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THE GOVERNMENT OF MAURITIUS

THE STUDY ON LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

FINAL REPORT

MAIN REPORT

[illegible]

JAPAN INTERNATIONAL COOPERATION AGENCY

THE GOVERNMENT OF MAURITIUS

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NOVEMBER 1990

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PREFACE

In response to a request from the Government of Mauritius, the Japanese Government decided to conduct a study on the Landslide Protection Project in Port Louis and entrusted the study to the Japan International Cooperation Agency (JICA).

JICA sent to Mauritius a study team headed by Dr. Masasuke Watari, and composed of members from Nippon Koei Co.,Ltd. and Nissaku Co.,Ltd. from April 1989 to July 1990.

The team held discussions with concerned officials of the Government of Mauritius, and conducted field surveys. After the team returned to Japan, further studies were made and the present report was prepared.

I hope that this report will contribute to the promotion of the project and to the enhancement of friendly relations between our two countries.

I wish to express my sincere appreciation to the officials concerned of the Government of Mauritius for their close cooperation extended to the team.

November 1990

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency

November, 1990

Yanagiya Kensuke
President
Japan International
Cooperation Agency
Tokyo

LETTER OF TRANSMITTAL

Dear Sir,

We have the pleasure of submitting to you a Final Report of the Study on Landslide Protection Project in Port Louis prepared for the consideration by the Government of Mauritius in implementing long term protective measures against landslide at La Butte in Port Louis.

This report consists of four separated volumes. The first volume is the Main Report which contains results of the feasibility level study on landslide at La Butte in Port Louis, and detail design level study on designing and cost estimating of the long term protective measures for the landslide. The study results indicate that it is the time to proceed the long term protective measures to stabilize the landslide. The second volume is the Supporting Report which consists of (I) to (III) parts. The part (I) contains field investigation results to support the study results presented in the Main Report. The part (II) contains studies on experimental investigation and emergency protective measures for mitigation of landslide movements. The part (III) contains planning, designing, cost estimate, and economic evaluation of the long term protective measures for stabilization of the whole landslide area. The third volume is the Summary Report which summarizes main results of the study. The fourth volume is the Data Book which contains monitoring data, unit price lists used for cost estimate of the long term protective measures, and core logs of drilled boreholes.

All members of the Study Team wish to express grateful acknowledgment to the personnel of your Agency, Advisory Committee, Ministry of Foreign Affairs, Ministry of Construction, Embassy of Japan to Madagascar as well as officials and individuals of Mauritius for their assistance extended to the Study Team.

In conclusion, the Study Team sincerely hopes that the study results

would contribute to realization of the long term protective measures for stabilization of La Butte landslide and to socio-economic development of the landslide area.

Yours sincerely,

M. Watari

Masasuke Watari
Team Leader
Landslide Protection Project
in Port Louis

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LIST OF MAIN ABBREVIATIONS

GOM:	Government of Mauritius
MOLG:	Ministry of Local Government
MOW:	Ministry of Works
MOH:	Ministry of Housing
MS:	Meteorological Services
MOPL:	Municipality of Port Louis
CEB:	Central Electricity Board
CWA:	Central Water Authority
SMF:	Special Mobile Force
UOM:	University of Mauritius
IMF:	International Monetary Fund
U.K:	United Kingdom
JIS:	Japanese Industrial Standard
Rs:	Mauritian Rupees
J.Yen:	Japanese yen
CBR:	Cost-benefit ratio
NPV:	Net present value
IRR:	Internal-rate-of-return
F.s:	Safety factor
PF.s:	Planned safety factor
Deg:	Degree
Dia:	Diameter
El:	Elevation
Pvc:	Polyvinyl chloride
SPT:	Standard penetration test
CH:	Inorganic clays of high plasticity, fat clays
UU:	Unconfined and undrained

1. INTRODUCTION

1.1 Location

After continuous rainfall in the rainy season from 1987 to 1988, a landslide occurred at La Butte on a lower slope of Signal mountain (323 m) in the southwest part of Port Louis which is the capital of Mauritius. Active movements of the landslide were recognized in June 1987 as damage to structures such as houses, roads, water supply pipes and so on. The active landslide covers an area of about 400 m x 700 m. The lower half of the landslide area is in a highly populated area whereas the upper half is grass land belonging to the government. The location of the landslide including the surrounding areas is shown in Fig.1.1-1.

1.2 Background

The Government of Mauritius designated the district of about 12.5 ha as a high risk area for landslide damages. In the restricted area, 327 houses are packed and 479 families are living. Anticipated potentially endanger area is assumed in the outside lower parts of the landslide area. Total population of the high risk area and the anticipated potentially endanger area is about 3700. More than 50 buildings or houses were demolished because of their serious damages by landslide movements. A mosque built more than 120 years ago and Ecole de la Montagne with 550 students, which is now closed, are included in these demolished buildings. About 1 m differences in level, which cause traffic interference, were created on main roads in the landslide area. Damages caused by landslide movements are shown in the attached photographs of Fig.1.1-2.

A sizable number of houses in the landslide area are damaged by cracks but residents are still living in these houses. Residents in the area are anxious about safety of their houses, landslide movements, and repair of the houses. From these facts the landslide seems to be a large social problem in Mauritius. The Government of Mauritius decided to secure the landslide area by providing protective measures because residents in the area are unwilling to move to other places and it is difficult to find preferable lands to move to in the vicinity.

Based on the condition the first preliminary assessment of the landslide was made by an expert dispatched from the United Kingdom from September to October 1987. According to the assessment results, the Government of Mauritius acted immediately to abandon a water reservoir, to replace water mains with a flexible type, to close an elementary school, and with other measures.

After the completion of the U.K. assessment, five members of a Japanese expert team were dispatched to make preliminary investigation in September 1988 and to install several kinds of monitoring equipment at and around the landslide area. This study covered protective measures against landslide movements, emergency evacuation procedures, and the needs and methods for further investigations.

Based on the results of these studies by the Japanese team, the Government of Mauritius requested the Government of Japan to undertake the necessary investigations for preparation of both urgent and long term protective measures. The "Scope of Works" for the investigation was signed in March 1989 and the investigation was initiated in April 1989.

1.3 Objective of the Study

The objectives of these investigations were:

1) to clarify movement and cause of the landslide in La Butte area; to carry out investigations of topography and geology, and to install various monitoring devices for measurement and analysis.

2) to plan long-term protective measures including non-structural measures against a large scale landslide.

3) to plan urgent protective measures to deal with situations such as cracks and uneven settlement as are seen in parts of the area.

4) to transfer the technology to Mauritian counterparts throughout

the entire operation.

1.4 Study of the Project

The study was divided into two stages each of which was further divided into two phases. The study was initiated in April 1989 and is scheduled to be completed at the end of November 1990 by completion of the final study report.

The study was organized as shown in Table 1.1 to comprise four parties: the JICA Study Team, Mauritian Technical Team, Japanese Advisory Committee, and JICA Coordinator.

1.4.1 The first stage study (April - August 1989)

The phase-1 of the first stage from April to July 1989 consisted of field investigations including ground survey, geological investigations, installation of monitoring equipment, and collection and analysis of data from monitoring equipment.

The phase-2 from July to August 1989, included geological investigation, collection and analysis of data from the monitoring equipment, and preparation of an experimental investigation plan.

An interim report was prepared on completion of the phase-2 to document all the geological investigation results, and all collected data from monitoring equipment. Planning and design for an experimental investigation for mitigation of the landslide movements and a cost estimate were included into the interim report.

1.4.2 The second stage study (September 1989 - November 1990)

The phase-1 of the second stage from September 1989 to March 1990 mainly consists of construction of experimental investigation, soil removal works, and construction of counterweights. The phase-2 of the second stage from April to November 1990 mainly consists of construction of remained parts of the experimental investigation and preparation of a final report of this study.

Based on the planning and design for the experimental investigation, construction works of one 20 m deep drainage well and 1000 m of drainage boreholes (20 holes) was completed in phase-1 of the second stage. Drawings of the drainage well are shown in Fig.1.4-1.

For improvement of overall stability, soil removal works were planned and executed in parallel with progress on the experimental investigation. About 50000 m^3 of soil were removed from the central upper part of the landslide area to improve landslide stability by decreasing weight on the upper slope.

Improvement of safety factors of 3 % to 8 % were expected in the surrounding area by the soil removal works. However, the expected safety factors after soil removal works will not be sufficient to secure a safety factor of 1.2, which is considered to be necessary as a general long term protective measure. This insufficiency of the safety factor has to be compensated by steel piling works.

