INVESTIGATION OF SOME SELECTED LANDSLIDES IN 1998 (VOLUME 7)

GEO REPORT No. 114

Fugro Scott Wilson Joint Venture

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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Prepared by:

Geotechnical Engineering Office,
Civil Engineering Department,
Civil Engineering Building,
101 Princess Margaret Road,
Homantin, Kowloon,
Hong Kong.

PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. A charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the last page of this report.

R.K.S. Chan
Head, Geotechnical Engineering Office
August 2001

EXPLANATORY NOTE

This GEO Report consists of two Landslide Study Reports on the investigation of selected slope failures that occurred in 1998. The investigations were carried out by Fugro Scott Wilson Joint Venture (FSW) for the Geotechnical Engineering Office as part of the 1998 Landslide Investigation Consultancy.

The LI Consultancies aim to achieve the following objectives through the review and study of landslides:

(a) establishment of an improved slope assessment methodology,

(b) identification of slopes requiring follow-up action, and

(c) recommendation of improvement to the Government's slope safety system and current geotechnical engineering practice in Hong Kong.

The Landslide Study Reports prepared by FSW are presented in two sections in this Report. Their titles are as follows:

The Landslip Investigation Division of the Geotechnical Engineering Office worked closely with the LI Consultants and provided technical input and assistance to the landslide studies.

SECTION 1 : DETAILED STUDY OF THE LANDSLIDES BELOW RAMP G OF THE TING KAU BRIDGE & APPROACH VIADUCT CONTRACT FROM 4 MAY 1998 TO 9 JUNE 1998

Fugro Scott Wilson Joint Venture

This report was originally produced in November 1999 as GEO Landslide Study Report No. LSR 17/99

FOREWORD

This report presents the findings of a detailed study of a series of landslides that occurred between 4 May 1998 and 9 June 1998 at two newly-formed fill slopes below Ramp G of the Ting Kau Bridge and Approach Viaduct Contract along Tuen Mun Road, Ting Kau. Debris from the landslides remained largely on the slopes. Cracking observed in the hard shoulder of Ramp G following the landslides resulted in the closure of one lane of the carriageway. No casualties were reported following the landslides.

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

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

YCKoo

Y C Koo
Project Director/Fugro Scott Wilson Joint Venture

CONTENTS

1. INTRODUCTION

Between 4 May 1998 and 9 June 1998 a succession of landslides occurred in two newly-formed fill slopes located at Ting Kau below Ramp G of the Highways Department (HyD) Contract HY/93/38: Ting Kau Bridge and Approach Viaduct (TKB&AV Contract), which was let by HyD under a design and build arrangement to the Ting Kau Contractors Joint Venture (TKCJV). The landslides affected about 40% of the slope face area (3300 m^2 out of 7200 m^2). Landslide debris remained largely within the slope boundaries. Cracking in the soft verge and road pavement behind the crest line of the slopes was also observed. No casualties were reported following the landslides.

Fugro Scott Wilson Joint Venture (FSW, the 1998 Landslide Investigation Consultants) carried out a detailed study of the failures for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. 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 landslides, present relevant background information and establish the probable causes of the failures. The scope of the study was limited to site reconnaissance, 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 tasks:

(a) a review of relevant documents relating to the site,

(b) analysis of rainfall data,

(c) discussions with various parties involved in construction,

(d) observations and measurements at the site,

(e) engineering analysis of the slopes, and

(f) diagnosis of the probable causes of the landslides.

2. THE SITE

2.1 Site Description

The two newly-formed fill slopes are located below the south-east edge of Ramp G of the TKB&AV Contract, off Tuen Mun Road at Ting Kau (Figure 1 and Plate 1). Ramp G provides access to the west-bound lanes of Tuen Mun Road from Ting Kau Bridge.

The natural topography surrounding the landslide site comprises a series of south-easterly trending spurs and valleys. Tuen Mun Road follows an approximate north-east to south-west alignment and has been formed in cut and fill. The portion of Ramp G within the landslide site has been formed mainly by the placement of additional fill to the Tuen Mun Road embankment, extending three existing fill slopes.

The Ramp G carriageway comprises a flexible pavement and a conventional surface drainage system with a concrete kerb arrangement to direct runoff along the edge of the carriageway into gully pits. The extent of filling works in relation to the road construction is shown in Figure 1.

The north-eastern fill slope (designated Slope A in this report) is approximately 80 m long and 28 m high, comprising four batters with a face angle of 30° to 39° separated by three 1.5 m-wide berms. A crib wall is located along the toe of the slope with a maximum height of about 5 m. The south-western fill slope (designated Slope B) is approximately 130 m long and 40 m high, and comprises seven batters (face angle of about 33°). The two slopes are approximately triangular in plan and both flanks meet at the valley floor. The slopes are separated by a spur line trending south-east. The cover of both slopes consisted of hydroseeded grass. Both slopes have a surface drainage system comprising open surface channels on berms draining into stepped channels located around the perimeter of the slopes which discharge into the natural stream courses extending beyond the slope toes.

The valleys extending beyond the toe of each slope fall away to Castle Peak Road, located 100 m to 150 m to the south-east, and are largely uninhabited. Private housing developments "Vista Del Mar" and "Villamar" are located on the spur line separating Slopes A and B, and a second spur line to the north-east of Slope A respectively (Figure 1).

2.2 Geology

Sheet 6 of the Hong Kong Geological Survey 1:20 000 scale Map Series HGM 20 (GCO, 1988), the relevant portion of which is reproduced in Figure 2, indicates that the solid geology of the landslide site and its surroundings comprises predominantly fine-grained granite with megacrysts, with a body of medium-grained granite indicated to the south-west. The geological boundary between the two formations is oriented in a north-south direction and is located near the south-western end of Slope B.

Predominant joint sets in the fine-grained granite are indicated as generally dipping steeply to the north with a dip angle of 70° to 80°.

2.3 Water-carrying Services

A plan of the landslide site showing the locations of existing utilities and services is presented in Figure 3.

The only water-carrying services in the vicinity of the landslides comprise stormwater drainage provisions associated with Tuen Mun Road and the adjacent roadside slopes. Three 900 mm diameter culverts (referenced P042, P043 and P044) traversing Tuen Mun Road from catchpits collect runoff from slopes along the north-western boundary of the road. The culverts pass beneath the fill slopes to discharge into surface drainage along the perimeter of the new works (P042 and P044), or directly into the natural drainage channel at the slope toe

(P043). In the case of culvert P042, the exit point into the surface drainage works for Slope A is relatively high on the slope, being located a short distance below the uppermost berm. Desk study indicates that the precast culverts were installed during the original Tuen Mun Road formation in 1975 and have been extended under the TKB&AV Contract (Figure 3) to suit the new slope geometry.

New stormwater drainage provisions installed in 1997/98 include a 450 mm diameter pipe located along the crest of the new slopes and outlet structures, which discharge runoff collected from the road into surface drainage on the slope face mid-way down the uppermost batter of both slopes (one outlet on Slope A and two on Slope B).

Subsurface drains are present along the crest of Slope A within the carriageway. These comprise a gravel-filled trench with concreted invert and perforated pipes, which discharge into the road drainage system.

2.4 Maintenance Responsibility

The affected fill slopes, which are yet to be registered in the GEO's New Catalogue of Slopes, are presently within the maintenance period for the TKB&AV Contract. HyD will be responsible for maintaining the slopes when they are handed over by the Contractor.

3. SITE HISTORY AND PREVIOUS STUDIES

3.1 General

The site history and details of previous studies have been compiled through a review of the available documentation and aerial photograph interpretation (API). Detailed findings of the API are summarised in Appendix A and Figure 4.

3.2 Site History

Aerial photographs from 1963 to 1972 indicate a natural hillside at the landslide site comprising well-defined spurs and valleys trending south-east and vegetated with low scrub and small isolated erosion scars (Figure 4).

The Tuen Mun Road Stage I works commenced in October 1974 with Scott Wilson Kirkpatrick and Partners (SWKP) as consultants to the Highways Office (HO). The Stage I works are first visible in the 1975 aerial photographs. Site formation works within the landslide site resulted in the formation of cut slopes along the north-western edge of the Tuen Mun Road corridor, and infilling of three natural valleys, with the resultant embankment slopes (subsequently registered in the 1977/78 Catalogue of Slopes as Nos. 6SE-C/FR13, 6SE-C/F14 and 6SE-C/F15 respectively) extending to the south-east.

Formation of the above fill slopes (Figure 4) was completed by October 1976. Slope No. 6SE-C/FR13 comprised a single batter from crest to toe and was 60 m long and 16 m high, with a face angle of 33° and a 4 m to 5 m high masonry retaining structure at the toe

(Binnie and Partners (B&P), 1983a). Slope Nos. 6SE-C/F14 and 6SE-C/F15 each comprised two batters separated by an intermediate berm and had height and face angle of 22 m and 33°, and 24 m and 31°, respectively (B&P, 1983 b & c).

The Stage I fill slope construction, as indicated by As-constructed Drawings (SWKP, 1979a to 1979e) included benching of the original ground surface and placement of a drainage blanket prior to placement of the fill material. Gravel-filled subsoil drains were installed along the natural stream courses with discharge provided at the slope toe. Culverts were installed through the fill embankments to drain runoff from the hillside and slopes on the opposite side of Tuen Mun Road.

The Tuen Mun Road Stage I works continued through 1977 and 1978. The Tuen Mun Road Stage II works commenced in late 1978 and continued through to 1982. These works appear to have had little effect on the fill slopes, which are observed to remain in good condition in 1978 photographs.

Between 1982 and 1994 little change is observed in the three fill slopes. Erosion features can be observed in the 1985, 1986, 1987, 1990 and 1993 aerial photographs (Figure 4), with periodic maintenance indicated by the clearing of drainage channels and re-establishment of vegetation in 1986, 1991 and 1994. The (September) 1995 aerial photographs indicate major works associated with the Tuen Mun Road Widening, Route 3: Country Park Sector and the TKB&AV Contract underway in the vicinity of the existing fill slopes, which remain unaffected by these works, but erosion features are visible on the slopes.

A landslide occurred in the lower portion of slope No. 6SE-C/FR13 in October 1995 (Figure 4 and Plates 2 to 4). The volume of the landslide, estimated at between 50 m^3 and 100 m^3, occurred within an active construction site and was not reported to the GEO. Temporary stabilisation measures comprised placement of rockfill ballast to the landslide scar.

Construction works associated with the Ramp G fill slopes commenced in late 1995. The (November) 1996 aerial photographs indicate extensive earthworks on slope No. 6SE-C/FR13, including the formation of a haul road, trimming of the natural ground to the south-west of the existing fill. The 1997 photographs indicate earthworks continued on slope No. 6SE-C/FR13 and construction activities underway on slope Nos. 6E-C/F14 and 6SE-C/F15, comprising stripping of the slope faces and trimming of the spur separating the two slopes, resulting in the formation of a single combined face in preparation for the placement of additional fill. The completed Ramp G slope works are indicated in Figure 1.

3.3 Previous Studies

Specific details of the Tuen Mun Road Stage I fill slopes design could not be located. However, subsequent correspondence from early 1977 has been located which indicates that further stability analyses were carried out during construction of the concerned fill slopes, based on soil parameters derived from laboratory testing of samples of fill compacted to a range of densities. The analyses resulted in a number of slopes being flattened to a gradient of IV : 2H from the original design gradient of 1V : 1.5H. The formation of fill slope

Nos. 6SE-C/FR13, 6SE-C/F14 and 6SE-C/F15 at the original design gradient of 1V : 1.5H suggests that the further analyses confirmed adequate stability for these slopes.

In October 1976, the Civil Engineering Office informed other Government departments of an upgraded specification for fill slope construction. This required compaction to at least 95% of the standard maximum dry density (MDD) for new slopes and for investigation, testing and analysis of existing fill slopes.

A series of re-assessments of the Tuen Mun Road Stage I fill slopes followed in response to this requirement by SWKP, HO and subsequently the Geotechnical Control Office (GCO, re-named GEO in 1991). The objective of the assessments was to determine the need for recompaction of the fill slopes. It was found that the average degree of compaction of the Stage I fill slopes was about 92% to 94% of the MDD. Stability analysis by SWKP gave calculated factors of safety of about 1.4 for the three concerned fill slopes.

Correspondence in May 1983 confirmed the acceptance of the Stage I fill slopes by GCO subject to a number of conditions (GCO, 1983), including acceptance of maintenance responsibility by the HO and prohibiting new residential development in the area beyond the toe of the slopes which could be affected by potential failures.

In May 1983, the three fill slopes were inspected by the GCO. Field sheets from these inspections were subsequently included in the report "Preliminary Studies for the Special Investigation into Fill Slopes" commenced in 1977 by B&P (B&P, 1983a, 1983b and 1983c). The field sheets indicate that the slopes are in a reasonable condition, though cracking in surface channels is common to all three slopes, minor erosion and gullying are indicated for slope Nos. 6SE-C/FR13 and 6SE-C/F15 and "movement" is noted in the lower portion of slope No. 6SE-C/F14.

The TKB&AV Contract was let by HyD under a Design and Build arrangement in early 1994 to the TKCJV, who engaged Binnie Consultants Limited (BCL) as their designer for geotechnical works. Independent checking was carried out by Flint & Neil (F&N) who was appointed directly by HyD. Initial geotechnical submissions were made by BCL in September 1994 to the HyD and the GEO. Submissions relating to the geotechnical works continued until late 1996.

Response from the Mainland West Division of the GEO in October 1994 limited their comment to the design basis, stating that "A safety factor of 1.4 (for soil slopes) is considered acceptable", groundwater conditions "...shall be in accordance with the "Geotechnical Manual for Slopes [GCO, 1984]", and "design parameters and assumptions .... shall be subject to review as ... results become available". Agreement was reached between the GEO and the HyD in respect of limiting the GEO's involvement to an "In-Principle" check of the geotechnical design. This is supported by a later correspondence from the GEO District Division which states "..... as the report is to be checked in Detail by the Design Checker, ... have only examined ..... technical standards".

BCL submitted a "Design Manual for Ramp G Fill Slope/Crib Wall" in August 1996 (BCL, 1996a). In relation to the new fill slopes, the manual specifies compliance with the earlier submission in November 1994 presenting the design aspects of new slope works "Design Manual -/DM170.1..." (BCL, 1994). This earlier document states that the fill materials shall be in accordance with "Section 6 Volume 10, Outline Construction Specification". The HyD have advised that this specification is basically identical to the General Specification for Civil Engineering Works (Government of Hong Kong, 1992) and therefore the compaction of soil fill to at least 95% of MDD is a requirement of the TKB&AV Contract.

BCL also submitted a "Geotechnical Interpretative Report: Ramp G Fill Slope and Crib Wall" (GIR) (BCL, 1996b). The report includes logs and testing and piezometer monitoring data from eleven drillholes and four trial pits. An assessment of the geological profile and laboratory test results was made and design soil parameters were specified. The soil types and associated design parameters differ between the texts of the design manuals and the GIR. However, the analyses presented in the GIR were based on parameters specified in the earlier of the two design manuals (BCL, 1994).

The groundwater conditions for both proposed Slopes A and B were assessed using data from four piezometers within Slope B. Two of these were located in the crest area and the other two were located near the toe of the pre-existing fill slopes. The landslide that occurred on fill slope No. 6SE-C/FR13 (beneath Slope A) in 1995 was back-analysed. This yielded an estimated groundwater level at failure which was higher than the monitored levels. Design phreatic surfaces based on the back-analysed surface were used in the subsequent stability analyses for Slopes A and B.

Three cross-sections (two through Slope A, with and without the crib wall, and one through Slope B) were analysed using the Morgenstern & Price (1965) method of analysis and only deep-seated failures were analysed. The design parameters for the new CDG fill are: $c' = 0$ and $\phi' = 39°$. It is noteworthy that the report states that “The analyses have been run on the basis that shallow slips in the landscaping medium, i.e. CDG, fill have been ignored”. In all cases, a factor of safety in excess of 1.4 was achieved. Sensitivity analyses were undertaken to assess the effect of an increase in groundwater level up to the base of the proposed rock fill layer (i.e. base of the new works). The minimum factor of safety achieved in these analyses was in excess of 1.1.

In April 1995, the existing fill slopes were inspected by consultants appointed by HyD under the project entitled “Roadside Slope Inventory and Inspections”. The Engineer Inspection Records (FMR Consultants, 1995) indicate the overall state of maintenance as “fair” to “good”.

The GEO initiated the consultancy agreement entitled “Systematic Inspection of Features in the Territory” (SIFT) in 1992 which, inter alia, aims to identify features not registered in the 1977/1978 Catalogue of Slopes and to update information on registered slopes, based on studies of aerial photographs and limited site inspections. The findings of Phase 2 SIFT Study Reports for the three fill slopes, compiled in June 1996, are summarised as follows:

(a) 6SE-C/FR13: assigned Class “WIP” (i.e. works in progress).

(b) 6SE-C/F14: assigned Class “B1” (i.e. assumed not to have been checked by GEO).

(c) 6SE-C/F15: assigned Class “B1”.

A routine maintenance inspection was carried out by HyD on slope No. 6SE-C/F15 in December 1996. General recommendations were made for routine maintenance, i.e. clearance of drainage channels, repair of cracking and removal of vegetation.

4. DESCRIPTION OF THE LANDSLIDES

4.1 Description of the Landslides

4.1.1 General

The landslides comprise a number of separate failures recorded during three separate rainstorm events. The final extent of the failed areas (not including debris runout) amounts to about 3300 m^2 in plan, which corresponds to about 40% of the combined face areas of Slopes A and B. Both slopes were affected to a similar degree.

4.1.2 Landslide No. 1 on 4 May 1998

The description of the initial incidence of landslides recorded on 4 May 1998 has been based on a review of record photographs (Plates 5 to 18) and post-failure site observations.

Failures occurred mainly in Slope B with an estimated failure volume of about 500 m^3 (Figure 6 and Plates 6, 7 and 15). The individual landslide scars were generally confined to the batters (Figure 7). The surface of rupture was typically 0.3 m to 0.4 m below, and approximately parallel to, the slope face, with fairly well-defined sub-vertical flanks. The mode of failure involved principally a shallow planar slide of the near-surface material (Plates 8 to 11 and Plate 15). Secondary outwash was also observed at isolated locations (Plates 9 to 14).

Debris from the landslides was deposited on berms, blocking the berm channels, and in many cases flowed down onto the batter below (Plates 10, 13 and 17). The debris comprised material of a silt/sand particle size and was in a relatively wet state (Plates 8, 9 and 11) immediately following failure. In some instances (Plate 9) the hydroseeding, although disturbed, gives the appearance of debris having moved down-slope as discrete units or slabs. The travel angle of the debris (Wong & Ho, 1996) was about 33°, which is within the typical range for rain-induced sliding failures of fill slopes.

A succession of scars/runouts was observed over almost the full height of the slope, with the length/width ratios of the failures on the uppermost batter being at the high end of the observed range. Failures also occurred on either side of the road drainage outlets on the slope face, blocking the berm drainage channels. A succession of scars with pronounced secondary outwash was observed beneath the southern drainage outlet. A similar observation is made in respect of the northern drainage outlet, where a broad scar is present beneath a 20 m length of berm channel isolated by debris from failures in the top batter at either end and fed by the road drainage outlet.

Following the landslides, cracking was observed at the crest of Slope B (Plate 5) and between the crest of the Slope A and the kerb (Plate 18). Separation between berm apron slabs and U-channels was also noted on Slope B (Plate 16).

4.1.3 Landslide No. 2 on 24 May 1998

The description of the second incidence of landslides is based on site observations and a review of record photographs (Plates 19 to 24).

Failures occurred mainly in Slope A (Plate 19) with an estimated total volume of about 400 m^3 (Figure 5). The mode of failure comprised shallow sliding along a surface of rupture about 0.3 m to 0.4 m below and parallel to the slope face (Figure 7).

Debris from the landslides comprised essentially fine material deposited on berms, blocking the berm channels, and in many cases flowed down onto the batters below. A succession of scars/runouts was observed over almost the full height of the slope. The debris deposition zone extended beyond the toe of the slope (Plate 20), with a travel angle of about 33°.

The photographic records suggest that the cribwall at the toe of Slope A (Plates 20 & 22) was not significantly affected by the landslides, when compared to the initial condition (Plate 49), except for the undermining of the access stairway at the toe of the wall (Plate 23).

Additional cracking was also noted in the hard shoulder behind the crest of Slope A, as well as between the hard shoulder pavement and kerb at the crest of Slope B (Plate 24).

4.1.4 Landslide No. 3 Between 24 May 1998 and 9 June 1998

A third incidence of failures was identified by FSW following a comparison of observations and record photographs (Plates 25 to 37) on 9 June 1998 and 15 June 1998 with the photographic records from the previous two incidences.

The failures were generally limited to Slope B with an estimated combined volume of about 400 m^3 and affected previously intact areas of the slope. The mapped extent of the failures and the condition of existing landslide features are indicated on Figure 6.

The landslides generally occurred in the third batter (52 mPD to 60 mPD) below the slope crest (Plate 25) with an approximately 30 m wide progression of failures and debris runout in lower batters to the slope toe from the 52.5 mPD berm (Plate 26). A failure was also noted at the northern end of the slope below the 37.5 mPD berm (Plate 28).

The mode of failure comprised shallow sliding along a surface of rupture located 0.3 m to 0.4 m below and parallel to the slope face (Figure 7 and Plate 25). Secondary outwash of the scars was observed, cutting sharp channels and exposing rock fill material beneath a thin (200 mm to 300 mm) layer of finer material (Plate 27). Debris runout extended beyond the slope toe with a travel angle of about 33°. Debris included gravel and cobble-sized particles.

4.2 Observations Prior to the Landslides

The Ramp G fill slopes were completed in early 1998. No instability in the new works was recorded between the time of completion and the initial failure on 4 May 1998.

It is noted that retro-planting of tree seedlings was underway on Slope B at the time of the initial failure on 4 May 1998. This is indicated by comparison of Plates 6 and 7, which show retro-planting underway. The operation involved manual excavation of small pits about 0.2 m by 0.2 m in plan on the slope face on a regular grid of about 2 m and placement of excavated material back into the pits after planting.

4.3 Observations Following the Landslides

The initial failure at Slope B was first noted by the site staff at around 09:00 hours on 4 May 1998. No definite time of failure could be established. Following the landslide, plastic sheeting was placed over the scars and surface drainage channels were cleared. The failure was not reported to the GEO.

The second incidence of failure was reported at around 20:00 hours on 24 May 1998 by the Hong Kong Police Force (HKPF) who observed cracking in the hard shoulder of Ramp G behind the crest of Slope A. No definite time of slope failure could be established.

The GEO inspected the failure on 30 May 1998, following the report by HyD on 29 May 1998. The inspection was limited to Slope A. Recommendations made by the GEO included the provision of plastic sheeting to cover the exposed scars and closure of the hard shoulder to traffic.

A joint site meeting was subsequently held between the GEO District Engineer and HyD site personnel to discuss primarily the cracking in the hard shoulder pavement and between the pavement and the adjacent kerb. It was noted that the cracking was being monitored and that settlement monitoring stations had been established along the hard shoulder and on berm apron slabs.

During May 1998, the designer of the geotechnical works, Binnie Black and Veatch Hong Kong Ltd. (BBVHKL, previously BCL until 1997) visited the site. It was noted that "Debris scars through the CDG fill layer were observed", and that a particular scar was "..large and deep with some rockfill exposed". It was further noted that CDG fill "..that had not failed was loose without any sign of horizontal compaction layers ..", and that "..the CDG fill had not been properly compacted in layers .." (BBVHKL, 1998).

The TKCJV subsequently engaged ESA Consulting Engineers Ltd. (ESA) to establish the causes of the landslides and cracking in the Ramp G pavement. It was noted by ESA (1998b) that "..slumping is generally confined to the upper "soil" layer .." and that

" ..subsequent erosion of the exposed CDG fill had also taken place in the slumped areas of the slope face".

FSW staff first visited the site on 9 June 1998 and again on 15 June 1998 and 9 July 1998. At this time, the majority of Slope A was covered with tarpaulin, as was a substantial portion of Slope B, which prevented a detailed inspection of the scars. It was observed that the most extensive distress at the slope crest comprised separation of the kerb and road pavement over the majority of the combined length of the slopes by up to 10 mm (Plate 36). The cracking within the hard shoulder adjacent to Slope A was approximately 15 m long, 3 mm wide located 2 m to 2.5 m behind the kerb (Plate 37).

During heavy rainfall on 9 June 1998 (hourly rainfall of 30 mm recorded), no overtopping of the kerb was observed during the inspection by FSW.

It was further noted during the 9 July 1998 inspection that a trial pit (ATPI on Figure 8) about 1 m to 1.5 m deep, excavated as part of the TKCJV investigation into the landslides at the location of the cracking behind the crest of Slope A, revealed that the cracks extended vertically into the rock fill beyond the base of the pit (Plates 38 and 39).

A Closed Circuit Television (CCTV) survey was subsequently arranged by FSW in April 1999 to assess the portion of Culvert No. P043 located beneath Slope B (estimated depth 3 m to 15 m below the slope face) and the 450 mm diameter stormwater drain located along the crest of Slope A with a measured depth of about 1.5 m to 2.5 m below road level (Section 2.3).

The objective of the CCTV survey was to check for signs of displacement or damage, which might indicate mass movement of the surrounding ground and possible presence of a deep-seated instability within the slope. Survey of the stormwater drain was aimed at the identification of any damage or displacement resulting from the observed settlement at the slope crest to indicate the extent of the settlement behind the crest and the potential for leakage.

The CCTV survey was carried out on 28 April 1999 (EGS, 1999). Culvert No. P043 was found to be in fair condition with no major defects or signs of gross movement. Survey of the 100 m length of stormwater drain along the crest of Slope A indicated no signs of damage or defect.

5. SUBSURFACE CONDITIONS

5.1 General

The subsurface conditions within the landslide site have been assessed on the basis of information obtained from the desk study and ground investigation carried out by ESA and BBVHKL following the failure.

5.2 Previous Ground Investigation

A number of previous ground investigations were carried out at the landslide site (Figure 8). These investigations provide information on the general stratigraphy, except for the recently placed fill.

5.3 Construction Records

5.3.1 Construction Drawings

The relevant parts of the construction drawings are presented in Figures 9 to 11. Figure 9 shows the typical slope construction. It is noted that:

(a) no minimum or maximum thickness of the "decomposed granite (DG) plant support medium" (designated "DG fill" in this report) has been specified,

(b) a straight dashed line is used to define the boundary between the DG fill and the "soil" (designated "landscaping fill" in this report). No indication of surface preparation of the DG fill (i.e. benching) prior to the placement of the landscaping fill is indicated,

(c) the specification of the landscaping fill permits material " ..taken from top layers of excavation elsewhere or made up by mixing DG with soil conditioner in ratio 3:1 ..", and

(d) the apron formed together with the berm drainage channel does not cover the full width of the berm, with an uncovered surface of around 500 mm wide.

Figure 10 indicates the typical construction of the crib wall. Figure 11 shows the typical section applicable to the Ramp G carriageway construction.

5.3.2 Construction Methods

The method statement by TKCJV (1997) presents details of slope formation as follows:

" - When access to the lower part of the slope is available work on the toe drainage can be carried out, e.g. extension of 900Ø drain, head wall, cut off drain and U channels etc.
- When the associated drainage work is completed the existing fill slope will be benched to receive the filter layers. Before laying Filter A SOR's staff will be notified to inspect the condition of the prepared slope surface and their permission obtained.
- A backhoe will be used for spreading filter materials, assisted by manual labour.

