REPORT ON THE ROCK SLOPE FAILURE AT CUT SLOPE 11NE-D/C7 ALONG SAU MAU PING ROAD ON 4 DECEMBER 1997

GEO REPORT No. 94

B.N. Leung, S.C. Leung & C.A.M. Franks

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
THE GOVERNMENT OF THE HONG KONG
SPECIAL ADMINISTRATIVE REGION

REPORT ON THE ROCK SLOPE FAILURE AT CUT SLOPE 11NE-D/C7 ALONG SAU MAU PING ROAD ON 4 DECEMBER 1997

GEO REPORT No. 94

B.N. Leung, S.C. Leung & C.A.M. Franks

This report was originally produced in December 1998
as GEO Landslide Study Report No. LSR 23/98

First published, September 1999

Prepared by:

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

This publication is available from:

Government Publications Centre,
Ground Floor, Low Block,
Queensway Government Offices,
66 Queensway,
Hong Kong.

Overseas orders should be placed with:

Publications Sales Section,
Information Services Department,
Room 402, 4th Floor, Murray Building,
Garden Road, Central,
Hong Kong.

Price in Hong Kong: HK$88
Price overseas: US$14.5 (including surface postage)

An additional bank charge of **HK$50** or **US$6.50** is required per cheque made in currencies other than Hong Kong dollars.

Cheques, bank drafts or money orders must be made payable to
The Government of the Hong Kong Special Administrative Region

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
September 1999

FOREWORD

This report documents the investigation of the failure of a rock slope along the Sau Mau Ping Road which happened on 4 December 1997. The rock slope was the lower part of a cut slope of reference 11NE-D/C7, the top portion of which had been removed under an ongoing Hong Kong Housing Authority site formation contract.

The investigation was carried out by Mr B N Leung with the assistance of Mr S C Leung. The aerial photograph interpretation and geological mapping of the failure site were carried out by Dr C A M Franks of the Planning Division. Colleagues from the Mainland East and Mines & Quarries Divisions, the Housing Department's Civil Engineering Section and their consultants Ove Arup & Partners provided useful information for the investigation. The report has been reviewed by them and Mr D J Howells (GGE/M). The Drafting Unit of the Special Projects Division assisted in the preparation of the tables, figures and plates of this report. All contributions are gratefully acknowledged.

P.L.R. Pang
Chief Geotechnical Engineer/Special Projects

ABSTRACT

On 4 December 1997, a rock slope along Sau Mau Ping Road failed. The rock slope was the lower part of a cut slope of reference 11NE-D/C7, the top portion of which had been removed under an ongoing Hong Kong Housing Authority site formation contract.

The failure completely destroyed a section of the protective fence along the toe of the slope. The failure debris blocked the entire four lanes of the Sau Mau Ping Road, covering 25 m of its length. While there was no injury, the road had to be closed for a period of 17 days, during which the failure debris was cleared, the slope inspected, the fence repaired and other works undertaken to ensure safety prior to re-opening of the road.

A comprehensive investigation of the slope failure was carried out. The investigation included a review of relevant documentary records, examination of aerial photographs, interviews with eye-witnesses to the failure, topographic survey, geological (including rock discontinuity) mapping, analysis of ground vibration monitoring data due to blasting and diagnosis of the failure.

The investigation concluded that the blasting which took place on 4.12.1997 close to the crest of the slope caused the slope failure. The location of the nearest blast area was assessed from the evidence collected. The closest distance between the blastholes in this area and the crest of the failed slope was found to be within 3 m. While this area was within the permitted blasting area limits, the amount of explosives used was found to have exceeded the permitted value stipulated in the Blasting Permit. Theoretical analysis has indicated that the blast-induced ground vibration alone could not have resulted in complete detachment of the largest rock block from the slope. However, with blasting carried out at this close distance from the slope, it is considered that the failure could have been triggered by the shock waves and gas pressures generated by the blast.

CONTENTS

1. INTRODUCTION

Around 2:20 p.m. on 4 December 1997, a rock slope along Sau Mau Ping Road failed (see Figure 1 and Plate 1). The rock slope was the lower part of a cut slope of reference 11NE-D/C7, the top portion of which had been removed under an ongoing Hong Kong Housing Authority (HKHA) site formation contract.

The failure happened a few seconds after the blasting which was carried out to the east of the slope under HKHA Contract No. "72 of 1994" entitled "Site Formation for Redevelopment of Sau Mau Ping Estate Phases 5 & 6 and Realignment of Sau Mau Ping Road". It completely destroyed a section of the 7.1 m high protective fence erected along the toe of the slope. The failure debris blocked the entire four lanes of the Sau Mau Ping Road, covering 25 m of its length. The metal hoarding on the far side of the road was punched through at several locations. A bus stop was located directly at the toe of the failed slope. This section of the Sau Mau Ping Road had been closed to the traffic and the area cleared of people at the time in accordance with the normal safety precautions adopted during blasting. Block 32 of the Sau Mau Ping Estate, which was directly opposite the failure and on the other side of the road, had been demolished at the time of the failure. There was no injury reported. However, the road had to be closed for a period of 17 days, during which the failure debris was cleared, the slope inspected, the fence repaired and other works undertaken to ensure safety prior to re-opening of the road.

The Geotechnical Engineering Office (GEO) of the Civil Engineering Department commenced an investigation of the failure in the afternoon of 4 December 1997. The investigation included the following key tasks:

(a) desk study, including review of relevant documentary records, examination of aerial photographs and old topographic maps of the site, and rainfall and earthquake data,

(b) interviews with witnesses to the failure and with other concerned persons,

(c) topographic surveys and detailed observations and measurements at the failure site,

(d) geological mapping,

(e) laboratory testing,

(f) collection and analysis of ground vibration monitoring data due to blasting, and

(g) theoretical stability analysis.

This report presents the findings of the investigation.

2. DESCRIPTION OF THE FAILURE

Figure 2 shows a plan of the failure and Figure 3 shows a cross section across the failure site.

The failed rock slope was about 25 m high with a berm at a level of about 6 to 7 m above the road. The failure mainly originated from the upper part of the slope. The volume of the failure was about 1 000 m^3. Although the failure debris was scattered along the bottom of the slope covering the road surface, soil and individual pieces of rock could be seen covering the entire slope face. The debris contained a number of very large and smaller angular blocks of rock of predominantly moderately to slightly decomposed medium-grained granite, completely decomposed locally. Blasting damage to the rock material and opening up of rock joints were generally observed on rock faces found in the vicinity of the failure site (see Plate 2).

The largest block of rock was about 150 m^3. As no signs of rolling were observed for this block and due to its non-circular shape, it is considered to have failed by sliding down the slope. The block rested partly against the lower unfailed portion of the slope and partly on the road, leaving a void underneath its base. Observations of the sub-vertical face of this rock block revealed extensive slickensiding on the joint infill material (Plate 3). This sub-vertical joint face had an apparent orientation of about 86°/190° and is considered to be part of joint set J1 (see Section 8.2).

Inspection of the failed slope indicated that it is formed in partially weathered fine- to medium-grained granite. The lateral boundaries of the failure are constrained by sub-vertical joints striking E-W, i.e. approximately perpendicular to the slope face. Sub-vertical joints, widely spaced and of very high persistence (mostly > 6 m and up to 10 m for some joints), which strike generally in the dip direction of the slope can be seen exposed at the top of the slope bounding the northern and southern ends of the failure (see Plates 4 and 5). Judging from the disposition of the largest block of rock which came down the slope, this block was bounded on its two sides by the sub-vertical joints observed.

The protective fence along the slope toe, which was built of vertical steel stanchions with horizontal steel connecting beams and wooden planks in between, failed to retain the debris.

According to the site supervisory staff, the failure did not take place instantaneously after the blasting. Instead it took a few seconds before the top of the slope started to move and noise of falling debris was heard, and the whole event came to a halt within a minute. Based on the comments from the site supervisory staff and the workers who were on Sau Mau Ping Road at the time of the blasting, it was the largest rock block that eventually broke through the protective fence, releasing smaller rock fragments caught behind the fence onto Sau Mau Ping Road.

The unfailed portions of the slope on both sides of the failure are lightly vegetated with shrubs and small trees. These areas dip generally at an angle of about 55° to 60° and locally at 70°, with a dip direction of about 260°.

Blast-induced rock fractures and joint openings could be seen on the excavated rock faces around the failure site (see Plate 2). Blasting which had taken place progressively around and towards the failure site might have resulted in progressive deterioration of the stability of the slopes. However, as the locations of the blast areas were not accurately surveyed, the blasting history of the areas around the failure site cannot be known with certainty and its effects on the stability of the failed slope cannot be quantified.

3. HISTORY OF THE SITE AND PROPOSED DEVELOPMENT

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

Plate 6 shows the appearance of the site about three months before the failure.

Table 1 summarizes all previous failures involving the cut slopes along Sau Mau Ping Road in the vicinity of the failure site, and Figure 4 shows their locations.

As part of the re-development of the Sau Mau Ping Estate and the re-alignment of the Sau Mau Ping Road by HKHA, the cut slope 11NE-D/C7 will be totally removed under the planned site formation works.

Figure 5 shows the proposed development plan around the site.

4. THE BLASTING

4.1 The Blast Assessment

A blast assessment report (AsiaConsult Pacific Ltd, 1994) was submitted to the GEO by the consultants to the HKHA in October 1994. The report recommended that "slope that has failed in the past to the north of the site along Sau Mau Ping Road" should be one of the blast sensitive receivers around the development site, and a maximum peak particle velocity (PPV) limit of 18 mm/s was recommended for this slope. However, the exact location of this slope was not given in the report. The report also identified that the gas main near the toe of the existing slopes along the Sau Mau Ping Road and the two temples at about 100 to 150 m to the north of the failure site were sensitive receivers, and maximum PPV limits of 10 mm/s and 25 mm/s were recommended for the former and the latter respectively. To mitigate the impact on the existing Sau Mau Ping Road and the housing blocks on the other side of the Road due to blasting, the report recommended that blasting should as far as possible be carried out with free faces orientated in a direction opposite to the Road. Moreover, a 'natural wall' for reducing the effects of the blasting to the surrounding should always be formed between the works site and the Road (see Figure 6).

GEO's comments on the blast assessment report and the responses received are given in Appendix A.

4.2 The Blasting Permits and Requirements under the Contract

Blasting Licence and Permit were first issued on 21 November 1995 (and subsequently renewed twice in November 1996 and in November 1997) by the Commissioner of Mines to the Contractor and the Blasting Subcontractor respectively for the HKHA Contract. As part of the permit conditions, the Contractor and Subcontractor were required to take all necessary safety precautions during blasting, including the erection of protective screens or cages and clearing and cordoning off areas which could pose a public safety risk before ignition. An area had been delineated, based on the recommendation of the HKHA's consultants, outside which blasting was not permitted. The limits of the permitted blasting area close to the failure site are shown in Figure 7. The maximum amount of explosives permitted per delay period had also been stipulated for sensitive receivers at various distances from the blasting area, including the gas main along the toe of the failed slope (see Table 2).

Under the HKHA Contract, a 20 m wide non-blasting zone measuring from the toe of the slopes along the Sau Mau Ping Road had been specified (see Figure 2) and a protective fence had been also been erected at the toe of the cut slopes along Sau Mau Ping Road. Figure 8 shows the typical details of the fence.

4.3 The Blasting Event

The general arrangement of the blast holes and the blasting details for the Blasting Areas 'A' and 'B', as provided by the Contractor, are given in Table 3 and Table 4 respectively. The amount of explosives per delay was 3 to 5 kg in Area 'A' and 21 to 22 kg in Area 'B'. Cartridge type explosives were used in the blasting.

The failure took place immediately after blasting in Areas 'A' and 'B' which were fired almost simultaneously. An assessment has been made of the locations of Areas 'A' and 'B'. For Area 'A', the assessment relied on photographs taken both before (see Plates 7 and 8) and after the incident, the interim record survey plan dated 29.11.97 provided by the Resident Engineer (see Figure 9), site inspections made before and after clearing up of the failure debris as well as interviews with the shotfirer. Plate 7 which was a progress photograph taken by the Resident Site Staff in the morning on the day of the failure (i.e. 4.12.97) shows a view of the relevant part of the site before the failure, and Plate 8 is an enlarged part of Plate 7 giving a close-up view of the crest and the cut face at the back of the failed slope. From Plate 8, holes can be seen to have been drilled close to the crest of the failed slope. It had subsequently been confirmed by the shotfirer that the Blasting Area 'A' was at the location of these drillholes. The assessed locations of Areas 'A' and 'B' are shown in Figure 7 (see also Figures 2, 3 and 9 for location of Blasting Area 'A').

The locations of the Blasting Areas 'A' and 'B' have been checked and these were confirmed to be within the permitted blasting area limits. However, the amount of explosives used is found to have exceeded that permitted based on the assessed distance from Blasting Area 'A' to the gas main located near the toe of the slope.

Vibration monitoring was carried out by the Contractor using seismographs at four locations in the vicinity of the site during the blasting event. The two seismographs (Seismographs No. 2 and No. 3, see Figure 7) nearest to the failed slope registered a ground vibration duration of about 1.8 seconds during the event, and Seismograph No.2, at a distance 80 m away from Blasting Area 'A', recorded a maximum peak particle velocity of 10.6 mm/s. Table 5 summarizes the peak particle velocities recorded by the seismographs. However, no detailed records of acceleration/velocity-time history were available, and no monitoring was carried out at the subject failed slope.

193 sets of vibration monitoring data were obtained at the site by the Contractor during the 3-months period before the failure. These data were plotted with peak particle velocity (PPV) versus scaled distance and also peak particle acceleration (PPA) versus scaled distance, and they are shown in Figures 10 and 11 respectively.

5. THE POSSIBLE TRIGGERS

The weather was fine at the time of the failure. Apart from 2 mm of rainfall recorded on 29 November 1997, no rainfall was recorded by the nearest rainguage no. K03 located at about 850 m to the west of the failure site (see Figure 1) during the week preceding the failure. No seepage or signs of seepage were observed on site during the inspection of the failure on 5 December 1997 and also during the subsequent inspections carried out in the following week.

The Hong Kong Observatory (HKO) was consulted on the possible occurrence of earthquakes in the vicinity of Hong Kong shortly before or at the time of the failure. HKO confirmed that no significant ground vibration was recorded by the eight seismic stations in Hong Kong.

Based on the above, the blasting which took place within the site shortly before the failure should be the only possible trigger for the failure.

6. FIELD STUDY

After the failure, an engineering geology field study was carried out to determine the rock mass and rock joint characteristics in the vicinity of the failure site. The study included general observations and discontinuity surveys in areas close to the crest and along the toe of the failed slope.

The joint wall compressive strengths of a number of discontinuities daylighting the rock face at the failure site were determined, and the rock joint surface roughness profiles of the sliding surfaces were also assessed.

Details of the engineering geological study have been reported by Franks (1998).

7. LABORATORY TESTING

A limited number of tests were carried out as part of this study to determine the range of shear strength of rock joints found at the site. The results are given and discussed together with an analysis of the failure in Appendix B.

8. ENGINEERING GEOLOGY OF THE FAILURE SITE

8.1 Geology

The site is underlain by fine- to medium-grained granite (Strange & Shaw, 1986). It is pinkish grey in the fresh state with an average grain size of close to 2 mm. The granite falls between the two main categories of fine- and medium-grained and is generally equigranular with shiny black clumps of biotite crystals up to 3 mm and scattered quartz. Occasional thin aplite dykes have been seen on site in association with hydrothermally altered zones, containing quartz and chlorite, along sub-vertical joints.

8.2 Rock Mass and Rock Joint Characteristics

Excavation in the crest area of the slope had removed much of the overlying completely to moderately decomposed rock within the partially weathered rock mass. Following the failure of the slope, inspection of the site revealed that much of the partially weathered rock mass is comprised of moderately to slightly decomposed rock with completely and highly decomposed rock forming infills. Hydrothermal alteration is evident with choritization and kaolinisation along many of the joints. Decomposition of the rock materials along the joint walls is limited to a maximum thickness of about 75 mm of completely decomposed granite but is more generally less than 25 mm. The decomposition products occasionally contain thin impersistent veins (2 to 3 mm) of kaolinitic clay, which locally may reduce the shear strength along the joint surface. However, it is considered that this reduction in shear strength is insignificant on a large scale.

Rock joint surveys were carried out at the site after the failure. A total of 45 joint orientation measurements were taken at the site, and the results are shown on the stereographic projection in Figure 12. Joint infills, roughness and weathering state were also noted. As can be seen from the stereographic projection and based on field observations, five major joint sets, J1 to J5, could be identified. Four of the joint sets, J1 to J4, are sub-vertical, and generally widely spaced, and the joint set J5 is very widely spaced and dipping at a low angle. Most of these joint sets are persistent. The characteristcs of these joint sets are summarised in Table 6. The major joint sets identified were generally consistent with those given in the geotechnical design report prepared for the site formation works by the consultants to the HKHA (OAP, 1994).

The most dominant joint sets at the failure site are joint sets J1, J4 and J5 with mean orientations of 88°/193°, 86°/321°, 29°/243° respectively. Joint sets J2 and J3 are less well developed and have mean orientations of 84°/239° and 72°/283° respectively. Although these two joint sets are less well developed, they could form a potential release surface to the back scarp of the landslide.

The low-angle dipping joints (of joint set J5) are considered to be the sheeting joints which dip generally in the direction towards the Sau Mau Ping Road. The largest rock blocks and a number of other large rock blocks in the failure debris are believed to have failed along joints which belong to this joint set.

