SECTION 2: DETAILED STUDY OF THE LANDSLIDE ON SLOPE No. 11NW-A/FR84 BELOW CASTLE PEAK ROAD NEAR KAU WAH KENG VILLAGE

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FOREWORD

This report presents the findings of a detailed study of the landslide (GEO Incident No. MW 2000/01/07) on slope No. 11NW-A/FR84 above Kau Wah Keng Village, below Castle Peak Road, Kwai Chung, which was reported on 14 January 2000. Debris from the failure was transported along the natural stream course leading to the Kau Wah Keng Village. There were no casualties as a result of the slope failure.

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

The report was prepared as part of the 1999 Landslide Investigation Consultancy (LIC), for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED) under Agreement No. CE 101/98. This is one of a series of reports produced during the consultancy by Fugro Maunsell Scott Wilson Joint Venture (FMSW).

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1. INTRODUCTION

On 14 January 2000, a failure (GEO Incident No. MW2000/01/07) was noted to have occurred on slope No. 11NW-A/FR84, above Kau Wah Keng Village, below Castle Peak Road, Kwai Chung (Figures 1 and 2 and Plate 1). The exact time of the failure is unknown. Debris from the failure was transported along a natural stream course leading to Kau Wah Keng Village. There were no casualties as a result of the incident.

Fugro Maunsell Scott Wilson Joint Venture (FMSW), the 1999 Landslide Investigation Consultants, commenced a detailed study of the failure in January 2000 for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED) under Agreement No. CE 101/98.

The key objectives of the detailed study were to document the observations and facts about the incident, present relevant background information and establish the probable causes of the failure. The scope of the study comprised site inspections, detailed site mapping, limited ground investigation, desk study and analysis. Recommendations for follow-up actions are reported separately.

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

(a) review of relevant documentary records relating to the site history and the sequence of events leading to and following the slope failure,

(b) aerial photograph interpretation (API),

(c) mapping of the failure scar,

(d) limited ground investigation works,

(e) analysis of rainfall data, and

(f) diagnosis of the probable causes of the slope failure.

2. SITE DESCRIPTION AND DEVELOPMENT

2.1 Site Description

Slope No. 11NW-A/FR84 is located below Castle Peak Road about 150 m to the northeast of Kau Wah Keng Village (Figure 1). The feature, which comprises a fill slope portion and a retaining wall, was formed by filling over the valley and natural stream course at this location to provide support to Castle Peak Road. The fill slope portion is about 16 m high by 50 m long with a maximum depth of fill of about 9 m. The concrete retaining wall, which runs along the toe and southern portion of the fill slope, is about 70 m long with a maximum height of 6.5 m (Figure 2).

The fill portion has been formed in 2 batters, each about 8 m high and separated by a 1.5 m wide berm. Each batter is covered with vegetation and mature trees and dips southwestwards at a gradient of 1 in 1.6 (32°). Surface drainage channels traverse the mid-portion of each batter, the intermediate berm and the toe of the fill slope (Figures 2 and 3).

In accordance with the record drawings, the retaining wall along the toe and southern portion of the fill slope comprises two types of construction, namely a mass concrete gravity wall and a reinforced concrete L-shaped retaining wall. The retaining wall along the southern flank and in the central portion of the toe of the fill slope is of mass concrete construction, while the remaining sections are reinforced concrete L-shaped retaining walls.

A squatter hut is located at the toe of the retaining wall to the southeast of the stream course (Figure 2). The resident of the squatter hut recollected that the incident occurred in early 1999 (Section 3.3).

Construction activities along Castle Peak Road above the crest of the subject slope at the reported time of observing the incident included slope works under the Landslide Preventive Measures (LPM) Programme under CED Contract No. GE/98/10 (Figure 1). From site inspections carried out by FMSW, it is evident that the slope has not been affected by the LPM works.

2.2 Water-carrying Services

A 900 mm diameter steel watermain passes through the upper portion of the fill slope at a maximum depth of about 4 m below ground level (Figures 2 and 3). According to the Water Supplies Department (WSD), the watermain supplies water under gravity feed from Tsuen Wan Service Reservoir No. 1, at an elevation of 97.5 mPD, to Kau Wah Keng Pumping Station. The fractured section of watermain is located in the vicinity of a local low-point at an elevation of about 66 mPD before the watermain rises to the Kau Wah Keng Balance Tank at an elevation of about 91.4 mPD. The section of the watermain within the fill slope is housed in a leakage detection chamber that has three overflow pipes discharging into the surface drainage channel traversing the mid-portion of the upper batter, details of which are shown in Figures 2 to 4.

A 750 mm diameter steel stormwater drain passes along the base of the fill slope in the central portion of the feature and discharges into the natural stream course below the retaining wall at the toe of the fill slope. This drain connects to a cross-road culvert beneath Castle Peak Road (Figures 2 and 3).

2.3 Maintenance Responsibility

According to the Slope Maintenance Responsibility Information System of the Lands Department, the maintenance responsibility for slope No. 11NW-A/FR84 lies with the Highways Department (HyD).

Maintenance responsibility of the 900 mm diameter fresh watermain lies with the WSD. No records were found to identify the party responsible for maintenance of the leakage detection system after its construction, but in accordance with the WSD it was not "handed over to this Department for operation and maintenance". The WSD had no knowledge of the existence of the chamber prior to the subject landslide incident.

The Drainage Services Department (DSD) is responsible for maintaining the cross-road culvert and the 750 mm diameter stormwater drain.

2.4 Site History

2.4.1 General

The history of the site has been established through API, together with a review of documentary records. The salient information is summarised below and illustrated in Figures 5 to 7. A more detailed account of the API is presented in Table 1.

2.4.2 History of Development

In accordance with documentary records, the feature was formed in association with filling works along Castle Peak Road. Plans from the HyD of proposed widening and realignment works for Castle Peak Road, dated 1953, show the feature to comprise a 60 m wide by 20 m high fill slope supported by a 18 m long toe wall founded at an elevation of about +50 mPD (Figure 5).

The 1959 aerial photographs show an area of fill in the central portion of the present-day slope No. 11NW-A/FR84. In the 1963 aerial photographs, Castle Peak Road has been realigned and widened into a two-lane dual carriageway. The area of fill identified in the 1959 aerial photographs does not appear to have been extended significantly, but a small retaining wall can be seen at the toe of the fill slope, which was not observed in previous photographs. The 1963 aerial photographs also show a linear feature, characterised by a lighter tone, between the crest of the fill slope and the footpath, which suggests the possibility that the watermain located within the crest of the fill slope was installed during this period. No records relating to the date of installation of the watermain were found in the file search of the WSD's records.

In the 1978 aerial photographs, the fill slope portion was covered with rigid surface protection and surface drainage channels were constructed, possibly because of instability first noted in the 1974 aerial photographs (Section 2.4.3). In addition, the entire verge at the crest of the slope has a bright tone, suggesting recent slope works. The October 1982 aerial photographs show that the feature has been modified to its current configuration and that the fill portion was covered with grass. Between 1982 and 2000, the aerial photographs shows that the vegetation on the slope becomes denser and comprises mostly trees.

The feature was referenced as slope No. 11NW-A/F9 in the 1977/78 Catalogue of Slopes. The field sheet prepared by Binnie and Partners (B&P), dated May 1977, recorded the fill slope as being about 20 m high by 40 m long and standing at an angle of 35° to the horizontal. Slope No. 11NW-A/F9 is recorded as an old slope in the GEO's Slope Information System (SIS), which is located in the central portion of the present-day slope No. 11NW-A/FR84 (Figure 5) and has similar dimensions to that recorded in the B&P field sheet.

In 1977/78, B&P carried out a Phase II – Detailed Study for slope No. 11NW-A/FR9 (Section 2.5.1). In the design report, the feature boundary and location was as that depicted for slope No. 11NW-A/F9 in the SIS database. Plans and cross-sections in the report show slope No. 11NW-A/FR9 to comprise a 40 m wide by 33 m long by 20 m high fill slope portion, supported by a 2 m high mass concrete wall along the toe of the fill slope (Figures 5 and 6). The investigation found the feature to be substandard and recommended that it be upgraded, following which the fill slope and retaining wall were modified to the current configuration under the LPM Programme between 1981 and 1982 (Section 2.6).

The subject slope was inspected in October 1995 under the "Systematic Identification and Registration of Slopes in the Territory" (SIRST) project, and was registered as slope No. 11NW-A/FR84 (Section 2.5.2).

2.4.3 Past Instability

The fill slope portion shows evidence of previous instability in the aerial photographs taken in 1959, 1963, 1967, 1972, 1974, 1980 and 1982 (Figure 7). Of these, the scars noted in 1959, 1963, 1967 and 1982 appear to be predominantly associated with erosion. However, the scar noted in the 1974 aerial photographs affected the western flank of the fill slope and may have involved a degree of mass displacement. The majority of the scars were located at the crest of the northwest and central portions of the fill slope.

Two distinct scars are noted on the aerial photographs taken on 9 December 1999 and on 11 December 1999 (Plates 2 & 3), which correspond to the main scar and lower scar observed by FMSW in January 2000 (Section 3.2 and Figure 7). These scars were not present in the aerial photographs taken on 8 February 1999, which indicates that the incident occurred sometime between February and December 1999. This is consistent with the squatter living below the fill slope, who stated that the failure occurred in early 1999 (Section 3.3). Based on the aerial photographs, the main scar appears to be limited in extent to the slope area between the berm and U-channel in the upper batter of the fill slope, which was severed by the time the slope was first inspected by FMSW in January 2000. This indicates that further significant instability could have occurred between December 1999 and January 2000.

2.5 Previous Studies

2.5.1 Binnie & Partners Study (1978)

The slope was assessed by B&P in 1978/79 (Binnie, 1978), at which time it was referenced as slope No. 11NW-A/FR9 (Section 2.4.2). As part of the study, ground investigation works comprising two trial pits and one drillhole were carried out. The drillhole was located at the crest of the slope and identified granitic fill to a depth of 9.6 m below ground level. A total of seven insitu dry density tests were undertaken in the trial pits at depths of between 0.3 m and 1.9 m, which indicated very loose fill with dry densities in the range of 0.94 Mg/m^3 to 1.19 Mg/m^3. Two piezometers were installed in the drillhole, with one piezometer tip positioned at the base of the hole at a depth of 11.5 m below ground level and the other positioned at the interface between the fill and the original ground level at a depth of 9.6 m. Groundwater monitoring was carried out between March 1978 and April 1979. The highest recorded water level occurred at the original ground level. The report did not mention whether piezometer buckets had been installed.

The study identified that both the fill slope and the retaining wall were sub-standard with regard to slope stability and sliding stability respectively and were in need of upgrading. The following recommendations were made:

(a) provide suitable subsurface and surface drains to control runoff and rises in the groundwater level,

(b) recompact the surface 3 m of fill to reduce the risk of liquefaction failure,

(c) regrade the slope to a gradient of 1 on 1.7 (30°) to improve the factor of safety against slip failure,

(d) raise the retaining wall at the slope toe by 2 m to accommodate regrading works,

(e) place grass by hydroseeding on the recompacted slope surface to reduce erosion, and

(f) inspect and maintain the culvert at the base of the slope to ensure it does not become blocked.

The report also identified that water-carrying services, including a 0.9 m diameter watermain, lay at the crest of the slope. Recommendations to “Relay services and mains (including the 900 mm water mains) in drained channels in accordance with the Independent Panel Report” were made in B&P’s report. The 750 mm diameter stormwater drain was noted as being in satisfactory condition and recommendations were made for regular inspection and maintenance only.

More intensive ground investigation works, comprising 3 drillholes and 34 trial pits, were carried out between November 1979 and July 1980 under PWD Contract No. 416/79 as part of the detailed design for slope upgrading works (Enpack, 1979 & 1980). Insitu density tests in the trial pits gave a range of dry densities between 1.28 Mg/m^3 and 1.41 Mg/m^3 for the top 2.5 m thick layer of fill. The drillholes identified fill to a maximum depth of 8 m and 3.7 m at the crest and toe of the slope respectively (Figure 6).

2.5.2 SIFT and SIRST Studies

In mid-1992, the GEO initiated a consultancy agreement, entitled “Systematic Inspection of Features in the Territory” (SIFT), to search systematically for slopes not included in the 1977/78 Catalogue of Slopes and to update information on previously registered features, by studying aerial photographs and limited site inspections. The slope was categorised as a Class B2 feature, i.e. “Assumed to have been checked by GEO (assumed formed post 1977) or Housing Department or studied to Stage 2 or equivalent”. The SIFT report also had the following remarks “Road continually upgraded since before 1945, main fill deposited prior to 1978. Between 1980 and 1984, drainage was added”.

In 1994, the GEO commenced a consultancy agreement, 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 slope on 9 October 1995 and reported no signs of leaking services, slope distress or previous failure. The concrete retaining wall at the slope toe was noted to be in fair condition and seepage was observed near the wall toe. No signs of wall distress or previous failure were reported along the wall.

2.6 LPM Works

The slope was upgraded to the present-day configuration under the LPM Programme between March 1981 and April 1982. These works were designed by B&P following their detailed study in 1978 (Section 2.5.1). The upgrading works comprised the recompaction of the top 3 m of fill on the slope and construction of a new retaining wall at the slope toe (Figure 3). The 0.9 m diameter watermain at the crest of the slope was encased in a new concrete trough that was intended to act as a leakage detection system (Figure 4). The trough comprised three separate sections, each with an overflow pipe discharging to the slope drainage channel at the mid-height of the upper batter. No details were shown on the record drawings as to how the overflow pipes were to connect to the trough. The finished slope surface was vegetated by means of hydroseeding.

2.7 Slope Inspections

2.7.1 Fill Slope Inspections by Binnie and Partners

In 1993, the GEO initiated a programme of inspection of 122 fill slopes re-constructed in or before 1981. The purpose of the project was to review the slope surface conditions, conditions of slope drainage systems, access problems, maintenance responsibilities, and assess the need for special monitoring and remedial works. It was also intended to identify possible areas for improvement or modifications in future designs. All of the 122 fill slopes, including the subject slope, were inspected by B&P during the period of May 1983 to January 1984. The inspection for the subject slope noted that the feature was in moderate condition, with very few defects that were of a minor nature.

2.7.2 Inspections by the Highways Department

The HyD has been undertaking slope inspections and maintenance work for slope No. 11NW-A/FR84 since 1995. Some record photographs showing the slope condition as observed in the course of Engineer Inspection (EI) and Routine Maintenance Inspections

(RMI) are enclosed in Appendix A. The EI and RMI reports referred to the feature as slope No. 11NW-A/F9, the boundary of which is larger and located further to the northwest of the present-day slope in reports prior to 1999 (Figure 5). Key observations are discussed below.

In December 1995, the first Engineer Inspection of the feature was undertaken by Fugro Mouchel Rendel Joint Venture (FMR), the consultants appointed by the HyD for a project entitled "Roadside Slope Inventory and Inspections" to carry out Engineer Inspections of about 4 000 HyD's slopes. The feature was referenced as slope No. 11NW-A/F9 in the inspection report and the feature boundary was as that depicted in the HyD's files (Figure 5).

The Engineer Inspection report identified that the full extent of the slope to be inspected and maintained had not been established and that the feature was "…a HyD defined "Cannot be Located" slope". The fill slope was classified as being in a good state of overall maintenance, although it was noted that the drainage channels and catchpits were blocked. The retaining wall at the toe of the fill portion of the feature does not appear to have been inspected during the EI as no details were recorded in the EI report.

RMI and Routine Maintenance Works (RMW) have been undertaken at between six month and yearly intervals from 1996 onward. The first recorded RMI was undertaken on 1 November 1996, and record photographs attached to the inspection report showed strong water flow discharging from one of the overflow pipes connected to the leakage detection chamber (Appendix A, Plate A). The RMI report noted the following:

(a) many of the surface drainage channels were cracked and badly choked and some had been severed in places,

(b) extensive voiding was present behind the small upstand wall along the toe of the lower batter indicating erosion and/or lateral movement of the upstand wall, and

(c) loose and wet areas of the slope surface were observed.

Based on the above, an immediate EI was identified as being necessary, but no records of the recommended EI having been carried out could be located in this study. Subsequent RMI's conducted in April 1997, November 1997, April 1998 and January 1999 made no recommendations for conducting an immediate EI.

In accordance with the RMI report, maintenance works to the slope and retaining wall, including the clearance of blocked channels and repair of damaged or cracked channels, were reportedly completed on 15 March 1997.

The next RMI in April 1997, identified very similar signs of distress across the feature as noted in the previous RMI. In particular, severed drainage channels and extensive voiding behind the upstand wall were still reported. There was no record of strong water flows discharging from the overflow pipe from the leakage detection chamber as noted in the previous RMI. The corresponding RMW to the observed distress was reported to have been completed by 23 September 1997.

RMI undertaken in November 1997 and April 1998, again identified cracked and blocked channels. According to the recorded observations and photographs taken as part of these inspections, the defects observed in April 1997 had previously been repaired. In particular, the extensive voiding to the upstand wall had been backfilled and a shotcrete apron constructed at the upslope portion.

The RMI of 14 January 1999 noted that apart from some unplanned vegetation and debris blocking the drainage channels, there were no obvious signs of distress that could be identified. The corresponding RMW were noted as being complete by 19 March 1999. Record photographs attached to the inspection report indicated slight seepage from one of the overflow pipes and on the slope surfaces in the lower batter.

2.7.3 Inspections by the Water Supplies Department

According to the WSD, the watermain has been included in the list of pipes for leakage detection under WSD Contract No. 16/WSD/97. An acoustic detection test carried out on the watermain on 25 March 1999, under the supervision of Maunsell Consultants Asia Ltd. and Camp Dresser & Mckee International Inc. (M & CDM), indicated that the watermain was in "normal/satisfactory situation with no leaks existing". M & CDM did not report any irregularities following their visual inspection of the slope carried out on 21 July 1999.

3. THE LANDSLIDE INCIDENT

3.1 General

In late December 1999, the villagers of Kau Wah Keng Village observed an unusually high sediment content in the water flowing along the service channel through the village. This had led to an accumulation of sand and mud partly blocking the service channel. This issue was brought to the attention of the District Office of Kwai Tsing. The complaint was then forwarded to the Resident Site Staff for the LPM works being undertaken by Scott Wilson Hong Kong Limited (SW) under Contract No. GE/98/10 above Castle Peak Road (Figure 1). In response to the complaint, the upstream area of the Kau Wah Keng Village was inspected. During the inspection, which was undertaken on 14 January 2000, it was noted that the central portion of the upper batter of slope No. 11NW-A/FR84 had failed (Figure 2 and Plates 1 & 4). Based on observations made during SW's inspection, the failed area was about 15 m long by 5 m wide by 2.5 m deep. At the time of inspection, it was noted that a large quantity of water was discharging into the failed portion of slope from the 900 mm diameter watermain exposed below the main scar (Plates 5 and 6).

3.2 Field Mapping of Distress

FMSW first inspected the site on 26 January 2000, at which time the 900 mm diameter watermain had been repaired. A further inspection was made on 9 March 2000, to undertake detailed mapping of the distressed slope. The details of field mapping are presented in Figures 7 and 8. Field mapping identified that the incident observed by SW comprised a large depression with steep sub-vertical sides (hereafter referred to as the main scar), that measured

17 m long by 8 m wide by 4 m deep (maximum), which indicates that the failure may have enlarged slightly since SW's original inspection on 14 January 2000. It is estimated that the volume of material originating from the main scar was about 300 m^3.

The width of the main scar narrowed towards the base, where a large subsurface void of about 1.5 m in diameter was identified (Plate 7). It was evident that the void extended further downslope and significant surface distress was noted along the base of the lower batter below and to the west of the main scar (Figure 7). This took the form of two voids and associated surface distress, one located below the main scar (Plate 8) and the other located some 15 m to the west (Plate 9). In the vicinity of these voids, there was evidence of surface washout/erosion. Also, the small upstand wall along the toe of the lower batter had tilted outwards over a length of about 15 m (Plate 10). Slope wash material had been deposited along the crest of the retaining wall indicating that large volumes of water had discharged from the surface voids and cascaded over the top of the wall, carrying material with it.

Based on the spatial relationship between the mouth of the void in the main scar and the voids and distress observed downslope, it is apparent that the two areas of distress are linked. On this basis, it is estimated that the total disturbed area measured 28 m long by 20 m wide with an overall disturbed volume of about 700 m^3, assuming an average depth of between 2 m and 3 m. Cross-sections through the disturbed slope are presented in Figure 8.

With the exception of a small shallow scar immediately upslope of the main scar, there did not appear to be any signs of major instability within the feature, pedestrian walkway or Castle Peak Road.

The retaining wall at the base of the slope was inspected for signs of distress. The weepholes along the wall were blocked, except for a few along the bottom row. Some hairline cracks were noted over the face of the wall. Overall, the condition of the wall appeared to be fair.

Inspection of the watermain and concrete trough exposed at the crown of the main scar showed that repair works had recently been carried out. A U-shaped hole of about 150 mm in diameter was present at the bottom of the trough (Plate 11). This hole is located at a construction joint in the trough approximately below the repaired portion of the watermain. The hole appeared old and exposed corroded reinforcement. Sub-vertical striations were evident on the exposed concrete surface of the hole, suggesting that mechanical action was probably used to form the hole.

FMSW were advised by the WSD that the U-shaped hole in the bottom of the concrete trough was present when they first inspected the site on 14 January 2000, following the report of the burst watermain. The WSD further noted that no complaints/reports on leakage of the 900 mm diameter watermain were received preceding the incident. No records were found in the WSD's files to show the construction details of the foundation to the watermain.