A small landslide 60 m x 80 m in size occurred in the upper part of the soil removal area. For mitigation of instability of this landslide, planning and design of counterweighting were prepared and performed, to bring it into a stable condition. However, this small landslide could be reactivated by continuous heavy rainfall.

About 400 m of surface water drainage channels were constructed in the upper parts of the landslide area to prevent rain water from infiltrating into the ground. About 1500 m of drainage channels were installed along berms in the soil removal work site for the same purpose. The alignment of drainage channels is shown in Fig.1.4-2.

In phase-2 of the second stage, the remaining experimental investigations were continued and completed at the beginning of July 1990. These comprised construction of one intermediate well 15 m in depth, drilling 120 m of drainage boreholes and installation of drainage facilities.

Collection and analysis of the monitoring data were continued throughout this period to clarify the mechanism of the landslide movements.

1.5 Planning of a Long Term Protective Measures

Planning and design of long term protective measures were made on the basis of the results obtained from this study. For stabilization of landslides many kinds of protective measures or combination of them are employed. In this case lowering of groundwater levels and ground reinforcement by steel piling works were considered to be appropriate.

The main protective measures consist of three (3) drainage wells with the drilling of 2100 m of groundwater collection boreholes for drainage of deeper groundwater, drilling of horizontal boreholes from the ground surface for drainage of shallower groundwater, and 8996 m of steel piling works.

The required work period for construction of the long term protective measures is estimated to be 22 months including preparation of tender documents, evaluation of tenders, mobilization for the construction work, and main construction work.

1.6 Cost Estimate and Project Justification for the Long Term Protective Measures

The total project cost is estimated to be about Rs.272.3 million. at the price level of January in 1990. The exchange rate applied for the cost estimate is one United States dollars equivalents to Mauritian Rs.15.3 and Japanese Yen 146.0.

Project evaluation is based on the with-project and without project principle. Project benefits relate to avoidance of potential damage which would be caused by a landslide. Project costs are the required amounts to provide the necessary protective measures.

The total project cost is estimated to be Rs.272.300 million and it is calculated to be Rs.223.99 million after disbursement for 35 years with the assumption of social discount rate to be 10 %, and the total benefit is estimated to be Rs.438.09 mill. Project justification is shown by 1.96 CBR, Rs.214.1 million in NPV, and 47.7 % IRR.

2 FIELD INVESTIGATIONS

2.1 Installation of Monitoring Equipment

Following preliminary studies by JICA expert team, additional monitoring equipment was installed in and around the landslide area in the initial stage of this study. Monitoring data were collected from the following equipment:

EQUIPMENT	No.
Rain gauge	1 unit
Tiltmeters	7 unit
Extensometers	16 unit
Borehole inclinometers	8 boreholes

Rainfall data were collected from a rain gauge installed at Ecole de la Montagne (the elementary school) in the center of the landslide area for collection of reliable rainfall data in the landslide. Tiltmeters were installed at and around the landslide for clarifying the general ground surface movement. Extensometers were installed at the crown, the toe, and the center of the landslide for clarifying tension movements, compression movements, and combined movements respectively. Measurement by borehole inclinometers was carried out in eight drilled boreholes for estimation of depth to slide surfaces. Location of the monitoring equipment is shown in Fig.2.1-1.

2.2 Topographic Survey

For stability analysis, which were carried out afterwards, of the whole landslide area, five observation lines, Line-V, Line-W, Line-X, Line-Y and Line-Z were selected to cross the landslide area from south to north. The observation lines are shown in Fig.2.1-1. Leveling surveys along these observation lines were made for obtaining topographic surface conditions, and for preparing 1:500 scale topographic profiles. A contour map at 1:1000 scale was also prepared for planning of the field investigations and the long term protective measures.

Leveling survey lines, Line-1, Line-2. Line-3 and Line-4, had previously been established along existing roads by the Mauritius government. Line-1 is aligned along the abandoned water main route upper slope of the landslide area. The survey lines are shown in Fig.2.2-1.

In the upper central part of the landslide which coincides with sections 20 and 33 on Line-1, more than 500 mm of subsidence was observed in May 1989. However, the subsidence movement did not continue not only due to decreased rainfalls but also effect of soil removal performed in the central part of the landslide area during the second stage of this study. Results of the leveling survey are shown in Fig.2.2-2.

There is no topographic abnormalities which are considered to imply the possibility to divide the landslide area into several blocks. Tension cracks are observed along the upper margin of the landslide and compressive upheavals of the ground surface are traceable along the lower margin.

In the course of soil removal some leveling points had to be removed and continuous measurement became impossible. Accordingly seven new lines were established along berms created among the excavated slopes in the soil removal area. These observation lines are shown in Fig.2.2-3.

2.3 Geological Investigations

Geological investigation consisted 235 m (10 holes) of borehole drilling for measurement of groundwater levels and 195 m (8 holes) for measurement by borehole inclinometer. Drilling was accompanied by recovery of core samples, and the conduct of permeability tests and standard penetration tests clarifying subsurface conditions.

Subsurface condition of the landslide area were confirmed by observation of the recovered core samples. The landslide area is composed of scree deposits, composed of clayey soil with basaltic gravel, overlying basaltic bedrocks. Permeability coefficients of the scree deposits ranged from $x10^{-1}$ cm/s to $x10^{-5}$ cm/s mostly in the orders of $x10^{-2}$ cm/s to $x10^{-3}$ cm/s, indicating the relatively high permeability of the scree deposits. The quantity of borehole drilling and permeability test results are summarized in Tables 2.3-1 and 2.3-2.

The maximum confirmed depth of the scree deposits is about 38 m, in borehole BV-W3, in the central part of the landslide. The depth to the bedrock surface decreases gradually from the observation W-line to the eastern and western parts of the landslide. Bedrock surface lines show smooth crescent shape in general, implying that the deepest bedrock surfaces are in the central part of the landslide.

The slide surfaces are assumed to be in the scree deposits in general except in some places where the slide surfaces lie on the bedrock surfaces directly. Clayey layers which probably indicate possible slide surfaces were recovered at depths of 15 m to 26 m from boreholes drilled in the upper part of the landslide and at depths of 4 m to 11 m from boreholes in lower part. Dark brownish fat clayey materials were confirmed in the layer between the basaltic bedrocks and the scree deposits in the toe portions of the landslide. Geological profiles are shown in Fig.2.3-1 to Fig.2.3-6 and contour maps of bedrock surfaces and slide surfaces are shown in Fig.2.3-7.

Since bedrock surfaces and slide surfaces appear to be rather smooth features without any clear discontinuity parallel to the direction of the maximum slope inclination, the landslide is considered to depend on only one uniform sliding mass movement. The landslide is therefore treated as such for stability analysis and for planning of protective measures.

2.4 Strength of Clayey Material along Slide Surface

Clayey materials are considered to be created along slide surfaces by repeated slide movements. Earth masses above slide surfaces are believed to slip down from upper to lower slopes with decreasing friction and cohesion of the clayey materials caused by increasing of pore pressure in the earth masses during heavy rainfall. It is therefore important to clarify the strength of clayey materials for verifying stability of landslides.

For laboratory tests clayey materials have to be collected if possible from the slide surface. In this study the assumed slide surfaces did not outcrop around the landslide area.

Although the sliding earth masses are composed of clayey soil with basaltic gravel, the clayey material was mainly observed in a stiff condition. However, development of slickensides was observed very frequently in the drainage well and the soil removal site during their excavation. Generally slickensides are observed typically along slide surfaces and faults and are considered to be created by shearing movements.

The slickensides observed in slide masses in the project site are believed to have been caused by previous sliding movements and then left under intensive sliding stress.

Samples of clayey materials were collected from the clayey masses in the drainage well and the soil removal site because they were believed to have almost the same properties as the expected clayey materials along the slide surfaces.

Samples were collected as undisturbed samples for mechanical tests and as disturbed samples for physical tests. The obtained test results are summarized in Tables 2.4-1 and 2.4-2. The obtained mechanical properties of the clayey materials are summarized as follows:

PROPERTY	VALUE
Cohesion	1.0 - 3.0 t/m^2
Internal friction angle	5.5 - 28.0 deg.

Stability analysis was carried out for improvement of safety factors by providing a drainage well in the experimental investigation. In this stability analysis, the mechanical properties of the clayey materials were estimated to be 1.0 t/m^2 of cohesion; and 6.0 deg. to 9.9 deg. of internal friction angle. These estimated values for the clayey materials are believed to be reasonable assumptions by comparison with those from the laboratory tests.

2.5 Analysis of Monitoring Data

Monitoring data were collected from; one set of rain gauge; seven sets of tiltmeters; sixteen sets of extensometers and eight sets of inclinometers.