After the clearing up of the failure debris from the failed slope, five persistent joint surfaces were exposed dipping at an average angle ranging from 16° to 29° in a direction (bearing in the range of 215° to 252°) towards the Sau Mau Ping Road. These joint surfaces, annotated as J5a to J5e (see Plate 9), belong to the joint set J5 and are considered to be the surfaces along which the failure took place. Joint surface J5c, which has an average dip angle of 25°, is considered to be the most likely failure surface along which the largest block of rock slid.

9. DIAGNOSIS OF THE FAILURE

Based on the assessed location of the Blasting Area 'A' as discussed in Section 4.3, the closest distance between the blastholes within this Area and the crest of the failed slope was found to be within 3 m (see Figures 2 and 3). At such a close distance, the surface of the failed slope facing west (i.e. towards the Sau Mau Ping Road) could have become another free face for the blast (in addition to the one at the back of the failed slope facing east), and the failure could have been triggered by the shock waves and gas pressures generated within the blastholes in the Blasting Area. No data on gas pressures were collected at this site. More importantly, the current state of knowledge does not permit any meaningful stability analysis of rock slopes subjected to near-field blast-induced gas pressures to be carried out. Therefore, no analysis of the failed slope subjected to gas pressures was undertaken. Nevertheless, an attempt had been made to estimate the cumulative displacement of the rock mass that moved down the slope along the shallow dipping failure surface (25° on average, see Section 8.2), assuming that it had been subjected to blast-induced ground vibration alone (see Appendix B). It was found that the cumulative displacement due to ground vibration alone could not have resulted in the complete detachment of the largest rock block from the slope.

The contribution of the vibration due to the blasting within the Blasting Area 'B' was negligible as compared with the blasting in Area 'A' (see Section B.3.2 in Appendix B).

10. CONCLUSIONS

From the investigation carried out, it was found that the blasting which took place within the site on 4.12.97 was the only possible trigger for the slope failure. The blasting carried out close to the crest of the slope at Blasting Area 'A' caused the failure, and the protective fence along the toe of the slope failed to retain the debris from reaching the Sau Mau Ping Road. The Blasting Area 'A' was confirmed to be within the permitted blasting area limits. However, the amount of explosives used was found to have exceeded the permitted value derived based on limiting the vibration at the gas main located near the toe of the slope. The closest distance between the blastholes within Area 'A' and the crest of the failed slope was assessed to be within 3 m. Theoretical analysis has indicated that the blast-induced ground vibration alone could not have resulted in complete detachment of the largest rock block from the slope. However, with the blasting at such a close distance, the slope failure could have been triggered by the shock waves and gas pressures generated by the blast.

11. REFERENCES

AsiaConsult Pacific Ltd (1994). Site Formation for Redevelopment of Sau Mau Ping Estate Phases 5 & 6 and Redevelopment of Sau Mau Ping Road - Blast Assessment Report. Report prepared for Ove Arup & Partners, 27 p. plus 18 Plates.

Franks, C. A. M. (1998). Engineering Geological Study of the Landslide at Slope No. 11 NE-D/C7, Sau Mau Ping Road on December 4, 1997. Advisory Report ADR 1/98, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, 66 p.

Ove Arup & Partners (1994). Site Formation for Redevelopment of Sau Mau Ping Estate Phases 5 & 6 and Realignment of Sau Mau Ping Road - Geotechnical Design Report. Report prepared for the Hong Kong Housing Authority, 20 p. plus 9 Figures, 6 Appendices and 11 Drawings.

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

LIST OF TABLES

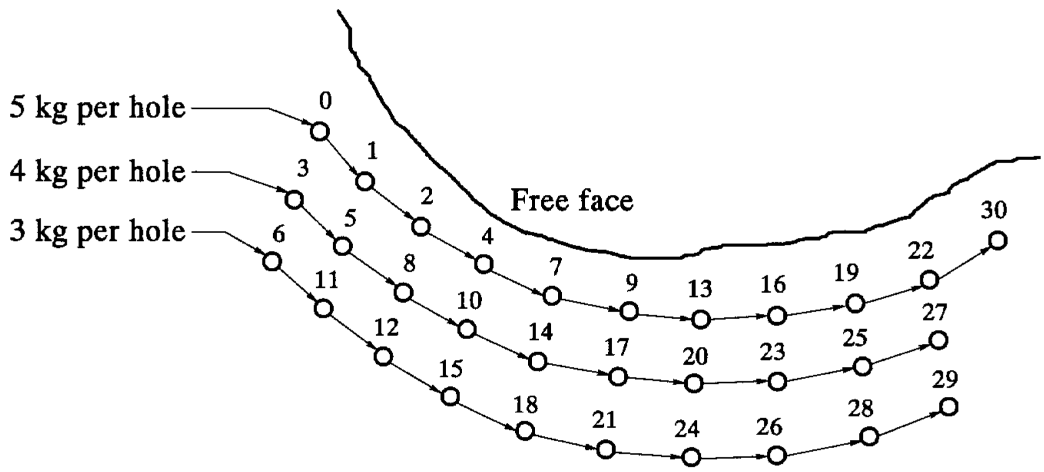
Table 1 - Summary of Previous Failures Involving Cut Slopes along Sau Mau Ping Road and Close to the Failure Site

Slope Ref.	Incident No.	Date of Failure	Failed Materials	Failure Volume (m^3)	Consequence
11NE-D/C4	(i) K89/5/3	2.5.89	debris with boulders	300 - 400	Road and pedestrian pavement blocked.
	(ii) K94/7/11	25.7.94	rock	150	A taxi damaged.
	(iii) None (Pre-GCO failure, details not known)	1964 (Exact date not known)	-	-	-
11NE-D/C5	(i) K84/5/14	30.5.84	soil	2	Pedestrian pavement and road affected.
	(ii) K3/17/83 (Details not known)	-	-	-	-
	(iii) K4/2/83 (Details not known)	-	-	-	-
	(iv) K3/12/83 (Details not known)	-	-	-	-
	(v) None (Pre-GCO failure, details not known)	1967 (Exact date not known)	-	-	-
11NE-D/C7	(i) K85/4/11	21.4.85	boulders	3	Pedestrian pavement affected.
	(ii) K89/5/9	2.5.89	soil	200	A vehicle damaged and road and pedestrian pavement blocked.
	(iii) K89/5/57	5.5.89	soil	very minor	Huts affected.
11NE-D/C8	(i) K84/8/3	5.8.84	fill	not known	Road and pedestrian pavement affected.
	(ii) None (Pre-GCO failure, details not known)	1972 (Exact date not known)	-	-	-
11NE-D/C32	(i) K6/13/83	18.6.83	soil	very minor	Huts affected.
	(ii) K94/7/12	23.7.94	weathered rock	40	Country park (?) affected.

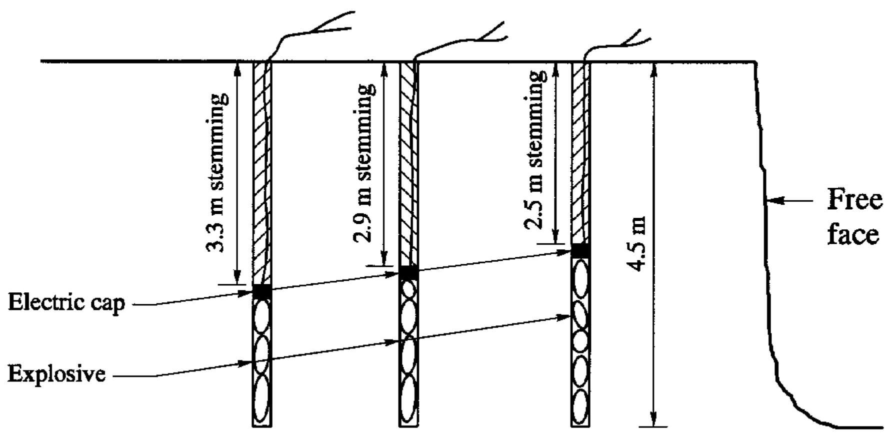
Table 2 - Permitted Maximum Amount of Explosives Per Delay Period

Distance (m)	Maximum Amount of Explosives Per Delay Period (kg)		
	Gas Mains	Water Tank and Pump House	Temples, Water Mains, Sau Mau Ping Estate, Structures, Installations, etc
0-20	Blasting not required		
20-25	0.43	0.67	1.96
25-30	0.68	1.04	3.06
30-35	0.97	1.50	4.40
35-40	1.33	2.04	5.99
40-45	1.73	2.67	7.82
45-50	2.19	3.37	9.90
50-55	2.71	4.16	12.23
55-60	3.28	5.04	14.79
60-65	3.90	6.00	17.60
65-70	4.58	7.04	20.66
70-75	5.31	8.16	23.96
75-80	6.09	9.37	27.51
80-85	6.93	10.66	31.30
85-90	7.82	12.04	35.33
90-95	8.77	13.49	39.61
95-100	9.77	15.04	44.13
100-105	10.83	16.66	48.90
105-115	11.94	18.37	53.91
115-125	14.32	22.03	64.67
125-135	16.92	26.03	76.41
135-145	19.74	30.36	89.12
145-155	22.77	35.03	102.82
155 and over	26.02	40.02	117.49
Note: This Table is reproduced from Blasting Permit No. A004687 issued by the Commissioner of Mines on 13.11.97.			

Table 3 - Details of Blasting Area 'A'



Blast Pattern
(N.T.S.)



Cross Section of Blast Hole
(N.T.S.)

BLAST DETAILS

Bench Height : 4.5 m

Burden : 2 m

Spacing : 2.2 m

Initiation System : Electric

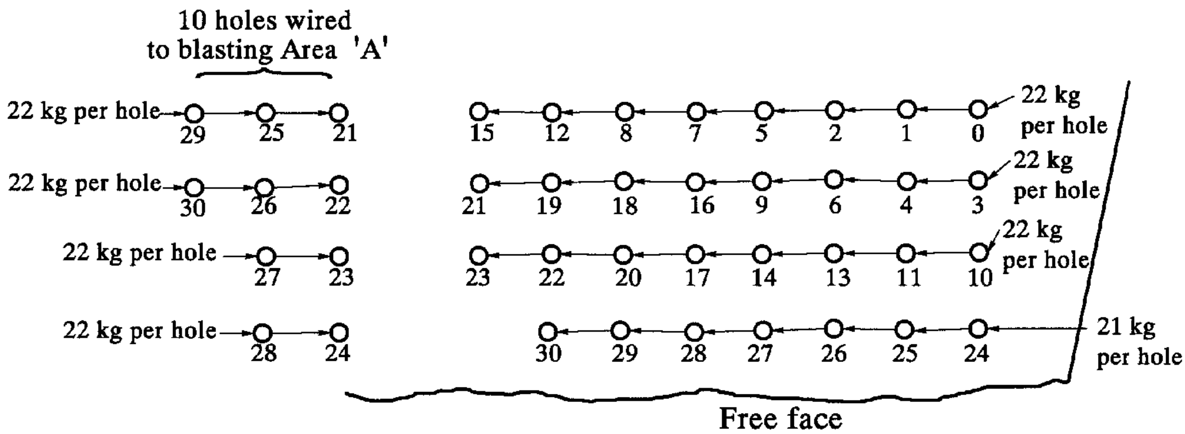
Powder Factor : 0.16 - 0.27 kg/m^3

Charge Weight / Delay : 3 to 5 kg

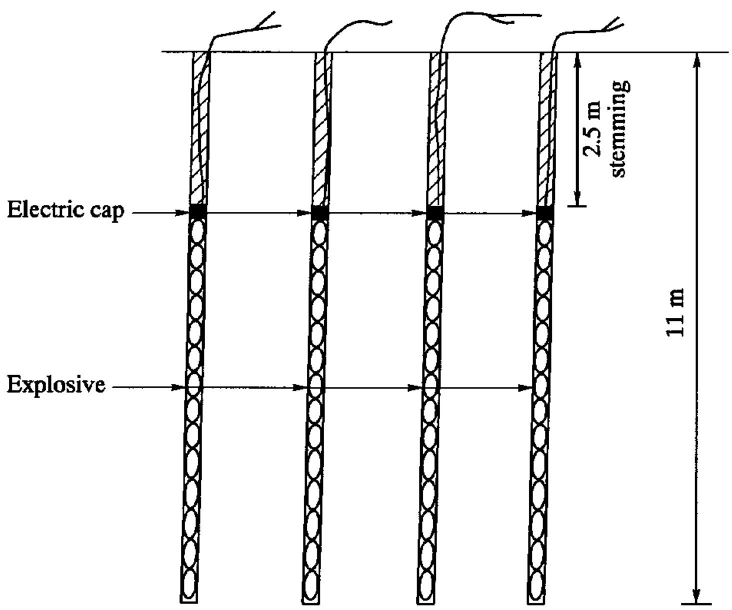
Delay between consecutive holes : 25 msec

Notes: (1) For location of Blasting Area 'A' , see Figure 7.
(2) The blasting pattern and details were provided by the Contractor.

Table 4 - Details of Blasting Area 'B'



<u>Blast Pattern</u>
(N.T.S.)



<u>Cross Section of Blast Hole</u>
(N.T.S.)

<u>BLAST DETAILS</u>

Bench Height : 10 m
Burden : 2.5 m
Spacing : 2.5 m
Initiation System : Electric
Power Factor : $0.3 kg/m^3$
Charge Weight / Delay : 21 kg to 22 kg
Delay between consecutive holes : 25 msec

Notes: (1) For location of Blasting Area 'B', see Figure 7.
(2) The blasting pattern and details were provided by the Contractor.

Table 5 - Peak Particle Velocity (PPV) Values Recorded

Seismograph No.	Peak Particle Velocity (PPV) Value Recorded (mm/s)
1	2.7
2	10.6
3	7.1
4	did not register any vibration

Notes: (1) For locations of the seismographs see Figure 7.
(2) The PPV values were recorded at the time of blasting by the Contractor.

Table 6 - Characteristics of Major Joint Sets

Set No.	Type	Dip/ Direction in degrees (Range)	Spacing (m)	Persistence (m)	Unevenness (Small Scale)	Waviness (Large Scale)	Weathering State and Infill	Joint Wall Compressive Strength (JCS) MPa	Remarks
1	Joint Side Release	88/193 (±3/±6)	0.6 to 8.0	Over 6	Smooth to rough	Planar to undulating	Stained, locally highly decomposed	31	Widespread occurrence
2	Joint	84/239 (±2/±2)	0.6 to 1.0	2 to 5	Smooth to rough, occasionally stepped	Planar to undulating	Stained, moderately to highly decomposed	52	Widespread occurrence
3	Joint	72/283 (±2/±1)	0.4	Over 3	Smooth to rough	Planar to undulating	Stained, locally highly decomposed	48	
4	Joint	86/321 (±3/±5)	0.2 to 0.3	Over 5	Smooth to rough	Planar to undulating	Stained, locally highly decomposed	54	
5	Joint/ Failure Plane of Large Blocks	29/243 (±4/±3)	1.5 to 4	Over 6	Smooth to rough, occasionally stepped	Planar to undulating	Stained, locally with thin kaolin and completely decomposed infill	56	Widespread occurrence steepens to the south of the site

