SECTION 2:
DETAILED STUDY OF THE LANDSLIDE AT SHING MUN TUNNEL ROAD ROUTE 5, TAI WAI ON 2 JULY 1997

Halcrow Asia Partnership Ltd

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FOREWORD

This report presents the findings of a detailed study of a landslide (GEO Incident No. MW97/7/66) which occurred on 2 July 1997 on a cut slope located adjacent to Shing Mun Tunnel Road, Route 5, Tai Wai. Landslide debris accumulated at the toe of the slope. No fatalities or injuries were reported. The landslide was located within a section of the cut slope that had exhibited previous instability and which shows evidence of possible deep-seated instability.

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

The report was prepared as part of the 1997 Landslip Investigation Consultancy (LIC) for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 68/96. This is one of a series of reports produced during the consultancy by Halcrow Asia Partnership Ltd (HAP). The report was written by Mr P Smith and reviewed by Dr R Moore and Dr S Hencher. The assistance of the GEO in the preparation of the report is gratefully acknowledged.

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1. INTRODUCTION

On 2 July 1997, a landslide (GEO Incident No. MW97/7/66) occurred on a cut slope located adjacent to Shing Mun Tunnel Road, Route 5, Tai Wai (Figure 1). Debris from the landslide accumulated at the toe of the slope adjacent to Shing Mun Tunnel Road. No fatalities or injuries were reported. The landslide was located within a section of the cut slope that had exhibited previous instability and which shows evidence of possible deeper-seated movement.

Following the landslide, Halcrow Asia Partnership Ltd (the 1997 Landslip Investigation Consultants) carried out a detailed study of the failure including the observed signs of distress for the Geotechnical Engineering Office (GEO), Civil Engineering Department (CED), under Agreement No. CE 68/96. This is one of a series of reports produced during the consultancy by Halcrow Asia Partnership Ltd (HAP).

The key objectives of the detailed study were to document the facts about the landslide, present relevant background information and establish the probable causes of the landslide. The scope of the detailed 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 key tasks:

(a) a review of relevant documents and aerial photography relating to the history of the site,

(b) analysis of rainfall records,

(c) interviews with persons involved with the landslide,

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

(e) diagnosis of the probable causes of the landslide.

2. THE SITE

2.1 Site Description

The landslide occurred on the lower part of the western end of cut slope No. 7SW-D/C866, which is located on the northern side of Shing Mun Tunnel Road, Route 5, Tai Wai (Figure 1).

The southwest-facing cut slope is about 220 m long and has a maximum height of about 70 m. At the failed section, the slope comprises nine batters and eight berms. The landslide affected the lowest batter (Figure 2). The batters are around 7.5 m high, inclined at about 34° to the horizontal and separated by 1 m-wide berms. U-channels are present on the berms and these drain from west to east. Approximately 50 m east of the landslide, a 225 mm- to 450 mm-wide stepped channel conveys water from the berm U-channels of the lower five batters to a catchpit at the toe of the slope. Catchpits are positioned at the intersection of the berm U-channels with the stepped channel. The cut slope does not have an impermeable surface cover and is extensively vegetated with trees and shrubs.

According to the SIMAR (Systematic Identification of Maintenance Responsibility of Slopes in the Territory) consultancy, the Highways Department (HyD) is responsible for the cut slope.

2.2 Site History

The history of the site was established from a review of documented information (Table 1) and aerial photographs (Table 2).

In 1924, the aerial photographs show that the site was a vegetated natural hillside to the north of a tributary of the Shing Mun River (Figure 3). On these photographs two scarp features are visible on the lower hillside which probably indicate the presence of relict landslides. The southern-most scarp was about 30 m wide and extended upslope from the tributary at 20 mPD to about 70 mPD. On the northern part of the hillside a 40 m-wide depression was located downslope of an arcuate scarp with rock exposures at about 80 mPD. The arcuate scarp was located within the head of a pre-existing stream course.

Between 1924 and the commencement of construction of the Route 5 highway in 1986, the upper part of the natural hillside remained essentially unchanged. The lower hillside was locally cultivated and buildings erected during this period were subsequently abandoned prior to highway construction.

Slope formation works for this section of the Route 5 highway began in April 1986 and were substantially completed by the end of 1989. The highway, at about 55 mPD, was formed predominantly in soil by cutting into the natural hillside. The upper hillside was cut back, removing up to 15 m of in situ material to form cut slope No. 7SW-D/C866 (Figures 3 and 4). A series of 300 mm × 300 mm gravel trench drains (Appendix A, Figure A1) was constructed on the lower two batters (Figure 2) in the vicinity of the 1997 landslide, possibly to lower localised groundwater encountered during construction work. The drains were connected to the berm and toe U-channels. The slope batters were hydroseeded, trees were planted in 1990, and by 1995 the tree cover on the slope was well-established.

2.3 Previous Studies

2.3.1 Slope Registration

In 1992, the GEO initiated the consultancy agreement entitled "Systematic Inspection of Features in the Territory" (SIFT) which, inter alia, aims to identify features not registered in the 1977/78 Catalogue of Slopes and to update information on registered slopes based on studies of aerial photographs and limited site inspections. In 1992, the SIFT study assigned the cut slope to Class "C2", i.e. a slope "formed or substantially modified after 30.6.78".

In 1994, the GEO commenced the consultancy agreement entitled "Systematic Identification and Registration of Slopes in the Territory" (SIRST) to update the 1977/78 Catalogue of Slopes and to prepare the New Catalogue of Slopes. The SIRST project registered the cut slope as No. 7SW-D/C866. The SIRST consultant inspected the slope in October 1995 and reported slight signs of seepage and cracking of a concrete drainage apron at the toe of the slope but no signs of distress. No emergency follow-up action was considered necessary.

2.3.2 Design Submissions and Checking

The cut slope was designed by Maunsell Consultants Asia (MCA) for the New Territories Development Department, as part of the Route 5 Sha Tin Connection project (MCA, 1986). The design was submitted to the Geotechnical Control Office (GCO, renamed GEO in 1991) for checking in November 1986. The key events of the geotechnical design submission are presented in Table 3.

The geotechnical submission by MCA (1986) reported on the geology and groundwater conditions and presented a stability assessment of the cut slope. As part of the Route 5 project, MCA carried out a ground investigation along the route that included several boreholes in the proposed cut slope. Undisturbed (Mazier) samples, retrieved from these boreholes, were subjected to single and multi-stage triaxial testing to determine shear strength parameters. The parameters adopted for slope design were as follows:

Colluvium	$c' = 3$ kPa	$\phi' = 33°$,
Completely decomposed granite	$c' = 9$ kPa	$\phi' = 34°$, and
highly decomposed granite	$c' = 5$ kPa	$\phi' = 38°$.

Based on the monitoring of piezometers in the vicinity of the proposed soil cut slope, MCA stated in their report that the "recorded groundwater regime is high and is close to the proposed slope profile". MCA recommended monitoring of the groundwater table in this area and the installation of additional piezometers following cutting "to verify the draw down profile of (the) groundwater table".

Based on the above assumptions a stability assessment of the proposed cut slope was undertaken (Appendix B, Figure B1). The report stated that "For global stability at chainage 1560, the F.O.S. (factor of safety) corresponding to a 1 in 10 years and 1 in 1 000 years is 1.40 and 1.31 respectively".

The geological model adopted by MCA showed the proposed lower three batters were formed in highly decomposed granite with the upper batters within completely decomposed granite. A superficial layer of colluvium, up to 5 m thick, was shown in the three highest batters.

The maximum groundwater level used by MCA in the stability analysis (1 in 1 000 years rainfall) was about 4 m below ground level in the lower batter of the cut slope. According to the theoretical analysis, the critical slip surface extended from the sixth batter to the toe of the slope.

Since "Groundwater levels were monitored for the 1986 wet season up to July only", the GCO requested further groundwater monitoring information to verify the design assumptions. Piezometer monitoring data for the period July 1988 to August 1990 were submitted to GCO by MCA. GCO requested "proper graphical plots of all available groundwater monitoring records together with associated analyses and interpretation" (Table 3). This outstanding matter had not been resolved by the time the Route 5 Shatin Connection was opened to the public in April 1990.

Ten months after road opening, in February 1991, the Project Manager of the New Territories Development Department wrote to MCA and requested the outstanding submission for GCO (Table 3). HAP has been unable to locate any subsequent correspondence on the submission.

2.3.3 Slope Maintenance

The HyD has prepared maintenance inspection reports for the cut slope twice a year since July 1994 (Table 1). Routine maintenance, comprising clearance of drainage channels, general slope clearance and trimming of overgrown vegetation, was recommended following the inspections.

2.4 Subsurface Conditions

Sheet 7 of the Hong Kong Geological Survey 1: 20 000-scale Map Series (GCO, 1986) shows the cut slope to be formed in coarse-grained granite. The engineering geology map of the Geotechnical Area Studies Programme (GASP) for the Central New Territories (GCO, 1987) shows the natural hillside before the construction of cut slope No. 7SW-D/C866 to be affected by general instability associated predominantly with in situ terrain. The GASP Report II (GCO, 1987), produced for regional appraisal and outline and strategic planning purposes at a scale of 1: 20 000, designated the terrain in the area of the 1997 landslide as an area of "General Instability". In this case the term "General Instability" denotes "in-situ terrain where many failures and other evidence of instability occur" which "provides an indication of the inherent weakness of the terrain and/or the occurrence of unfavourable groundwater conditions."