2.5.1 Rain gauge

Rainfall data from 1951 to 1987 were obtained from Line Barracks about 0.7 km northeast to the landslide area, indicating 960 mm as the mean annual rainfall for the period. In this period about 80 % of the rainfall was recorded in the rainy season from November to April.

Significant displacement by landslide movement was observed in 1987 after heavy rainfall brought on by a cyclone which attacked Mauritius in May 1987. One rain gauge was installed by the JICA expert team in 1988 at the Ecole de la Montagne in the central part of the landslide. Rainfall data have been collected from this additional rain gauge since 1988. According to the rainfall data so far normal, rainfall is about 1000 mm per year with less in dry years and more than 1300 mm in rainy years. The rainfall in rainy years is generally concentrated in the rainy season from November to April. The rainfall in recent years is shown in Figs.2.5-1 and 2.5-2.

Relatively heavy rainfalls of 1338 mm and 1301 mm was recorded in the rainy season from November to April in 1986 to 1987 and 1988 to 1989. Lower rainfall of 504 mm and 591 mm was recorded 1987 to 1988 and 1989 to 1990. Movement of the landslide was observed in 1987 initially after heavy rainfall. Though there are no monitoring records on landslide movements in 1988, movement was indicated as ground surface subsidence. Continuous subsidence movement was shown in Fig.2.2-2.

The rainfall in the rainy season from November 1989 to April 1990 was small and the landslide movement on the whole landslide area was almost negligible. However, the absence of landslide movements was probably due mainly to the effects of the soil removal works.

2.5.2 Borehole inclinometer

Five boreholes were drilled in the previous stage for measurements by borehole inclinometer. These boreholes, A002, A003, A005, A006 and A007, were drilled more or less straight on the Line-W in 1987 to 1988. However, these boreholes were sheared by slide movement afterwards, and measurement in these boreholes was confirmed to be impossible in May and June 1989 after starting this study. The location of these boreholes are shown in Fig.2.1-1.

Depths where possible displacement was occurred was confirmed to be at 3.0 m, 11.0 m and 27 m in boreholes, A002, A003 and A007. No shearing deformation was observed in boreholes, A005 and A006. The depth of these two boreholes appears to have been insufficient to reach to the slide surface.

In this study eight boreholes totaling 195 m were drilled along the observation lines and guide pipes were installed in the drilled boreholes for measurement by borehole inclinometer. Slight shearing displacements were measured in the rainy season from 1989 to 1990 in boreholes, BV-X3, BV-X4, BV-Z2 and BV-Z3, which were drilled in the eastern part of the landslide area. However, no shearing displacement was observed in the boreholes drilled in the western and lower parts of the landslide. Measured displacement records are shown in Fig.2.5-3 to Fig.2.5-10.

Intensive shearing displacements were measured in the records from BV-X1 drilled in the central upper part of the landslide. The shearing continued at depths of 2.5 m and 8.0 m in the period from November 1989 to February 1990. Shearing displacement reached about 7.5 cm and 5 cm at depths of 2.5 m and 8.0 m in February 1990 when further measurement became impossible because of large deformation of guide pipes.

These large shearing displacements were considered to be caused by the soil removal works. The measurement records are shown in Fig.2.5-5.

2.5.3 Tiltmeter

Tilting movement of ground surface has been measured by seven units of tiltmeters installed in and around the landslide area. No very clear cumulative tendency was observed in the records. However, the records from T-4 and T-6 show a relatively clear cumulative tendency in the N-S direction.

The tiltmeters T-4 and T-6 had been installed in the eastern part of the landslide. Tiltmeters T-3 and T-5 which had been installed in the eastern part of the landslide showed no clear cumulative tendency except during the short period just after the installation of the tiltmeters. Records from tiltmeters T-1, T-2 and T-7 shows stable condition after installation of the tiltmeters. The locations of T-1, T-2, T-7 seem not to have been affected by the landslide movements. Records of the tiltmeters are shown in Fig.2.5-11 to Fig.2.5-13.

2.5.4 Extensometer

Four extensometers were installed by the JICA expert team across tension cracks which had appeared in the uppermost part of the landslide. A further 12 units of extensometers were installed in this study in the toe and central portions of the landslide.

Large displacements were recorded by extensometers along the tension cracks in the rainy season from 1988 to 1989. The maximum displacements of about 250 mm were recorded by E-3 and E-4 in the period from the beginning of December 1989 to the beginning of May 1989. Further displacement ceased with decreasing of rainfall. After this rainy season no intensive displacement was measured. The records of the extensometers are shown in Fig.2.5-14 to Fig. 2.5-15.

Displacement was recorded again from extensometer E-3 in November 1989. This displacement was considered to be caused by a small secondary landslide which developed at and around extensometer E-3. The soil removal works were considered to have triggered the occurrence of this small secondary landslide.

Since the total amount of displacement reached about 120 mm between

October 1989 and January 1990 and displacement seemed to be continuing, use of an counterweight was planned and executed at the end of January 1990 as a protective measure. The displacement was instantly ceased with this first counterweight for a short period. However, displacement was commenced again in March 1990 after heavy rainfall from the middle of February to the beginning of March. For prevention of further displacement, a second counterweight treatment was performed from the end of May to the beginning of June 1990. The displacement was gradually controlled by this counterweight.

However, it seems possible that this small landslide may be accelerated again by continuous heavy rainfall sometime in the future. For protection from this small landslide in the long term, determent works by steel piling works were accordingly considered to be necessary.

2.5.5 Measurement of groundwater level

Measurement of groundwater levels had been conducted in ten boreholes, 235 m in total depth, since the beginning of this study. Groundwater tables could be measured in all boreholes after completion of drilling in June 1989. However, large drawdowns took place in all boreholes except those drilled in the toe portion, and further measurement of groundwater levels became impossible to conduct in many boreholes. Predominant slime sediments also occurred in some boreholes after completion of drilling, and this also made measurement of groundwater levels impossible.

Recovery of groundwater levels after relatively heavy rainfalls had been observed in boreholes, BV-Y2, BV-Y3 and BV-Z4 drilled in the eastern part of the landslide. The same phenomena had not been observed in the boreholes drilled in the western part. Recovery of groundwater levels was observed in BV-W3 and BV-V1 only immediately after the completion of drilling. One of the reasons why recovery of groundwater levels was not observed in the boreholes in the western part seems to have been due to the slime sediments, though there are no actual records of groundwater level recovery. Fluctuation of groundwater levels is shown in Fig.2.5-16.

Stability of the landslide seems to be intensively decreased by increasing of groundwater levels in the landslide area. A peak cut-off function will be intensely required for drainage wells during and after heavy rainfall to ensure the stability of the landslide by decreasing groundwater levels in the landslide area. The drainage well constructed in the experimental investigation is expected to functionate effectively by draining groundwater when the levels are increased after heavy rainfall. The drainage well is considered to be sufficient for draining groundwater in the western part of the landslide. In this respect installation of drainage wells is likely to be more effective in the eastern part rather than the western part.

2.6 Landslide Mechanism

La Butte landslide is a colluvial landslide according to a general classification of landslide types as follows:

1) Bedrock landslide
2) Weathered rock landslide
3) Colluvial landslide
4) Clayey soil landslide

Very thick scree deposits overlie basaltic bedrocks for about 40 m in maximum thickness. Frequently developed slickensides are observed in clayey soil in the scree deposits, implying that intensive sliding stress had been acted on the sliding masses above the slide surfaces of previous times.

The slide surface of the landslide is assumed to be developed in the scree deposits mostly except in some places where slide surfaces seem to touch bedrock surfaces. Geological investigation composed of borehole drilling was made on the five observation lines crossing landslide area from south to north. The slide surfaces are assumed to curve smoothly along the observation lines, according to the investigation results. The maximum depth to the slide surface is estimated to be about 25 m in the central part of the landslide. The depth to the assumed slide surfaces is estimated to decrease gradually from the central part to the western, eastern and southern marginal portions.

The depth to bedrock surfaces is confirmed to be maximum at the central part of the landslide. The maximum depth of about 40 m was confirmed by observation of the recovered core samples from borehole BV-W2. The depth to bedrock surfaces was confirmed to be about 20 m in the western and eastern parts of the landslide. The depth of scree deposits decreases to about 10 m in the toe portion of the landslide. The clayey layer which is about 5 m in thickness is confirmed to be in the layer between the scree deposits and bedrock surfaces in the southern marginal portion.

In regard to groundwater levels in the landslide, measurement of groundwater levels was made immediately after completion of borehole drilling but it has not been possible in some boreholes since the rainy season of 1988/1989 because of considerable drawdown of groundwater levels in the landslide area.