LIST OF FIGURES

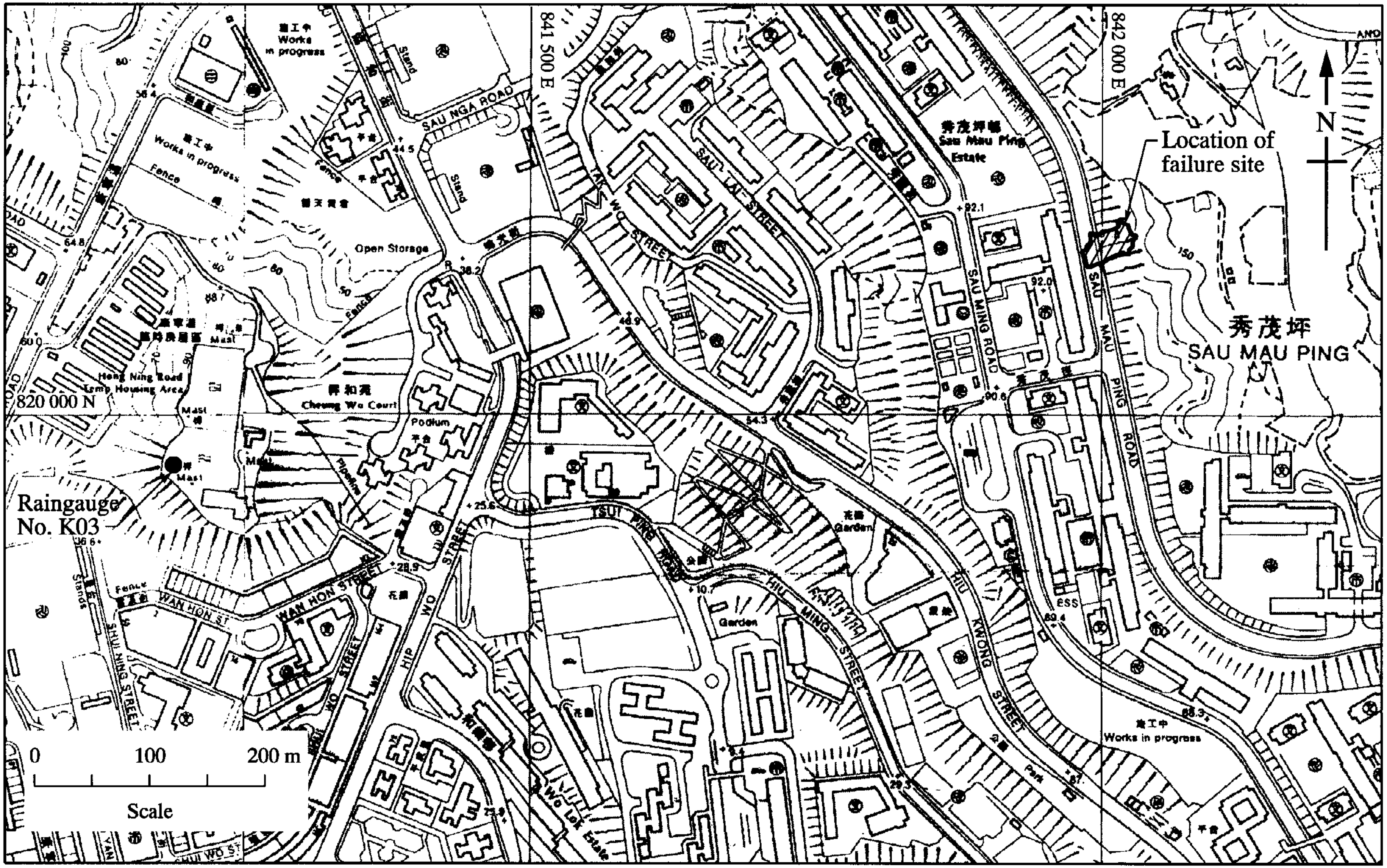


Figure 1 - Location of the Failure Site

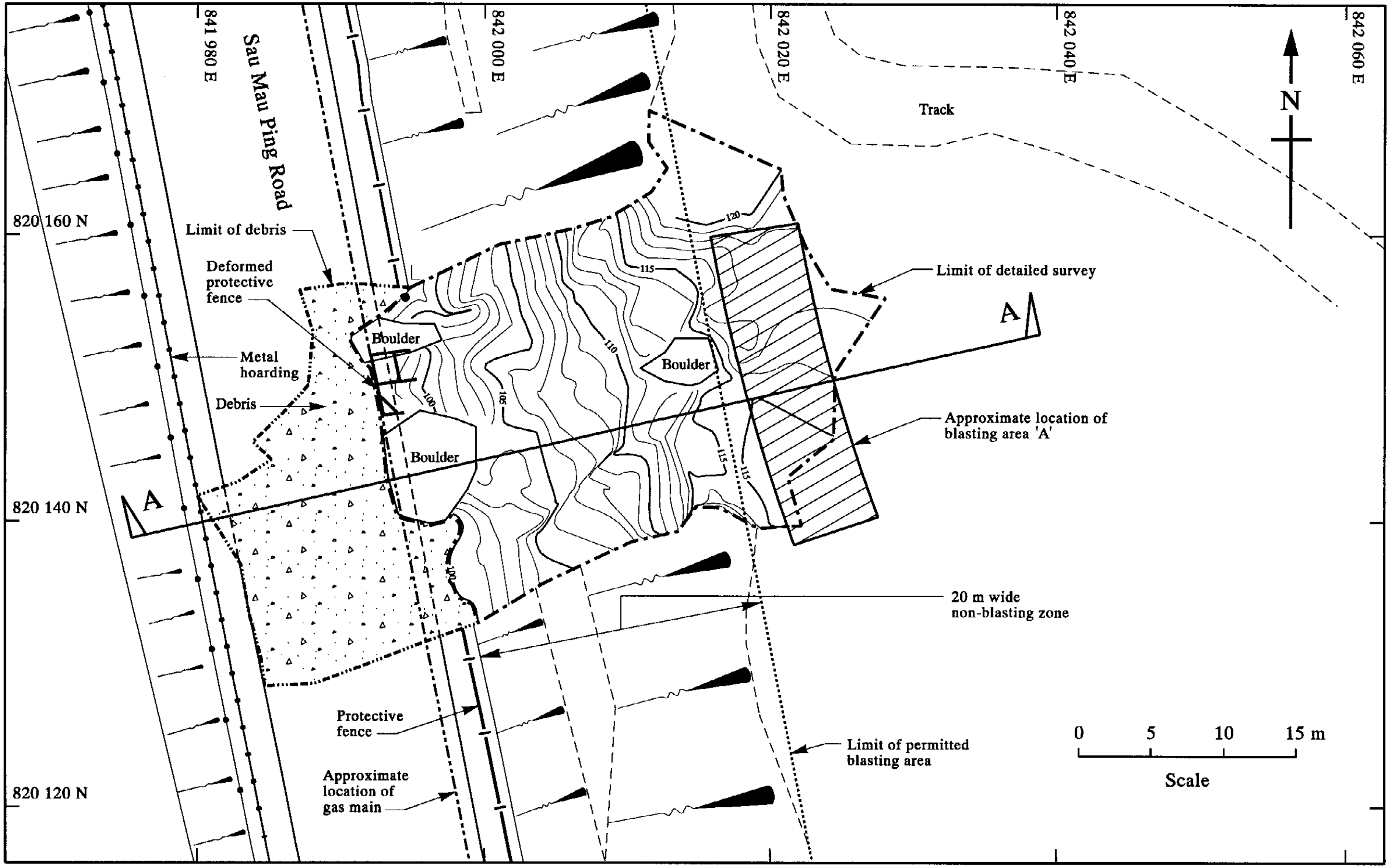


Figure 2 - Plan of the Failure

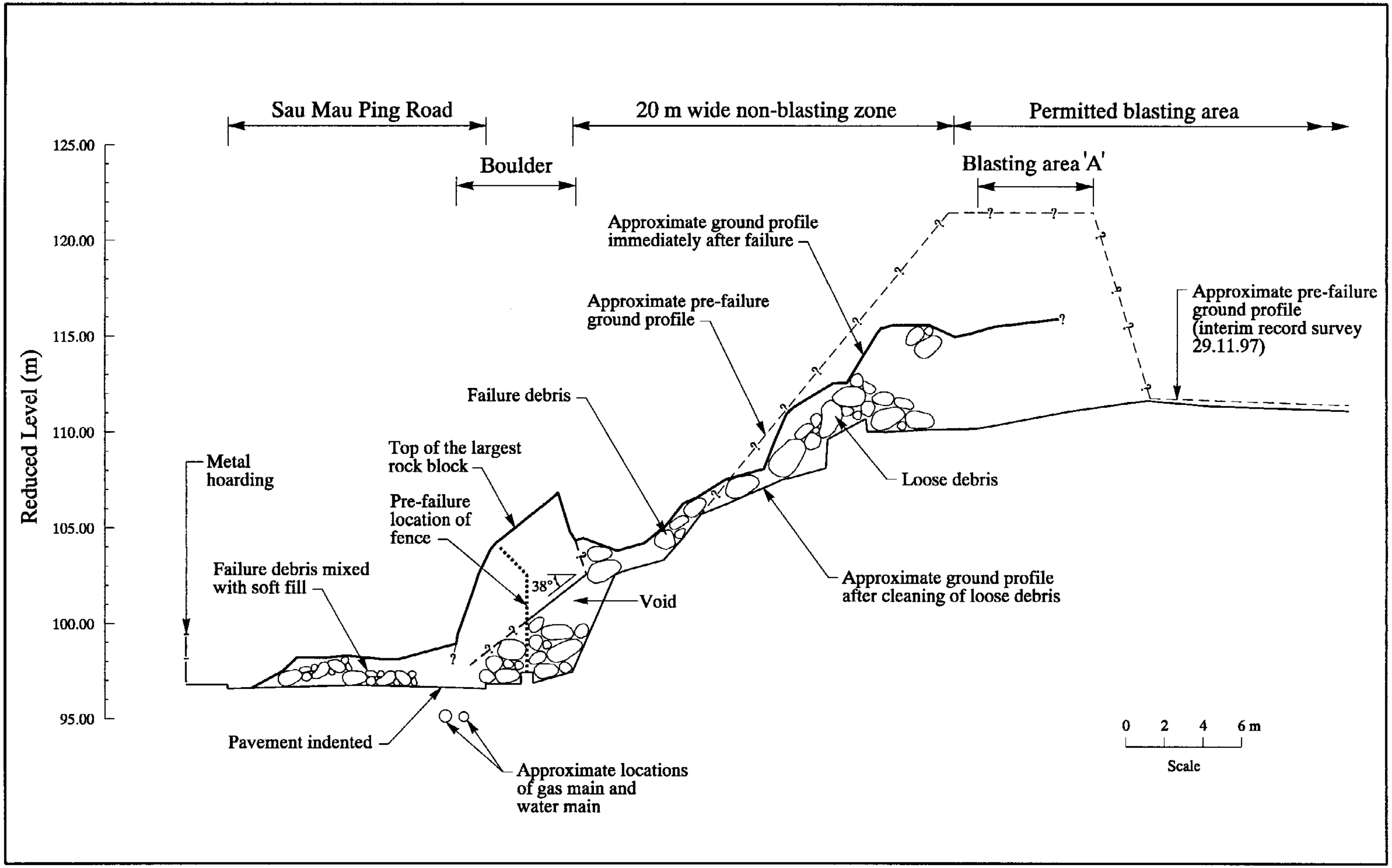


Figure 3 - Cross Section A-A

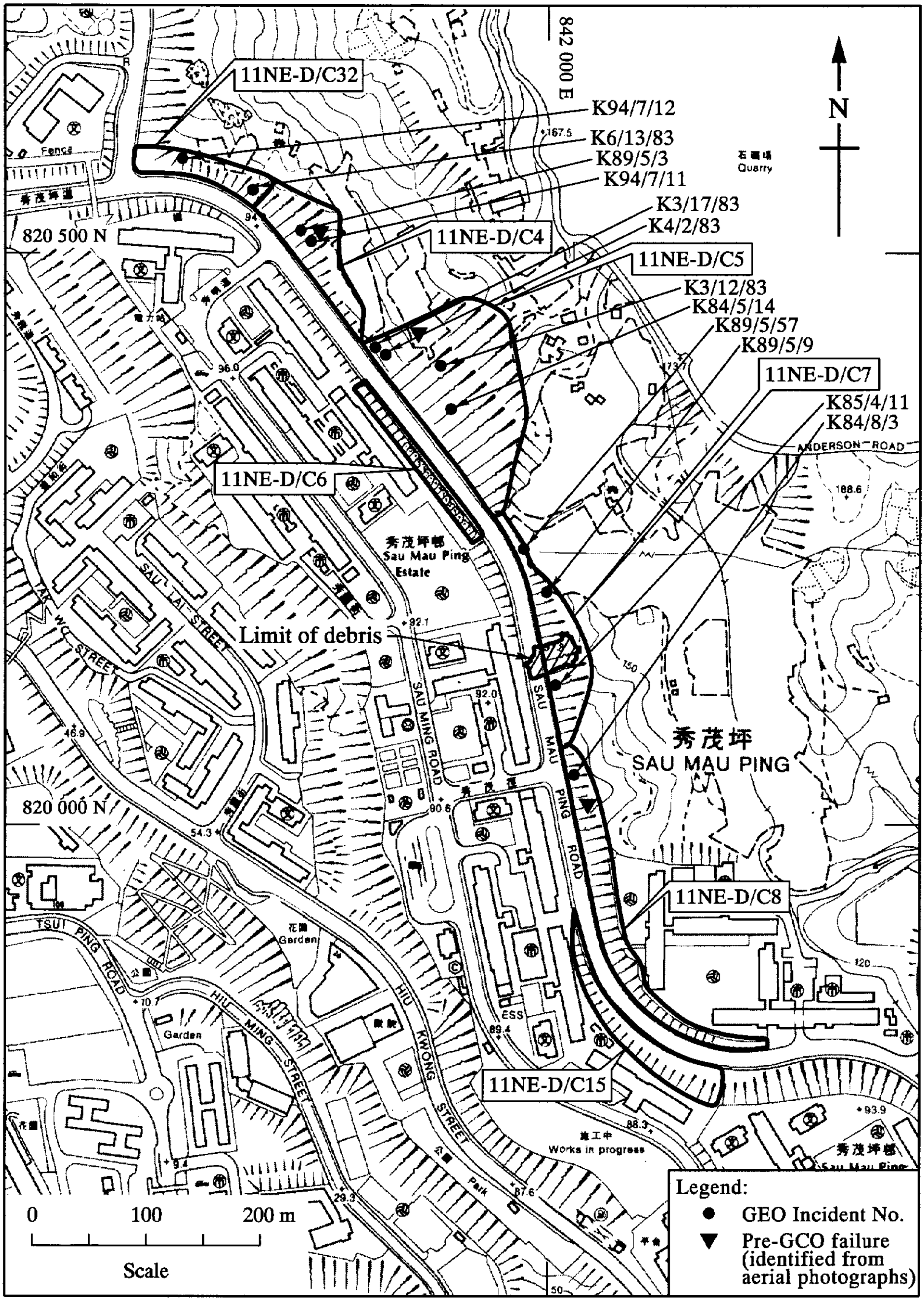


Figure 4 - Locations of Previous Failures Involving Cut Slopes along Sau Mau Ping Road