- Grade 400 rock will be obtained mainly from the project "Route 3 Country Park Section" and partly from Ramp G and Ramp H excavation.
- By inspection oversized rock will be broken down into smaller pieces and undersized material will not be used and disposed off site.
- Profile rails will be set out by subcontractor's surveyor and checked by TKC's surveyor at the toe of the fill slope.
- Surface of deposited rock will be sealed with fragments before compaction (OCS Clause 6.44 (2)).
- Rockfill is then placed and compacted in approximately 500mm layers by 10 passes of a Vibratory roller "BOMAG BW212D". (Catalogue enclosed).
- Where locally it is impossible to operate a big roller due to confined space the rockfill will be compacted by the bucket of a backhoe.
- Rockfilling operation will continue keeping the face of slope within the reach of a backhoe. The surface of slope will be trimmed back to a smooth profile.
- Benching for soil infill will be cut in the rockfill. Soil is then spread and compacted by a 3-tonne baby roller in 200 mm thick layers.
- The berm above the lower slope is then formed. Rockfilling proceeds on the slope above this berm and during this operation soil mixed with conditioner (300-500 mm thick) is spread on the lower completed slope and drainage work on the berm starts. A mini-backhoe may be used for this work.
- The same sequence repeats until the top of slope reaches Ramp G formation. ”

A checklist attached to the method statement indicates the various “Witness/Hold Points” during the construction process. Also attached is an “Inspection, Test and Monitoring Plan”. DG fill and landscaping fill do not appear in the list of materials included in the plan.

5.3.3 Record Photographs

The early stages of formation of Slope A over the existing slope No. 6SE-C/FR13 in April 1997 are presented in Plates 40 to 42. Plate 40 shows the replacement reservoir structure completed at the toe of Slope A and the mass concrete portion of the crib wall footing. A general view across Slope A is presented in Plate 41, which shows the haul road formation and stockpiling of rockfill. A caption on the HyD record photograph index states “.....rockfill material which were too fine”.

Plates 43 and 44, also taken in April 1997, show the initial works for the Slope B formation, with the extension of the existing culvert P043, located beneath slope No. 6SE-C/F14 to the new slope toe. Plate 44 also provides a general view across the slope and the spur line separating this feature from slope No. 6SE-C/F15.

Plate 45 indicates the progress of the crib wall footing construction by mid-September 1997, and Plate 46 provides a general view across the Ramp G works from late October 1997, with bulk filling works for Slope B and haul road construction visible on the left of the frame.

Plate 47 provides the initial indication of slope formation at Slope A in late October 1997. Compaction plant is just in frame on the left, and an excavator can be seen trimming/tamping the DG fill placed over the rock fill. Of significance is the planar face formed in the DG fill and the upstand remaining on the drainage channel at the slope perimeter, indicating that the final 300 mm to 500 mm layer of landscaping fill is yet to be placed. Inclusion of some rock fill in the DG fill is also visible.

Plate 48, taken five days after Plate 47, shows the landscaping fill placed over the DG fill on the lowest batter of Slope A, and surface drainage formed on the 64.5 mPD berm above. A significant quantity of fine material is visible in the rock fill.

Plates 50 to 58 give an indication of the slope formation works carried out on Slopes A and B from late October 1997 through to essential completion at the end of 1997. The series of plates is deliberately repetitive in order to demonstrate that construction techniques were apparently applied consistently. The photographs do not provide clear confirmation of the placement of the filter layer at the existing fill surface prior to rockfilling. The main points to note are as follows:

(a) the height between benches in the rock fill prior to placement of DG fill appears to be often far in excess of the 2 m maximum indicated on the working drawings (Plates 50, 52 and 53 and Figure 9),

(b) very little overfilling of DG fill appears to have been applied in practice (Plates 50 and 56),

(c) the sloping face of the DG fill was formed to a planar profile (i.e. no benching) prior to placement of the landscaping fill (Plates 51 to 56),

(d) the DG fill layer was of the order of 200 mm to 300 mm thick in places (Plate 53) and compaction plant was applied to the sloping face in these instances, rather than on a horizontal plane,

(e) placement of the landscaping fill appears to have been by tipping from the berm above (Plates 52 and 54), and

(f) the apron slab formed adjacent to the surface drainage channels on berms did not extend to the edge of the berm (Plates 57 and 58), resulting in an exposed horizontal soil surface between the edge of the apron slab and the crest of the batter below as per the construction drawings (Section 5.3.1).

Plate 59 provides a general view along Ramp G, indicating construction progress to the end of 1997. Slope B appears to be essentially complete whereas Slope A is nearing completion and the natural spur line in between is being profiled.

5.4 Post-Failure Investigation

ESA, engaged by TKCJV (Section 4.3), reported on their findings on the slope failures in August 1998 (ESA, 1998b). Ground investigation conducted by ESA comprised six drillholes (two on Slope A and four on Slope B) extending to 5 m below rockhead (Grade III or better rock), twelve trial pits (five on Slope A and seven on Slope B) to a maximum depth of about 4 m. Piezometers and standpipes were installed in all drillholes for groundwater monitoring. The locations of investigation stations are indicated on Figure 8.

Standard penetration tests (SPT's) were carried out and undisturbed samples were recovered from drillholes and trial pits. Insitu density testing (sand replacement and nuclear densometer) was carried out in the trial pits. These test results have not been made available to FSW.

The details of the trial pitting and associated logs are given in ESA (1998a). Drillhole logs for the remainder of the ground investigation and initial groundwater monitoring are given in Fong On (1998).

Laboratory testing on recovered samples included moisture content and density determinations, direct shear box tests and triaxial tests (Geotechnics and Concrete Engineering ,1998; MateriaLab, 1998; Soil and Materials Engineering, 1998).

ESA carried out ground movement monitoring from 3 June 1998 to 6 January 1999 as part of their investigation.

Additional investigation was carried out by BBVHKL following their review of ESA's report, comprising a trial trench extending from the slope crest to the first berm and exposing the rock fill beneath the DG fill (BBVHKL, 1998). Sand replacement insitu density tests (4 Nos.) and laboratory compaction testing were carried out on the DG fill.

5.5 Deduced Conditions

5.5.1 Ground Profile of Slope A

Three geological sections through Slope A are presented in Figures 13 to 15, which indicate the various strata encountered and the postulated interface locations. Construction records for the Tuen Mun Road and the TKB&AV Contract have been superimposed on the geological sections and they show reasonable agreement.

The stratigraphy indicated for the slope, as deduced from available ground investigation data, comprises up to 12 m of fill underlain by an insitu weathering profile of granitic rock, rockhead being encountered up to 10 m below the fill/natural ground interface.

The fill strata comprise an upper zone 1 m to 3 m (typically 1 m) thick, of silty, fine to coarse sand with occasional gravel (landscaping fill and DG fill). This overlies a layer of fine to coarse gravel, cobbles and boulders of slightly decomposed granite (rock fill), up to 8 m thick, which in turn overlies a slightly clayey, silty, fine to coarse sand with occasional gravel-sized fragments of granite (Tuen Mun Road fill). Where present, this layer is up to

6 m thick. The insitu decomposed rock beneath the fill layers generally comprises an upper zone of Grade V to IV with zones of Grade III material. This is generally described as a fine to medium-grained decomposed granite recovered as fine to coarse sand with fine to medium gravel-sized rock fragments. The weathering profile ranges up to 10 m in thickness.

The visual appearance of the landscaping fill in the landslide scars is generally consistent with the description provided in the contractor's method statement of a CDG fill mixed with a 'conditioner' (Section 5.3). The DG fill exposed in the landslide scars has the appearance of a "typical" CDG fill. The relative compaction of the DG fill ranged from 79% to 91% of MDD according to the BBVHKL Report (1998), which also quotes a range of relative compaction of 77% to 101% obtained from in-situ density testing (using nuclear densometer) in one trial pit in Slope A and three trial pits in Slope B by ESA.

The rock fill material is described in drillhole logs as a predominantly gravel and cobble-sized mix of rock fragments. However, BBVHKL noted from their investigation that the rock fill layer did not appear to have been benched and that oversized rock fill material was present at the rock fill/DG fill interface.

The older underlying fill is found to be similar in composition to the more recent DG fill, although the investigation logs and laboratory test results indicate a higher clay/silt content, with the material occasionally described as a sandy silt. From only four available SPT test results in this material, 'N' values are found to generally range from 10 to 18 (average 13) with a high value of 57 in one test. This would tend to suggest a medium dense material.

The decomposed granite beneath the fill has 'N' values generally range from 41 to over 200. These values indicate that the insitu decomposed granite is in a dense to very dense state, with a trend of increasing density with depth. This corresponds with the descriptions of an extremely weak to moderately strong material given on the investigation logs.

Triaxial testing was carried out on three undisturbed samples of insitu CDG recovered from drillholes ABH3, ABH4 and ABH6 (all located on Slope B) during the ground investigation by ESA. A best-fit line of the p' – q plot for the combined test data has yielded shear strength parameters of $c' = 3$ kPa, $\phi' = 38°$. These values are comparable to the parameters assumed for insitu CDG in the design of the Ramp G Slopes ($c' = 5$ kPa, $\phi' = 37°$).

5.5.2 Ground Profile of Slope B

A geological section for Slope B is presented in Figure 12, which indicates the various strata encountered and the postulated interface locations. As with Slope A, construction records have been superimposed on the geological section and they show reasonable agreement.

The stratigraphy is essentially the same as that described above for Slope A, comprising up to about 22.5 m of fill underlain by an insitu profile of decomposed granite, with rockhead encountered at a maximum depth of about 10 m below the fill/natural ground interface.

The depth of fill is generally more variable on Slope B than on Slope A. The old underlying fill (Tuen Mun Road fill) is recorded as up to 14.7 m thick (as opposed to about 6 m for Slope A). The recent filling works are locally much thinner than for Slope A, particularly where the new fill overlies the former spur line between the original fill slopes (slope Nos. 6SE-C/F14 and 6SE-C/F15), formed as part of the Tuen Mun Road construction.

The rock fill material is described on drillhole and trial pit logs as predominantly gravel and cobble-sized rock fragments.

There are more SPT and GCO Probe test results available for the older underlying fill deposit within Slope B than within Slope A. These indicate that the fill was in a more dense state in this area. The SPT 'N' values generally range from 7 to 52, with blow counts of greater than 100 noted in two tests. The average number of blows was 34 (from 21 tests). This indicates a variable density and would tend to suggest a dense insitu state. GCO Probe test results (SWKP, 1981) vary from 3 to 65 blows with an average value of 19 (from 1048 results). These results suggest a loose to dense material, which corresponds with descriptions given in the investigation logs.

The SPT test results for the decomposed rock encountered beneath the fill, generally ranging from 14 to greater than 200 with an average value of greater than 90 (from 19 results), show these deposits to be of variable density.

5.5.3 <u>Groundwater</u>

Groundwater monitoring data from the post-failure investigation, which was collected during July and August 1998, is limited and has been found to be anomalous in a number of instances, e.g. recorded water level apparently below piezometer tip level. Nevertheless, the overall indication given is that the base groundwater table is located between 3 m and 7 m below the fill/ natural ground interface in both slopes. Piezometers installed within the fill (old and new) generally recorded a 'dry' result. However, one installation located at the toe of slope A recorded groundwater levels 2 m above the fill/ natural ground interface, within the old fill. Available groundwater monitoring data from previous ground investigations shows generally reasonable agreement with the recent data. The measured groundwater levels are indicated on geological sections in Figures 12 to 15.

The inferred groundwater levels for each slope show reasonable agreement with design assumptions adopted in the GIR (BCL,1996b) for the new works. This was also the conclusion reached by ESA (1998b).

5.5.4 <u>Fill Slope Construction</u>

In broad terms, the Ramp G slope works appear to have been completed to the details indicated in the construction drawings and generally in accordance with the method statements. However, a detailed review of record photographs suggests a number of points of possible departure from documented procedures, which are given below.

(a) Record photographs (Plates 52 to 56) generally show compaction of the rockfill within the 2 m to 3 m zone at the sloping face of the layer (plant access track width) and an irregular surface of stockpiled material behind. It is unclear whether the rockfill behind the outer zone received proper compaction.

(b) The placement and compaction of the DG fill with minimal overfill (Plates 47 to 56) and subsequent minor trimming to the slope face prior to placement of landscaping fill may have resulted in a thin layer of less well-compacted fill material being present at the face. At certain locations on Slope B (Plate 53), the thickness of this fill material placed appears to be insufficient to permit access by roller compaction plant. Placement and compaction were probably done by an excavator normal to the slope face, with a vibrating plate compactor also used on the sloping face. There is a potential for layering parallel to the slope face and variable compaction by this approach. The range of relative compaction indicated by the post-failure investigations (Section 5.5.1) suggests variable compaction of the DG fill.

(c) The landscaping fill was essentially a 'conditioned' DG fill material based on the construction method statement and post-failure site observations. This fill was placed by tipping material down the planar slope face and apparently compacted by tamping with the use of an excavator bucket (Plates 52 and 54). The potential exists for layering parallel to the slope face, variation in the compaction state and the likelihood of the lowest degree of compaction near the planar interface between the DG fill and landscaping fill.

5.5.5 Road Construction

The Ramp G Pavement construction has been confirmed by the trial pits (Plates 38 and 39) to conform essentially with the construction details indicated on Figure 11. The assembly of pavement courses is about 500 mm in thickness and is underlain by a soil/rock levelling layer about 300 mm thick, placed to make formation, which directly overlies the rock fill material utilised in the slope construction.

5.5.6 Ground Movement Monitoring

An assessment of ground movements that occurred in the Ramp G fill slopes has been made based on the vertical and lateral movement monitoring carried out after the failure (Figure 8). Monitoring data covers the period 3 June 1998 to 6 January 1999.

Each set of monitoring data shows a general trend of both vertical settlement and lateral movement away from Ramp G. The rate of settlement was relatively rapid (5 mm to 10 mm/month) during the first two months of monitoring, stabilising to a lower rate of movement after this time.

Settlements recorded at the crest and first berm of Slope A to the end of July 1998 were in the range of 5 mm to 50 mm (generally less than 15 mm) and 10 mm to 60 mm (generally less than 20 mm) by early January 1999. Settlements recorded along the crest of Slope B over the same period were of the order of 10 mm to 20 mm, increasing to 10 mm to 25 mm by early January 1999. Settlement of the first berm was greater, at 15 mm to 30 mm, increasing to 15 mm to 35 mm with time. The lower berms of each slope and the crib wall at the toe of Slope A experienced settlements of 5 mm or less to the end of July and 10 mm or less by early January. Lateral movements in the range of 0 to 20 mm were typically recorded and followed a trend similar to settlement.

On the basis of the above, it can be seen that approximately 80% of the movements recorded to date occurred prior to the end of the wet season. The settlement rates beyond this time are of the order of 0 to 2 mm/month. The post-wet season settlement rate for the marker showing the greatest vertical displacement (Marker 13 at the middle of the crest line of Slope A) lies toward the upper end of this range.

It is noted that the movement monitoring has comprised surface monitoring only and that the berms on which these markers are located have generally been undermined by the individual landslides and surcharged by accumulated debris. The recorded ground movements should not necessarily be interpreted as an indication of mass movement of the fill body.

The results of the CCTV survey of the 450 mm stormwater drain along the crest of Slope A indicate no obvious signs of movement. Survey of the culvert No. P043 located below Slope B also indicates no major defects in the manhole structure and the culvert extension completed under the TKB&AV Contract.

6. ANALYSIS OF RAINFALL RECORDS

6.1 General

Each incidence of landslides in the newly-formed fill slopes has been recorded either during or not long after specific rainstorm events on 4 May 1998, 24 May 1998 and 9 June 1998 respectively.

As the specific time of each incidence of landslides is not known, only the timing of initial observation of failures can be used as a guide.

The estimated return periods for maximum rolling rainfall for various durations determined from the rainfall analysis are based on historical rainfall data at the Hong Kong Observatory (Lam & Leung, 1994).

The nearest GEO automatic raingauge (No. N10) is located at Emmanuel Primary School on Castle Peak Road at Sham Tseng, about 1.2 km to the west. The raingauge records and transmits rainfall data at 5-minute intervals via a telephone line to the GEO.

6.2 Landslide No. 1 on 4 May 1998

Slope failures were initially observed at around 9 a.m. on 4 May 1998. Daily rainfall for the period 1 April 1998 to 30 June 1998 and hourly rainfall for 48 hours before and 8 hours following the observation are given in Figures 16 and 17 respectively. The daily record shows that rainfall was recorded over the three days prior to the observed failures (2 to 4 May 1998), with the hourly data indicating relatively concentrated peaks from 07:00 hours to 09:00 hours on the day the failures were observed.

Isohyets of rainfall for the 24-hour period preceding the failures are presented in Figure 18.

Table 1 presents the estimated return period for the maximum rolling rainfall for various durations. A return period of less than 2 years is indicated in all cases.

6.3 Landslide No. 2 on 24 May 1998

Slope failures were observed at around 8 p.m. on 24 May 1998. Hourly rainfall for 48 hours before and 8 hours following the observation is presented in Figure 19. The daily record presented in Figure 16 shows that a storm occurred on this day, with the hourly data indicating peaks from 12:00 hours to 14:00 hours.

Isohyets of rainfall for the 24-hour period preceding the landslides are presented in Figure 20.

Table 2 presents the estimated return period for the maximum rolling rainfall for various durations. A return period of less than 2 years is indicated in all cases.

6.4 Landslide No. 3 Between 24 May 1998 and 9 June 1998

The daily rainfall record from Figure 16 indicates that relatively minor rainfall was recorded between 24 May 1998 and 8 June 1998, with a rainstorm event recorded on 9 June 1998 (day of inspection by FSW). Hourly rainfall for 48 hours before the site visit by FSW and 8 hours following is presented in Figure 21. The hourly data indicates peaks from 02:00 hours to 16:00 hours (time of inspection by FSW).

Isohyets of rainfall for the 24-hour period preceding the site visit are presented in Figure 22.

Table 3 presents the estimated return period of the maximum rolling rainfall for various durations. The 24-hour rainfall was the most severe, with a corresponding return period of 5 years.

7. THEORETICAL STABILITY ANALYSIS

Stability analyses have been carried out using the rigorous method of Morgenstern & Price (1965) to assess two possible failure modes for the observed instabilities in the Ramp G fill slopes. These are:

(a) failure in the surficial layers, and

(b) a deeper-seated failure giving rise to the cracking observed in the hard shoulder behind the slope crest.

The method of construction of the fill slopes has been taken into account in assessing the observed failures in the surficial layers of the two slopes.

For the first failure mode, groundwater was modelled as a phreatic surface parallel to the slope face and above the interface between the DG fill and landscaping fill. The results of the analyses are presented in Figure 23. Shear strength parameters included in the analyses are considered to represent a reasonable range of expected values for the nature of fill materials involved, given the method of placement and the likely variable compaction of the fill material. The analyses suggest that a transient elevated water table in the range of about 100 mm to 200 mm above the DG fill is required to initiate failure.

Two sections have been analysed for the deeper-seated failure mode, one relating to each of the fill slopes, and are presented on Figures 24 and 25 respectively.

The analyses comprised the calculation of a minimum factor of safety under what would be considered "normal" design conditions in respect of shear strength parameters and base groundwater table indicated by monitoring data, from which the effects of certain imposed conditions could be assessed. The shear strength parameters for the various soil layers, as assumed in the GIR (BCL, 1996b), and groundwater table, as deduced from available information (Section 5.5.4), were adopted. The calculated minimum factors of safety for potential slip surfaces in Sections A and B (relevant to Slopes A and B respectively) were 1.34 and 1.45.

For sensitivity analyses, a more extreme groundwater condition was then imposed on the cross-sections, corresponding to a phreatic surface located at the base of the Ramp G rock fill (i.e. pre-1995 ground level). The analyses gave minimum factors of safety of 1.22 and 1.12 for Slopes A and B respectively.

Based on this work, the observation may be made that a deeper-seated instability is probably unlikely, even given fairly extreme groundwater conditions.

8. REVIEW OF POST-LANDSLIDE REPORTS BY OTHER PARTIES

8.1 ESA Report

With regard to the shallow failures at the slope face, ESA (1998b) note that the drawing (Figure 9) does not specify any benching of the DG fill prior to placement of the landscaping fill and accordingly there would have been no 'keying' effect. With regard to the cause of failure, they state "...that the surface slumping has occurred because of the relatively low strength of the "soil" fill and the occurrence of a pre-existing line of weakness along the interface of the CDG fill and "soil" fill". This conclusion was based on visual inspection of the failed areas, detailing and the likely properties of the materials involved. ESA did not make reference to the design or construction aspects of the underlying DG fill, in relation to the shallow failures.

With regard to the cracking in the Ramp G pavement, ESA considered the following possible mechanisms:

> "1. Mass movement of the slope due to existing planes of weakness in the underlying CDG triggered by the increased pressure due to the placed fill, the surcharge effect of Tuen Mun Road and the higher rockhead/water levels. None of these factors were fully considered in the original design.
>
> 2. General settlement of the rockfill due to poor construction.
>
> 3. Migration of fines material into the coarse grade rockfill.
>
> 4. Poor workmanship during construction of the road pavement.
>
> 5. Shallow slips associated with the surface slumping. "

ESA discounted mechanisms 3 to 5 on the basis of observation of the shape and orientation of cracking. In view of the difficulties involved in investigating the condition of the rock fill, ESA assessed the possibility of instability in the CDG.

ESA concluded that the assumptions of rockhead and base groundwater levels in the original design were reasonable and that although relict jointing with kaolinite development had been identified in the insitu CDG, the results of laboratory testing indicated that the mass shear strength parameters were not adversely affected. ESA considered that deep-seated shear failure was unlikely, and that settlement of the Ramp G rock fill due to "poor construction during the fill works" was postulated as the most likely cause of the cracking in the road pavement. ESA suggested that the settlement may have been exacerbated by the "..surcharge effect on the slope due to the high number of heavy vehicles using Ramp G following the opening of Ting Kau Bridge" and "..build-up of groundwater or perched water in the embankment during periods of heavy rainfall."

8.2 BBVHKL Report

With regard to the shallow failures at the slope face, BBVHKL (1998) note that "...some of the failures were restricted to the upper layer of topsoil [landscaping fill] sliding along the interface between the topsoil and the CDG [DG fill]...", but that "...some of the failures extend through the CDG layer to expose the underlying rock fill". Their findings were based on the observation of slips extending to the CDG at one location on Slope B, where failure "...occurred as a single slip, and not as the staged failure implied by ESA's report".

BBVHKL's conclusions, based on observations and results of testing on the DG fill from their additional investigation carried out in June and July 1998, are that "...the CDG [DG fill] appears to be in a loose condition and has not been compacted in accordance with the specification. Furthermore, there is no evidence that the surface of the rock fill was benched in accordance with the specification".

BBVHKL did not provide any specific comments on the postulated mechanism of cracking at the slope crest in their report.

9. DIAGNOSIS OF THE PROBABLE CAUSES OF INSTABILITY

9.1 Shallow Slope Failures

The precise timing for the series of failures on the Ramp G fill slopes is not certain. However, given the rainfall preceding the failures and in the absence of major de-stabilising construction activities, the failures were probably triggered by rainfall.

The mode of the failures involved primarily sliding of the upper 0.3 m to 0.4 m of fill material on individual slope batters along a surface approximately parallel to the slope face and located at or near the planar interface between the DG fill and the overlying landscaping fill. Based on site inspections, the presence of the planar interface probably acted as a pre-existing weak plane and played a significant role in the failure.

The gradients of the fill slope batters (33° to 39° were relatively steep considering the nature of the fill material. It is postulated that the failures were caused by transient elevated water pressures near the interface between the landscaping fill and the DG fill. This is supported by the results of stability analyses, which suggest that failure of the landscaping fill will occur given a modest, transient perched water table above the DG fill.

Direct surface infiltration is considered to be the most likely source of water affecting the two vegetated slopes. In addition, the areas opened up for tree planting could have allowed concentrated water ingress into the slope. The detailing of the slope involving termination of the berm aprons before the crest of the batter below (i.e. a horizontal uncovered surface in the landscaping fill) also promoted infiltration (Figure 9 and Plate 58).

Significant overland flow from the road at the slope crest, resulting from overload of the road drainage system and overtopping of the kerb is considered unlikely on the basis of observations made by FSW during the rainstorm event of 9 June 1998. Furthermore, there were no signs of erosion near the slope crest indicative of overland flow from the road.

Leakage from stormwater drainage is unlikely, based on the results of the CCTV survey (Section 4.3).

A transient rise of the base groundwater table resulting in the shallow failures is considered unlikely given that there is no permanent high groundwater table in the slopes.

Other factors that may also have contributed to the failures are given below.

(a) the method of placement of the landscaping fill, which was observed from record photographs to have been tipped down the slope face, and the subsequent compaction of this material normal to the slope face could have resulted in segregation and layering parallel to the slope face,

(b) the landscaping fill is relatively thick (300 mm to 500 mm) such that the root growth had probably not penetrated into the underlying DG fill to tie the landscaping fill onto the slope at the time of the rainstorm events, and

(c) the nominal thickness of the DG fill in certain areas has probably prevented access by roller compaction plant. Where compaction was carried out normal to the slope face for such areas, variable compaction of this material and layering parallel to the slope face may result. In addition, it was noted from site record photographs that some of the DG fill was compacted with little overfilling, potentially resulting in a less dense layer close to the interface with the landscaping fill.

9.2 Ground Movements and Distress

The ground movements recorded in the slope have comprised a downward and outward movement at slope crests, on the slope faces, and along the crest of the crib wall at the toe of Slope A.

The possible causes of the slope movement and cracking of the road pavement may include the following:

(a) deep-seated failure through the old fill or in-situ CDG,

(b) settlement of the Ramp G rock fill,

(c) loss of fines from the overlying fill into the rock fill due to lack of a "separator" layer or geotextile,

(d) compression of the 1975 Tuen Mun Road fill due to surcharge from the Ramp G fill, and

(e) reduction of lateral support at the slope crest due to the near-surface instability (Section 9.1).

Based on currently available information (e.g. disposition of the cracks at crest, lack of bulging at toe and the results of CCTV survey which do not indicate any major movement of the buried drains) and the supporting theoretical analyses, deep-seated failure within the old fill body and the natural ground below would appear to be unlikely.

Regarding the other postulated causes, in the absence of information other than surface movement monitoring, it is not possible to positively ascertain the primary causes of the cracking in the road pavement.

It should be noted that the magnitude of ground movement measured from June 1998 to January 1999 is typically of the order of 10 mm to 20 mm, with a maximum of 50 mm vertically and 20 mm horizontally. This is relatively small when compared to the likely average particle size of the rock fill (i.e. about 200 mm) and the total layer thickness of Ramp G and Tuen Mun Road fill (some 10 m to 20 m).

10. CONCLUSIONS

It is concluded that the landslides were probably triggered by moderate to heavy rainfall (return periods of about 2 to 5 years). The slopes failed during their first wet season following construction and during the early part of the maintenance period of a Design and Build contract.

The landslides were extensive and affected about 40% of the face area of the slopes. The volume of failure was estimated to be about 1300 m^3. The mode of failure was principally shallow slides along a surface approximately parallel to the slope face and located at or near the planar interface between the landscaping fill and the underlying DG fill. Distress in the form of cracking has also been observed in the road pavement and soft verge behind the slope crest.

The failures were probably caused by the development of transient elevated water pressure in the near-surface fill following direct infiltration. Deficiency in detailing of the fill slope (i.e. the presence of a planar interface between the DG fill and the overlying landscaping fill), together with the method of placement of fill actually adopted which was liable to result in some of the near-surface fill being less well compacted, contributed to the near-surface failures. The principal failure mode that occurred apparently was not considered in the design submissions.