The inferred geological cross-section through the landslide based on interpretation of borehole information, aerial photography and field examination is shown in Figure 4. A simplified hydrogeological cross-section is shown in Figure 5.

Prior to construction of the cut slope, two boreholes (DHA7 and DHA21) were drilled by Gammon (Hong Kong) Limited (1986) in the vicinity of the 1997 landslide (Figure 2). Borehole DHA7, located upslope of the landslide encountered "boulders" of granite generally within a dense to very dense, medium to coarse sand to 21.5 m depth, overlying completely to highly decomposed granite to 37.2 m above moderately to slightly decomposed granite defined as bedrock (Figure 4). Core recovery in the borehole between about 14.5 m and 17.5 m below ground level (bgl) within the "boulders" was only about 60%.

The upper 21.5 m in borehole DHA7 was interpreted on the borehole log as colluvium, although that seems unlikely given the lack of any surface expression of colluvium visible on aerial photographs. It is more likely that borehole DHA7 encountered a heterogeneous in situ weathered rock profile.

Borehole DHA21, located downslope of the landslide, encountered completely decomposed granite to a depth of 10.9 m overlying granite bedrock.

HAP carried out field inspections of the slope, and noted that the materials downslope of the tension crack (see Section 3.2) comprise predominantly completely decomposed granite with few exposures of moderately to slightly decomposed granite rock within a partial rock mass weathering PW 0/30 zone (Figure 2). Immediately upslope of the tension crack, rock exposures of moderately to slightly decomposed granite are present within completely decomposed granite of PW 30/50, which increases to PW 50/90 in the fourth batter. Above the fourth batter, the percentage of rock exposure again decreases (PW 0/30).

On the 1989 aerial photographs (Table 2) evidence of a seepage line obliquely crossing the second to fourth batters (Figure 3) may be seen. The western end of the seepage line fades out just above the first batter about 20 m east of the 1997 landslide. The seepage line is possibly associated with the presence of a dyke.

A review of borehole data indicates that, in the vicinity of the 1997 landslide, the depth of weathering appears to be more pronounced, with a maximum of about 20 m (Figure 4). The greater depth of weathering coincides approximately with the former head of the stream course associated with earlier landsliding in the natural terrain (Section 2.2 and Figure 3).

Following formation of the cut slope, piezometers P10 and P11 were installed in the fourth berm and at the toe of the slope respectively (Figures 2 and 5). Between 1988 and 1990 intermittent monitoring of piezometer P10 recorded a 5.8 m range in water level. The highest water level recorded was 10.3 m below ground level (bgl) on 5 August 1988 following a period of heavy rainfall and the lowest water level was 16.1 m bgl on 30 March 1990. Monitoring records for piezometer P11 were only available for April and August 1990 which were relatively dry periods. The records showed fairly consistent water levels around 5.3 m bgl. Observations following the 1997 landslide appear to indicate a much higher water level in the lower part of the cut slope (see Section 3.2).

2.5 Previous Landslides

On the 1924 aerial photographs two natural terrain landslides that predate 1924, are visible on the original hillside close to the 1997 landslide site (Figure 3). A further two smaller natural terrain landslides on the same hillside, to the south of the site, can be seen on the 1963 and 1976 aerial photographs. The GEO's Natural Terrain Landslide Inventory contains no records of landslides on the hillside under consideration.

Following cut slope formation a landslide (GEO Incident No. MW95/8/20) occurred on the lower batter at the western end of the cut slope on 14 August 1995 (Figure 2). The GEO incident report recorded that the failure involved "weathered rock" and affected the east-bound carriageway of Route 5, that it was shallow and about 24 m^3 in volume, and that infiltration was the probable cause of failure.

During a site inspection in March 1998, HAP noted an apparent failure scarp, which is covered with chunam. The scarp, which is situated at the toe of the second batter above GEO Incident No. MW97/7/66, is about 2 m high, 2 m wide and 0.5 m deep (Figure 2). The landslide occurred after the cut slope was completed in 1990 but the precise date of failure is not known.

3. THE LANDSLIDE

3.1 Time of the Landslide

HyD reported GEO Incident No. MW97/7/66 to the GEO on 7 July 1997. A HyD works supervisor first observed the failure before 17:00 hours on the afternoon of 2 July 1997. The precise time of failure is not known. A Landslip Warning had been issued by the GEO at 06:25 hours on 2 July 1997 and was cancelled at 08:40 hours on 5 July 1997.

3.2 Description of the Landslide

The GEO and HyD jointly inspected the landslide on 9 July 1997. The landslide was located on the lowermost batter of the cut slope (Figure 2). The Incident Report prepared by the GEO indicated that the failure affected the upper 4 m of the batter and was 5 m wide and 1 m deep and that the estimated volume of landslide debris was 10 m^3. A thin layer of debris covered the lower slope with further minor debris deposited on the verge of the east-bound carriageway of the Shing Mun Tunnel Road. Seepage from the base of the failure scar was also recorded during the inspection on 9 July 1997.

The morphology of the failure scar suggests that the minor landslide was a shallow slide. The travel angle of the landslide debris was about 32°, as measured from the crest of the failure scar to the limit of debris run-out. This is within the typical range of 30° to 40° for rain-induced landslides in Hong Kong (Wong & Ho, 1996). After the landslide, GEO advised HyD to carry out urgent repair works which comprised the provision of a drainage pipe to collect seepage from the base of the failure scar, and reinstatement of the failed area with no-fines concrete.

HAP carried out a number of inspections of the site between December 1997 and July 1998. The urgent repair works as specified above had been implemented. A steady seepage was noted from the drainage pipe installed in the failure scar during the urgent repair works.

Seepages from several small scarps, the toes of the lower two batters, and the trench drains in the vicinity of the failure were also noted (Figures 2 and 5).

On the lower batter, about 15 m west of the 1997 landslide, signs of distress in the form of an area (about 25 m^2) of minor bulging and slumping were noted (Figure 2).

During an inspection by HAP in March 1998, an arcuate open subvertical tension crack was identified on the second batter above the 1997 landslide and above the unreported failure in the second batter (Figure 2). The tension crack was 40 m long, 0.4 m wide and was open to a depth of at least 1.5 m. The weathered appearance of the tension crack and the presence of vegetation in it indicated that the feature had been present for some time. At the ends of the tension crack fresh soil scarps up to 0.1 m high and a fallen tree were observed, which suggest renewed movement and lateral extension of the crack. The central portion of the crack coincided with a steeply-dipping discontinuity within a localised exposure of moderately to slightly decomposed granite rock. Both ends of the crack had propagated through intact completely decomposed granite.

While inspecting the cut slope in March 1998, cracking of catchpits along a stepped channel was identified 50 m east of the 1997 landslide (Figure 2). In the lower two batters, to the east of the stepped channel, trees were displaced in a downslope direction.

3.3 Follow-up Actions

Following inspection by HAP and GEO in April 1998, HyD was informed of the presence of the tension crack. At the recommendation of the GEO, HyD subsequently implemented urgent repair works which comprised vegetation clearance, infilling of the tension crack with compacted soil cement and covering with tarpaulin, and the placement of an oil drum safety barrier at the toe of the slope. The drainage pipe installed at the base of the 1997 failure scar was extended to direct water flow into the U-channel at the toe of the slope.

In April 1998, HyD instructed its consultant to undertake an Engineer Inspection of cut slope No. 7SW-D/C866 through its "Roadside Slope Engineer Inspections" programme. The instability and tension crack reported by HAP, including minor cracking, in the lower two batters to the east of the landslide were identified during this inspection. The consultant who carried out the Engineer Inspection recommended "an urgent stability assessment". The Advisory Division of the GEO, at the request of HyD, was assessing the stability of the slope at the time of preparing this report.

4. RAINFALL

The nearest GEO automatic raingauge No. N02 is located at Shun Wo House, Wo Che Estate, about 2.1 km northeast of the landslide (Figure 1). The precise time of failure is not known but for the purpose of analysis, all rainfall recorded up to 17:00 hours on 2 July 1997 has been included by which time the landslide is known to have occurred. Figure 6 shows that 385.5 mm of rain fell during the 24 hours up to 17:00 hours on 2 July 1997.

The daily rainfall recorded between 1 June 1997 and 3 July 1997 and hourly rainfall between 29 June and 2 July 1997 are shown in Figures 6a and 6b respectively. Intermittent rainfall commenced at about 01:00 hours on 2 July 1997 before intensifying between 05:00 hours and 06:00 hours on that morning. The peak hourly rainfall of 124 mm was recorded between 05:25 hours and 06:25 hours (Table 4). Rainfall of lower intensity continued for the rest of the day.

Table 4 presents the estimated return periods for maximum rolling rainfall for selected durations based on historical rainfall data recorded at the Hong Kong Observatory (Lam & Leung, 1994). The 2-hour rainfall ending at 07:00 hours on 2 July 1997 was the most severe with a corresponding estimated return period of about 38 years. The 31-day rainfall ending at 17:00 hours on 2 July was also severe with an estimated return period of about 34 years.

