservoir cut face, the locations of which appear to roughly correspond to the western release plane of the landslide of 9 June 1998. By 1996, the lower cutting had been excavated to provide an additional access lane at the toe, encroaching onto the sitting-out area alongside the old road at the crest. By November 1997, the lower slope had been trimmed back to its present profile, with earthworks still ongoing. The older upper cutting appears to have been just re-shotcreted at this time.

Based on GEO's landslide database and file records, no previous landslides are reported in the vicinity of the 1998 landslide site. However, as previously discussed, evidence of possible instability has been noted on the 1949 photographs, in the form of a scar in the eastern upper part of the cutting, prior to cutting back. This particular scar may be a relict failure, with the photolineation forming its western margin.

3.2 Previous Studies and Inspections

The existing upper cut slope flanking the reservoir and the original cut slope bordering Tai Po Road were registered as Nos. 11NW-B/C69 and 11NW-B/C60 respectively in the 1977/78 Catalogue of Slopes. The earliest file information at the GEO is an inspection report by Binnie & Partners in 1976, for both cut slopes. The report noted that both slopes showed no signs of distress or seepage.

In mid-1992, the GEO initiated a consultancy entitled "Systematic Inspection of Features in the Territory" (SIFT) to search systematically for slopes not included in the 1977/78 Catalogue of Slopes, based on studies of aerial photographs and limited site inspections. The two slopes were identified by the SIFT project in May 1995 and both were classed as Category C1 (i.e. "Assumed formed pre-1978 or illegally formed").

In 1994, the GEO commenced a consultancy entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST) to systematically update the 1977/78 Catalogue of Slopes and to prepare the New Catalogue of Slopes. The GEO's consultants for the SIRST project inspected the two slopes in October 1997. The inspection report stated that no signs of seepage or distress were noted, and no emergency action was considered necessary.

Slope No. 11NW-B/C69 was inspected in February 1995 under HyD's consultancy entitled "Roadside Slope Inventory and Inspections". The record of the inspection stated that the rigid cover was cracked, spalled and affected by vegetation and recommended that the surfacing be replaced, along with the removal of vegetation. It also stated that "Geotechnical conditions not known, slope considered to be very steep – stability assessment considered to be required".

The submission by PAC to the GEO for the detailed design of the new cutting was first made on 31 March 1998 after the cutting had been constructed. The design report, dated December 1994, recommended design strength parameters for completely decomposed granite (CDG) of $c' = 14$ kPa and $\phi' = 42°$ based on site-specific ground investigation (Section 5.2) and laboratory triaxial tests. The parameters were considered by the designer to be conservative, as they gave a minimum factor of safety for the existing cut slope of less than 1.0. The calculated factors of safety for the proposed new 40° cutting were between 1.37 and 2.27, assuming conventional circular failures of various sizes, although it appears a c' value of 5 kPa, not 14 kPa, was actually used in the analyses. The design groundwater level was assumed to be 2 m above rockhead (Section 5.4). An extract from the design report by PAC is given in Appendix B. The stability analyses did not include the upper cut slope above the old Tai Woh Ping Road. The topographic survey given in the submission also did not show the existing reservoir and the upper cut slope (flat ground was assumed at the crest of the proposed 40° cut slope). Surface water drainage design considered rainfall runoff from the slope itself and did not consider other surface water flow introduced onto the slope.

The GEO commented on the submission on 12 May 1998, requesting further information to support the submission, in particular location plans and a photograph of the site. Supplementary information was received by the GEO on 2 June 1998, but it was still incomplete. On 2 June 1998, the GEO further requested for the outstanding information, namely plans with existing and new slopes (with designations), retaining walls and bridges (with marked up pier numbers).

According to the Mainland West Division of the GEO, the outstanding information was required in order that the setting of the new cutting could be checked since it was suspected that the older cutting flanking the reservoir site could be close to the new cutting, which had not been shown, nor taken into account, in the PAC submission. Thus, the submission, which was made after completion of the new cutting, had not been accepted by the GEO when the landslide occurred.

4. DESCRIPTION OF THE LANDSLIDE

4.1 Observations Made Prior to the Landslide

The sequence of events leading to the landslide during heavy rain on 9 June 1998 has been re-constructed from accounts of witnesses and summarised below in chronological order:

(a) The new cutting, the construction of which commenced in October 1994, was inspected on completion in December 1997 by the Assistant Resident Engineer for the project, who observed no signs of distress or seepage.

(b) The Resident Engineer for the project advised that flooding of the old Tai Woh Ping Road was not unusual at times of “heavy rain”. A photograph (Plate 2) taken on 8 June 1998 (the day before the landslide) showed that erosion had taken place.

(c) At 16:50 hours on 9 June 1998, the Assistant Inspector of Works for HyD noticed soil being washed onto Tai Po Road from the new cutting. On closer inspection, it was apparent that surface water channelled down the cracks in the vicinity of the eastern margin of the landslide site and that the shotcrete in the upper cutting had cracked.

4.2 Time of the Landslide

At 17:15 hours on 9 June 1998, the contractor’s Site Agent inspected the slopes in view of the heavy rain. He observed flooding of the old Tai Woh Ping Road at the crest of the new cutting and a torrent of water cascading down the slope “like a yellow river” in the vicinity of the eastern margin of the landslide. It was noted that movement of both the lower and upper cut slopes had already taken place.

Based on the above observations, slope deformation and failure probably occurred some time before 16:50 hours on 9 June 1998.

4.3 Field Mapping and Observations after the Landslide

The extent and profile of the landslide and resultant debris were determined by topographic surveys carried out by HyD, under the direction of their consultant for the Lung Cheung Road and Ching Cheung Road Improvements Project. The extent of the landslide is shown in Figures 1 and 2, and a cross-section through the landslide is given in Figure 3. General and close-up views of the landslide site are shown in Plates 3 to 12.

The landslide appears to consist of essentially two discrete masses, which have been subjected to different amounts of movement, separated by a major tension crack at the back edge of the old Tai Woh Ping Road (Figure 3 and Plate 5). Another major tension crack was located near the crest of the upper cutting above Tai Woh Ping Road (Figure 3). The landslide comprised a main, lower failure within the new cutting (No. 11NW-B/C60), and an upper failure with comparatively less displacement, within slope No. 11NW-B/C69. The main failure mass moved approximately 1 m in a south-westerly direction essentially in a translational mode along a shallow dipping basal failure plane, whilst the upper failure mass moved approximately 0.5 m, along a steeper, and possibly curved, failure surface, essentially in a rotational mode. The basal failure plane was encountered in six of the eight trial pits dug in August to October 1998. A detailed description of the nature of the basal failure plane is given in Section 5.2.

The landslide resulted in the movement of material that was trapezoidal in plan, about 17 m high and 37 m wide at its base, tapering to about 10 m wide at the crest and wedge-shaped in section, with an average depth of about 4 m. The total volume of the landslide is estimated to be about 1,400 m^3.

The landslide mass remained relatively intact, apart from the shear/tension cracks dividing it into separate blocks, and did not break up significantly. However, extensive signs of distress, in the form of significant cracking and dislocation of berm slabs, were observed (Plate 6).

Although the displaced material remained largely on the slopes, a large amount of sand and fine gravel was washed out following the failure. This was evidenced by two deep fan concentrations of deposits at the base of the slope, immediately beneath the margins of the landslide. The eastern margin in particular was eroded to a 2 to 3 m wide channel, where weathered granite was washed out from around corestones (Plate 7).

The western margin was delineated by a planar, persistent, subvertical discontinuity running the whole height of the landslide (Figure 2 and Plates 8 and 9). This relict discontinuity remained relatively tight (except near the ground surface, where run-off/ seepages had eroded a narrow shallow channel), at least for the lower half of its length, suggesting that movement had probably been sub-parallel to the discontinuity. The strike of the discontinuity is about 40° west of the slope-facing direction. The general orientation of the discontinuity was measured as 85°/330° (dip/dip direction), whereas the slope is typically orientated at 40°/200°, implying a direction of movement oblique to the direction of the slope face. The discontinuity, extending outside the landslide area down to the slope toe, was seen to be heavily iron stained and infilled with manganese oxide. The manganese infilling had been polished to a slightly undulating planar surface where movement had taken place.

The eastern margin was less well defined as a linear feature, partly as a result of the extensive post-failure erosion (Plate 7), and partly as a result of the presence of two joint sets that acted as release planes. One set was measured as 80°-90°/080° (dip/dip direction) and the other as 80°-90°/180°. This margin was more open than the western relict discontinuity, as a result of the main soil mass sliding obliquely away from these release planes (Plate 10).

The only underground water-carrying services of relevance to the landslide was the 150 mm concrete butt-jointed pipe, in gravel surround, immediately beneath the northern verge of the former Tai Woh Ping Road at the toe of the upper cutting (Figure 3). A photograph of this pipe is shown in Plate 11. It can be seen that the exposed end of the dislocated pipe is at the junction of the main tension crack and the eastern margin of the landslide. Although rain was continuing at the time of the inspection by FSW on 10 June 1998, some 20 hours after the landslide, only a small amount of seepage was observed coming out of the exposed end of the debris-filled pipe.

The upper shotcreted cutting above the former Tai Woh Ping Road showed significant cracking above the projected margins of the failed cutting below. A major tension crack with a 500 mm (approximately) vertical displacement had developed along the fence line bordering the reservoir compound. A second nominal fracture (25 mm approximately) was noted further back along a previously slightly open (as indicated by grass root development) construction joint between a U-channel and a concrete pavement running around the perimeter of the reservoir (Figures 2 and 3).

The former tarmac road surface forming the crest of the new cutting was severely cracked and pot-holes were noted in places, particularly along wheel ruts (Plate 12). The nature of the damage suggests that these may be a result of excessive trafficking. At the time of FSW's inspection on the morning of 10 June 1998, workers were seen to be laying a concrete screed over part of the former carriageway surface, immediately upslope of the landslide, which might have been in a particularly poor condition.

4.4 Consequences of the Landslide

As a precautionary measure, WSD drained the saltwater service reservoir immediately behind the landslide site during the night of 9 June 1998, at the recommendation of the GEO.

In view of the possibility of further slope failures which could affect Tai Po Road below, HyD and their consultant decided to close the highway until completion of the temporary stabilisation works. These works comprised the placement of rockfill at the toe of the lower cut slope and trimming back of the upper cut slope, together with surface drainage provisions to intercept and dispose surface water flowing along the old Tai Woh Ping Road.

The works were carried out in the week following the landslide and Tai Po Road was re-opened on 17 June 1998.

Permanent remedial works, designed by PAC, were undertaken during the period between August and December 1998, and comprised the following:

(a) trimming of the lower cut slope back to its original design profile,

(b) excavation of the margins of the landslide and replacement with mass concrete,

(c) installation of 18 m long soil nails (at 2.5 m centres) through the slipped mass of the lower cut slope,

(d) installation of 15 m long raking drains (at 3 m centres) and provision of shotcrete cover (with weepholes) to the lower cut slope,

(e) construction of new cut-off drains to intercept surface water flowing down the old Tai Woh Ping Road,

(f) reinstatement of the surface water drainage system on the lower cut slope, and

(g) trimming of the upper cut slope to 45° and provision of a new crest channel and shotcrete cover (with weepholes) to the slope surface.

5. SUBSURFACE CONDITIONS OF THE SITE

5.1 General

The subsurface conditions at the site were determined using information from the available documentation and results of post-failure field mapping carried out by FSW. Field mapping of the landslide was carried out on 10 June 1998 before placement of the rockfill toe berms, and subsequent field mapping was undertaken on 15 June 1998 after removal of landslide debris from the failure in the upper cutting (slope No. 11NW-B/C69). Further information was collected from a series of trial pits sunk by Gammon, under the direction of FSW and PAC, between August and October 1998 during the permanent remedial works.

5.2 Geology and Ground Investigations

Sheet 11 of the Hong Kong Geological Survey 1:20 000 Map Series HGM20 (GCO, 1986a) indicates that the landslide site is underlain by medium-grained granite. Although no structural features are recorded at the landslide site, the map indicates a number of southwest – northeast trending faults and photogeological lineaments, and also a number of southeast – northwest trending sub-vertical joints in the general region. The Hong Kong Geological Survey Memoir No. 2 (GCO, 1986b) noted that these lineation orientations are typical of the region. The orientations of these discontinuities coincide approximately with the general trend of the principal lateral release planes involved in the landslide of 9 June 1998.

Drillhole Nos. LD45A, LD46, LD46A and LD50 (Figure 5) were constructed in 1993 for the geotechnical design of the Lung Cheung Road and Ching Cheung Road Improvements project. These indicate that the landslide site is underlain principally by corestone-bearing CDG, which grades into Grade II rock at about 10 m below, and approximately parallel to, ground surface. Drillhole Nos. LD46 and LD46A, which are closest to the landslide area, have been superimposed onto the section through the landslide (Figure 3).

The trial pits dug in August to October 1998 (Figure 6) aimed to locate the tension cracks, release planes and failure surfaces associated with the landslide. This involved eight machine-dug trial pits (Nos. TP1 to TP8) to a maximum depth of about 5 m and a hand-dug shallow trench (No. TRA) approximately 1 m deep.

These trial pits encountered highly to completely weathered granite, with a high (30% to 50%) proportion of slightly to moderately weathered granite. Less weathered granite rock occurs as corestones of sizes varying from less than 1 m^3 to structured masses of tens of cubic metres.

The trial pits in the upper western part of the landslide (particularly TP1) showed closely fractured, Grade II and III rock, as well as the lateral release plane (Plate 13). These fractures were up to 50 mm in width. Hand probing using reinforcement bars proved a minimum depth of fracturing to 5 m below original ground (road) level at TP1.

Trial Pit No. TP3 encountered part of the lateral tension crack within the main failed mass (Plate 14). The 150 mm (approximately) crack had become infilled with rockfill that was used to surcharge the slope as part of the emergency stabilisation works.

The basal slip plane to the main lower failure was encountered in six of the trial pits (Nos. TP3, TP4, TP5, TP6, TP7 and TP8). This consisted of a thin (10 to 50 mm), apparently persistent, slightly undulating layer of soft to firm sandy silty clay, which exhibited signs of movement and dipped at angles of between 10° and 20° in a south-westerly direction. At the location of the major tension crack at the toe of the upper cutting, the clay layer was located at approximately 4 m below ground (road) level. Based on its elevation at the various locations encountered, the clay layer appears to have an average dip of about 14° in a direction sub-parallel to the strike of the western release plane, and oblique to the slope face. Typical views of this clay layer are shown in Plates 15 to 17.

It is considered that the apparently persistent clay layer possibly represents an infilled sheeting joint. The infilling may be a deposit left in the joint by leaching or infiltration, above a weathering zone of lower permeability. Alternatively, the clay layer may be the product of a long period of near-surface weathering of the insitu material.