Another hole was observed at the eastern end of the concrete trough. This was a square hole about 250 mm wide that had been formed in the upstand wall to the concrete trough to provide an outlet connection to the overflow pipe (Plate 12). Inspection of the hole identified corroded reinforcement and that the internal sides of the hole were relatively smooth indicating that the hole had probably been formed by boxing out the section during the original concreting works. The outside face of the upstand wall was also smooth and had no reinforcement projecting from it. A section of concrete drainage pipe lay on the floor of the main scar below the hole, which was probably the section of pipe that used to connect into the concrete trough. Inspection of the pipe end that probably used to connect to the trough revealed that the joint had been formed by inserting the pipe into the outlet hole and forming a concrete haunch at the junction between the two. No evidence of a positive key to ensure a water-tight joint could be identified. As noted above (Section 2.6), the detailing of the joint was not shown in the record drawings.

3.3 Events Following the Incident

After identifying the incident, SW immediately reported the broken watermain to the WSD for urgent action. An inspection team from the WSD visited the site on 14 and 15 January 2000. They noted a 100 mm long crack along the edge of a welded joint in the lower quarter of the pipe on the downslope side of the watermain (Plates 13 and 14). The pipe was reported to be in sound working condition in general and the edges of the crack were recorded as being rough and relatively fresh. Remedial works, comprising the repair of the fracture and formation of a mass concrete buttress to support the concrete trough, were completed on 15 January 2000 (Plate 15).

FMSW interviewed the resident of a squatter hut located at the toe of the retaining wall, below the area of the distressed fill slope. The resident of the squatter hut, who claimed to have resided there since 1984, recalled seeing a large quantity of water and debris cascading over the top of the retaining wall in early 1999, but could not give precise details as to the volume and nature of flow involved. He reported that the flow of water over the retaining wall continued until January 2000, when it stopped abruptly.

Following the incident, FMSW interviewed the village representative from Kau Wah Keng Village, during which information came to light regarding details of the service channel flowing through the village. In accordance with the village representative, the service channel was constructed along the line of an existing stream course between 4 and 5 years ago, at which time only moderate stream flow had been observed along the channel. Sometime after the completion of the service channel the flow condition increased, such that it was strong enough to sweep away rubbish and waste in the channel. Following the repair of the broken watermain by the WSD on 15 January 2000, the villagers observed that the flow condition had reduced significantly.

In mid-May 2000, the HyD carried out repair works to the distressed slope. These works were substantially completed by 26 July 2000. The main scar was backfilled with rock fill and concrete was pumped into the erosion voids at the bottom of the main scar. The distressed slope surfaces were protected by shotcrete after removal of loose debris. The broken U-channels on the slope and at the toe were re-constructed and the overflow pipes were reinstated (Plate 16).

4. SUBSURFACE CONDITIONS

4.1 Geology

With reference to the published geological map (GEO, 1986), the site is underlain by Mesozoic fine-grained granite (Figure 9). Coarse-grained granite outcrops in the areas to the northwest and southeast of the site and superficial debris flow deposits cover the area to the south. The geology of the site is not considered to have been significant in the incident.

4.2 Ground Investigation

4.2.1 Previous Ground Investigation and Groundwater Monitoring

Previous ground investigation works, comprising a total of 4 drillholes and 36 trial pits were carried out between 1978 and 1980 as part of the detailed study undertaken by B&P (Figure 10). Details of these works are discussed in Section 2.5.1.

4.2.2 Present Ground Investigation

Investigation works under the current study comprised 5 trial pits and 80 GCO probe tests, the locations of which are shown in Figure 10. Trial pit logs are provided in Appendix B.

Trial pits were excavated to investigate the type, nature and thickness of materials at various locations over the slope. Bulk samples were collected from the trial pits to enable Proctor tests, soil classification and index tests to be undertaken. In-situ density tests were also performed in the trial pits.

A total of 8 in-situ density tests were undertaken within the 3 m thick re-compacted cap of fill material. The test results indicate that the degree of compaction in this zone is over 95%. A total of 3 in-situ density tests were undertaken in the old fill layer beneath the 3 m thick re-compacted cap. The test results indicate that the degree of compaction of the underlying old fill is between 83% and 90.5%. There is no significant difference in nature between the re-compacted fill and the old fill in terms of moisture content, plasticity index and particle size distribution from the samples tested. A summary of the results is presented in Table 2.

GCO probes were positioned around the failed/distressed area in an attempt to identify the extent of the distressed zone associated with internal erosion. GCO probes were also distributed across the remaining areas of the fill slope, considered to be unaffected by the internal erosion associated with the leaking main, to establish the typical GCO probe blow count versus depth profile.

A total of 80 GCO probe tests were undertaken, of which 39 tests reached refusal (taken to be >100 blows per 100 mm) at a depth of <2.0 m. Of the remaining 41 GCO probe tests, 11 showed a strong indication of disturbance covering an area approximately 20 m wide by 15 m long that lay between the main scar and the lower scar above the retaining wall (Appendix C, Figures C2 and C3). The portions of the slope outside this area do not seem to have been affected by erosion forces and the slope failure itself, but GCO probe Nos. GP6B and AP14, located in the upper northwestern part of the fill slope by trial pit TP4, and GCO probe Nos. AP12 and AP13, located in the southeastern part of the fill slope by trial pits TP5 and TP2 respectively, did identify the possible local disturbance to depths of 3 m maximum (Appendix C, Figure C4).

A CCTV survey was carried out through the 750 mm diameter stormwater drain underlying the slope. The stormwater drain was shown to be in reasonable condition by the CCTV survey, with no evidence of significant cracking that could have led to a loss of fines from the fill slope above.

5. ANALYSIS OF RAINFALL RECORDS

The nearest GEO automatic raingauge No. N04 is located at Kwai Fong Lau, Cho Yiu Estate, about 1 km to the west of the subject feature. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the GEO.

In view of the fact that the exact date and time of the incident is uncertain, a detailed rainfall analysis is not possible. Instead, rainfall records have been analysed to determine the monthly rainfall for the years 1996 to 1999 (Figures 11 and 12).

It can be seen from Figure 11 that at the time of RMI in 1996, there had been little rainfall in the area for more than a month before the inspection. Accordingly, the significant discharge from the overflow pipe observed in the upper batter of the slope and wet slope surface in the lower batter were unlikely to have been the result of surface infiltration from rainfall. Also, based on previous groundwater monitoring data this is unlikely to have been due to a rise in the main groundwater table. This suggests that the probable source of water observed in the 1996 RMI was leakage from the watermain.

The monthly rainfall plots for the remaining years (Figures 11 and 12), show heavy rainfall between the months of June and August ranging from a minimum of 700 mm in June 1998 to a maximum of about 1150 mm in August 1999.

6. POSTULATED SEQUENCE OF EVENTS LEADING TO FAILURE

Based on the evidence available, it is considered that the likely sequence of events leading to the 2000 incident probably commenced with water from the leaking main entering the body of the fill slope through the 150 mm diameter U-shaped hole in the floor of the concrete trough. As advised by the WSD, the hole was present when they first inspected the watermain on 14 January 2000, shortly after the incident was reported. The hole appeared to be old and was located below the welded joint in the watermain where the fracture occurred, but the reason for its presence is unknown.

As the bottom of the concrete trough was formed at about 4 m below ground level at this location, water leaking through the hole in the trough would have penetrated into the loose fill below the 3 m thick recompacted cap. It is likely that the loose fill below the recompacted cap was prone to internal erosion through the loss of fines. Consequently, it is considered that over time the constant flow of water from the leaking main led to a degree of settlement below the trough (Figure 13 (a)).

The settlement-induced void in the loose fill layer would have led to a loss of support to the overlying material, causing the void to propagate upwards into the 3 m thick recompacted fill layer through the localised collapse of unsupported material. The lack of support to the recompacted fill layer would eventually bring to bear increased stresses onto the overflow pipe connecting to the leakage detection trough as the internal void enlarged (Figure 13 (b)). The additional stresses would cause the pipe joints and connections to open and allow more water to leak into the body of the fill slope, thereby speeding up the settlement and erosion processes (Figure 13 (c)).

Based on post-failure observations by FMSW of poor workmanship during construction to the connection between the overflow pipe and the leakage detection trough, it is unlikely that the connection between the two was watertight (Section 3.2), thereby providing another outlet for leaking water to infiltrate into the fill slope. Furthermore, the poor detailing of the connection between the overflow pipe and the leakage detection trough (Section 2.6) may have led to a concentration of stresses at this ‘weak’ connection and caused the eventual detachment of the overflow pipe from the leakage detection trough (Figure 13 (d)). Based on the RMI undertaken by HyD, it is considered probable that this occurred sometime between November 1996 and April 1997, because the significant water flow noted in the November 1996 inspection was not referred to in the April 1997 inspection report. However, the WSD have doubts as to the validity of this postulation because this would mean that the watermain had been leaking for a prolonged period of time. This postulation apparently contradicts WSD’s observations made of the pipe crack following the 2000 incident and information from the leakage detection test carried out by WSD’s consultant on the watermain on 25 March 1999 (Section 7).

Following the collapse of the overflow pipe, all water leaking from the main would have entered the body of the fill slope. A photograph taken on 15 January 2000 by the WSD, following the identification of the failed slope, shows water cascading out of the hole in the concrete trough where the overflow pipe used to connect (Plate 6). This constant, preferential sub-surface flow probably resulted in an increased rate of internal erosion and the development of erosion cavities and further surface distress at the toe of the fill slope (Figure 13 (e)).

The blocked weepholes along the retaining wall at the toe of the slope, as observed by FMSW (see Section 3.2), would have inhibited the leaking water from escaping through the wall. Water would have backed up behind the wall until it reached a level where it was able to discharge out of the fill portion of the slope above the wall, causing surface distress. This would explain the reason for the distressed upstand wall observed at the toe of the fill slope during RMI in November 1996 (Section 2.7.2). Furthermore, it is possible that the stability of the retaining wall may have been compromised by water backing up behind the wall to a level higher than the design groundwater table. In this respect, blockage of weepholes along the retaining wall would have exacerbated the extent of distress that has occurred.

It is conceivable that the layer of dense recompacted fill material forming the roof above the erosion cavity remained stable through arching effects, but as slope conditions deteriorated through continued erosion over time, the stability of the roof reached limiting conditions of equilibrium and eventually collapsed (Figure 13 (f)). The exact date at which the underground cavity collapsed is unknown, but from desk study records and API, it occurred sometime between July 1999 and December 1999. Therefore, internal erosion of the slope may have been ongoing for over two years.

Observations in December 1999, of unusually high sediment content in the service channel by the residents of Kau Wah Keng Village suggest that the last stage of failure may have been towards the end of 1999. However, this increased sediment content may alternatively have been related to construction activities associated with the LPM works ongoing above Castle Peak Road.

7. DISCUSSION

Based on the available information, it is probable that the failure of fill slope No. 11NW-A/FR84 involved significant internal erosion of fill material leading to the formation of a large sub-surface void and the eventual collapse of the near-surface materials. This diagnosis is supported by the absence of landslide debris corresponding to the formation of the failure scar. Furthermore, the very extensive nature of the internal erosion formed within the recompacted fill slope prior to the 2000 incident points to the fact that the water source initiating the internal erosion had been present and operating for a prolonged period of time.

Internal erosion is considered to have been the result of leakage from the 900 mm diameter watermain located at the crest of the slope. The watermain was housed in a concrete trough constructed as part of slope upgrading works in 1982. The trough was intended to act as a leakage detection system with any leaking water being conveyed into the slope surface drainage system via overflow pipes, but the presence of a sizeable hole at the base of the trough would have created the potential for any leakage to infiltrate into the fill slope.

Based on a detailed review of the available documentary information, it is considered possible that the watermain could have been leaking from as early as November 1996, when significant water flows were observed from one of the overflow pipes during the first RMI by the HyD. The report prepared following the November 1996 RMI recommended that an immediate EI be carried out, but no records of the recommended EI having been carried out could be located in this study. Moreover, no records were found to indicate that the November 1996 leakage was repaired prior to the 2000 incident. Subsequent RMI's conducted in April 1997, November 1997, April 1998 and January 1999 made no reference to and contained no details of water flowing from the overflow pipes and no recommendations were made for conducting an immediate EI.

The WSD had expressed reservations as to the reliability of the observations of the RMI's undertaken by HyD for the feature, as well as the postulation of detachment of the overflow pipe sometime between November 1996 and April 1997 leading to internal erosion for a prolonged period of time (Section 6). WSD suggested that the watermain could not have been leaking for such a long time, as they considered it unreasonable to implicate that the leakage detection work carried out by their consultant on 25 March 1999, reportedly using the latest technology, had failed to detect the leakage. WSD also noted that their consultant did not report any irregularities following a visual inspection of the slope on 21 July 1999.

Furthermore, WSD noted that at the time the leaking main was being repaired following the 2000 incident, the pipe was still in sound working condition in general and that the edges of the crack were found to be rough and relatively fresh, from which they opined that the pipe crack did not appear to them to be of 3 years of age. However, as noted before, other evidence collected during this study strongly points to the likelihood of leakage from the watermain over a prolonged period of time.

The exact reason as to why the joint weld at the underside of the watermain became open is not certain. It is possible that differential settlement of the loose fill layer beneath the surface 3 m recompacted fill induced tensile stresses at the bottom of the watermain where the welded joint was pulled open. Inadequate support to the watermain during construction of the leakage detection chamber may also have been a possible contributory factor.

8. CONCLUSIONS

Having regard to all the evidence and stakeholders' views collected from this study, in particular the photographic evidence of strong water flow from the leakage detection system during maintenance inspection and significant internal erosion of the fill slope as revealed by the post-failure ground investigations, it is postulated that the slope failure was most probably a result of settlement and internal erosion of the fill material due to prolonged leakage from the 900 mm diameter watermain buried in the upper batter of the slope. The roof collapse above the major subsurface erosion feature in the fill slope is considered to have occurred sometime between July 1999 and December 1999.

The slope was previously upgraded under the LPM Programme in 1982, during which time the slope was re-profiled to a shallower gradient and the top 3 m of fill was recompacted. As part of the LPM works, the watermain at the crest of the slope was encased in a concrete trough as part of the leakage detection system.

Discharge from the leakage detection system was observed during a Routine Maintenance Inspection by the Highways Department in November 1996. The effects of leakage were exacerbated by the presence of a 150 mm diameter hole in the bottom of the concrete trough below the watermain. Poor detailing of the concrete trough, i.e. inadequate connection between the overflow pipe and the outlet in the trough, probably also played a role in the failure by allowing additional concentrated water ingress into the fill slope.

The exact reason as to why the welded joint at the underside of the watermain became open is not certain. It is possible that differential settlement of the loose fill layer beneath the 3 m recompacted fill cap induced tensile stresses at the bottom of the watermain where the welded joint was pulled open. Inadequate support to the watermain during construction of the leakage detection chamber may also have been a possible contributory factor.

9. REFERENCES

Binnie (1978). Slope No. 11NW-A/FR9 Castle Peak Road, Kwai Chung. Special Investigation into Fill Slopes – Detailed Studies, Binnie & Partners (Hong Kong) for Geotechnical Control Office, Hong Kong, 21 p.

Enpack (1979). Final Site Report (Report No. 76). Landslide Study Site Investigation for Slope No. 11NW-A/FR9, Castle Peak Road. Enpack (HK) Ltd., 29 p.

Enpack (1980). Final Site Report (Report No. 76 A). Landslide Study Site Investigation for Slope No. 11NW-A/FR9, Castle Peak Road. Enpack (HK) Ltd., 43 p.

GEO (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.

LIST OF TABLES

Table 1- Record of Aerial Photographs Studied (Sheet 1 of 4)

Year	Photograph Reference No.	Altitude (feet)	Observations (Refer to Figure 7)
1959	Y4617, Y4618	40000	Castle Peak Road has been constructed as a single-lane dual carriageway. Fill is noted below the road in the location of the present-day slope No. 11NW-A/FR84. At this time, the slope shows indications of filling by end tipping (i.e. little or no compaction) over an existing drainage line. An apron/verge is noted between the road edge and the crest of the slope. The verge has a bright tone, which may suggest recent activity such as excavation and/or filling in this area. The fill slope appears to be covered with thin vegetation and a small erosion/landslide scar is noted at the crest of the northwest part of the slope.
1963	Y8773, Y8774	3900	Castle Peak Road is now a two-lane dual carriageway. The fill slope does not appear to have been extended significantly. A small retaining wall is noted at the toe of the fill slope. Three small erosion/landslide scars are noted near the crest and mid-slope portions of the slope. Rill erosion is noted on the mid-slope scar suggesting a mainly erosion-type failure. A linear feature, characterised by a bright tone, is noted between the crest of the fill slope and the pedestrian walkway. This may represent buried services near the crest of the slope. A prop-supported pipeline is noted below the road about 130 m to the west northwest of the fill slope. The pipeline may be connected to that observed within the main scar, in January 2000.
1964	Y10961, Y10962	2700	The entire slope cannot be seen on the photographs. The vegetation on the slope appears to be denser. A small erosion/landslide scar is noted at the crest.
1967	Y13415, Y13416	6250	As above. An erosion/landslide scar is noted on the filled slope to the south of the scar noted in 1963 and 1964.
1972	261, 262 272, 273	Unknown	An erosion/landslide scar is noted in the northwest corner of the slope. Several boulders are also noted on the slope below the scar.

Table1- Record of Aerial Photographs Studied (Sheet 2 of 4)

Year	Photograph Reference No.	Altitude (feet)	Observations (Refer to Figure 7)
1973	5313, 5314	6000	Vegetation on the fill slope is denser.
1974	9781, 9782	12500	An erosion/landslide scar is noted along the western margin of the slope. The crest of the scar is located at the crest of the slope.
1976	14674, 14675, 14676	2000	The scar noted in 1974 is still apparent on the western margin of the slope. The upper portion of the scar is still exposed and several boulders can be seen towards the toe of the scar. A linear feature, characterised by a light grey tone, is noted between the crest of the slope and the pedestrian walkway, i.e. on the verge between the slope and road. The feature can be traced for the full length of the fill slope crest. It may represent buried services as noted in 1963.
1977	20247, 20248	4000	The vegetation on the slope is now denser and includes trees, except within the area of the scar noted in 1974 and 1976. The upper portion of the scar is still exposed but the trial is covered with grass.
1978	23986, 23987	4000	The slope has been covered with surface protection, i.e. chunam/gunite or shotcrete. Two down channels and two mid-slope, herringbone surface drains are located on the slope surface. Some trees are also noted along the southern part of the slope toe and at the transition zone with the natural slope at the northern boundary. It is not clear (due to the angle of the photographs) if the slope is supported by a retaining wall. The entire verge at the crest of the fill slope has a bright one, suggesting recent alteration in association with the slope works.
1980	30145, 30146	4000	A retaining wall is noted at the slope toe (and was probably part of the slope in 1978). A linear feature, characterised by a bright tone, is noted along the full length of the verge at the fill slope crest. This may suggest that the buried services have been altered. Some vegetation has grown through the surface cover of the slope.
1980	32850, 32851	4000	As above. An area of slope in the northwest portion of the slope is characterised by a bright tone, which suggests that the slope has been repaired in this

Table1- Record of Aerial Photographs Studied (Sheet 3 of 4)

Year	Photograph Reference No.	Altitude (feet)	Observations (Refer to Figure 7)
1980 cont.			location. Part of the repaired surface coincides with the location of the scar first noted in 1974. Above this, and for the entire length of the slope, the verge also has a bright tone.
1981	39103, 39104	10000	Slope works are on going at the fill slope. A retaining wall is under construction along the toe and eastern margin of the slope. The retaining wall has been erected at the previous slope toe. The previous slope appears to be unchanged apart from the construction of the wall (i.e. it is still covered with surface protection). A temporary fence has been erected at the slope crest (denoting the site boundary).
1982	44560, 44561	10000	Slope No. 11NW-A/FR84 has been constructed to its present-day configuration. The surface contains drainage channels and is covered with grass. A small area, characterised by a light tone, is noted at the centre of the southern part of the slope near the slope crest, which may represent a small erosion scar. An area, characterised by a light grey tone is noted on the northern side of a central down channel, i.e. close to the main scar of the 1999 landslide. This may represent surface instability but the quality of the high level photographs prevents detailed examination of the feature.
1983	52148, 52149	10000	The scar features described above are no longer apparent.
1984	57018, 57019	4000	No obvious changes. A patch of low vegetation noted in the northwest corner of the slope.
1985	A2689, A2690	15000	The vegetation on the slope is denser.
1986	A6299, A6300	4000	As above.
1988	A14708, A14709	4000	The slope is uniformly covered with dense vegetation, including trees, except for a relatively bare patch on the slope above the culvert outlet, i.e. along the toe of the centre of the slope.
1990	A20940, A20941	4000	The vegetation on the slope is denser.