The cause of ground movements resulting in the observed cracking at the crest of the slopes cannot be established from the available information, although it appears unlikely to be associated with the progressive development of a deeper-seated failure mechanism.

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LIST OF TABLES

Table 1 - Maximum Rolling Rainfall at GEO Raingauge No. N10 for Selected Durations Preceding the Landslide of 4 May 1998 and the Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 Minutes	8.5	08:10 on 4 May 1998	< 2
15 Minutes	19.0	08:10 on 4 May 1998	< 2
30 Minutes	25.0	08:20 on 4 May 1998	< 2
1 Hour	38.5	08:25 on 4 May 1998	< 2
2 Hours	43.5	09:05 on 4 May 1998	< 2
4 Hours	43.5	09:05 on 4 May 1998	< 2
8 Hours	44.0	12:40 on 4 May 1998	< 2
12 Hours	44.0	12:40 on 4 May 1998	< 2
24 Hours	50.5	10:55 on 4 May 1998	< 2
48 Hours	134.0	13:05 on 4 May 1998	< 2
4 Days	153.5	09:05 on 4 May 1998	< 2
7 Days	194.0	09:05 on 4 May 1998	< 2
15 Days	309.0	09:05 on 4 May 1998	< 2
31 Days	344.5	09:05 on 4 May 1998	< 2

Notes :

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

2 Maximum rolling rainfall was calculated from 5-minute data for durations up to 48 hours, and from hourly rainfall 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 & Leung (1994)'s data, which are based on hourly rainfall for these durations.

Table 2 - Maximum Rolling Rainfall at GEO Raingauge No. N10 for Selected Durations Preceding the Landslide of 24 May 1998 and the Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 Minutes	7.0	12:20 on 24 May 1998	< 2
15 Minutes	16.5	12:20 on 24 May 1998	< 2
30 Minutes	26.0	12:35 on 24 May 1998	< 2
1 Hour	40.0	13:05 on 24 May 1998	< 2
2 Hours	68.0	14:00 on 24 May 1998	< 2
4 Hours	77.5	14:00 on 24 May 1998	< 2
8 Hours	116.0	14:30 on 24 May 1998	< 2
12 Hours	119.0	16:05 on 24 May 1998	< 2
24 Hours	119.0	16:05 on 24 May 1998	< 2
48 Hours	119.0	16:05 on 24 May 1998	< 2
4 Days	151.5	16:05 on 24 May 1998	< 2
7 Days	160.0	16:05 on 24 May 1998	< 2
15 Days	196.0	16:05 on 24 May 1998	< 2
31 Days	474.0	16:05 on 24 May 1998	< 2

Notes :

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

2 Maximum rolling rainfall was calculated from 5-minute data for durations up to 48 hours, and from hourly rainfall 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 & Leung (1994)'s data, which are based on hourly rainfall for these durations.

Table 3 - Maximum Rolling Rainfall at GEO Raingauge No. N10 for Selected Durations Preceding the Landslide of 9 June 1998 and the Estimated Return Periods

Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 Minutes	8.5	10:40 on 9 June 1998	< 2
15 Minutes	21.0	10:40 on 9 June 1998	< 2
30 Minutes	34.5	10:55 on 9 June 1998	< 2
1 Hour	47.0	11:25 on 9 June 1998	< 2
2 Hours	70.5	04:25 on 9 June 1998	< 2
4 Hours	122.0	06:10 on 9 June 1998	2
8 Hours	150.0	10:50 on 9 June 1998	2
12 Hours	206.0	14:25 on 9 June 1998	4
24 Hours	305.0	18:20 on 9 June 1998	5
48 Hours	321.0	16:55 on 9 June 1998	4
4 Days	381.0	16:55 on 9 June 1998	3
7 Days	462.5	16:55 on 9 June 1998	3
15 Days	488.5	16:55 on 9 June 1998	< 2
31 Days	605.0	16:55 on 9 June 1998	< 2

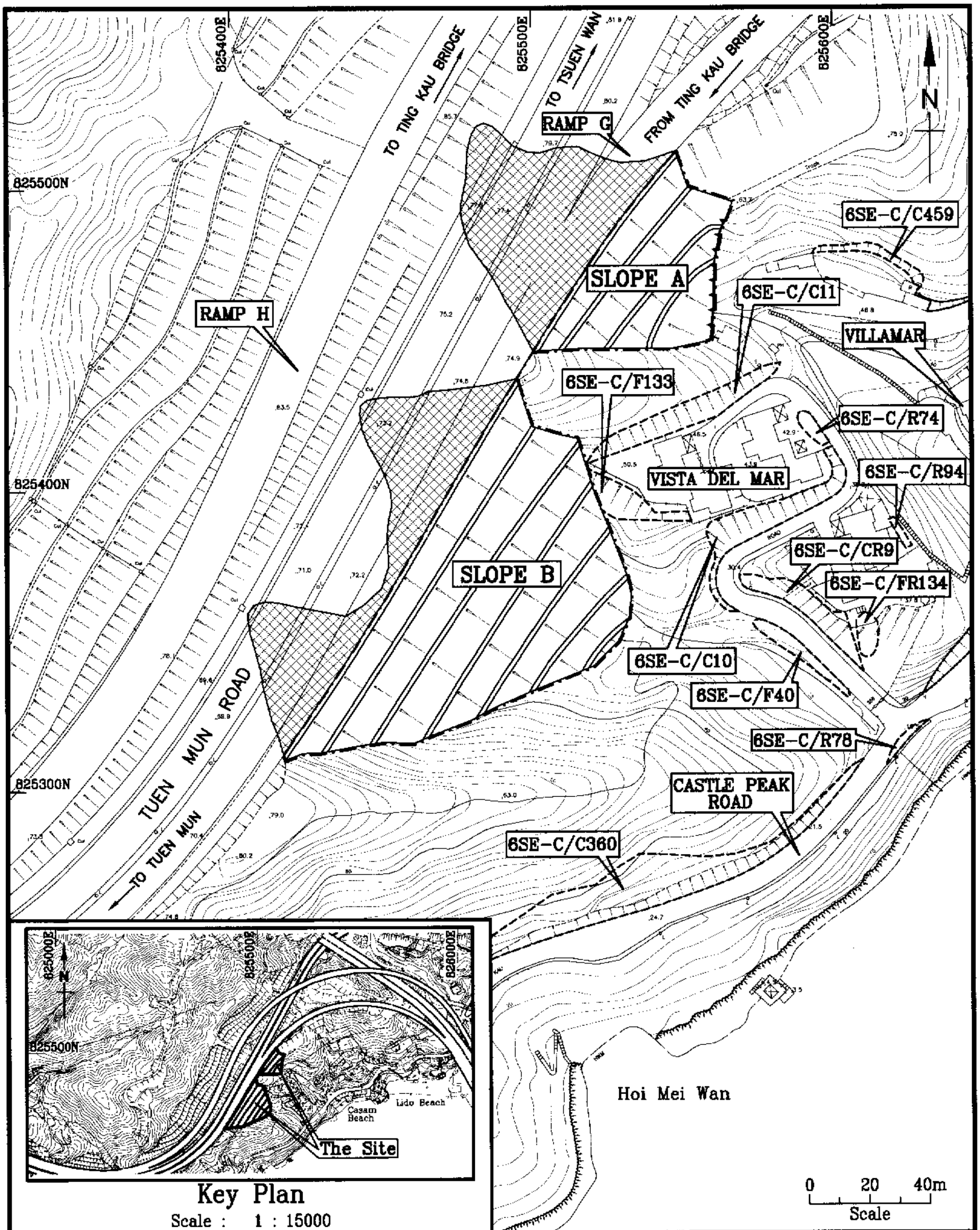
Notes :

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

2 Maximum rolling rainfall was calculated from 5-minute data for durations up to 48 hours, and from hourly rainfall 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 & Leung (1994)'s data, which are based on hourly rainfall for these durations.

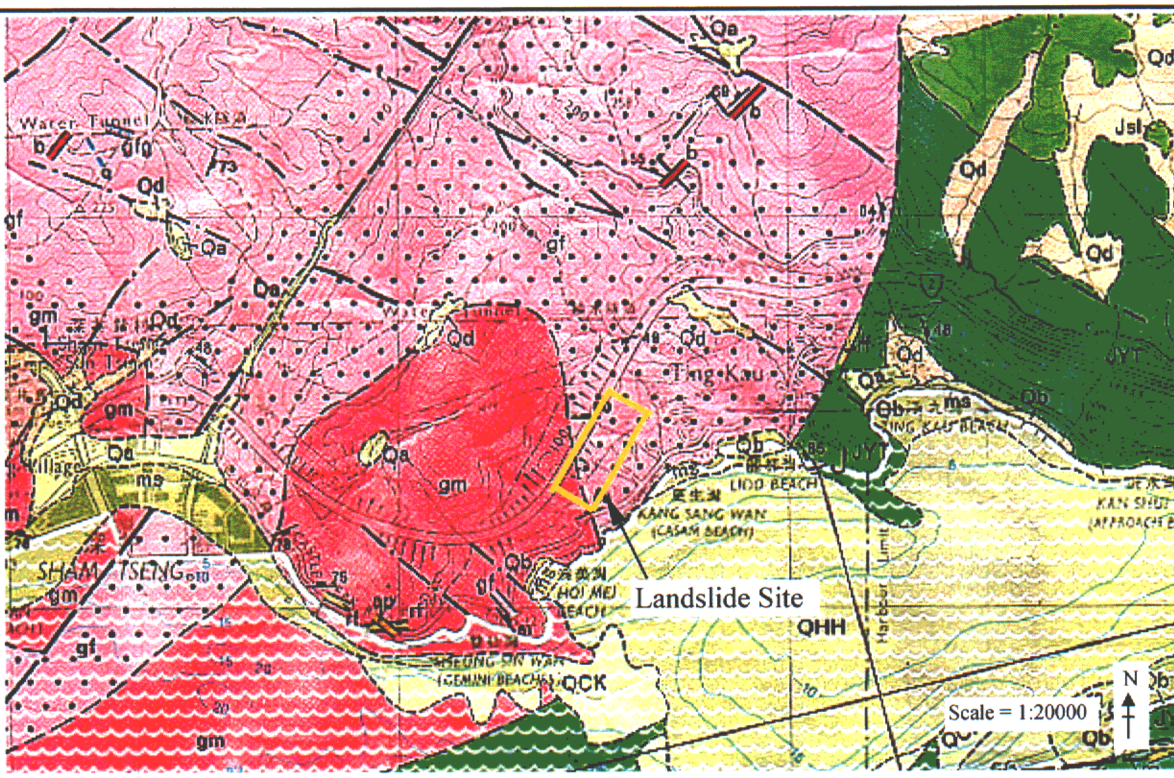
LIST OF FIGURES



Legend: 6SE-C/C11 Registered Feature Number; Crib Wall; Study Feature; Approximate Extent of Fill Beneath Tuen Mun Road and Ramp G; Feature Boundary

Notes:
1. Base Map is extracted from 1 : 1000 scale Survey Sheet Nos. 6SE-17D and 6SE-18C.
2. Feature boundaries and registration numbers taken from sheets 6SE-17D and 6SE-18C of the GEO SIRST study (GEO, 1996).

Figure 1 - Site Location Plan



Legend:

Quaternary Superficial Deposits (Onshore)

- Qa — Alluvium – Clay/silt, sand and gravel; well-sorted to semi-sorted
- Qb — Beach Deposits – Sand
- Qd — Debris Flow Deposits – Unsorted sand, gravel, cobbles & boulders; clay/silt matrix

Quaternary Superficial Deposits (Offshore)

- QHH — Undivided, mainly dark grey marine mud
- ms — Marine sand, part silty
- QCK — Undivided, red yellow and grey clay, silt, sand and gravel

Solid Geology

- JYT — Coarse ash crystal tuff
- Jsl — Tuff-breccia
- gf — **Fine-grained granite** } **Megacrystic**
- gm — **Medium-grained granite** }
- b — Basalt
- ap — Aplite

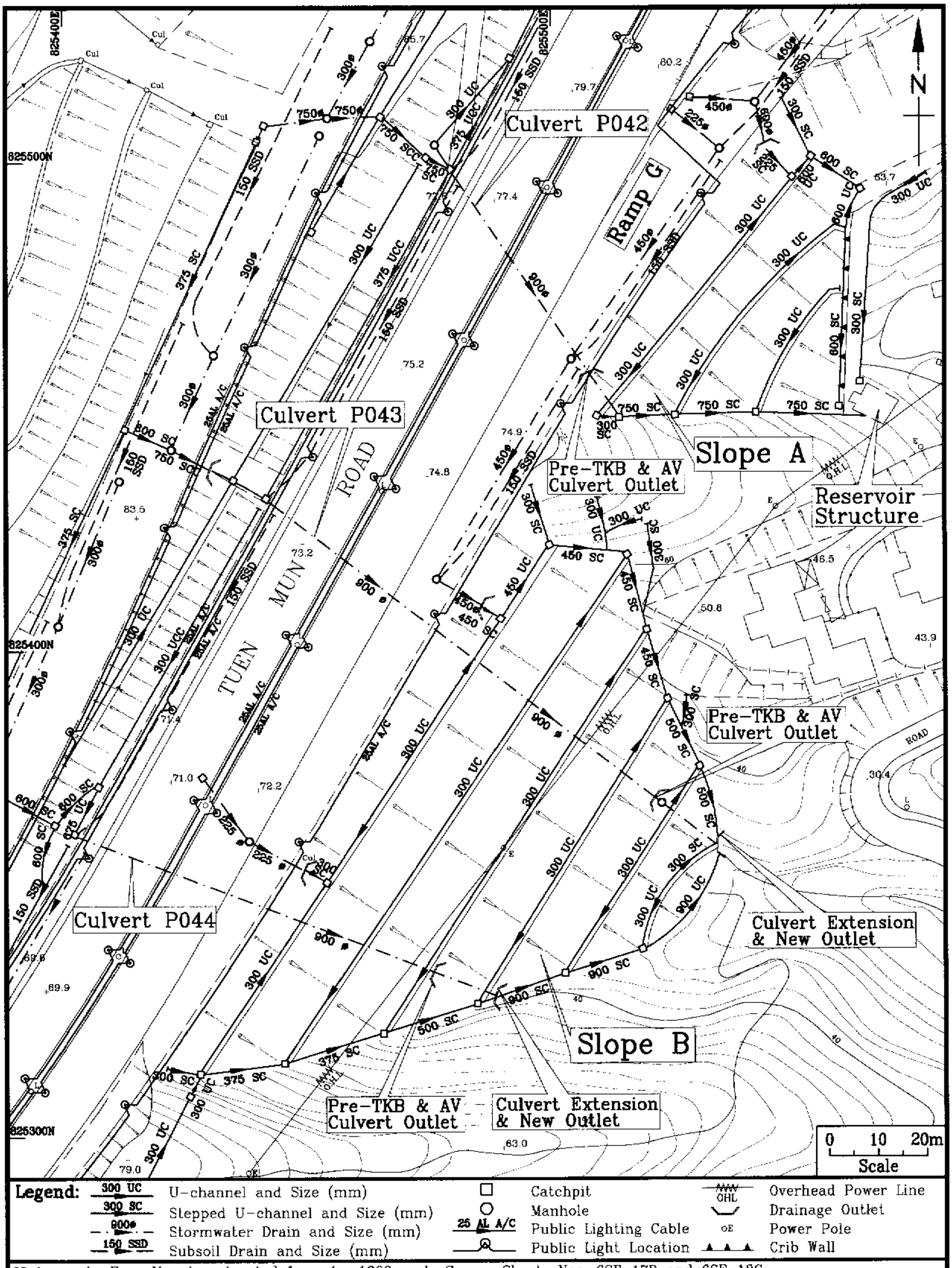
Geological Lines

- Geological Boundary, Superficial Deposits
- Geological Boundary, Solid Rock *
- Faults (crossmark indicates downthrow side) *
- Photogeological Lineament

* Broken lines on map face denote uncertainty

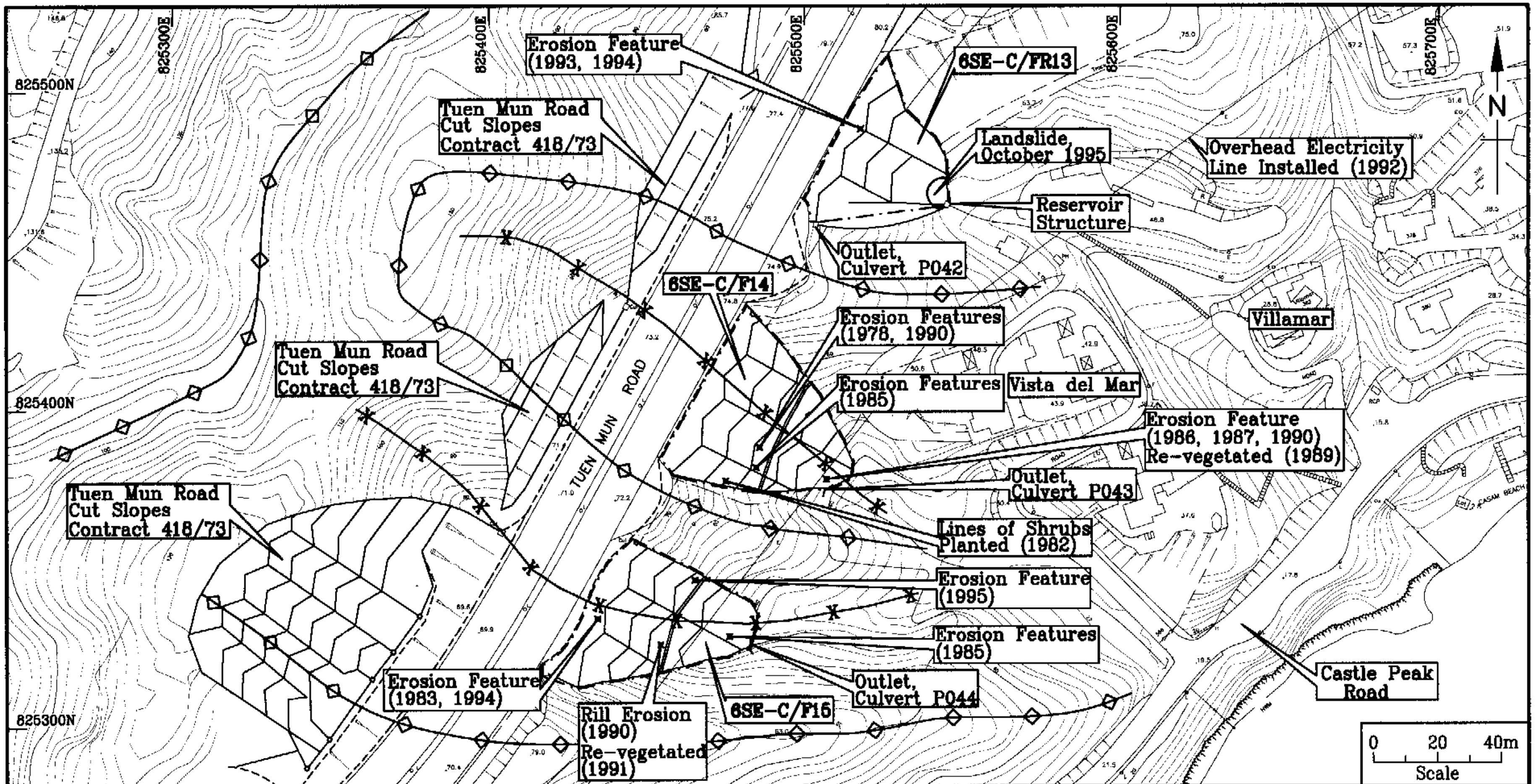
Notes: 1. Geology relevant to study area shown bold.
2. Extract from Sheet 6 HGM20 (Scale 1:20000) by the Hong Kong Geological Survey.

Figure 2 - Solid and Superficial Geology of the Landslide Site



Notes:
1. Base Map is extracted from 1 : 1000 scale Survey Sheets Nos. 6SE-17D and 6SE-18C.
2. Drainage details of the area are based on Binnie Consultant Limited's Drawing Nos. TKB/GEN/BC/1101 and TKB/GEN/BC/1102 of Contract No. HY/93/38 "Ting Kau Bridge and Approach Viaduct".
3. Drainage details of Slope A and Slope B are based on Highways Department's Drawing No. E05598/WD/001 of Contract No. HY/93/38 "Ting Kau Bridge and Approach Viaduct".
4. Location of power cables and overhead lines are based on CLP Utility Plans.

Figure 3 - Existing Services and Utilities

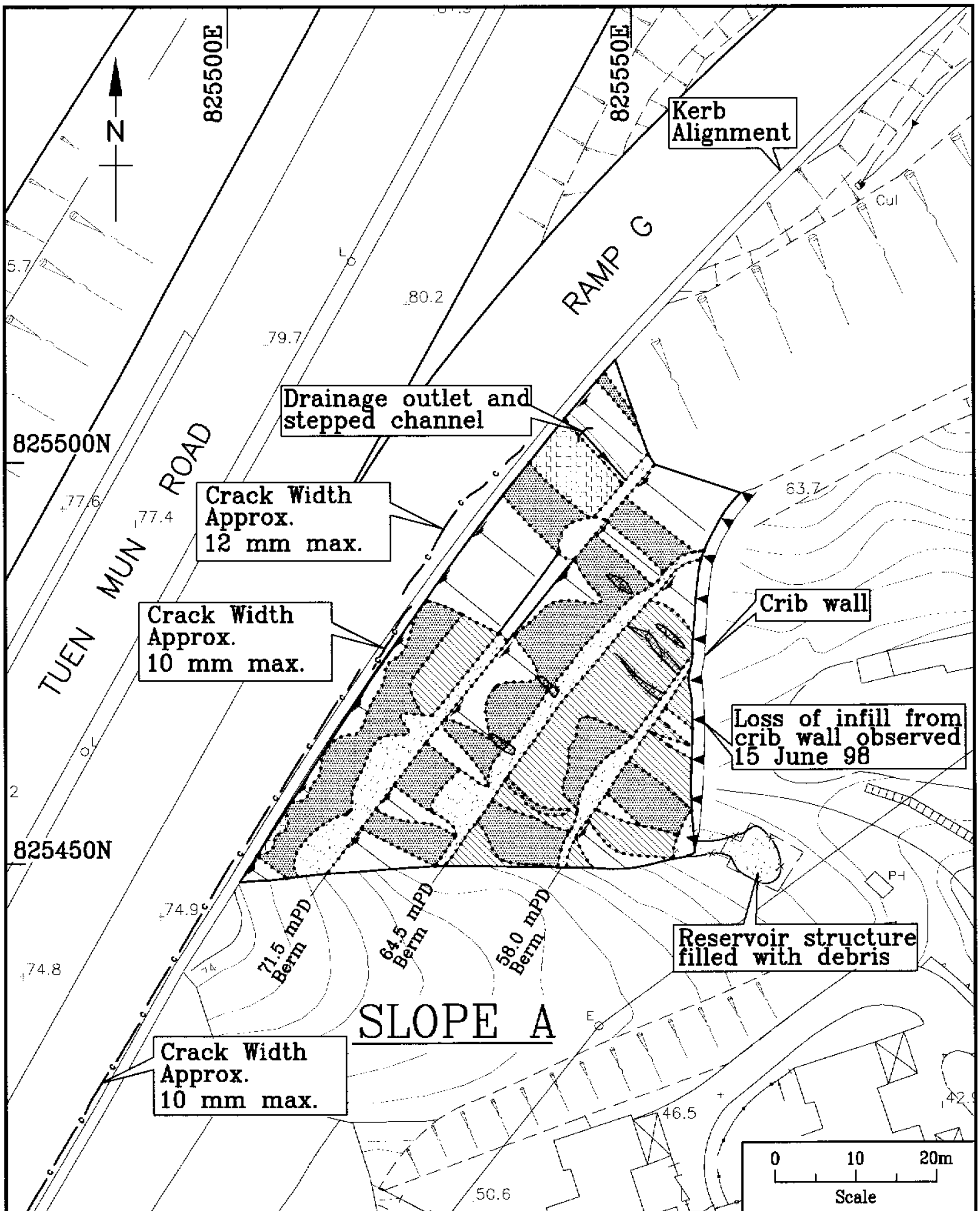


Legend:

6SE-C/F15	Registered Feature Number
—·—	Feature Boundary
(Chevron symbol)	Chevron Drains (Partially destroyed by erosion (1976), Replaced (1977))
80	Ground Surface Contour (mPD)
◇—◇	Ridge Crest / Spur Line
X—X	Valley Axis

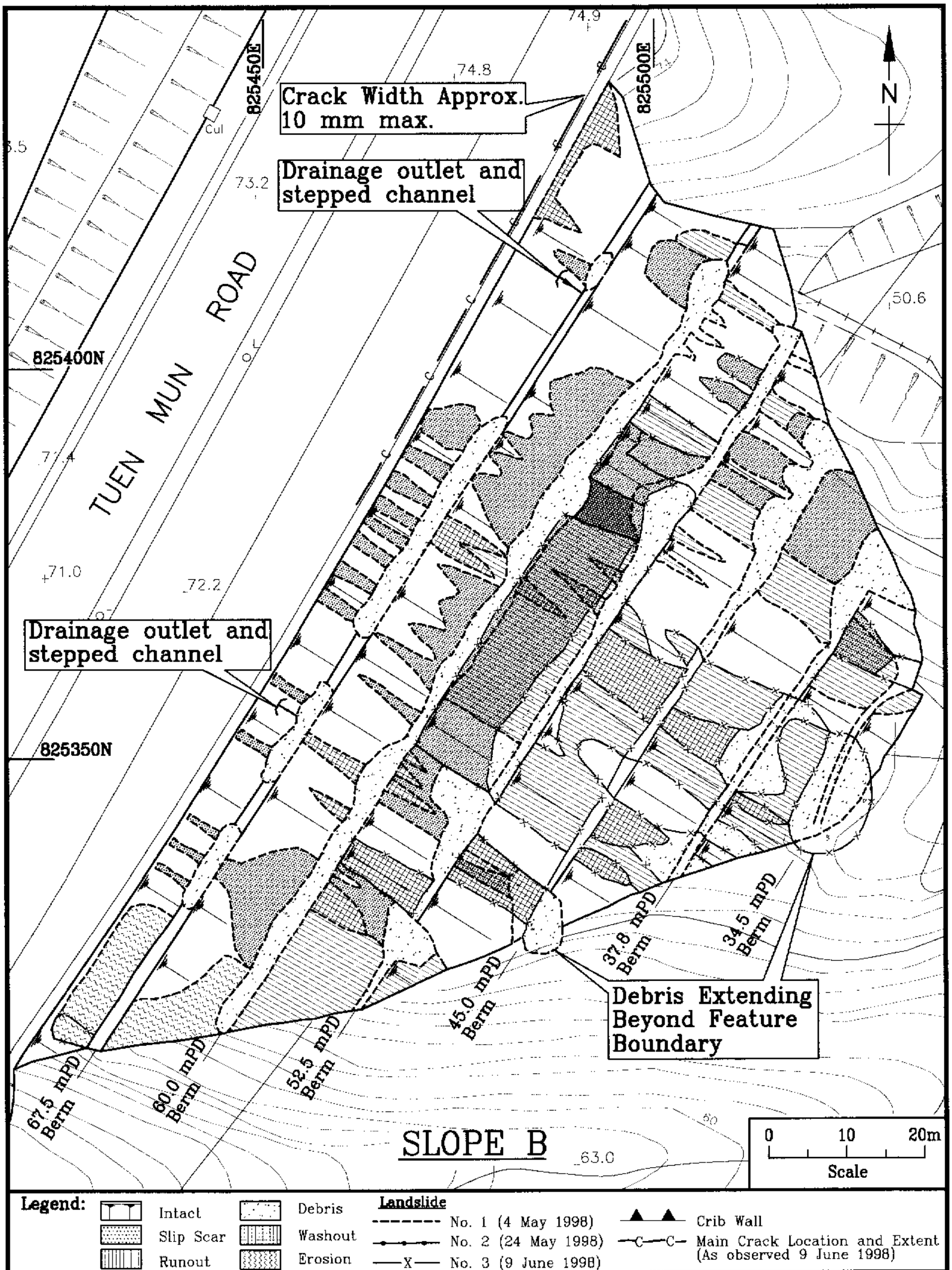
Notes:
1. Base Map is extracted from 1 : 1000 scale Survey Sheet Nos. 6SE-17D and 6SE-18C.
2. Pre-1995 slopes and contours are based on Drg. Nos. TKC/TKB/SVR/001 & 002 of "Ting Kau Bridge and Approach Viaduct" Contract.