It is noted that this clay layer was not recorded in the logs of the two previous drillholes (Nos. LD46 and LD46A), which are closest to the landslide area (Figure 3). As these drillholes were sunk by water-flush drilling with Mazier sampling at about 2 m intervals, the clay layer may not be readily identified by such routine drilling.

5.3 Laboratory Tests

5.3.1 Soil Samples

A series of direct shear box tests was carried out at the Public Works Central Laboratories (PWCL) on undisturbed specimens of relict jointed CDG, cut from a block sample taken from the continuation of the western release plane, beyond the area of movement (Figure 2). Each of the three specimens was tested under drained conditions, with three cycles of shearing, in order to determine the peak and post-peak shear strength parameters.

The average peak shear strength was measured as $c' = 6$ kPa and $\phi' = 34°$. These tests were carried out before the actual basal slip plane was revealed in the trial pits, and were carried out on the premise that the results might be relevant to the main failure plane. In the event, this proved not to be the case, as the persistent sandy silty clay layer was subsequently observed to have acted as the basal failure plane and controlled the failure.

Direct shear box tests were carried out at the PWCL on a sample of the sandy silty clay from the basal slip plane (taken from TP4). Unfortunately, the tests were not successful due to inappropriate testing procedures being inadvertently adopted.

5.3.2 Water Samples

In order to assess the possible ingress of salt water from the reservoir into the landslide site, chloride content tests were carried out on the water-soluble residue of a sample of CDG washed out from the landslide onto the slip road under construction below, together with a sample of water taken from seepage along the western release plane of the landslide. A water sample was also taken from the nearby stream to the east of the site for comparative purposes. A further water sample was taken from a small permanent flow in a stepped channel feeding into the stream, in the valley opposite the reservoir valve chamber. The locations of the sampling points are shown in Figure 7.

The results of the tests show that the chloride contents of the samples from the landslide site were below the detectable limit. The control sample from the stream had a slightly elevated chloride content (9 mg/l), whilst the sample, from the small flow feeding into the stream, showed a higher chloride content of 630 mg/l. This particular flow was fed into some nearby squatter huts. The squatters confirmed that this was a reliable and permanent source of water, although it was not used for drinking as a result of its salinity. The relatively high chloride content of this source is probably the reason for the slightly elevated chloride values of the stream lower down. The typical chloride content for seawater is in the region of 17,000 mg/l. Consequently, although the concentrations measured in the two water samples in the stream area suggest a degree of contamination, the extent is relatively small. Furthermore the relevant flows from which samples were taken are sufficiently remote from the landslide site to be of direct relevance to the failure. However, the above findings illustrate the possibility of localised leakages from the reservoir or its associated pipe runs.

According to the WSD, the source of the chloride contamination is probably diffusion from the reservoir site.

5.4 Surface Water Drainage System

The original drainage system for the former Tai Woh Ping Road comprised a shallow dished channel on the northern, down-camber side of the road, which flows towards the crest of the landslide site (Figure 7). It is intercepted by a double grated gully/catchpit located near the reservoir valve chamber upslope of the landslide site, where a cross-road culvert falls in the general direction of a catchpit located on the valley side above the nearby flowing stream. Dye tests carried out by Gammon under the direction of FSW in August 1998 were unable to confirm a positive outflow for the gully. The tests however confirmed that the drain was only able to cope with a very nominal flow of water, beyond which the system backed up to ground level and water then spilled back into the verge-side dished channel, in a westerly direction, towards the landslide site. It is suspected that the culvert drain was effectively blocked, and the nominal flow that it did accept (measured as about 0.2 litres per second) could be leakage from pipe joints, etc.

As a result of the blockage in the culvert drains, the landslide site probably received surface water flows effectively from a larger catchment area than that considered in the design of surface water drainage for the newly-formed cut slope.

Given the limited quantity of water that would be accepted by the gully/catchpit and the large catchment area, and based on observations by FSW (see also Section 4.1), the possibility of water overtopping the new drainage system at this narrowest part of the old road, particularly during heavy rainfall, cannot be excluded.

5.5 Groundwater Conditions

Groundwater was not encountered in any of the trial pits excavated during August to October 1998.

Groundwater records for both the original (August to December 1993) and recent (June to October 1998) monitoring exercises have been reviewed. The locations of both sets of groundwater monitoring stations are shown in Figure 5, and sections summarising the findings are given in Figure 8.

The 1993 water levels were measured in two standpipes (drillholes LD46 and LD49) and a piezometer (drillhole LD50). Readings were taken for one week after installation, weekly for two months thereafter, with a final reading one month later. No Halcrow buckets were provided in any of the installations to monitor the transient peak groundwater levels. The monitoring records indicate that the groundwater table during the monitoring period was generally about 10 m below, and approximately parallel with, the new slope which is approximately 3 m below the inferred rockhead level. For design purposes, PAC assumed a water level 2 m above rockhead.

The water levels recorded in the 1993 drillholes at the start of each morning shift during drilling with water as the flushing medium, though less reliable than piezometers, suggest a potential for a higher, possibly perched, water table in the soil mass.

The recent monitoring exercise has included daily readings (since the end of June 1998) of four new sets of groundwater monitoring installations, two at the toe of the cut slope and two on the old Tai Woh Ping Road, each of which has a deep piezometer and a shallower standpipe installed at levels indicated in Figure 8. Readings from June 1998 to the end of October 1998 have been made available by PAC. It should be noted that the peak transient water levels may not have been recorded by the piezometers, because no Halcrow buckets were installed. The two higher installations (P1 and P2) indicate water levels at about 2 m above rockhead, similar to the original assessment by PAC. The two lower installations near the slope toe (P3 and P4), however, indicate that water levels up to over 7 m above rockhead can develop.

The records for standpipe P3, in particular, have indicated a rapid and significant transient response in groundwater level to rainfall, rising by over 10 m almost to ground level. It is considered likely that the rapid response was due to the presence of relatively impermeable layers in the soil portion which contains relict geological defect features and zones of corestones.

It should be noted that post-failure piezometer monitoring in the vicinity of the landslide may not necessarily be representative of conditions at the time of the landslide of 9 June 1998 because of the disturbance to the soil mass and the opening up of tension cracks as a result of significant slope deformation.

When the site was inspected by FSW some 16 hours after the failure, seepages were noted to be issuing along the persistent discontinuity marking the western boundary of the slip. The base and sides of the emptied service reservoir above the landslide were inspected by FSW on 15 June 1998, and no significant structural defects which might affect its watertightness were observed. There were however isolated damp patches on the reservoir floor slab, with a surrounding chloride crust (Plate 18), which was probably the result of evaporation of salt water following emptying of the reservoir.

There is considerable uncertainty about the actual groundwater condition at the landslide site. There is limited monitoring data prior to the failure and the available data are not necessarily consistent, whereas the post-landslide monitoring may not be entirely representative of the conditions at the time of failure because of the significant opening up of the ground. Notwithstanding the above, the following are what is known about the landslide site with reasonable confidence:

(a) there was no permanent high main groundwater table and that the main groundwater table was unlikely to have risen significantly to affect the failure,

(b) the catchment area was small, and

(c) there was much erosion by surface water.

It is likely that the main source of water ingress was direct surface infiltration which was probably promoted by the presence of surface water directed onto the site. If the relict joints were open prior to failure, they would probably have enhanced infiltration. However, there are no records of significant open joints being observed prior to the landslide.

6. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge is No. K06, which is located at Carnation House, So Uk Estate, about 600 m to the west of the site. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the GEO.

Daily rainfall for one month preceding and seven days following the event, and hourly rainfall for 48 hours before and 8 hours following the landslide are given in Figure 9. The daily rainfall records show that the storm was concentrated around 9 June 1998 (the day of the landslide), with the hourly data indicating intense peaks from 05:00 to 08:00 hours, 14:00 to 15:00 hours and 16:00 to 17:00 hours.

For the purposes of rainfall analysis, the time of the landslide has been taken to be 17:00 hours on 9 June 1998. Isohyets of rainfall for the 24-hour period preceding the landslide are shown in Figure 10.

Table 1 presents the estimated return period for the maximum rolling rainfall for various durations based on historical rainfall data at the Hong Kong Observatory (Lam & Leung, 1994). The 12 hour and 24 hour-rainfall (290 mm and 364 mm respectively) was the most severe with a corresponding return period of about 10 years.

A comparison of the 9 June 1998 rainstorm with that of other past major storms recorded by raingauge No. K06 is presented in Figure 11. The 9 June 1998 rainstorm was less severe than other previously recorded significant storms since installation of the raingauge in 1983. However, it is pertinent to note that the rainfall preceding the landslide was the first severe rainstorm that the cutting was subjected to since its substantial modification in late 1997.

Whilst return periods for various durations within the storm have been assessed based on Lam & Leung (1994), it is recognised that this method does not necessarily give the true return periods for a particular site (Wong & Ho, 1996). However, it does provide an objective ranking of the likely relative severity of the different rainfall characteristics assessed and is considered adequate for the present purposes.

7. THEORETICAL STABILITY ANALYSES

Stability analyses using the rigorous method of Morgenstern & Price (1965) were carried out to assist in the diagnosis of the probable causes and mechanism of the failure. A representative cross-section through the landslide is presented in Figure 3. The pre-failure ground profile was established from topographic survey plans, and the geometry of the failure surface was based on FSW's field observations, including the findings from the trial pitting exercise.

There is great uncertainty about the actual sequence of the failure. The presence of a major tension crack at the back edge of the old Tai Woh Ping Road (Section 4.3) suggests that the failure possibly occurred in two stages. However, it is also possible that the failure occurred in one go, with the tension crack formed as a result of internal deformation of the displaced mass or as a result of post-failure movement of the lower slope. For illustrative purposes, the landslide has been analysed in two parts:

(a) a slip in the lower cut slope, with the scarp along the back of the old Tai Woh Ping Road, and

(b) a subsequent slip in the upper cut slope, along a steeply-dipping surface, with the back scarp along the fence line of the reservoir compound behind the crest.

The lower cut slope is considered to have failed along a thin, probably persistent sandy silty clay layer dipping at a shallow angle (about 14°) out of the cut face, based on observations in the trial pits. For the purposes of stability analyses, shear strength parameters with c' value of zero and ϕ' values ranging from 20° to 32° were considered, together with various perched water tables above the clay layer.

The results of the stability analyses are shown in Figure 12. These indicate that the main failure was likely to have been the result of the presence of a material with relatively low shear strength and high groundwater conditions. Given the uncertainties in groundwater conditions acting on the slip surface at the time of failure, and that laboratory testing could not adequately establish the representative shear strength parameters for the clay layer, it is not possible to determine the operational mass shear strength and critical groundwater conditions from the analyses.

The failure surface of the upper landslide generally passed through relatively intact CDG. There was no field evidence that relict joints with weak infill material played a significant role in this failure. For the purposes of analysis, a range of c' values of between 6 and 14 kPa, and ϕ' values of between 35° and 42° have been considered. Dry conditions (i.e. fully saturated but with no positive water pressure) were assumed. The results are given in Figure 13. The findings suggest that the failure can be explained by fairly typical shear strength parameters of the CDG.

8. DIAGNOSIS OF PROBABLE CAUSES OF THE FAILURE

The close correlation between the rainfall and the time of failure indicates that the landslide was probably triggered by rainfall. A number of principal factors probably combined to cause the failure of the newly-formed cutting in its first wet season under heavy rainfall, including:

(a) the presence of an adversely orientated persistent, thin sandy silty clay layer providing a relatively low-strength basal shear plane and a relatively impermeable barrier, which is favourable to the build-up of perched water table,

(b) the presence of persistent relict discontinuities acting as lateral release planes, and

(c) the existence of cracks in the damaged road pavement of the old Tai Woh Ping Road which promoted concentrated water ingress into the lower cut slope.

The main source of water ingress was probably direct infiltration of rainfall, which was possibly enhanced by the surface water overtopping the road at the crest of the lower cut slope during the heavy rainstorm on 9 June 1998. This would have led to wetting up of the material and probably formation of a perched water table above the persistent sandy silty clay layer, resulting in failure.

Another possible contributory factor to the failure is the influence of the 150 mm diameter pipe and its gravel surround below the road above the crest of the lower cut slope. This may have contributed to feeding water into the landslide area, although the fact that the pipe was largely blocked and that the pipe apparently acted only as a subsoil drain, with no surface water connection, would have limited this effect. However, the gravel filled trench could have conducted a fairly large amount of water and may have acted as a release plane at the back of the main failure. It is also noted that poor maintenance of the cross-road culvert probably resulted in surface water flow along the carriageway from a relatively large catchment area to the area above the landslide site. The possibility of concentrated water ingress through cracks in the road and/or water overtopping the road during rainstorms cannot be excluded.

Although traces of elevated chloride content were detected in the vicinity of the landslide site, there is no evidence of significant leakage from the reservoir and the associated pipe system. It is therefore probable that leakage of salt water had a negligible contribution to the landslide.

9. DISCUSSION

The failure occurred at a newly-formed cut slope that had previously been designed to the required geotechnical standards based on detailed site-specific ground investigation and laboratory testing. However, the design was yet to be checked and accepted by the GEO at the time of failure. It would appear that the design did not take into account the following key factors:

(a) the presence of an adverse geological structure in the form of a persistent adversely-orientated low-strength sandy silty clay layer, which also gave rise to an unfavourable hydrogeological setting, and

(b) the possibility of high groundwater levels within the slope.

It would appear that the presence of the persistent clay layer was not identified in the conventional ground investigation carried out for the design of the site formation work. Also there is no record of such a clay layer having been exposed in the cut face following the excavation work.

The excavation of the lower cut slope for road widening in 1997 involved the removal of a significant volume of material (about 3,000 m^3 and average about 6 m thick). However, there are no records of relict discontinuities opening up following the excavation which, if present, would have enhanced surface infiltration.

The failed mass associated with the apparently ductile landslide travelled only a fairly short distance (about 1 m) and the displaced mass remained largely on the relatively shallow cut slope (about 40°). This may be due to the re-establishment of equilibrium as a result of the change in slope profile following slope movement (the slope gradient was not particularly steep) and the possible release of water pressure along tension cracks and relict joints which dilate or open up during slope movement. The clay layer, that largely controlled the basal shear plane, possibly behaved in a relatively plastic manner in terms of its stress-strain characteristics which may contribute to the limited mobility of the unstable soil mass.

10. CONCLUSIONS

It is concluded that the landslide was probably triggered by heavy rainfall (10-year return period for the worst duration of 12 to 24 hours).

The failure involved significant deformation (about 1400 m^3 in volume) of a newly-formed, 23 m high and 40° steep soil cut slope. The landslide occurred in its first wet season, i.e. some six months after the cut slope was built.

The failure was principally controlled at its base by an adversely orientated and gently dipping (about 14°) persistent sandy silty clay layer, and its lateral release surfaces by sub-vertical manganese coated persistent discontinuities. Inadequate maintenance of the road surface and road drainage system above cut slope No. 11NW-B/C60 probably resulted in uncontrolled surface water flows causing erosion and increased infiltration into the slope.