The maximum drawdown of groundwater levels was about 30 m in the boreholes drilled in the central part of the landslide. A drawdown of about 20 m is observed in the western part. On the other hand, it is less than 5 m to 20 m in the eastern part. From these observation results, drainage of groundwater by installation of drainage wells is likely to be more effective in the eastern part than the western part.

Groundwater tables observed in June 1990 were considered to be the levels after intensive drawdown because of low rainfalls in the rainy season from November 1989 to April 1990. On the other hand, groundwater levels after heavy rainfall was considered to be higher than the levels observed in June 1989, and landslide movement might be accelerated again in the future by heavy and continuous rainfall which may cause a large recovery of groundwater level to higher than the level in June 1989.

Depths to bedrock surfaces and slide surfaces are maximum in the central part of the landslide where the maximum drawdown of groundwater levels were also observed. Depths to slide surfaces and bedrock surfaces are assumed to decrease gradually from the central part of the landslide to the marginal portion. Depth to groundwater levels also gradually decreases from the central part to the western and eastern portions.

From the observation results on depths to the slide surfaces, bedrock surfaces, groundwater levels, and location of tension cracks, the landslide that occurred at La Butte is considered to be one landslide, that is, the landslide cannot be divided into several blocks.

The assumed strengths of clayey materials of 1.0 t/m^2 for cohesion and 6.0 deg. to 9.9 deg. for internal friction angle for stability analysis for the whole landslide area are confirmed to be reasonably estimated by the laboratory tests.

3. URGENT PROTECTIVE MEASURES

3.1 Preparation of Urgent Protective Measures

Protective measures for landslides are classified into prevention works and determent works in general. They may be summarized as follows:

LANDSLIDE CONTROL WORKS

Prevention Works:	Surface drain works
	Drain works
	Interceptor wall works
	Groundwater drain work
	  Horizontal borehole drilling from the ground surface
	  Drainage wells
	  Drainage tunnels
	Soil removal works
	Counterweight works

Determent Works:	Pile works Large diameter cast-in-place shaft works Anchor works Retaining wall works

Urgent protective measures planned for mitigation of landslide movement, were included: installation of surface water drainage channels; construction of one drainage well; and soil removal works.

3.1.1 Surface drainage channels

Increased groundwater levels in landslides can suddenly affect stability. Drainage channels are for preventing rain water from infiltrating into the ground. Installation of the surface water drainage channels was conducted in the field investigation period in the upper parts where open tension cracks had been observed.

Additional drainage channels were installed in the period of the soil removal works. The alignment of surface drainage channels is shown in Fig.1.4-2.

3.1.2 Drainage well

Taking into consideration the need for efficient drainage of groundwater, the construction site for the drainage well was selected near Line-W because the depth to slide surfaces or bedrock surfaces is deeper in that area and drainage of groundwater was considered to be most effective.

For assessing the effects of drainage well construction, stability analysis was made on the assumption that the safety factor (F.s) was 1.0 before providing a drainage well in the landslide. For the stability analysis the slice method, which is the most popular for landslide stability analysis, was conducted for all the selected observation lines. For obtaining physical properties of sliding earth masses in the whole landslide area, calculation of the internal friction angle of the sliding surfaces was carried out with assumption of 1.0 t/m^2 for cohesion. Divided slices for the stability analysis are shown in Fig.3.1-1 to Fig.3.1-5. The calculated internal friction angles for each observation line are summarized as follows:

Line	Cohesion (t/m^2)	Internal friction angle (deg.)
V	1.0	9.4
W	1.0	9.7
X	1.0	9.3
Y	1.0	9.5
Z	1.0	9.0

Stability analysis was made along W-line using the calculated value of soil strength parameters and with assumption of 1.8 t/m^3 for the unit weight of sliding earth masses. Values such as force along the normal line, force along the tangential line, and pore pressure of slide surfaces of each slice, and the length of slide surface were obtained by accumulating the values for each slice. In the stability analysis groundwater levels in sliding earth masses are treated as pore pressure.

The drawdown of groundwater levels by construction of a drainage well is expected for about 2 m along W-line. The safety factor was calculated as follows:

	BEFORE CONSTRUCTION OF THE DRAINAGE WELL	AFTER COMPLETION OF THE DRAINAGE WELL
SAFETY FACTOR (F.s)	1.00	1.061

About 6 % of improvement of safety factor is expected by construction of a drainage well along the W-line.

The depth of the drainage well was decided from the depth to the assumed slide surfaces so as not to penetrate the slide surfaces for avoiding possible damages to the well by shearing displacement along slide surfaces. Groundwater surrounding the drainage well was considered to be drained through groundwater collection boreholes. The distance between the drainage well and the ground surface, to which the collected groundwater was designed to be drained out, is about 120 m and drilling of horizontal boreholes with sufficient accuracy was considered to be difficult. Therefore, one intermediated well was required between the drainage well and the ground surface for accurate drilling. The drainage boreholes are therefore divided into two sections accordingly.

The main features of the drainage well works may be summarized as follows:

1) Depth of the drainage well is 20 m.
2) Diameter of the drainage well is 3.5 m.
3) Total length of groundwater collection boreholes is 1000 m (20 holes).
4) Depth of the intermediate well is 15 m.

5) Total length of drainage boreholes is 120 m (2 holes).

3.1.3 Soil removal work

For mitigation of the landslide, installation of drainage channels, and construction of a drainage well with drilling of groundwater collection boreholes were planned to conduct by JICA's expense. However, these urgent protective measures seemed to be insufficient for securing the stability of the landslide if there should again be heavy rainfalls as in the rainy season from 1988 to 1989. While the study was in progress soil removal works were considered to be necessary for reducing the sliding force.

The soil removal works were planned to conduct by expense of the government of Mauritius in the central upper slope of the landslide, and covering an area of about 100 m x 170 m. The soil removal work was designed to provide five berms of 3 m in width at every 3 m height interval in the zone between El.40 m and El.60 m. The location and design of the soil removal work site is shown in Fig.3.1-6.

3.2 Experimental Investigation

As one urgent protective measure, an experimental investigation including construction of a drainage well and an intermediate well, drilling of groundwater collection boreholes and drainage boreholes, and installation of a drainage facility were planned and executed during this study.

3.2.1 Design of the drainage well and the intermediate well

Design of the experimental investigation was prepared by following the general design concepts as follows:

1) Diameter of the drainage well is 3.5 m.
2) Depth of the drainage well is 20.0 m.
3) Diameter of the intermediate well is 3.5 m.
4) Depth of the intermediate well is 15 m.
5) Length of groundwater collection boreholes is 1000 m (20 holes).
6) Length of drainage boreholes is 120 m.

The stress which is expected to act on the drainage well structure is considered to be only earth pressure. For finding out the earth pressure, the calculation method based on the static earth pressure was adopted. According to the calculation results the static earth pressure is expected as follows depending on the well depth. The relationship between design earth pressure and well depth is summarized in Table 3.2-1.

Depth of Well	Static Earth Pressure
15 m	13.5 t/m^2
20 m	18.0 t/m^2

The wells were designed to be protected by steel liner plates of 2.7 mm in thickness with steel stiffeners of 125x125x6.5 mm in size. The strength of the well structure was examined against buckling pressure and compressive stress. Calculation results indicated that a drainage well made of perforated steel liner plates of 2.7 mm in thickness plus stiffener rings at 2 m interval would be sufficiently strong for a depth of 20.0 m taking compressive stress and bending moments into consideration. The structure of the drainage well is shown in Fig.1.4-1.

Examination of the strength of the intermediate well was also performed by the same procedure as for the drainage well.

3.2.2 Material for the wells

Materials for the wells have to be of sufficient strength against the expected earth pressure. The main materials for the wells are steel liner plates, stiffeners, bolts and nuts, steel covers and ladders. The required specification and quantity are summarized in Table 3.2-2. For construction of the wells, materials which are available locally were to be utilized as far as possible, and the availability of the materials was confirmed through examination of local materials.

3.2.3 Plan of construction of the wells

The procedure for well construction may be summarized as follows:

1) Preparatory works
2) Transportation
3) Temporary works
4) Installation safety facility
5) Main construction works
6) Transportation (including dismantling)
7) Clearing works

Outline of the construction works process is shown below.