The maximum rolling rainfall for the rainstorm has been compared with selected past severe rainstorms recorded at raingauge No. N02 since the raingauge was installed in 1982 (Figure 7). The maximum rolling rainfall for the rainstorm of 2 July 1997 for durations between 15 minutes and 50 hours exceeds those of previous rainstorms recorded at the raingauge.

5. PROBABLE CAUSES OF FAILURE

5.1 Diagnosis of Failure Trigger and Key Factors Causing the Instability

Although the precise time of the 1997 landslide is not known, the close correlation between the severe rainfall and reporting of the incident suggests that the failure was probably triggered by rainfall.

The presence of a significant tension crack as well as localised bulging of the slope surface and cracking of channel and catchpit suggests that material detachment (e.g. the 1977 landslide) and the various signs of distress could be surface expressions of the development of deeper-seated instability.

In the diagnosis of the causes of the instability, the contribution of three key factors need to be considered, namely, possible influence of the relict landslide, groundwater conditions, and possible presence of unusually weak material. These are discussed further in the following.

5.2 Possible Influence of Relict Landslide

A large relict landslide was interpreted from the 1924 aerial photographs (Section 2.2) as being approximately coincident in plan with the distressed area observed following the 1997 landslide incident. Such relict landsliding could adversely affect material strength and the groundwater regime. It is possible that the abrupt changes in the gradient of the natural hillside prior to cutting back, as deduced from old topographical maps, were the result of this landslide (Figure 4). However, it is important to note that the coincidence (in plan) of the tension crack with a relict landslide does not necessarily imply that the latter would have affected the present cut slope given that much material (up to 15 m) was removed during slope formation. The probable pre-existing natural topography (Figure 4) suggests that the geometry of the relict landslide is unlikely to have extended much below the surface of the two lowermost batters of the cut slope and hence relict landsliding probably did not have a direct and significant effect. Notwithstanding this, the occurrence of a large-scale relict landslide highlights potential instability problems at the site and could provide an indication of inherent weakness of the terrain and/or unfavourable groundwater conditions. Such destabilising factors could still operate, to a certain extent, and affect the present distressed slope.

5.3 Groundwater Conditions

The groundwater regime in the weathering profile, particularly where the material has been subjected to displacement and disturbance, is likely to be complicated and highly variable, with infiltration and subsurface seepage through preferential flow paths and open cracks and joints. No post-failure groundwater monitoring was carried out but it is possible that the groundwater conditions could have been altered by the instability and hence may not be representative of that prevailing prior to the instability. Although there is much uncertainty regarding the precise groundwater model, there is however much evidence of high groundwater conditions within the two lowermost batters of this part of the slope, viz. piezometer monitoring records during the design and construction phases, presence of trench drains, and seepage observations during the detailed landslide study. A review of the piezometer data obtained during cut slope formation indicates that the highest reading of piezometer P10 in August 1988 was about 5 m higher than that assumed for the design of the slope (see Figure 5).

5.4 Possible Presence of Unusually Weak Material

In the absence of a detailed post-failure ground investigation including laboratory testing, it is not possible to establish whether the presence of unusually weak material (e.g. persistent clay-infilled adversely oriented discontinuity) could have played a significant role in bringing about the instability and governing its geometry.

Preliminary theoretical stability analyses were carried out based on the same set of shear strength parameters ($c' = 9$ kPa and $\phi' = 34°$) adopted in the original design in 1986 and the assumption of groundwater conditions that are consistent with seepage observations. The analyses considered a potential slip surface extending from the tension crack just in front of the second berm to the toe of the slope. The results suggest that deep-seated instability within the deeply-weathered profile, given typical shear strength parameters and a high groundwater condition, is credible. This suggests that the instability may not necessarily be affected by unusually weak material. However, the possible presence and contribution of weak material would need to be further verified by means of ground investigation.

5.5 Discussion

It is postulated that the cause of the instability is primarily related to elevated groundwater pressures in response to severe rainfall, superimposed on an already high groundwater regime, which may be a result of convergent subsurface groundwater flow possibly associated with an old stream course.

The mode of instability consists of slope displacement with only localised slope detachments. This may be related to the relatively large-scale and deep-seated nature of the instability of a deeply-weathered profile, variability in piezometric response of the unstable soil mass, both spatially and temporally, and possible release of matrix and discontinuity pore water pressure as the ground mass dilates and joints open up upon slope displacement. The comparatively shallow inclination of the slope with a limited driving force may also have been a contributory factor to the relatively ductile response to date as opposed to a fast-moving uncontrolled failure with major slope detachment.

The ground deformation and instability occurred on a slope which had previously been subjected to a detailed stability assessment based on site-specific ground investigation and laboratory testing. Groundwater monitoring was carried out during construction but it would appear that the monitoring results were not interpreted to check the validity of the design groundwater assumptions, despite a request by the GEO. In this respect, the cut slope cannot be considered to have been designed and checked to current standards, as it has not been accepted by the GEO as being up to standard.

The detailed landslide study did not commence until early 1998 and the presence of the tension crack was not identified during the preliminary inspection made in July 1997. The timing of tension crack formation cannot be ascertained with confidence although its geometry in plan in relation to the location of 1997 failure would suggest that the tension crack may have been formed at the same time, in July 1997, during an intense rainstorm which was the most severe since slope formation for short to medium-duration rainfall (Figure 7).

6. CONCLUSIONS

It is concluded that the 1997 landslide was triggered by severe rainfall.

It is possible that the landslide together with the tension crack and signs of distress on the slope could be surface expressions of the development of deeper-seated instability. The cause of the instability is probably primarily related to elevated groundwater pressures in a deeply-weathered profile in response to severe rainfall, superimposed on an already high groundwater regime.

The cut slope was previously subjected to a detailed assessment based on site-specific ground investigation and laboratory testing. However, it would appear that the validity of the design assumptions with respect to groundwater levels was not verified and that the slope had not been accepted by the GEO as being up to current geotechnical standards.

7. REFERENCES

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LIST OF TABLES

Table 1 – Sources of Information

Source	Documents
Drainage Services Department	Existing Utility Information
Highways Department / New Territories East Region	(a) Maintenance Responsibility (b) As Built Drawings 62783/1101B, 62783/1102C (General Layout), 62783/1130C, 62783/1131D (Drainage and Sewerage Layout) (c) Routine maintenance records dated July 1994, April 1995, July 1995, February 1996, July 1996, March 1997, July 1997, August 1997, January 1998 and April 1998
GEO Mainland West Division	(a) Slope file GCMW 2/11/7SW-D/C292 (b) Correspondence file GCMd 2/B3/49 (c) Correspondence file GCMd 2/E2/97-1 Pt 3)
Geotechnical Information Unit (GIU) at the Civil Engineering Library	GIU Ref. 7496 (Gammon (Hong Kong) Limited, Route 5 – Revised Alignment Boreholes)
GEO Landslide Incident Report Database	Details of past landslides reported to GEO
GEO Planning Division Natural Terrain Landslide Inventory (NTLI)	(a) Aerial Photographs from 1924, 1963, 1975, 1976, 1977, 1978, 1980, 1987 to 1996 (b) Details of past natural terrain landslides
GEO Publications, Reports, Maps and Memoirs	(a) Sha Tin: Solid and superficial geology, Hong Kong Geological Survey map Series HGM 20, Sheet 7, 1: 20 000 scale

Table 2 – Summary of Aerial Photograph Interpretation (Sheet 1 of 2)

Year	Photo Ref.	Altitude	Observations (see also Figure 3)
1924	H51/14 & 15	-	Subject area is natural hillside lying to the north of a tributary of the Shing Mun River. The hillside is covered with scrub vegetation and falls from a ridge line at about 120 mPD to the tributary at about 20 mPD. Agricultural fields are present to the west of the tributary. The lower part of the hillside has been affected by slope instability. A pronounced landslide scar is present on the lower southern part of the hillside with a width of 30 m and upslope extent from 20 mPD to 70 mPD. On the northern part of the hillside an arcuate scarp feature is present at about 80 mPD beneath which a depression extends downslope to the tributary. The scarp feature is possibly the head of a relict landslide and is coincident with the recently observed tension crack.
1963	5350, 51	3 900 ft	Agricultural terraces are present along the lower hillside as well as buildings, several tracks cross the upper hillside. Along the arcuate scarp, identified above, rock exposures are present. Minor failure present within drainage line on southern part of hillside.
1975	11552, 53	3 000 ft	Agricultural terraces have been abandoned on the lower hillside. Further buildings are present towards the toe of the hillside. Minor agricultural terrace sited on the southern part of the crest line. Tree growth present in the central part of the hillside.
1976	16981, 92	4 000 ft	Expansion of agricultural terrace on the crest line. Hillside remains relatively unchanged. Minor landslide on lower southern part of hillside.
1977	20079, 80	4 000 ft	Hillside remains relatively unchanged.
1978	24070, 71	4 000 ft	Hillside remains relatively unchanged.
1980	31610, 11	6 000 ft	Hillside remains relatively unchanged.
1987	A09309, 10	4 000 ft	Majority of buildings on hillside appear abandoned.