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Table 1 – Maximum Rolling Rainfall at GEO Raingauge No. K06 and Estimated Return Periods for Different Durations Preceding the 9 June 1998 Landslide

Duration	**Maximum Rolling Rainfall (mm)**	**End of Period**	**Estimated Return Period (Years)**
5 minutes	11	05:40 on 9 June 1998	2
15 minutes	24	05:40 on 9 June 1998	1
1 hour	66	05:45 on 9 June 1998	2
2 hours	107	06:45 on 9 June 1998	3
4 hours	147	07:50 on 9 June 1998	4
12 hours	290	16:45 on 9 June 1998	10
24 hours	364	16:55 on 9 June 1998	10
48 hours	374	16:55 on 9 June 1998	5
4 days	394	17:00 on 9 June 1998	4
7 days	452	17:00 on 9 June 1998	4
15 days	509	17:00 on 9 June 1998	3
31 days	704	17:00 on 9 June 1998	3

Notes:

(1) Return periods were derived from Table 3 of Lam and Leung (1994).

(2) Maximum rolling rainfall was calculated from 5-minute data for durations up to 48-hours, and from hourly data for longer rainfall durations.

(3) The use of 5-minute data for durations between 2 hours and 48 hours results in better data resolution, but may slightly over-estimate the return periods using Lam and Leung (1994)'s data, which are based on hourly rainfall for these durations.

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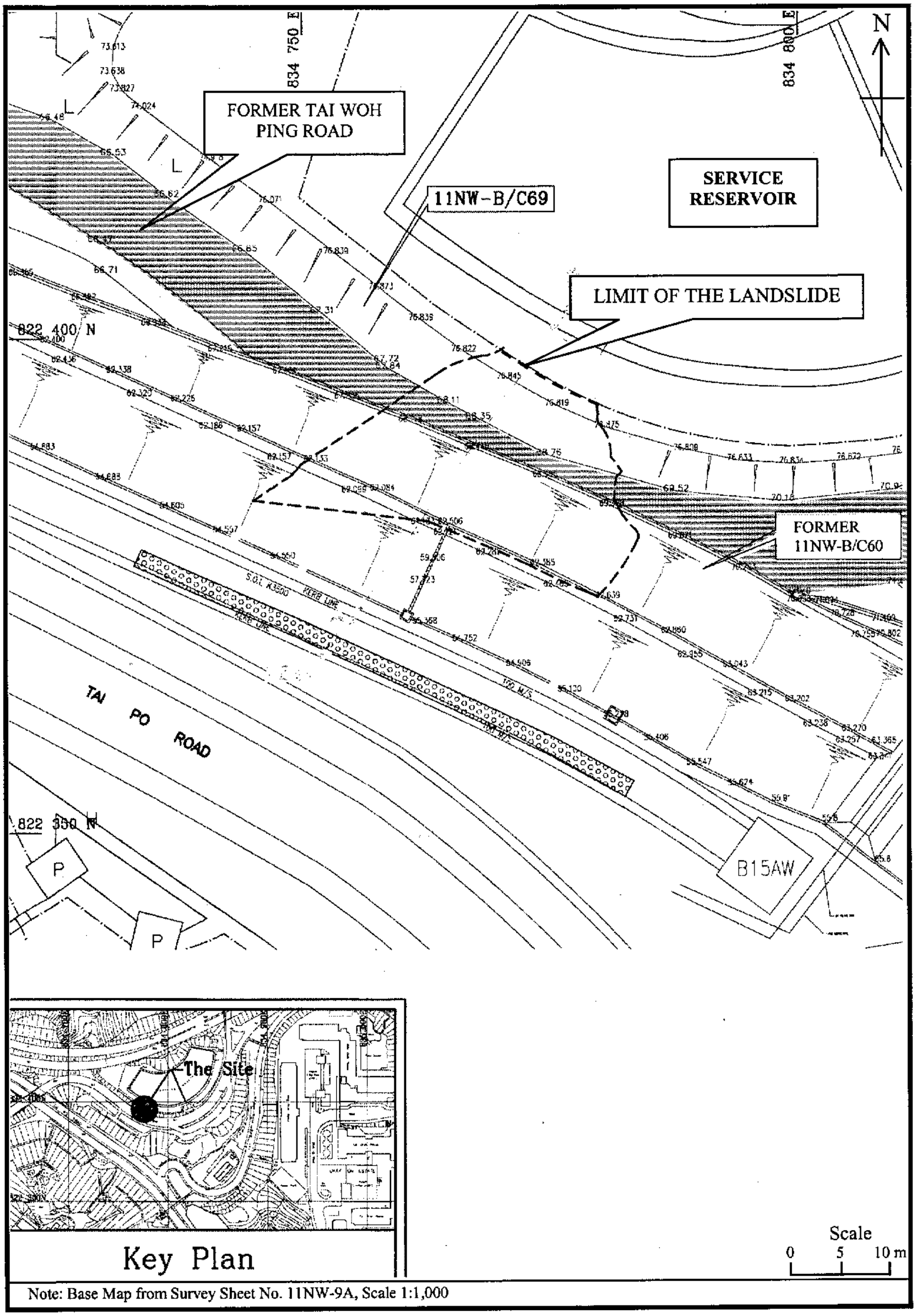


Figure 1 – Site Location Plan

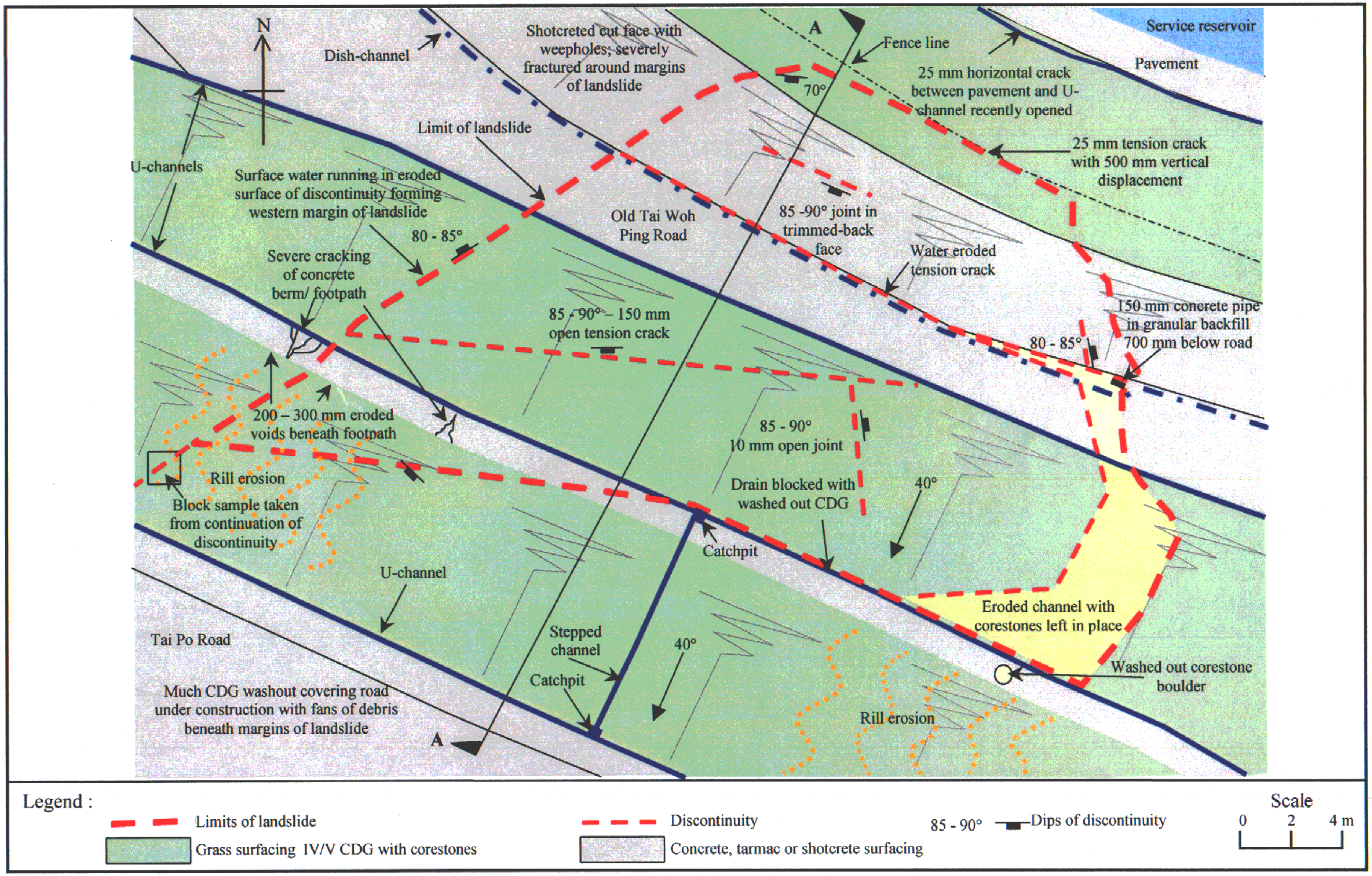
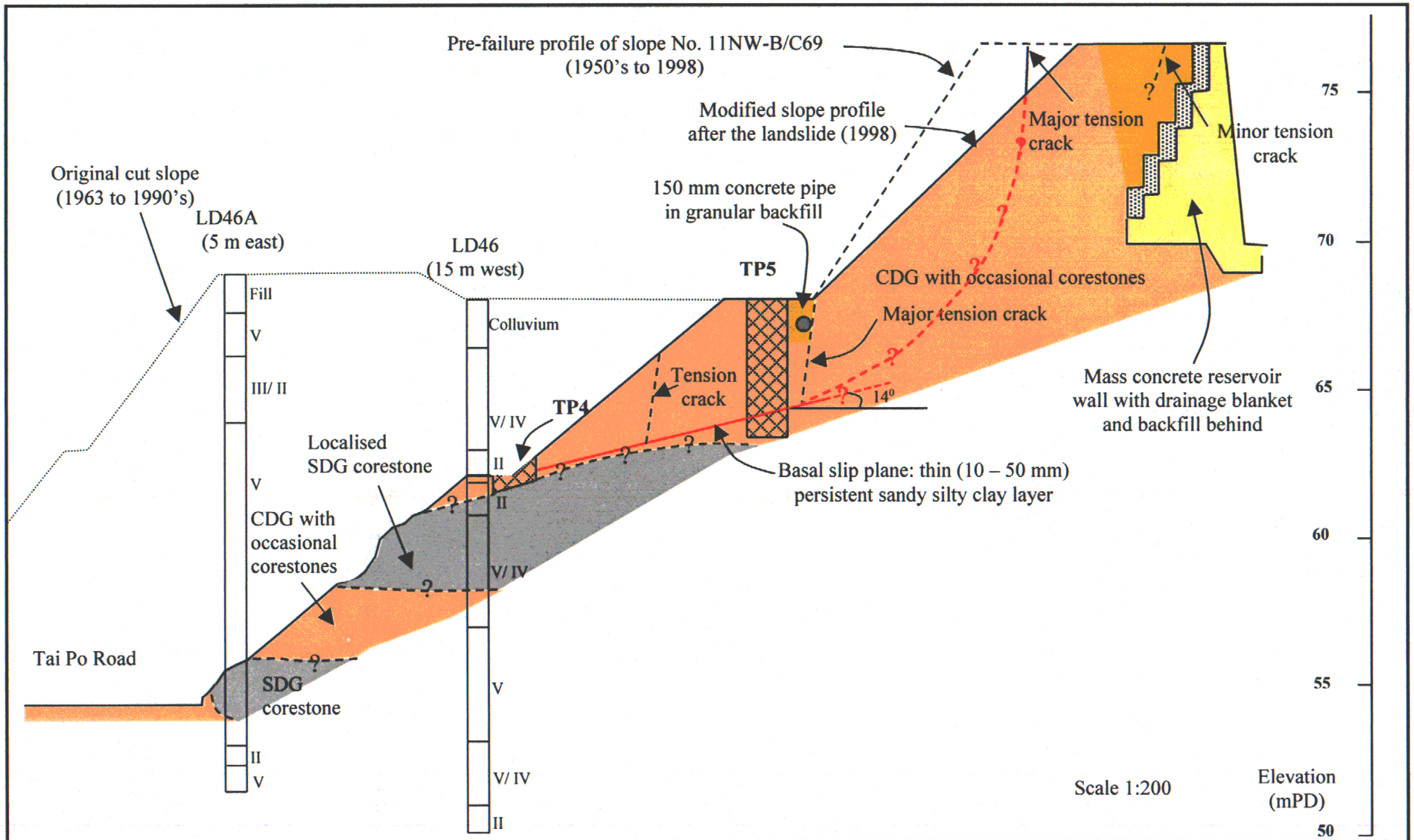


Figure 2 – Plan of the Landslide



Notes : (1) See Figure 5 for location of drillholes
(2) See Figure 2 for line of section

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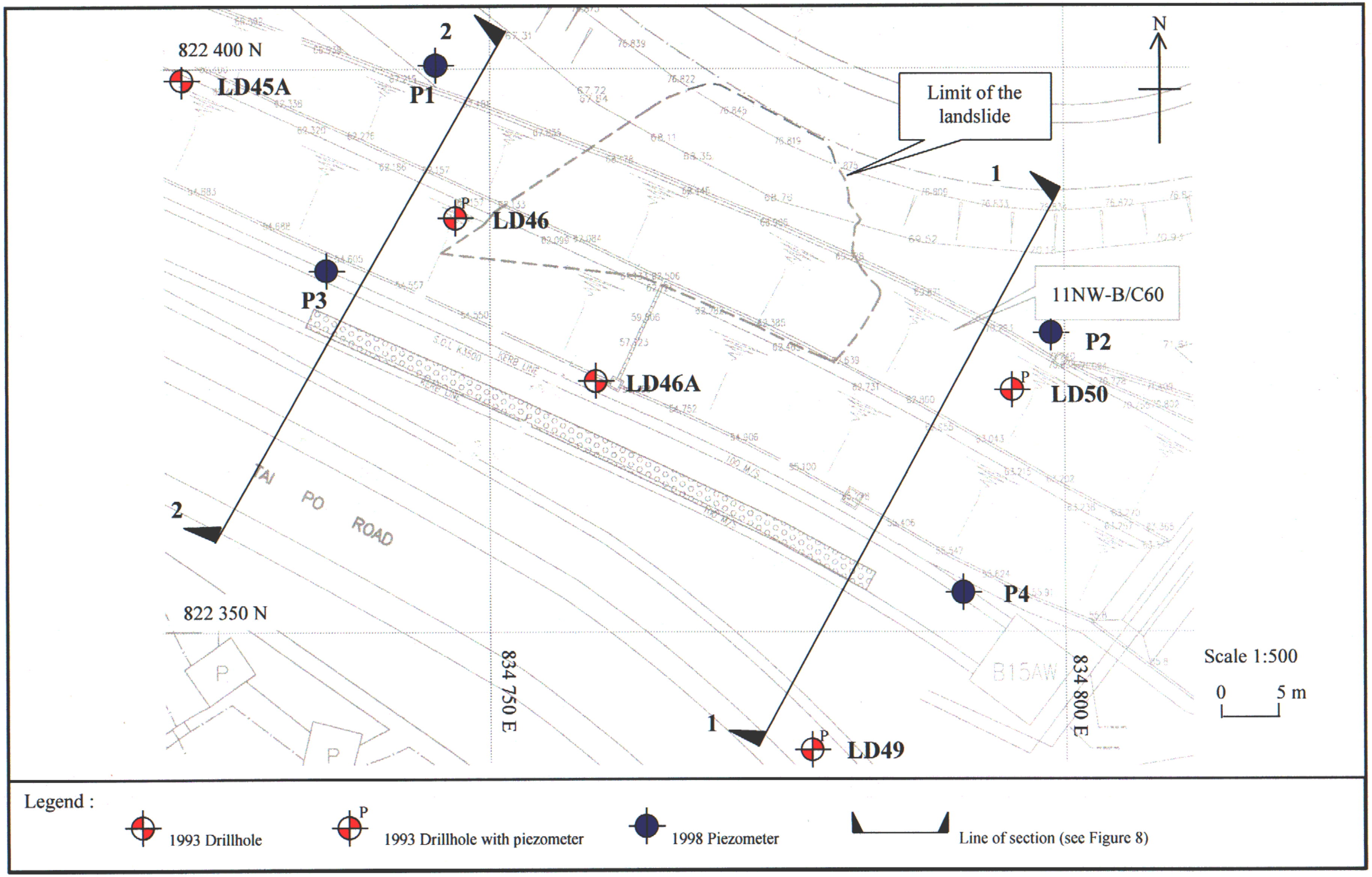