1) Excavation Works:	Excavation and mucking
2) Metal Works:	Liner plate installation
3) Concrete Works and Plate Fixing:	Excavation and concreting

4) Drilling of Groundwater Collection Boreholes: Machine installation
5) Drainage Channel Construction: Excavation and masonry works
6) Concrete Works: Concrete placing
7) Related works:

Total working period for the experimental investigation was estimated to be 14 months. Working days are counted to be 30 days for a month and working hours are counted to be 7 hours for a day. However, the actual works including all related works were completed within 10 months about 4 months shorter than the expected work period. This shortened work time was largely due to the relatively low rainfalls in the rainy season from 1989 to 1990.

3.2.4 Cost estimate for the experimental investigation

The main construction cost for the experimental investigation was composed of direct construction cost, indirect construction cost, and general supervision cost. The exchange rate was based on the rate in May 1990 as follows:

US$ 1.00 = J.Yen 135.0 = Rs. 15.3

Basic unit price was classified into labor cost unit price, materials unit price, and equipment unit price. These prices were accumulated separately for local supply and supply from abroad.

Total construction cost was estimated to be about J.Yen 91.4 mill. for all required works.

3.2.5 Execution of experimental investigation

For mitigation of the landslide movement by decreasing groundwater levels in the landslide area, the experimental investigation works were planned in the central part of the landslide area. The experimental investigation was started in September 1989 and completed in June 1990.

Excavation of the drainage well and the intermediate well was made 0.5 m at a time with excavation and installation of steel liner plates of 0.5 m in height alternately. After completion of excavation to the target depth, the base concrete was made at the bottom of the wells for protection from water leakage through the well bottoms.

Groundwater collection boreholes were designed to be drilled into the mountain side in a fan shape with a lateral opening angle of about 10 deg to 15 deg. The boreholes were drilled into the mountain side at an angle of elevation of 0 deg. to 5 deg. for effective groundwater drainage. The groundwater collection boreholes were drilled to a diameter of 66 mm. The arrangement of the groundwater collection boreholes is shown in Fig.3.2-1 to Fig.3.2-2.

After completion of the drainage well, groundwater collection boreholes, and the intermediate well, drilling of drainage boreholes was conducted from the intermediate well in the direction of the drainage well and the ground surface where collected groundwater had to be drained. For securing smooth gravitational drainage of collected groundwater from the drainage well to the ground surface, the drainage boreholes were inclined to be 0.8 - 2.0 deg. The profile along drainage boreholes is shown in Fig.3.2-3.

The maximum length of accurate horizontal drilling is considered to be about 50 m in general. The location of the intermediate well was decided to be at 45 m from the drainage well accordingly. On the other hand, the section between the intermediate well and the ground surface was decided to be 75 m because the accuracy for drilling was not required so precisely as for the section between the drainage well and the the intermediate well. After completion of drilling of the drainage boreholes, steel pipes of 114.3 mm in diameter were installed for protection of the boreholes. The alignment of drainage boreholes are shown in Fig.3.2-4.

The drainage facility mainly consists of an open channel for about 18 m which was installed for smooth drainage of groundwater from the drainage well to ditches along existing roads. A drawing of the drainage facilities is given in Fig.3.2-5.

3.2.6 Subsurface conditions

Earth materials composed of clayey soil with basaltic gravel were encountered during excavation of the drainage well and the intermediate well and drilling of groundwater collection boreholes and drainage boreholes. Inclusion of gravel materials increased slightly beyond a depth of 10 m. Clayey materials became rather sandy in the deeper sections and dry in general. Subsurface conditions surrounding the drainage well and the intermediate well are shown in Fig.3.2-6 to Fig.3.2-7.

Slickensides were frequently observed to the full depth of the drainage well and the intermediate well, implying intensive sliding stress had been acting on the earth masses above the slide surfaces in the landslide area.

3.2.7 Effect of the experimental investigation

The drainage well works were conducted for mitigation of landslide movement by effective drainage of groundwater from the surrounding zone of the drainage well. After the rainy season from 1988 to 1989, in which total rainfall reached 1301 mm, groundwater levels in the landslide area were greatly reduced and no recovery of the groundwater levels was observed even in the rainy season from 1989 to 1990 in which the total rainfall amount was only 591 mm. The groundwater levels in the borehole drilled from the bottom of the drainage well and in the borehole, BV-W2, which is located most close to the drainage well, are shown in Fig.3.2-8, with comparing the groundwater levels in 1989 with 1990.

Fig.3.2-8 indicates that the groundwater level in June 1990 was more than ten meters deeper than the levels of groundwater collection boreholes. Therefore, it is understandable that the groundwater was not able to drain after completion of the drainage well. However, groundwater levels is considered to be recovered remarkably in and after heavy rainfalls as higher groundwater levels were recorded in June 1989.

Taking the higher permeability of the sliding earth masses ranging mainly from $x10^{-2}$ cm/s to $x10^{-3}$ cm/s into consideration, the drainage well is believed to be effective for draining groundwater from the landslide area. This fundamental concept is applied for planning of the drainage works for the long term protective measures.

3.3 Soil Removal Work

In a landslide area upper earth masses always acts on the lower earth masses to push them down along slide surfaces until attaining equilibrium. Reduction of weight from upper slopes in landslides is therefore often effective in mitigating landslide movement generally. Accordingly soil removal was proposed in this study.

3.3.1 Design for soil removal work

The designing for the soil removal was prepared by the study team and the execution of the work was done by a selected contractor at the expense of the government of Mauritius. The main excavation works were performed between the end of October and the beginning of December, 1989.

The soil removal site was chosen between El.=40 m and El.=60 m in the central upper part of the landslide area. Cut slopes was designed to be 1.0 to 1.5 (horizontal to vertical) with three meter wide berms between excavated slopes. The excavated slopes were designed on a semicircular curve of about 120 m diameter to fit in smoothly with the natural mountain slopes. A large half circular flat area was created at the bottom of the soil removal site.

About 50000 m^3 of earth was excavated in accordance with the design. The excavated materials were transported to bare land facing Caudan basin in Port Louis harbor, a distance of less than 1 km in a crow line.

3.3.2 Stability analysis for soil removal work

Stability analysis was done on W-line and X-line which cross the soil removal area from south to north. The stability analysis was based on the slice method using the values assumed and calculated in the landslide stability analysis for the experimental investigation. The obtained safety factors may be summarized in the below table. In the table improvement of safety factor after experimental investigation, which includes construction of one drainage well, is shown also.

Since the experimental investigation is expected to affect on the observation W-line. The expected total improvement of safety factors are 9.2 % and 7.1 % for the W-line and X-line respectively.

IMPROVEMENT OF SAFETY FACTOR BY SOIL REMOVAL WORKS

Observation Line	Before Soil Removal	After Soil Removal	After Experi. Investigation	Total F.s
W	F.s=1.00	F.s=1.031	F.s=1.061	1.092
X	F.s=1.00	F.s=1.071		1.071

3.3.3 Effect of soil removal work

The stability of the landslide seems to have benefited from the combined effects of low rainfalls and the protective measures, though it is difficult to evaluate the effect of the protective measures on their own. Some indication can be observed however from the continuous records of the leveling survey along the line-1 which follows the abandoned water supply pipe line which crosses the upper part of the landslide area. Leveling survey data are available from the end of 1987 and the survey results are shown in Fig.2.2-2 and Fig.3.2-9.

Fig.3.2-9 indicates a large subsidence of ground surface in the section between survey points, No.28 and No.33 in the period between the end of 1987 and the middle of 1989. The rainfall was very small, 504 mm in the rainy season from 1987 to 1988 but remarkable subsidence had been continued until the middle of 1989. On the other hand, subsidence was disappeared or minimized in the rainy season from 1989 to 1990 in which the rainfall was very small again, 591 mm. From these facts subsidence of the ground surface was concluded to be minimized by the soil removal work.

Monitoring records from the extensometers do not indicate any clear sliding movement of the landslide as a whole. However, extensometer E-3 indicates a significant displacement caused by a small secondary landslide. The stability of the whole landslide seems therefore to have benefited not only from the relatively small rainfall in the rainy season 1989 to 1990 but also from completion of the soil removal works.

3.4 Emergency Counterweight

3.4.1 Small secondary landslide development

Monitoring records from extensometer, E-3, showed intensive displacement after the start of the soil removal works. This displacement was confirmed to be a secondary development from the main slide surface from measurement records of the borehole inclinometer in borehole, X-1, located in the slope below E-3. Remarkable displacement of open cracks in the area surrounding E-3 and continuous protrusion of soil layers, which seemed to coincide with the secondary slide surface, in the excavated slope between El.=49.0 m and El.=52.0 m in the soil removal area indicated the occurrence of intensive movements in the area surrounding E-3.