Table 2 – Summary of Aerial Photograph Interpretation (Sheet 2 of 2)

Year	Photo Ref.	Altitude	Observations (see also Figure 3)
1988	A15399, 400	4 000 ft	Slope (7SW-D/C866) is under construction with the majority of the excavation complete except for the lower northern batters.
1989	A16148,49	4 000 ft	Slope excavation is nearing completion. Significant rock exposures are present on the fourth batter from the toe in the northern part of the slope. An oblique line of darker reflectance crosses the second to fourth batters, which may be a line of seepage and is possibly associated with the presence of an intrusive dyke.
1990	A23426,27	4 000 ft	Increasing tree cover on the slope particularly in the area of the 1997 landslide.
1991	A27120,21	4 000 ft	Increasing tree cover on the slope particularly in the area of the 1997 landslide.
1992	A31942,43	4 000 ft	
1993	A34663, 64	4 000 ft	
1994	A390086, 87	4 000 ft	
1995	A46132, 33	4 000 ft (?)	Increasing tree cover on the slope particularly in the area of the 1997 landslide. High reflectance patch on lower batter with previous tree cover. Possible repair works to slope following failure.
1996	CN14942, 43	4 000 ft	Increasing tree cover on the slope particularly in the area of the 1997 landslide.

Table 3 – Key Events of the Geotechnical Design Submission (Sheet 1 of 3)

Event	Reference/Date
Maunsell Consultants Asia (MCA) submitted the cut slope design, including slope stability assessments for checking by the Geotechnical Control Office, (GCO) through Sha Tin Development Office (STDO). The Geotechnical Submission is entitled "Sha Tin New Town, Stage 2, Route 5 – Sha Tin Connection, Phase 1, Ch. 1260 to 2050 (Volume 1 of 2)" and dated October 1986. In the submission, the section of slope where the 1997 landslide occurred falls within chainage 1520 to 1740.	MCA letter dated 29.10.1986 to STDO (Ref. SKY: MC: 62783/13)
GCO commented on the submission and noted that "the stability analysis indicates that all of the proposed slopes achieve the required F.O.S.". GCO also noted that "very extensive cutting and filling works have been proposed" and requested the consultant "to examine alternatives". GCO pointed out that "Groundwater levels were monitored for the 1986 wet season up to July only. The stability analyses are based on the maximum recorded groundwater levels plus allowance for the 1 in 10 years and 1 in 1000 years storm". Further monitoring results were requested ("when they become available") from "newly installed piezometers so as to verify the design assumptions".	Memo from CGE/ME of GCO to Project Manager, Sha Tin (PM/ST) of STDO dated 17.12.1986 (Ref. GCMd 2/B3/49)
In response to GCO comment (17 December 1986) in particular, on the extensive cutting, MCA replied that "extensive cutting is required" because of a "design constraint", and "all the proposed slopes can achieve the required F.O.S." MCA "therefore believe the present slope configurations are satisfactory. Other options would be expensive and were not worthwhile to be considered". In their letter, MCA also enclosed "the proposed piezometers to be installed under the captioned contract" and stated that "The results [of groundwater monitoring] will be submitted in due course".	MCA Letter to STDO dated 19.02.1987 (Ref. SKY:MC:62783/13)
On receiving MCA's letter dated 19 February 1987, GCO wrote to PM/ST saying that they had no further comments on the extensive cutting since MCA confirmed that "the proposed slope works between CH1260 and 2050 are the optimum alternative". GCO also requested PM/ST to "ensure that all the piezometers on site should be well protected and maintained for monitoring during construction" and went on the request that "results to be submitted should include analysis and interpretation".	Memo from CGE/ME of GCO to PM/ST of STDO dated 24.03.1987 (Ref. GCMd 2/B3/49)

Table 3 – Key Events of the Geotechnical Design Submission (Sheet 2 of 3)

Event	Reference/Date
Piezometer readings for the period between 11 November 1988 and 29 February 1989 were submitted by MCA to GCO on a number of occasions without analysis and interpretation.	MCA letters to CGE/ME of GCO dated 29.12.1988 (Ref. EP:LSM:62783/13), 22.11.1989 (Ref. AKWL: YHC:62783/13) and 02.03.1990 (Ref. AKWL: YHC:62783/13), etc.
GCO wrote to PM/North East N.T. (NENT) in April 1990 on the outstanding items of submission with regard to the groundwater monitoring. In particular, GCO requested "graphical plotting of the updated piezometer readings together with analyses and interpretations" and asked PM/NENT to "urge your consultant to make the outstanding submissions as soon as possible".	Memo from CGE/ME of GCO to PM/NENT of STDO dated 06.04.1990 (Ref. GCMd 2/B3/49 Pt. 2), copied to MCA.
In March and April 1990, MCA submitted to GCO a number of "charts showing the variation of water table at piezometers…."	MCA letters to CGE/ME of GCO dated 29.03.1990 and 09.04.1990 (Ref. AKWL: YHC:62783/13)
In response to GCO memo (6 April 1990), MCA wrote to PM/NENT that "Graphical plotting of piezometer readings have been sent to GCO" via our letter dated 29th March and 9th April 1990".	MCA Letter to GCO/ME of GCO dated 25.04.1990 (Ref. AKWL: YHC: 62783/13)
After reviewing the graphical plotting of the piezometer readings submitted by MCA, GCO replied to PM/NENT stating that "The design ground water level should also be included in the plotting to facilitate comparison".	Memo from CGE/ME of GCO to PM/NENT of STDO dated 09.05.1990 (Ref. GCMd 2/B3/49 Pt 2), copied to MCA
The Senior Resident Engineer (SRE) of STDO wrote to GCO in May 1990 stating that "Route 5 – Shatin Connection has been opened to Public in mid April 1990…..The piezometers are ready to be handed over to your department now". The SRE then called a site meeting and joint inspection for the hand over.	Memo from SRE/NENT of STDO to CGE/ME of GCO dated 09.05.1990 (Ref. RL:IL: AT: 62783/44/57)

Table 3 – Key Events of the Geotechnical Design Submission (Sheet 3 of 3)

Event	Reference/Date
GCO wrote to PM/NENT to advise that "GCO is not the office responsible for the maintenance of the captioned works and therefore not be able to take over the monitoring of the 34 nos. of piezometers". GCO then requested that "when the queries… in connection with groundwater levels are resolved and that the groundwater assumptions adopted in the design are verified and accepted by GCO, the consultant should be asked to critically assess the need for long term monitoring of the piezometers".	Memo from CGE/ME of GCO to PM/NENT of STDO dated 21.05.1990 (Ref. GCMd 2/B3/49), copied to SRE/NENT
In response to GCO memo (21 May 1990), MCA replied that "it is not necessary to undertake long term monitoring of groundwater level. However, in order to keep a complete groundwater record. We would continue the measurement of groundwater levels until August 1990".	Letter from MCA to PM/NENT of STDO dated 07.06.1990 (Ref. AKWL: YHC: 62783/13), copied to CGE/ME
In response to MCA letter (7 June 1990), GCO replied to PM/NENT in July 1990 that "Regarding the termination of groundwater monitoring after August 1990 …. I can only comment on this issue when all the updated and properly plotted graphs of groundwater monitoring records together with the associated analyses and interpretations… are available".	Memo from CGE/ME of GCO to PM/NENT of STDO, dated 06.07.1990 (Ref. GCMd 2/B3/49 Pt 2), copied to MCA
GCO wrote again to PM/NENT in January 1991 that "proper graphical plots of all available groundwater monitoring records together with associated analyses and interpretation are still outstanding". GCO urged further that "in view that the entire project is almost complete, please submit the outstanding details promptly so that I can comment on the groundwater monitoring records…..".	Memo from CGE/ME of GCO to PM/NENT of STDO dated 24.01.1991 (Ref. GCMd 2/B3/49 Pt. 2), copied to MCA
PM/NENT wrote to MCA in February 1991 referring to the GCO memo of January 1991 "Please provide the outstanding information to CGE direct, with copies to this office at your earliest convenience."	Letter from PM/NENT of STDO to MCA dated 04.02.1991 (Ref. in NEST 2/41/102), copied to CGE/ME

Table 4 – Maximum Rolling Rainfall Recorded at GEO Raingauge No. N02 for Selected Durations Preceding the 2 July 1997 Landslide and The Corresponding Estimated Return Periods