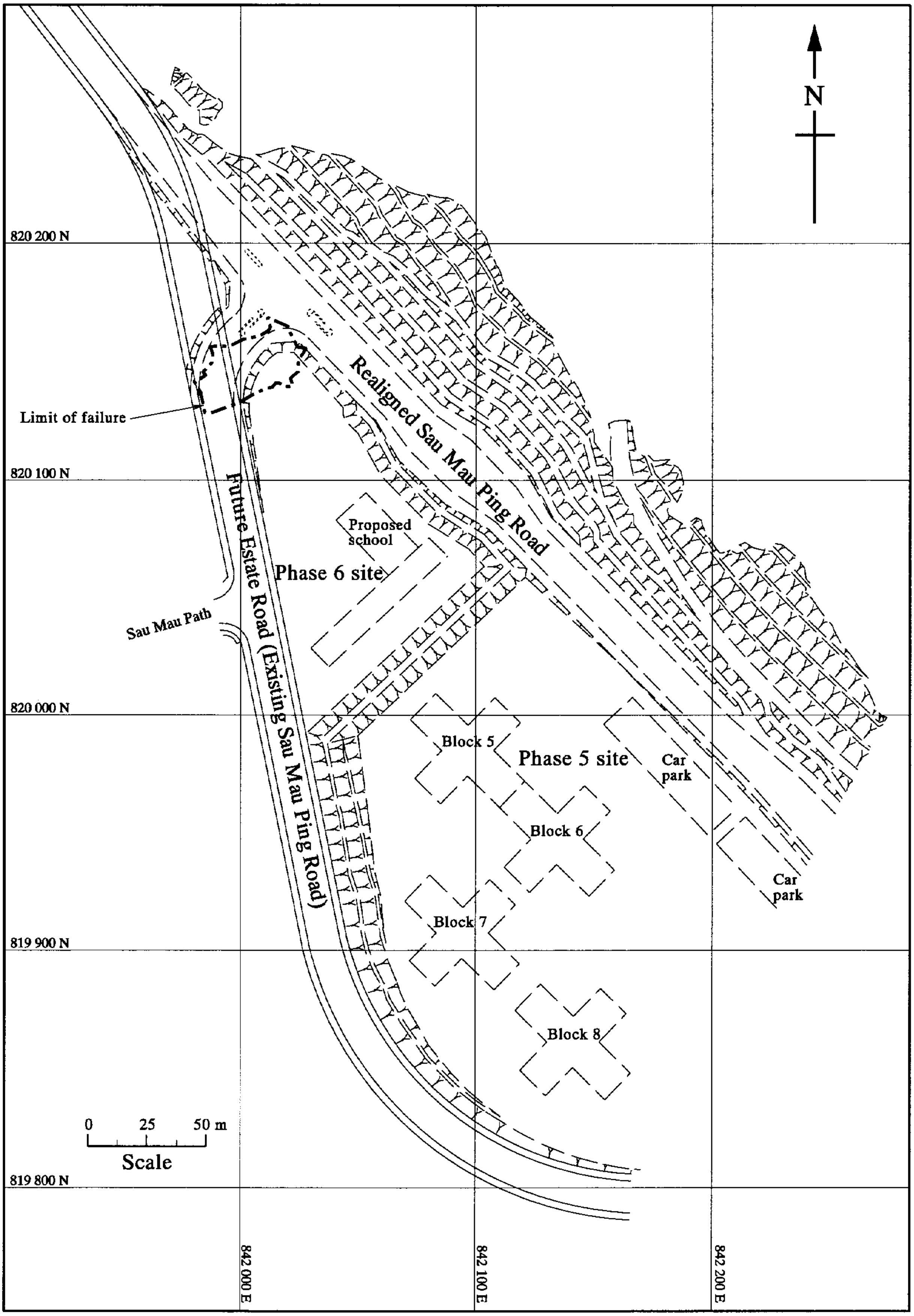


Figure 5 - Proposed Development Plan

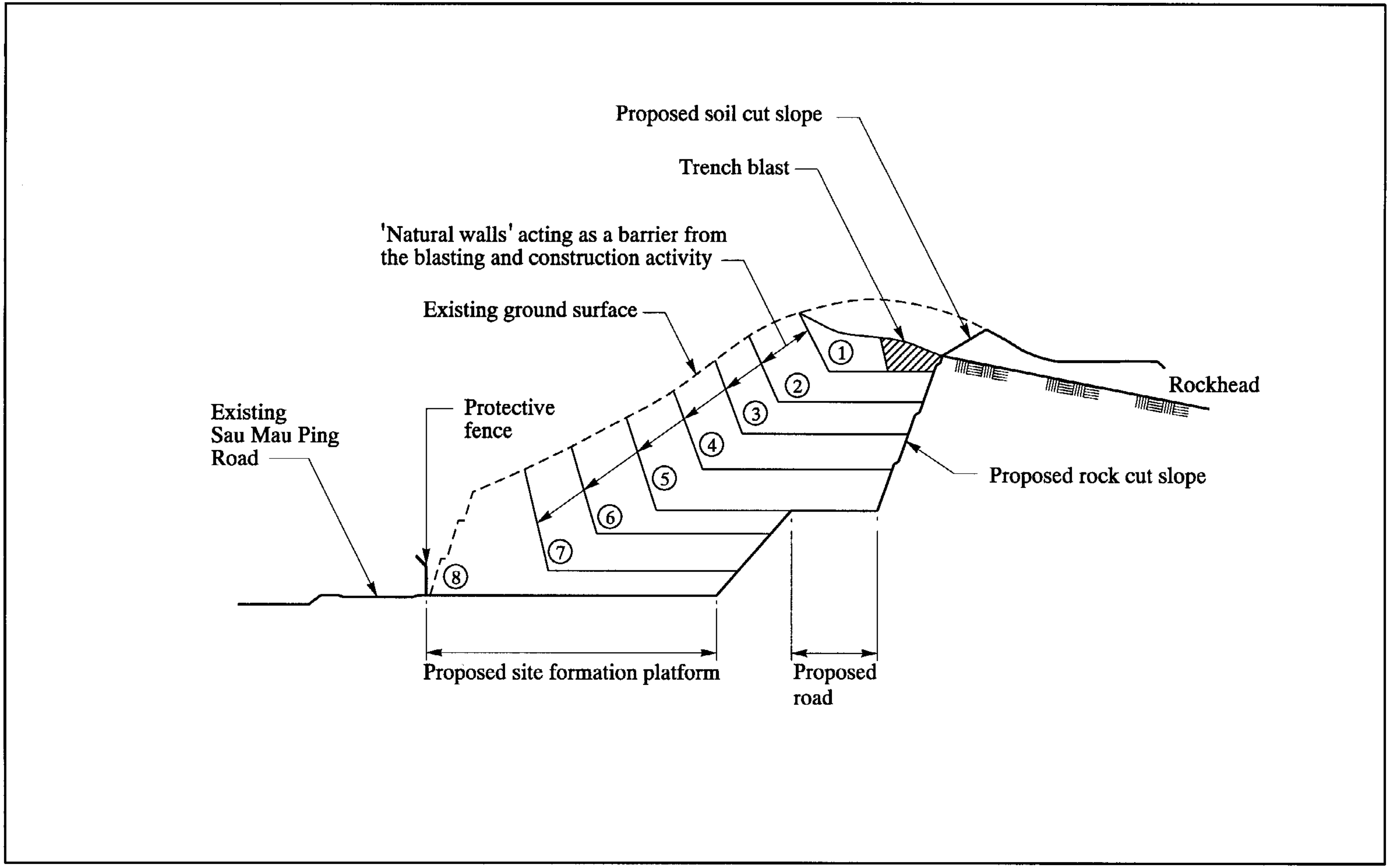
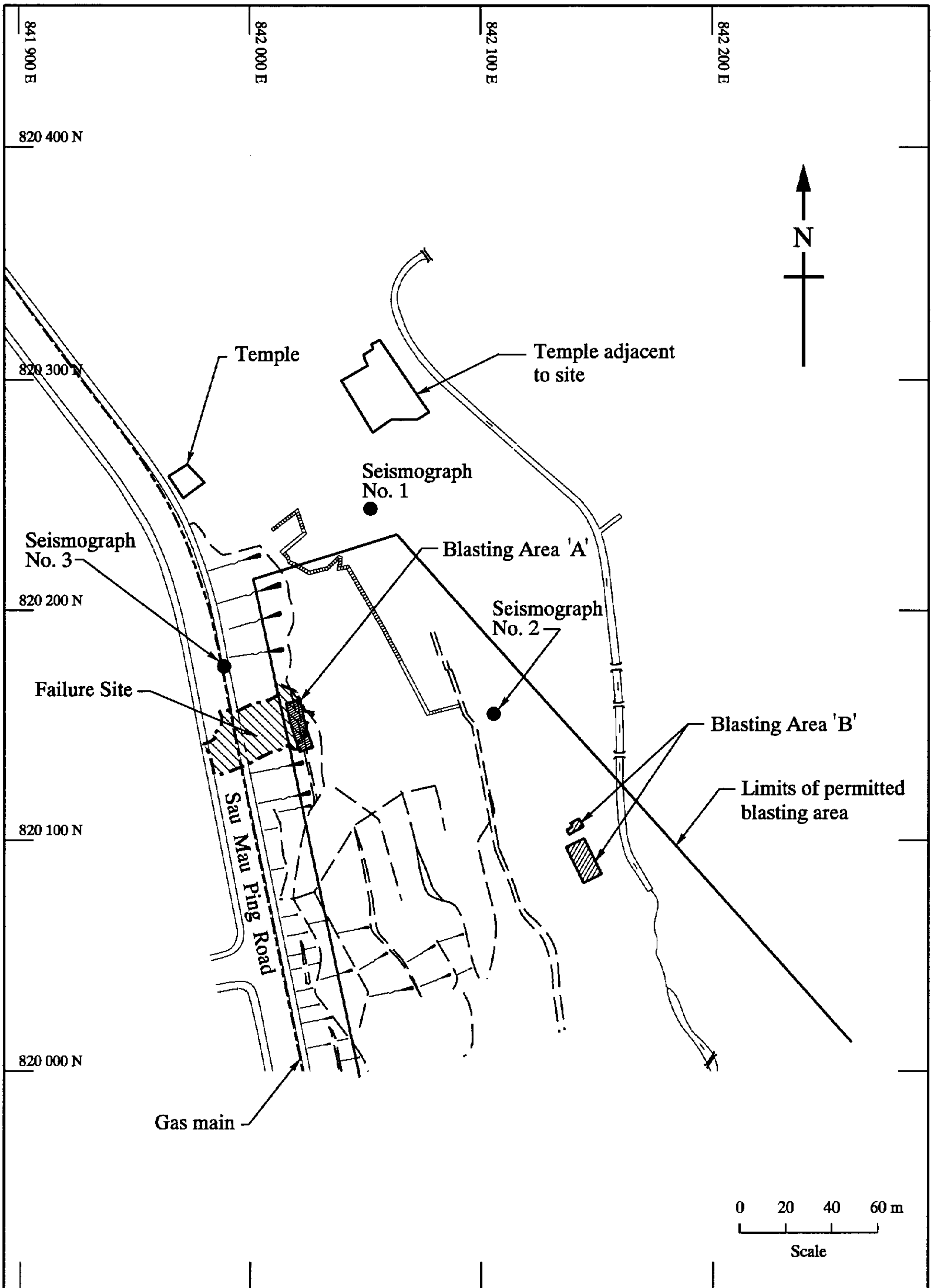
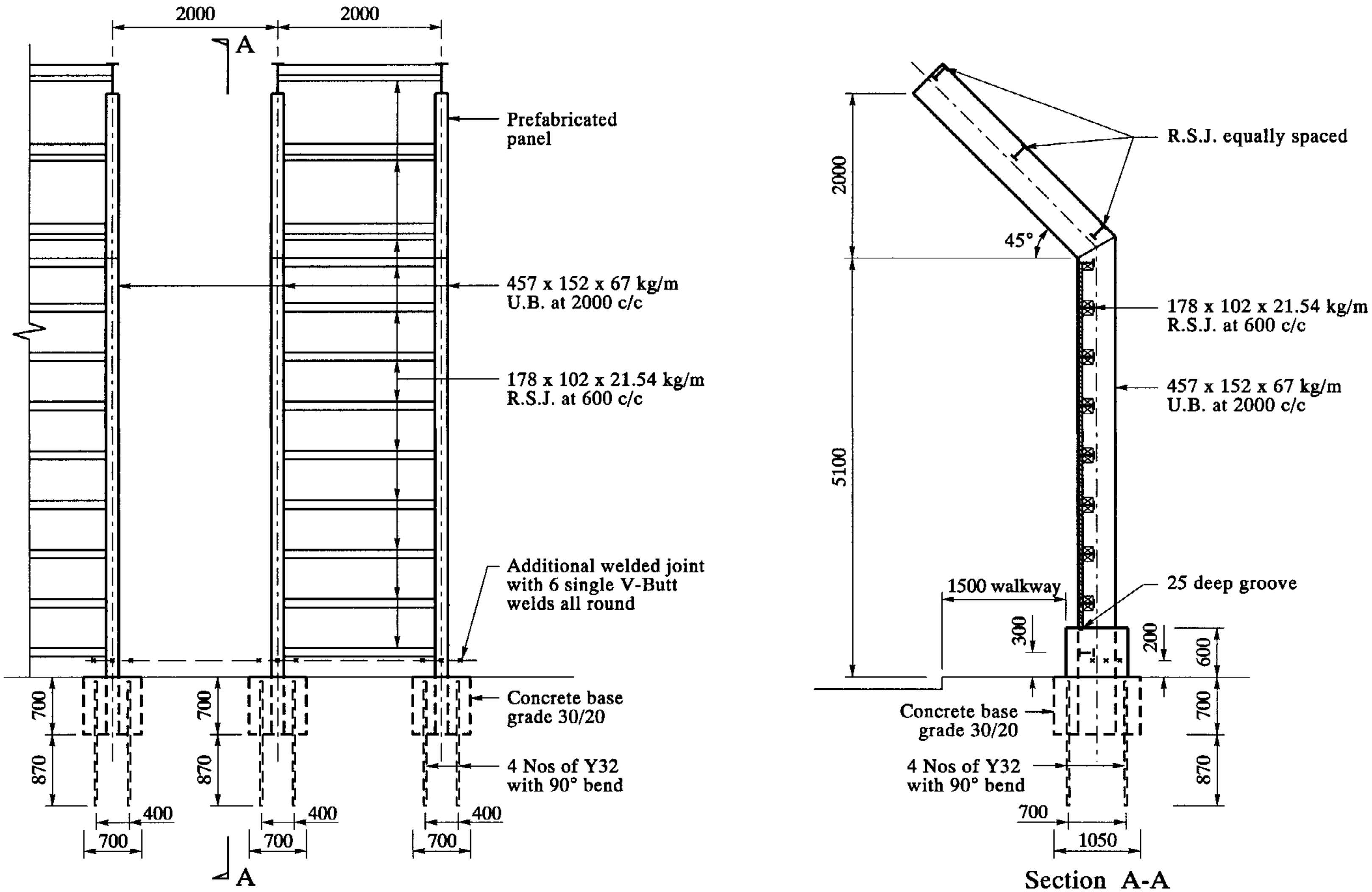


Figure 6 - Proposed Typical Sequence of Excavation in Rock



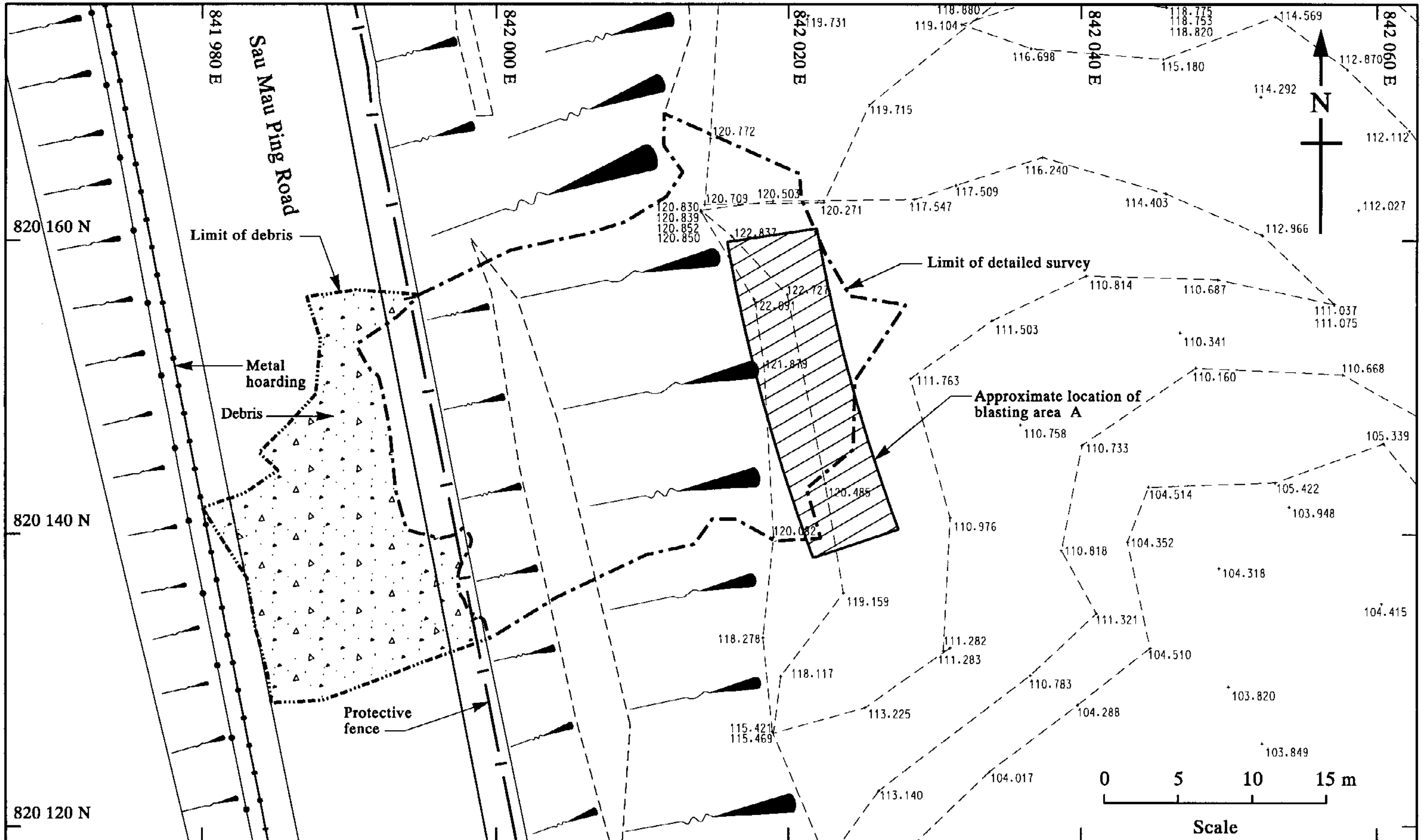
Note: The 4th seismograph is outside the coverage of this Figure to the south of the site.

Figure 7 - Limits of Permitted Blasting Area



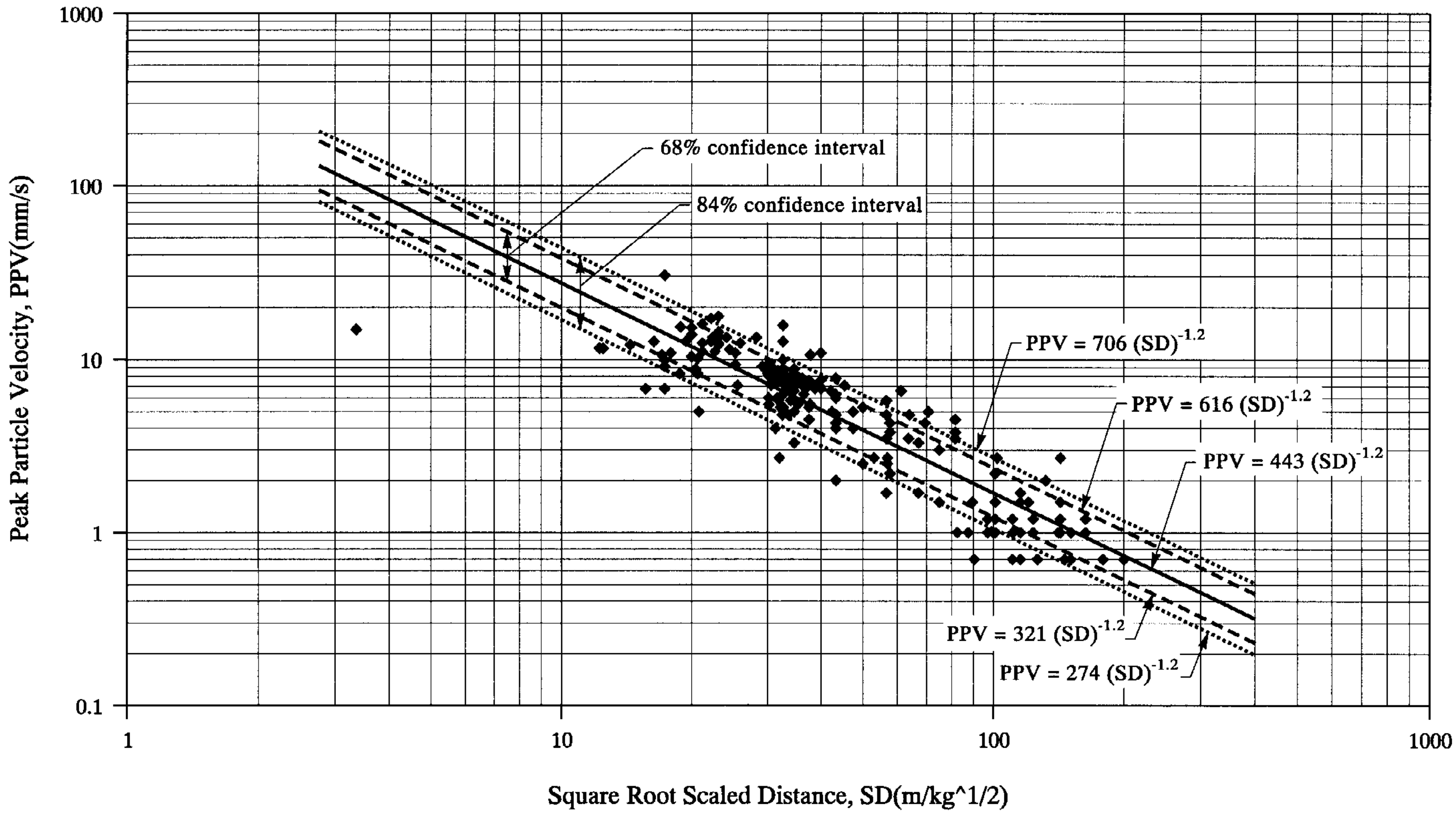
Note: All dimensions are in millimetres.