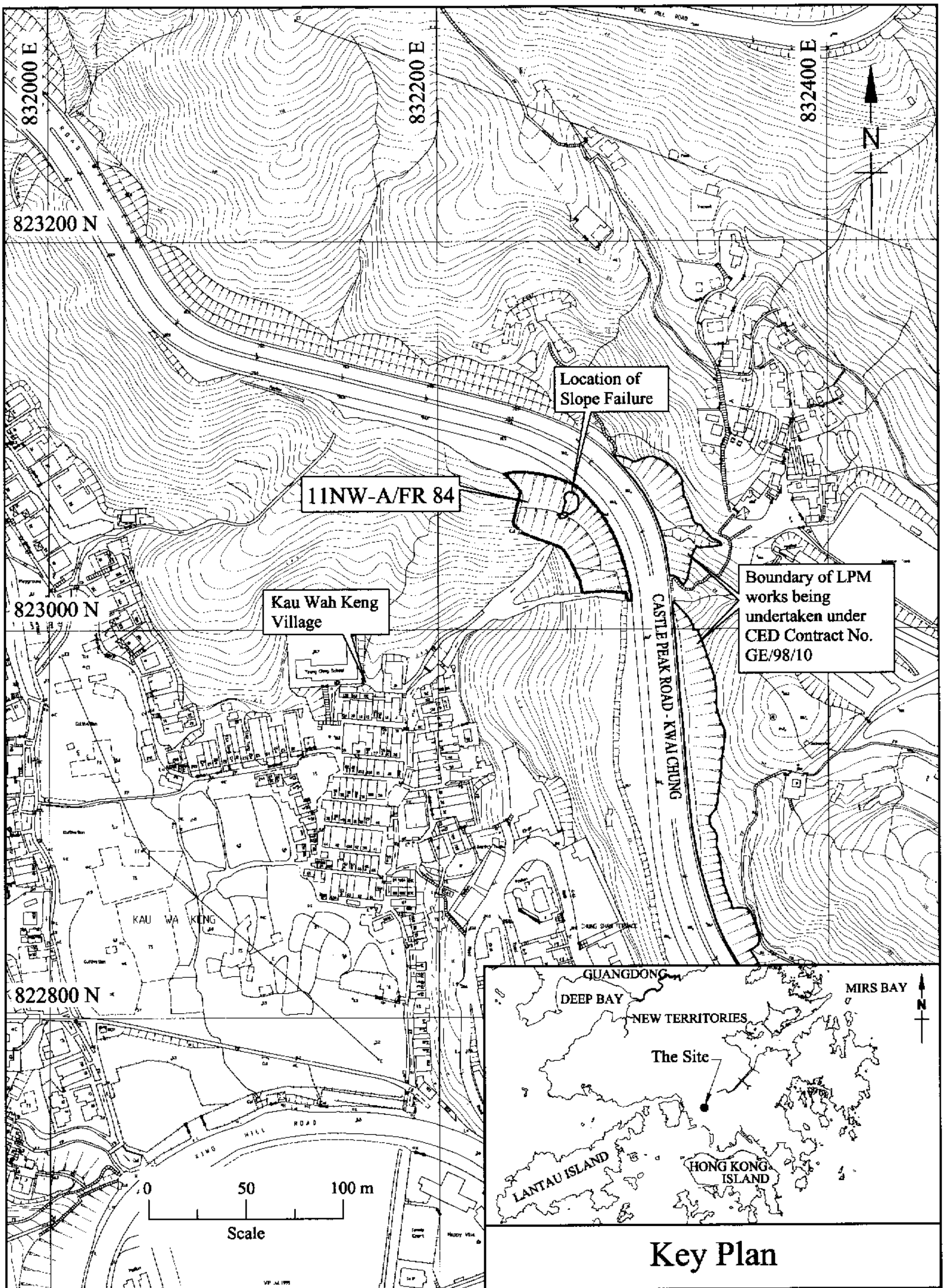
Table1- Record of Aerial Photographs Studied (Sheet 4 of 4)

Year	Photograph Reference No.	Altitude (feet)	Observations (Refer to Figure 7)
1990	A23579, A23580	4000	As above.
1991	A27504, A27505	4000	As above.
1991	A27635, A27636	4000	No obvious changes.
1992	A30457, A30458	4000	The vegetation on the slope is denser.
1992	A32840, A32841	4000	As above.
1993	CN4779, CN4780	5000	A small hut (squatter) noted at the slope toe.
1995	CN12268, CN12269	10000	No obvious changes.
1996	CN16195, CN16196	10000	No obvious changes.
1997	CN17282, CN17283	4000	The vegetation on the slope is noticeably denser.
1998	CN20689, CN20690	3500	Stereo pair not available. The vegetation on the slope is denser.
1998	CN21255, CN21256, CN21257	4000	The vegetation on the slope is denser.
1999	CN25420, CN25421	8000	Two erosion/landslide scars are noted on the fill slope, which correspond to the main scar and lower scar (noted at the toe of the western flank of the slope) identified during the site inspection. The main scar appears to be limited in extent to the slope area between the berm and the J-channel, which was severed by the time that the slope was first inspected by FMSW in January 2000.
1999	CN25316, CN25317	4000	The angle of the photograph prevents detailed examination of the two scars noted in the 09/12/99 photographs. The main scar can be seen on one photograph. However, no further details are discernible.

Table 2 - Summary of Resuts of Laboratory Testing

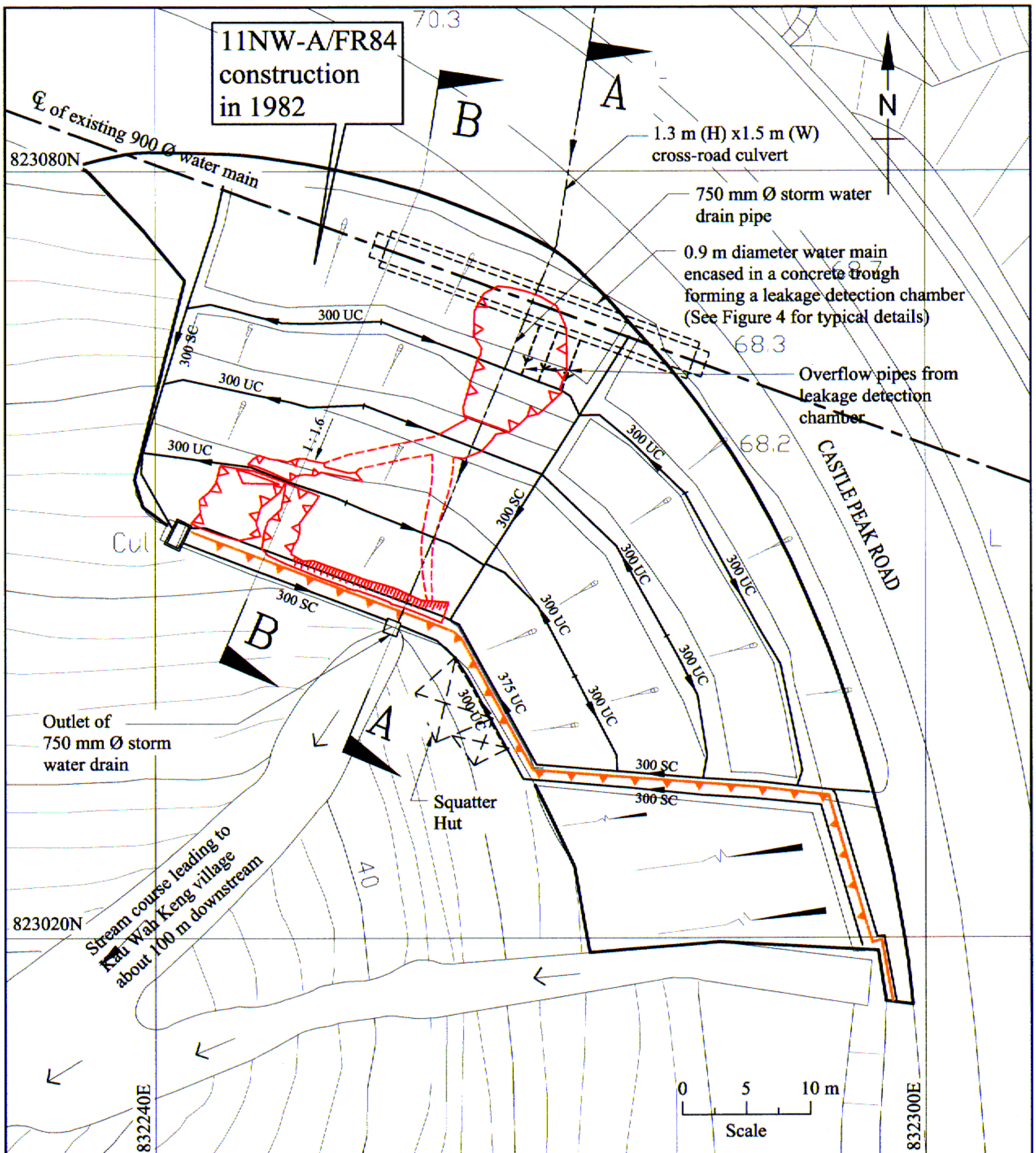
Trial Pit No.	Depth (m)	Visual Description of Soil	Determination of Relative Compaction of fill material			Moisture Content	Percentage of material retained on the 425 μmm test sieve	Liquid Limit	Percentage of material retained the 425 μmm test sieve	Plastic Limit	Plasticity index	Liquidity index
			Max. Dry Density	Optimum Moisture Content (%)	Relative Compaction							
TP1	0.5	Moist brown silty / clayed very sandy GRAVEL (Recompacted Fill)	N/A	N/A	N/A	13	71	38	29	20	18	1.3
TP1	1	Moist brown silty / clayed very sandy GRAVEL (Recompacted Fill)	N/A	N/A	N/A	13	80	33	20	18	15	2.95
TP2	1	Moist brown clayed / silty very gravelly SAND (Recompacted Fill)	1.89	12	100	12	52	40	48	21	19	0.22
TP2	1.5	Moist brown silty / clayed very sandy GRAVEL (Recompacted Fill)	2.13	8.1	100	5.1	76	25	24	Non-plastic	-	-
TP2	1.8	Moist grey silty / clayed very sandy GRAVEL (Recompacted Fill)	2.18	6.7	100	4.2	79	24	21	Non-plastic	-	-
TP2	3	Moist brown very gravelly very silty / clayed SAND (Recompacted Fill)	1.9	12	90.5	12	52	37	48	21	16	0.24
TP3	0.5	Moist brown very gravelly very silty / clayed SAND (Recompacted Fill)	1.88	13	98.4	15	52	41	48	21	20	0.47
TP3	1	Moist brown clayed silty gravelly SAND (Recompacted Fill)	1.9	12	97.4	14	55	36	45	20	16	0.55
TP4	1	Moist brown very gravelly very silty / clayed SAND (Recompacted Fill)	1.89	13	98.4	16	51	41	49	21	20	0.62
TP4	2	Moist brown slightly gravelly very sandy FINE SOIL (Colluvium)	1.79	16	87.7	19	26	48	74	20	28	0.23
TP4	3	Moist brown slightly gravelly silty / clayed SAND (Residual Soil)	1.86	13	87.1	12	45	34	55	18	16	0.28
TP5	1	Moist brown very gravelly very silty / clayed SAND (Recompacted Fill)	1.9	12	98.9	14	50	42	50	23	19	0.24
TP5	2	Moist brown very silty / clayed very gravelly SAND (Recompacted Fill)	1.85	13	97.3	15	54	39	46	21	18	0.54
TP5	3	Moist brown very gravelly very silty / clayed SAND (Old Fill)	1.87	13	90.4	16	53	41	47	21	20	0.69
TP5	3.5	Moist brown very gravelly very silty / clayed SAND (Old Fill)	1.86	14	83.3	18	44	44	56	23	21	0.41

LIST OF FIGURES



Note: Base Map is extracted from Survey Sheet No. 11NW-2C, 2D, 7A & 7B, dated 1988.

Figure 1 - Site Location Plan

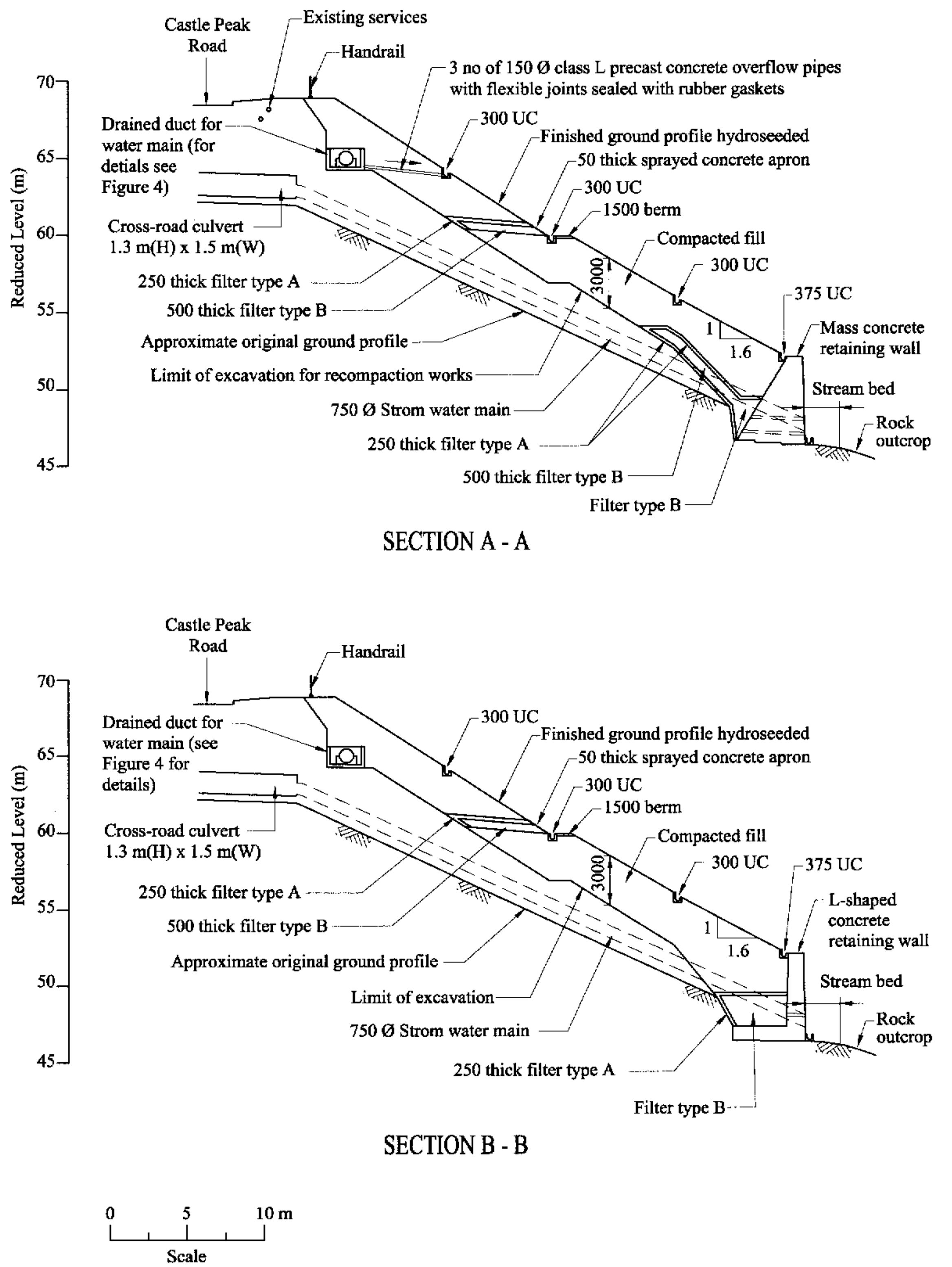


Legend:

- Boundary line
- Mass concrete retaining wall
- Strom water drain pipe
- 300 UC Existing channel
- 1999 landslide scar
- Area of apparent sub-surface voiding, exact route and depth unknown
- Natural stream course

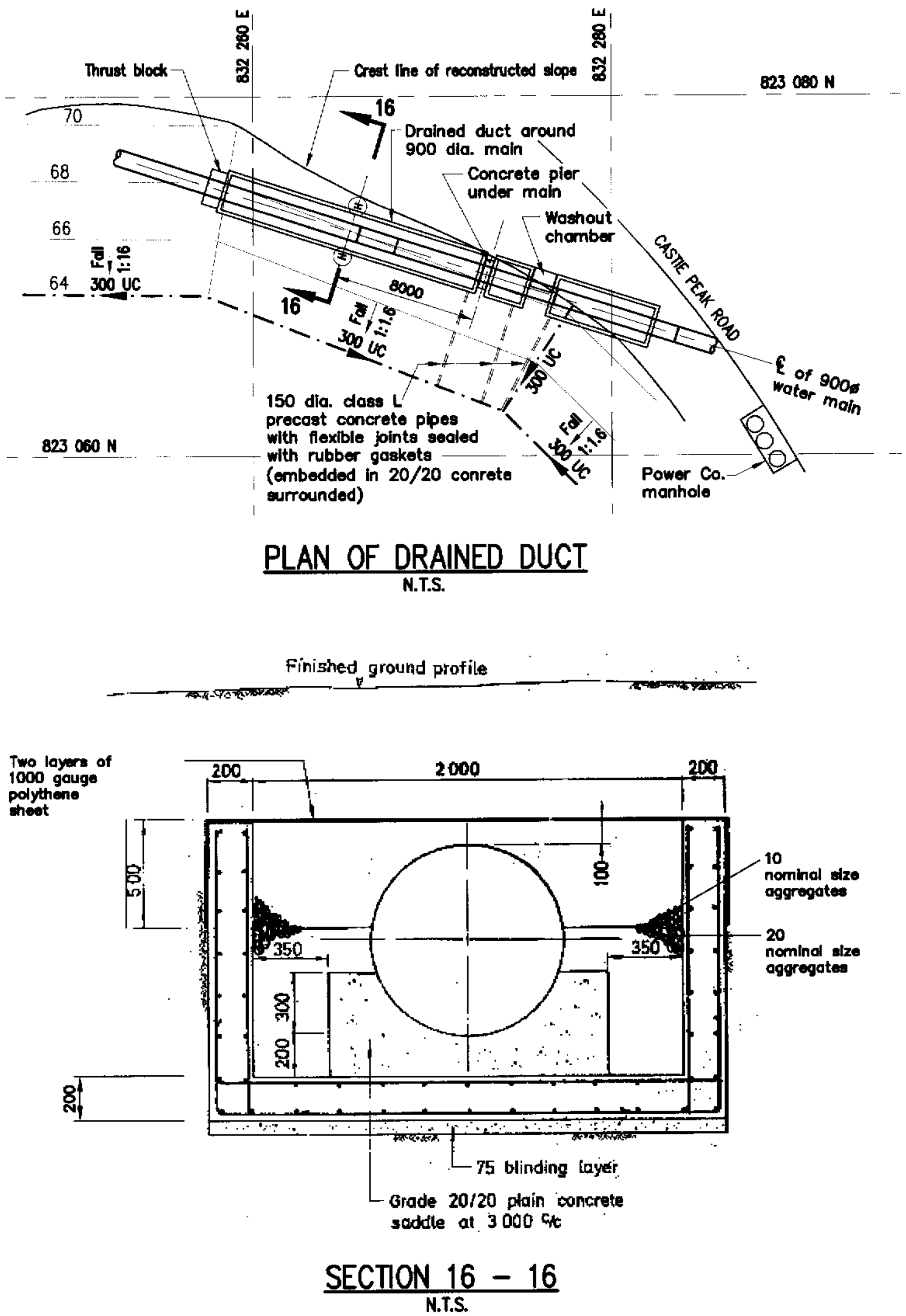
Notes:
(1) Slope details extracted from Binnie & Partner's Record Drawing No. 32, August 1982.
(2) For details of cross-sections refer to Figure 3.

Figure 2 - Layout of LPM Works in 1981/82



Notes:
(1) Slope details extracted from Binnie & Partner's Record Drawing No. 32, August 1982.
(2) For location of section refer to Figure 2.

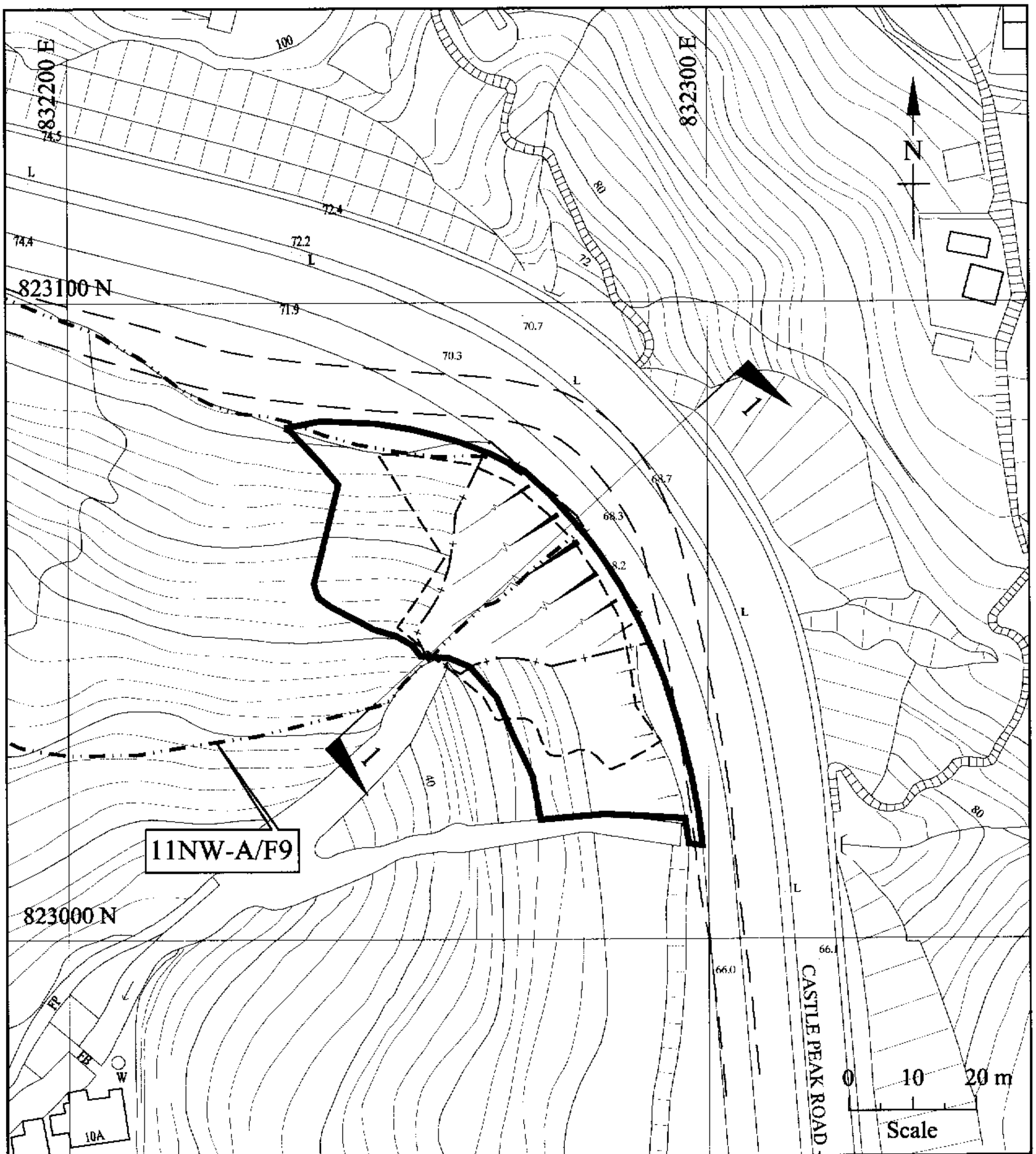
Figure 3 - Cross-sections through the Slope Showing Slope Profile Following LPM Upgrading Works



Notes :

(1) Details extracted from Binnie & Partner's Record Drawing No. 36, August 1982.
(2) All dimensions are in mm.