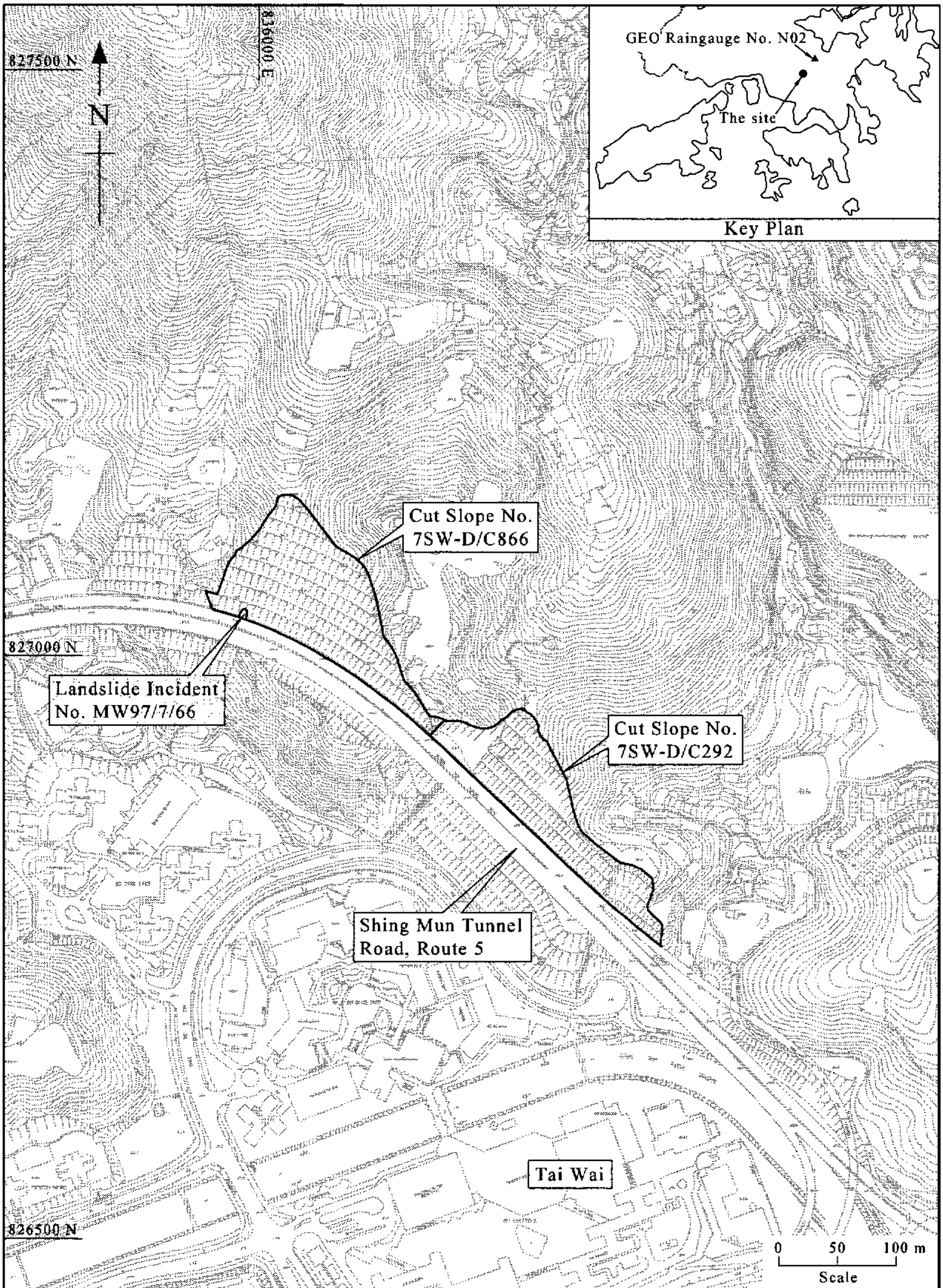
Duration	Maximum Rolling Rainfall (mm)	End of Period	Estimated Return Period (Years)
5 minutes	12.5	06:15 hours on 2 July 1997	2
15 minutes	34.5	06:15 hours on 2 July 1997	5
1 hour	124	06:25 hours on 2 July 1997	32
2 hours	183.5	07:00 hours on 2 July 1997	38
4 hours	229	07:00 hours on 2 July 1997	24
12 hours	366	15:00 hours on 2 July 1997	33
24 hours	385.5	17:00 hours on 2 July 1997	12
2 days	455	17:00 hours on 2 July 1997	12
4 days	468.5	17:00 hours on 2 July 1997	6
7 days	511.5	17:00 hours on 2 July 1997	6
15 days	630	17:00 hours on 2 July 1997	5
31 days	1146	17:00 hours on 2 July 1997	34

Notes:

(1) Return periods were derived from the Gumbel equation and data published in Table 3 of Lam and Leung (1994).

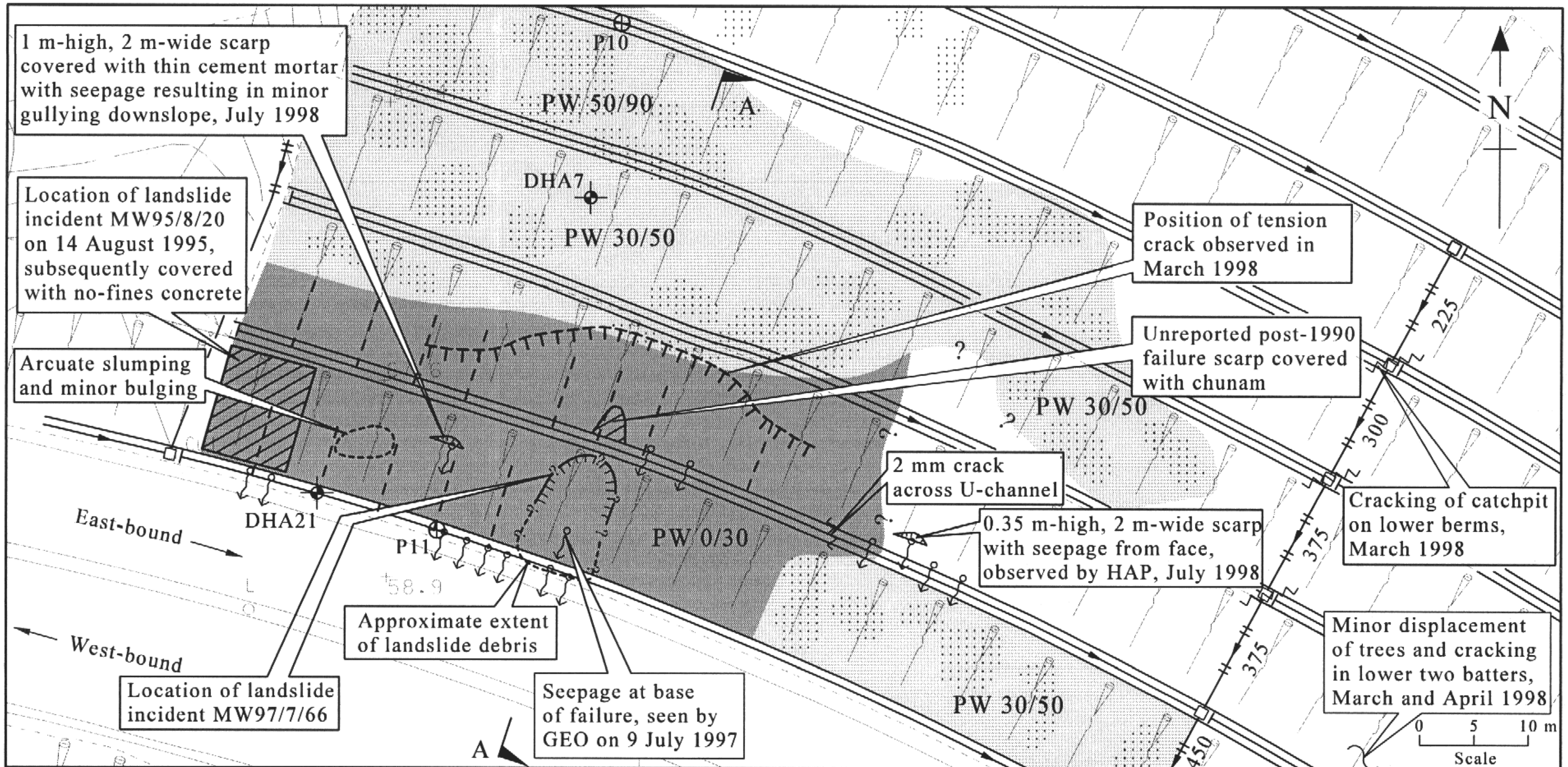
(2) Maximum rolling rainfall was calculated from 5-minute data for durations up to one hour and from hourly data for longer durations.

LIST OF FIGURES



Note : Base map is extracted from Survey Sheet Nos. 7SW-14B, 7SW-14D, 7SW-15A and 7SW-15C dated October 1992 (Original scale 1:1 000).

Figure 1 - Site Location Plan



Legend : ---- Gravel trench drain; Stepped channel; U-channel; Borehole by Gammon (Hong Kong Ltd) (1986); Rock exposure; Piezometer installed following formation; Completely decomposed granite (PW 0/30); Seepage; Moderately to slightly decomposed granite rock within completely decomposed granite (PW 30/50 to PW 50/90))

Notes :
(1) All U-channels and stepped channels are 225 mm wide unless otherwise stated
(2) Location and extent of rock exposures are approximate and have been estimated from site inspection and aerial photography.
(3) Boundaries of PW zones are indicative.
(4) Refer to Figure B1, Appendix B for as-built detail of trench drains.