Figure 5 – Drillhole and Piezometer Location Plan

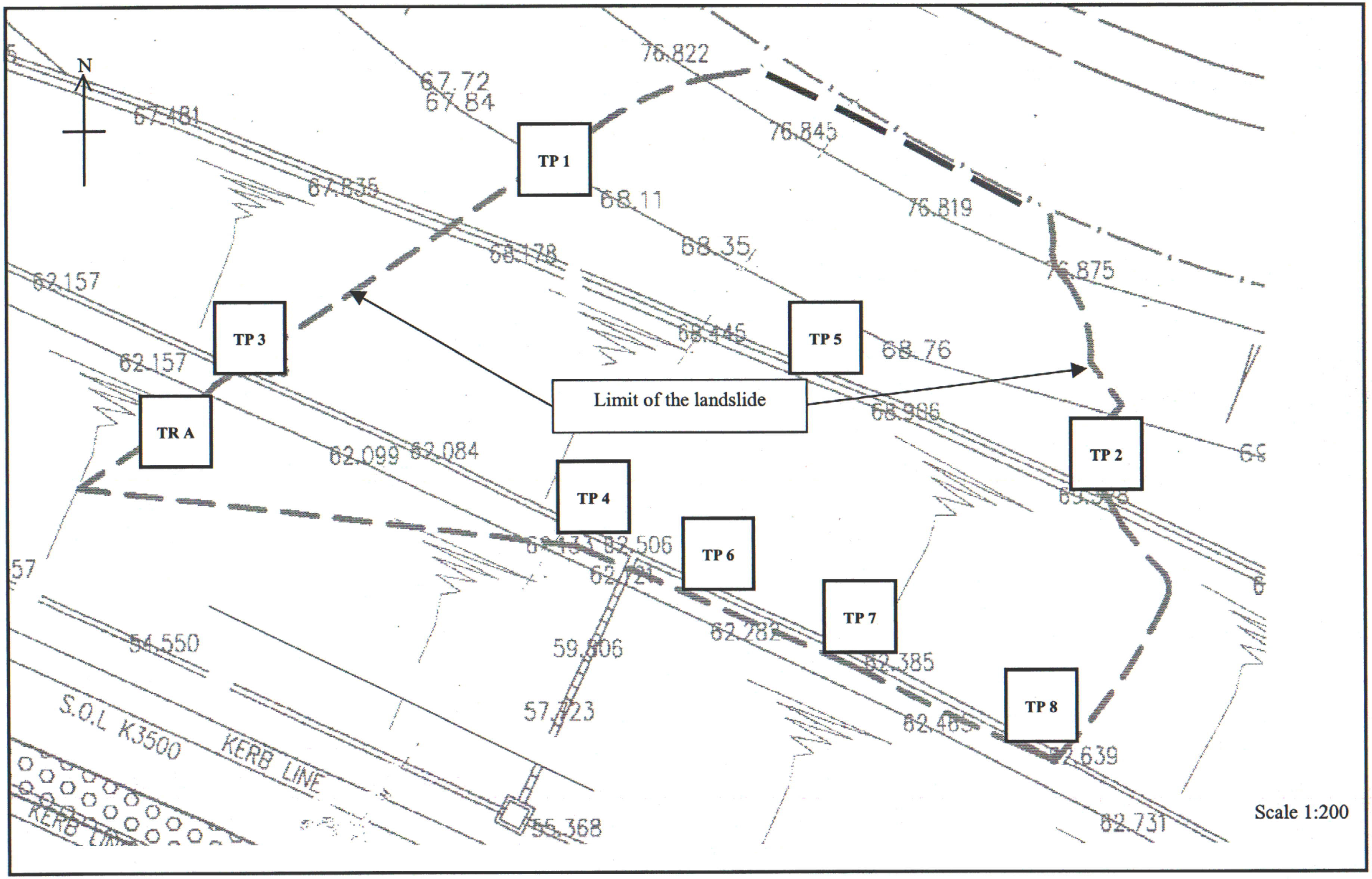


Figure 6 – Trial Pit and Trial Trench Location Plan

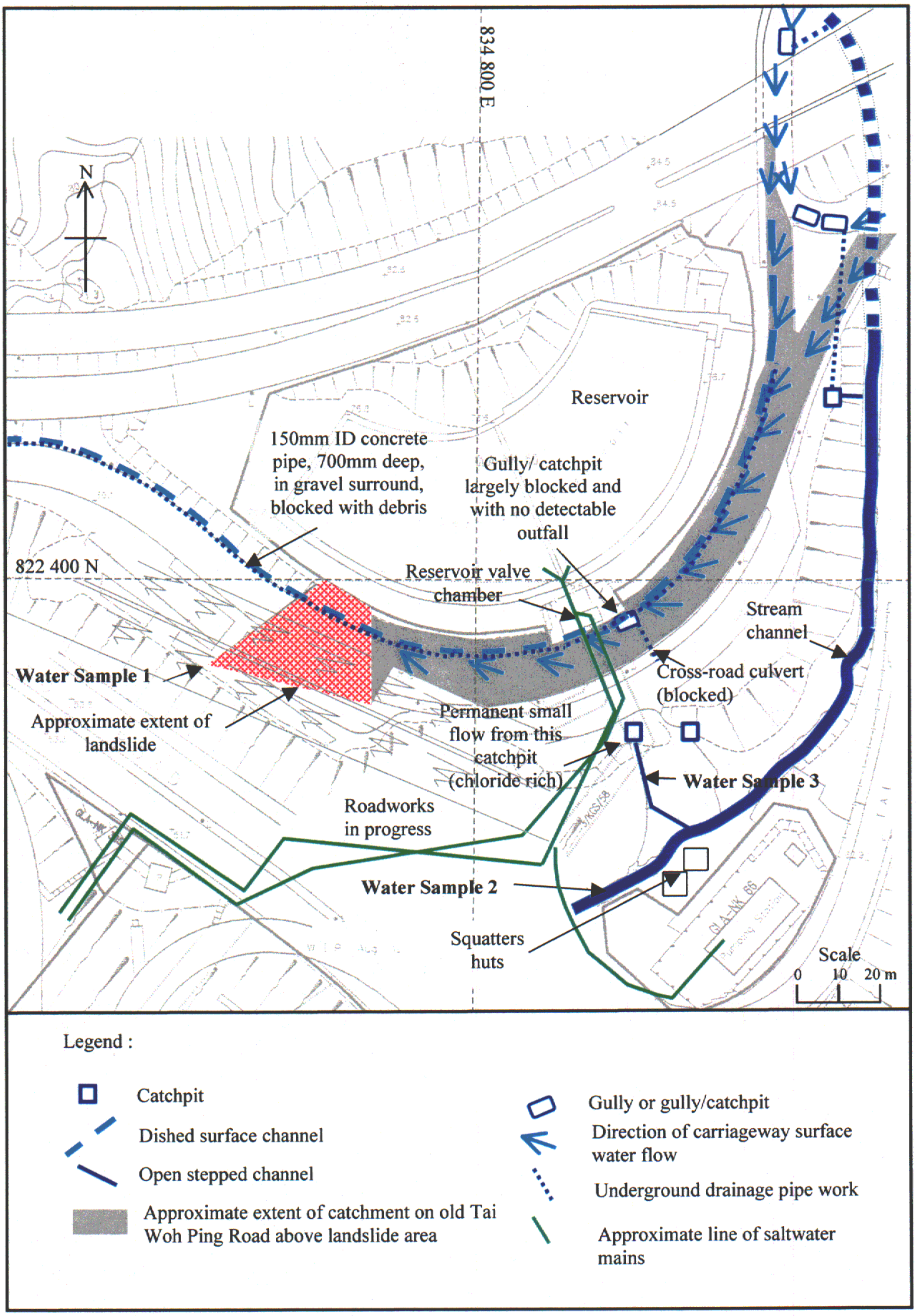


Figure 7 - Sketch Plan Showing Surface and Underground Water-carrying Services

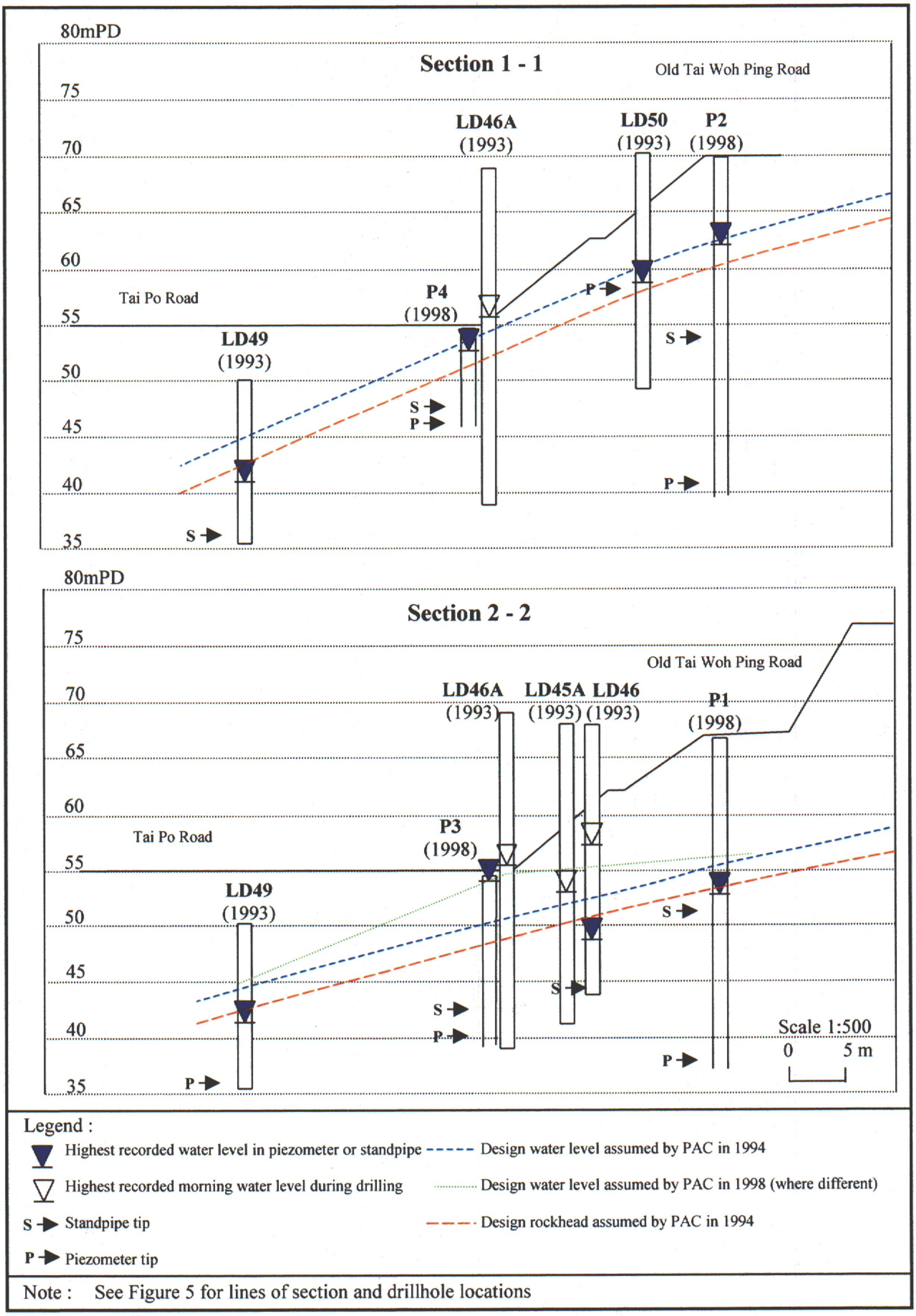
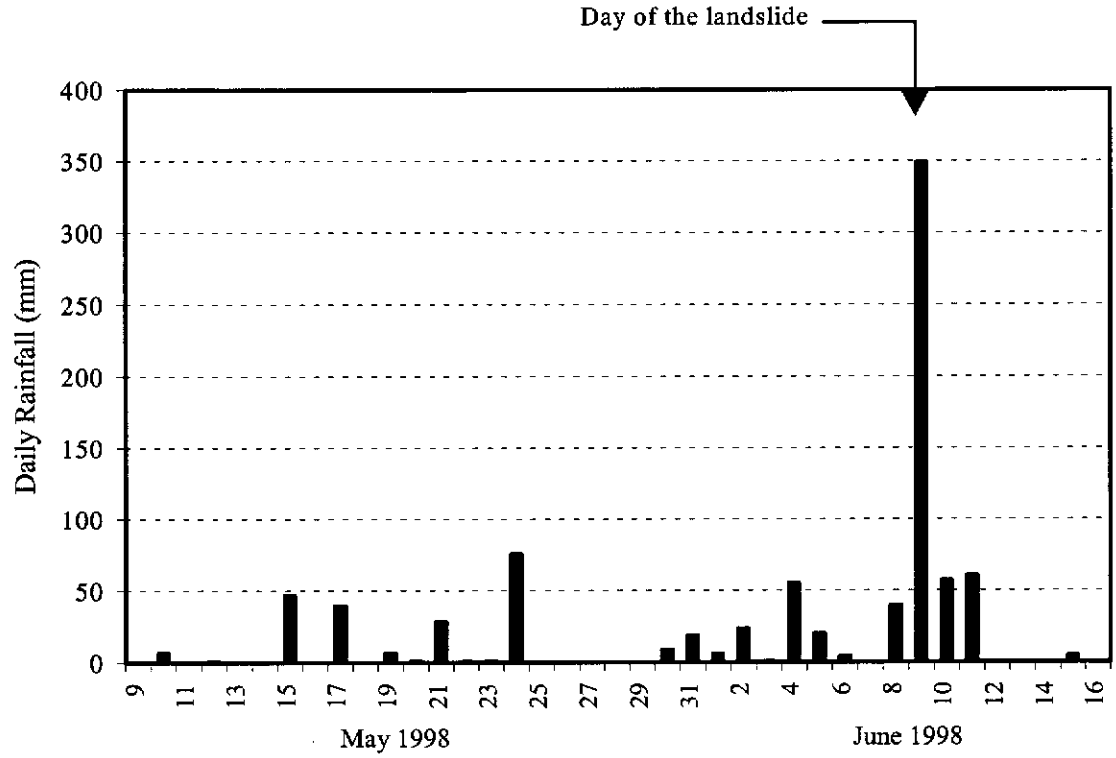
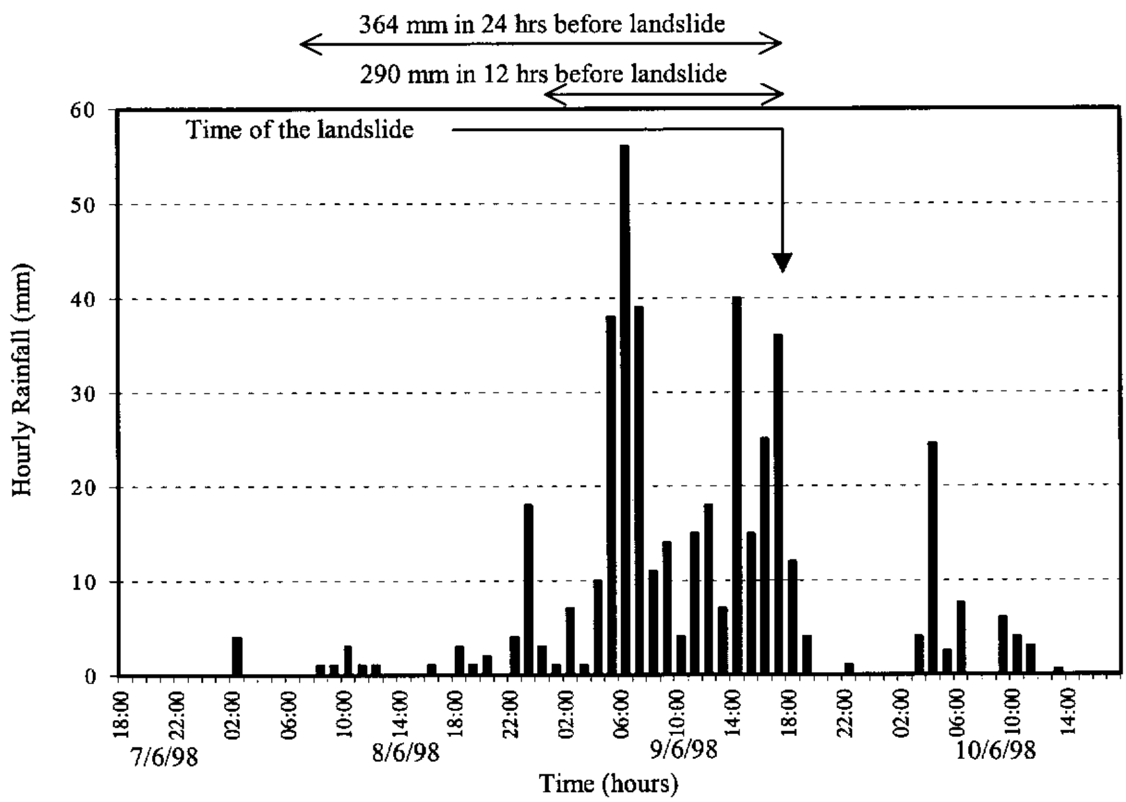