Figure 4 - Pre-1995 Slope Formation



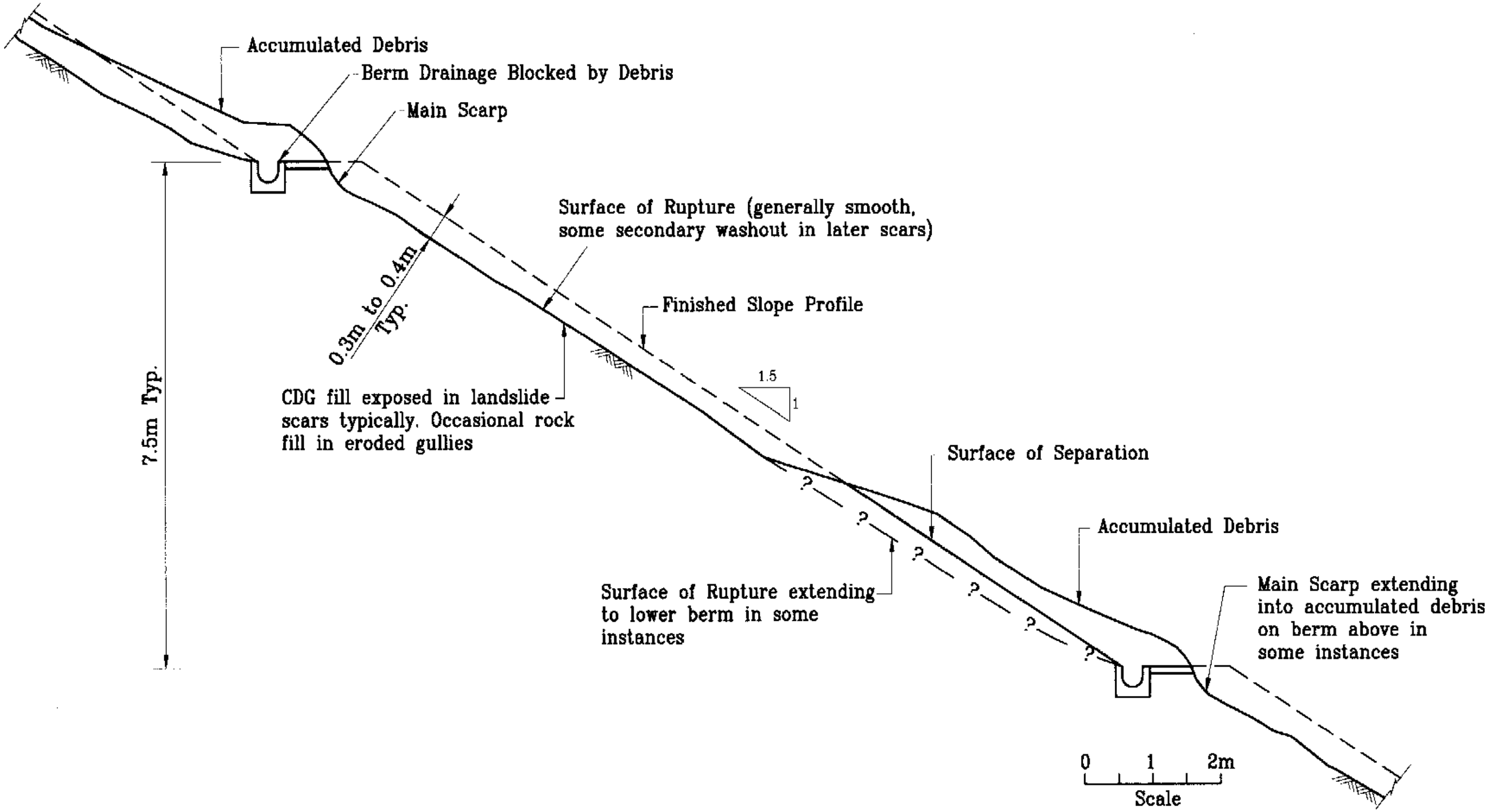
Notes:
1. Base Map is extracted from 1 : 1000 scale Survey Sheet Nos. 6SE-17D and 6SE-18C.
2. Extent of crack at crest indicated is extent as of 15 June 1998.
3. Majority of scars for Landslide No. 1 (4 May 1998) covered with plastic sheeting prior to Landslide No. 3 (9 June 1998).

Figure 5 - Completed Formation (Slope A) Showing Observations after Failure



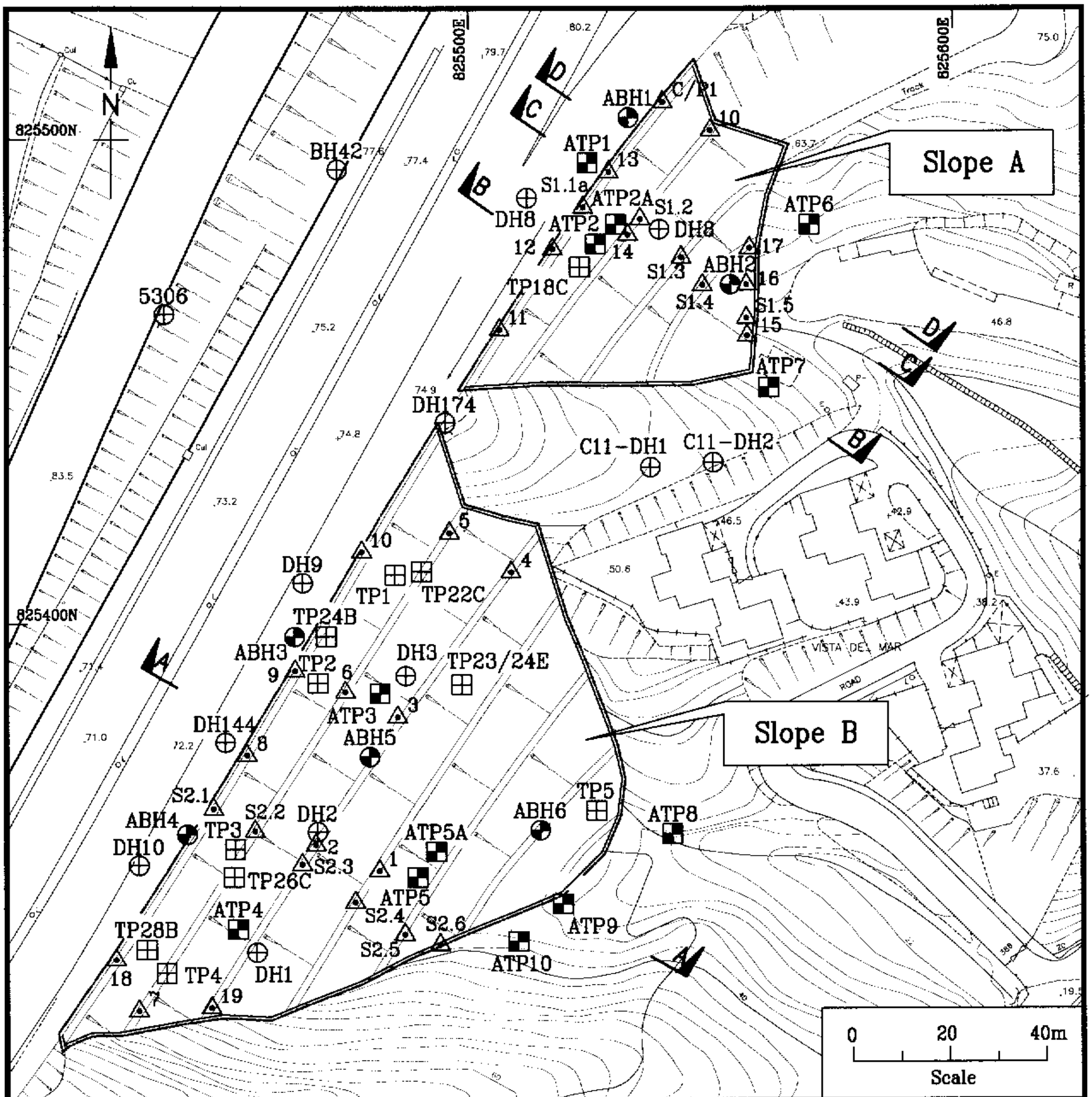
Notes:
1. Base Map is extracted from 1 : 1000 scale Survey Sheet Nos. 6SE-17D and 6SE-18C.
2. Extent of crack at crest indicated is extent as of 15 June 1998.
3. Majority of scars for Landslide No. 1 (4 May 1998) covered with plastic sheeting prior to Landslide No. 3 (9 June 1998).

Figure 6 - Completed Formation (Slope B) Showing Observations after Failure



Notes: 1. Slope geometry indicated is typical for Slopes A and B. Individual batter gradients may vary.
2. Refer Plates 5 to 37 for actual appearance of individual scars.

Figure 7 - Typical Section through Landslides



Legend:

Feature Boundary

Geological Section Location (Refer Figures 12 to 15)

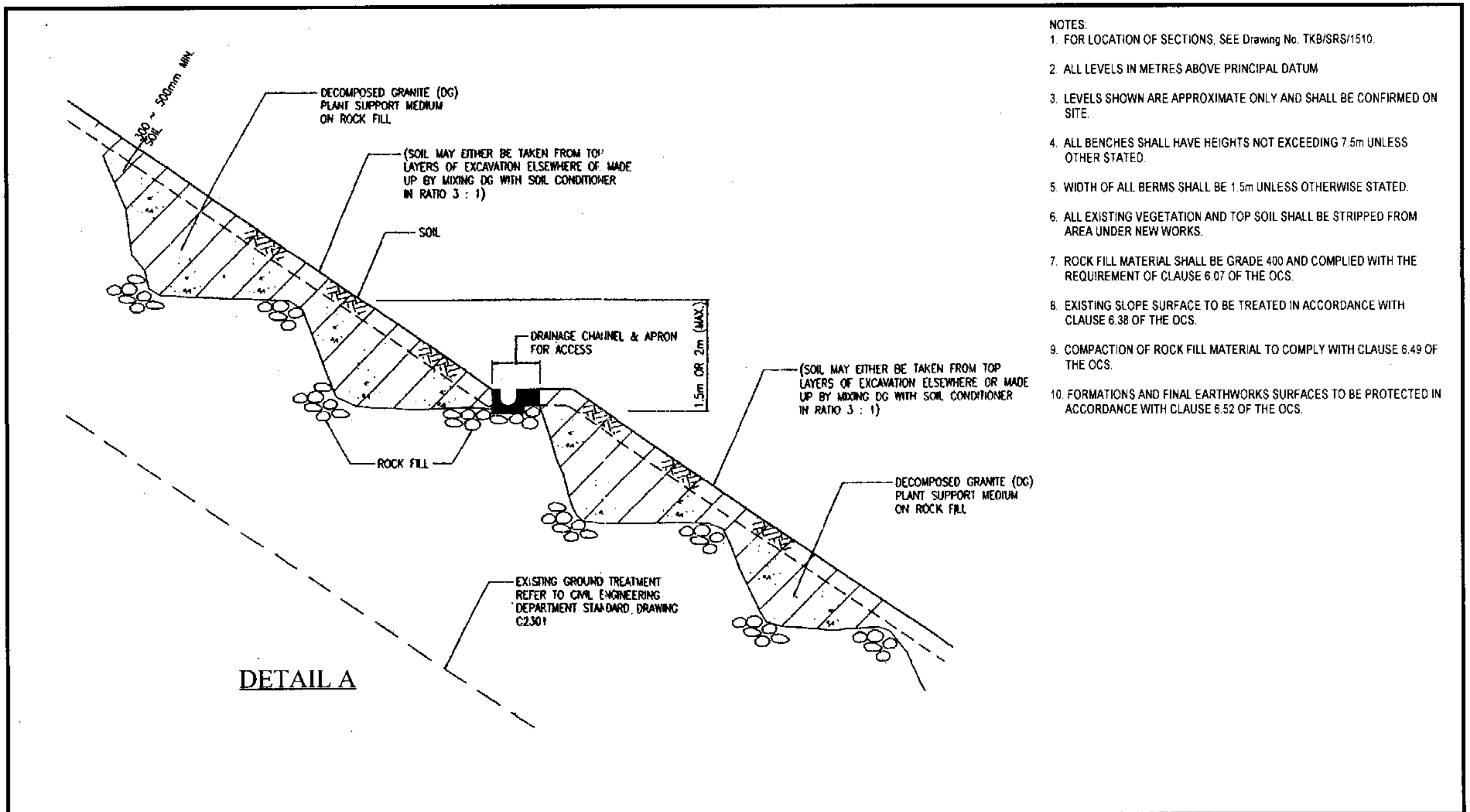
ABH4 Drillhole; ATP8 Trial pit — Locations and reference numbers of post-failure GI stations. (Fong On, 1998) and (ESA, 1998)

10, S2.4 — Locations and reference numbers of settlement monitoring stations. (ESA 1998)

DH10 Drillhole; TP28B Trial pit — Locations of GI station's completed prior to Ramp G slope works.
TP18C, -/22C, -/23/24E, -/24B, -/26C, -/28B (SWKP,1981)
C11-DH1 & DH2 (Lam Geotechnics,1987)
5306 (Bachy Soletanche, 1987)
BH42 (Intrusion Prepakt, 1993)
DH8-10, TP1-5 (Geotechnics and Concrete Engineering, 1993)
DH144 & -174 (Enpack, 1994)
DH1-4 (Enpack, 1995)
DH8 (Binnie, 1995)

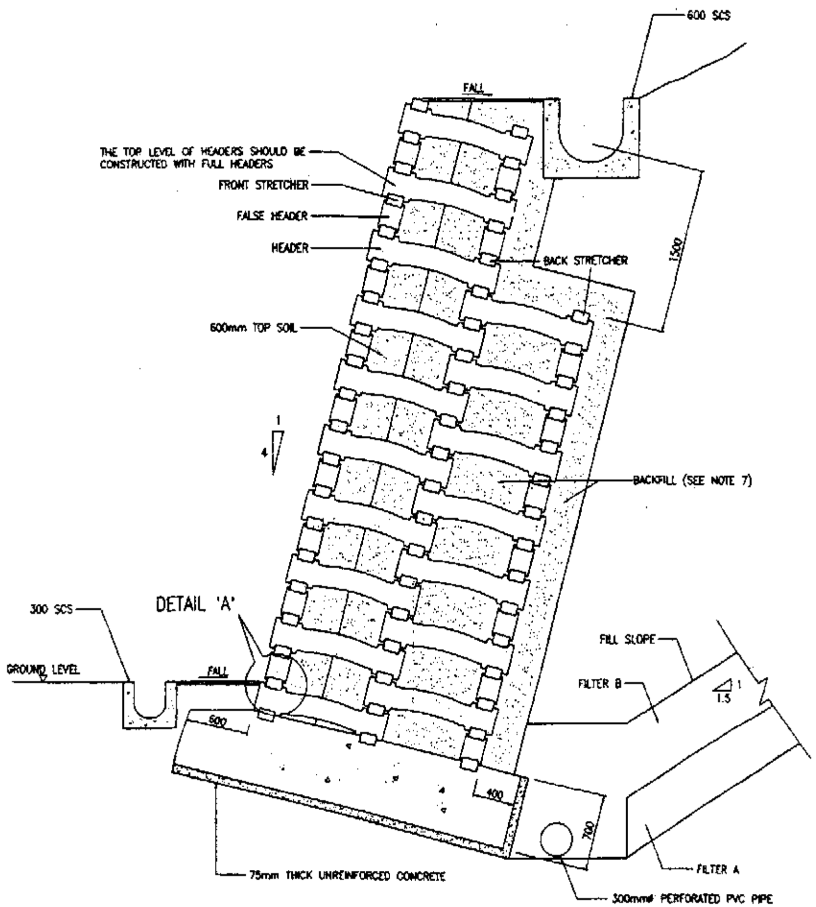
Notes: 1. Base Map is extracted from 1 : 1000 scale Survey Sheet Nos. 6SE-17D and 6SE-18C.
2. Section locations based on Working Drawing TKB/SRS/BC/1510 from the TKB & AV Contract (BCL 1996).

Figure 8 - Investigation Stations



Notes : 1. Detail A and Notes extracted from Working Drawing No. TKB/SRS/BC/1511 of Highways Department Contract No. HY/93/38 Ting Kau Bridge and Approach Viaduct, Ting Kau Contractors Joint Venture.
2. The "OCS" referred to in notes on the drawing relates to the HyD "Outline Construction Specification".

Figure 9 - Typical Detail – Ramp G Slope Construction



NOTES:

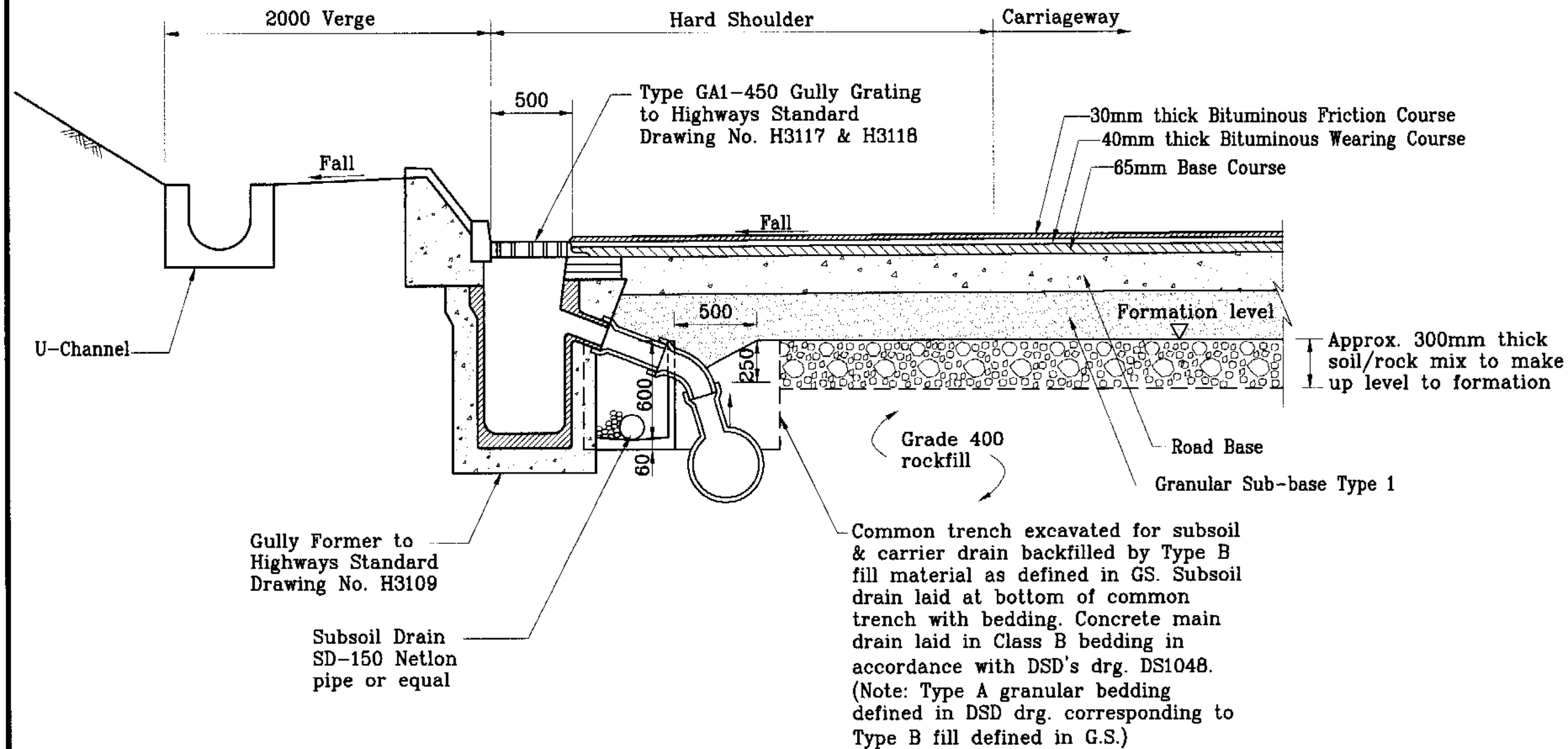
1. ALL DIMENSION ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
2. CRIB WALL UNITS SHALL BE DORAN CRIB WALL OR SIMILAR TO SUPERVISING OFFICER'S APPROVAL. SEE "DORAN I.C.B. CRIB WALLING" CATALOGUE FOR COMPONENT DIMENSIONS & DETAILS.
3. NO PRECAST CONCRETE CRIB WALLIONG UNITS SHALL BE ERECTED UNTIL THE CONCRETE OF THE FOUNDATION HAS REACHED 7 DAYS PREMATURE STRENGTH AND STABLISING BERM HAS FILLED UP TO THE TOP OF FOUNDATION.
4. ALL CONCRETE TO CRIB WALL FOUNDATION SHALL BE 30D/20.
5. LOCATION AND DIMENSION OF THE CRIB WALL FOUNDATION ARE SUBJECT TO CHANGE TO SUIT THE CONTRACTOR'S PROPOSED CRIB WALLING UNIT AND TO ALLOW FOR CONSTRAINTS IMPOSED BY ADJACENT STRUCTURES OR OTHER WORKS. FINAL LOCATION AND DIMENSIONS OF THE CRIB WALL FOUNDATION SHALL BE DETERNMINED ON SITE BY THE ENGINEER. FOUNDATION MATERIAL SHALL BE DENSE CDG OR BETTER WHICH REQUIRES THE USE OF A PICK FOR EXCAVATION, ASSUMED GROUND BEARING CAPACITY FOR THE CRIB WALL FOOTING IS TAKEN AS 300 kPa.
6. EXPOSED SURFACE OF CONCRETE BASE SHALL BE CLASS F3 FINISH. UNEXPOSED SURFACE SHALL BE CLASS F2 FINISHED.
7. BACKFILL MATERIAL FOR THE CRIB WALL SHALL BE IN ACCORDANCE WITH CLAUSE NO 9.4.3.(3) OF GEOGUIDE 1 "GUIDE TO RETAINING WALL DESIGN"..
8. THE BACKFILL SHALL BE EXTEND BEHIND THE WALL FOR AT LEAST 300mm AND SHALL BE SPREAD EVENLY IN HORIZONTAL LAYERS NOT EXCEEDING 300mm LOOSE DEPTH EACH 300MM LAYER OF MATERIAL SHALL BE THOROUGHLY COMPACTED WITHIN AND BEHIND THE WALL USING HAND-OPERATED COMPACTION MACHINARY, TO THE SATISFACTION OF THE ENGINEER.
9. THE OUTER 600mm INFILL MATERIAL SHALL BE TOP SOIL.
10. THE AREA WHICH DO NOT COMPLY WILL BE REMOVED AND REINSTATED WITH FOUNDING MATERIAL DENOTED IN NOTE 5.

DOUBLE CELL CRIB WALLING

(For wall height equal to or exceed 3m)

Note : Details and Notes extracted from Working Drawing No. TKB/SRS/BC/1512 of Highways Department Contract No. HY/93/38 Ting Kau Bridge and Approach Viaduct, Ting Kau Contractors Joint Venture.

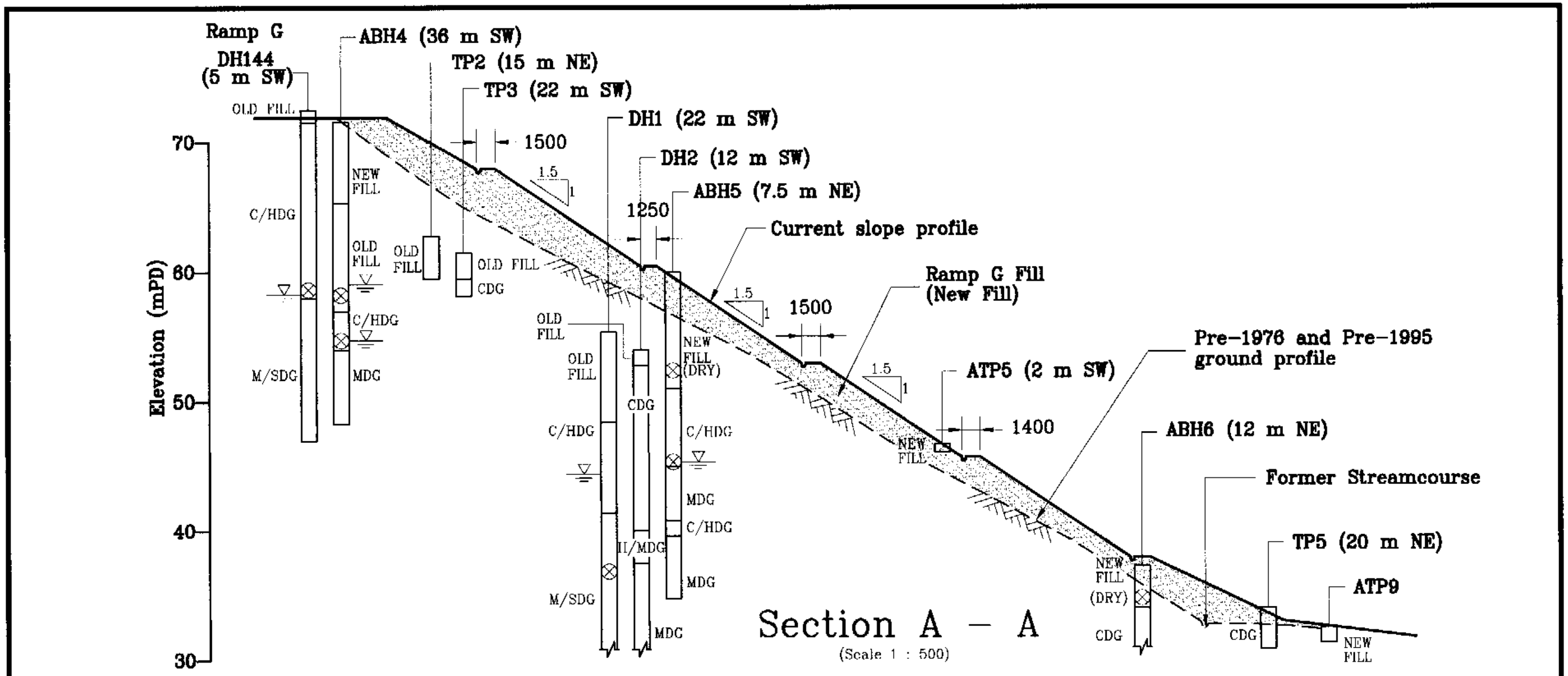
Figure 10 – Cribwall Construction



TYPICAL SECTION ON AT-GRADE RAMPS & APPROACH VIADUCT AT TING KAU
(WITH HARD SHOULDER)

Notes: 1. Information shown on this figure is based on Highways Department Contract No. HY/93/38 Construction Drawing.
2. Typical section is shown for "toe of slope" condition. Actual section is at crest of slope. Drainage channel not included at crest.