Figure 2 - Site Plan

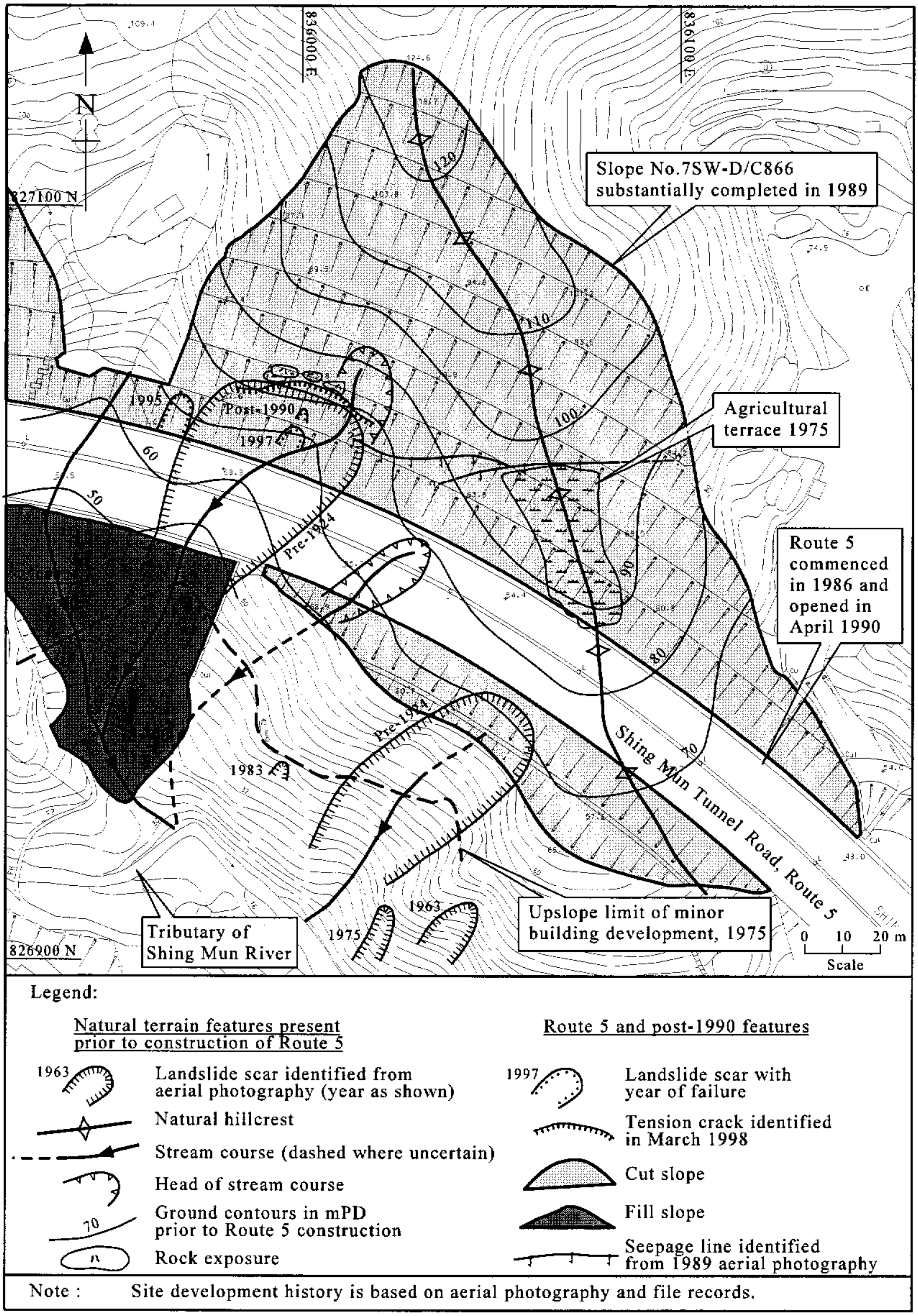
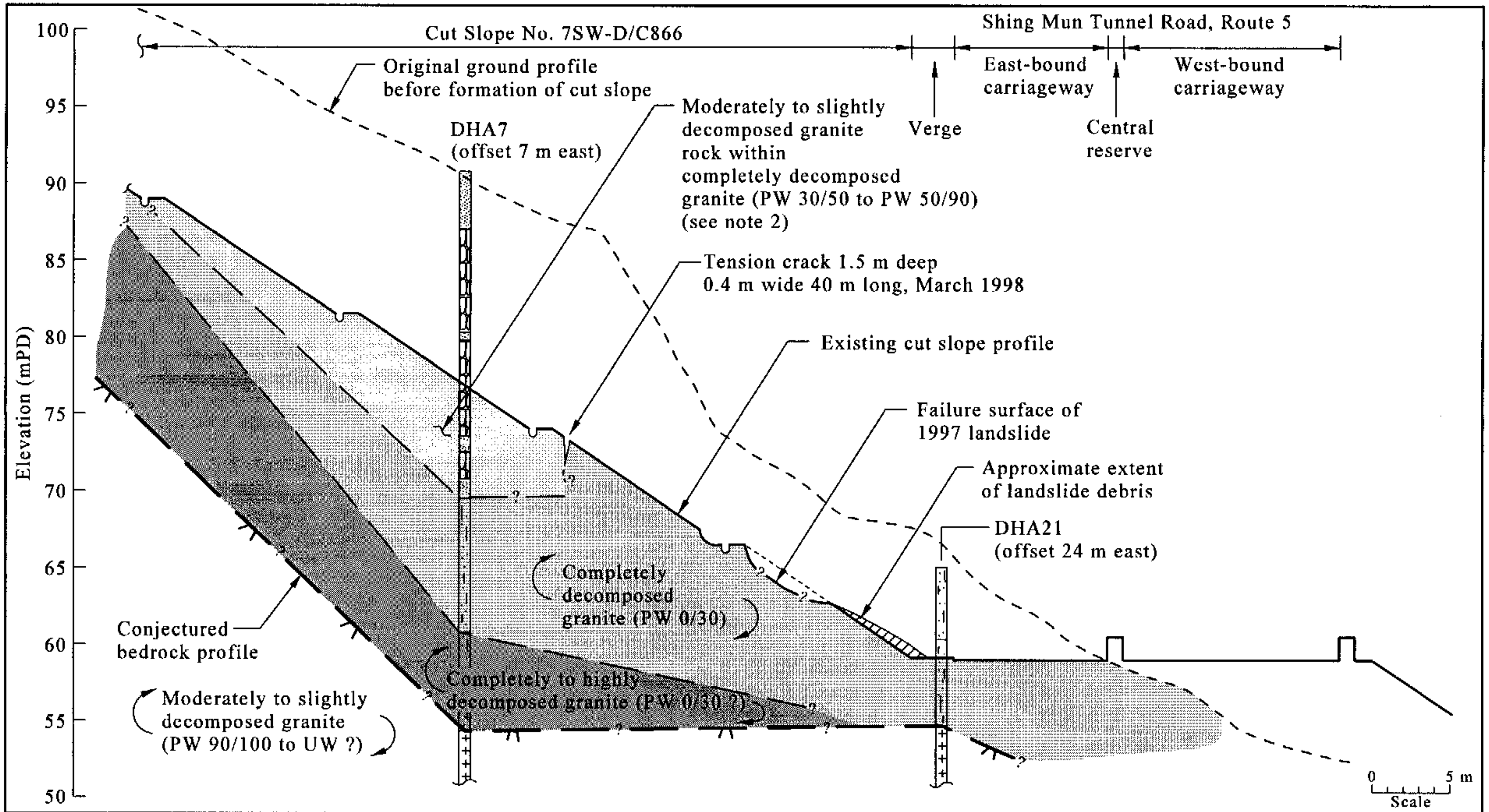