Figure 8 - Typical Details of the Protective Fence



Note: This Figure is reproduced from the interim survey plan dated 29.11.97 prepared by the Resident Engineer's Office.

Figure 9 - Interim Record Survey of the Area Close to the Crest of the Failed Slope on 29 November 1997



Legend:

◆ Data collected from 5.9.97 to 3.12.97

Figure 10 - Peak Particle Velocity versus Square Root Scaled Distance

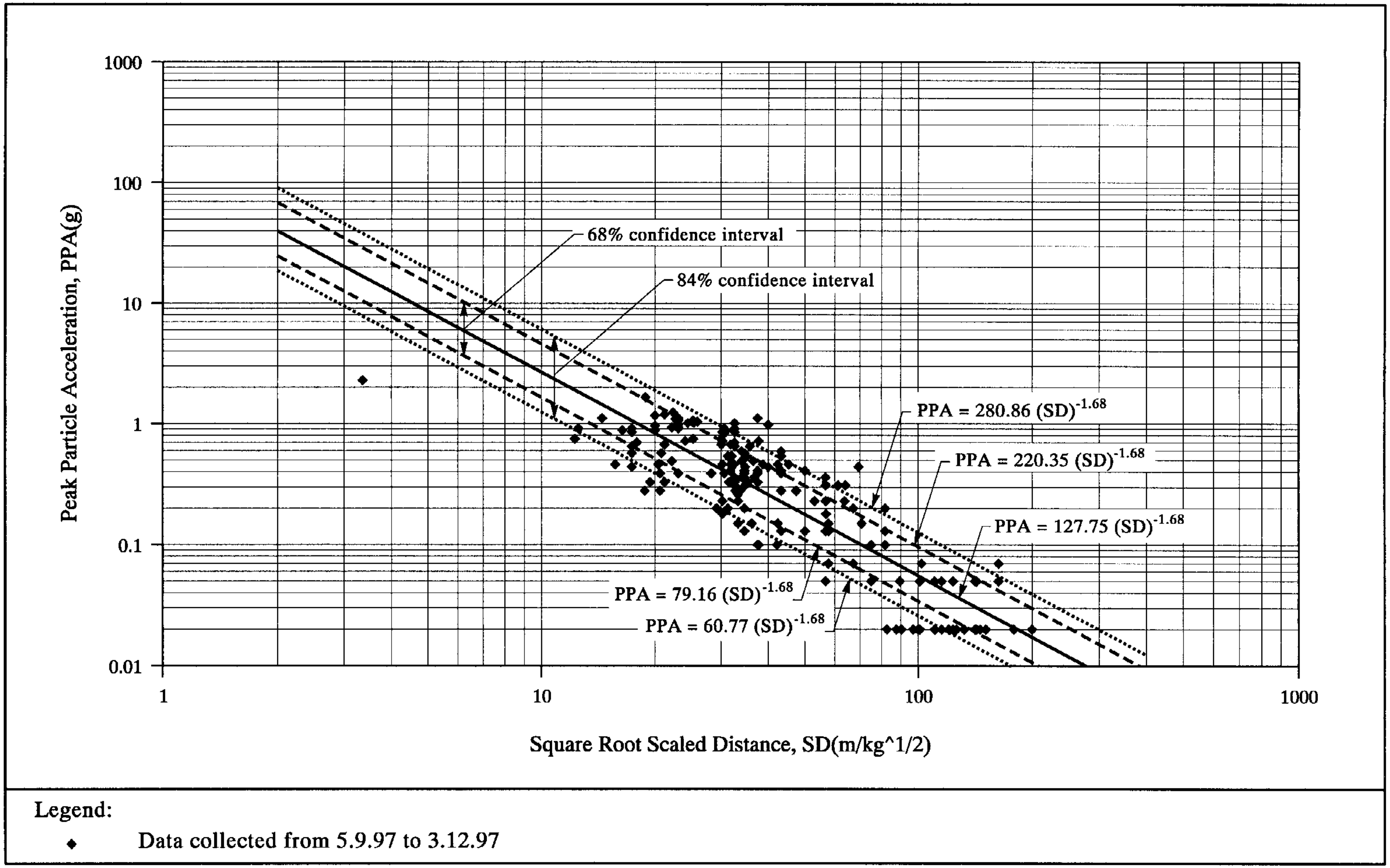
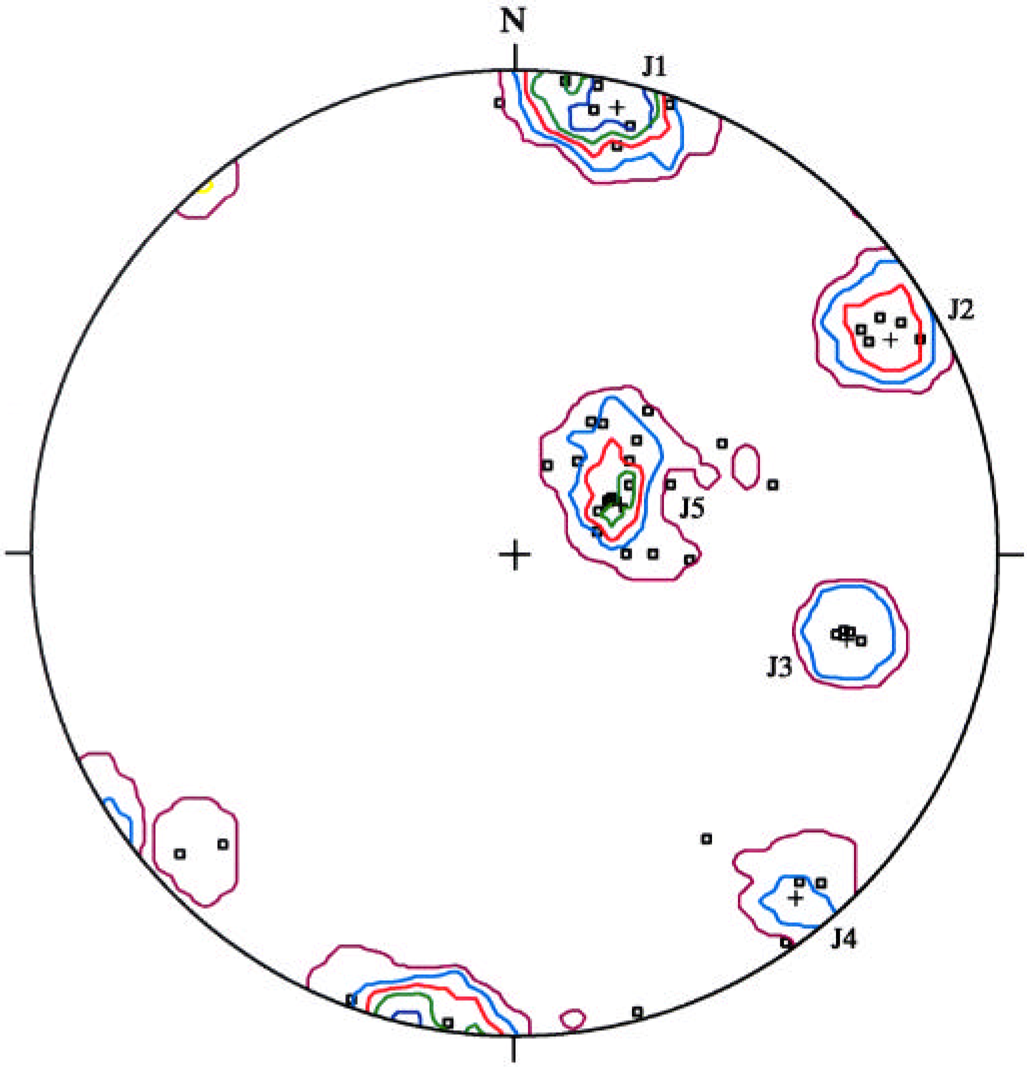


Figure 11 - Peak Particle Acceleration versus Square Root Scaled Distance

Equal Angle Equatorial Net



Legend:

▫ Pole to discontinuity (total 45 poles)

Pole Concentrations % of total per 1.0 % area

- < 3 %
- < 6 %
- < 9 %
- < 12 %
- < 15 %

Joint Set	Dip	Dip Direction
J1	88°	193°
J2	84°	239°
J3	72°	283°
J4	86°	321°
J5	29°	243°

Figure 12 - Stereoplot of Poles to Joints at the Failure Site

LIST OF PLATES

Plate 1 - Aerial View of the Failure Taken on 4.12.97 Shortly after the Failure (Reproduced from Sing Pao dated 5.12.97)

Negative No : SP 9710/C13

Plate 2 - Blast Damages Observed in the Vicinity of the Failed Site

Negative No : SP 9710/C23

Plate 3 - Slickensiding Observed on the Sub-vertical Face of the Largest Block of Rock

Negative No : SP 97105/99

Plate 4 - View of the Sub-vertical and Very High Persistence Rock Joint Bounding the Northern End of the Failure

Negative No : SP 97106/9

Plate 5 - View of the Sub-vertical and Very High Persistence Rock Joint Bounding the Southern End of the Failure

Plate 6 - View of the Failure Site about Three Months before the Failure
(Photograph Taken on 11.9.97 by Resident Site Staff)

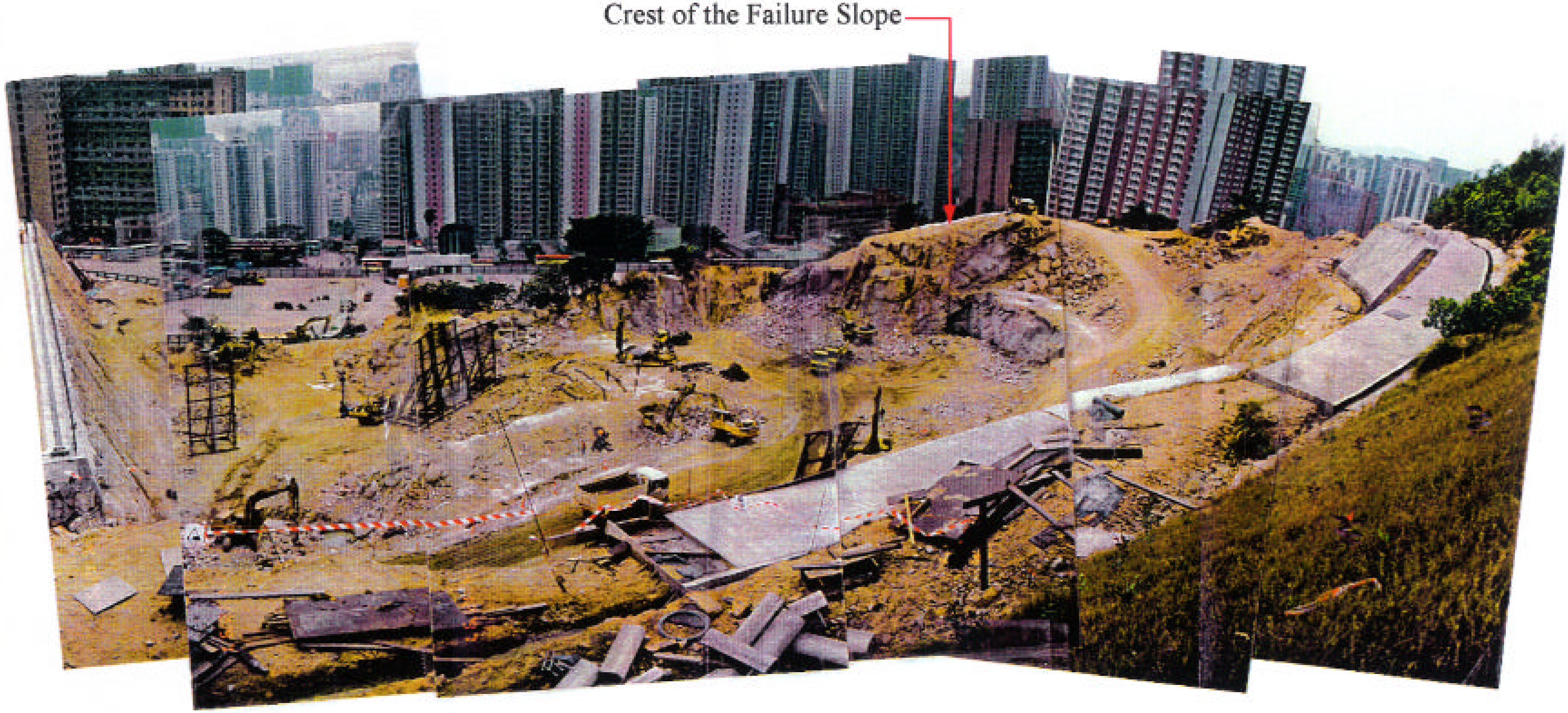


Plate 7 - View of the Site Looking West Taken on 4.12.97 before the Failure
(Bi-weekly Progress Photographs Taken by the Resident Site Staff)

Plate 8 - An Enlarged View of the Crest and the Cut Face at the Back of the Failed Slope before the Failure
(Extracted and Enlarged from Plate 7)

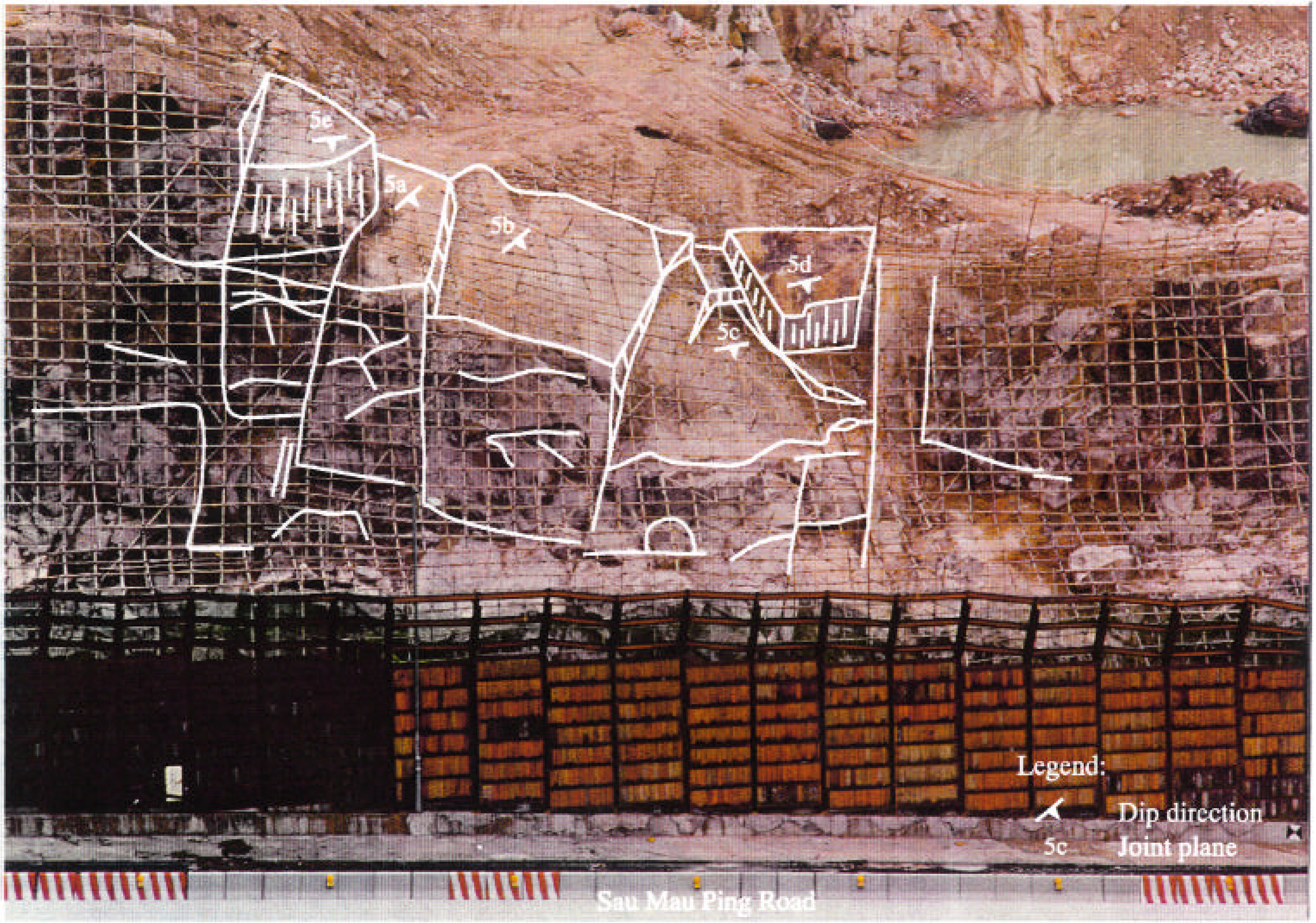


Plate 9 - View of the Site Following Removal of Failure Debris

APPENDIX A

SUMMARY OF SITE HISTORY

CONTENTS

A.1 SITE DEVELOPMENT

References

The earliest available aerial photographs of the site were taken in 1945. At that time, the site was undeveloped and drainage lines trending ENE-WSW and N-S across the site were evident. From the 1963 aerial photographs, two tributary drainage lines trending predominantly E-W could be seen traversing the slope 11NE-D/C7. One of the drainage lines was located near the northern edge of the subject failure site.