Displacement amount of about 1 mm/day to 3 mm/day was recorded by E-3 at the end of October 1989 of the soil removal works. It then increased to about 120 mm by the end of January 1990. The displacement is shown in Fig.2.5-14. From interpretation of borehole inclinometer records, it was assumed that there were two secondarily developed slide surfaces in that area. Shearing displacements were observed in borehole, BV-X1, at 5.0 m and 10.5 m in depth. The displacement by shearing reached about 50 mm and 75 mm at depths of 5 m and 10.5 m. The displacement records are shown in Fig.2.5-5. This small landslide occupies an area 60 m x 80 m in size.

3.4.2 Execution of counterweight

For mitigation of landslide movement by this small landslide, a protective measure by counterweight was planned. This was implemented on the berm at El.=46.0 m in the soil removal work site at the end of January 1990. Volume of the counterweight embanked was about 655 m^3.

According to monitoring records from E-3, movement of this small landslide ceased instantly after provision of this first counterweight. The landslide remained stable for about a half month but movement started again at the end of February 1990 after relatively heavy rainfall from the middle of February to the beginning of March 1990. Measurement by borehole inclinometer became impossible in borehole BV-X1 because the probe of the borehole inclinometer was not able to enter the borehole due to the large shearing deformation in the guide pipes. Very intensive displacement of about 200 mm was recorded within one month from the beginning of March to the beginning of April 1990.

In May a second counterweight was planned for mitigation of this accelerated displacement, and this was implemented between the end of May and the beginning of June 1990. The counterweights were constructed on the berms at El.=49.0 m and El=52.0 m in the sizes of about 485 m^3 and 154 m^3. After completion of these counterweights, sliding gradually ceased. Drawings of the counterweights are shown in Figs.3.2-10 and 3.2-11. The sizes of counterweights is summarized in Table 3.4-1.

3.4.3 Stability analysis

Before constructing the counterweights, stability analysis was made to determine the appropriate sizes for the counterweights. Two secondarily developed slide surfaces A and B were estimated in the lower slope of E-3. The first counterweight was to counteract sliding along slide surface A and the secondary counterweights were to counteract sliding along slide surface B.

Stability analysis was done on the assumption of a safety factor to be 1.0 (F.s=1.0) at the start of the small landslide with the unit weight and cohesion of sliding earth masses to be 1.8 t/m^3 and 1.0 t/m^2. After finding out the internal friction angle by using these assumed values, stability analysis was made for slide surface A after completion of the first counterweight. The obtained safety factors for the slide surfaces A and B were 1.094 and 1.103. Improvement of the safety factor was expected to be about 10 %. this was considered to be sufficient for short term protection.

However, additional sliding along the assumed slide surface B occurred after completion of the first counterweight after continuous heavy rainfall. The calculated safety factor for slide surface B is therefore considered to be rather overestimated. Stability analysis was repeated to determine an appropriate value for the internal friction angle of slide surface B with assumption of a safety factor of 1.0 for the slide surface in the condition after completion of the first counterweight. Through these procedures a safety factor of 1.119 was obtained for slide surface B after completion of the first and second counterweights. The results of the stability analysis are summarized in Table 3.4-2.

Movement of the small landslide diminished after completion of the counterweights and the stability analysis results indicated that the counterweights had been properly made as a short term protective measure and that the landslide was in a stable condition. However, further movements of the small landslide could occur in the future after continuous heavy rainfall. Determent works such as steel piling works for the small landslide are considered to be necessary on a long term protective measure.

4. LONG TERM PROTECTIVE MEASURES

4.1 Planning of Long Term Protective Measures

4.1.1 General planning

Planning of long term protective measure for La Butte landslide is the final target of this study. Full account must be taken of all collected data from the field investigation results, the experimental investigation results, and the results of the urgent protective measures conducted during the study.

Protection against landslide movements may be classified into prevention works and determent works. Prevention works, which include surface drainage works, subsurface drainage works, soil removal works, and counterweight works, aim to control landslide movements by improving natural condition. Determent works, which include pile works, large diameter shaft works, anchor works, and retaining wall works, aim to control landslide movements by reinforcing the landslide area by providing steel piles and anchors. The most suitable approach for an individual landslide must be decided according to actual site condition.

For planning of the long term protective measures for La Butte landslide the main points of the concepts are as follows:

1) Landslide movements depend on only one united mass movement i.e. the landslide is not divided into several blocks because any clear feature which imply to divide the landslide into several blocks was not confirmed by topographic and geological survey.

2) Planned safety factor should be PF.s=1.2 because there are many important facilities such as buildings and roads in the lower half and surrounding parts of the landslide and with consideration of the landslide protective technical standards in Japan.

3) The protective measures will be a combination of drainage works and steel piling works because protective measures by natural control approach and artificial control approach are necessary.

4) Groundwater levels in the landslide area is assumed to be drawdown about 2 m by surface drainage works and subsurface drainage works because drawdown for 2 m is the minimum target value for drainage well works in Japan.

5) For preparation of steel piling works, improvement of the safety factor by soil removal works will be considered because soil removal work was already conducted and safety factor is already improved in and surrounding parts of the soil removal area.

6) The designing of the steel piling will be made to secure the planned safety factor of PF.s=1.2 with taking effects of drainage well works into consideration.

4.1.2 Planned safety factor

Necessary safety factor for the stabilization of landslides is generally decided before planning of protective measures. This safety factor is called the planned safety factor.

Planned safety factors for securing stabilization of slopes by providing landslide protective measures are generally required to be F.s=1.1 to 1.2. The planned safety factor of F.s=1.2 was selected for protection from the concerned landslide because there are many lives at risk and important facilities in that area.

4.1.3 Selection of protective measures

For protection from the landslide, measures composed of surface drainage works, subsurface drainage works and steel piling works have been selected as long-term protective measures as shown in Fig.3.4-1.

4.2 Stability Analysis for the Long Term Protective Measures

4.2.1 Safety factor

Stability analysis was made to find the safety factors after the soil removal works and for all observation lines on the assumption that groundwater levels will be depressed 2.0 m below the level observed in June 1989 by drainage well works including the well constructed in the experimental investigation. Strength parameters of the sliding surfaces were those calculated in the planning of the experimental investigation.

Calculated safety factors for all observation lines are summarized below:

OBSERVATION LINE	SAFETY FACTOR (F.s)
V	1.071
W	1.092
X	1.134
Y	1.084
Z	1.094

4.2.2 Required deterrent force

The planned safety factor for long term protective measures is PF.s=1.2 for all the observation lines. Safety factors calculated for all the observation lines are mentioned above. The difference between the planned safety factor (PF.s) and the calculated safety factor (F.s) after the soil removal works is the insufficiency in safety factor which has to be made up by the steel pile works.

Required deterrent force (P) is calculated by multiplying the insufficiency in safety factors by the tangential force (T) of each slice of each observation line. The required deterrent forces for all the observation lines were calculated and are summarized below. The required deterrent force for the secondary small landslide was calculated also with the assumption that the planned safety factor is 1.1 (PF.s=1.1) and the present safety factor is 1.0 (F.s=1.0).

LINE	PF.s	F.s	T(t/m)	P(t/m)	L(m)	Total P(t/m)
V	1.20	1.071	975.12	125.83	180	22649.40
W	1.20	1.092	1508.25	162.82	120	19538.40
X	1.20	1.134	1152.79	75.94	120	9112.80
Y	1.20	1.084	683.20	79.25	120	9510.00
Z	1.20	1.094	494.20	52.39	210	11001.90
				Total	750	71812.50
Small Landslide	1.10	1.000	478.33	47.83	70	3348.31

P = (PF.s - F.s) x T

Line: observation line
PF.s: planned safety factor
F.s: safety factor
T: force along tangential line
P: required deterrent force
L: section length between each observation line

Required total deterrent force is calculated by multiplying the required deterrent force for each observation line by the section length between each observation line.

Steel piling works will be performed in the compressive zone of the landslide, and the actual alignment for the piling route was designed to be about 750 m in total length, following the existing roads in the landslide area. The alignment of the steel piling work route is shown in Fig.4.1-1.

Through these calculation the total required deterrent force for the whole landslide was found to be about 72000 tons. The required deterrent force for the unit section length is calculated to be 95.75 t/m accordingly. On the contrarily, total required deterrent force and the required deterrent force for the unit section length for the small landslide is calculated to be 3348 tons and 47.83 t/m.

It is confirmed that safety factor for the whole landslide after completion of steel piling works is calculated to be F.s=1.217 that is bigger than the planned safety factor (PF.s=1.2) with applying steel piles (SM50 or better quality) which will be used for the long term protective measures. The reason why the calculated safety factor is bigger than the planned safety factor is that the selected steel piles have shear strength of 96.0 t/m which is larger than the required deterrent force of 95.75 t/m.