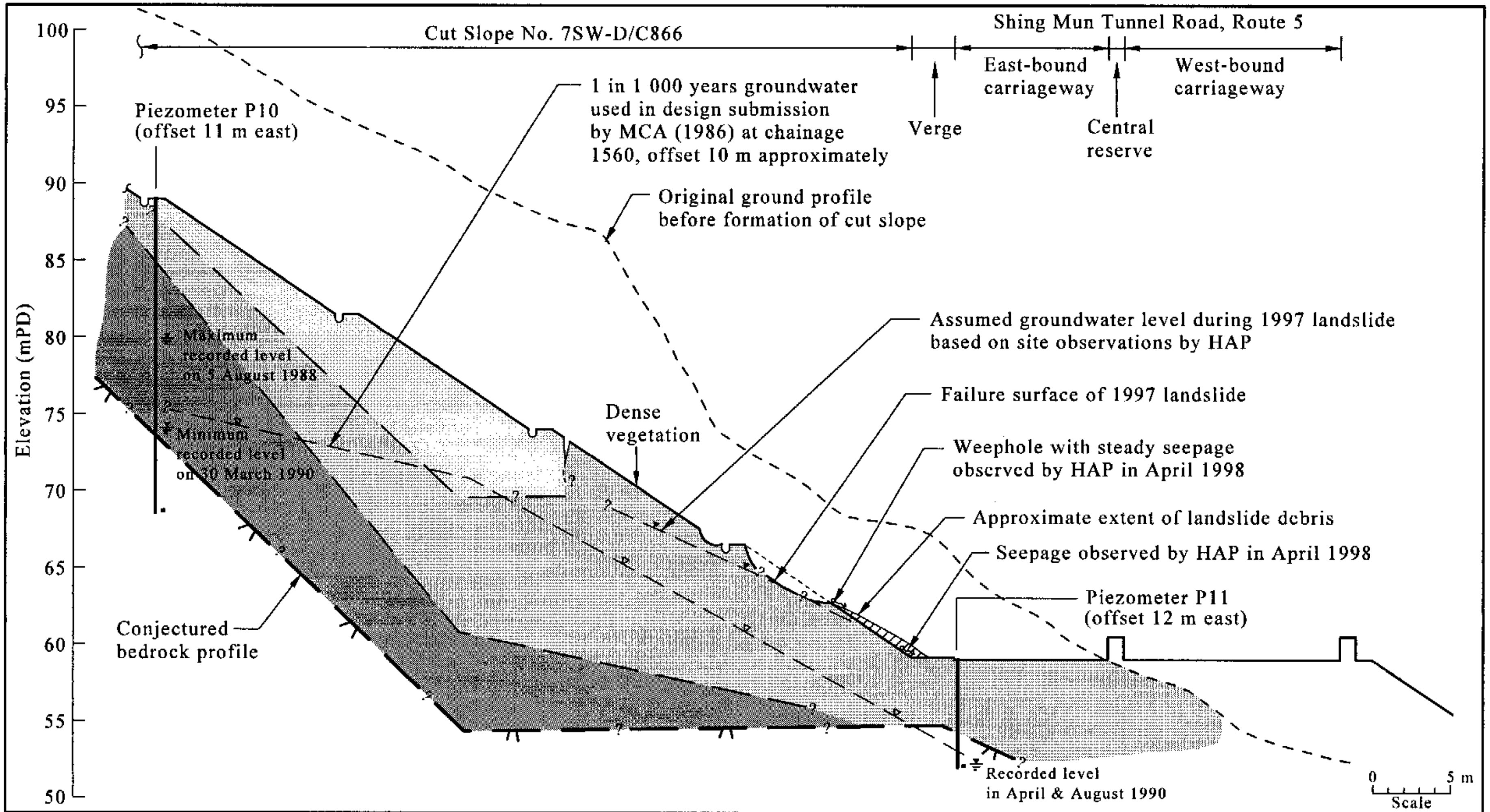
Figure 3 - Site Development History



Borehole Legend : Moderately to slightly decomposed granite "boulders" | Dense to very dense medium to coarse sand | Completely or completely to highly decomposed granite | Moderately or moderately to slightly decomposed granite

Notes : (1) For location of section see Figure 2.
(2) Interpreted as "colluvium" by Gammon (Hong Kong) Limited (1986) but probably heterogeneous in situ weathered rock.

Figure 4 - Inferred Geological Cross-section A - A through the Landslide

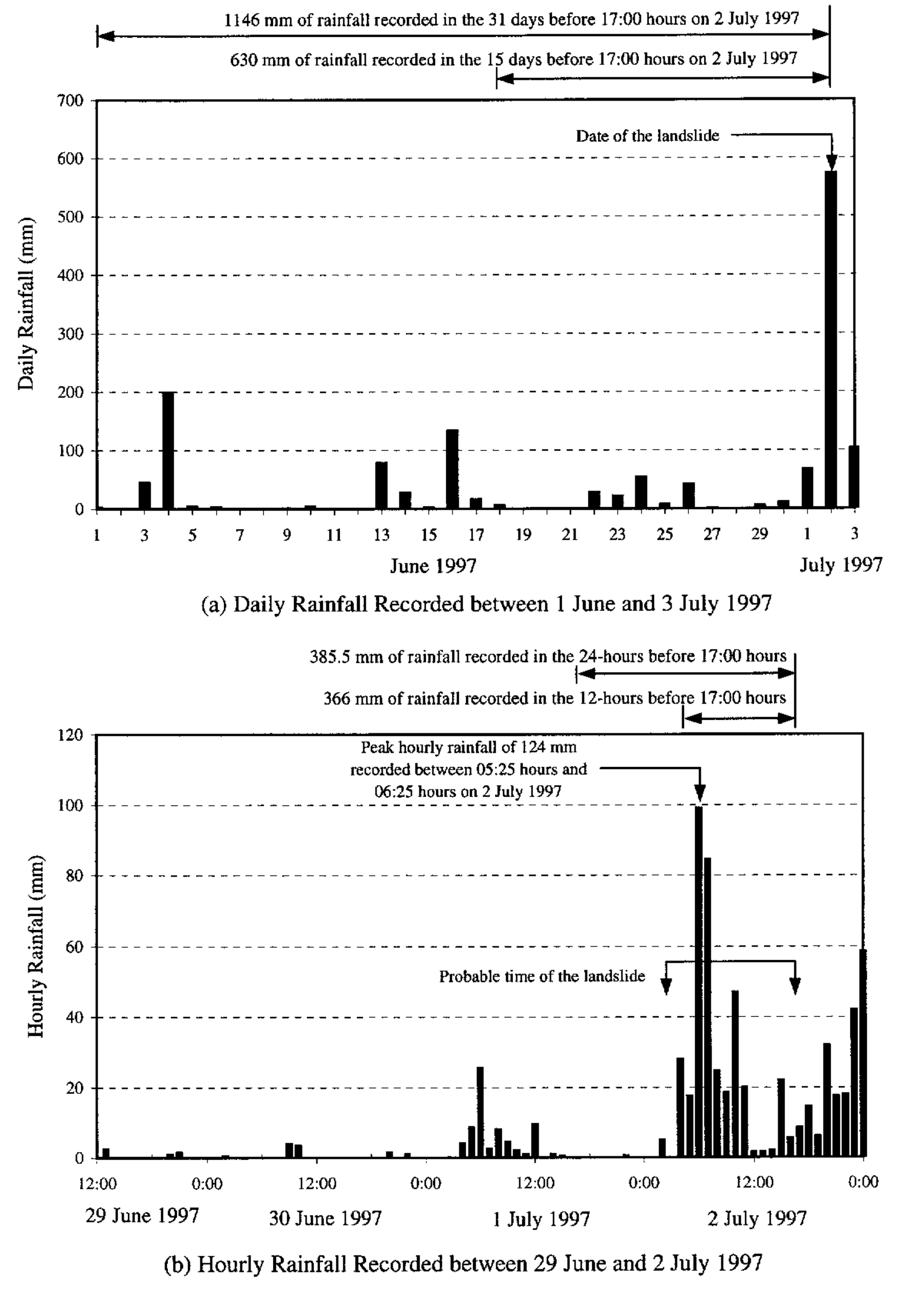


Legend :

- Moderately to slightly decomposed granite rock within completely decomposed granite (PW 30/50 to PW 50/90)
- Completely decomposed granite (PW 0/30)
- Completely to highly decomposed granite (PW 0/30 ?)

Note : For location of section see Figure 2.

Figure 5 - Hydrogeological Cross-section A - A through the Landslide



Note : Landslide reported to have occurred on 2 July 1997 but the precise time of failure is not known.

Figure 6 - Rainfall Records at GEO Raingauge No. N02

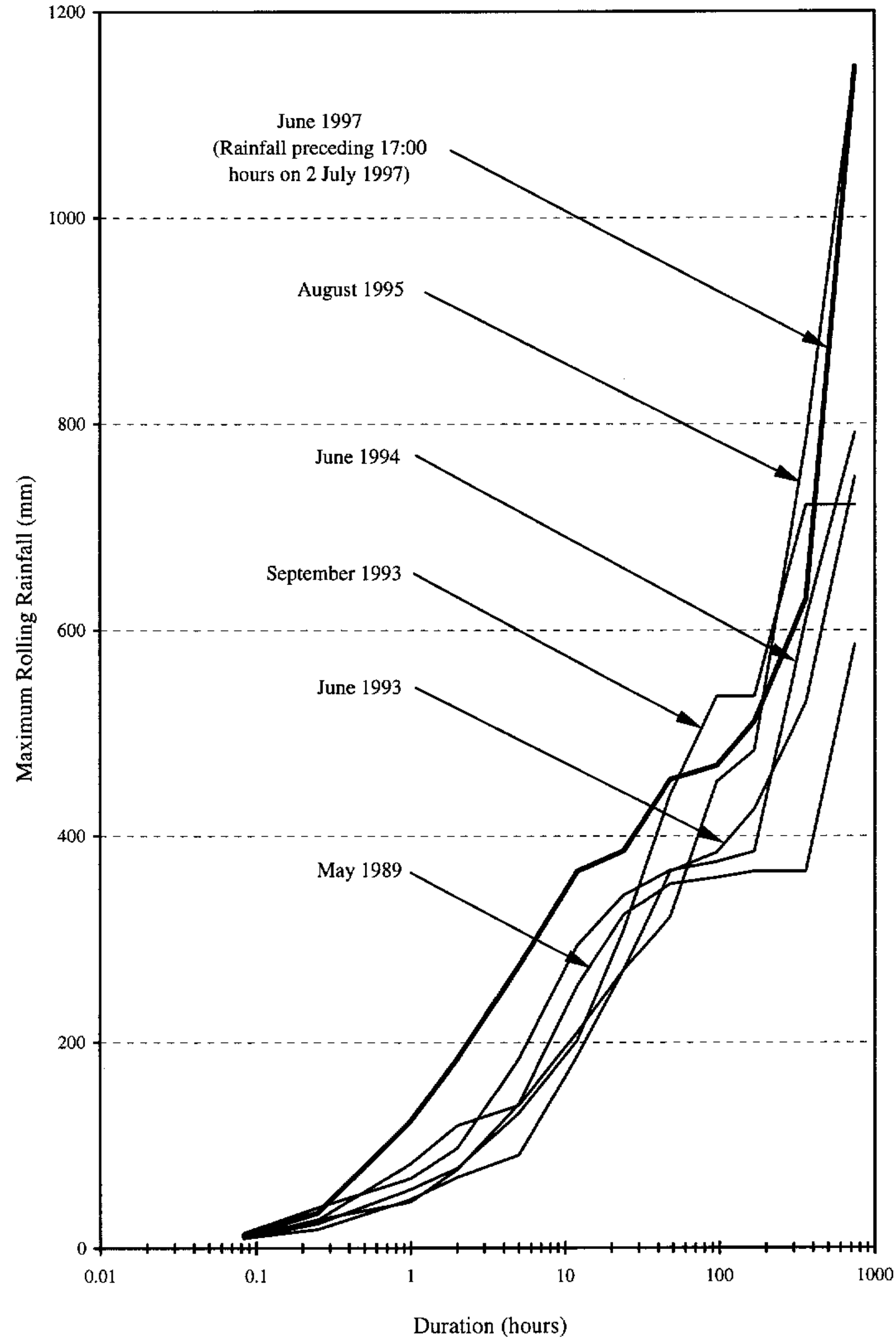


Figure 7 - Maximum Rolling Rainfall at GEO Raingauge No. N02 for Major Rainstorms