As evident from the aerial photographs, the construction of the Sau Mau Ping Estate, the Sau Mau Ping Road and also the slope 11NE-D/C7 was in progress in February 1967 and the construction of the Road and the slope could be seen to have been completed in February 1972.

From the aerial photographs, seepage from the slope 11NE-D/C7 was evident in February 1972 and October 1973, and the slope appeared to have become deeply eroded in October 1973. Squatter huts were evident in the northern part of the slope in December 1978, and more squatter huts could be seen in June 1980. The squatter huts between slope 11NE-D/C7 and C8 were cleared in January 1984, and eventually all squatter huts were seen to have been cleared in February 1988.

Minor slope works were seen to have been carried out on slope 11NE-D/C7 in September 1985 and March 1986. A landslide scar was evident near the northern end of the slope 11NE-D/C7 in November 1989, and slope works could be seen to have been carried out in this part of the slope in July 1993.

Site formation works carried out for the redevelopment of the Sau Mau Ping Estate to the east of the slope 11NE-D/C7 were in progress in September 1995. The upper part of the slope was being removed in May 1997 and standing water within the site formation area to the east of the slope was apparent.

A.2 PREVIOUS ASSESSMENTS

A.2.1 Slope Registration

In June 1977, the slope was registered by Binnie & Partners (Hong Kong) Consulting Engineers (B&P), engaged by the Hong Kong Government to prepare a catalogue of cut slopes, fill slopes and retaining walls (now known as the 'Catalogue of Slopes').

Binnie & Partners (HK) Field Sheet for Slope No. 11NE-D/C7

A.2.2 Study Report, Area 20 - Sau Mau Ping Road

In August 1973, at the request of the Chief Highway Engineer, Scott Wilson Kirkpatrick & Partners (SWKP) prepared a study report on the assessment of the safety of Sau Mau Ping Road with regard to the stability of the natural and man-made slopes. The study included field inspections, ground

SWKP (1973)

investigation and stability analyses and covered mainly three slopes (identified as Left-hand Cut, Main Cut and Right-hand Cut) along the road. Slope 11NE-D/C7 was identified as the Right-hand Cut. Professor E. Hoek of Imperial College inspected the site in April and July 1973, and his assessments were included in SWKP's Report as Appendices.

Detailed joint surveys were carried out and the results showed that "the dip and direction of the sheet joints are similar for the three cuts. For the Right-hand cut, the joint dips centre around 25° to 30° and the joints themselves are only partially developed. Therefore a significant, albeit unknown, amount of cohesion can be relied on besides friction on the joint planes." Page 3 of SWKP (1973)

Stability analyses were carried out for the slopes under three groundwater cases: dry, at existing groundwater level and fully saturated. The report noted that "The Right-hand Cut has no fully developed sheet joints although there are many in embryonic form. A considerable, although unknown, cohesion can be relied on for the solid sections of rock between the sheet joints and, even assuming the worst condition, failure of this slope is thought to be remote." The report concluded that "the stability of the Right-hand Cut is not in question". Page 5 of SWKP (1973)

A.2.3 Landslip Preventive Measures (LPM)

In 1989, the slope was considered by the Geotechnical Control Office (GCO) (renamed as Geotechnical Engineering Office (GEO) in 1991) for inclusion into the 1990-91 LPM programme.

However, the slope was subsequently deleted from the programme, as it would be significantly modified within the following 5 years due to Housing Department's (HD) proposed Redevelopment of the Sau Mau Ping Estate.

Later, the slope was re-considered for inclusion in the LPM 95/96 Programme. However, the slope was once again deleted from the programme as the slope would be removed under the site formation contract for the Sau Mau Ping Estate Phases 5 & 6 Redevelopment which was scheduled to be tendered in April 1995.

A.2.4 Geotechnical Design for the Site Formation for Redevelopment of the Sau Mau Ping Estate Phases 5 & 6 and Realignment of Sau Mau Ping Road

A.2.4.1 Geotechnical Design Report (OAP, 1994)

A geotechnical design report, prepared by the Ove Arup & Partners (OAP), was submitted to GEO on 28 October 1994 for the site formation works for redevelopment of Sau Mau Ping Estate Phases 5 & 6 and realignment of Sau Mau Ping Road. The report covered the geotechnical design of slopes, retaining walls and surface drainage associated with the site formation works. OAP (1994)

In the covering letter (ref. 21282/PF/VN (3347) dated 28.10.94) for the submission, OAP noted "....sheeting joints in rock....as the major areas of geotechnical concern."

The cut slope 11NE-D/C7 fell within the area for the Phase 6 of the redevelopment which was described in the report as "The Phase 6 site is a natural hillside with gentle wooded slopes dipping to the west. However adjacent to Sau Mau Ping Road, the hillside has been excavated leaving a large cut slope (ref. 11NE-D/C7 in Figure 3), partly in soil and partly rock, which was formed during road construction." Para 3.1.3, OAP (1994)

"In order to meet the programme, blasting will be the medium of excavation in rock...." Para 5.4.3, OAP (1994)

"The site is surrounded by sensitive receivers; high rise residential blocks, water supply services and other utilities. However the presence of a high pressure gas pipe under the east side footpath of Sau Mau Ping Road represents the major constraint. Hong Kong Gas has advised that vibration levels must not exceed 10 mm/sec at this pipe." Para 5.4.4, OAP (1994)

"An extensive joint mapping exercise was carried out on existing slopes in and around the site. Slopes inspected are shown in Figure 4. Impression packer tests were also undertaken in six boreholes and joint data taken." Slope 11NE-D/C7 was one of the slopes inspected. Para. 6.4.2, OAP (1994)

"However some caution is warranted in respect of joint set P4 for the future road cutting (Figure 8a). These are part of the same set as the sheeting joints which are exposed in the slopes north of the site (slopes 11NE-D/C4 and 11NE-D/C5 in Figure 3)." Para 6.4.6, OAP (1994)

"Sheeting joints are another concern even though the kinematic analysis for the future slope (Figure 8a) showed there to be no failure mechanisms. Cut slopes above Sau Mau Ping Road, immediately to the north of the site (11NE-D/C5 & C4 in Figure 3) have failed in the past along sheeting joints. It is likely that this was due to poor control of blasting." Para. 6.5.5, OAP (1994)

A.2.4.2 GEO's Comments on the Report and OAP's Responses

GEO commented that "OAP need also to review the stability of the existing slopes (e.g. cross-sections 4-4, 5-5 & 10-10) within the site boundary and upgrade them to current standards where necessary. The stability of the overall slope including under blasting vibration should also be checked." Cross-section 4-4 cuts across slope 11NE-D/C7. Para 2.4.1 (g), GEO letter to OAP dated 12/12/94

OAP responded that "The slope stability analysis for the existing slopes (i.e. those that will remain) are included in the Appendix B of our original submission.... Existing slopes north of these (i.e. essentially those north of Section 5-5) will be removed entirely." The existing slopes north of Section Page 6, OAP letter to GEO dated 27/11/95

5-5 included the slope 11NE-D/C7.

"Blast vibration at the existing slopes to remain will be limited as the Contract specifies a 20 m non blasting zone (see plan) adjacent to Sau Mau Ping Road and Po Lam Road and a maximum PPV of 10 mm/sec at gas pipes located below the Sau Mau Ping Road footpath adjacent to the site." Page 7, OAP letter dated 27/11/95

GEO noted that "The kinematic stability assessment.... showed that a few individual joints fall within the sliding zone for face angle of 60 degree. Many joints, joint set P4 and the line of intersection J6 lie within the daylighting envelope....joint set P4 should be treated with extreme caution. These are part of the same set as the sheeting joints which are exposed in the slopes north of the site." Para 2.4.2 (a) GEO letter dated 12/12/94

OAP agreed and noted that stabilization measures would be installed if persistent joints of set P4 are exposed and have a potential for sliding. Page 7, OAP letter dated 27/11/95

GEO reminded HD that ".... adequate and proper site supervision, with the necessary input by experienced geotechnical engineers, should be provided for the project." Para 3, GEO Memo to HD dated 5/3/96

OAP responded that "We have an adequate and proper resident site team. The team comprises one RE, one GE, three AREs, one IOW, one AIOW, one WSI and one SSO." Page 3 of OAP letter to GEO dated 30/9/96

A.2.5 Blasting

A.2.5.1 Blast Assessment Report

A blast assessment report dated 28 October 1994, prepared by AsiaConsult Pacific Ltd (APL), the Sub-consultant to OAP was submitted to GEO on 31 October 1994. OAP letter to GEO dated 31/10/94

The report noted that a meeting was held on 24 June 94 between APL and the Mines and Quarries Division of GEO, and in that meeting "The Mines Division also expressed concern with regards to the stability of adjacent slopes to the work site. A major slope failure has occurred in the past in a cutting north of the site and since then, the slopes have failed on two occasions, both in soil. The long term effects of blasting and to what degree the existing slopes would be affected by blasting during construction are pertinent factors highlighted." Section 3.0, APL (1994)

APL identified the rock slopes along Sau Mau Ping Road which had a history of instability as one of the blast sensitive receivers. However, which particular slopes should fall into this category were not mentioned. Section 4.0 APL(1994)

With regard to the adjacent housing blocks, APL noted that "....Eventually, blasting could get as close as 20 m from the residence.... With correct preventive measures, ...blasting could be carried out at these distances Section 4.1 APL (1994) from the buildings.... To further mitigate any impact, blast free faces can be orientated to produce a general direction of movement of rock away from the sensitive receivers."

With regard to the existing rock slopes along Sau Mau Ping Road, APL noted that "We have been advised by OAP that rock joint mapping has shown that a random joint pattern exists within the granite rock. Despite this sheeting joints do exist in the cutting above Sau Mau Ping Road north of the site (GEO Slope Ref. 11NE-D/C5). During the formation of Sau Mau Ping Road there has been a number of failures although it is not clear whether these have been failures in rock.... There are also a number of other large slopes both within the estate in the site which are to be removed. In the case of the latter, the stability of these must be maintained before being removed.... OAP advises that the Factor of Safety on these slopes is probably close to unity in many cases. Using the GEO Report 15 "Assessment of Stability of Slopes subjected to Blasting Vibration" as a guideline, the critical PPV for a factor of unity is 18 mm/sec at the failure plane on the slope surface. The PPV represents the limit for small failures which is more critical than for large failure mechanisms." Section 4.3 APL (1994)

With regard to Services and Utilities, APL noted that "Intermediate pressure mains are located along the east side of Sau Mau Ping Road. (Drawing No. 4.9).... Vibrational effects are a concern with both intermediate and high pressure mains and the Hong Kong Gas have set PPV limits of 10 mm/sec at the mains." Section 4.5 APL (1994)

The method of excavation was also described by APL that "In general a sequence of operations will have to be adopted such that there is always a natural wall between the works area and the residential environment.... The success of the method depends on the steepness of the slope that is being cut. However, reviewing sections through the site....this principle can be adopted for the majority of the site." Section 6.0 APL (1994)

"It is imperative that on this site, blasting is carried out such that all blasts, as practicable as possible be orientated free-facing the direction opposite to the residential developments and the existing Sau Mau Ping Road. This will reduce the risk of any potential negative effects of blasting having an impact on the surrounding existing development." Section 6.3 APL (1994)

Trial blasting was recommended to be carried out on site at the initial stage of the project to determine "site factors K and B for determining the interpolated ground vibration values on sensitive receivers.... It is envisaged that five to ten trial blasts will be carried out for the above purpose." Section 6.6 APL (1994)

A.2.5.2 <u>GEO's Comments on the Blast Assessment Report</u>

GEO's comments on the assessment report are as follows:

"It would be useful to put on a plan the anticipated zone of influence due to blasting for identifying the possible blast-sensitive receivers." Para 4.3.1 GEO memo dated 2/12/94

"It is considered that not only rock slopes along Sau Mau Ping Road which have a history of instability be assessed, but also other rock slopes along Sau Mau Ping Road and along Po Lam Road and within the zone of influence." Para 4.3.2 GEO memo dated 2/12/94

"There are numerous large boulders scattered on the slopes along Sau Mau Ping Road. In some cases, their factor of safety may be very close to unity. Would blasting have any adverse effects on their stability?" Para 4.3.4 GEO memo dated 2/12/94

"The parameters assumed in the calculation of the critical PPV on rock slope should be submitted for consideration." Para 4.4.1 GEO memo dated 2/12/94

"Will there be any loss of shear strength at the joints after blasting? The long term stability of the rock slopes and boulders should be assessed." Para 4.4.2 GEO memo dated 2/12/94

"The recommended max PPV limits for rock and fill slopes should be more clearly specified in the Table." Para 4.7 GEO memo dated 2/12/94

"On-site monitoring is recommended to check the actual level of vibration of the rock (sheeting) joints and boulders caused by blasting and the displacement of the joints and boulders. This would provide the data required for the improvement of blast design, as well as of methods for assessing rock slope and boulders stability." Para 4.8.1 GEO memo dated 2/12/94

"It is considered that there is a need for regular slope (soil and rock slopes) inspection and slope movement monitoring throughout the blasting period." Para 4.8.2 GEO memo dated 2/12/94

OAP sent a copy of the particular specification for blasting which would be incorporated into the HD's contract document to GEO and noted that "The purpose of the non-blasting zones and limitations on PPV is to protect the gas mains on Sau Mau Ping Road and adjacent temples." OAP also noted that they would respond within the following few days to GEO to address the outstanding queries raised by GEO on the Blast Assessment Report. However, nothing was received up to the time of the rock slope failure. OAP letter to GEO dated 3/11/95

A.2.5.3 General Specification (GS) and Particular Specification (PS) Clauses of the HKHA Contract "72 of 1994" Related to Blasting

The PS for the Hong Kong Housing Authority (HKHA) Contract "72 of 1994" for the site formation of the "Redevelopment of Sau Mau Ping Phases 5 & 6 and Realignment of the Sau Mau Ping Road" stipulates the following: PS under GS HKHA Contract "72 of 1994"

"The peak particle velocity at the gas main along the east side of Sau Mau Ping Road shall not exceed 10 mm/sec. The peak particle velocity at the two temples at the west end of the site shall not exceed 25 mm/sec." Clause 6.13, HKHA Contract "72 of 1994"

".... Road closures will apply at all times regardless of whether blasting is close to the road or away from the road anywhere else on Site." Sub-clause (b)(vi), HKHA Contract "72 of 1994"

"Blasting will not be permitted within a 20 metres zone adjacent to Sau Mau Ping Road...." Sub-clause (c)(iii), HKHA Contract "72 of 1994"

"....perform on site 'up to ten trial blasts'.... at selected locations.... can enable the determination of site factors such that peak particle velocity values can be reasonably predicted and recorded." Sub-clause (d)(i), HKHA Contract "72 of 1994"

A.2.5.4 Blasting Licence and Permit

In October 1995, China Road and Bridge Corporation(CRBC) submitted a method statement to GEO for the application of Blasting Licence and Permit. The statement included proposed safety measures, site clearance method, procedure for blasting and typical blasting pattern to be adopted and the responsibility of staff for blasting works. The statement also included a table showing the charge weights which were proposed to be adopted for blasting adjacent to gas mains and sketches showing typical blast pattern and details of blasting sequence and protective measures. CRBC letter to GEO dated 9/10/95

CRBC also submitted sketches showing the locations of the Gas Main and other sensitive receivers to GEO. Revised details of the protective screens, typical protective measures as well as a table showing the charge weights were also submitted. CRBC letter to GEO dated 24/10/95

A.2.5.5 Category 1 Dangerous Goods Licence and Permit to Use Category 1 Dangerous Goods

On 21 November 95, the Commissioner of Mines issued a Licence and a Permit respectively to China Road and Bridge Corporation and Sang Hing Civil Contractors Co. Ltd to possess and to use explosive at the Sau Mau Ping site for a period from 24 November 95 to 23 November 96. The Licence and Permit were subsequently renewed twice to cover the periods from 24 November 96 to 23 November 97 and from 24 November 97 to 23 November 98. Commissioner of Mines Permit and Licence dd 21/11/95

The permitted blasting area was marked on a plan attached to a set of conditions for approval of the permit and licence. Some of the conditions are:

"For the area hatched in green, the site shall be developed along the lines indicated X-X and worked in the direction marked Y-Y on the attached plan." Para 6

"Delay blasting techniques shall be used for all primary and presplit blasts." The maximum amount of explosives per delay period versus distance from the nearest gas main, water tank and pump house, adjacent temple, water Para 20 mains etc. were also stipulated in a table.