Figure 8 – Summary of Groundwater Monitoring Results

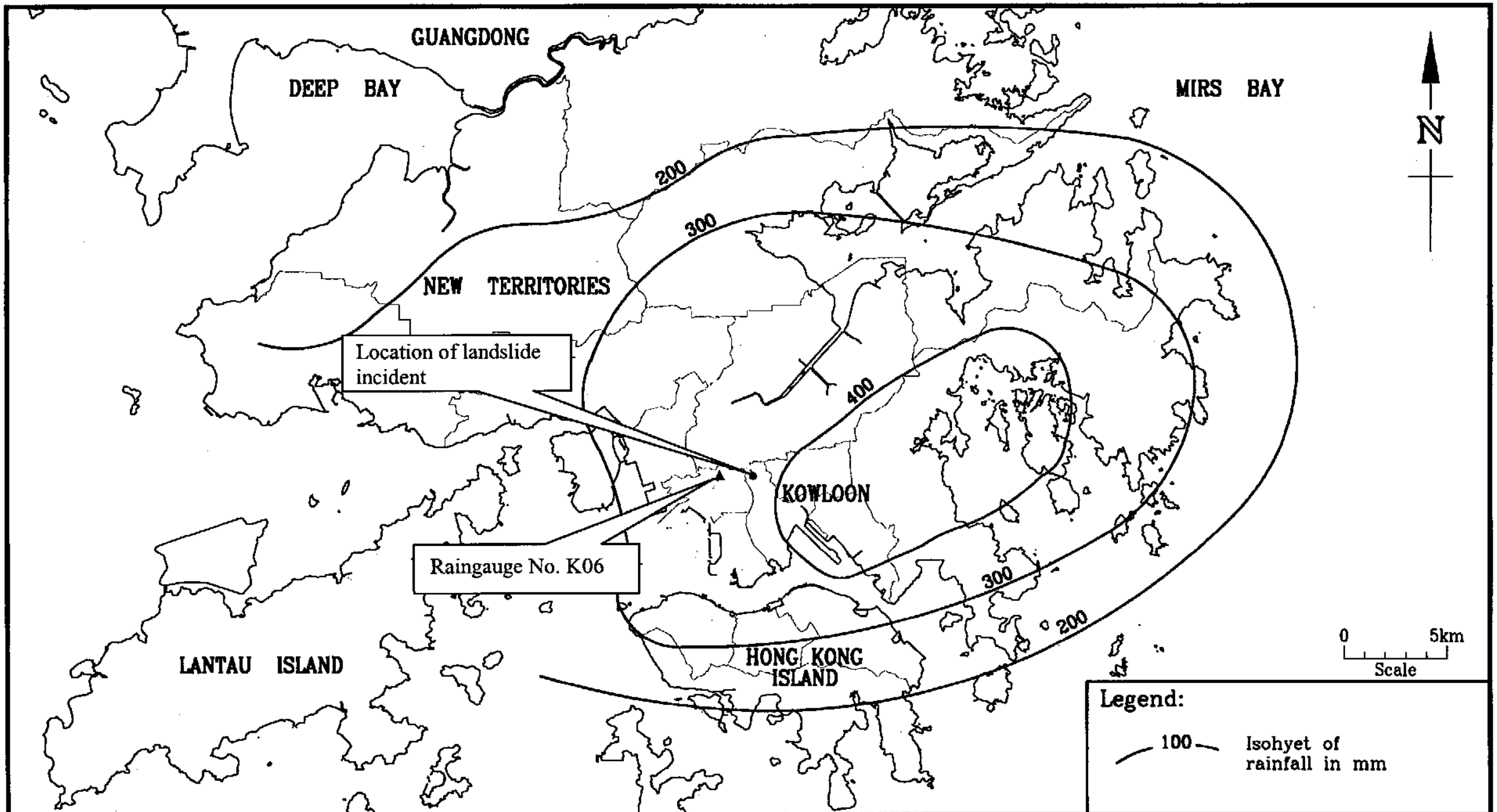


(a) Daily Rainfall Recorded at GEO Raingauge No. K06 from 9 May to 16 June 1998



(b) Hourly Rainfall Recorded at GEO Raingauge No. K06 from 7 June to 10 June 1998

Figure 9 – Daily and Hourly Rainfall Records from GEO Raingauge No. K06



Note: Data from GEO Raingauges

Figure 10 – Isohyets of Rainfall for 24 hour Period Ending 17:00 hrs. on 9 June 1998

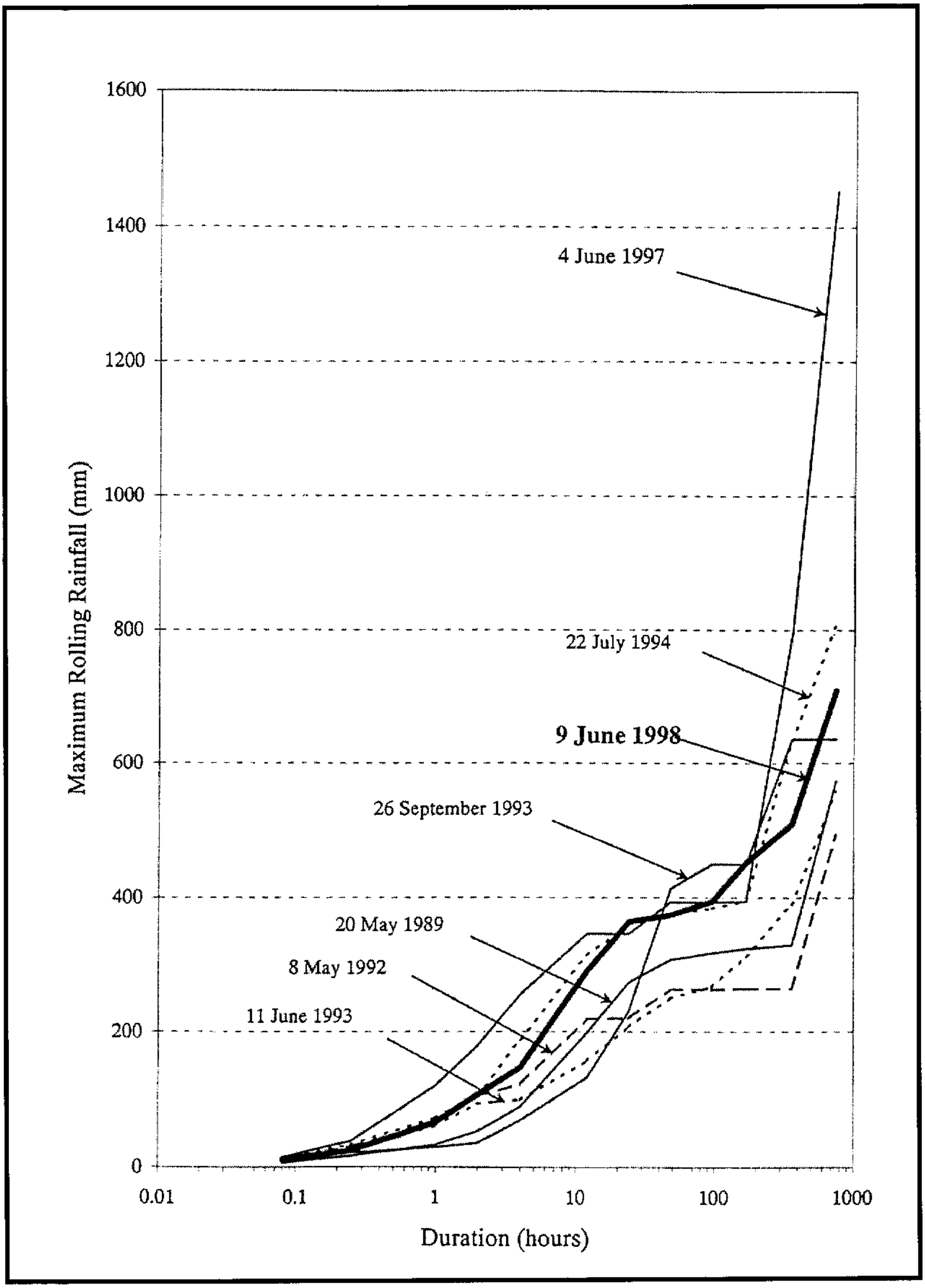


Figure 11 – Maximum Rolling Rainfall Preceding the Landslide of 9 June 1998 and Other Major Rainstorms Recorded by Raingauge No. K06