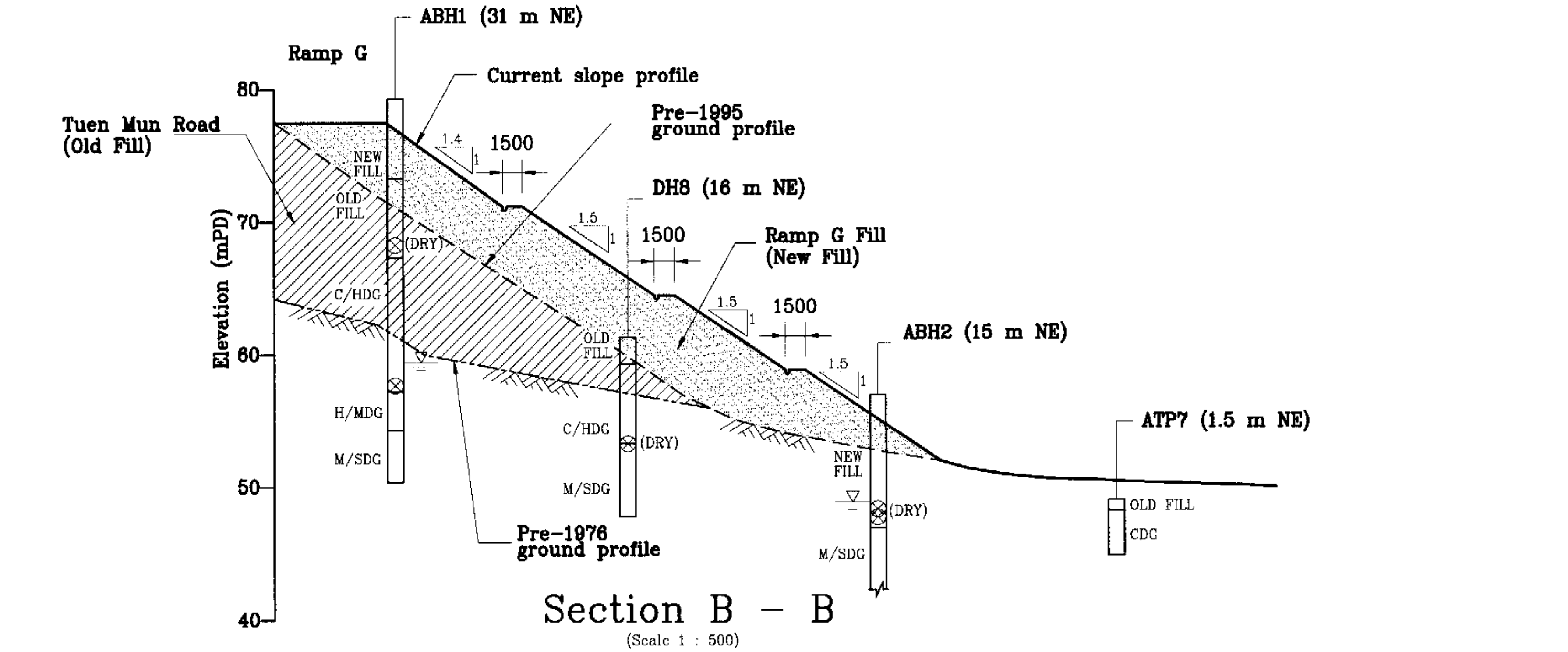
Figure 11 - Ramp G Pavement Construction



Legend : ABH1 (31 m NE)	Drillhole ABH1 offset 31m NORTH EAST		Observed groundwater level (maximum) Dry where noted	OLD FILL	Tuen Mun Road Formation	CDG	Completely Decomposed Granite (Grade V)
						HDG	Highly Decomposed Granite (Grade IV)
				NEW FILL	TKB & AV Ramp G Formation	MDG	Moderately Decomposed Granite (Grade III)
⊗	Piezometer Tip	COLL	Colluvium			SDG	Slightly Decomposed Granite (Grade II)

Notes:
1. See Figure 8 for location of section.
2. The current slope profile is based on the final survey of the Ramp G Fill Slope between chainages 810 and 1004 by Highways Department Survey Section (TKB) (1998). Drg. ref No. E05598/WD/001.
3. The Pre-1976 and Pre-1995 fill slope construction details are based on Scott Wilson Kirkpatrick & Partners Tuen Mun Road Stage I Construction Details (1979) Drg. No. 76811/G29, 76811/1052R and 71805/208R and on Binnie Consultants Ltd. Ting Kau Bridge and Approach Viaduct Contract Drg. No. TKB/SRS/BC/1510.
4. The Pre-1995 ground profile is based upon initial survey for Ting Kau Bridge and Approach Viaduct Contract by Highways Department Survey Section (TKB) (1995). Drg. No. TKC/TKB/SVR/002.

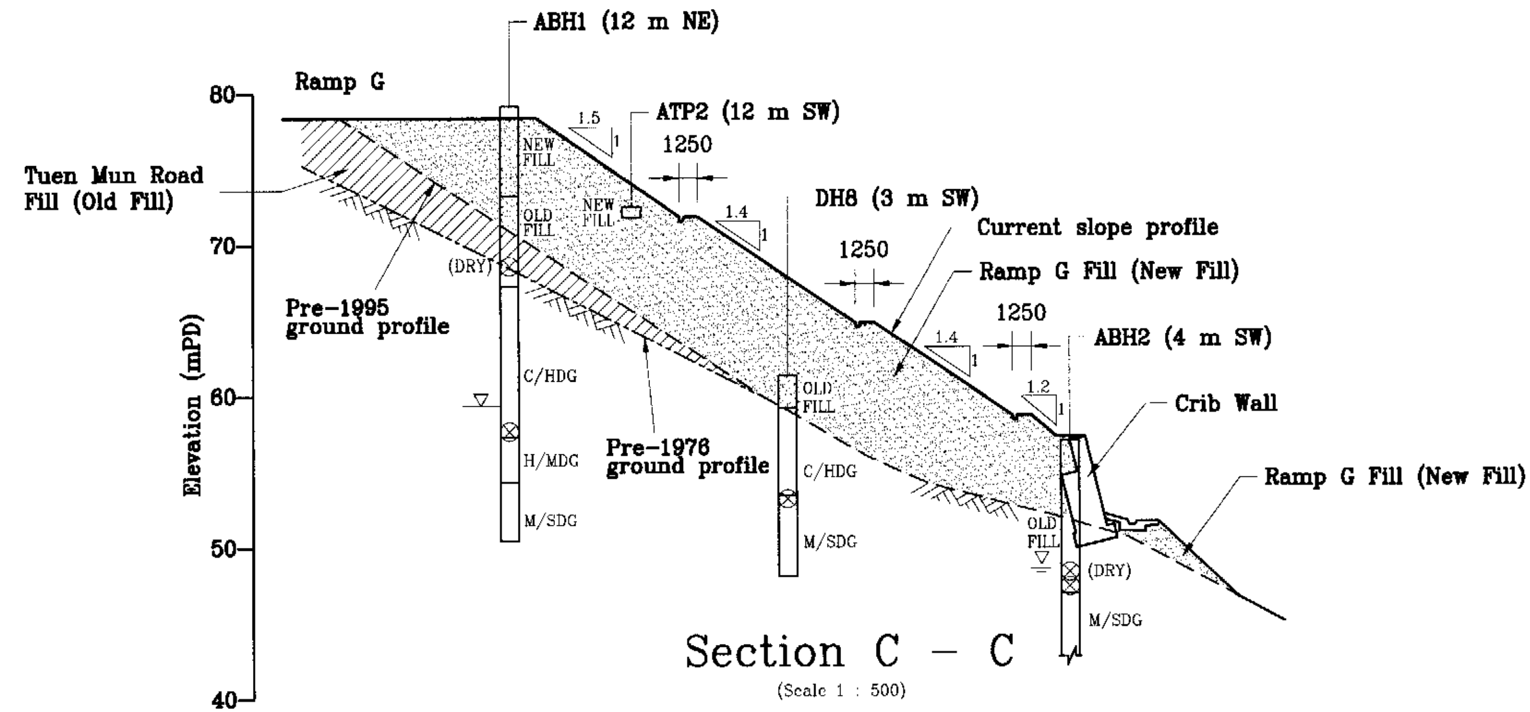
Figure 12 - Geological Sections (Sheet 1 of 4)



Legend : ABH1 (31 m NE)	Drillhole ABH1 offset 31m NORTH EAST	▽	Observed groundwater level (maximum) Dry where noted	OLD FILL	Tuen Mun Road Formation	CDG	Completely Decomposed Granite (Grade V)
						HDG	Highly Decomposed Granite (Grade IV)
				NEW FILL	TKB & AV Ramp G Formation	MDG	Moderately Decomposed Granite (Grade III)
⊗	Piezometer Tip	COLL	Colluvium			SDG	Slightly Decomposed Granite (Grade II)

Notes:
1. See Figure 8 for location of section.
2. The current slope profile is based on the final survey of the Ramp G Fill Slope between chainages 810 and 1004 by Highways Department Survey Section (TKB) (1998). Drg. ref No. E05598/WD/001.
3. The Pre-1976 and Pre-1995 fill slope construction details are based on Scott Wilson Kirkpatrick & Partners Tuen Mun Road Stage I Construction Details (1979) Drg. No. 76811/G29, 76811/1052R and 71805/208R and on Binnie Consultants Ltd. Ting Kau Bridge and Approach Viaduct Contract Drg. No. TKB/SRS/BC/1510.
4. The Pre-1995 ground profile is based upon initial survey for Ting Kau Bridge and Approach Viaduct Contract by Highways Department Survey Section (TKB) (1995). Drg. No. TKC/TKB/SVR/002.

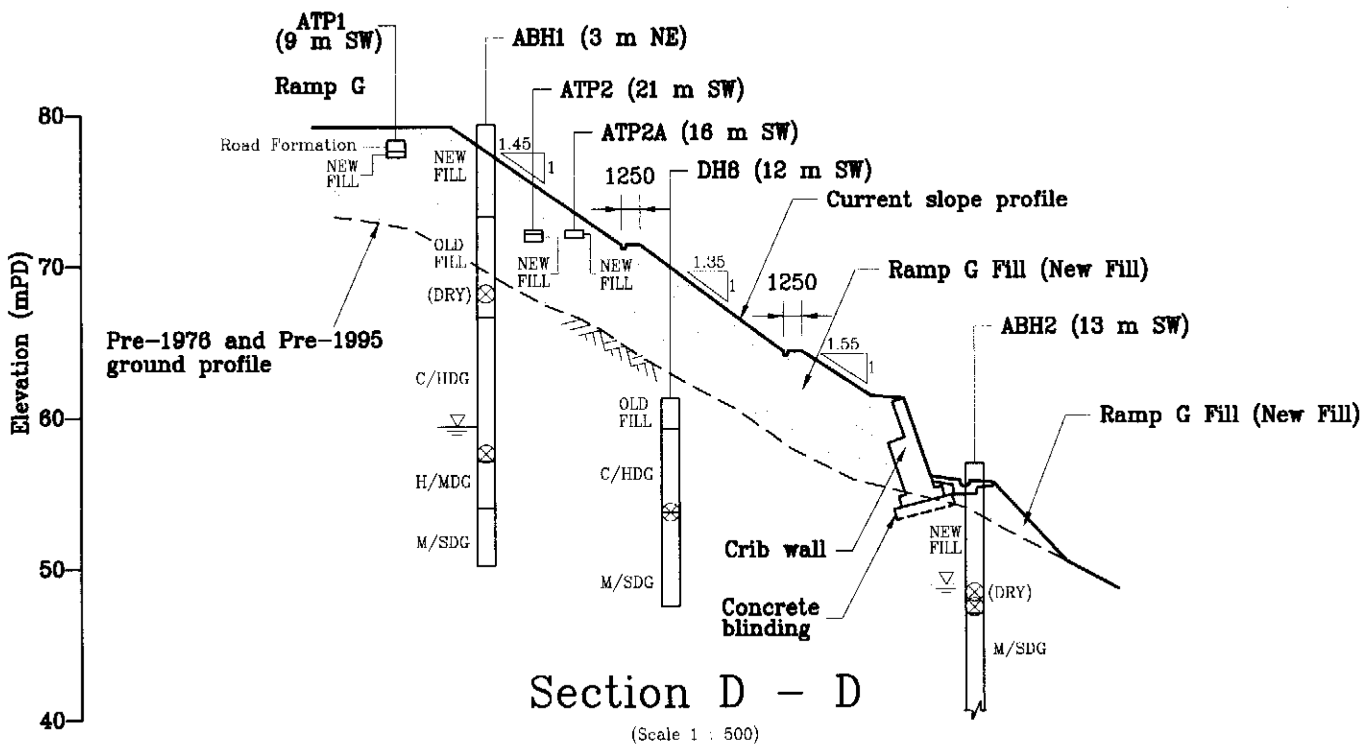
Figure 13 - Geological Sections (Sheet 2 of 4)



Legend : ABH1 (31 m NE)	Drillhole ABH1 offset 31m NORTH EAST	⊻	Observed groundwater level (maximum) Dry where noted	OLD FILL	Tuen Mun Road Formation	CDG	Completely Decomposed Granite (Grade V)
						HDG	Highly Decomposed Granite (Grade IV)
				NEW FILL	TKB & AV Ramp G Formation	MDG	Moderately Decomposed Granite (Grade III)
⊗	Piezometer Tip	COLL	Colluvium			SDG	Slightly Decomposed Granite (Grade II)

Notes:
1. See Figure 8 for location of section.
2. The current slope profile is based on the final survey of the Ramp G Fill Slope between chainages 810 and 1004 by Highways Department Survey Section (TKB) (1998). Drg. ref No. E05598/WD/001.
3. The Pre-1976 and Pre-1995 fill slope construction details are based on Scott Wilson Kirkpatrick & Partners Tuen Mun Road Stage I Construction Details (1979) Drg. No. 76811/G29, 76811/1052R and 71805/208R and on Binnie Consultants Ltd. Ting Kau Bridge and Approach Viaduct Contract Drg. No. TKB/SRS/BC/1510.
4. The Pre-1995 ground profile is based upon initial survey for Ting Kau Bridge and Approach Viaduct Contract by Highways Department Survey Section (TKB) (1995). Drg. No. TKC/TKB/SVR/002.

Figure 14 - Geological Sections (Sheet 3 of 4)



Legend : ABH1 (31 m NE)	Drillhole ABH1 offset 31m NORTH EAST	▽	Observed groundwater level (maximum) Dry where noted	OLD FILL	Tuen Mun Road Formation	CDG	Completely Decomposed Granite (Grade V)
				NEW FILL	TKB & AV Ramp G Formation	HDG	Highly Decomposed Granite (Grade IV)
⊗	Piezometer Tip	COLL	Colluvium			MDG	Moderately Decomposed Granite (Grade III)
						SDG	Slightly Decomposed Granite (Grade II)

Notes:
1. See Figure 8 for location of section.
2. The current slope profile is based on the final survey of the Ramp G Fill Slope between chainages 810 and 1004 by Highways Department Survey Section (TKB) (1998). Drg. ref No. E05598/WD/001.
3. The Pre-1976 and Pre-1995 fill slope construction details are based on Scott Wilson Kirkpatrick & Partners Tuen Mun Road Stage I Construction Details (1979) Drg. No. 76811/G29, 76811/1052R and 71805/208R and on Binnie Consultants Ltd. Ting Kau Bridge and Approach Viaduct Contract Drg. No. TKB/SRS/BC/1510.
4. The Pre-1995 ground profile is based upon initial survey for Ting Kau Bridge and Approach Viaduct Contract by Highways Department Survey Section (TKB) (1995). Drg. No. TKC/TKB/SVR/002.

Figure 15 - Geological Sections (Sheet 4 of 4)

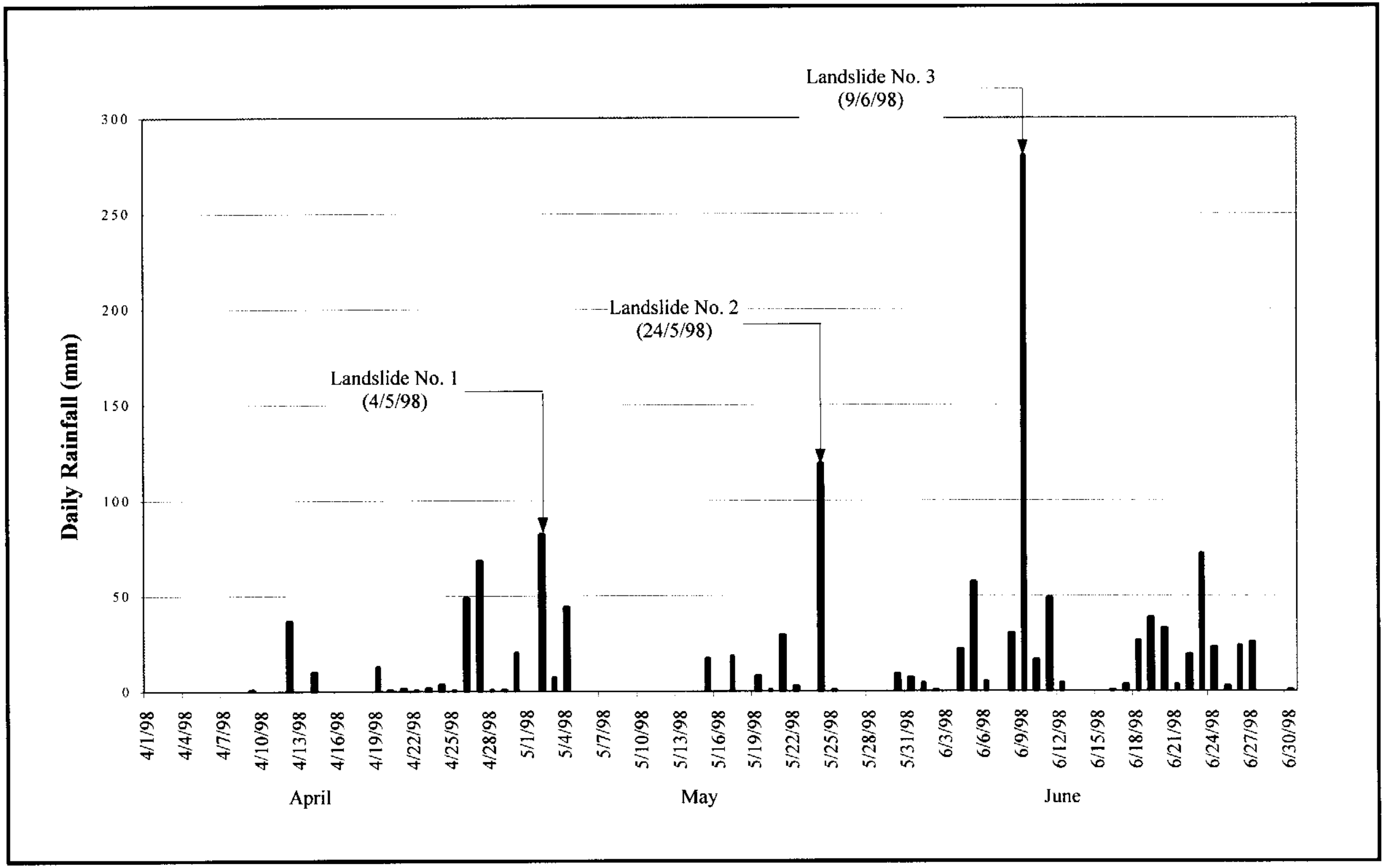


Figure 16 – Rainfall Records at GEO Raingauge No. N10 (1 April 1998 to 30 June 1998)

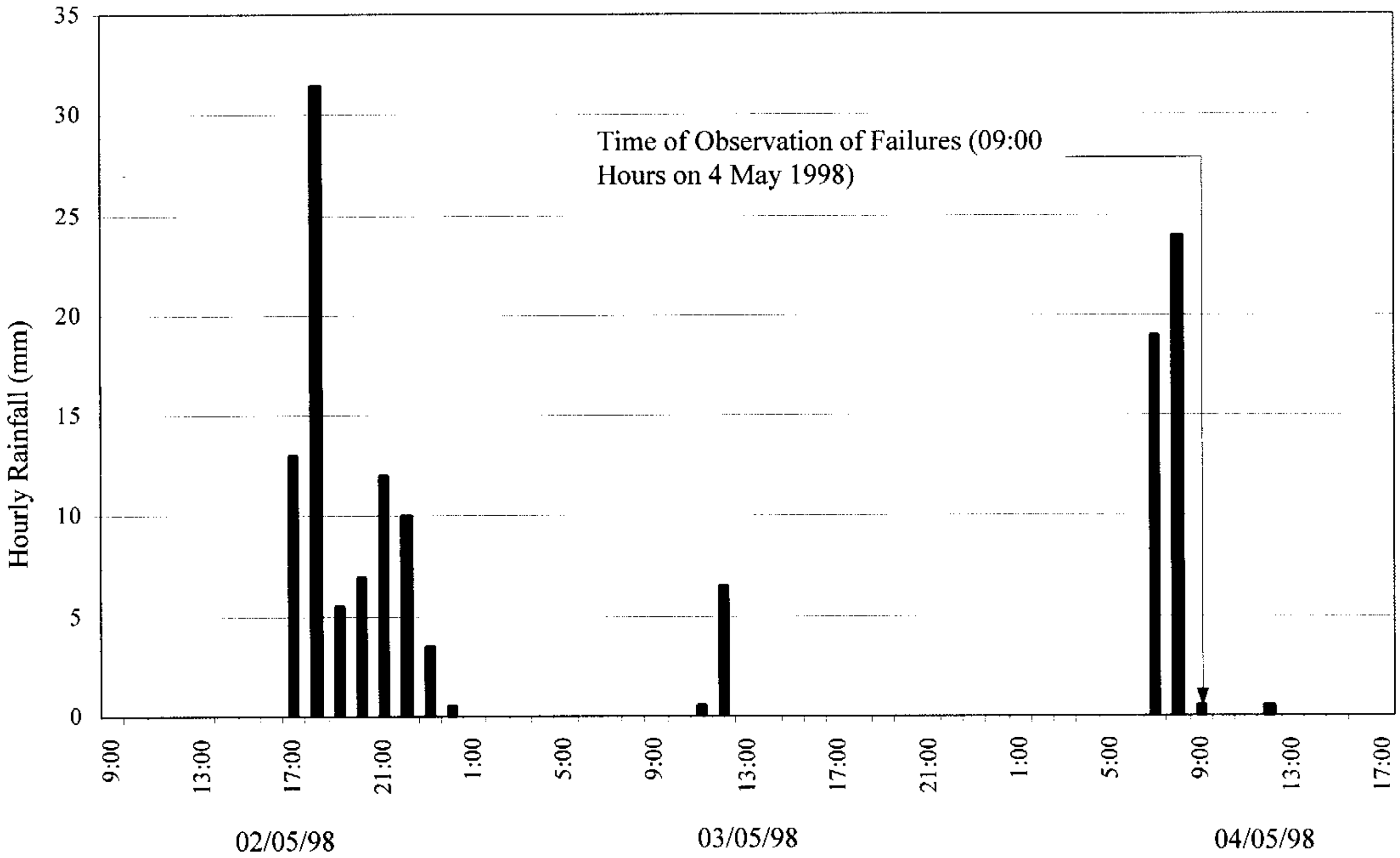
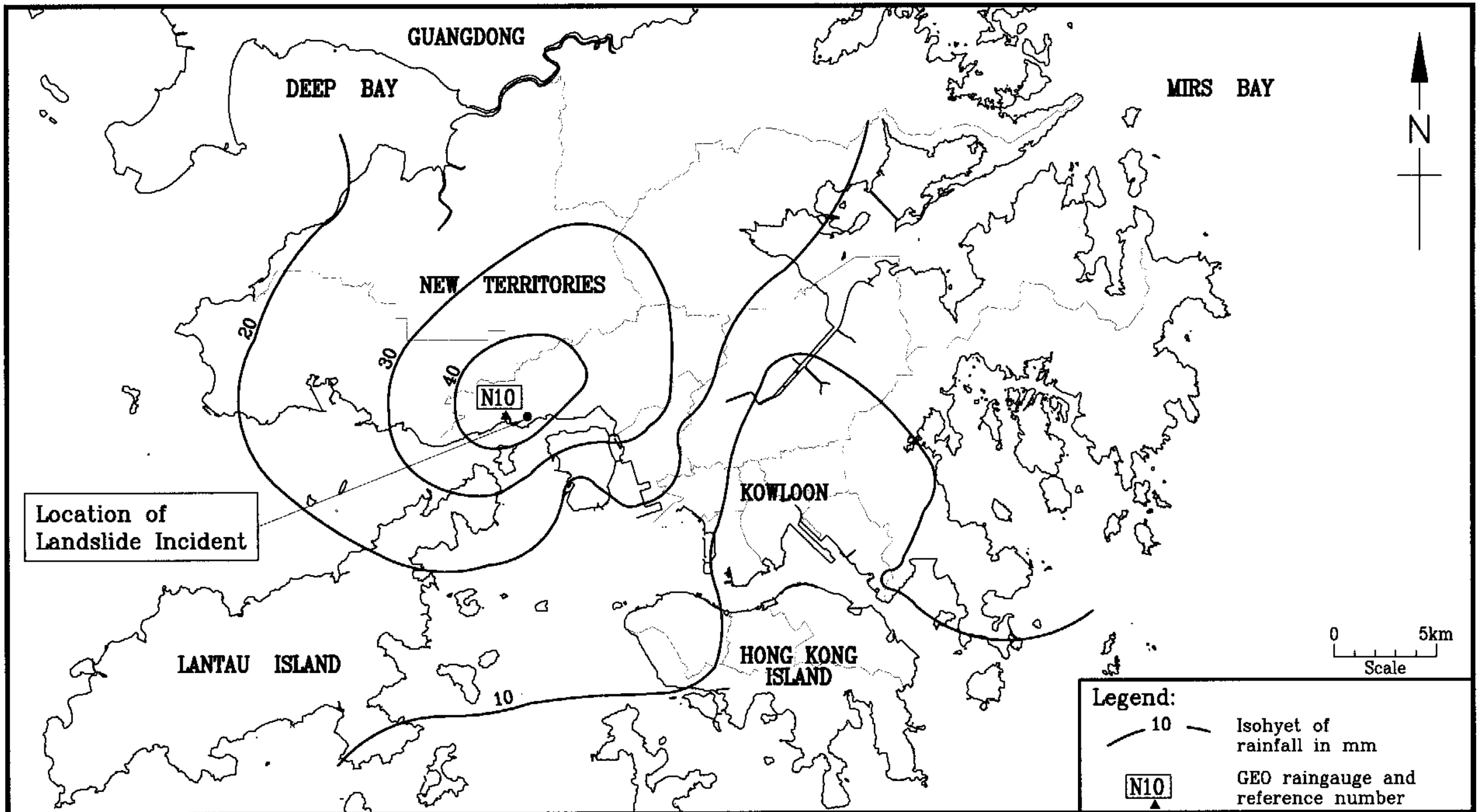


Figure 17 – Hourly Rainfall Data Records at GEO Raingauge No. N10 for Rainstorm Event No. 1 of 4 May 1998