Figure 4 - Typical Details of Slope Drainage System



Legend:

— — Original alignment of Castle Peak Road prior to road widening and realignment works in the late 1950's.

— · · — Boundary of slope No. 11NW-A/F9 as shown in HyD's file record and as used in early Engineer Inspection and Routine Maintenance Inspection Reports.

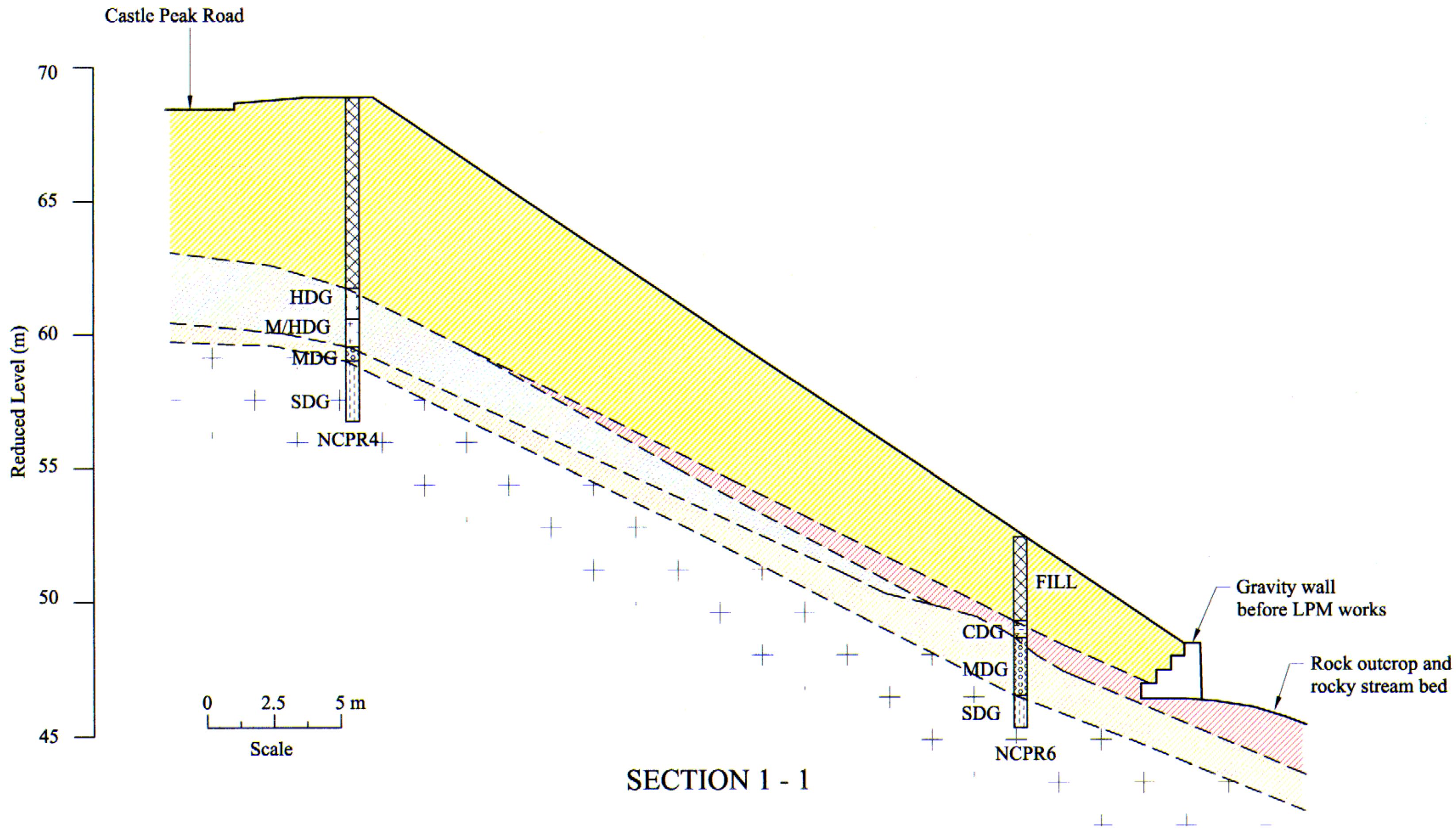
– – – – Feature boundary as depicted in the "Proposed Widening and Realignment of Castle Peak Road" drawings dated 1953 (Details extracted from PWD Drg No. R.K. 425/2B, dated 2/12/1953).

—+— Feature boundary of slope No. 11NW-A/F9 as shown in the GEO's SIS database and that of slope No. 11NW-A/FR9 as depicted in Binnie and Partners Phase II - Detailed Study Report, dated 1978.

▬▬ Feature boundary (slope No. 11NW-A/FR84) as identified by SIMAR.

Note: Base Map is extracted from Survey Sheet No. 11NW-2C & 2D, dated 1988.

Figure 5 - Plan Showing History of Development of the Feature



Notes:
(1) For location of section refer to Figure 5.
(2) Cross-section extracted from Binnie & Partners Study Report, dated 1978.

Figure 6 - Cross-section 1 - 1 through the 1963 Fill Slope

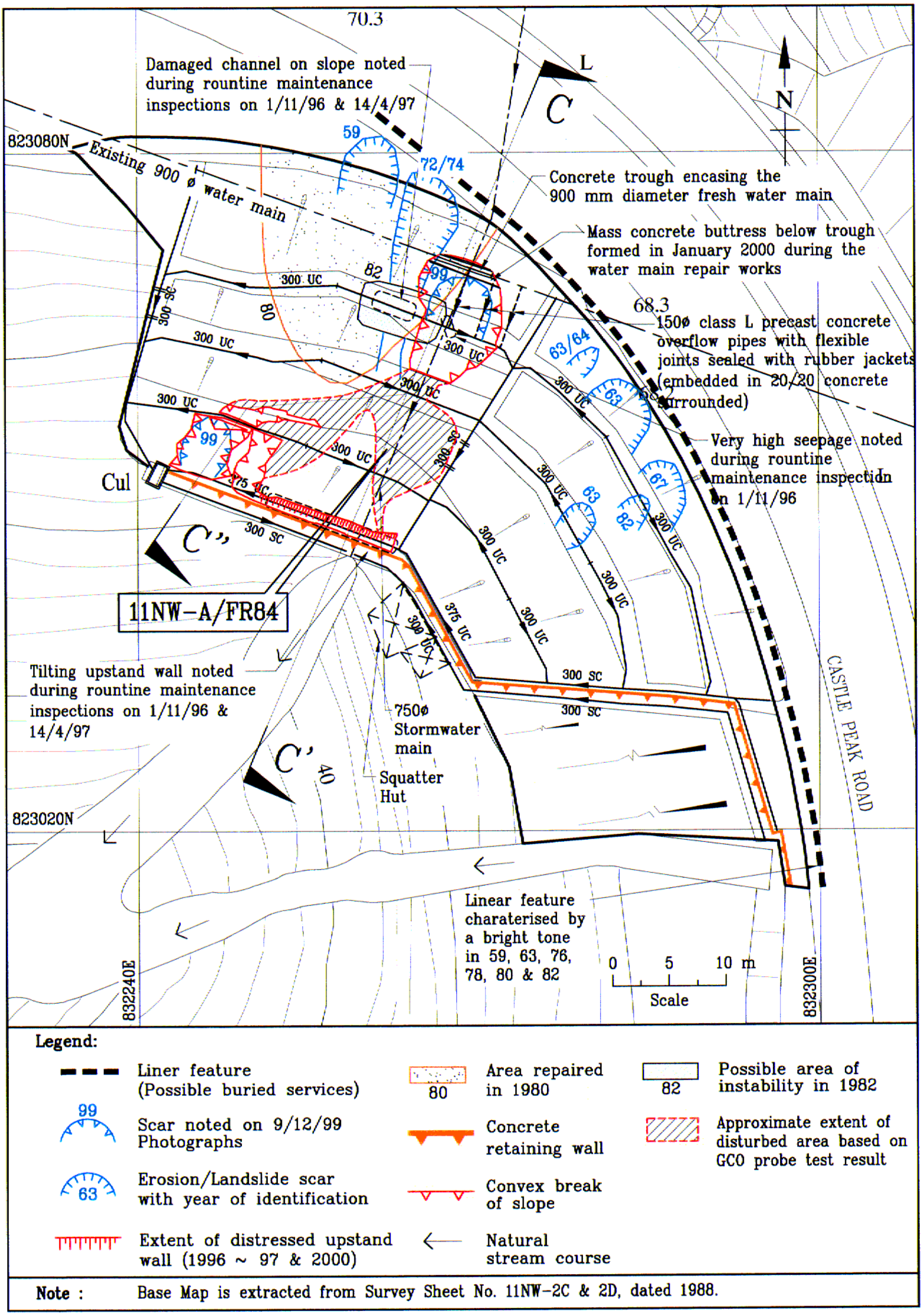
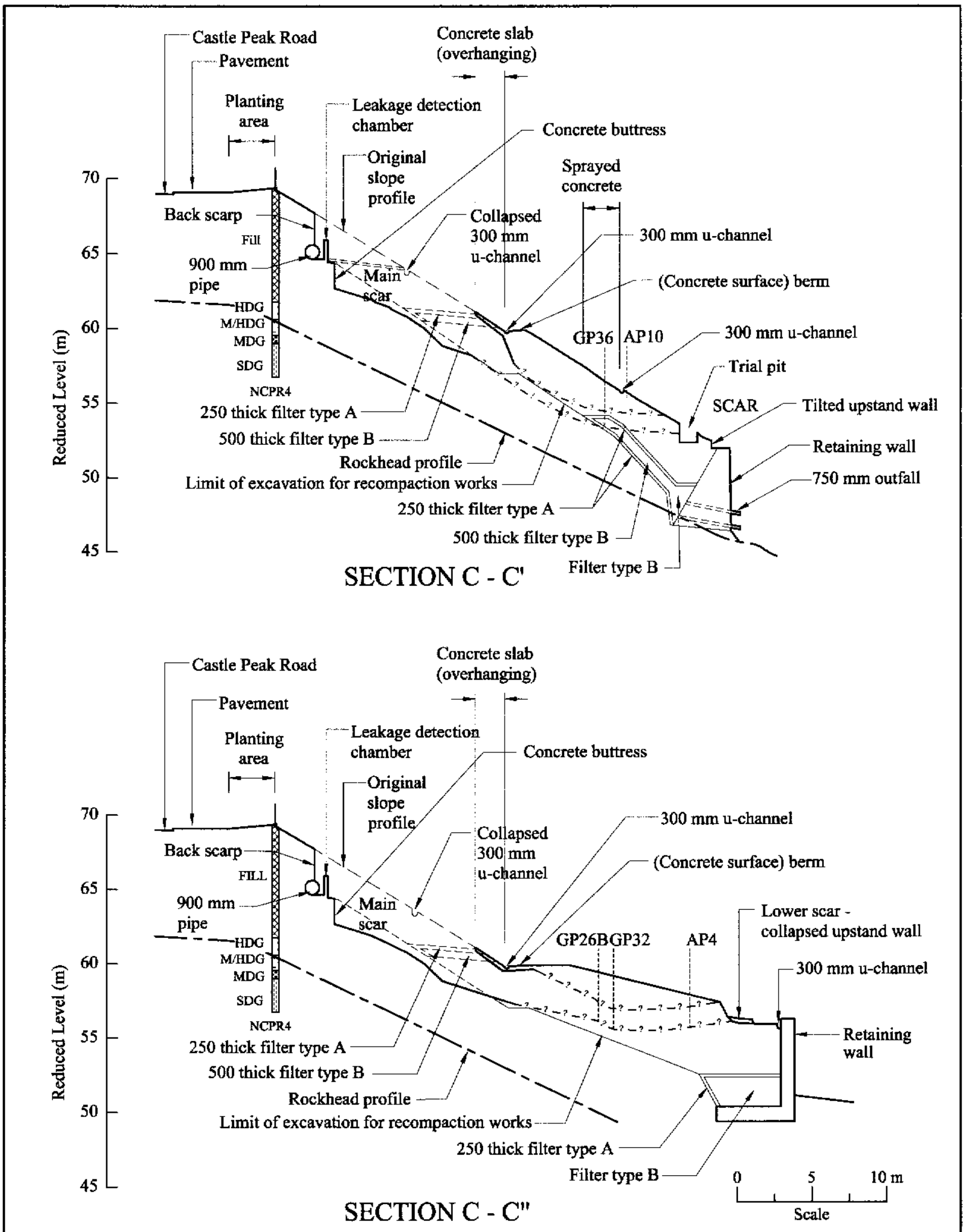


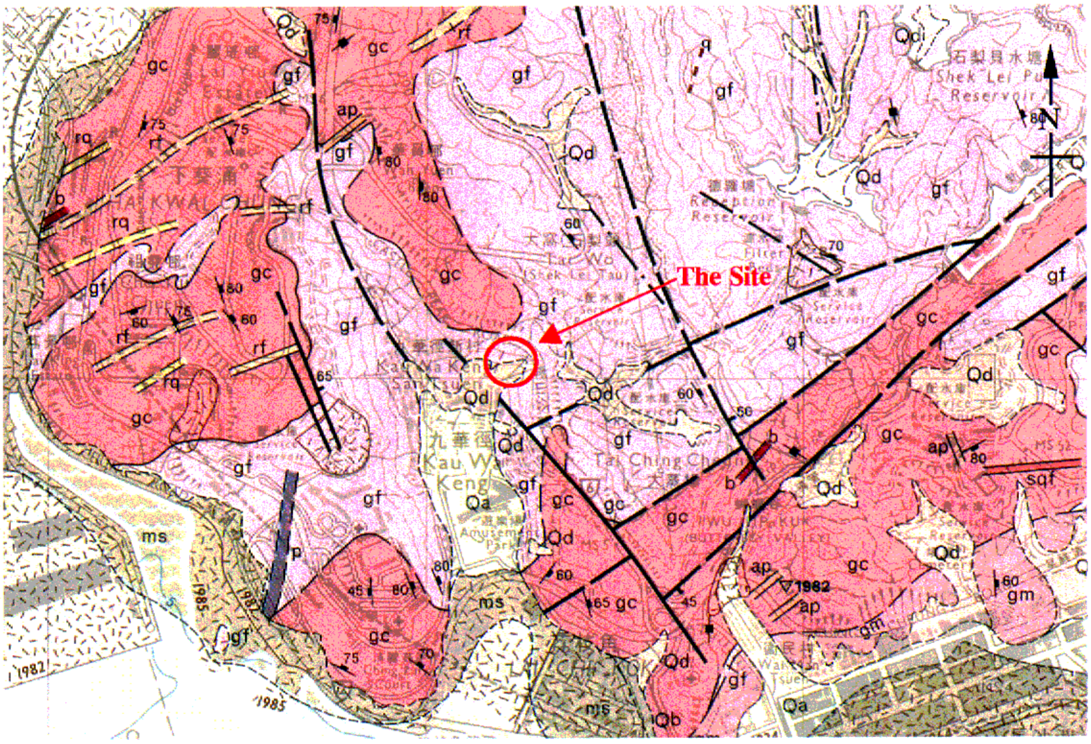
Figure 7 - Plan Showing Signs of Distress at the Landslide Site



Legend:	GP32	GCO probe number	NCPR4	Borehole number	CDG	Completely Decomposed Granite
	M/HDG	Moderately to Highly Decomposed Granite	SDG	Slightly Decomposed Granite	-·-·-·-	Possible routing of sub-surface erosion channel

Notes:
(1) For location of section refer to Figure 7.
(2) 750 mm diameter storm water drain not shown for clarity.

Figure 8 - Cross-sections through the Landslide



0m 500 m

LEGEND:

SUPERFICIAL DEPOSITS (ONSHORE)

- Qfs — Fill, sanitary fill (Qfs)
- Qb — Beach deposits
- Qa — Alluvium
- Qd — Debris Flow Deposits

SOLID GEOLOGY

- gf — **Fine-grained granite, < 2mm**
- gm — Medium-grained granite, 2-6mm
- gc — Coarse-grained granite, > 6mm
- sqf — Fine-grained quartz syenite, < 2mm
- b — Basalt
- rq — Quartzphyric rhyolite
- rf — Feldsparphyric rhyolite
- ap — Aplite
- p — Pegmatite

SUPERFICIAL DEPOSITS (OFFSHORE)

- ms — Marine sand, part silty

GEOLOGICAL LINES

- Geological boundary, superficial deposits
- Fill boundary, with limit of reclamation at date shown
- Geological boundary, solid rock *
- Fault (crossmark indicates downthrow side) *
- Mineral vein *

(* Broken lines on map face denote uncertainty)

STRUCTURAL SYMBOLS

- 20 — Inclined jointing
- Vertical jointing

Notes: (1) Geology relevant to study area shown bold.
(2) Extracted from Sheet 11 of the Hong Kong Geological Survey 1 : 20,000 Scale Map Series HGM20 (GCO, 1986).