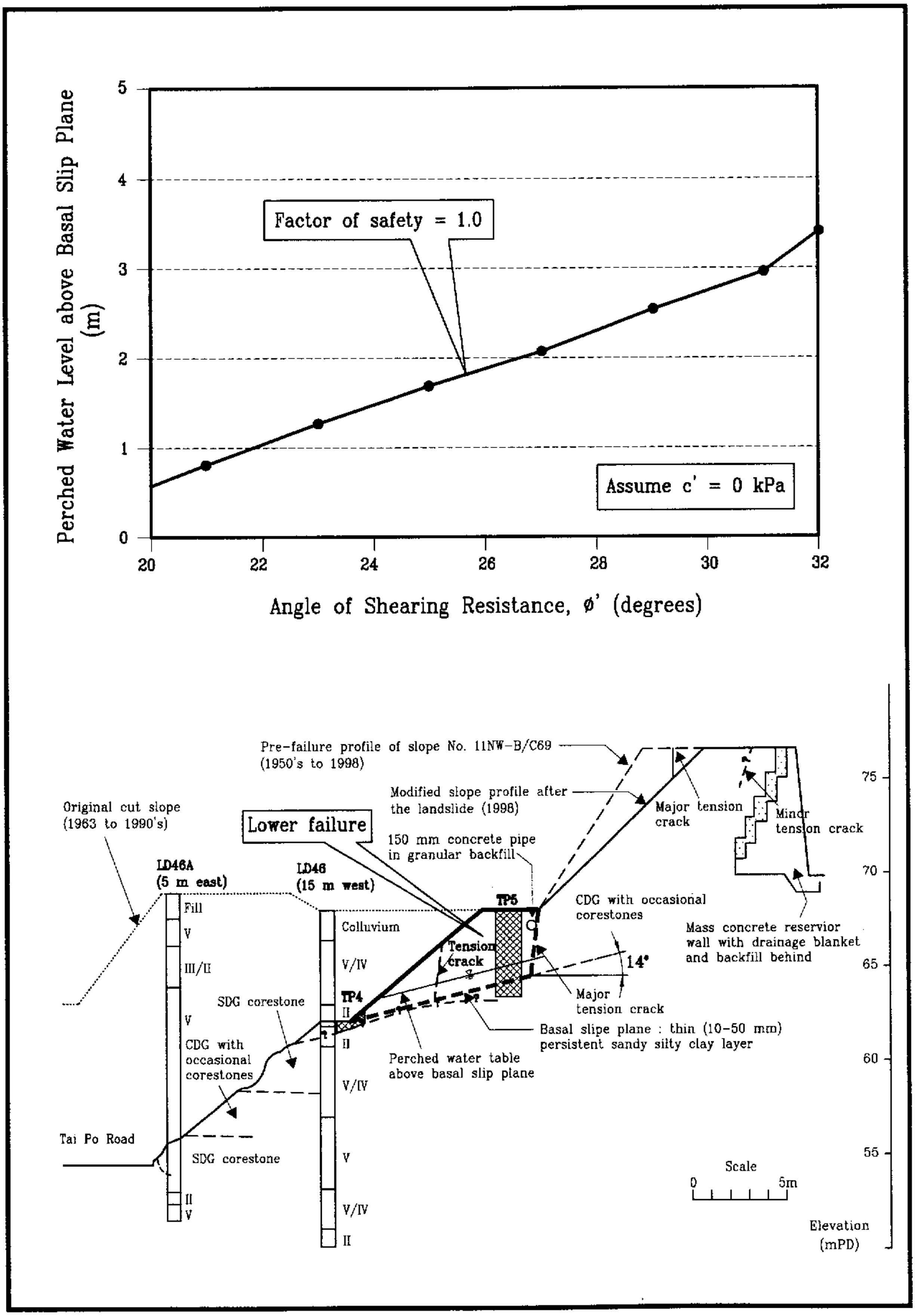
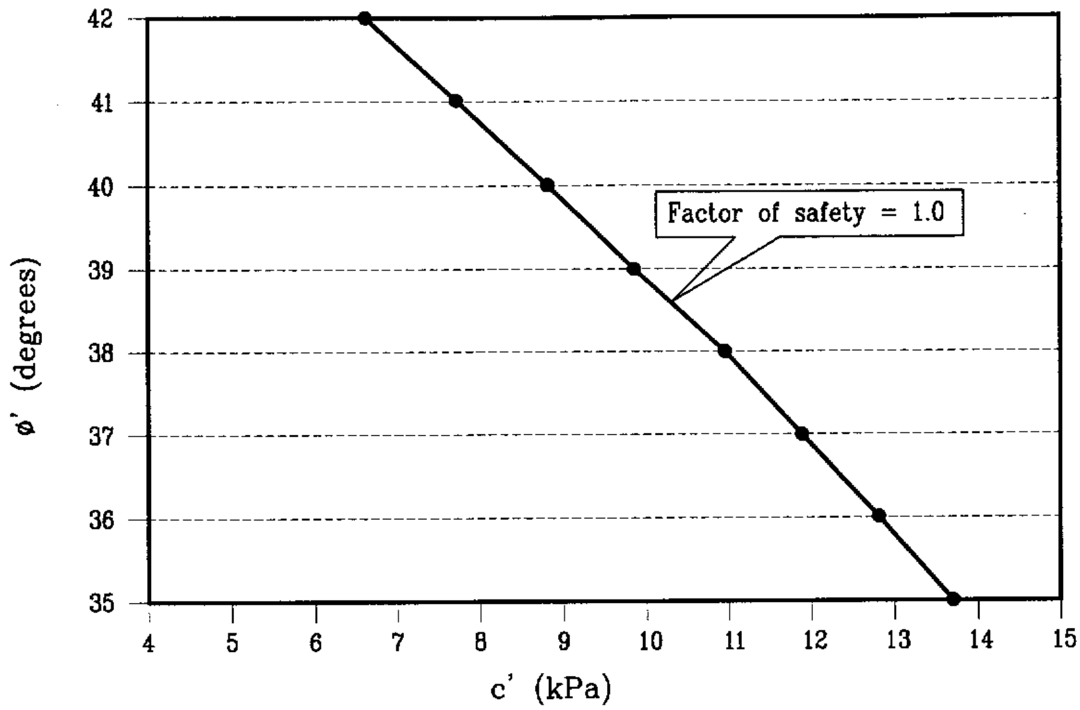
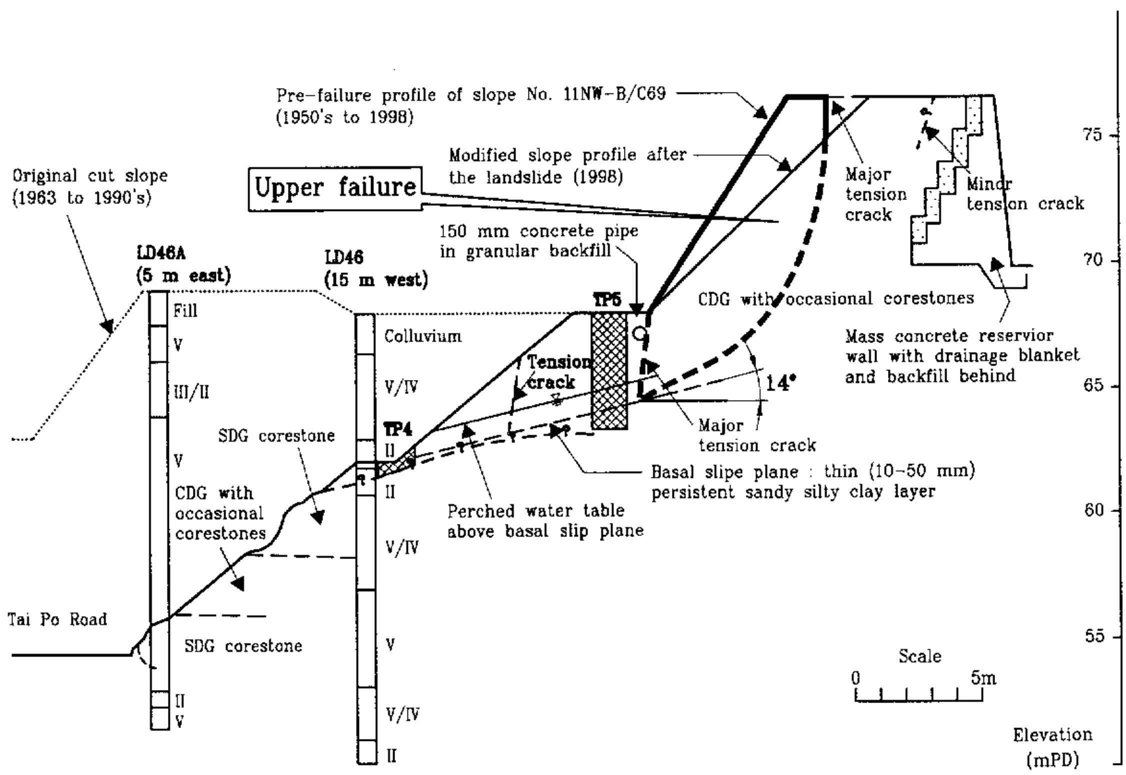


Figure 12 - Sensitivity of Lower Failure to Height of Perched Water Level





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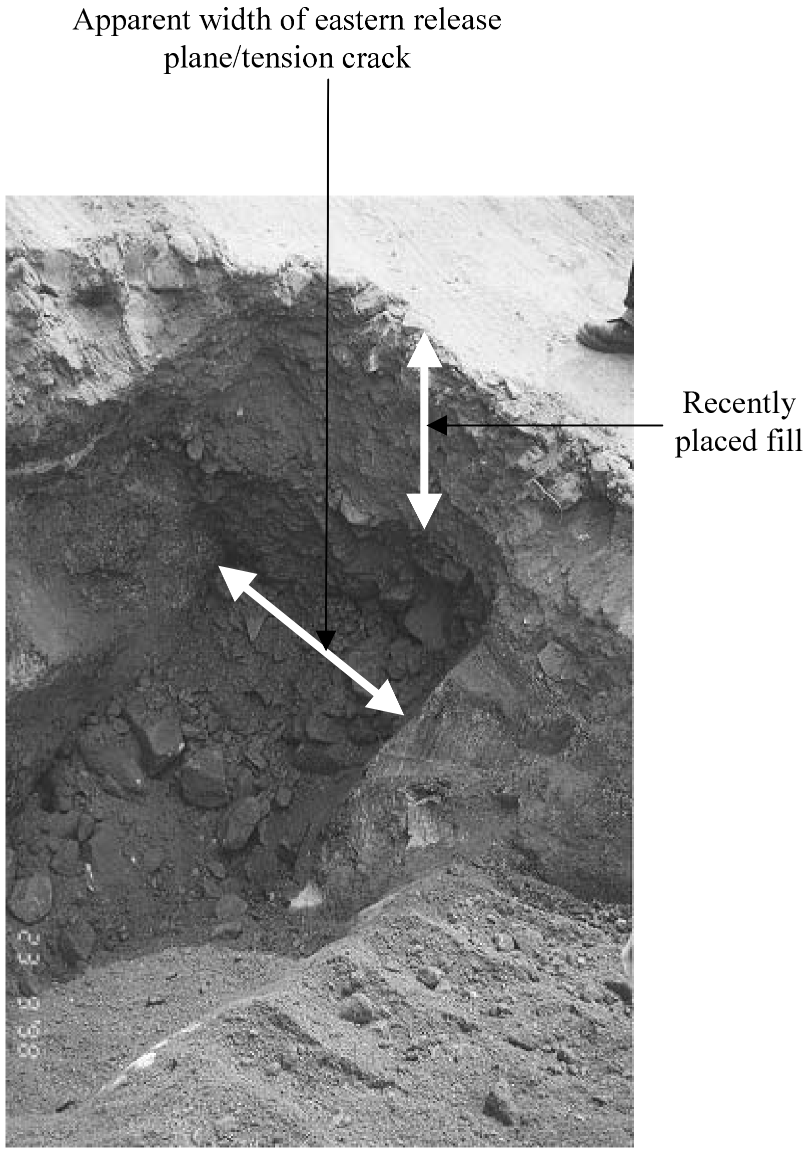

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(Photograph Taken on 23 September 1998)

Plate 11 – View of 150 mm Pipe Exposed in Main Tension Crack
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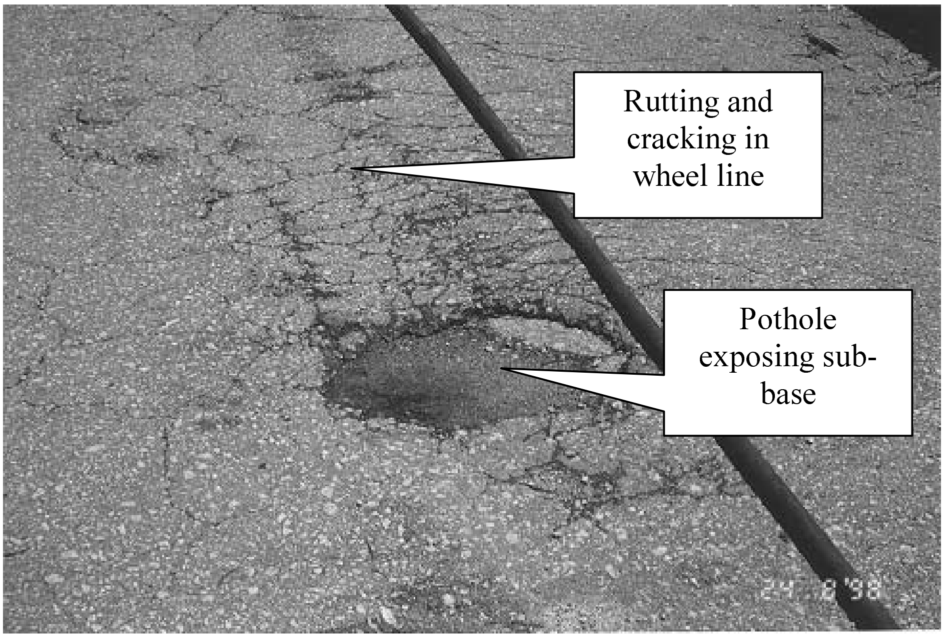

Plate 12 – View of Condition of the Old Tai Woh Ping Road Pavement
(Photograph Taken on 24 August 1998)

Opened joint forming release plane dipping 85°/330°

Plate 13 – View of Trial Pit No. TP1 Showing Western Release Plane
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Tension crack (dipping 86°/175°)
infilled with rockfill

Plate 14 – View of Trial Pit No. TP3 Showing Filled Minor Tension Crack
View Looking East
(Photograph Taken on 25 September 1998)