Note: Isohyets of Rainfall for the period from 09:05 on 3 May 1998 until 09:05 on 4 May 1998

Figure 18 - Rainfall Distribution in the 24-hour Period Preceding the Landslide of 4 May 1998

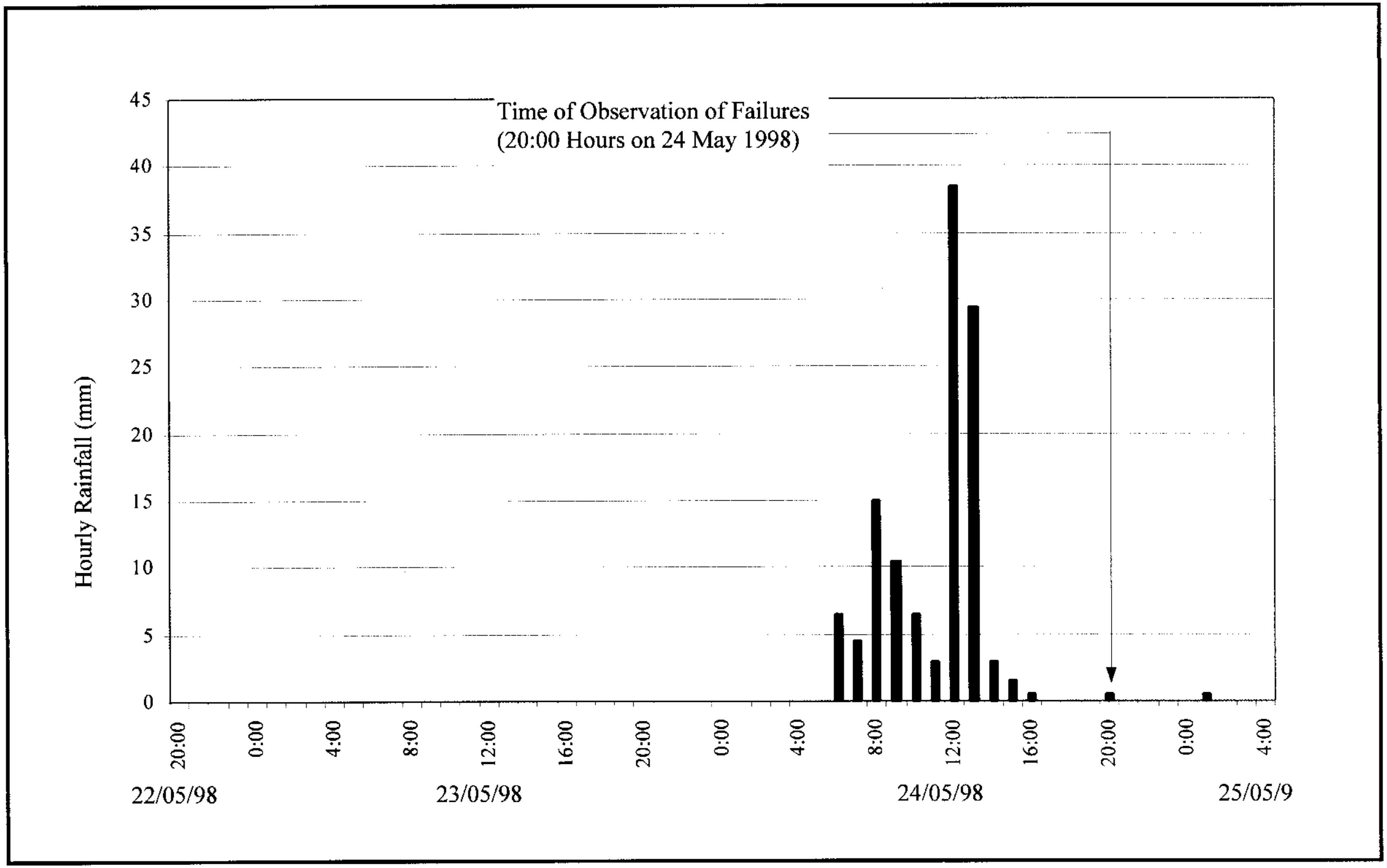
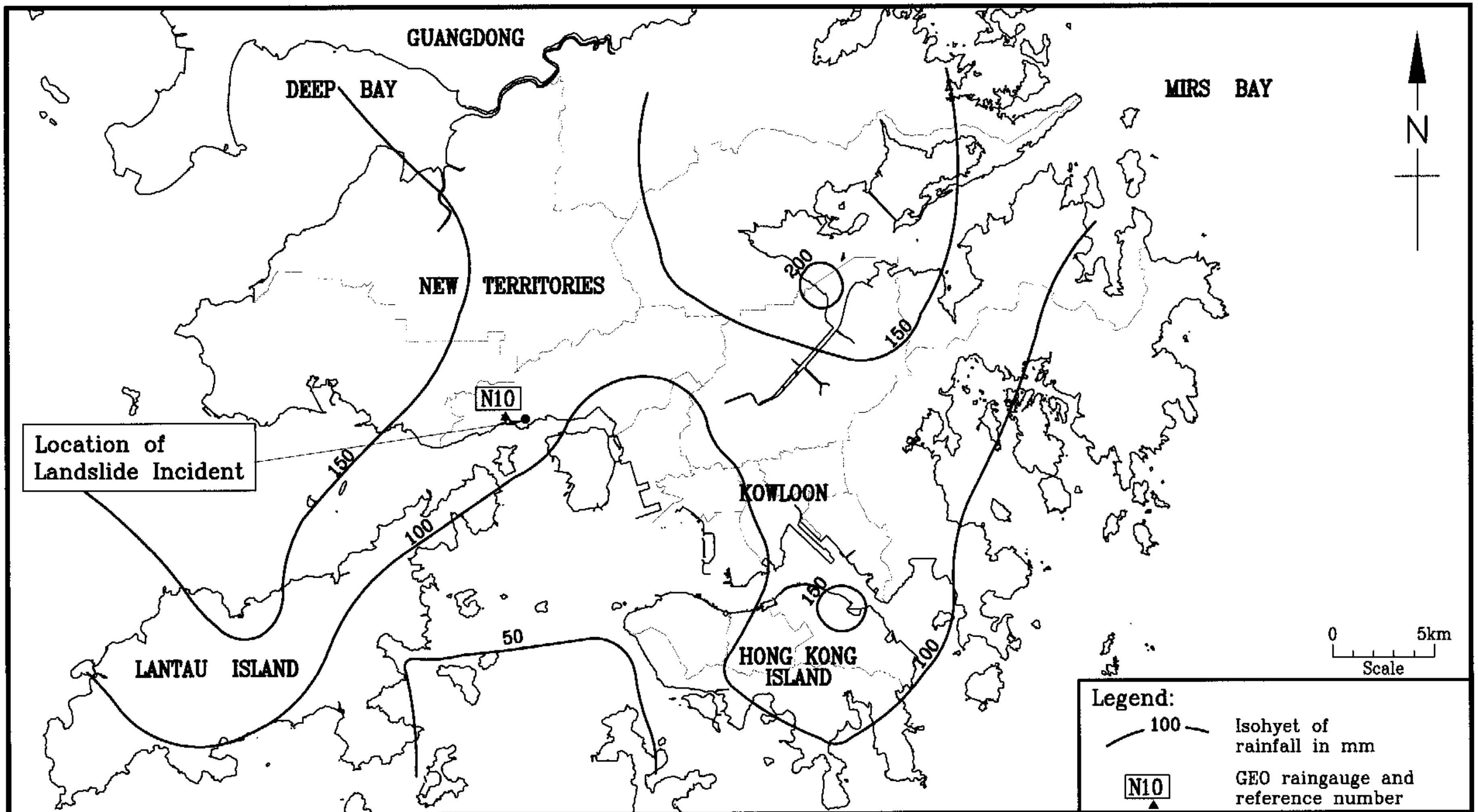


Figure 19 – Hourly Rainfall Data Records at GEO Raingauge No. N10 for Rainstorm Event No. 2 of 24 May 1998



Note: Isohyets of Rainfall for the period from 16:05 on 23 May 1998 until 16:05 on 24 May 1998

Figure 20 - Rainfall Distribution in the 24-hour Period Preceding the Landslide of 24 May 1998

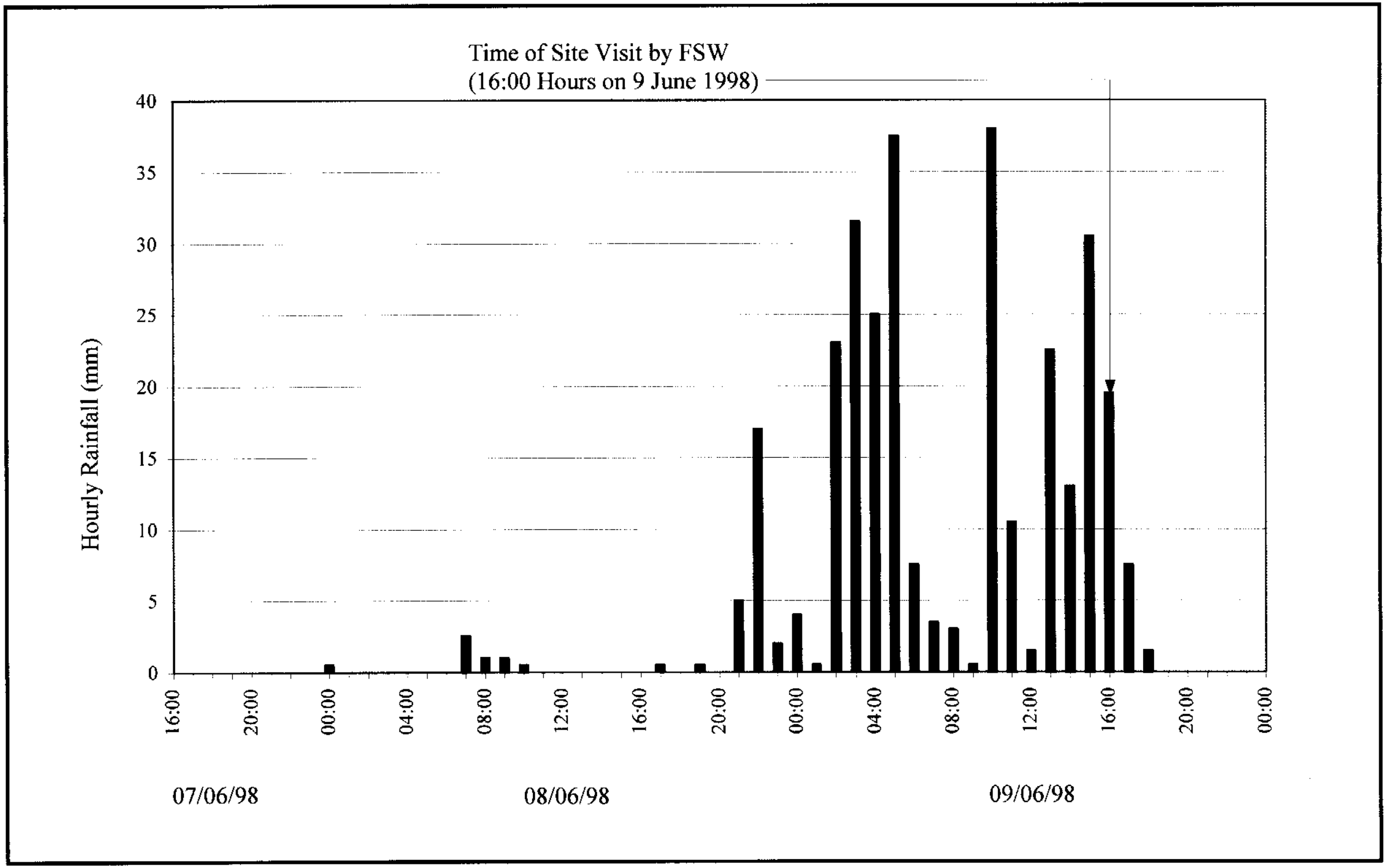


Figure 21 – Hourly Rainfall Data Records at GEO Raingauge No. N10 for Rainstorm Event No. 3 of 9 June 1998

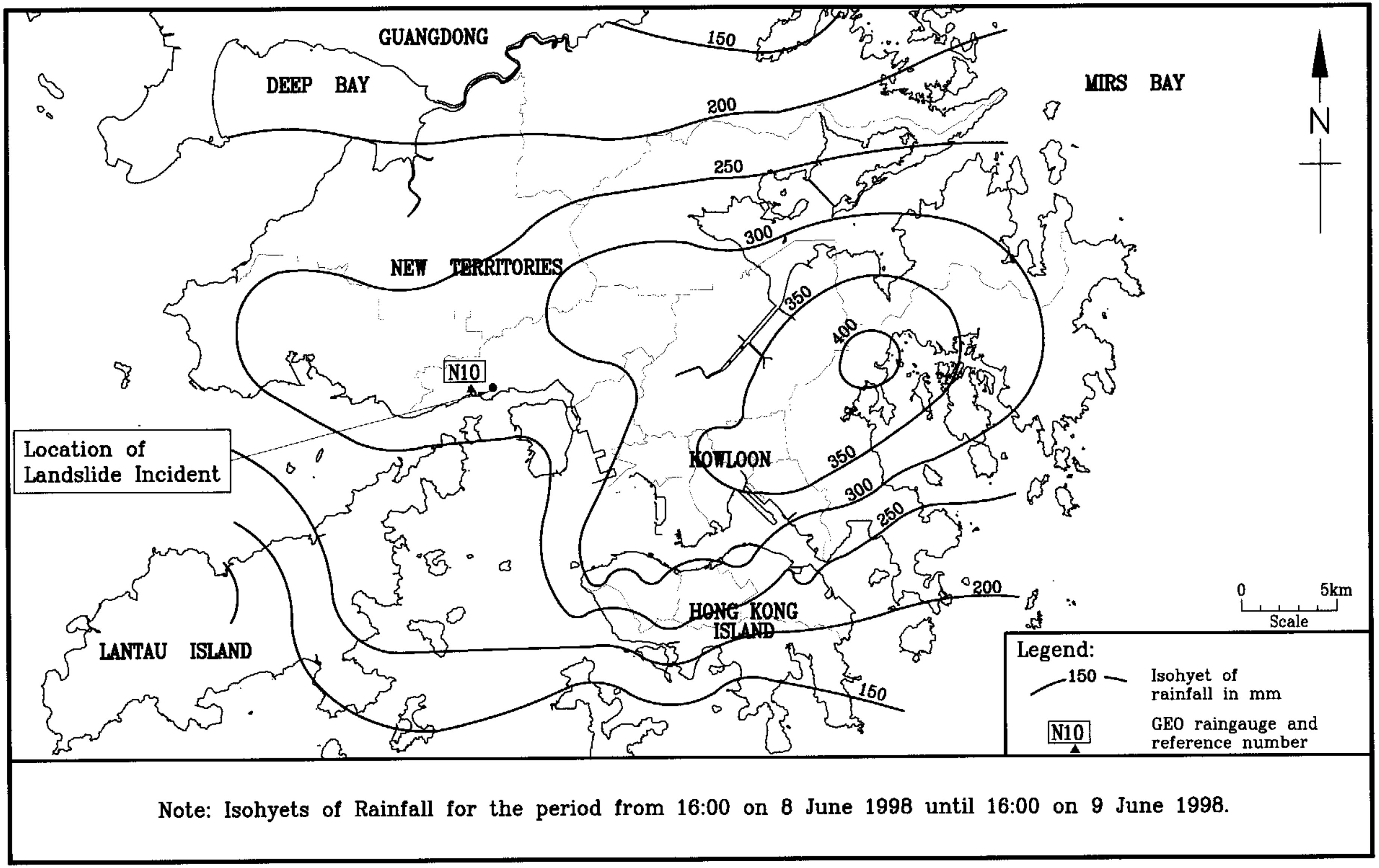
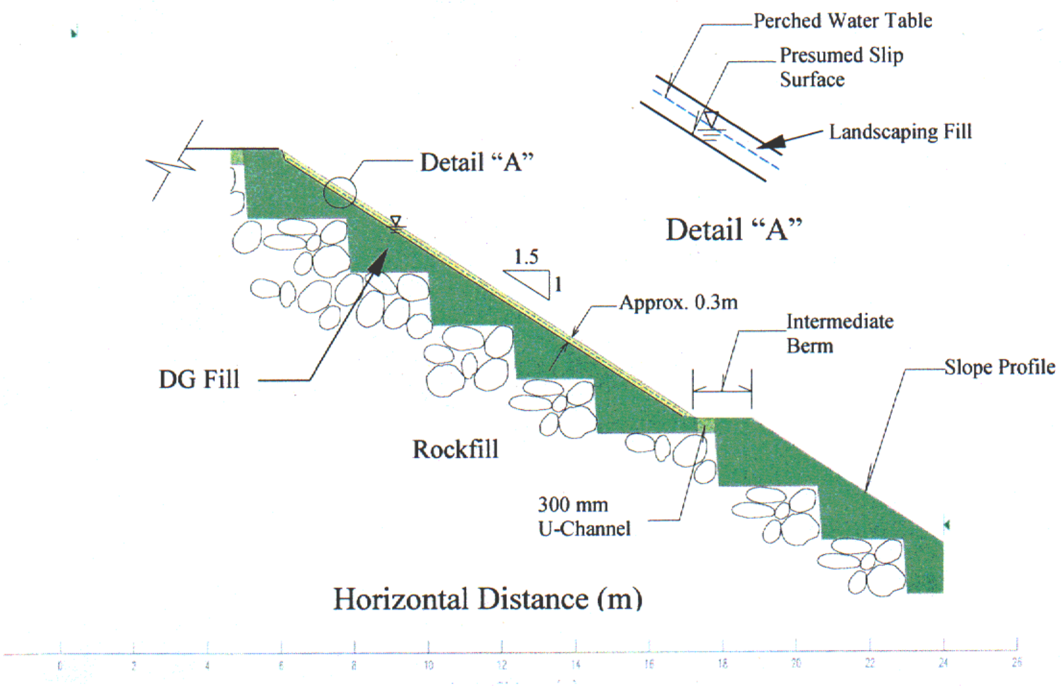
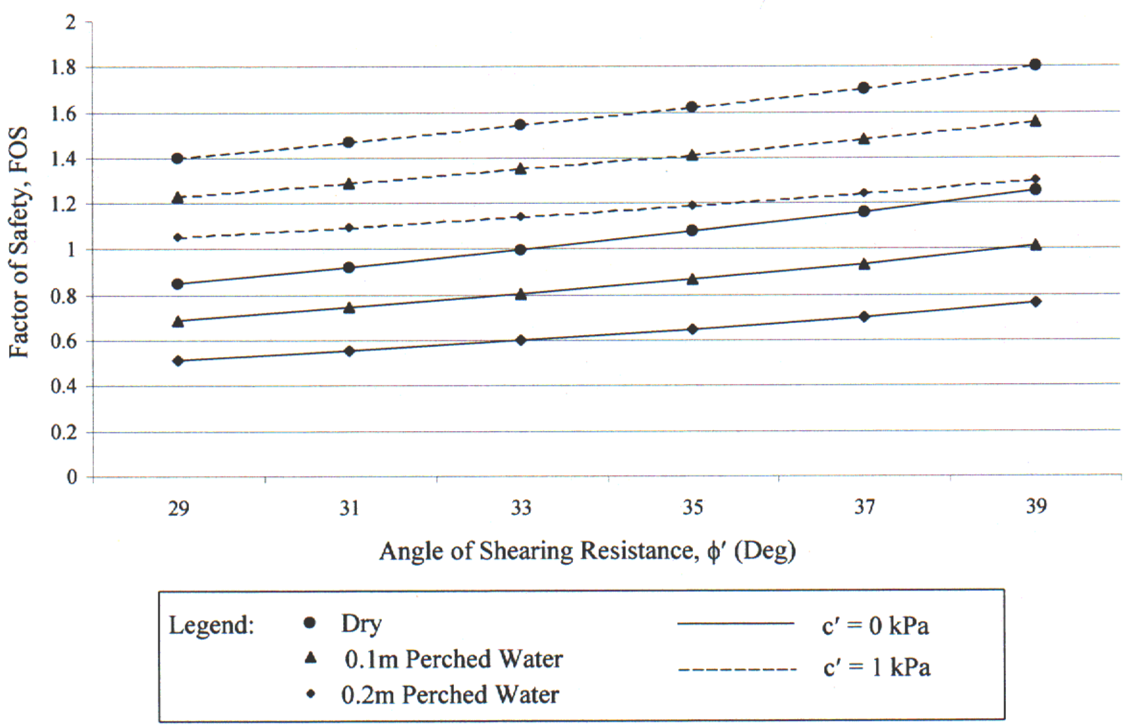


Figure 22 - Rainfall Distribution in the 24-hour Period Preceding the Landslide of 9 June 1998



(a) Typical Details of the Fill Slope (Scale not shown)



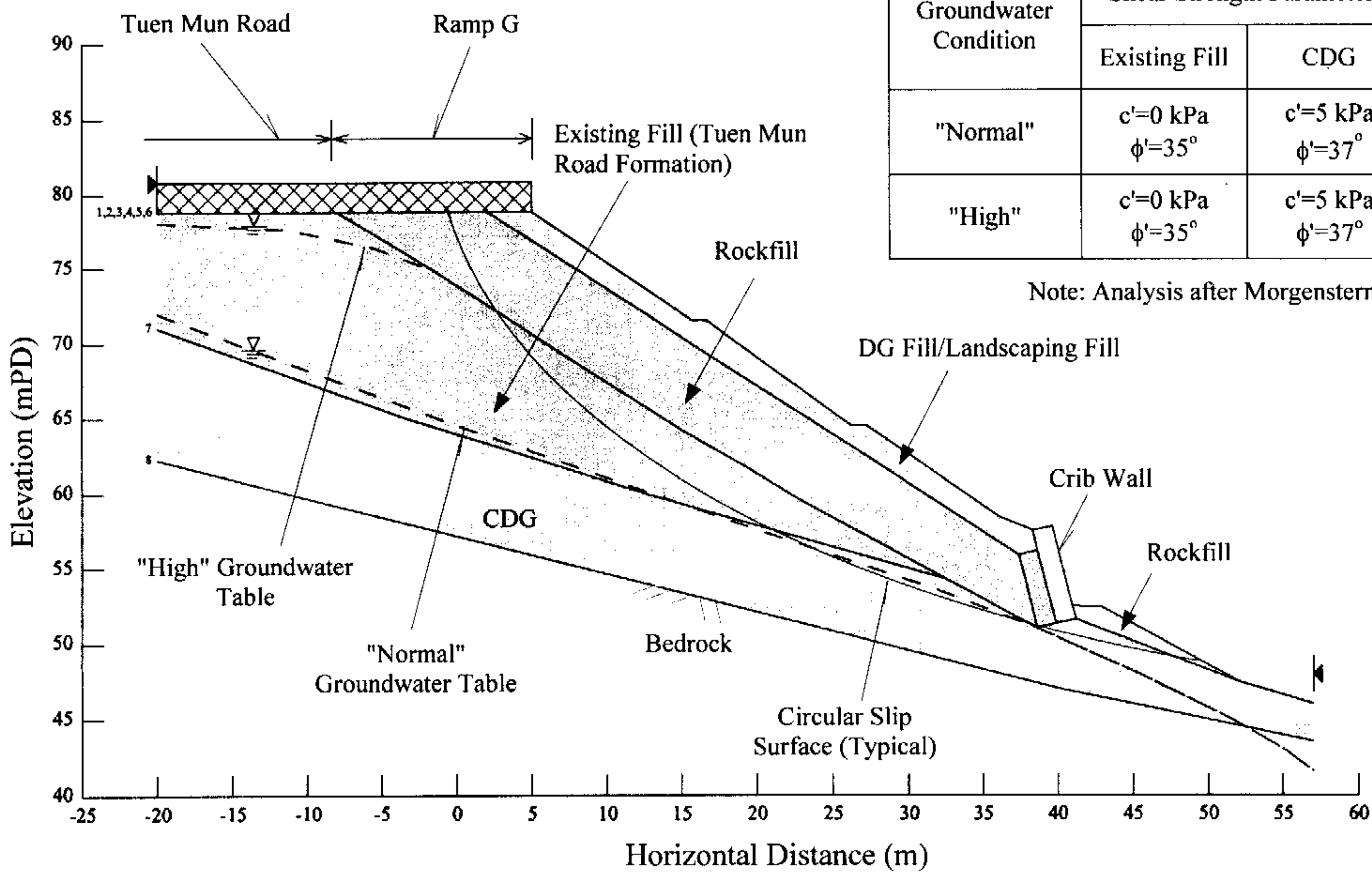
(b) Shear Strength along Surface of Rupture

Note: Analysis after Morgenstern & Price (1965).

Figure 23 – Theoretical Stability Analysis (Sheet 1 of 3)

Groundwater Condition	Shear Strength Parameters		F_{min}
	Existing Fill	CDG	
"Normal"	c'=0 kPa ϕ'=35°	c'=5 kPa ϕ'=37°	1.34
"High"	c'=0 kPa ϕ'=35°	c'=5 kPa ϕ'=37°	1.22

Note: Analysis after Morgenstern & Price (1965).



Slope A (Scale as shown)

Figure 24 – Theoretical Stability Analysis (Sheet 2 of 3)

Groundwater Condition	Shear Strength Parameters		F_{min}
	Existing Fill	CDG	
"Normal"	c'=0 kPa ϕ'=35°	c'=5 kPa ϕ'=37°	1.45
"High"	c'=0 kPa ϕ'=35°	c'=5 kPa ϕ'=37°	1.12

Note: Analysis after Morgenstern & Price (1965).