4.3 Drainage Work

Drainage works planned for landslide protection are classified into surface drainage works and subsurface drainage works.

4.3.1 Horizontal borehole drilling from ground surface

Surface drainage works are for securing slope surface stability by draining shallow groundwater from mountain slopes. For the protection of the excavated slopes in the soil removal area, horizontal boreholes of 1670 m total length are planned. The boreholes will be drilled from berms created during soil removal work. Boreholes are planned to be drilled from 11 sites, and three (3) to six (6) boreholes will be drilled from each site for 30 m to 50 m in individual length.

4.3.2 Drainage well work

Subsurface drainage works will include construction of three drainage wells and one intermediate well and drilling of groundwater collection boreholes and drainage boreholes.

Construction of the drainage wells is for mitigating the landslide movements by reducing groundwater levels surrounding the drainage wells through groundwater collection boreholes. The required quantity of drainage wells is 35 m (3 wells) in total depth. The groundwater collection boreholes are planned to be 1670 m in total length.

One intermediate well of 11 m depth will be constructed for conveyance of drainage water to the ground surface. The drainage boreholes are for draining groundwater from the drainage wells to the ground surface through the intermediate well.

Main function of the drainage wells is for controlling suddenly increasing groundwater levels after heavy rainfalls. This is a peak cut-off function because groundwater levels recover immediately after heavy rainfall especially in the eastern part of the landslide area.

4.4 Steel Piling Work

4.4.1 Steel piles

(1) Main landslide

Required deterrent force for the main landslide was calculated through stability analysis for the whole landslide. Selection of steel piles and examination of the diameter and interval of steel piles were made on the basis of the calculated deterrent force of 95.75 t/m.

Thin-walled steel piles are not sufficient to prevent landslide with the required deterrent force. Thick-walled steel piles, such as welding type centrifugal cast-iron pipes, are required for landslides. Diameter of steel piles was decided as 300 mm from actual experience in Japan. The intervals of the steel piles was decided in relation to the diameter of the steel piles and the depth to the slide surface. The wall thickness of the steel piles was determined as 17 mm to provide the necessary sectional area. The relationship between the sectional area and the diameter of piles is shown in Table 4.4-1.

The length for embedment of steel piles was decided to be more than one third of the pile length from site geological conditions and the length of embedment into bedrocks was decided to be 2 m minimum. The top of the steel piles was designed to be 1.0 m below the ground surface. Interval of steel piles is decided to be 2.0 m from the relation with diameter of steel piles. The alignment of steel piling works for the main landslide is shown in Fig.4.3-1. Length of steel piles range from 12 m to 36 m depending on the location of piling alignment.

(2) Small landslide

According to topographic condition and outcrops position of bedrocks in upper slopes, possible extent of slide surfaces of the small landslide is assumed to be less than 100 m to upper side from the toe portion of the small landslide.

Required deterrent force for the small secondary landslide was calculated to be 47.83 t/m. From this required force, 300 mm diameter steel piles of 9 mm thick are proposed to be provided at 2 m interval. Length of steel piles is decided to be 16 m from assumed depth to slide surfaces. Profile of steel pile arrangement is shown in Fig.4.3-2.

4.4.2 Procedure of piling work

Steel piles will be inserted into drilled boreholes with mortar filling in the steel piles and in space between the piles and boreholes afterwards. The work procedure for the piling works is shown below.

1) Decision on location of piling work
2) Temporary works
3) Drilling works
4) Installation of piles
5) Mortar filling
6) Dismantle temporary work and cleaning up

The quantities of the protective measures is summarized below and given in Table 4.1-1.

PROTECTION MEASURE	QUANTITY	DIMENSION
Drainage Works:		
Horizontal borehole	1670 m (30-50m)	66 mm (dia)
Drainage well	35 m (3wells)	3.5 m (dia)
Groundwater collection borehole	2100 m (50-60m)	66 mm (dia)
Intermediate well	11 m (1well)	3.5 m (dia)
Drainage borehole	200 m (50 m)	116 mm (dia)
Piling Works:		
1) Main landslide		
Vertical boring	8800 m (13-37m,380holes)	350 mm (dia)
Steel pile installation	8420 m (12-36m,380piles)	300 mm (dia)

		17 mm (thick)
2) Small landslide		
Vertical boring	576 m (16m,36holes)	350 mm (dia)
Steel pile installation	576 m (16m,36piles)	300 mm (dia)
		9 mm (thick)

These above quantities cover protective measures for both the main landslide and the small secondary landslide.

4.5 Construction Plan and Schedule

4.5.1 Implementation time schedule

The schedule for construction of the long term protective measures assumes completion within about 22 months from the loan procedure to completion of the main construction works. The implementation time schedule is shown in Fig.4.5-1.

The work program is considered to start with the loan arrangement which will require about 2 months. Selection of a contractor will require international competitive bidding and about 4 months will be required for contract signing. The notice proceed is assumed to be issued within one month after contract signing. The main construction works will take about 15.5 months.

4.5.2 Construction plan

(1) Drainage wells

There will be four wells in total including three drainage wells and one intermediate well. Construction of the wells is scheduled to be carried out by using two sets of equipment, and the work is scheduled to be commenced nine months after the beginning of loan procedure.

From the drainage wells groundwater collection boreholes will be drilled and perforated pvc pipes will be inserted afterwards for protection of the boreholes. Drainage boreholes will be drilled from the drainage wells and the intermediate well. Drilling of the boreholes will be performed by two sets of drilling machine, the drilling is scheduled to be commenced one month after starting of drainage well construction.

(2) Horizontal boring

For protection of excavated slopes in the soil removal area, horizontal boring will be made by using the same drilling equipment used for drilling of groundwater collection boreholes. Perforated pvc pipes will be inserted into boreholes afterwards for protection of the boreholes. The works are scheduled to be commenced at the same time of construction of drainage wells.

(3) Piling work

Piling work will be divided into three stages: the first stage is for drilling of vertical boreholes: the second stage is for installation of steel piles into the drilled boreholes: the third stage is for earth backfill after piling.

The period required to complete the piling works will be about 15.5 months, and the piling works are scheduled to be commenced at the same time of the construction of drainage wells.

4.6 Cost Estimate

4.6.1 Condition for cost estimate

The construction cost for the project was estimated at the price levels of January 1990. The cost estimate was made for local currency and foreign currency components separately. The constitution of the project cost is summarized below.

1) Construction cost of the project facilities
2) Administrative expenses
3) Price escalation
4) Physical contingency
5) Engineering services cost
6) Interest during construction

4.6.2 Unit price

Direct construction cost was estimated on an unit price basis multiplying the unit price of works by the corresponding work quantity. The unit price consists of labor cost, material cost, equipment expense and contractor's indirect cost.

4.6.3 Estimated project cost

The total project cost is estimated to be Rs.272.3 mill. comprising a foreign currency portion of Rs.219.5 mill. and a local currency portion of Rs.52.8 mill. A summary of the estimated total project cost is given below, and detail of the project cost estimated is summarized in Table 4.6-1.

(unit: Rs. mill.)

Item	F/C	L/C	Total
Construction cost	178.3	29.3	207.6
Administration expense	0.0	6.3	6.3
Price escalation	0.0	5.2	5.2
Physical contingency	17.8	4.1	21.9

Engineering service	16.5	2.4	18.9
Interest during construction	6.9	5.5	12.4
Total	219.5	52.8	272.3

4.6.4 Annual disbursement schedule

Disbursement schedule is assumed to start in July and end in June according to the fiscal year in Mauritius. The construction cost is scheduled to be disbursed for two years, the first year and the second year. Since the implementation time schedule of the whole project is estimated to be about 22 months, the disbursement is assumed to be allotted in the same period of 11 months in the first and the second years. The construction costs for the project were assumed to be disbursed as shown in Table 4.6-2 to Table 4.6-3.

The annual disbursement schedule of the total project cost may be summarized as follows:

(unit: Rs. mill.)

Year	F/C	L/C	Total
First year	51.2	13.1	64.3
Second year	158.3	39.7	208.0
Total	219.5	52.8	272.3

4.7 Economic Evaluation

The damages from landslides is two kinds: threats of disaster which will be caused by a landslide; and the substantial damage caused by a landslide. Although the damage by a landslide involves intangible factors such as loss of human lives, the economic evaluation was made to evaluate this in terms of the national and regional economy.

Project evaluation was made from estimation of the potential area of damage in the event of a landslide. The potential area of damage is shown in Fig.4.7-1. This is based on the assumption that landslide disaster will cause damages to the area that has almost the same length of slide surface from which steeply inclined portion is eliminated in the profiles of slide surfaces.