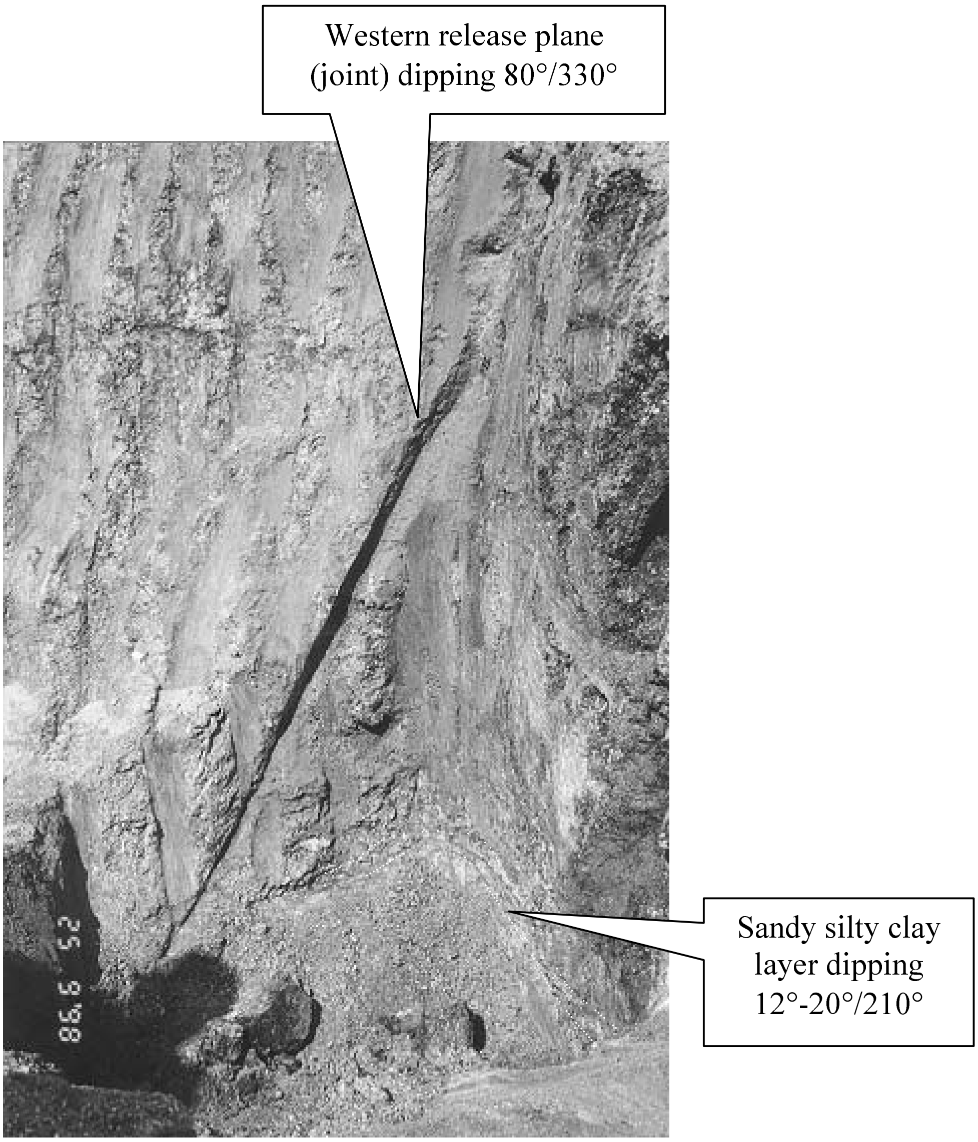

Plate 15 – View of Trial Pit No. TP3 Showing Release Plane and Sandy Silty Clay Layer
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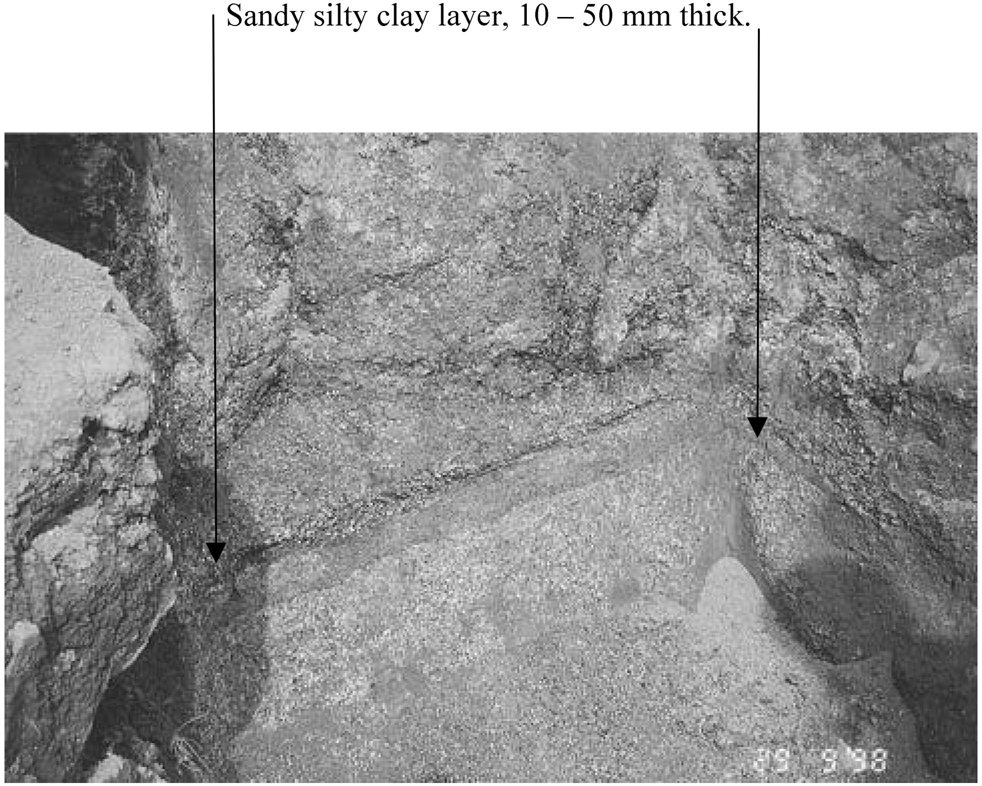

Plate 16 – View of Trial Pit No. TP4 Showing Sandy Silty Clay Layer Representing Probable Basal Slip Plane
View Looking North
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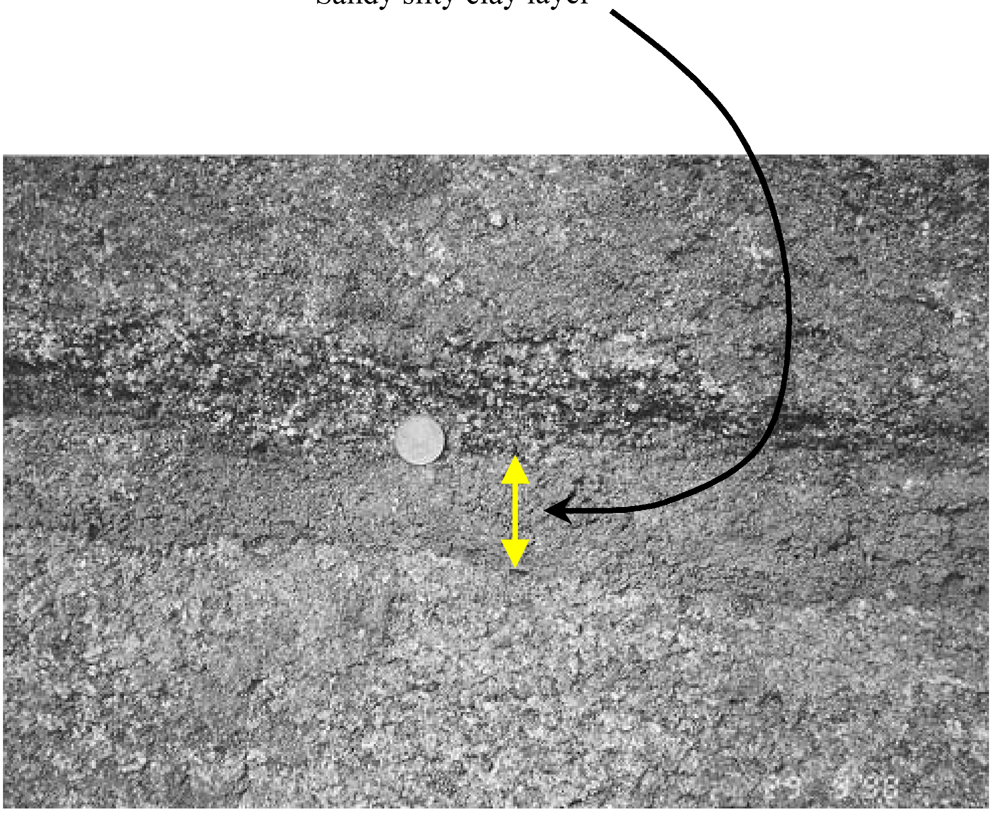


Plate 17 – Close-up of Sandy Silty Clay Layer, Trial Pit No. TP4
(Photograph Taken on 29 September 1998)

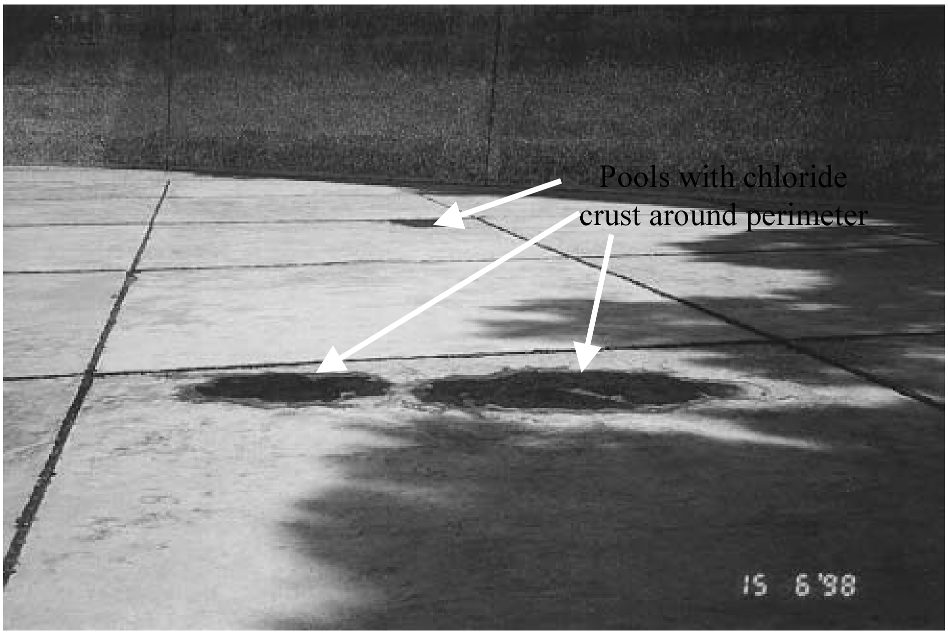


Plate 18 – View of Reservoir Floor After Emptying
(Photograph Taken on 15 June 1998)

APPENDIX A

AERIAL PHOTOGRAPH INTERPRETATION

A1. INTRODUCTION

An aerial photographic interpretation (API) has been carried out as part of the desk study for the detailed study of the landslide that occurred on 9 June 1998 in a new cutting slope, formed by substantially cutting back an older cutting, at Tai Po Road. The failure also affected another older cutting bordering a service reservoir immediately above it. The main objective of the API was to confirm site history and assess site conditions, particularly with respect to past instability of the former cutting. A series of aerial photographs, as listed in Section 7, have been studied.

A2. SITE HISTORY

1949 These earliest photographs show that the Tai Po Road is the only completed highway in the area at this time, although its junction with the future Tai Woh Ping Road is under construction. The cutting for the Tai Po Road in the vicinity of the subject site is some way south of the present cutting, bordering what is now the eastbound carriageway of Tai Po Road. The cutting is relatively steep, and apparently unsurfaced, with a mid-height berm. There appears to be a sub-vertical discontinuity (fault?) around the cuttings mid-length, which can be traced for 30 m or so beyond the cutting’ s crest towards the northeast. The section of cut to the east of the change, and above the berm, appears to have failed, and a distinct scar is visible in this area. Smaller similar shaped scars are also present at the cutting crest in the west of the cutting, and in a small cutting for a cut-off drain above the main scar. These scars may however represent sites of corestone removal. A second photolineation is distinguishable running on an almost east-west axis, intercepting the cutting as its western extremity. Much rill erosion is evident below this lineation at the western end of the cutting.

1963 By 1963 the Tai Woh Ping Road had been completed, as had the service reservoir above. Tai Po Road has been duelled, and the subject cutting excavated back, with the construction of two equidistantly spaced lateral drains across its face. No failures are apparent, although erosion is apparent at the toe of the cut. The face is bouldery with numerous apparently random corestones. A boulder/corestone group is left on the crest in the western half of the cutting.

1969 The crest of the cutting is being developed as a sitting out/viewing area at this time, with the western boulder group being incorporated as a feature.

1973 A change to the cut face drainage arrangements has been made with the upper lateral drain now only extending across the western half of the cutting, before dropping vertically to link with the lower lateral drain. The dual carriageway north of reservoir is under construction.

1975 No significant change.

1976 No significant change.

1977 No significant change.

1978 The cutting face appears to have been regulated and shotcreted, with just a few bouldery protrusions.

1979 No significant change.

1980 No significant change.

1981 No significant change.

1984 Lorry parking (normal to the road) is evident along the southern verge of Tai Woh Ping Road between the road and the sitting out area. Vegetation is becoming established through the shotcrete an along drains, including a tree in the middle west of the cutting face.

1986 No significant change.

1987 No significant change.

1988 Vague lineations are visible in the reservoir cut face, possibly representing the western (and eastern?) release planes.

1990 No significant change.

1991 No significant change.

1992 No significant change.

1993 The 1993 photographs show that the previously vegetated cut slope above the old road had been stripped of vegetation and possibly re-shotcreted.

1995 No significant change.

1996 By 1996, the cutting had been trimmed back to provide an additional access lane at the toe, reducing the width of the sitting out area alongside the old road at the crest.

1997 By November 1997, the slope had been trimmed again to its pre-failure profile, with earthworks still ongoing. The older upper cutting appears to have been freshly re-shotcreted at this time.

A3. PAST INSTABILITY

There is some evidence to suggest former instability in the original Tai Po Road cutting, based on the 1949 photographs. Although some of the scars evident in and around the cut face at this time may represent overbreak of excavation due to the presence of boulders/corestones, there is considered to be a possible planar structure controlled type failure in the upper eastern part of the cutting at this time. The position of this feature, relative to the landslide is shown in Figure 4 in the main report.

A4. FILL

No observations of fill being placed in the immediate vicinity of the landslide site have been made.

A5. SURFACE HYDROLOGY

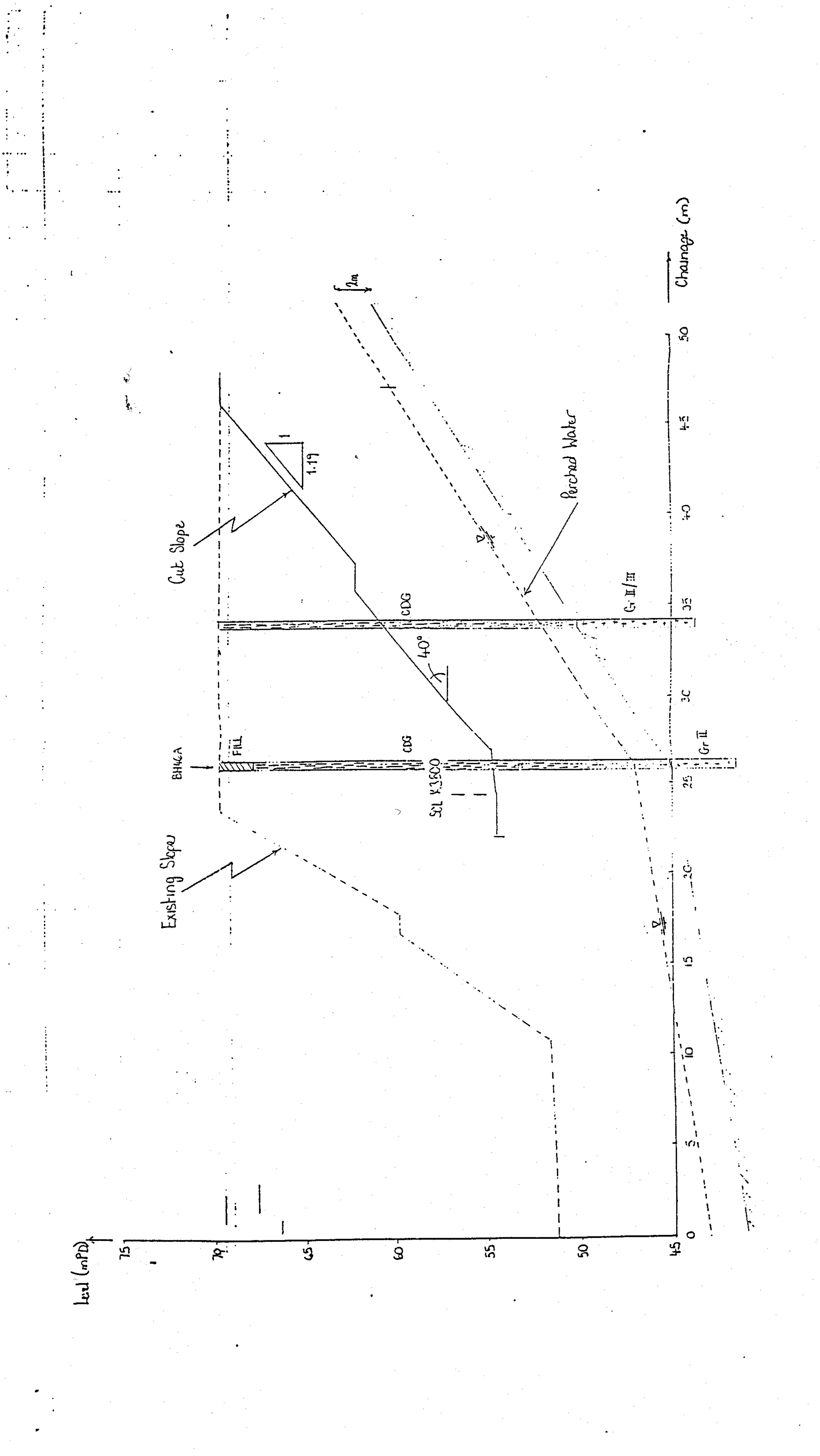
Aerial photography confirms that the former Tai Woh Ping Road drains surface water towards the landslide site from a relatively large catchment area.