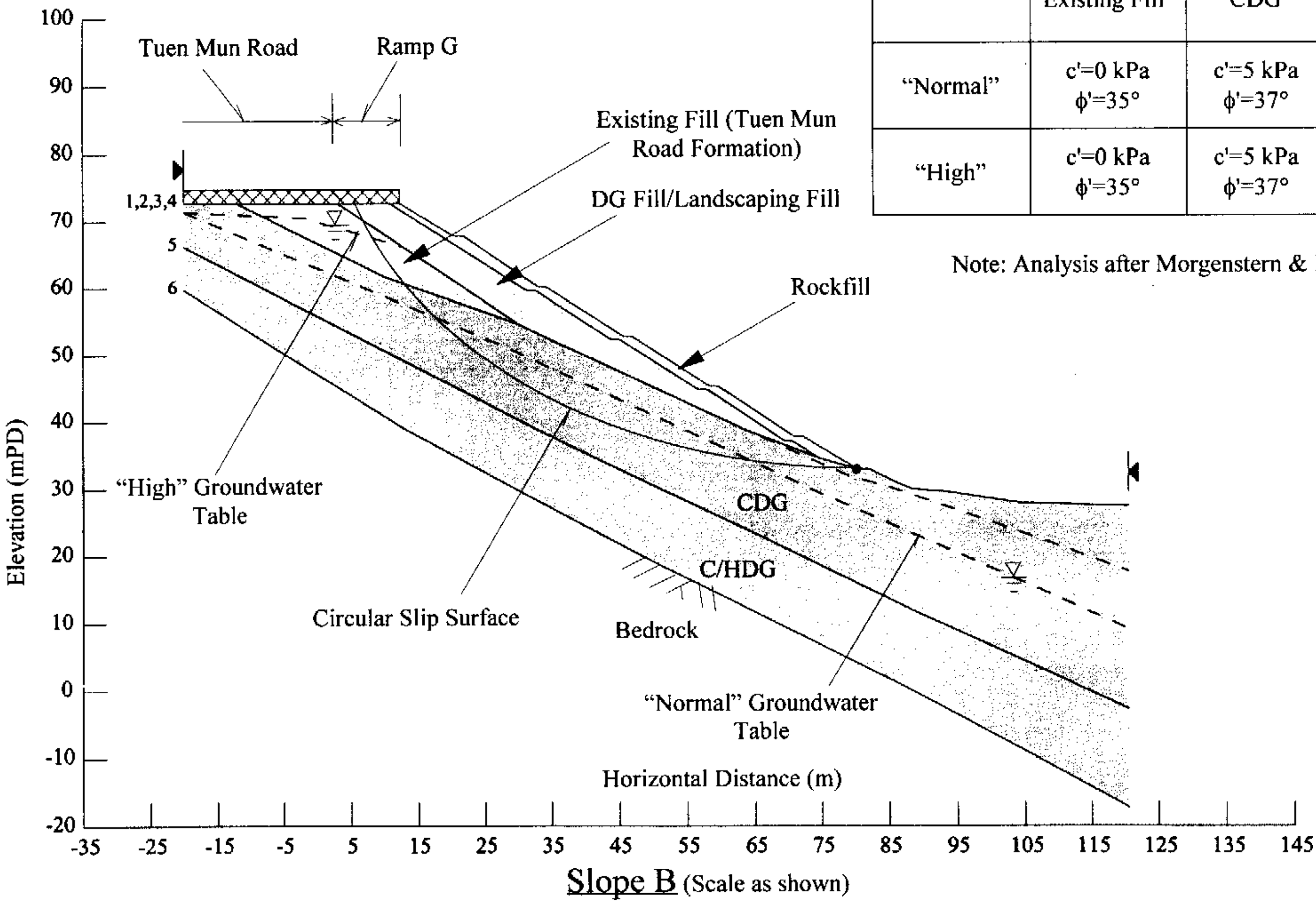


Figure 25 – Theoretical Stability Analysis (Sheet 3 of 3)

LIST OF PLATES

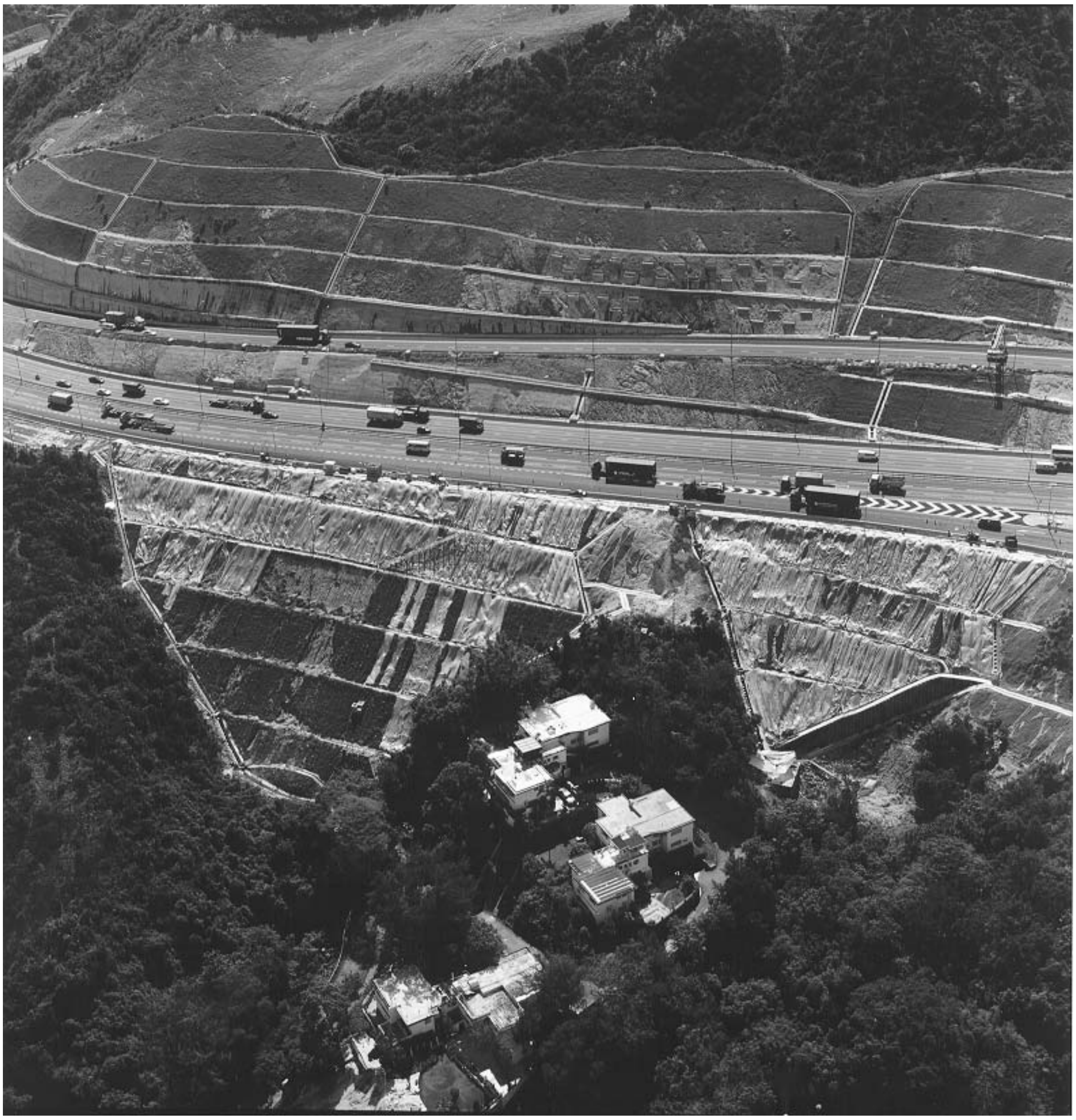


Plate 1 – Low-Level Oblique View Across the Landslide Site. (Photograph Taken on 29 June 1998)

Plate 2 – View North-East Across Landslide Scar in Slope No. 6SE-C/FR13. (Photograph Taken on 10 October 1995)

Plate 3 – View South-East Over Landslide Scar Indicating Debris Runout. Concrete Structure Visible in Debris on Left. Reservoir Filled With Debris Visible in Centre Background. (Photograph Taken on 10 October 1995)

Plate 4 – View North-West Across Landslide Scar Towards Main Scarp.
Horizontal Layering Visible in Exposed Fill.
(Photograph Taken on 10 October 1995)

Plate 5 - Event No. 1: 4 May 1998.
View North-East Along Crest of Slope B Indicating Tension Cracking South of Road Drainage Outlet on Slope (Outlet Visible on Right of Frame).
(Photograph Taken on 4 May 1998)

Plate 6 – Event No. 1: 4 May 1998. View North-East Along 67.5 mPD (Uppermost) Berm of Slope B and Lower Batters from Southern End of Slope. Note Extensive Blockage of 67.5 mPD Berm Drainage and Failed Areas of Slope on Right of Frame. (Photograph Taken on 4 May 1995)

Plate 7 – Event No. 1: 4 May 1998. View South-West Along Upper Batter of Slope B and 67.5 mPD Berm from Northern End. Note Road Drainage Discharge into Slope Drainage in Middle-Ground and Debris from Landslides Blocking Berm Drainage Channel on Either Side of Catchpit. (Photograph taken on 4 May 1998)

Plate 8 – Event No. 1: 4 May 1998. View North-East Along 60 mPD Berm from South End of Slope B Indicating Scar on Batter Above and Runout of Debris on Batter Below. (Photograph Taken on 4 May 1998)

Plate 9 – Event No. 1: 4 May 1998. View North-East Along 52.5 mPD Berm from South End of Slope B Indicating Scars on Batter Above and Debris Deposition on Berm and Batter Below. (Photograph Taken on 4 May 1998)

Plate 10 – Event No. 1: 4 May 1998. View North-West Towards Crest of Slope from 52.5 mPD Berm at South End of Slope B. White Tile to Right of Centre is at Edge of 60 mPD Berm. Note Landslide Scar in Batter Above 60 mPD and Debris Runout Over Batter Below (Hydroseed Intact). Note Also Slumping of Face of Batter to Left of Scar. (Photograph Taken on 4 May 1998).

Plate 11 – Event No. 1: 4 May 1998. View North-East Along Batter Between 45 mPD and 52.5 mPD Berms from South End of Slope B. Note accumulated Debris in Foreground, with Scar Extending from Batter Above 52.5 mPD Berm to 45 mPD Berm and Debris Accumulation on Berm. (Photograph Taken on 4 May 1998)

Plate 12 – Event No. 1: 4 May 1998. View North-East Along 45 mPD Berm from South End of Slope B. Accumulated Debris on Berm in Foreground. Note Also Debris on Same Berm in Distance Beyond Manhole on Slope Face, and Debris Deposition on 37.5 mPD Berm Below on Right of Frame. (Photograph Taken on 4 May 1998)

Plate 13 – Event No. 1: 4 May 1998. View North-West Towards Crest of Slope B from 45 mPD Berm up Succession of Landslide Scars Below Road Drainage Outlet Above 67.5 mPD Berm at CH 1000 Note Washout of Scars and Accumulation of Debris on Batter Above 45 mPD Berm in Foreground. (Photograph Taken on 4 May 1998)

Plate 14 – Event No. 1: 4 May 1998. View West Along Perimeter Drainage at South End of Slope B from Outlet of Culvert No. P044 Below 45 mPD Berm (Berm is Level with Catchpit). Note Scars and Debris Runout and Deposition on Batters Above. (Photograph Taken on 4 May 1998).

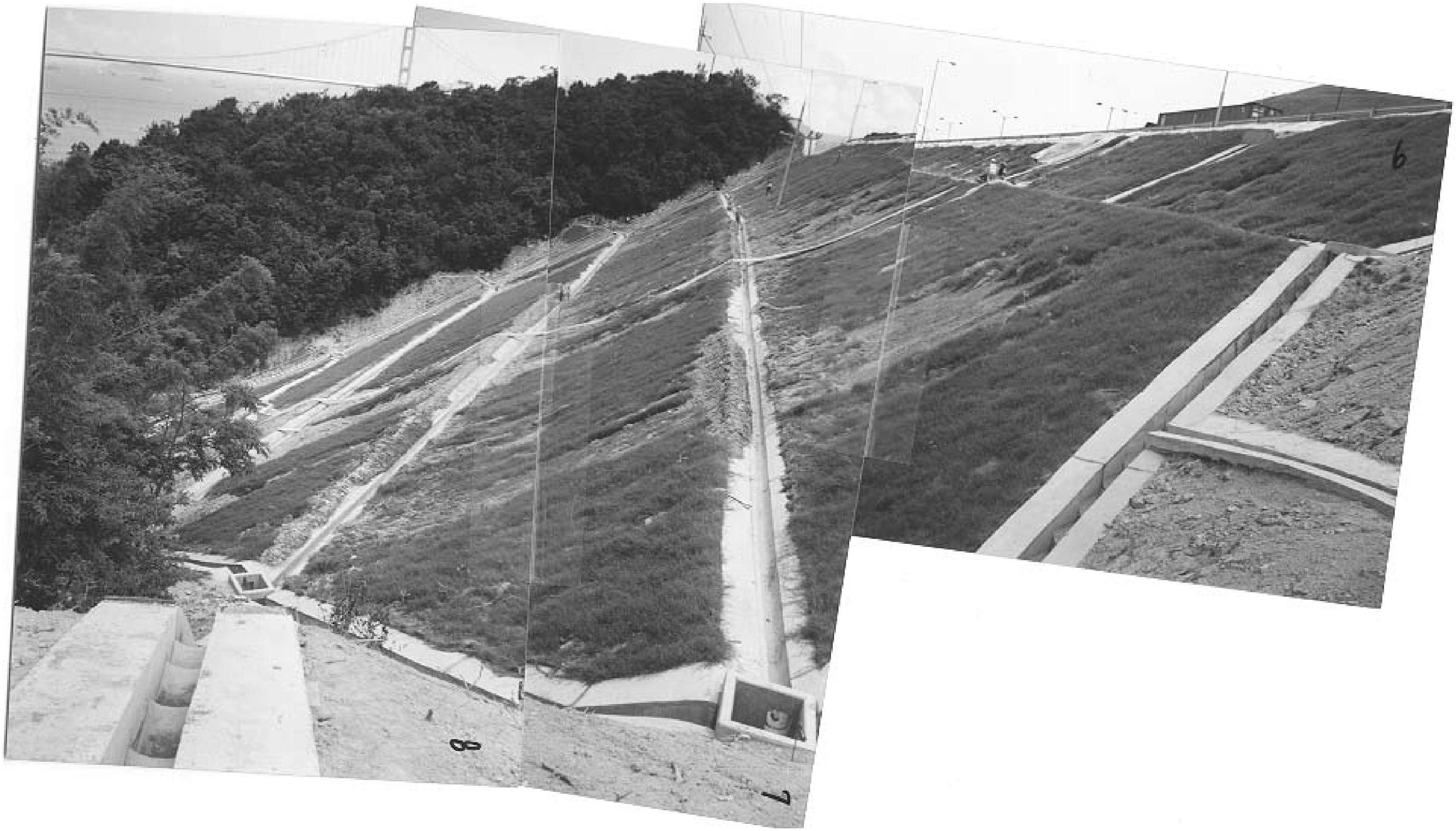


Plate 15 – Event No. 1: 4 May 1998. View South-West Across Slope B from Natural Spur Separating Slopes A and B. 60 mPD Berm in Centre of Frame. Note Extent of Scars in Slope Face. Note also Clearance of Berm Channels and Stock piling of Debris on Berm Apron. (Photographs Taken on 8 May 1998)

Plate 16 – Event No. 1: 4 May 1998. View South-West Along 60 mPD Berm Towards South End of Slope B. Note Separation of Apron Slab from Berm U-Channel. (Photograph Taken on 8 May 1998)

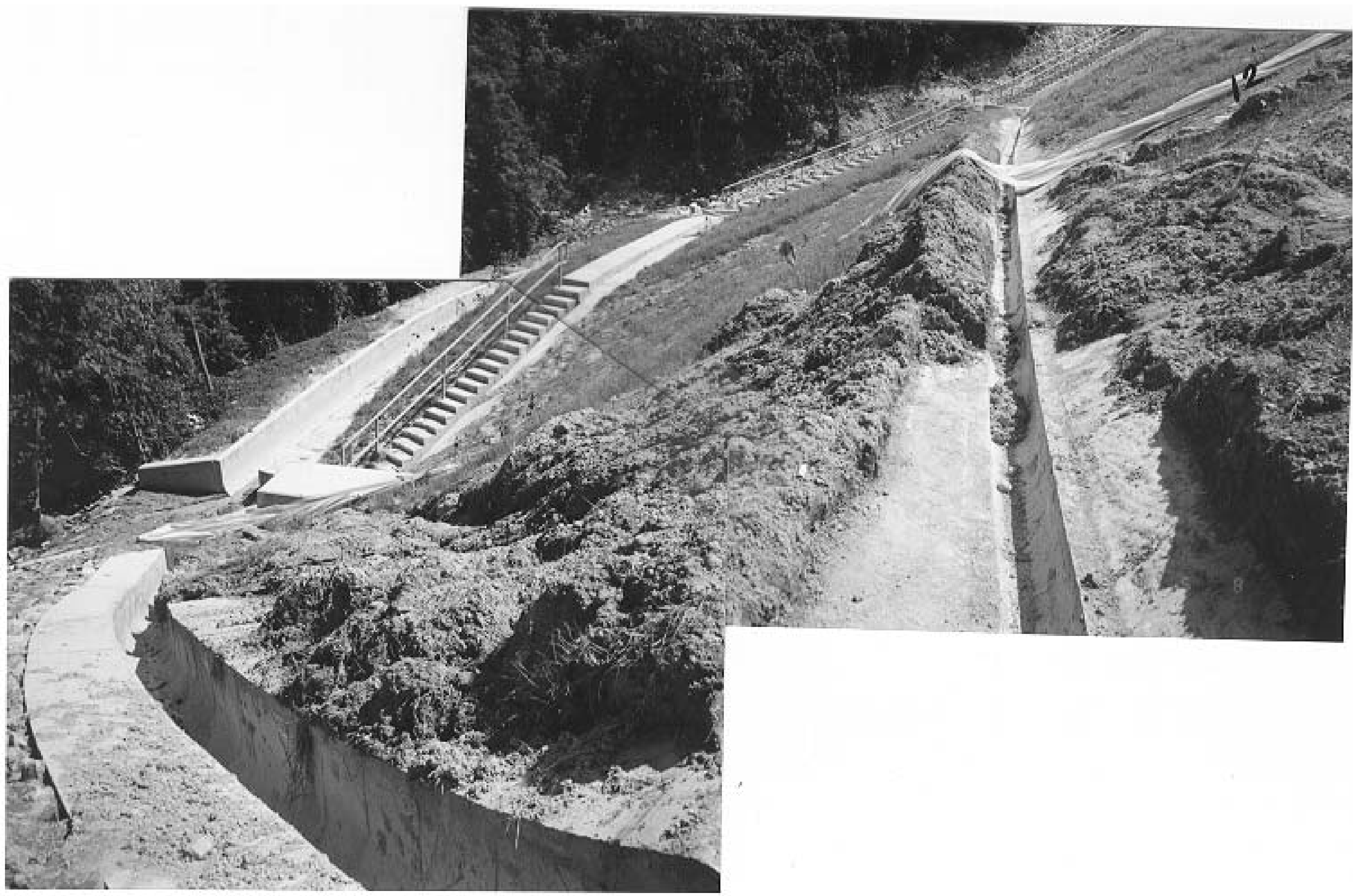


Plate 17 - Event No. 1: 4 May 1998. View South-West Along 37.5 mPD Berm Towards South End of Slope B Indicating Debris Deposition on Slope Face. Clearance of Berm Drainage in Progress. Note Also Apparent Runout Over Batter Below Berm in Middle Distance Indicated by Flattened Hydroseeding. Outlet for Culvert P043 Visible at Toe of Slope on Left of Frame. (Photograph Taken on 8 May 1998)

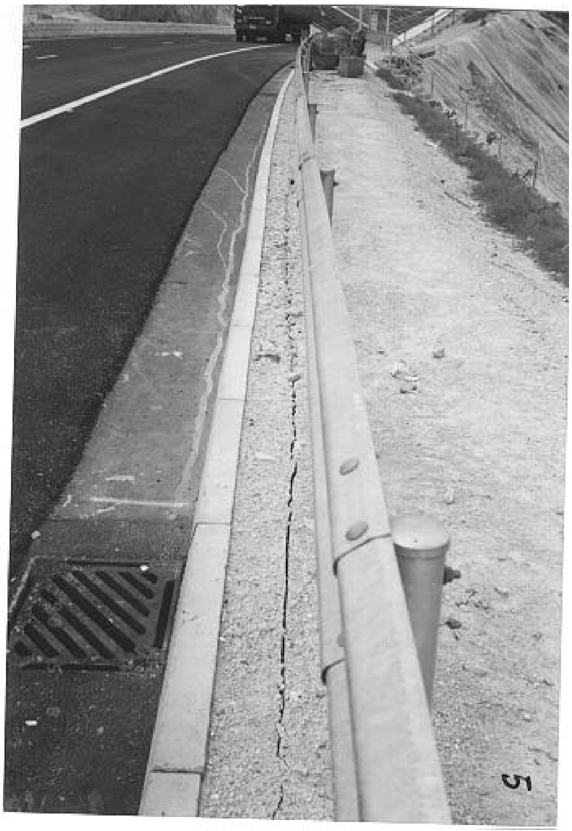


Plate 18 – Event No. 1: 4 May 1998. View North-East Along Crest of Slope A Indicating Cracking Behind Kerb Beam. (Photograph Taken on 8 May 1998)

Plate 19 – Event No. 2: 24 May 1998. View South-West Across Slope A and Slope B, Indicating Extensive Failure in Upper Two Batters of Slope A, with Runout of Debris and Additional Failures on Third and Fourth Batters (Below 64.5 mPD and 58 mPD Berms, Respectively). Clearance of Berm Drainage Channels Underway. Note Stockpiling of Debris on Berm Aprons. Note Also Little Change to Appearance of Slope B Since 8 May 1998 (Refer Plate 15). (Photograph Taken on 28 May 1998)

Plate 20 – Event No. 2: 24 May 1998. View South Towards Toe of Slope A Indicating Debris Deposition on Slope Face, with Over-spill onto Concrete Apron at Toe of Crib Wall and Reservoir Structure. (Photograph Taken on 28 May 1998)

Plate 21 – Event No. 2: 24 May 1998. View North-West Towards Crest of Slope A Indicating Scars and Debris Runout on Slope Face. Note Layering Parallel to Slope Face in Scar on Lower Batter and Secondary Washout of Scar. Note Also Main Scarp in Debris on Batter Above in Right of Frame. (Photograph Taken on 28 May 1998)

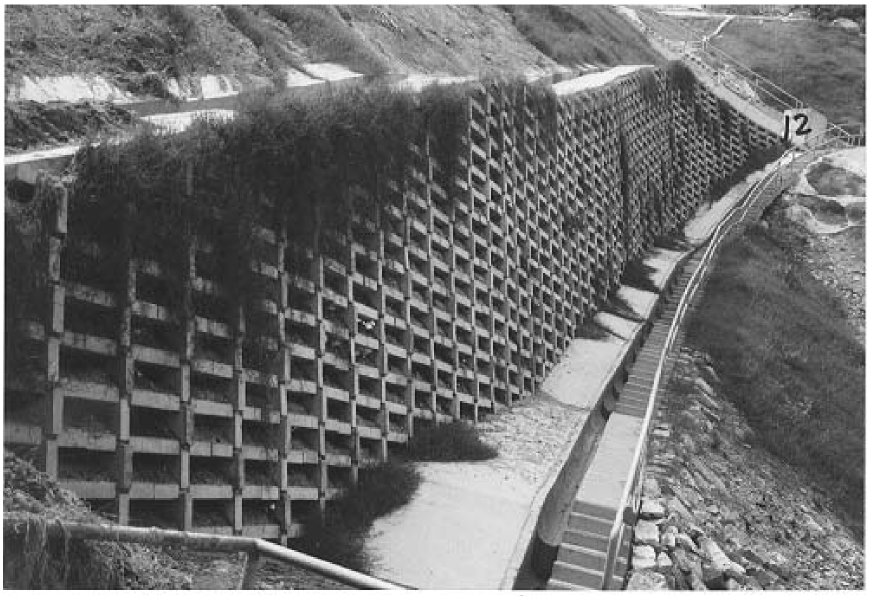


Plate 22 – Event No. 2: 24 May 1998. View North Along Crib Wall at Toe of Slope A. Note Lack of Debris Over-spill at Toe of Wall and Loss of Fine Material (Topsoil) From Wall Infill. (Photograph Taken on 28 May 1998)

Plate 23 – Event No. 2: 24 May 1998. Close-up of Access stairway Below Crib Wall Indicating Washout of Fill Material and Undermining of Stairs. (Photograph Taken on 28 May 1998)

Plate 24 – Event No. 2: 24 May 1998. View South-West Along Crest of Slope B Indicating Separation Between Asphalt pavement and Kerb Beam. (Photograph Taken on 28 May 1998)

Plate 25 – Event No. 3: 9 June 1998. View South-West Along 45 mPD Berm on Slope B Indicating New Failures in Slope B Affecting Batter Above 52.5 mPD Berm (Right of Frame). Note Debris Over-Spill onto Batter Below 45 mPD Berm and Blockage of Berm Drainage. (Photograph Taken on 15 June 1998)

Plate 26 – Event No. 3: 9 June 1998. View South from 45 mPD Berm on Slope B Indicating Debris Runout from New Failure in Batter Above 37.5 mPD Berm to Batter Below and Beyond Toe of Slope. (Photograph Taken on 15 June 1998)

Plate 27 – Event No. 3: 9 June 1998. View Towards Crest of Slope B From 45 mPD Berm Indicating Scars of New Failures. Note Secondary Washout of Scars Exposing Rock Fill Beneath. (Photograph Taken on 15 June 1998)

Plate 28 – Event No. 3: 9 June 1998. View North from 34.5 mPD Berm Towards New Failure in Batter Below 37.5 mPD Berm. Note Rock Fill Exposed in Scar and Main Scarp in Debris on Berm. (Photograph Taken on 15 June 1998)

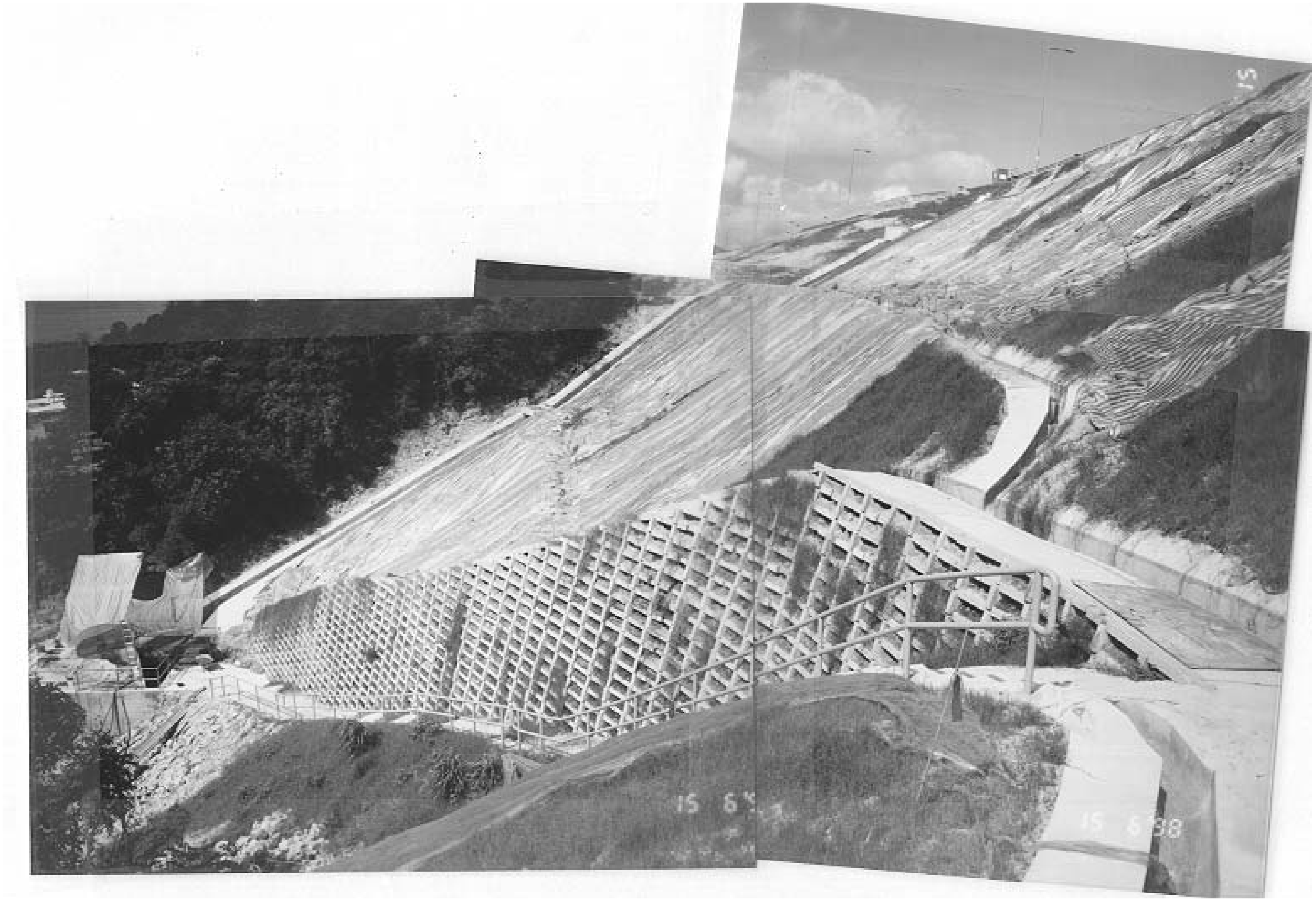


Plate 29 – Event No. 3: 9 June 1998. View South-West Across Slope A. Note Extensive Coverage of Slope Face with Plastic Sheeting and only Minor Washout of Debris at Joins in Sheeting. Note Also Blocked Drainage on 58 mPD Berm in Centre of Frame. (Photograph Taken on 15 June 1998)