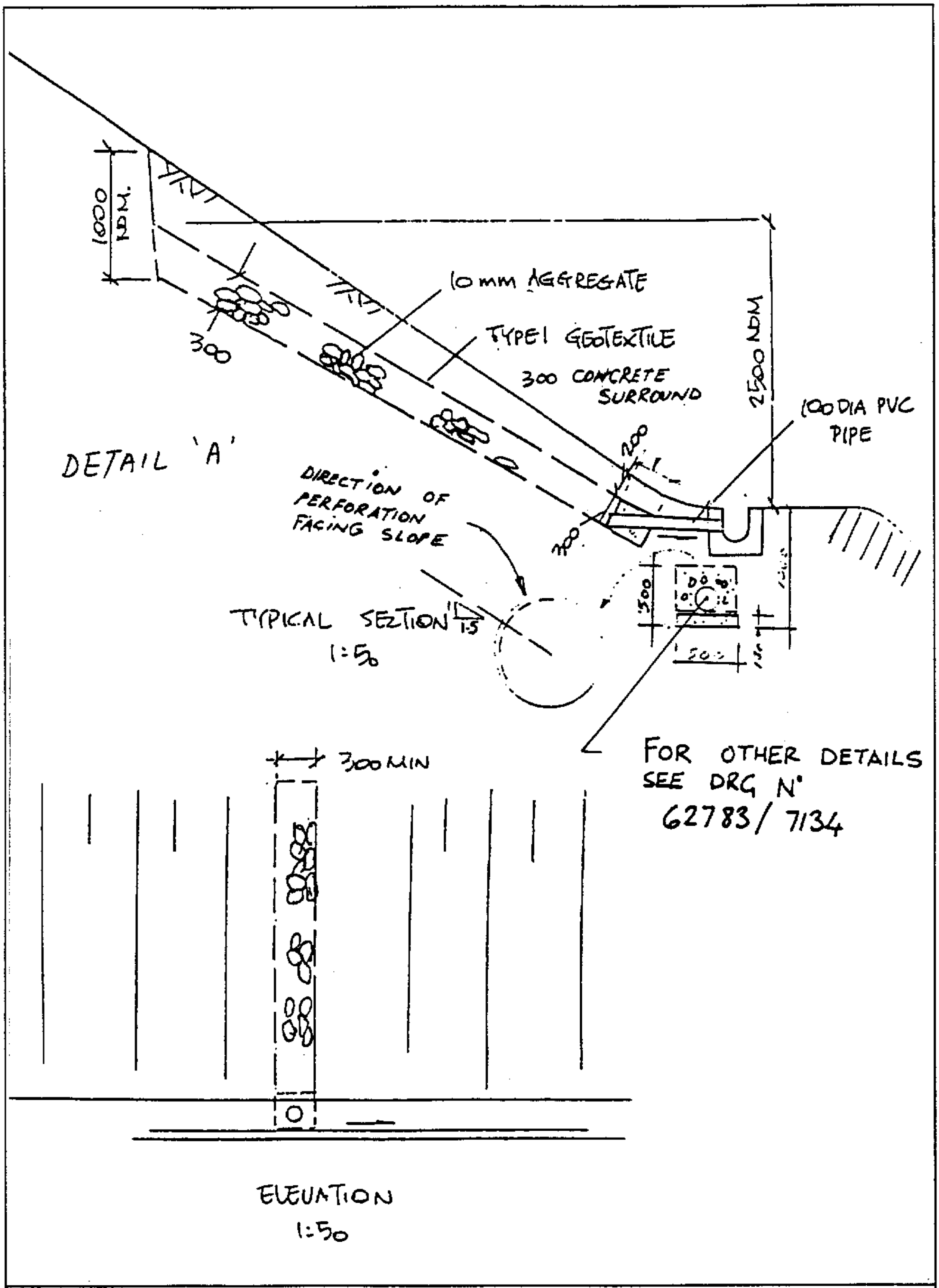
LIST OF PLATES

Plate 1 - View of Incident No. MW97/7/66 after Urgent Repair Works (Photograph Taken on 4 December 1997)

Plate 2 - Tension Crack Located on the Second Batter
(Photograph Taken on 15 September 1998)

APPENDIX A

TYPICAL DETAILS OF TRENCH DRAIN

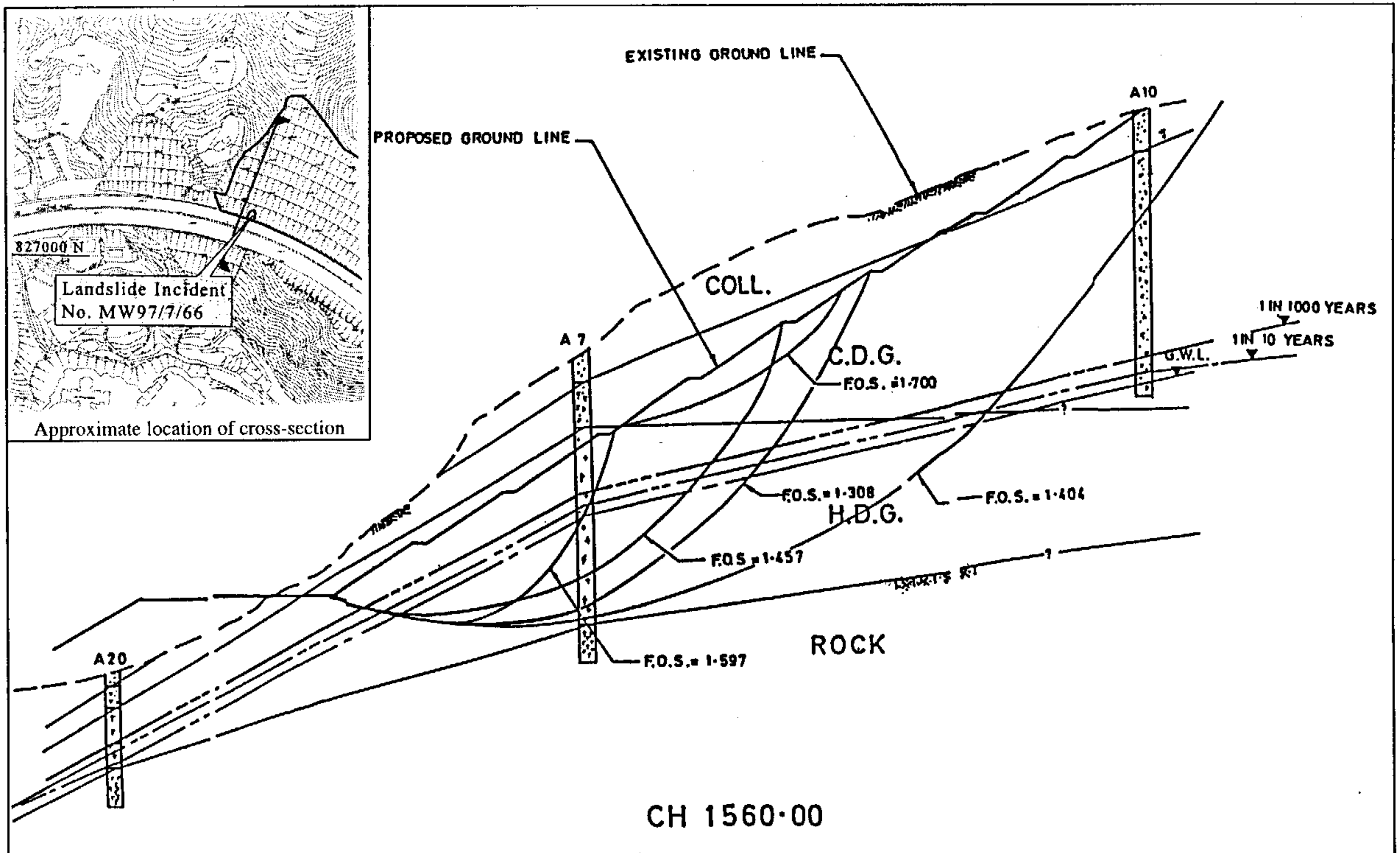


Note: Refer to Figure 2 for location of trench drains.

Figure A1 - As-Built Detail of Trench Drains Installed in Slope No. 7SW-D/C866

APPENDIX B

DESIGN SLOPE STABILITY SECTION



Note: For location of section see insert.

Figure B1 - Design Slope Stability Section X - X at Chainage 1560 (MCA, 1986)