Figure 9 – Regional Geological Map

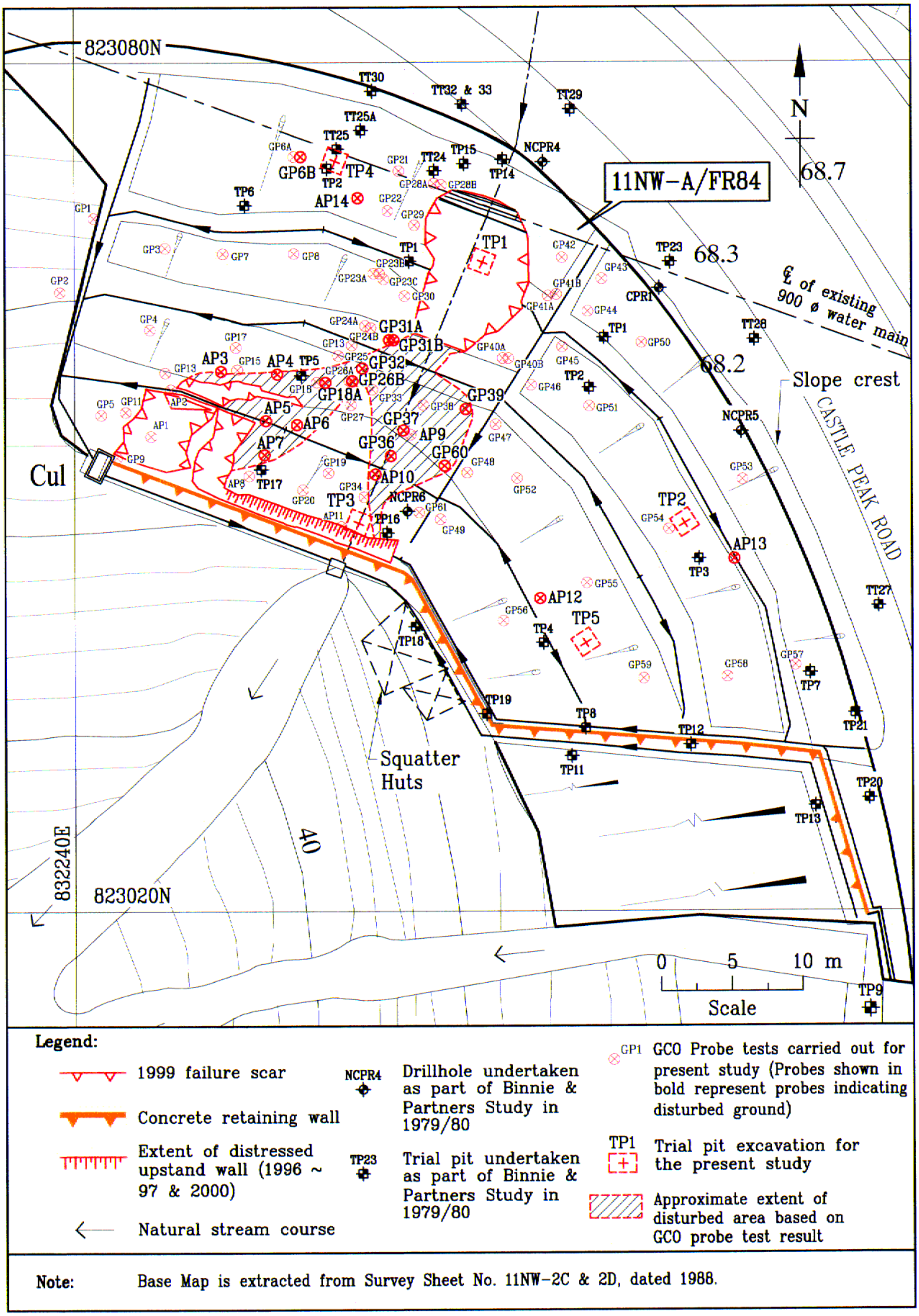


Figure 10 - Ground Investigation Plan

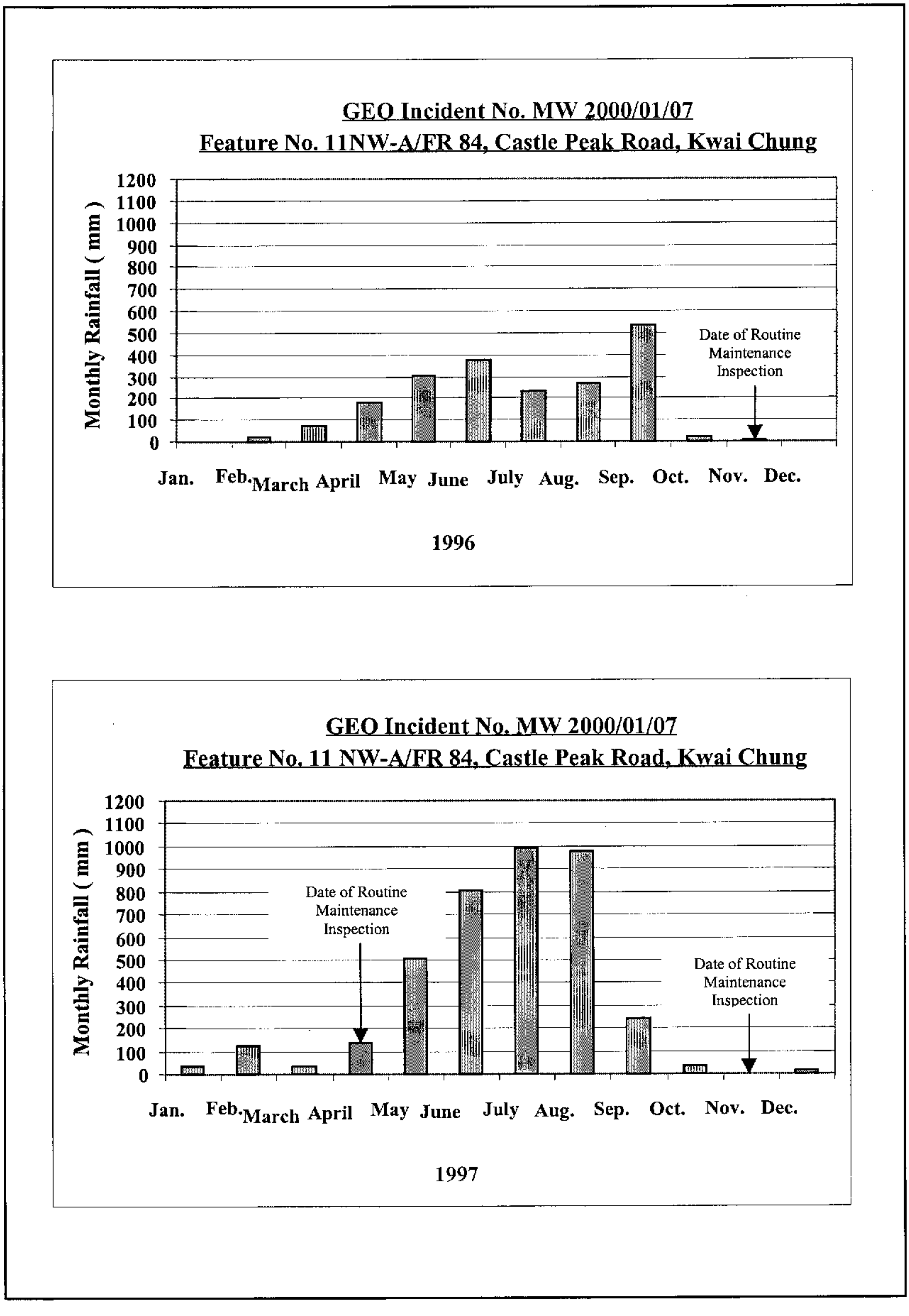


Figure 11 – Rainfall Records at Raingauge N04 for 1996 and 1997

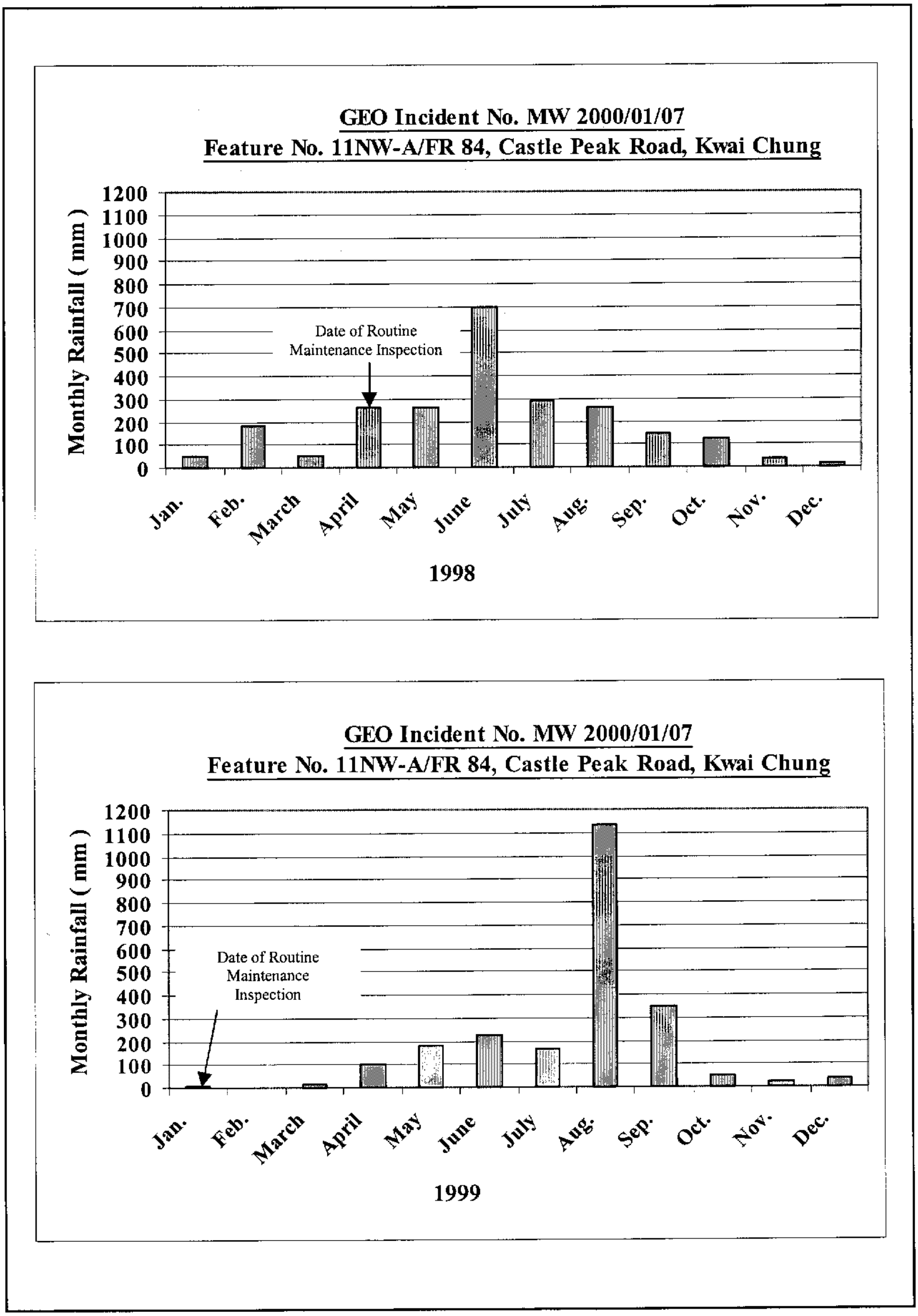
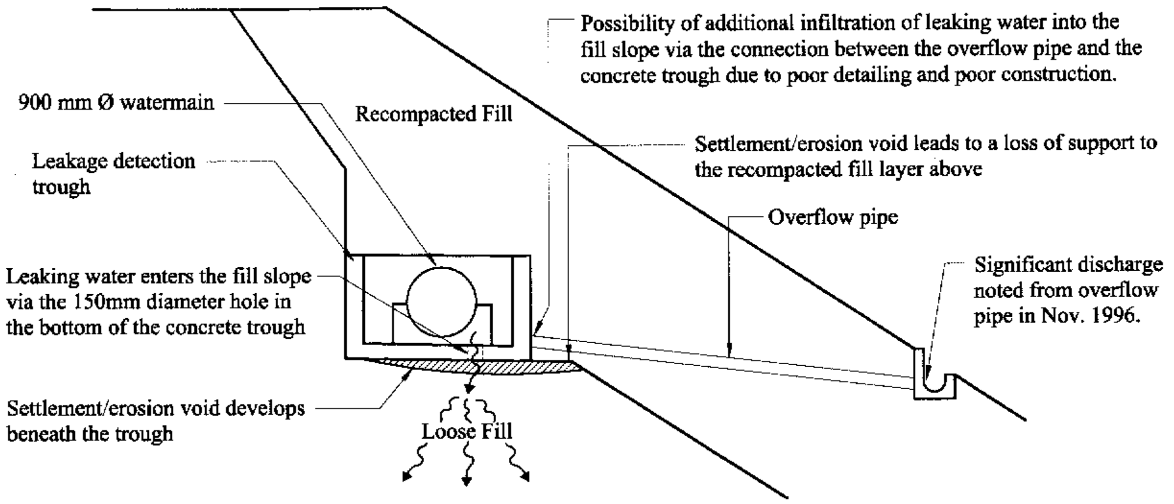
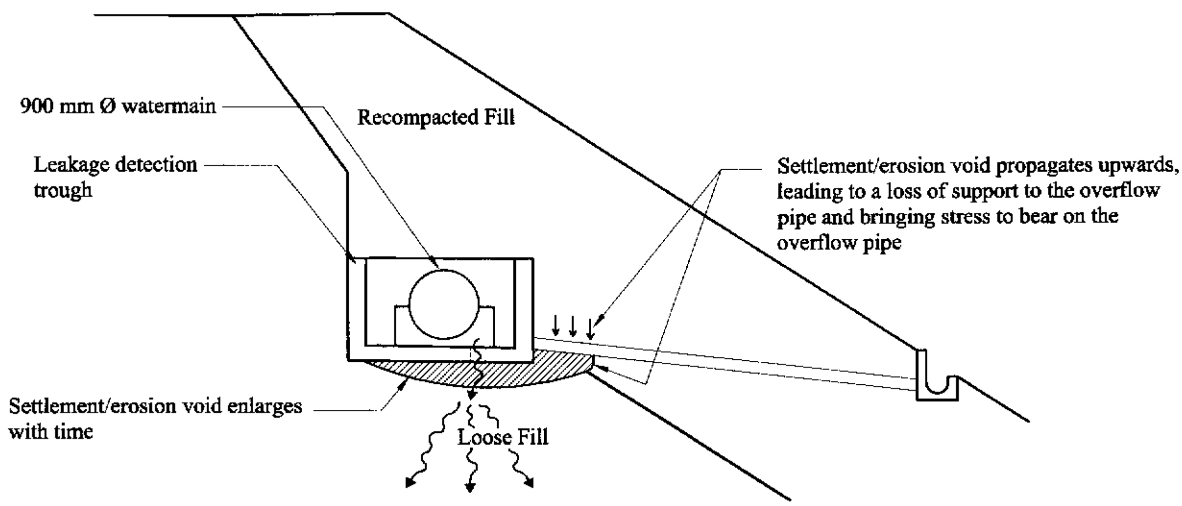


Figure 12 – Rainfall Records at Raingauge N04 for 1998 and 1999



a) Water from the broken watermain collected in the leakage detection chamber and discharges into the surface drainage system via the overflow pipe, but some water infiltrates into the fill below the bottom of the concrete trough through the 150 mm diameter hole in the base of the concrete trough, causing loss of fines and inducing settlement. This in turn results in a loss of support to and settlement of the recompacted fill layer above.

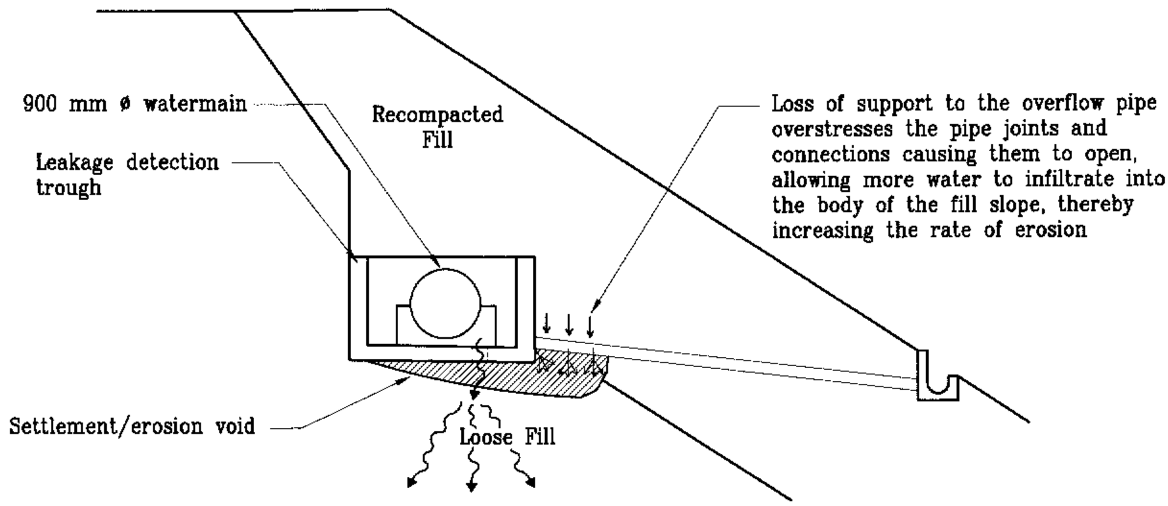


b) The void beneath the concrete trough enlarges due to further erosion and settlement and propagates upwards into the 3 m thick recompacted cap due to the localised collapse of unsupported material. This in turn results in the overflow pipe becoming over-stressed.

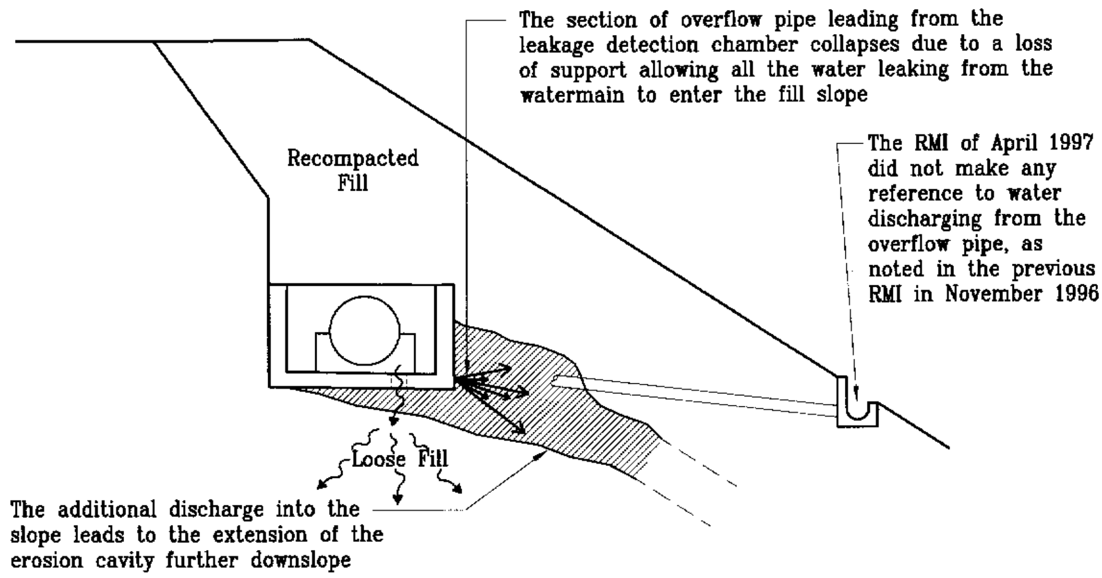
0 1 2 m

Scale

Figure 13 - Schematic Diagrams Showing Possible Sequence of Events Leading to Slope Failure (Sheet 1 of 3)



c) The loss of support to the overflow pipe leads to the pipe joints/connections becoming over-stressed causing them to open, allowing more water to infiltrate into the body of the fill slope, thereby increasing the rate of erosion. Water starts to back up behind the toe retaining wall due to the blocked weepholes and water pressures build up behind the small upstand wall at the crest of the retaining wall causing the top to rotate outwards.

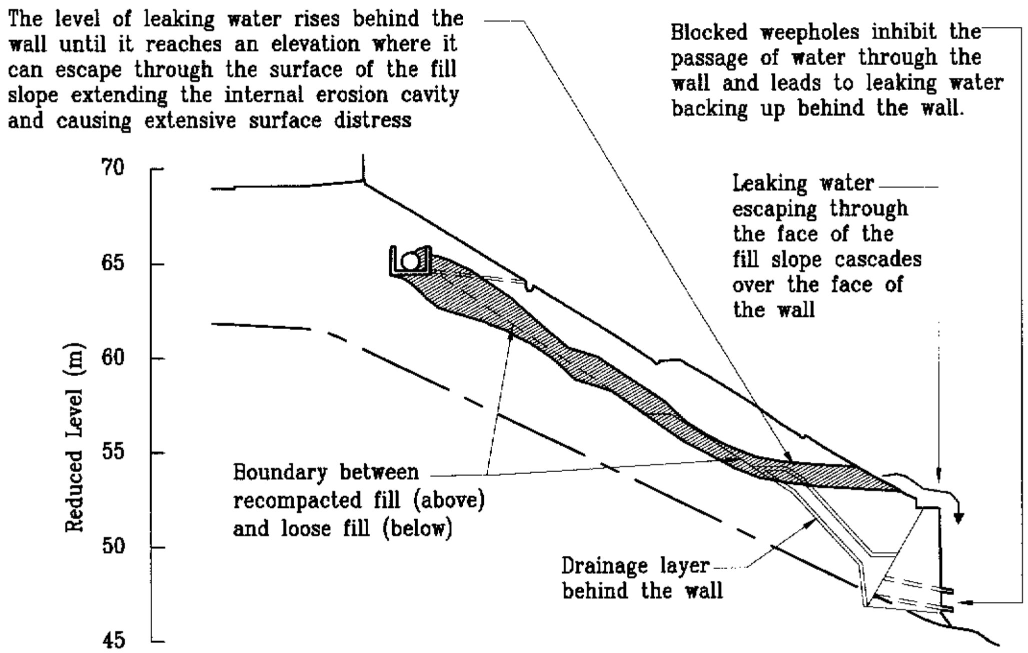


d) The section of the overflow pipe leading from the leakage detection chamber collapses due to a loss of support, allowing all the leakage from the broken watermain to discharge into the fill slope. Increased internal seepage flows within the fill cause the erosion cavity to extend downslope.

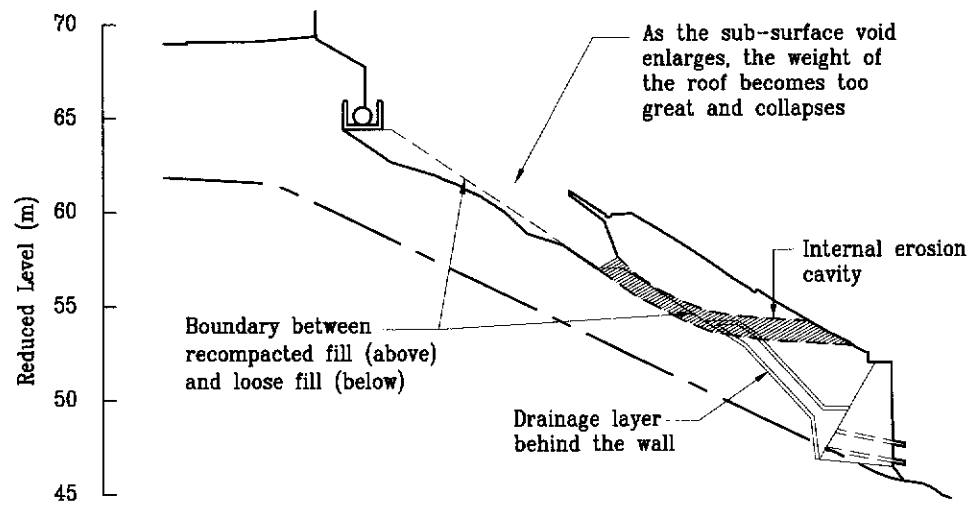
0 1 2 m

Scale

Figure 13 - Schematic Diagrams Showing Possible Sequence of Events Leading to Slope Failure (Sheet 2 of 3)



e) Increased water flows into the body of the fill slope leads to further backing up of water behind the retaining wall, forcing it to discharge out of the surface of the fill slope at an increased rate and causing extensive erosion and surface distress.