A6. AERIAL PHOTOGRAPHS VIEWED

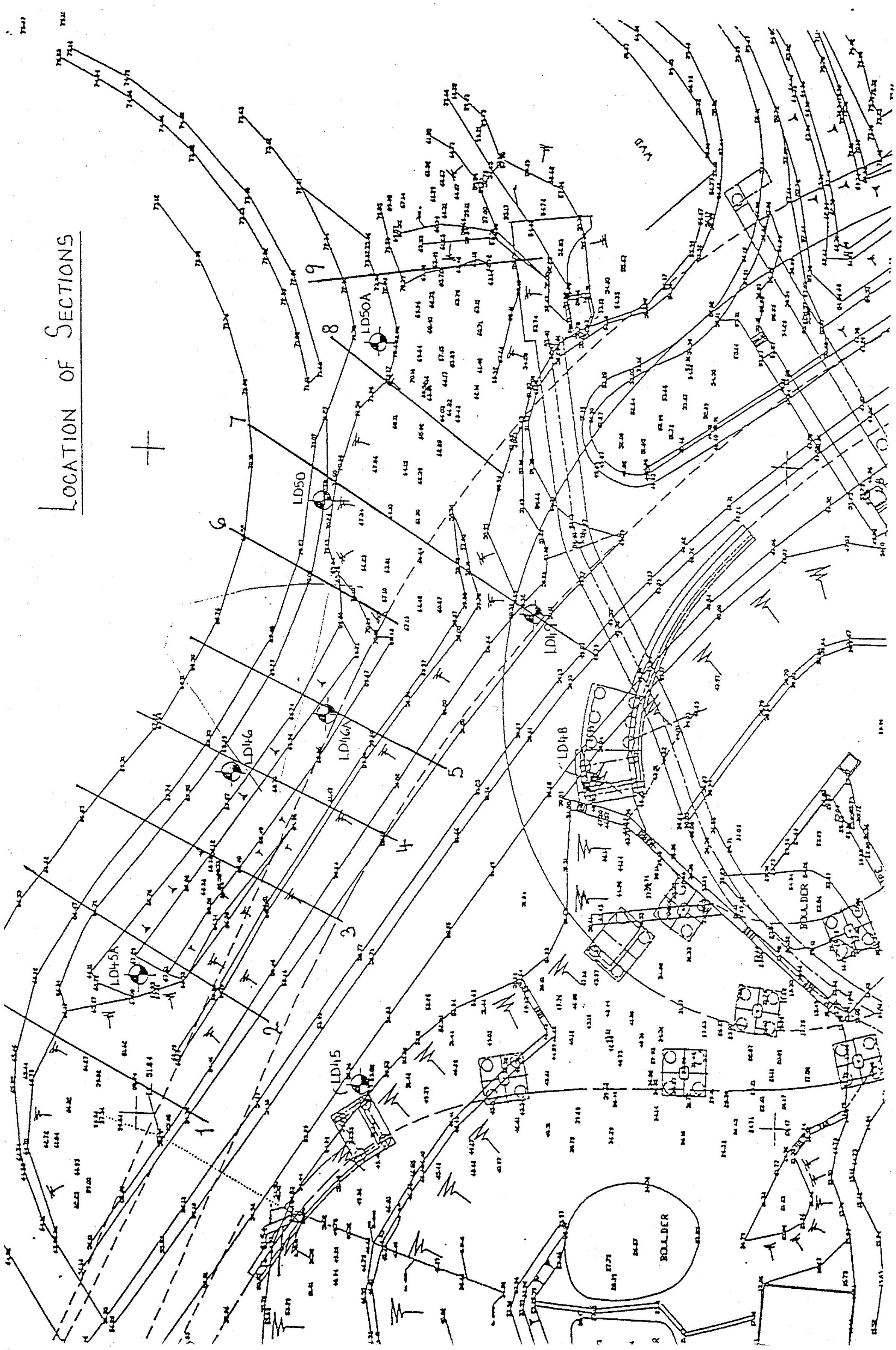
Year	**Altitude (feet)**	**Aerial Photograph Ref. Nos.**	
1949	8,000	Y01783	Y01784
1963	2,700	Y08070	Y08071
1967	6,250	Y13417	Y13418
1968	3,000	Y14273	
1969	1,800	Y14908	Y14909
1973	1,800	5372	5373
1975	2,500	11508	11509
1976	1,900	14700	14701
1977	2,000	19249	19250
1978	4,000	23982	23983
1979	4,000	27322	27323
1980	4,000	30116	30117
1981	5,500	36598	36599
1984	4,000	56987	56988
1986	4,000	A04469	A04470
1987	4,000	A09536	A09537
1988	4,000	A14662	A14663
1990	4,000	A20916	A20917
1991	4,000	A27508	A27509
1992	4,000	CN18908	CN18909
1993	4,000	CN13395	CN13396
1994	4,000	CN11323	CN11324
1995	3,500	A39256	A39257
1996	4,000	A35273	A35274
1997	4,000	A30445	A30446

APPENDIX B

EXTRACTS OF SLOPE STABILITY ANALYSIS FROM DESIGN REPORT BY PYPUN/ARUP CONSULTANTS (1994)



SECTION 5 (Ch. 817.00)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LD50A
LD50
LD46A
LD46
LD45A
LD45
LD48
BOULDER
BOULDER
1
2
3
4
5
6
7
8
9

ARUP - Ove Arup & Partners Hong Kong

LUNG CHEUNG & CHING CHEUNG ROAD
Slope SO56 (Revised Profile)
Section 5 (Ch.817.0)

Job No.	Sheet No.	Rev.
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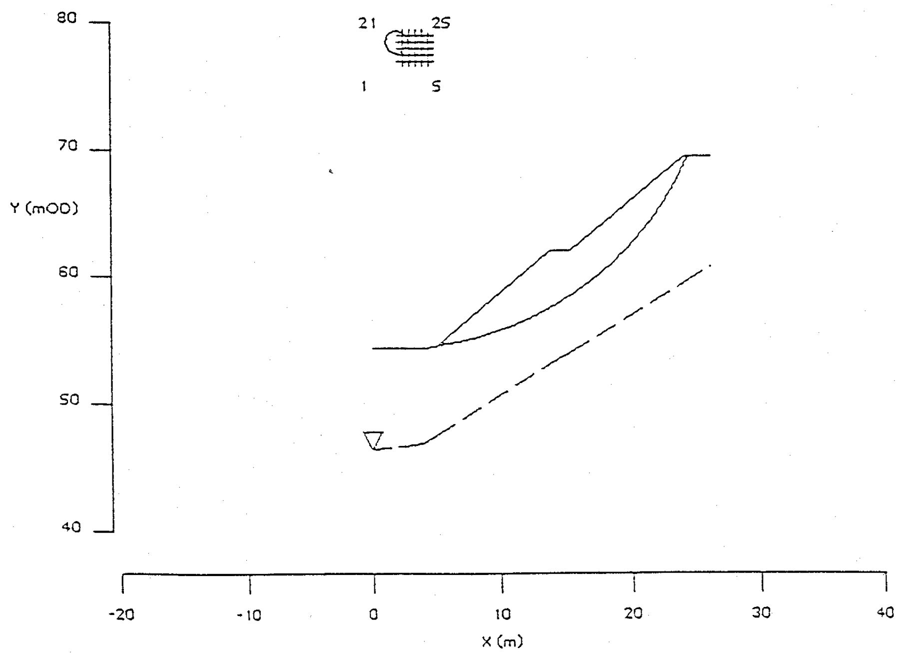
Org. Ref.		
Made by IA	Date 3-Dec-94	Checked JL



:ALE 1: 500

Circular slip surfaces drawn: 1

X (m)	Y (mOD)	Radius (m)	Weight (kN)	FOS	Drawn	Grid
2.00	78.50	24.00	1102	1.612	YES	---
2.50	79.00	24.00	1057	1.616	NO	21
2.50	78.50	24.00	1249	1.615	NO	16
1.50	78.50	24.00	955	1.618	NO	---
3.00	79.00	24.00	1201	1.621	NO	22
2.00	79.00	24.00	915	1.622	NO	---
2.50	77.50	23.00	1123	1.626	NO	6
3.00	78.00	23.00	1081	1.628	NO	12
2.00	77.50	23.00	979	1.629	NO	---
3.00	78.50	24.00	1397	1.632	NO	17

Centre of slip with lowest FOS is ringed

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DATA FOR ANALYSIS

SOIL DATA

Stratum No.	Description	Bulk Unit Weight (kN/m3) Above GWL	Bulk Unit Weight (kN/m3) Below GWL	Shear strength parameters drained / undrained	Phi' (deg) / c0 (kN/m2)	c' (kN/m2) / y0 (mOD)	k (kN/m2/m)	cu/p' ratio
1	CDG	20.00	20.00	DRAINED	42.00	5.00		

LEVELS OF TOPS OF STRATA AND PHREATIC SURFACES (mOD)

Stratum No.	Grid X Coordinates (m) ----> 0	2.50	4.00	5.00	13.80	15.30	24.00	26.00
1	54.40	54.40	54.50	54.60	62.10	62.10	69.50	69.50
...								
GWL	46.40	46.78	47.00	47.63	53.15	54.09	59.55	60.80

'ION METHOD

actor of safety on : SHEAR STRENGTH

Direction of slip : DOWNHILL

Minimum number of slices = 10

Minimum slip weight (kN) = 20

Method : JANBU VARIABLY INCLINED INTERSLICE FORCES

Maximum number of iterations = 60

Horizontal acceleration (%g) = 0

Initial distribution of surface loads : NO

GROUND WATER DATA

Pore pressure distribution type : HYDROSTATIC DISTRIBUTION
head equivalent to depth below phreatic surface

Maximum soil suction (m head of water) = 0

Unit weight of ground water (kN/m3) = 10.00

Number of phreatic surfaces = 1

SURFACE LOADS

None specified

C AR SLIP SURFACE DATA

ircle centre specification : GRID

Bottom left of grid x1 (m) 2.50
y1 (mOD) 77.00

Centres on grid: number in X direction 5 spacing (m) 0.50
number in Y direction 5 spacing (m) 0.50
extend grid to find minimum F.O.S. : YES

Circle radius specification : DEFINED RADII
initial value of radius incremented until circle invalid

Circle radii initial radius (m) 22.00
increment (m) 1.00

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ORST CASE OF THE CIRCULAR SLIP SURFACES ANALYSED: WATER CASE 1 OF 1
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ETAILED OUTPUT
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entre coordinates X(m) = 2.00 Y(mOD) = 78.50 Radius (m) = 24.00

terations =	22	Horizontal acceln. (%g) =	0
et vertical force (kN) =	1	Slip weight (kN) =	1102
et horiztl. force (kN) =	2	Disturbing moment (kNm) =	14551
		Restoring moment (kNm) =	23459

actor of safety = 1.612 F.O.S. Tolerance = 4.89360E-5

Point	Slip surface coordinates X(m)	Y(mOD)	Pore pressure u(kN/m2)	Interslice forces (kN) vertical T	horizontal E	E(u)
1	5.12	54.70	(-70.00)	0	0	0
2	6.94	55.01	(-61.71)	5.2	15.1	0
3	8.72	55.46	(-54.99)	15.2	40.1	0
4	10.47	56.04	(-49.86)	28.5	65.9	0
5	12.16	56.76	(-46.37)	42.6	86.0	0
	13.80	57.60	(-44.54)	54.1	96.0	0
[illegible]	15.30	58.52	(-44.34)	60.5	95.5	0
[illegible]	16.69	59.52	(-45.61)	62.2	88.3	0
[illegible]	18.01	60.62	(-48.33)	58.3	75.1	0
10	19.25	61.81	(-52.47)	49.2	58.2	0
11	20.40	63.09	(-58.03)	36.2	40.1	0
12	21.45	64.44	(-64.96)	21.7	23.5	0
13	22.41	65.87	(-73.23)	8.5	10.3	0
14	23.26	67.36	(-82.80)	-0.6	1.8	0
15	24.00	68.91	(-93.63)	-3.0	-1.9	0
16	24.25	69.50	(-97.99)	-1.5	-2.0	0

Slice number	Strength Parameters c' (kN/m2)	Tan phi'	Pore pressure u(kN/m2)	Slice weight W(kN)	Forces on base (kN) Normal P	Shear S
1	5.00	0.9004	0	22.4	24.7	31.5
2	5.00	0.9004	0	63.4	65.1	67.8
3	5.00	0.9004	0	96.5	96.0	95.7
4	5.00	0.9004	0	121.5	117.2	114.7
5	5.00	0.9004	0	138.2	128.5	124.9
6	5.00	0.9004	0	121.1	108.9	106.9
7	5.00	0.9004	0	102.3	88.7	88.4
8	5.00	0.9004	0	99.5	81.9	82.3
9	5.00	0.9004	0	91.9	71.4	72.9
10	5.00	0.9004	0	80.3	58.5	61.2
11	5.00	0.9004	0	65.8	44.6	48.7
12	5.00	0.9004	0	49.3	31.0	36.5

Slice number	Strength Parameters c' (kN/m2)	Tan phi'	Pore pressure u(kN/m2)	Slice weight W(kN)	Forces on base (kN) Normal P	Shear S
13	5.00	0.9004	0	32.2	18.9	25.6
14	5.00	0.9004	0	15.6	9.0	16.7
15	5.00	0.9004	0	1.5	1.2	4.3

Slice n	Surface load (kN/m2) vertical Q(V)	horizontal Q(H)	Water pressure on ground surface (kN/m2) vertical ug(V)	horizontal ug(H)
1	0	0	0	0
2	0	0	0	0
3	0	0	0	0
4	0	0	0	0
5	0	0	0	0
6	0	0	0	0
7	0	0	0	0
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0	0	0	0
14	0	0	0	0
15	0	0	0	0