Plate 30 – Event No. 3: 9 June 1998. View North-West Towards Top Batter of Slope A at South End of Slope. Scar Precedes Event No. 3. (Photograph Taken on 15 June 1998)

Plate 31 – Event No. 3: 9 June 1998. View North-East Along 64.5 mPD Berm on Slope A Indicating Debris Previously Stockpiled on Berms Washed into Surface Drainage. (Photograph Taken on 15 June 1998)

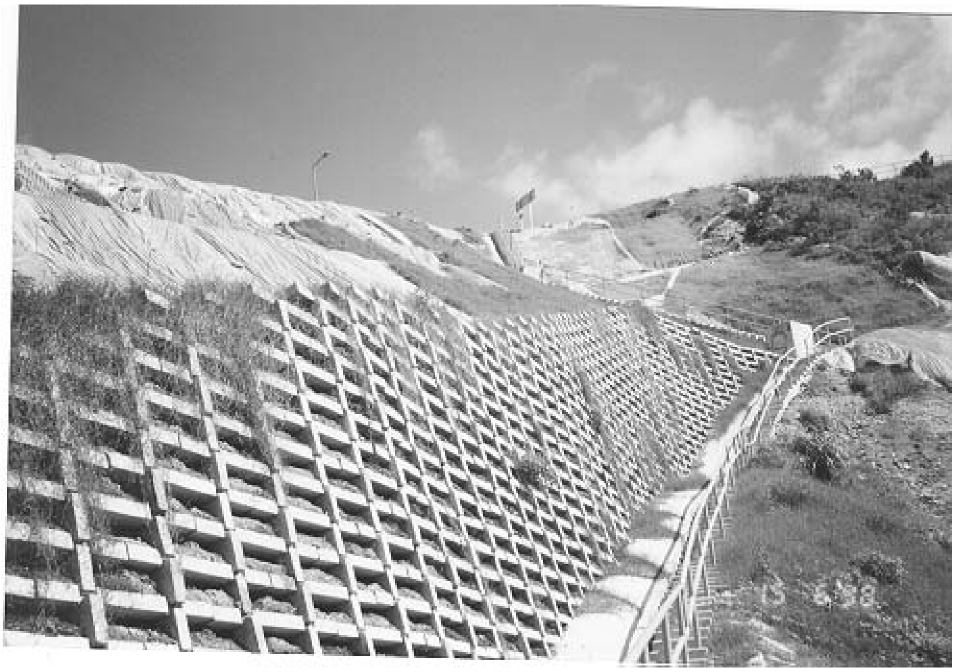


Plate 32 – Event No. 3: 9 June 1998. View North Along Crib Wall at Toe of Slope A. (Photograph Taken on 15 June 1998)

Plate 33 – Event No. 3: 9 June 1998. View South Across Lower Portion of Slope B from 45 mPD. Note Blockage of Surface Drainage by Debris Stockpiled on Berms and Debris from New Failure in Batter Above 37 mPD Berm in Distance. (Photograph Taken on 15 June 1998)

Plate 34 – Event No. 3: 9 June 1998. View Towards Crest of Slope B From 45 mPD Berm Indicating Secondary Washout of Existing Scar. Note Plastic Sheeting on Slope Above and Thin Layer of Rock Fill Exposed Beneath DG Fill, with Existing Fill Visible Behind Rock Fill. (Photograph Taken on 15 June 1998)

Plate 35 – Event No. 3: 9 June 1998. View North-East Along 60 mPD Berm on Slope B Indicating Separation of Berm Apron Slab from Berm U-Channel. Also Note Standing Water in U-Channel. (Photograph Taken on 15 June 1998)

Plate 36 – Event No. 3: 9 June 1998. Close-up of Filled Cracking Behind Kerb Beam Approximately Mid-Way Along Crest of Slope A. Note Also, Separation of Kerb Beam from Pavement. (Photograph Taken on 15 June 1998)

Plate 37 – Event No. 3: 9 June 1998. View North-East Along Hard Shoulder at Crest of Slope A Approximately 35 m from North End Indicating Cracking in Pavement. Cracking Filled with Expanding Foam. (Photograph Taken on 15 June 1998)

Plate 38 – Trial Pit ATP1 Excavation at Crest of Slope A. Note Line of Cracking Across Far Side of Pit Indicated by Yellow Foam on Either Side. Pavement Layers and Stormwater Drain Exposed in Pit. (Photograph Taken on 9 July 1998).

Plate 39 – Close-Up of Cracking Extending Vertically Beyond Base of Trial Pit ATP1. (Photograph Taken on 9 July 1998)

Plate 40 – View South-West from Toe of Slope No. 6SE-C/FR13. New Reservoir Structure on Left of Frame. Crib Wall Footing Adjacent to Reservoir Insitu Profile Exposed Beneath Previous Fill and Rockfill. (Photograph Taken on 7 April 1997)

Plate 41 – View North-East Across Upper Portion of Slope No. 6SE-C/FR13 Indicating Haul Road in Natural Slope to North-East and Rock Fill Material (This Rock Fill Apparently Did Not Meet Specification Due to High Fines Content). (Photograph Taken on 7 April 1997)

Plate 42 – View North-West from Toe of Slope No. 6SE-C/FR13. Wall of Existing Reservoir Exposed Beneath Loose-Placed Rockfill Material. Crib Wall Footing Visible in Foreground. (Photograph Taken on 8 April 1997)

Plate 43 – View North-West from Toe of Slope No. 6SE-C/F14 Indicating New Drainage Works as Extension of Existing Culvert P043 to the Toe of Slope B. (Photograph Taken on 14 April 1998)

Plate 44 – View South-West Across Slope No. 6SE-C/F14 and the Natural Spur Separating This Slope From Slope No. 6SE-C/F15. Extension of Existing Culvert P043 Visible at Toe of Slope. (Photograph Taken on 14 April 1997)

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Plate 46 – View South-West Along Ramp G. Slope B Bulk Filling Operations in Progress on Left of Frame. (Photograph Taken on 23 October 1997)

Plate 47 – View South-East Across Slope A Filling Operations. Note Excavator Profiling DG Fill Layer on Left of Frame (Planar Face Formed). Note Also Upstand on Stepped Channel Below Catchpit Indicating Final 300 mm to 500 mm of Landscaping Fill Yet to be Placed. (Photograph Taken 24 October 1997)

Plate 48 – View East Towards Toe of Slope A from Berm Elevation at 64.5 mPD. Note Rock Fill in Foreground, Channel for 58 mPD Berm Formed and Landscaping Fill Over DG Fill on Lower Batter. (Photograph Taken on 29 October 1997)

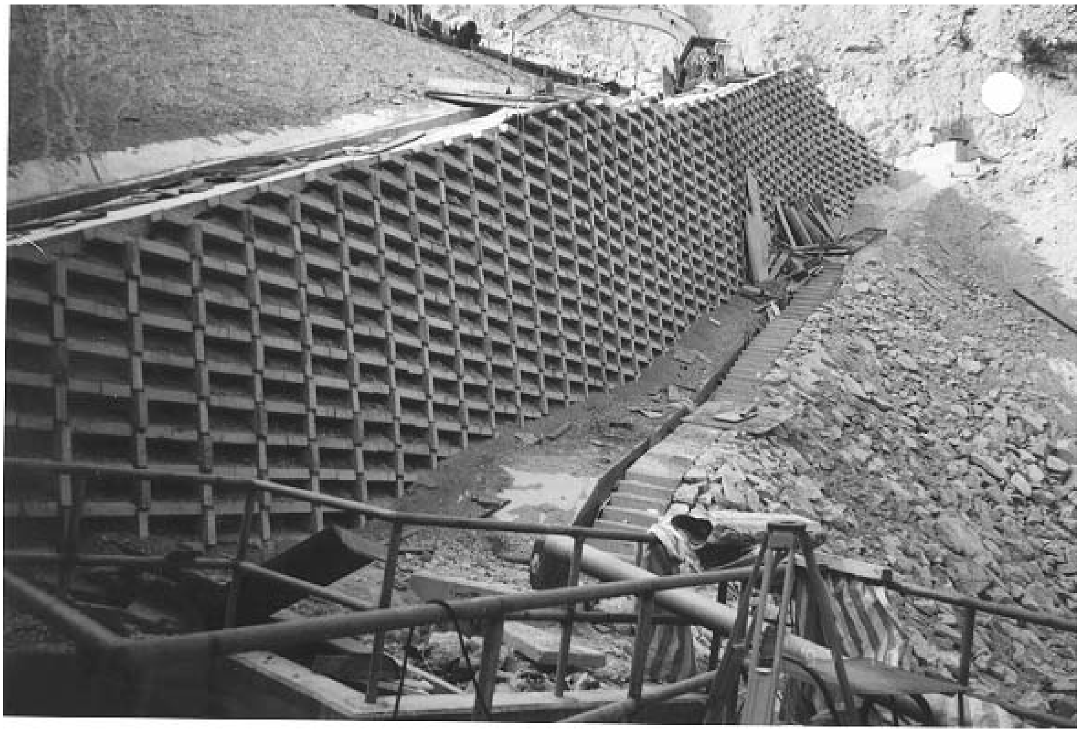

Plate 49 – View North Along Completed Crib Wall From New Reservoir Structure Note Rock Fill Placed Below Crib Wall. (Photograph Taken on 29 October 1997)

Plate 50 – View South Across Slope B Construction. Note Placement and Compaction of DG Fill Over Rock Fill in Centre of Frame on Batter Between 52.5 mPD and 45 mPD Berms (Small Roller Visible Behind Lower of two Large Excavators). Note Also Drainage Works in Progress Below Filling Operations, Upstand Remaining on Channel Structure on Left of Frame to Receive Landscaping Fill Layer and Planar Face on DG Fill. (Photograph Taken on 29 October 1997)

Plate 51 – View East Towards Toe of Slope A Indicating Placement of DG Fill Over Rock Fill Between 64.5 mPD and 58 mPD. Note Planar Face on DG Fill and Upstand at Toe of Batter. (Photograph Taken on 5 November 1997)

Plate 52 – View South Across Slope B indicating Placement of Landscaping Fill Over DG Layer Between 52.5 mPD and 45 mPD. Note Finished Slope Face Now Flush With Drainage Channel Above 45 mPD Berm. (Photograph Taken on 5 November 1997)

Plate 53 – View South Across Slope B Indicating Placement of DG Fill Over Rock Fill on Batter Between 60 mPD and 52.5 mPD. Note Shallow Depth of DG Fill, Apparently Minimal Benching in Rock Fill and Compaction of Fill by Vibrating Plate Compactor on Slope Face (Beneath Arm of Far Excavator) to Produce Planar Face. (Photograph Taken on 12 November 1997)

Plate 54 – View South Across Slope B Indicating Placement of Landscaping Fill Over DG Fill Between 60 mPD and 52.5 mPD. Note ‘Loose-Tipping’ Approach and Impressions on Upper Half of Slope Face Suggesting Tamping by The Backhole Bucket. (Photograph Taken on 9 November 1997)

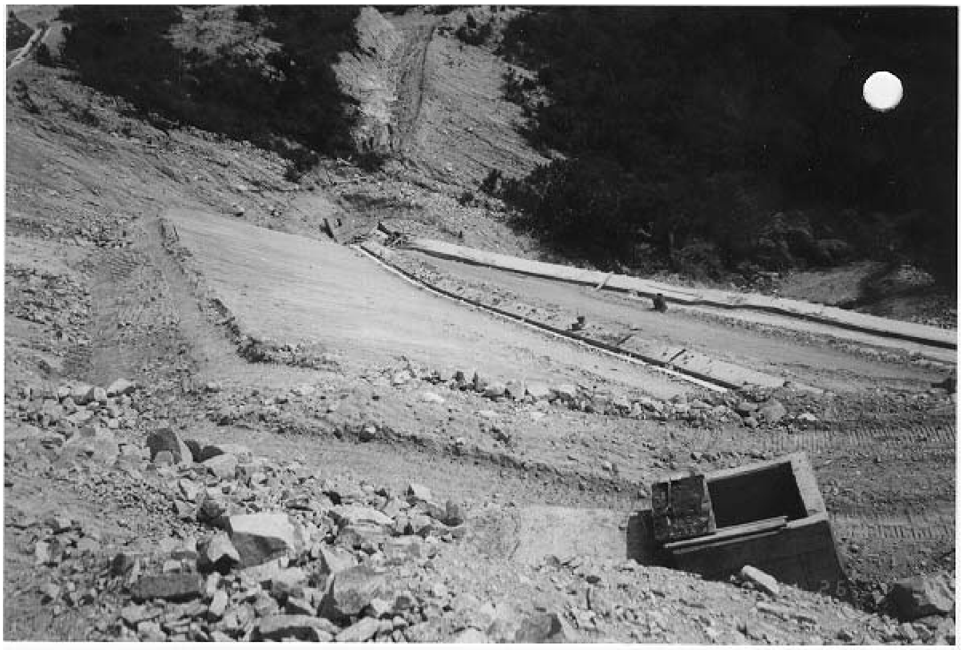

Plate 55 – View North-East Across Slope A From 64.50 mPD Berm Indicating Placement of DG Fill Over Rock Fill on Batter Between 64.50 mPD and 58 mPD, and Landscaping Fill on Batter Between 58 mPD and Crest of Crib Wall. (Photograph taken on 26 November 1997)

Plate 56 – View South Across Slope B Indicating Placement of DG Fill Over Rock Fill on Batter Between 67.5 mPD and 60 mPD. Note Planar Face to DG Fill and Apparently Minor Trimming of Slope Face Following Compaction. (Photograph Taken on 26 November 1997)

Plate 57 – View South-East Along Crest of Slope A. Landscaping Fill Placed on Upper Batter. Note Also that Berm Apron Slab (67.5 mPD) Does Not Extend to Edge of Berm. Landscaping Fill From Batter Below Trimmed Flush with Berm, Leaving Exposed Horizontal Surface. (Photograph Taken on 31 December 1997)

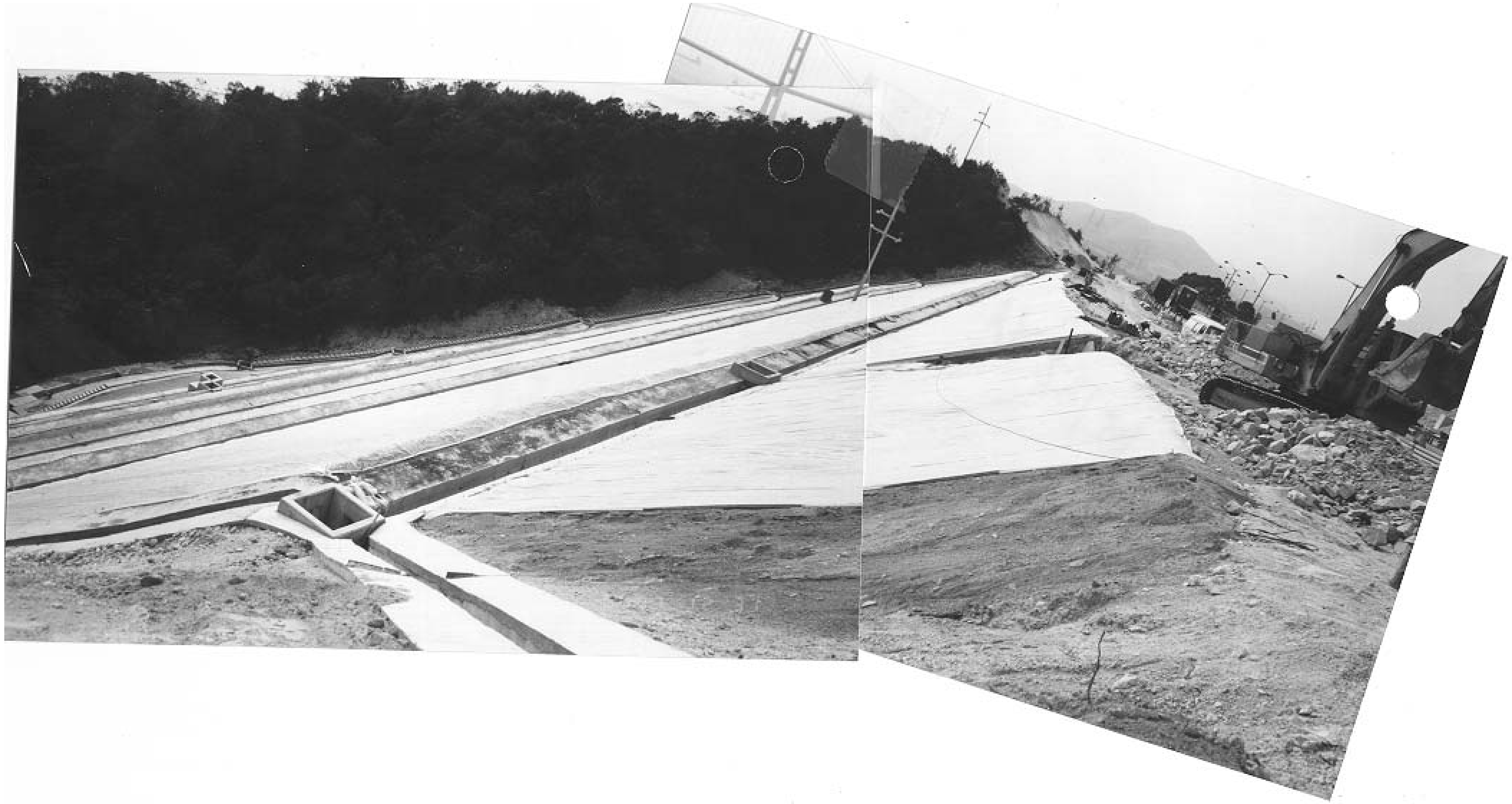

Plate 58 – View South Across Slope B Indicating Essentially Completed Slope Works. Drainage Works in Progress on Upper Batter. Similar Comment Regarding Exposed Horizontal Face of Landscaping Fill at Berm Level as for Plate 57. (Photograph Taken on 31 December 1997)

Plate 59 – General View South-West Along Ramp G Indicating State of Construction and Near Completion of Ramp G Fill Slopes. (Photograph Taken on 31 December 1997)

APPENDIX A

AERIAL PHOTOGRAPH INTERPRETATION REPORT

Aerial Photograph Interpretation (API) Report
Detailed Study of Failed Fill Slopes below Ramp G, Ting Kau Bridge

A.1 DETAILED OBSERVATIONS

The following comprise the detailed observations made from the photographs studied. Relevant photographs are referenced in Section 2.0 of this report.

YEAR	OBSERVATIONS
1963	Castle Peak Road is present. Terrain above Castle Peak Road is rugged with sharp ridges and valleys. Crests are relatively bare and vegetation is mainly shrubs. There is a small reservoir present (with associated platform and building) in the valley to the NW of Casam Beach. Site preparation works have started for the houses to the south of the reservoir and some of the lower houses are already present.
1964	Good quality photographs. Some small isolated erosion scars present on the hillsides above Castle Peak Road. Site preparation work ongoing for housing to the north of Casam Beach.
1972	Site is present on only one photograph, and only covers the area to the north of Casam Beach. Many boulders evident on the hillside.
1975	Very good quality photographs. Site formation works for Tuen Mun Road are well advanced. Cuttings for the east bound carriageway are almost completed and filling is ongoing for fill slope No. 6SE-C/F15. The stream course at the base of the fill has been culverted (concrete culvert evident at base) and there is a dozer and roller present on the slope.
1976 January	Slope No. 6SE-C/F14 and slope No. 6SE-C/F15 are mostly complete and are separated by a natural ridge line running NW-SE. A single berm is present midway up the slopes. The ridge line has been stripped of vegetation, and possibly used for temporary access between the two slopes, at around berm height. Both fill slopes have culverts in place underneath the fill (entry and exit culverts visible) and a stepped channel has been exposed in the upper reaches of slope No. 6SE-C/F14 underneath road formation level. The third fill slope, slope No. 6SE-C/FR13 has been formed and a curved drainage channel is present to the SW of the slope, which is separated from the fill slope by a natural spur. The drainage channel drains to the small reservoir (noted in the ‘63 photos) located at the toe of the slope. No berm is present on this slope. The culvert intake for slope No. 6SE-C/FR13 on the northern side of the road is obvious.
1976 (October)	Very good quality photos for detail of the fill slopes, which are now almost completed. Surface drainage has been completed in a chevron pattern though this has been destroyed by surface erosion in places. Asphalt pavement now present on the east bound carriageway of Tuen Mun Road from the SW up to the start of fill slope No. 6SE-C/FR13. Stepped drainage channels are present

along the edges of all fill slopes draining to natural stream courses below the fill slopes.

1977 East-bound carriageway of Tuen Mun Road has now been completely paved. Drainage which was destroyed (or covered) by erosion in 1976 has now been replaced. Slope surfaces are in good condition with no signs of distress. Vegetation on the slopes consists of grass with some small shrubs.

1978 Poor quality photographs. The east bound carriageway of Tuen Mun Road is now open to traffic. There appears to be some erosion on slope No. 6SE-C/F14 just above the berm between the centre and western stepped channels. The upper boundary of this apparent erosion is a straight line which may indicate vegetation stripping and repair, or possibly erosion protection sheets.

1979 Only one photograph. Most of slope No. 6SE-C/FR13 and slope No. 6SE-C/F14 are very light in colour which may indicate parched vegetation.

1981 Only one photograph. Vegetation re-established on the fill slopes which appear to be in good condition.

1982 Both carriageways of Tuen Mun Road now open to traffic. Lines of shrubs have been planted on slope No. 6SE-C/F14 on the upper face between the SW edge of the fill and the first stepped channel. No other changes.

1983 Only one photograph available showing the landslide site. No significant changes apparent.

1984 Poor quality photographs. No observable changes. Slopes appear to be in good condition.

1985 Very good quality photographs. Some small erosion scars present on the lower face of slope No. 6SE-C/F15 and just above the berm on slope No. 6SE-C/F14 (next to shrub line), otherwise the slopes are in good condition.

1986 Very good quality photographs. There appears to have been some slope maintenance works carried out, as all the drainage channels are clear and visible. There appears to be an area of erosion (or possibly vegetation removal) on the lower face of slope No. 6SE-C/F14 between the SW edge and the central stepped channel. The spur between slope No. 6SE-C/F14 and slope No. 6SE-C/F15 is still bare at around bench height. (as noted in ‘76 photographs and has remained unvegetated throughout).

1987 The bare patch noted on the fill slope No. 6SE-C/F14 in 1986 is still present and appears to have been cleared rather than eroded. No other changes apparent.

1988	No changes apparent. The bare section on the natural spur between slope No. 6SE-C/F14 and slope No. 6SE-C/F15 is still evident.
1989	Vegetation now re-established on slope No. 6SE-C/F14 at the bare sections noted in 1987. Slopes appear to be in good condition.
1990	The bare section noted in 1986/87 on the slope No. 6SE-C/F14 has returned. There is also a small erosion failure above the berm on the same slope, and some minor rill erosion on slope No. 6SE-C/F15 at around the same height. Slope No. 6SE-C/FR13 is well vegetated and appears to be in good condition.
1991	Vegetation has re-established itself on the bare sections as noted in the 1990 photographs. There are two small huts and a clearing on the natural ridge between slope No. 6SE-C/FR13 and slope No. 6SE-C/F14, and a footpath from them leading to the NE towards slope No. 6SE-C/FR13.
1992	Vegetation on the slopes has become much thicker. Electricity poles have been erected on both of the natural spurs between the fill slopes. No further changes.
1993 (July)	Slopes are heavily vegetated. No signs of instability and no further changes evident.
1993 (October)	Poor quality colour photographs. There are small erosion scars on the upper faces of slope No. 6SE-C/FR13 and slope No. 6SE-C/F15.
1994	Drainage channels on the slopes are clear which may indicate some vegetation clearance works. The two erosion patches noted in 1993 are still visible. Otherwise no further changes.
1995	Major changes are occurring on the hillside to the NW of Tuen Mun Road as part of the Tuen Mun Road widening, Route 3 CPS and the Ting Kau Bridge works. The cut slope immediately to the NE of slope No. 6SE-C/FR13 is being cut into, otherwise no other changes to the fill slopes. Possible small erosion feature on slope No. 6SE-C/FR15 adjacent to the spur at berm level. In addition, some slight erosion at the location of the bare area noted in 1987 on slope No. 6SE-C/FR13.
1996	Road widening works and slip road(Ramp G) to the NW at Tuen Mun Road are fairly well advanced. The upper surface of fill slope No. 6SE-C/FR13 has been extensively excavated, and there is a haul road leading from the cut to the NE, to the excavated slope. The curved drainage channel to the SW of this slope is exposed and it appears that the natural ridge between this channel and the fill slope has been removed. The slope appears to have been excavated down to culvert level in the lower section adjacent to the small reservoir. There appears to be erosion protection sheets on the central portion of the newly excavated back face, which may indicate possible instability. The excavation is circular in shape. Slope No. 6SE-C/F14 and slope

No. 6SE-C/F15 are as yet unaffected.

1997 (July)	Only slope No. 6SE-C/FR13 is visible on these low-level oblique photographs.
	The haul road noted in 1996 has been replaced and access to the slope face is now via a haul road leading from the NE corner of the slope off Tuen Mun Road. Excavators are present at the base of the slope and it appears that most of the upper surfaces of the fill slope have been removed. Loose material and gully erosion can be seen on the slope.
1997 (high level) (November)	Only single photographs available. Construction works now extending to the other two fill slopes. Appears to be only stripping works. The central and SW slopes have now become one slope, with the trimming and benching of the natural spur which separated them.

A.2 PHOTOGRAPHS

A full list of photographs used in the API study is provided below in Table 1:

Date	**Altitude**	Photographs	
01/1963	3900	Y08913	Y08914
02/1963	3900	Y08948	Y08949
12/1964	1800	Y11181	Y11182
06/1972	1800	1732	
12/1975	1500	11341	11342, 11343
01/1976	4000	13209	13210
10/1976	1500	15634	15635
12/1977	4000	20062	20063
12/1978	4000	24039	24040
07/1979	4000	26321	
11/1981	4000	40102	40103
07/1982	4000	43110	43111
01/1983	4000	47321	
10/1984	4000	56251	56252
10/1985	4000	67629	67630
11/1986	10,000	A07293	A07294
10/1987	4000	A10525	A10526
10/1988	4000	70304	70305
09/1989	4000	A18352	A18353
03/1990	4000	A20976	A20977
10/1991	4000	A27577	A27578
05/1992	4000	A31211	A31212
07/1993	4000	A35349	A35348
10/1993	5000	CN4788	CN4789
05/1994	5000	A38174	A38175
09/1995	3500	CN1115	CN1116
11/1996	4000	CN15773	CN15774
07/1997	900	74411-74413	74414-74417
11/1997	20000	CN19033	CN19064

Table 1 – List of Aerial Photographs