A.3 PREVIOUS LANDSLIDES OCCURRED IN THE VICINITY OF THE SUBJECT FAILED SLOPE

A minor failure involving approximate 3 cu.m. of boulders occurred on the lower part of the slope 11NE-D/C7 on 21 April 85 affecting the pedestrian pavement. GCO Incident Report No. K 85/4/11

The northern end of the cut slope 11NE-D/C7 was reported to have failed on 2 May 1989. The failure was in the upper soil portion of the slope (about 30 m above the level of Sau Mau Ping Road) and involved approximately 200 cu.m. of soil debris. A vehicle was damaged and the road and pedestrian pavement were blocked. Emergency works recommended by GCO were to trim back the failed slope and provide chunam protection and a surface drainage system. GCO Incident Report No. K 89/5/9

A very minor failure in soil occurred near the crest of the same slope on 5 May 1989 affecting the squatter huts. GCO Incident Report No. K 89/5/57

A.4 REFERENCES

AsiaConsult Pacific Ltd (1994). Site Formation for Redevelopment of Sau Mau Ping Estate Phases 5 & 6 and Redevelopment of Sau Mau Ping Road - Blast Assessment Report, 27 p. plus 18 Plates.

Binnie & Partners (HK) Ltd. (1977). Cut Slope No. 11NE-D/C7, Sau Mau Ping Road - Field Sheet, Binnie & Partners (HK) Ltd for Geotechnical Control Office, 2 p. (Unpublished).

GCO (1985). Incident Report No. K85/4/11. Geotechnical Control Office, Hong Kong.

GCO (1989). Incident Report No. K89/5/9. Geotechnical Control Office, Hong Kong.

GCO (1989). Incident Report No. K89/5/57. Geotechnical Control Office, Hong Kong.

GEO (1994). Mainland East Division File Ref. No. GCMd 3/2/208 Pt 3. Geotechnical Engineering Office, Hong Kong.

Housing Authority (1994). General Specification and Particular Specification, HKHA Contract 72 of 1994, Hong Kong Housing Authority.

Ove Arup & Partners (1994). Site Formation for Redevelopment of Sau Mau Ping Estate Phases 5 & 6 and Realignment of Sau Mau Ping Road - Geotechnical Design Report. Ove Arup & Partners, 20 p. plus 9 Figures, 6 Appendices and 11 Drawings.

Scott Wilson Kirkpatrick & Partners (1973). Licensed & Other Areas - Ground Investigation, Area 20 - Sau Mau Ping Road - Study Report. Scott Wilson Kirkpatrick & Partners for Highways Office, Hong Kong, 8 p. plus 3 Appendices.

APPENDIX B

DISPLACEMENT ANALYSIS OF THE FAILED ROCK MASS

CONTENTS

B.1 LABORATORY TESTING

For the analysis of the failure described in Section B.3, a limited number of tests were carried out as part of this investigation to determine the range of shear strength of rock joints found at the site.

Two direct shear tests were carried out on specimens prepared from a sheet of completely decomposed granite detached from the base of the largest block of rock, which is believed to be the weathered materials formed along a rock joint. Multi-stage tests were performed due to the limited materials available. An average friction angle of 37.5° is obtained for these materials (see Figure B1).

The 'peak' friction angle of a moderately decomposed rock joint within a core sample was determined using a tilt table in accordance with the method described by Barton & Choubey (1977) and was found to be equal to 39.5°. The joint roughness coefficient (JRC) of that rock joint was also determined by visually comparing the joint surface with typical profiles provided by Barton & Choubey (1977) and was found to be equal to 7.8.

The basic friction angle of slightly decomposed granite was also determined by sliding rock cores against each other on a tilt table and was found to be equal to 39° (dry) and 37.5° (wet).

All of the above tilt table tests were carried out on rock core samples recovered during the previous ground investigation for the design of the site formation works for the redevelopment of the Sau Mau Ping Estate, as no ground investigation works were carried out separately for the present investigation.

B.2 THE CHARACTERISTICS AND SHEARING RESISTANCE OF THE SLIDING SURFACES

After the clearing up of the failure debris from the failed slope, five persistent joint surfaces were exposed dipping at an average angle ranging from 16° to 29° in a direction towards the Sau Mau Ping Road. The characteristics of these sliding surfaces are summarised in Table B1.

As can be seen from Table B1, the large scale roughness angles of these surfaces range from 3.5° to 14° with an average of 8.5°, while the small scale roughness angles range from 6° to 14.5° with an average of 10.1°. The large scale roughness was also assessed by direct measurement on surveyed cross-sections of the slope and the average value was found to be 8.5°. As can be seen, there is apparently little difference amongst the average roughness angles measured from the failure surfaces.

Using the roughness amplitude measurement method as suggested by Barton (1990) as a first approximation, the JRC values determined on site for the sliding surfaces ranged from 1 to 20 with an average of about 8 (see Table B1). A JRC value of 7.8 was determined for a joint surface (of about 100 mm in length) in the laboratory (see Section B.1). After accounting for the scale effects as suggested by Barton (1990), a field JRC value of about 4 was derived from this laboratory value.

The persistence of the sliding surfaces ranges from 4 to about 10 m with an average of about 6.7 m.

Basic friction angles (ϕ_b) of 39° (dry) and 37.5° (wet) were obtained for the slightly decomposed granite using a tilt table (see Section B.1). Separately, ϕ_b values of 25° to 33.5° were obtained by deducting the small scale roughness of 6° to 14.5° from the friction angle of 39.5° determined for the rock joint in the moderately decomposed rock core in the laboratory (see also Section B.1). Barton (1973) has suggested that the ϕ_b values for coarse-grained and fine-grained granite are in the range of 31° to 35° and 29° to 35° respectively. A ϕ_b value of 33° was obtained by Irfan & Evans (1991) for the medium-grained granite at Shaukeiwan. After considering the likely sensitivity of the analyses to be carried out to the value of shear strength, a ϕ_b value of 33.5° was adopted for the rock joint for the present investigation. In some circumstances, the value of the basic friction angle, ϕ_b, can be larger than the residual friction angle, ϕ_r. However, the difference is negligible and for practical purposes ϕ_b and ϕ_r are interchangeable (CANMET, 1977).

At low stress level, the shear strength of a discontinuity is controlled by the interlocking of small surface irregularities, and for a ϕ_r value of 33.5° and an average small scale roughness (i) of 10.1° for the sliding surfaces, the effective friction angle ($\phi_{eff} = \phi_r + i$) was found to be equal to 43.6° in accordance with Patton (1966).

The effective friction angle (ϕ_{eff}) has also been assessed in accordance with Barton (1990), which takes into account the effects of scale and stress level, and was found to vary between 49.6° to 51.6° with an average of 50.6 ° (see Table B2).

At the failed slope, the amplitude of the asperities on the five exposed failure surfaces ranged from 4 to 42 mm, whilst the thickness of the infilling along the rock joints around the site ranges from 3 mm to about 75 mm locally, but generally less than 25 mm. Goodman (1970) has suggested that if the thickness of the infilling is more than 25 to 50% of the amplitude of the asperities, there will be little or no rock to rock contact, and the shear strength properties of the joint will be controlled by the properties of the infilling materials. As the thickness of infilling is slightly greater than 50% of the maximum size of the measured asperities, it is considered that some reduction in the effective friction angle due to the surface roughness should be made to account for this infilling. Assuming that the joint is partly infilled and partly clean in equal proportions, the average effective friction angle will be 40.6° (i.e. the average of 37.5° (determined for the decomposed infilling materials, see Section 7) and 43.6° (derived from Patton's method)) or 44.1° (i.e. the average of 37.5° and 50.6° (derived from Barton's method)). Therefore, the best estimate of the average operative friction angle along the failure surface is considered to be in the range of 40° to 44°.

B.3 DISPLACEMENT ANALYSIS OF THE FAILED ROCK MASS

B.3.1 Approach to the Analysis

The psuedo-static approach for the analysis of problems related to blasting vibration which is characterized by high frequency pulses is known to be very conservative (Wong & Pang, 1992). This is because the approach involves a simple force balance only, assumes a constant static force to apply throughout the period and does not account for the time history or loss of energy through frictional sliding (Dowding & Gilbert, 1988). Therefore, the psuedo-static approach is considered not suitable for the back analysis of the subject failed slope.

For the complete failure of a rock block to occur along a joint surface leading to it sliding down a slope (i.e. infinite downslope displacement), two necessary conditions must be satisfied (Wong & Pang, 1992). First, the induced peak particle velocity must exceed the critical peak particle velocity assessed for the concerned joint surface, and secondly the angle of inclination of the joint surface must be greater than the residual friction angle of the joint surface. At the subject failed slope, the second condition is obviously not satisfied as the inclination of the five failure surfaces found are shallow-dipping, with average dip angles ranging from 16° to 29° with an average of about 25°, which are appreciably lower than the residual friction angle of 33.5° for the failure surfaces. Therefore, it became necessary to assess whether or not the cumulative displacement of the failed rock mass due to blast vibration alone could amount to such a value that the centre of gravity of the failed rock mass could go beyond the lower limit of the sliding surface, lost its balance and failed. For this to occur the cumulative displacement of the rock mass must be in the order of a few metres as the length of the largest detached rock block in the direction of the failure is about 6 to 10 m.

B.3.2 Ground Vibration at the Failed Slope

The minimum distance between the centre of gravity of the failed slope and the nearest blast hole (charged with 3 kg of explosive) in Blasting Area 'A' is estimated to be about 6 m, thus giving a minimum scaled distance of about 3.5 $m/kg^{0.5}$. Assuming that a linear extrapolation of the ground vibration monitoring data collected from the site (see Figures 10 and 11) to such a short scaled distance is valid, the mean PPV and PPA are estimated to be about 100 mm/s and 10 g respectively. At a confidence interval of 68%, these values could increase to an upper limit of about 140 mm/s and 18 g respectively.

The critical peak particle velocity (PPV_c), the exceedence of which post-peak joint displacement would occur, was determined in accordance with the method suggested by Wong & Pang (1992). For JRC values of 4 to 8, friction angles of 40 to 44° and an average joint length of 6.7 m obtained for the failure surfaces dipping at an average angle of 25°, the PPV_c is estimated to be about 100 to 130 mm/s. The results are shown in Figure B2. As can be seen, the PPV_c can be exceeded within the possible ranges of the above parameters.

The distance between the failed slope and the blast holes (each charged with 21 to 22 kg of explosive) in Blasting Area 'B' is about 130 m, thus giving a scaled distance of about 28 $m/kg^{0.5}$. The corresponding mean PPV and PPA were estimated to be about 7.7 mm/s and 0.5 g. As can be seen, the contribution of the effects of the blasting in Area 'B' to the failure was negligible as compared with the blasting in Area 'A'.

B.3.3 Cumulative Displacement of the Failed Rock Mass

The estimation of the displacement of the failed rock mass was made using the graphical method suggested by Hencher (1981), the principles of which are illustrated in Figure B3. Newmark (1965) first proposed a model for the computation of the displacement of a sliding block due to ground vibration. Based on the same principles, Sarma (1975) later derived mathematical equations for the calculation of the displacement of a sliding block induced by earthquake. However, in using these equations a negative normal reaction will result at the base of the sliding block under a high ground acceleration encountered at the subject failure site. This is physically not possible and hence the method is not suitable for direct calculation of the displacement of the failed rock mass for the present failure. The graphical method suggested by Hencher (1981) which could easily account for the above situation was therefore adopted for the displacement estimation.

Assuming the peak ground acceleration attained at the failed slope during blasting event to be 10 g (see Section B.3.2) and the friction angle of the sliding plane to be conservatively equal to 33° (i.e. close to ϕ_r) the cumulative displacement of the failed rock mass when subjected to a horizontal and sinusoidal ground acceleration at that peak value and at a frequency of 30 Hz for a duration of 1.8 seconds (see Section 4.3) is estimated to be about 800 mm (Figure B4). The estimation is considered to be conservative as the ground acceleration is assumed to reach the peak value (i.e. at 10 g) in every cycle, which does not happen in reality. Moreover, displacement is assumed to take place in the downslope direction only but not in the upslope when the velocity of the rock mass changes sign and moves in the upslope direction. In reality upslope block displacement is possible if the acceleration in the upslope direction is sufficiently high to exceed the critical acceleration in that direction (Dowding & Gilbert, 1988). Unlike earthquake ground motions, blasting vibration is characterized by short duration high frequency pulses, which according to Mines & Quarries Division's records have a frequency content ranging typically from 30 to 100 Hz (Wong & Pang, 1992). For the same maximum peak particle acceleration, the higher the frequency, the lower would be the displacement (Dowding & Gilbert, 1988). For sinusoidal acceleration, it can be deduced that the displacement is inversely proportional to frequency. At a frequency of 100 Hz, the cumulative displacement will be reduced by more than two-third. Also, at increasing distance from the point of blasting, vibration of higher frequency will gradually be filtered out and lower frequency content will become dominant. Therefore, adopting a frequency of 30 Hz for the estimation of displacement is conservative.

The above estimation of the displacement has been repeated for the ground acceleration to act in a critical direction (i.e. at an angle of $\theta = (\phi' - \beta)$ above the horizontal, where β is the inclination of the sliding plane) as suggested by Sarma (1975), and the results are also shown in Figure B4. As can be seen, the direction of the acceleration has little effect on the displacement. The same conclusion was reached by Sarma (1975).

From the above calculation, it can be seen that there is a low probability that displacement due to ground vibration alone can amount to an order of 3 to 5 m during the event, thus causing the complete failure of the rock mass. Some other factors, such as the effects of blast-induced gas pressures, might have come into play. Delayed gas pressures can produce large permanent block displacement (Dowding & Gilbert, 1988). However, there are no data available to quantify these effects and to make any meaningful analysis for this failure.

Although as demonstrated in the above analysis that there is a low probability for the bulk of the rock mass to fail due to ground vibration alone, this does not preclude the possibility that there might be failure of smaller pieces of rock which could be located on steeply inclined surfaces above the main shallow-dipping sliding surfaces triggered by blast vibration. However, this could not be proved or otherwise from the available information.

B.4 REFERENCES

Barton, N. (1990). Scale effects or sampling bias. Proceedings of the First International Workshop on Scale Effects in Rock Masses, Loen, Norway, (ed. P. da Cunha), Balkema, Rotterdam, pp 31-55.

Barton, N. & Choubey, V. (1977). The shear strength of rock joints in theory and practice, Rock Mechanics, 10, pp 1-54.

CANMET (1977). Mechanical Properties. Pit Slope Manual, Chapter 3, Canada Centre for Mineral and Energy Technology, 87 p.

Dowding, C. H. & Gilbert, C. (1988). Dynamic stability of rock slopes and high frequency travelling waves. Journal of Geotechnical Engineering, American Society of Civil Engineers, vol. 114, pp 1069-1088.

Evans, N. C. & Irfan, T. Y. (1991). Landslide Studies 1991: Blast-induced Rock Slide at Shau Kei Wan. Special Project Report SPR 6/91, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, 115 p.

Goodman, R. E. (1970). The deformability of joints. Determination of the In-Situ Modulus of Deformation of Rock, Special Technical Publication 477, ASTM, Philadelphia, pp 174-196.

Hencher, S. R. (1981). Friction parameters for the design of rock slopes to withstand earthquake loading. Proceedings of the Conference on Dams and Earthquake, London, pp 79-87.

Newmark, N. M. (1965). Effects of earthquakes on dams and embankments. Géotechnique, vol. 15, pp 139-160.

Patton, F. D. (1966). Multiple modes of shear failure in rock. Proceedings of the First International Congress on Rock Mechanics, Lisbon, pp 1411-1420.

Sarma, S. K. (1975). Seismic stability of earth dams and embankments. Géotechnique, vol 25, pp 743-761.

Wong, H. N. & Pang, P. L. R. (1992). Assessment of Stability of Slopes Subjected to Blasting Vibration, (GEO Report No. 15). Geotechnical Engineering Office, Hong Kong, 111 p.