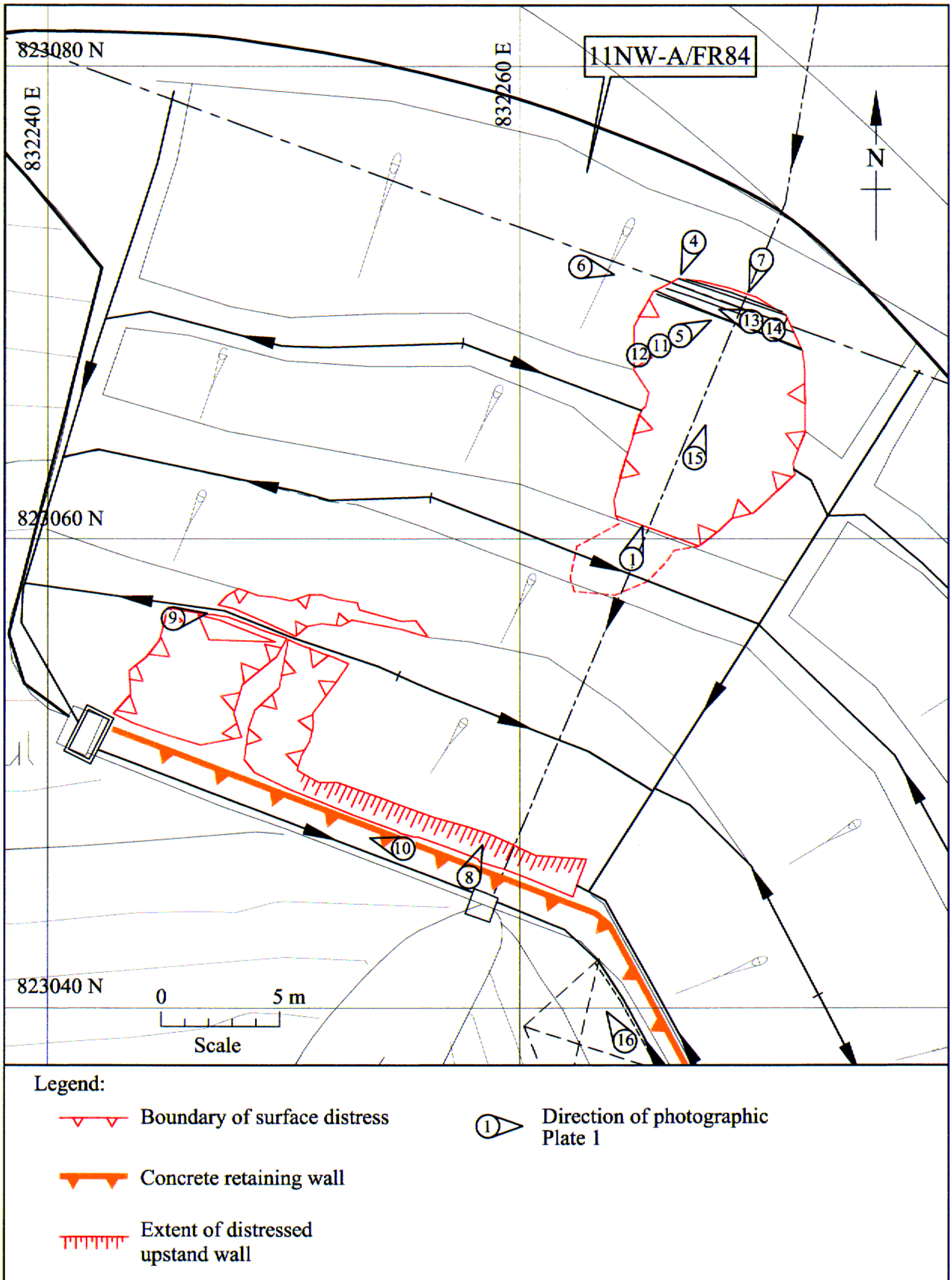


f) The continual enlargement of the erosion cavity brings the stability of the roof of recompacted fill material spanning the erosion hole to limiting conditions of equilibrium and eventually leads to collapse.

0 5 10 m

Scale

Figure 13 - Schematic Diagrams Showing Possible Sequence of Events Leading to Slope Failure (Sheet 3 of 3)



Notes:
(1) Base Map is extracted from Survey Sheet No. 11NW-2C & 2D, dated 1988.
(2) Plates 2 and 3 refer to extracts from aerial photographs.

Figure 14 - Location of Photographs

LIST OF PLATES

Plate 1 – View of Main Scar (Photograph Taken on 26 January 2000)

Note: See Figure 14 for Location of Photograph.

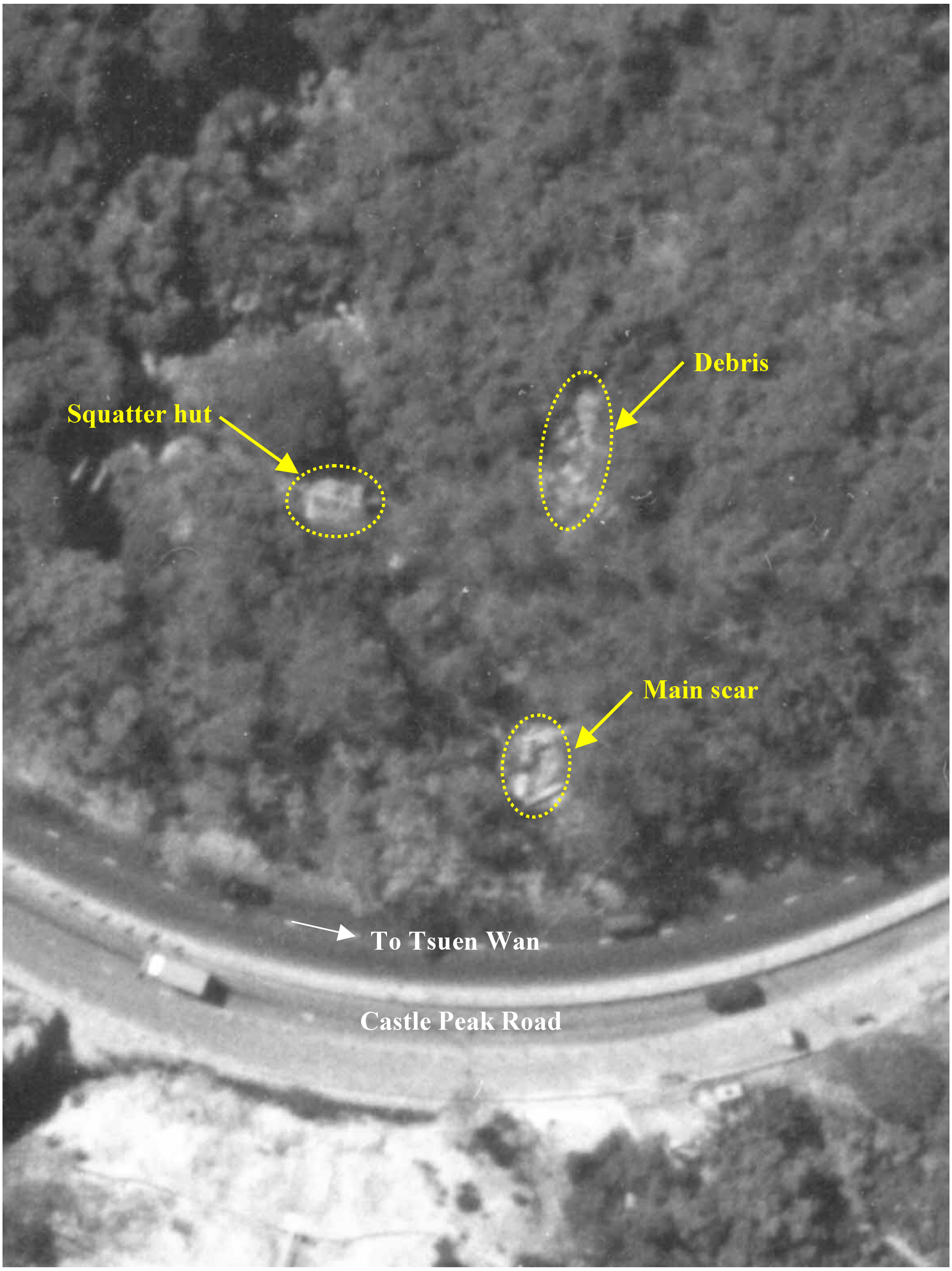


Plate 2 – Extract from Aerial Photograph CN 25421 Showing Slope Distress (Photograph Taken on 9 December 1999)

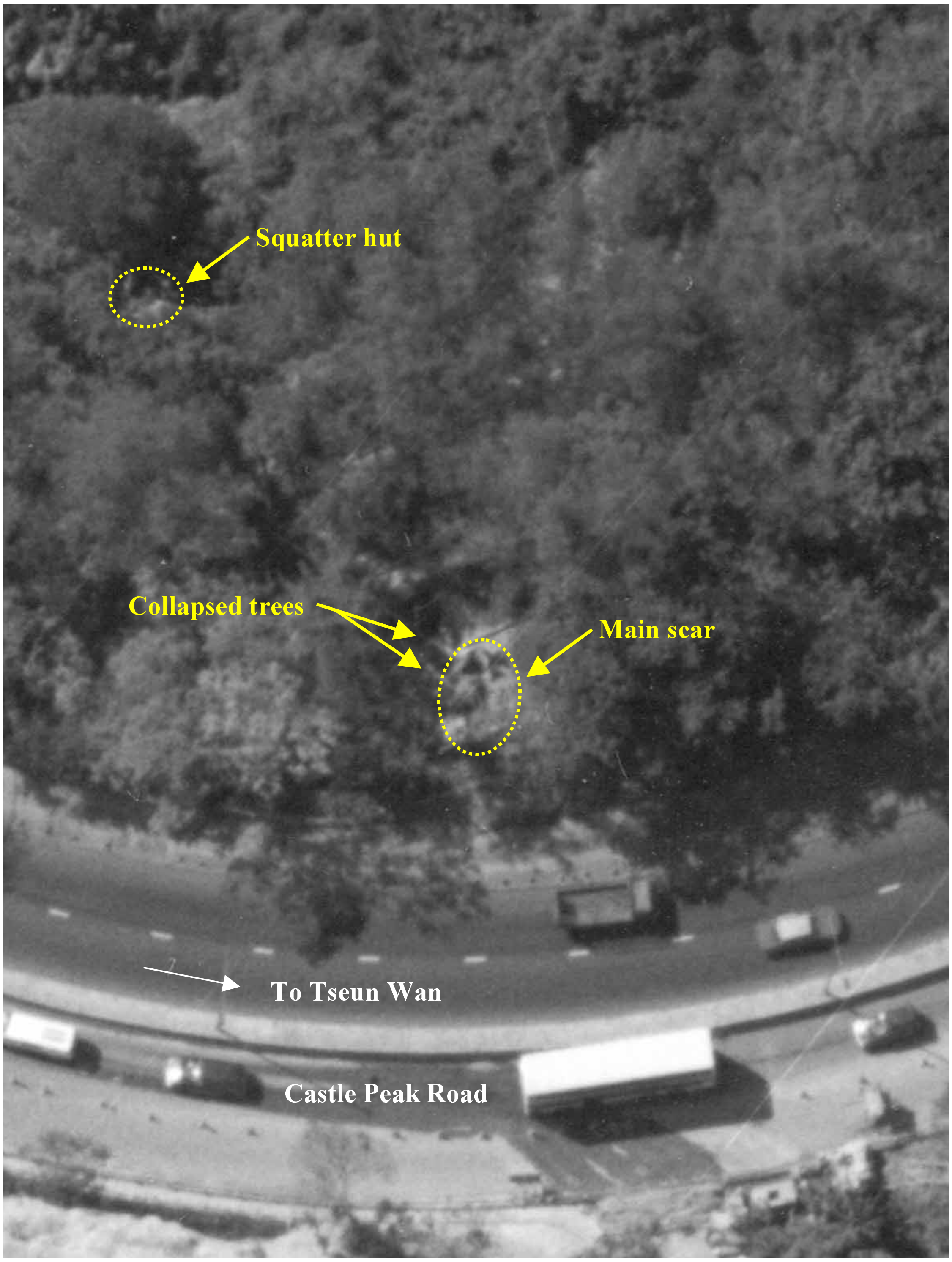


Plate 3 – Extract from Aerial Photograph CN 25316 Showing Slope Distress
(Photograph Taken on 11 December 1999)

Plate 4 – View of Failed Slope as Observed by Scott Wilson (HK) Ltd. (Photograph Taken on 14 January 2000 by Scott Wilson (HK) Ltd.)

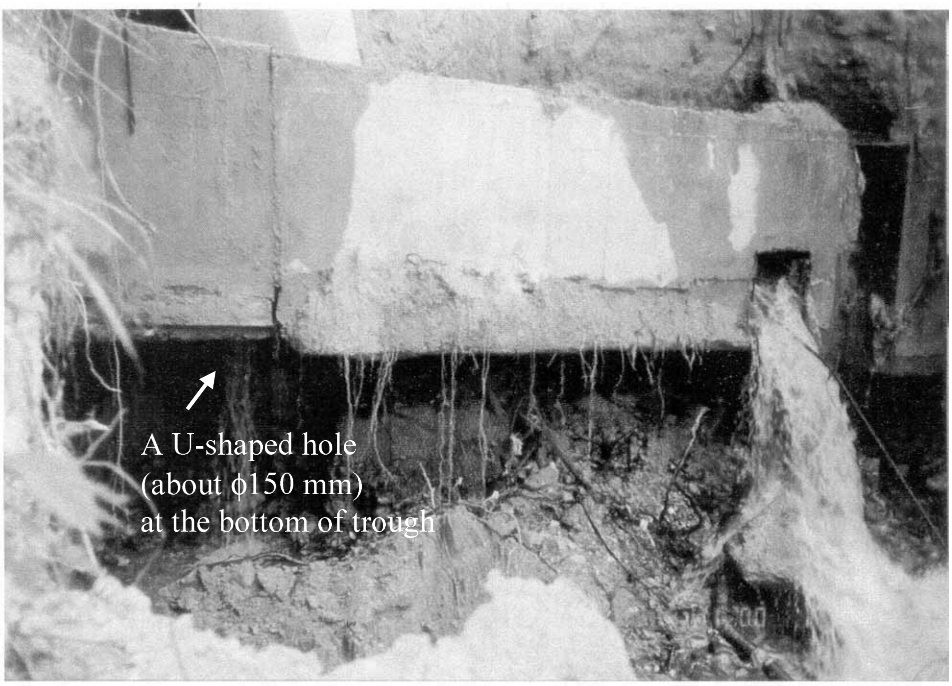


Plate 5 – View of Water Discharging from Leakage Detection Trough in Which the 900 mm Diameter Watermain was Housed (Photograph Taken on 14 January 2000 by Scott Wilson (HK) Ltd.)

Note: See Figure 14 for Locations of Photographs.

Plate 6 – View of Leaking Pipe at the Time of Inspection by WSD
(Photograph Taken on 15 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 7 – View of the Void at the Toe of the Main Scar
(Photograph Taken on 3 May 2000)

Note: See Figure 14 for Location of Photograph.

Plate 8 – Subsurface Void (approx. 0.6 m in diameter) in the Lowest Batter of the Slope below the Main Scar (Photograph Taken on 26 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 9 – Subsurface Void in the Lowest Batter to the Southwest of the Main Scar
(Photograph Taken on 26 January 2000)

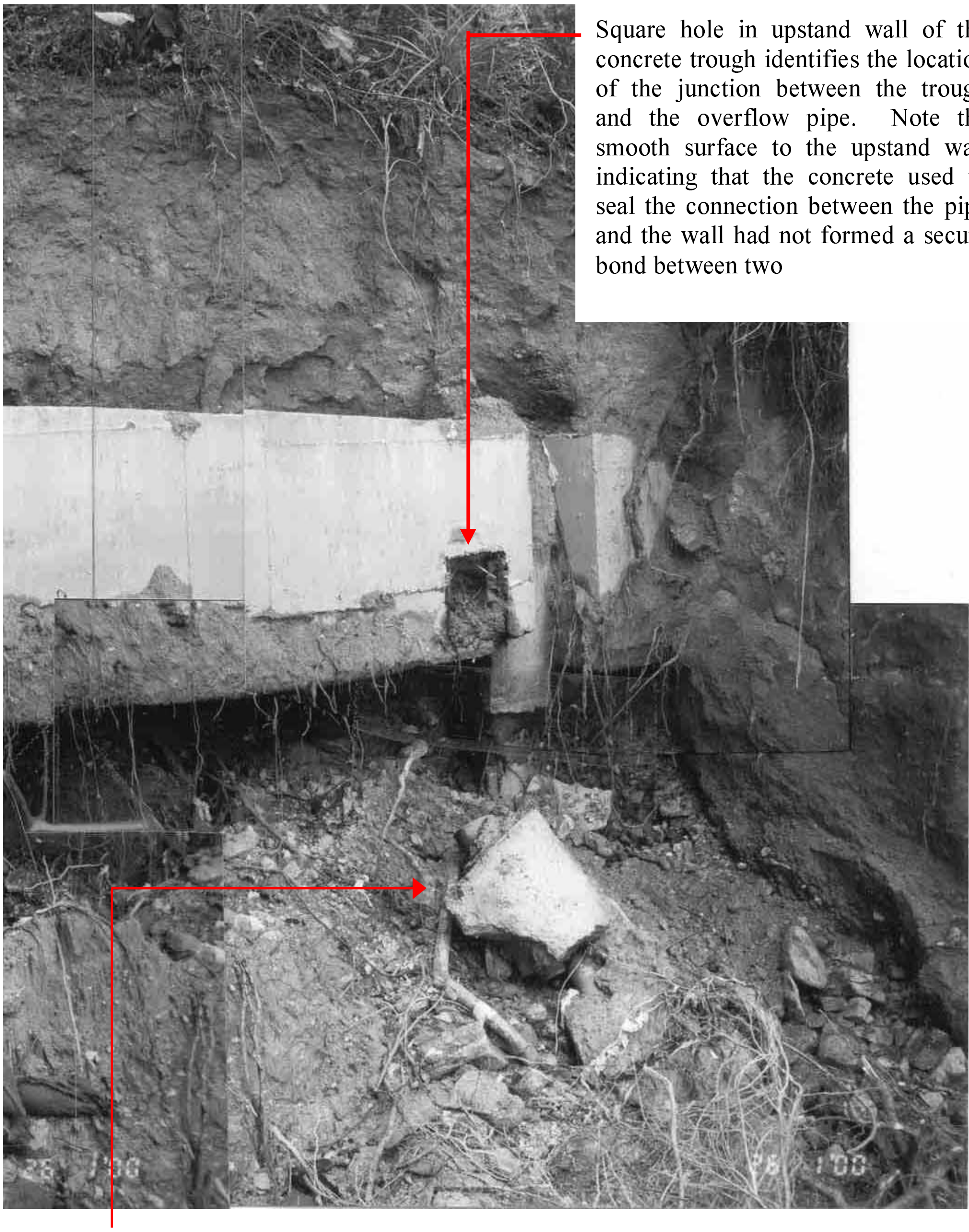
Note: See Figure 14 for Location of Photograph.

Plate 10 – Tilted Upstand Wall at the Toe of the Lowest Batter
(Photograph Taken on 26 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 11 – View of the Hole at the Bottom of Trough
(Photograph Taken on 26 January 2000)

Note: See Figure 14 for Location of Photograph.



Square hole in upstand wall of the concrete trough identifies the location of the junction between the trough and the overflow pipe. Note the smooth surface to the upstand wall indicating that the concrete used to seal the connection between the pipe and the wall had not formed a secure bond between two

End of overflow pipe that was secured into the concrete trough. The end was intact indicating that it had been drawn out of the connection rather than being shear by vertical pressure from above

Plate 12 – View of Junction between the Outlet Hole in the Concrete Trough and the Overflow Pipe (Photograph Taken on 26 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 13 – View of 100 mm Long Crack at the Welded Joint
(Photograph Taken by WSD on 15 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 14 – Close-up View of the Crack along the Welded Joint
(Photograph Taken by WSD on 15 January 2000)

Note: See Figure 14 for Location of Photograph.

Plate 15 – View of Concrete Buttress Built to Support the Trough after the Incident
(Photograph Taken on 3 May 2000)

Note: See Figure 14 for Location of Photograph.

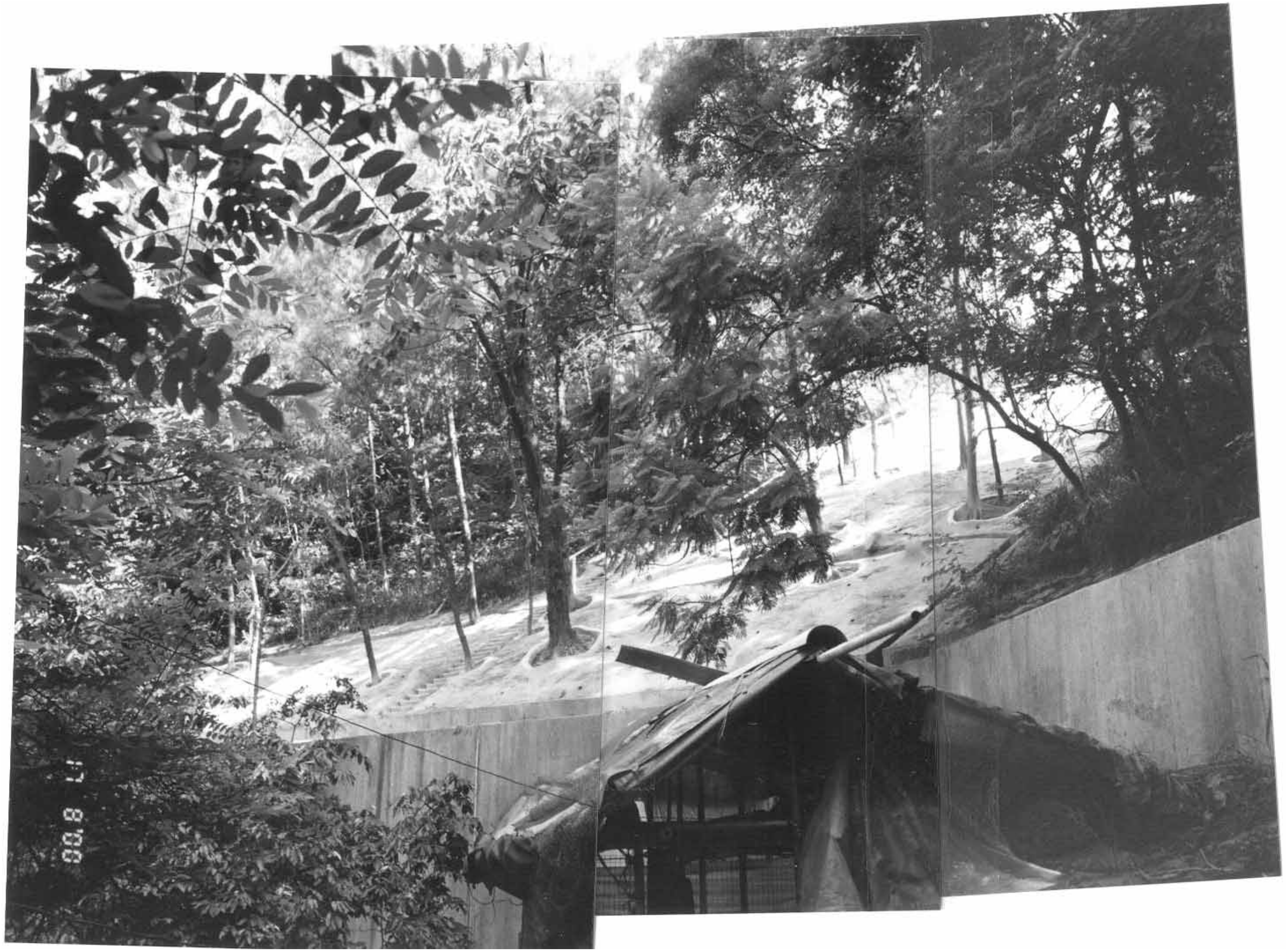


Plate 16 – General View of Slope after Repair Works by HyD
(Photograph Taken on 17 August 2000)

Note: See Figure 14 for Location of Photograph.

APPENDIX A

PHOTOGRAPHS TAKEN DURING ROUTINE MAINTENANCE INSPECTIONS BY HIGHWAYS DEPARTMENT

Plate A – View of Significant Discharge from Overflow Pipe Connecting to the Leakage Detection Chamber (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate B – View of Blocked Drainage Channel (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate C – View of Tilting Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate D – View of Tilting Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate E – View of Blocked Channel (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate F – View of Broken Channel (Record Photograph Extracted from HyD Report on RMI of 1 November 1996)

Plate G – View of Broken Channel (Record Photograph Extracted from HyD Report on RMI of 14 April 1997)

Plate H – View of Cracked Berm and Channel (Record Photograph Extracted from HyD Report on RMI of 14 April 1997)

Plate I – View of Partly Blocked Channel (Record Photograph Extracted from HyD Report on RMI of 14 April 1997)

Plate J – View of Tilting Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 14 April 1997)

Plate K – View of Repaired Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 17 April 1998)

Plate L – View of Repaired Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 17 April 1998)

Plate M – View of Slight Seepage/Dampness at the Discharge Point of the Overflow Pipe (Record Photograph Extracted from HyD Report on RMI of 14 January 1999)

Plate N – View of Seepage at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 14 January 1999)

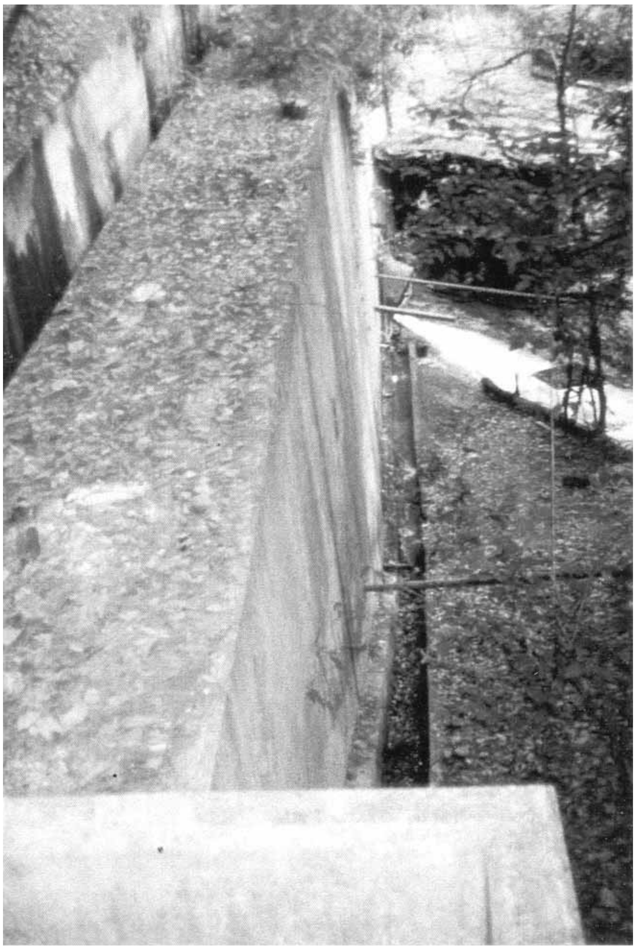
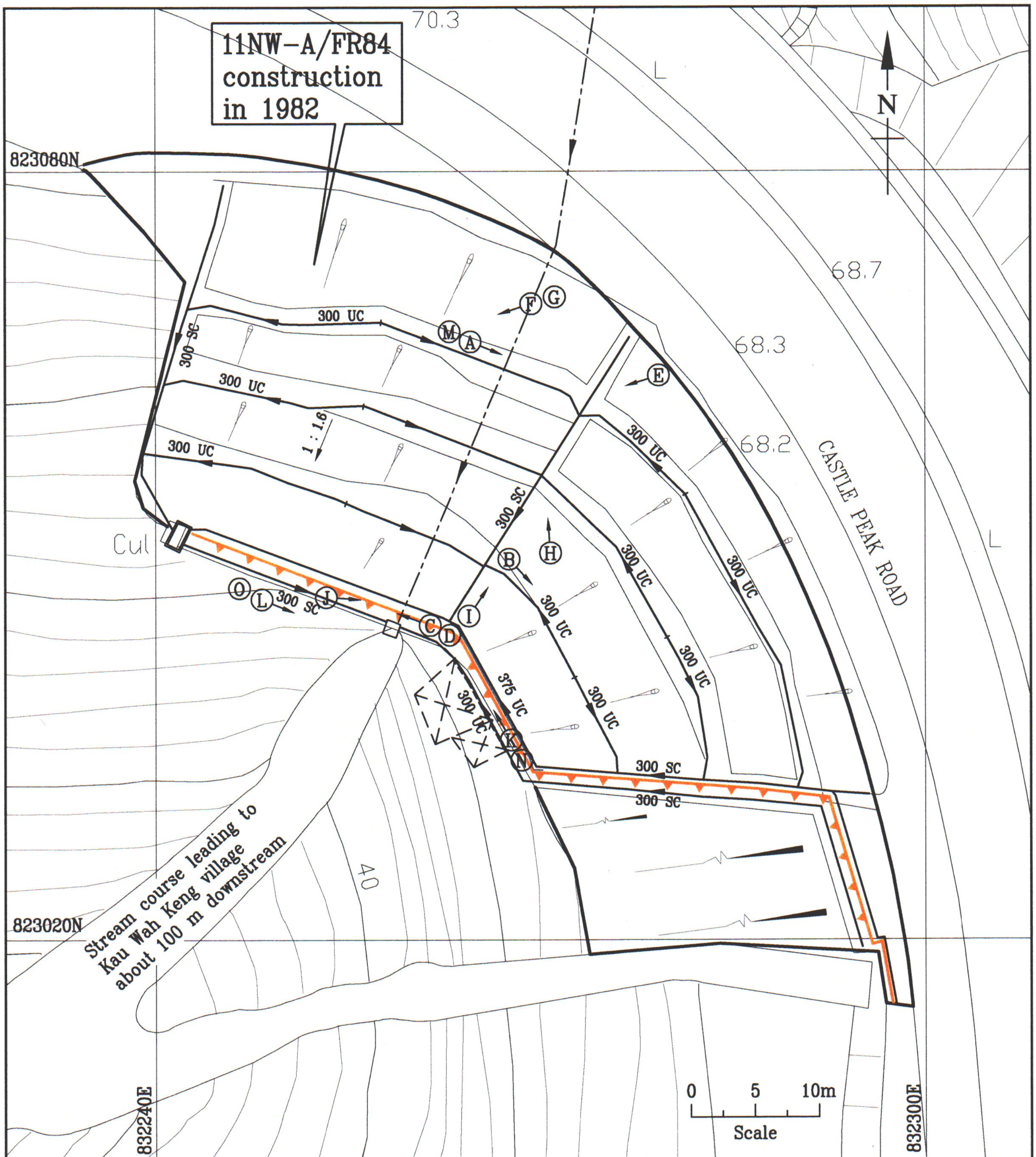


Plate O – View of Seepage/Dampness along Upstand Wall at the Toe of the Slope (Record Photograph Extracted from HyD Report on RMI of 14 January 1999)



Legend: Feature boundary

Ⓐ→ Direction of Photograph A

Note : Photograph A to F taken during routine maintenance in November 1996.

Photograph G to J taken during routine maintenance in April 1997.

Photograph K to L taken during routine maintenance in April 1998.

Photograph M to O taken during routine maintenance in January 1999.

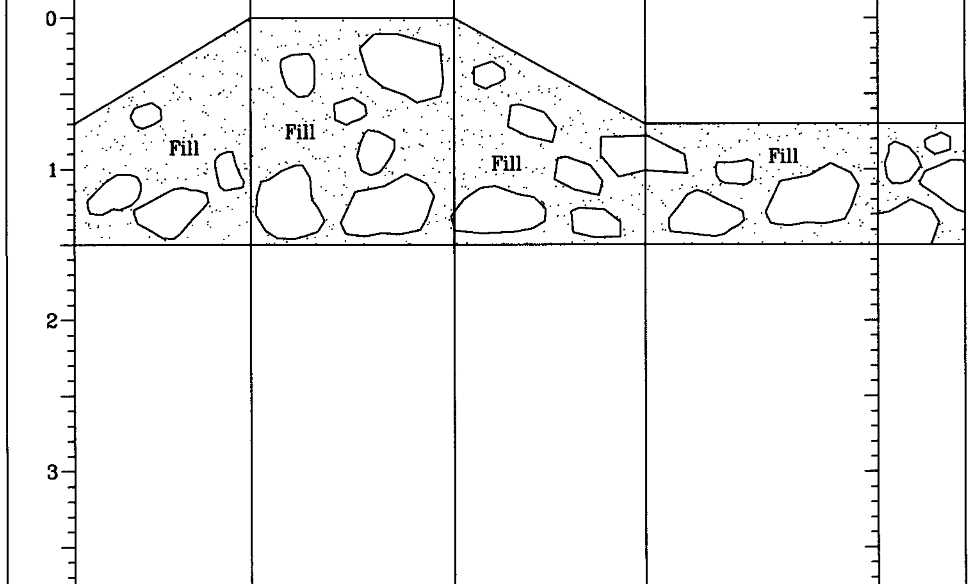
Figure A1 - Direction of Photographs Taken During Routine Maintenance Inspections By HyD (1996 - 1999)

APPENDIX B

TRIAL PIT LOGS

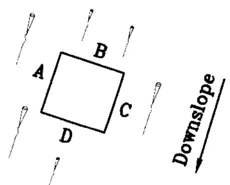
Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 21 Kau Wah Keng, Castle Peak Road, Kwai Chung, N.T.	TRIAL PIT RECORD TRIAL PIT No. TP1 CO-ORDINATES: 832268.6 E, 823066.1 N GROUND LEVEL: 60.2 mPD	Sheet 1 of 1 EXCAVATION DATE: 02~03/May/2000 BACKFILL DATE: 15~16/May/2000 LOGGED BY: K H Liu DATE: 04/May/2000 CHECKED BY: S L Chui DATE: 08/May/2000

Samples & tests	Depth (m)	FACE D: 1.5 m	FACE A: 1.5 m	FACE B: 1.5 m	FACE C: 1.5 m	Legend	Grade	Description
↨ ↨	0–1.5	Fill	Fill	Fill	Fill			Boulders, cobbles & gravels with very loose light yellow silty sand (Fill)



SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↨	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



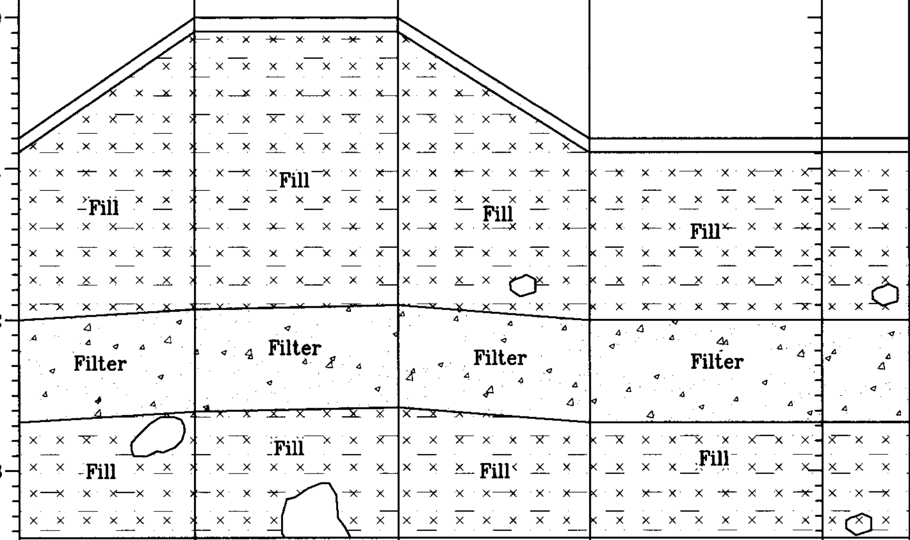
KEY

NORTH ARROW

REMARKS

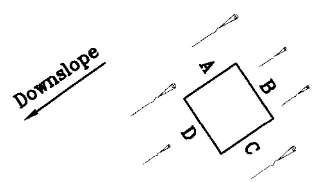
Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 21 Kau Wah Keng, Castle Peak Road, Kwai Chung, N.T.	TRIAL PIT RECORD TRIAL PIT No. TP2 CO-ORDINATES: 832283.4 E, 823047.6 N GROUND LEVEL: 63.1 mPD	Sheet 1 of 1 EXCAVATION DATE: 02~03/May/2000 BACKFILL DATE: 15~16/May/2000 LOGGED BY: K H Liu DATE: 04/May/2000 CHECKED BY: S L Chui DATE: 08/May/2000

Samples & tests	Depth (m)	FACE D: 1.5 m	FACE A: 1.5 m	FACE B: 1.5 m	FACE C: 1.5 m	Legend	Grade	Description
ꓵ ↕	0–0.9							Very loose, dark grey, clayey SILT & SAND (TOP SOIL)
ꓵ ↕ ꓵ ↕	1–2	Fill	Fill	Fill	Fill			Dense, dark yellow, clayey to slightly clayey, silty SAND with occasional cobbles/boulders (FILL)
	2–2.8	Filter	Filter	Filter	Filter			Dense, grey, sandy fine to medium GRAVEL (sized rock fragments) (FILTER)
ꓵ ↕	3	Fill	Fill	Fill	Fill			Dense, dark yellow, clayey to slightly clayey silty SAND with occasional cobbles/boulders (FILL)



SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
☒	BLOCK SAMPLE
ꓵ	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



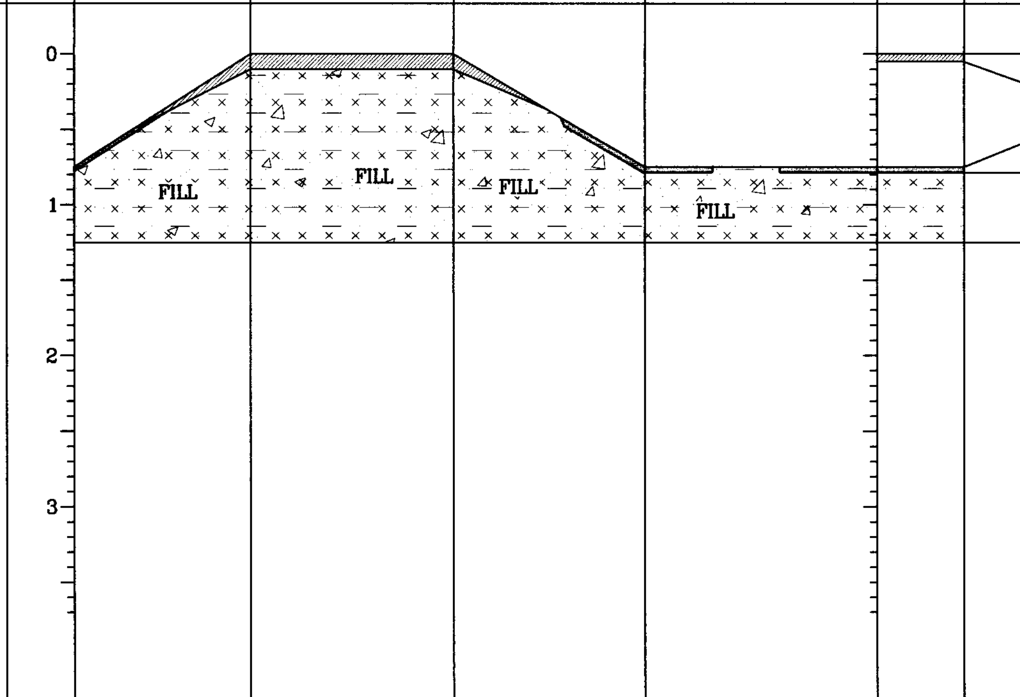
KEY

NORTH ARROW

REMARKS

Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 21 Kau Wah Keng, Castle Peak Road, Kwai Chung, N.T.	TRIAL PIT RECORD		Sheet 1 of 1
	TRIAL PIT No. TP3	EXCAVATION DATE: 02~03/May/2000 BACKFILL DATE: 15~16/May/2000	
	CO-ORDINATES: 832260.1 E, 823047.2 N GROUND LEVEL: 52.3 mPD	LOGGED BY: K H Liu DATE: 04/May/2000 CHECKED BY: S L Chui DATE: 08/May/2000	

Samples & tests	Depth (m)	FACE D: 1.5 m	FACE A: 1.5 m	FACE B: 1.5 m	FACE C: 1.5 m	Legend	Grade	Description



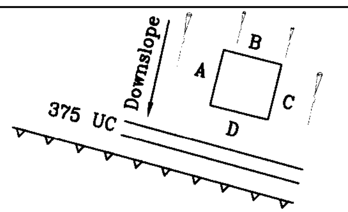
Loose, dark grey, silty SAND (TOP SOIL)

Shotcrete (25mm to 50mm thick)

Dense, brownish yellow, silty slightly clayey SAND with some gravels and cobbles (FILL)

SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

NORTH ARROW

REMARKS

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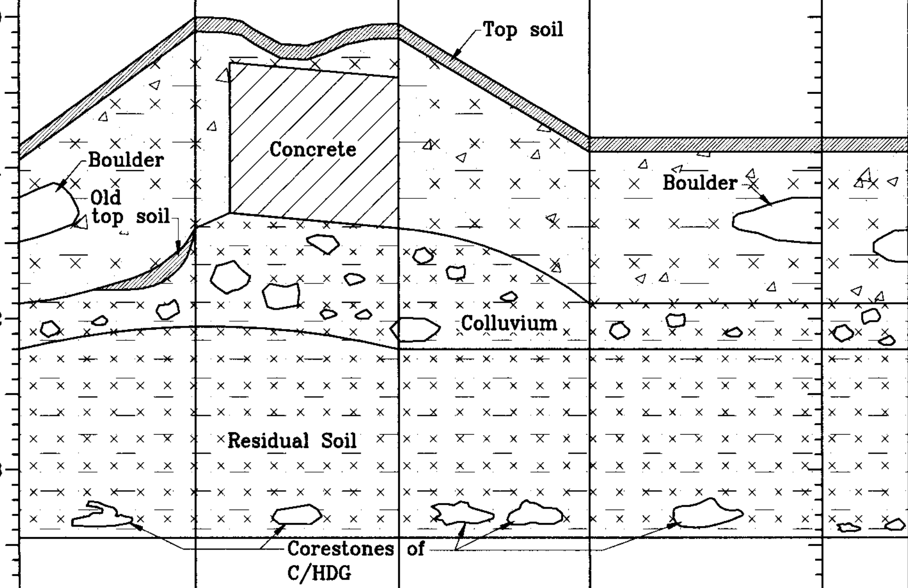
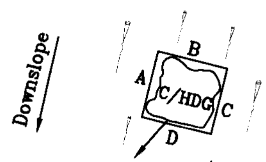
Fugro Maunsell Scott Wilson Joint Venture Agreement No. CE 101/98 1999 Landslide Investigation Consultancy Detailed Study No. 21 Kau Wah Keng, Castle Peak Road, Kwai Chung, N.T.	TRIAL PIT RECORD TRIAL PIT No. TP4 CO-ORDINATES: 832258.0 E, 823072.9 N GROUND LEVEL: 64.0 mPD	Sheet 1 of 1 EXCAVATION DATE: 02~03/May/2000 BACKFILL DATE: 15~16/May/2000 LOGGED BY: K H Liu DATE: 04/May/2000 CHECKED BY: S L Chui DATE: 08/May/2000

Samples & tests	Depth (m)	FACE D: 1.5 m	FACE A: 1.5 m	FACE B: 1.5 m	FACE C: 1.5 m	Legend	Grade	Description
	0							
⊔↕	1							Very loose, dark grey, clayey SILT & Fine SAND (TOP SOIL)
								Dense, dark yellow, silty slightly clayey SAND with some gravels and occasional cobbles (FILL), possibly part of the leakage detection chambers, Concrete structure was revealed on Face B
⊔↕	2							Firm to stiff, moist to dry, yellow and brownish yellow, sandy SILT & CLAY matrix with irregular HDG and H/MDG cobbles soundly embedded (COLLUVIUM - not younger than Holocene - Pleistocene)
⊔↕	3							Firm, moist, yellow, sandy SILT & CLAY with irregular C/HDG corestones at base of the trial pit (RESIDUAL SOIL)



SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

PLAN (NOT TO SCALE)



KEY

NORTH ARROW

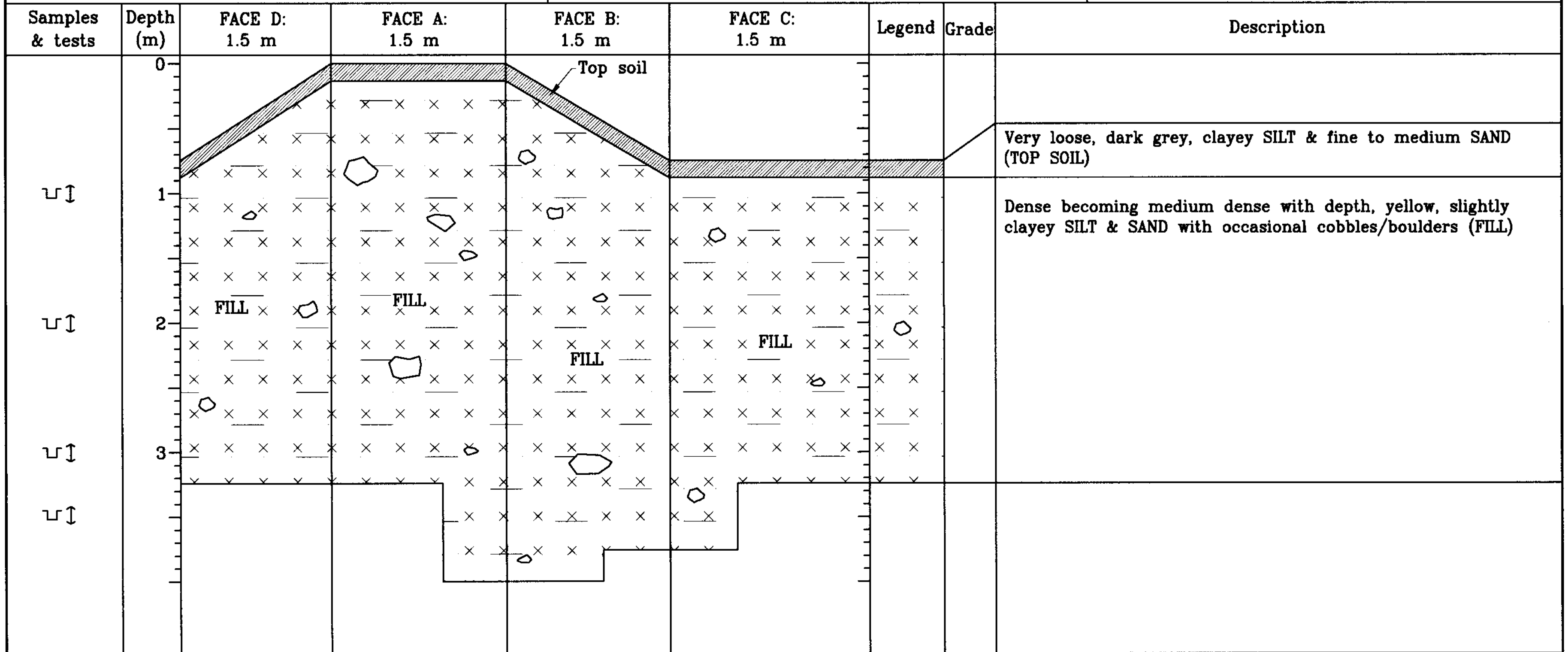
REMARKS

Fugro Maunsell Scott Wilson Joint Venture
Agreement No. CE 101/98
1999 Landslide Investigation Consultancy
Detailed Study No. 21
Kau Wah Keng, Castle Peak Road,
Kwai Chung, N.T.

TRIAL PIT RECORD

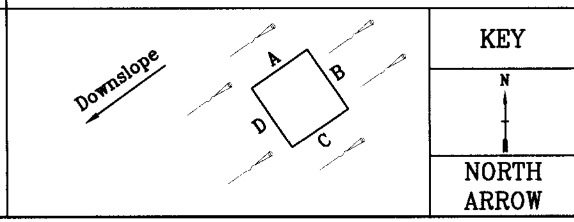
TRIAL PIT No. TP5

EXCAVATION DATE:	02~03/May/2000
BACKFILL DATE:	15~16/May/2000
CO-ORDINATES :	832276.2 E, 823039.4 N
GROUND LEVEL :	57.3 mPD
LOGGED BY:	K H Liu DATE: 04/May/2000
CHECKED BY:	S L Chui DATE: 08/May/2000



SYMBOL	SAMPLES/TESTS/WATER
•	SMALL DISTURBED SAMPLE
↕	BULK DISTURBED SAMPLE
▬	UNDISTURBED SAMPLE HORI. (U100/U76)
▯	UNDISTURBED SAMPLE VERT. (U100/U76)
⊠	BLOCK SAMPLE
⊔	IN-SITU DENSITY TEST
▵	WATER SAMPLE
∿	SEEPAGE

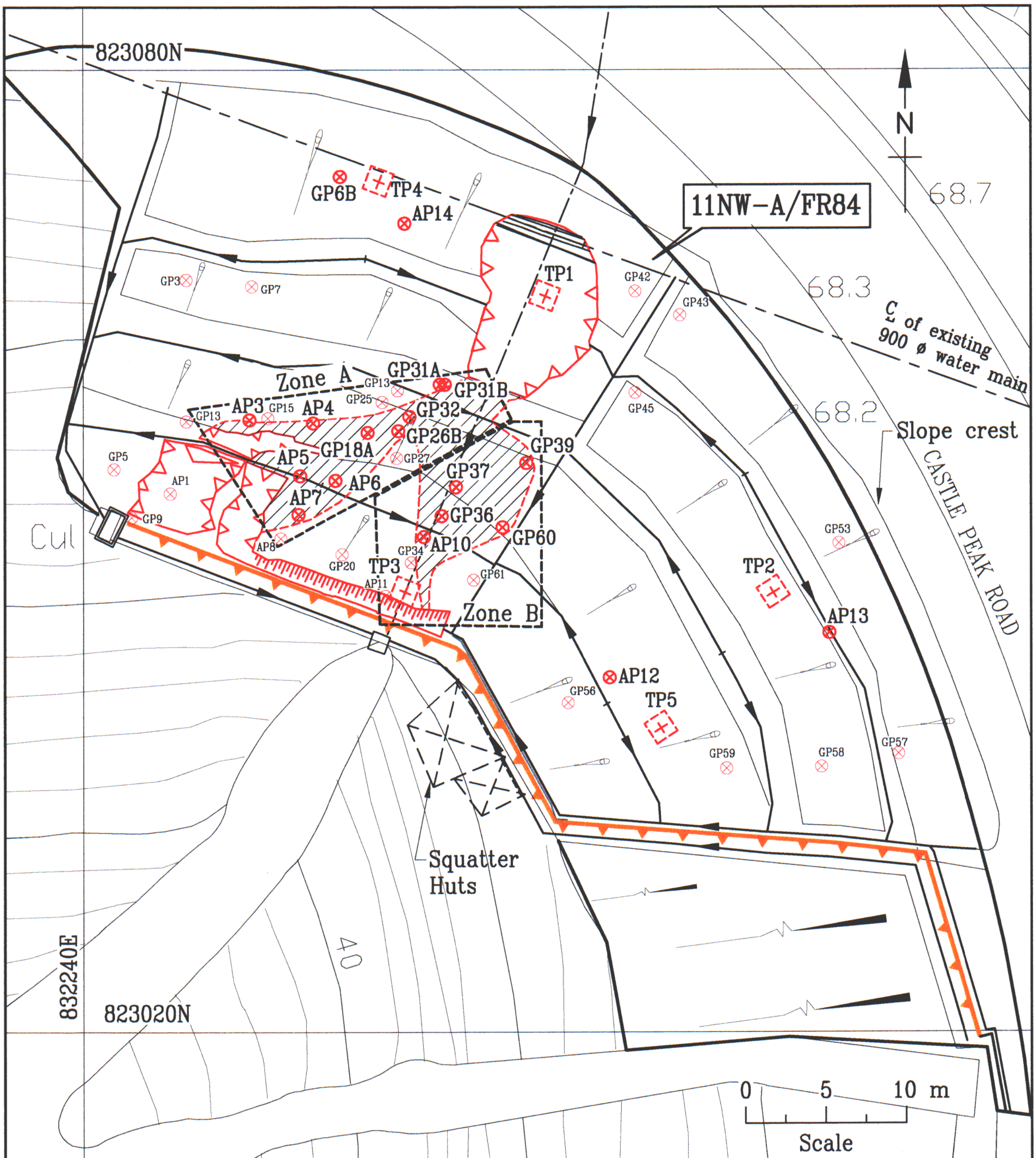
PLAN (NOT TO SCALE)



REMARKS

APPENDIX C

GCO PROBE TEST RESULTS



Legend:

1999 failure scar

Concrete retaining wall

Extent of distressed upstand wall

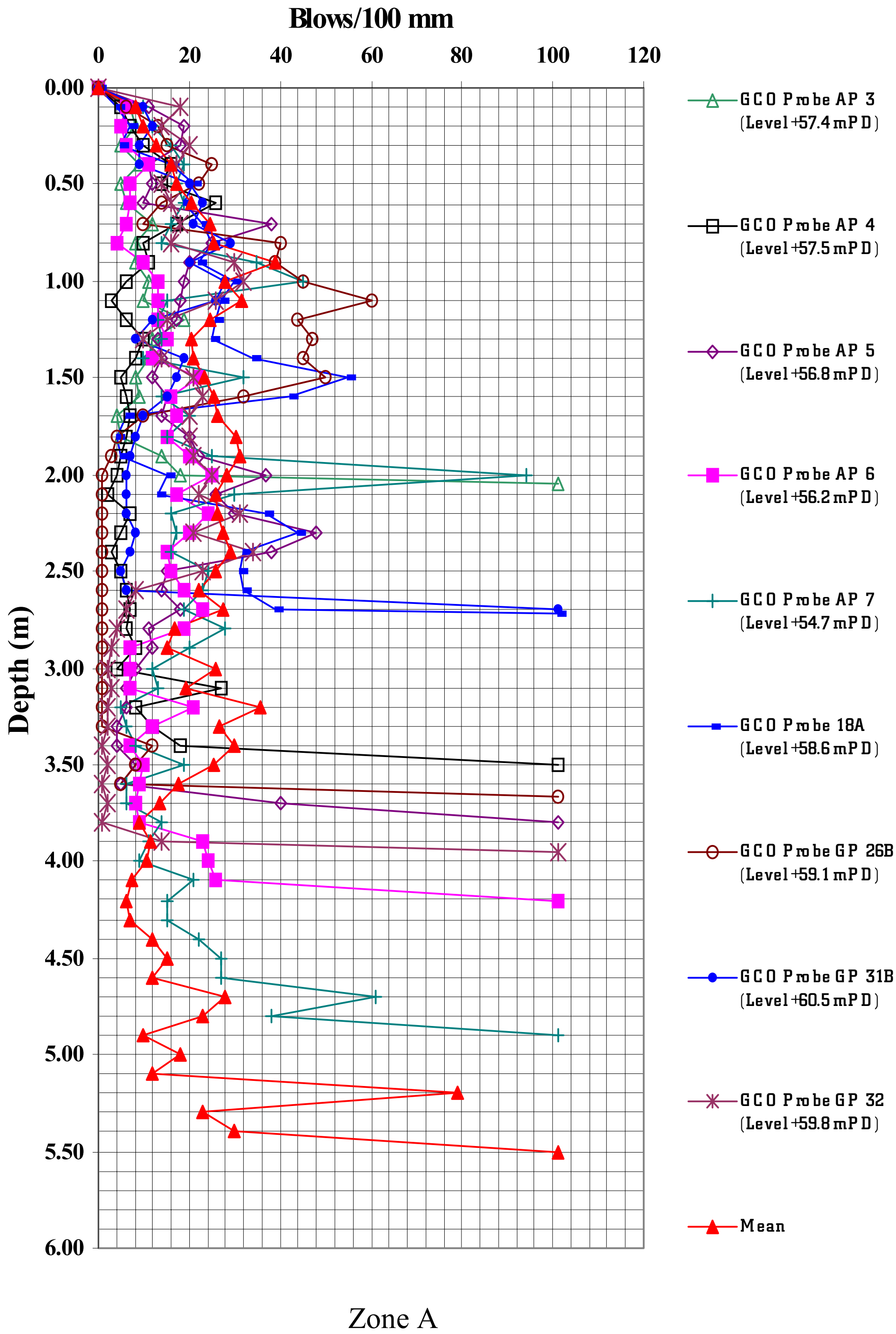
GP1 GCO Probe tests carried out for present study (Probes shown in bold identify probes indicating disturbed ground)

TP1 Trial pit excavation for the present study

Approximate extent of disturbed area based on GCO probe test result

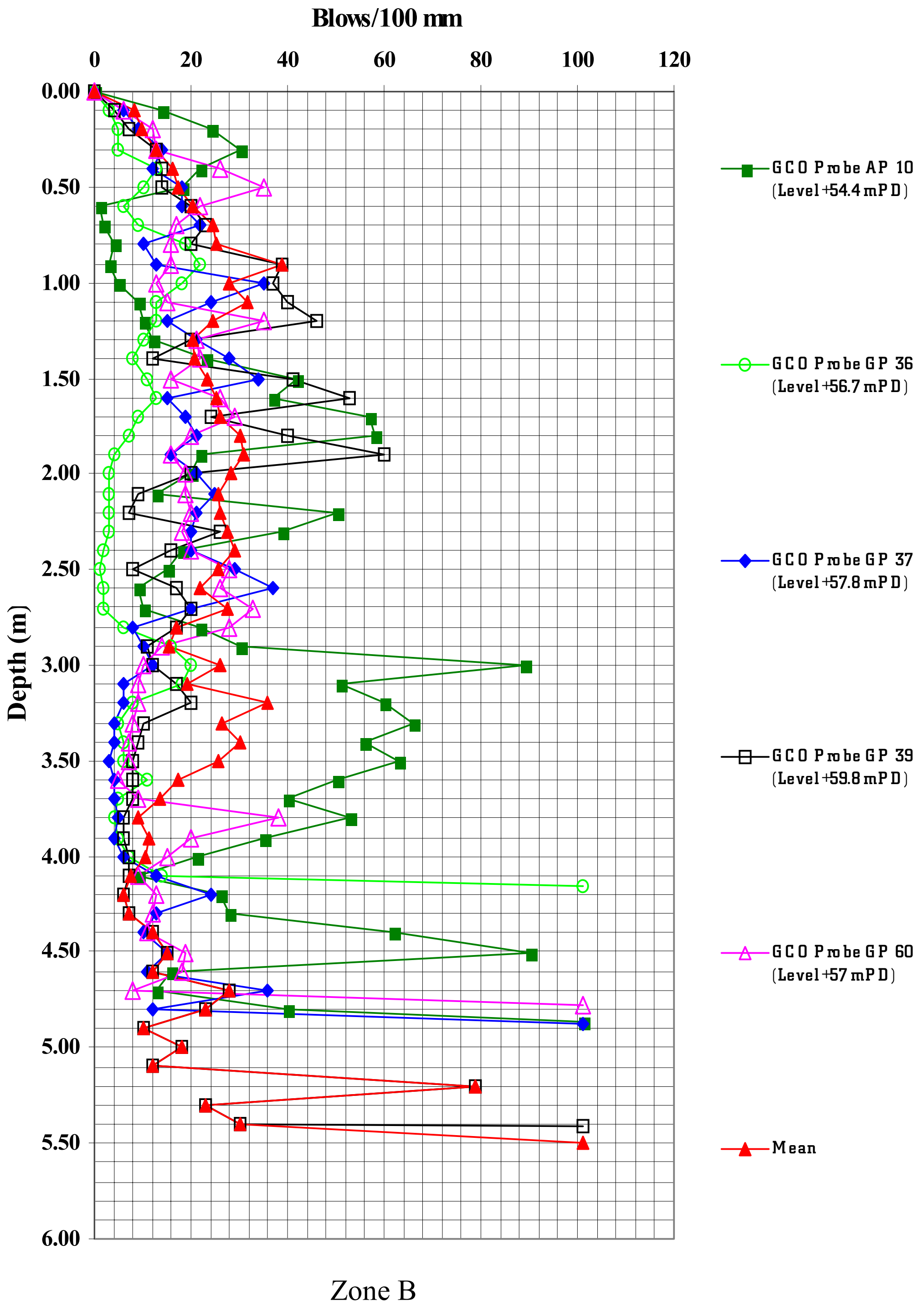
Note: For clarity, GCO probe tests which reached refusal (taken to be > 100 blows per 100mm) at <2.0m depth have not been shown.

Figure C1 - GCO Probe Test Location Layout Plan



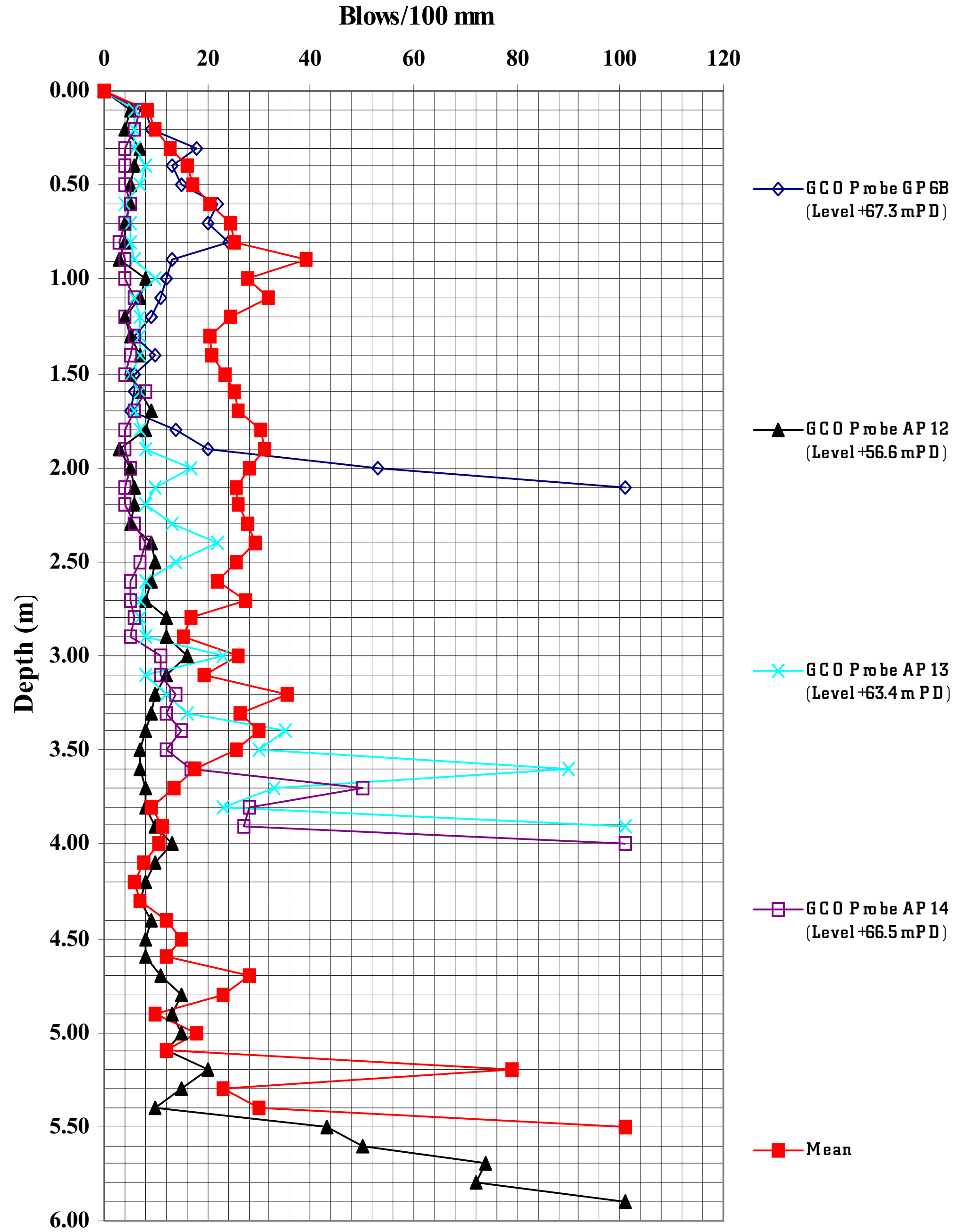
Note:
(1) The blow count for GCO Probes within Zone A indicating disturbance have been shown in full.
(2) The blow count for GCO Probes not indicating disturbance have been shown as a mean plot.
(3) Refer to Figure C1 for the location of the GCO Probe Tests in Zone A.
(4) Refer to Figure 10 for the location of all GCO Probe Tests.

Figure C2 - Plot of GCO Probe Test Indicating Disturbance in Zone A



Note:
(1) The blow count for GCO Probes within Zone B indicating disturbance have been shown in full.
(2) The blow count for GCO Probes not indicating disturbance have been shown as a mean plot.
(3) Refer to Figure C1 for the location of the GCO Probe Tests in Zone B.
(4) Refer to Figure 10 for the location of all GCO Probe Tests.

Figure C3 - Plot of GCO Probe Test Indicating Disturbance in Zone B



Note:
(1) The blow count for GCO Probes outside of Zone A and B, but indicating possible local disturbance have been shown in full.
(2) The blow count for GCO Probes not indicating disturbance have been shown as a mean plot.
(3) Refer to Figure C1 for the location of the GCO Probe Tests plotted above.
(4) Refer to Figure 10 for the location of all GCO Probe Tests.

Figure C4 – Plot of GCO Probe Test Results for Probes Indicating Possible Local Disturbance Outside of Zones A and B