LIST OF TABLES

Table B1 - Characteristics of the Sliding Surfaces

Set No.	Type	Dip/ Direction in degrees (Range)	Spacing (m)	Persistence (m)	Unevenness (Small Scale)	Waviness (Large Scale)	Roughness Angle (deg.)		Weathering State and Infill	Joint Roughness Coefficient (JRC)	Joint Wall Compressive Strength (JCS) MPa (avg.)
							Large Scale	Small Scale			
5a	Joint	16/221 (±6/±23)	1.5	5	Smooth to rough, occasionally stepped	Planar to undulating	6	6	Stained, locally highly to completely decomposed	3 to 16	15 to 69 (32)
5b	Joint	29/215 (±10/±12)	2	Over 10	Smooth to rough, occasionally stepped	Planar to undulating	10	14	Stained, locally fresh to moderately decomposed	5 to 20	32 to 94 (57)
5c	Joint/ Failure Plane of Largest Block	25/252 (±14/±22)	4	Over 10	Smooth to rough, occasionally stepped	Planar to undulating	14	14.5	Stained, locally fresh to highly decomposed with thin kaolin	8 to 20	20 to 150 (56)
5d	Joint	21/250 (±8/±12)	1.5	4	Smooth to rough, occasionally stepped	Planar to undulating	8	8	Stained, locally fresh to moderately decomposed	3 to 10	32 to 70 (55)
5e	Joint	26/243 (±3.5/±11)	2	4.5	Smooth to rough, occasionally stepped	Planar to undulating	3.5	8	Stained, locally with thin kaolin and completely decomposed infill	1 to 12	33 to 70 (47)

Table B2 - Effective Friction Angle ϕ' of the Sliding Surfaces Determined in Accordance with Barton's (1990) Formula

Parameter	Value
Joint Compressive Strength (JCS_o)	30 to 62 MPa
Joint Roughness Coefficient (JRC_o)	7.8
Average Natural Block Size (L_n)	6.7 m
Average Large Scale Roughness Angle (waviness) (i)	8.5°
Basic ('Residual') Friction Angle (ϕ_r)	33.5°
Average Normal Stress (σ)	100 to 150 kPa

Barton's (1990) modified joint shear strength formula, where L_o = laboratory size joint sample (100 mm nominal)

$$\phi' = JRC_n \log [JCS_n/\sigma] + \phi_r + I = (7.6° \text{ to } 8.9°) + \phi_r + i = (7.6° \text{ to } 8.9°) + 33.5° + 8.5° = 49.6° \text{ to } 50.9° \quad (\text{for } \sigma = 150 \text{ kPa})$$
$$= (8.3° \text{ to } 9.6°) + \phi_r + i = (8.3° \text{ to } 9.6°) + 33.5° + 8.5° = 50.3° \text{ to } 51.6° \quad (\text{for } \sigma = 100 \text{ kPa})$$

$$JRC_n = JRC_o[L_n/L_o]^{-0.02\,JRC_o}$$

$$JCS_n = JCS_o[L_n/L_o]^{-0.03\,JRC_o}$$

LIST OF FIGURES

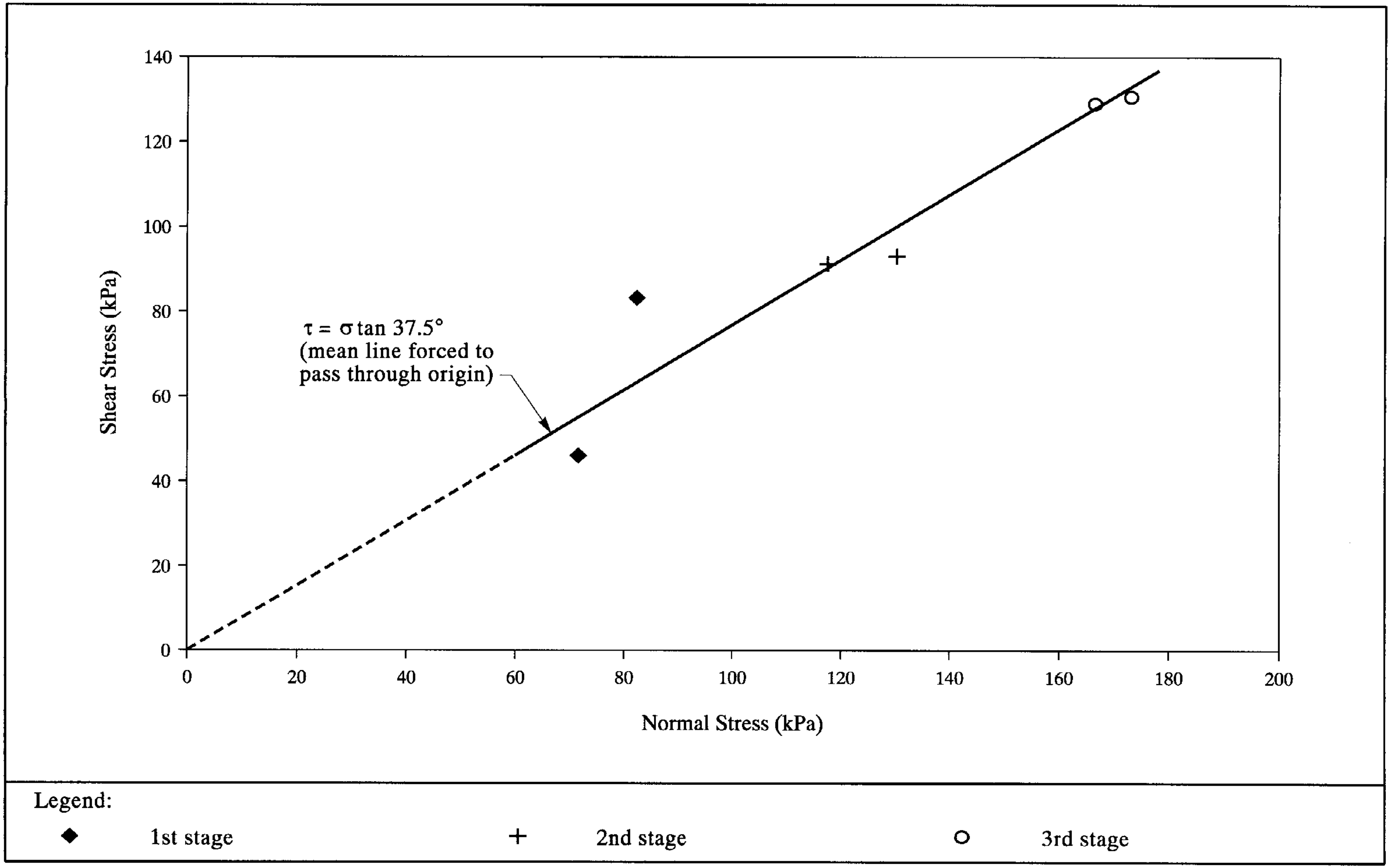
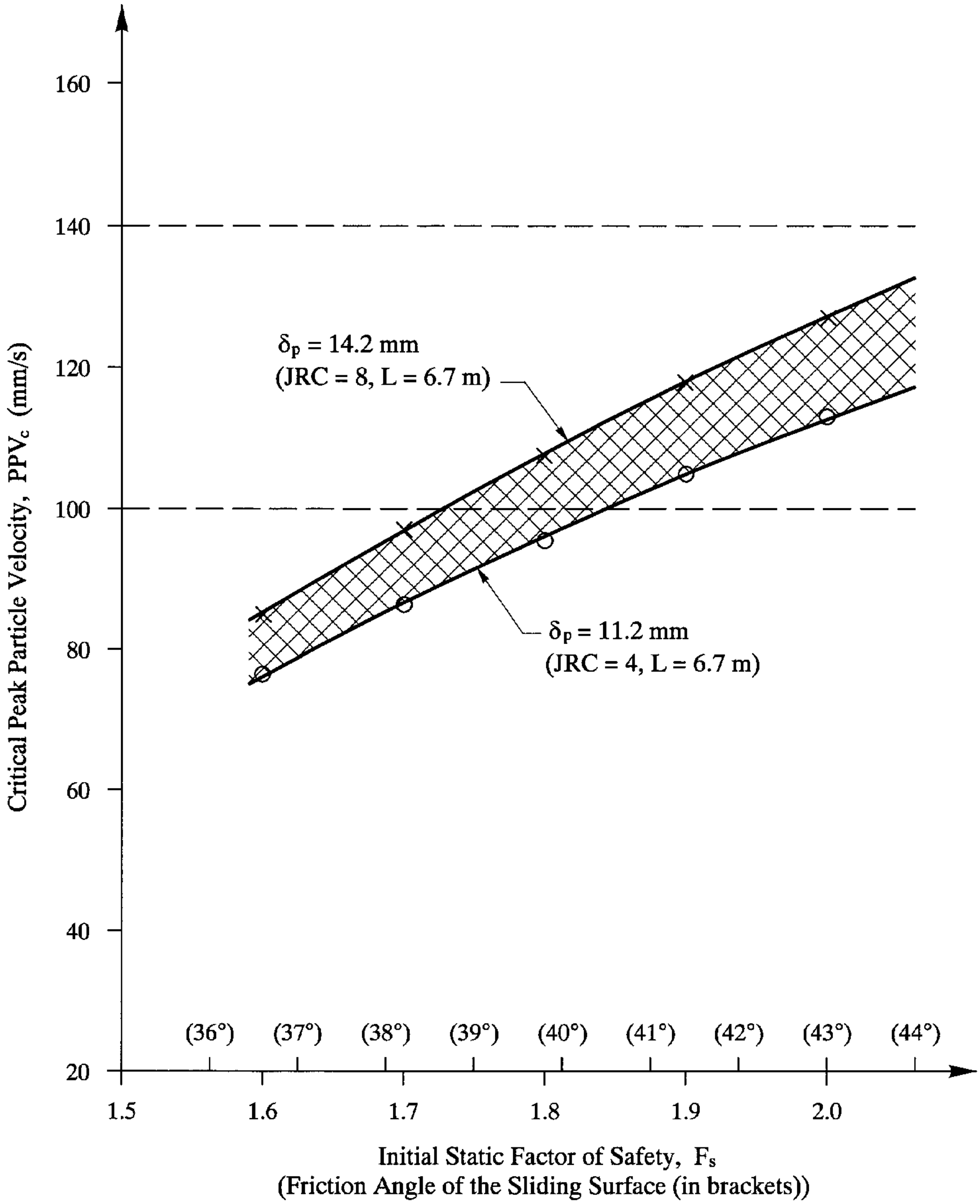


Figure B1 - Results of Multi-stage Direct Shear Tests on Specimens of Decomposed Granite Recovered from the Base of the Largest Block of Rock



Legend:

JRC	Joint roughness coefficient
δ_p	Joint displacement at peak stress = $\frac{L}{500}\left(\frac{JRC}{L}\right)^{0.33}$ (Barton, 1990)
L	Joint length

Note: The inclination of the sliding plane is assumed to be equal to 25°.

Figure B2 - Critical Peak Particle Velocity versus Initial Static Factor of Safety

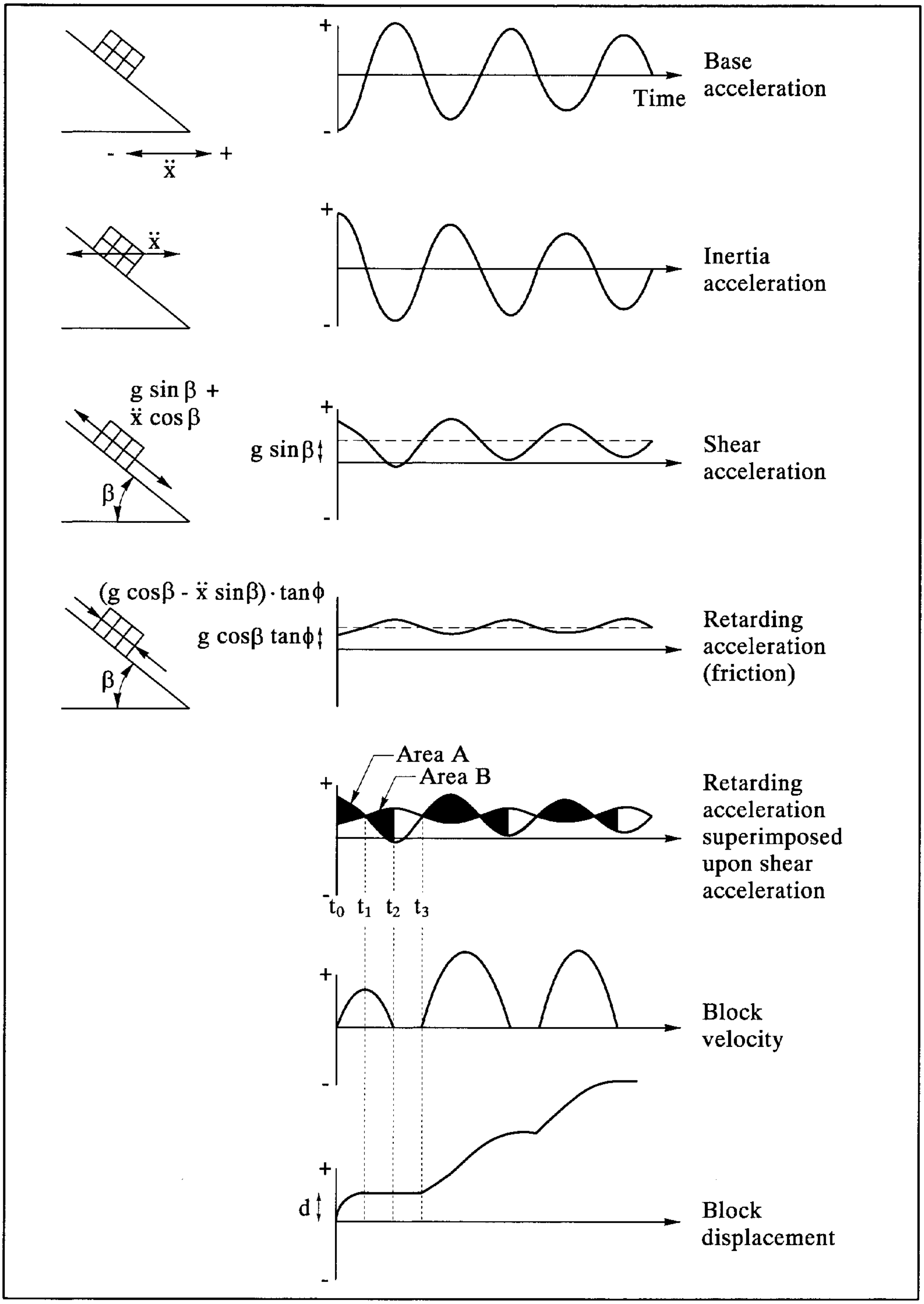
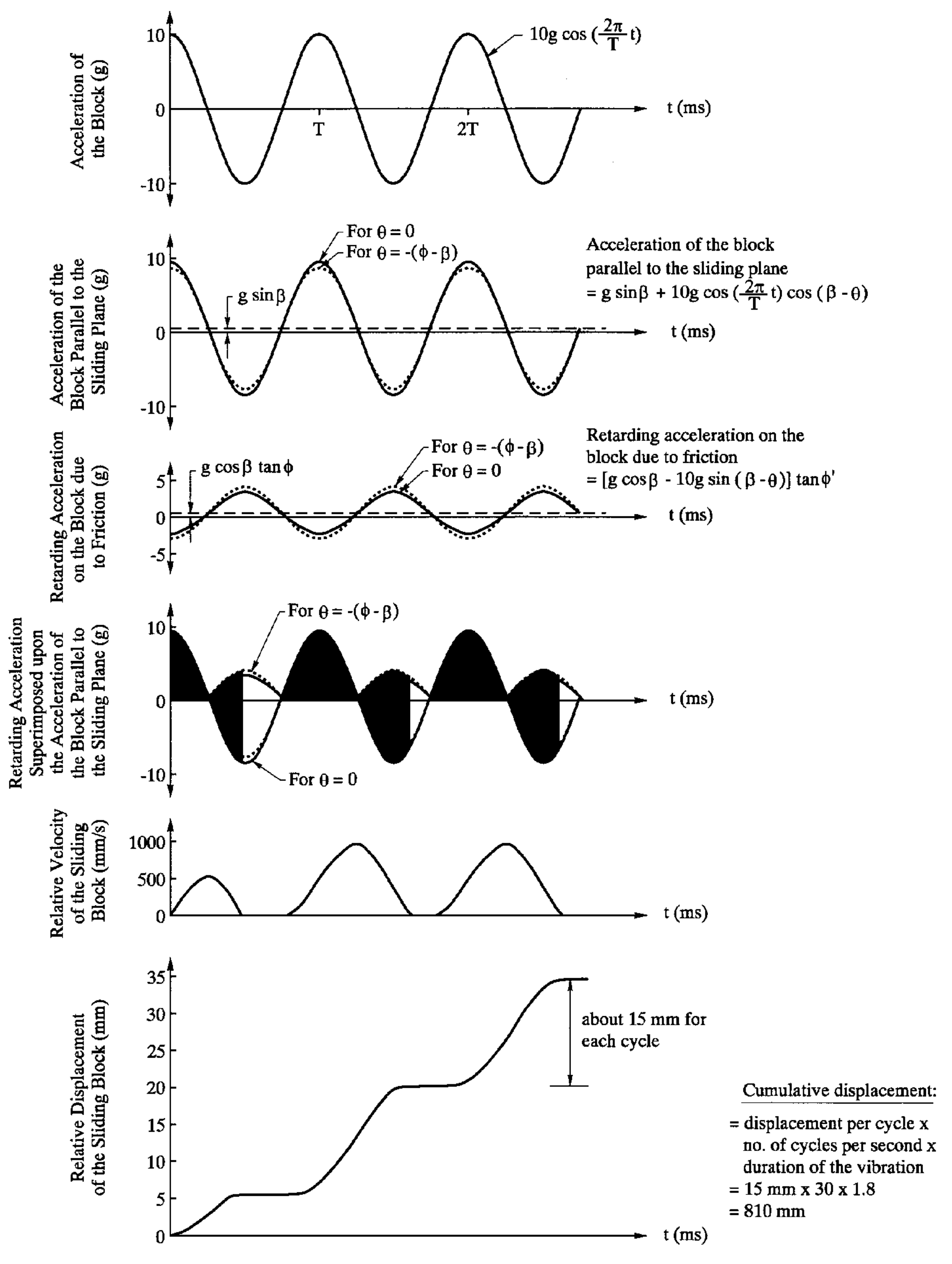


Figure B3 - Cumulative Displacement of a Block on an Inclined Plane Subjected to Horizontal Vibrations (after Hencher, 1981)



Legend:

β	Inclination of the sliding plane to horizontal (assumed 25°)	T	Period of the vibration
ϕ'	Effective friction angle of the sliding plane (assumed 33°)	g	Acceleration due to gravity
θ	Inclination of the direction of the acceleration of the rock block to the horizontal (θ is +ve if the direction points downwards below the horizontal, and θ is -ve if the direction points upwards above the horizontal)		

Figure B4 - Estimate of the Relative Cumulative Displacement of the Rock Mass Subjected to Sinusoidal Acceleration of a Maximum Magnitude of 10 g