Project evaluation was made on the basis of the with-project and without-project principle: that is, project benefits are potential damage which would be caused by a landslide without the project. On the other hand, project costs are the ones required for the measures to protect the area from landslide disasters.

The economic viability of the project is assessed by applying three discounting techniques; cost-benefit ratio method (CBR), the net present value method (NPV) and the internal-rate-of-return method (IRR).

For assessing the project benefit, damage costs to buildings and properties, traffic services, water supply, and electric supply are calculated. However, land value, loss of human lives, regional economic activities and increase in employment were treated as intangibles. The total damage costs are estimated at Rs.417.11 mill. The assumed damaged costs are summarized in Table 4.7-1. On the other hand, the project cost for the landslide protective measures was estimated at Rs.272.3 mill.

Cash flow of construction cost for the project is calculated with assumption that the landslide will be occurred at the fourth year after the commencement of the project, immediately after completion of the long term protective measures. The cash flow is given in Table 4.7-2. The prime discount rate applied for the first two methods is

10 %.

Economic viability of the project was evaluated by the three methods, and the results obtained were 1.96 for CBR, Rs.214.10 mill. for NPV, and 47.7 % for IRR. The results of the economic evaluation are summarized in Table 4.7-3.

In assessment of the potential landslide damages, number of buildings were totaled to be about 840 with 944 families in the landslide area and the potentially endanger area. Total population is counted to be about 3700 on the basis of data obtained from the government of Mauritius.

In view of the importance of the intangible items, especially the threat to human lives, the project viability is emphasized and therefore early implementation of the project is recommended.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The study on Landslide Protection Project in Port Louis was carried out by collection and analysis of monitoring data, field investigations, and an experimental investigation with provision of urgent protective measures for mitigation of the landslide. Through these studies, the mechanism and movements of the landslide were revealed as follows:

1) The landslide is classified as a colluvial landslide.

2) The landslide movements depend on one united single body of the landslide.

3) The depths to bedrock surfaces and slide surfaces are deepest at the center of the landslide, and decrease gradually to the marginal parts of the landslide.

4) Ground water levels decreased significantly in the central part of the landslide after the rainy season in 1989, but groundwater levels in the boreholes drilled in the eastern part of the landslide have recovered after heavy rainfall.

5) The whole landslide is in a stable condition at present but there is a possibility it would move again after heavy rainfall.

6) Movement of the whole landslide has been mitigated by the soil removal works.

7) Movements of a small secondary landslide ceased after counterweight treatment but movements may occur again after heavy rainfalls in the future.

For protection of the landslide at La Butte, the following protective measures are planned for long term protection.

Item	Quantity
Horizontal borehole drilling from ground surface:	
1) Drilling of horizontal borehole	1670 m
Drainage well works:	
1) Construction of drainage well	35 m (3 wells)
2) Construction of intermediate well	11 m (1 well)
3) Drilling of groundwater collection borehole	2100 m
4) Drilling of drainage borehole	200 m
Piling works:	
1) Vertical drilling (main landslide)	8800 m
2) Pile installation (- do -)	8420 m
3) Vertical drilling (small landslide)	576 m
4) Pile installation (- do -)	576 m

Total construction cost of the long term protective measures was estimated to be Rs.272.3 mill., consisting Rs.219.5 mill. for the foreign portion and Rs.52.8 mill. for the local portion.

The economic validity of the project was evaluated to be 1.96 in CBR, Rs.214.10 mill. in NPV and 47.7 % in IRR. These results confirm that the project is highly viable due to the fact that the project can gain a higher return than that yielded from the discount rate of 10 % representing social opportunity cost.

5.2 Recommendations

It has been revealed that the project will be justifiable technically and economically through the study on " Landslide Protection Project in Port Louis". In consideration of importance of intangible items, especially the threat to human lives, the project is highly viable. Therefore, it is strongly recommended to proceed with the necessary preparation works for executing construction of the long term protective measures (the Project) as soon as possible.

Large scale of construction works accompanied by soil excavation or soil embankment is recommended to be strictly prohibited until completion of the Project. The flat land behind Ecole de la Montagne created after the soil removal works is recommended to be left vacant for construction activities of the Project. After completion of the Project the area behind Ecole de la Montagne is recommended to be prepared as playing fields for public use.

TABLES

TABLE 1.1 ORGANIZATION OF THE STUDY

MAURITIAN TECHNICAL TEAM

Mr.R.Honore (Chairman)
Mr.H.Ganoo(Vice-chairman)
Mr.R.Dozeer
(Principal Engineer,MOW)
Mr.M.F.Boodhoo
(Engineer,MOW)
Mr.R.Vaghjee
(Assistant Director,MS)
Mr.S.Sokappadu
(Hdrometeorologist, MS)
Dr.Chan.C.Yuk
(Lecturer, UOM)
Mr.A.Rajarai
(Executive Engineer, CWA)
Mr.R.Seeras
(Land Surveyor, MOH)
Mr.C.Lefebure
(City Engineer, MOPL)
Mr.Soobratee
(Engineer, MOPL)
Mr.Prithipaul
(Economist, MOLG)
Mr.H.Jeanne
(Administrative Officer, MOLG)

JICA STUDY TEAM

Dr.M.Watari(Team Leader)
Mr.M.Yako(Deputy Team Leader,
Landslide Engineer)
Mr.M.Negishi
(Hydrogeologist)
Mr.M.Arai
(Topographic Surveyor)
Mr.Y.Kouzuki
(Drilling Expert)
Mr.M.Tatebayashi
(Monitoring/Analysis Expert)
Mr.M.Shirakura
(Facility Design Engineer)
Mr.H.Sakurai
(Construction Planner)
Dr.S.Kato
(Economist)

ADVISORY COMMITTEE

Mr.H.Ikeya(Chairman/
Ministry of Construction)
Mr.R.Tsunaki(Member/
Ministry of Construction)
Mr.S.Ohyagi(Member/
Ministry of Construction)

JICA COORDINATOR

Mr.M.Suemori(Social
Development Study Dept.)
Mr.T.Ito(Social
Development Study Dept.)

TABLE 2.3-1(1/2) INVESTIGATION QUANTITY FOR SECOND PHASE-1 IN FIRST STAGE

PHASE	BOREHOLE No.	DEPTH (m)	PERMEABILITY TEST OPEN-END	LUGEON	S.P.T	P.V.C	INCLINOMETER
Phase -1	BV-V1	30.0	4	…	1	30.0	…
	BV-V3	17.0	1	…	…	17.0	…
	BV-W1	30.0	4	…	1	30.0	…
	BV-W2	30.0	4	1	3	24.0	…
	BV-W3	40.0	2	…	…	31.5	…
	BV-X2	25.0	3	…	…	25.0	…
	BV-Y1	18.0	2	…	…	18.0	…
	BV-Y2	22.0	1	…	…	22.0	…
	BV-Y3	10.0	…	…	…	10.0	…
	BV-Z4	13.0	…	…	…	13.0	…
	TOTAL	235.0	21	1	5	220.5	…

TABLE 2.3-1(2/2) INVESTIGATION QUANTITY FOR SECOND PHASE-2 IN FIRST STAGE

PHASE	BOREHOLE No.	DEPTH (m)	PERMEABILITY TEST OPEN-END	LUGEON	S.P.T	P.V.C	INCLINOMETER
Phase -2	BV-V2	30.0	4	…	…	…	30.0
	BV-V4	20.0	1	…	…	…	20.0
	BV-X1	22.0	4	…	…	…	22.0
	BV-X3	35.0	4	…	2	…	35.0
	BV-X4	20.0	1	…	3	…	20.0
	BV-Z1	25.0	2	…	…	…	25.0
	BV-Z2	23.0	2	…	…	…	23.0
	BV-Z3	20.0	…	…	3	…	20.0
	TOTAL	195.0	18	…	8	…	195.0

TABLE 2.3.2(1/2) BOREHOLE DRILLING QUANTITY AND SUMMARY OF PERMEABILITY TEST

BOREHOLE No.	DEPTH (m)	WATER LEVEL (m)	CASING (m)	HEAD H(m)	CASING DIA. R (cm)	QUANTITY Q(l/min)	PERMEABILITY K (cm/sec)
BV-V1	10.0	9.50	0.00	9.50	4.2	4.0	3.04×10^{-3}
	15.0	15.00	0.00	15.00	4.2	72.7	3.50×10^{-2}
	20.0	20.00	0.12	20.12	4.2	>350.0	$>1.26 \times 10^{-1}$
	25.0	25.00	0.35	25.35	4.2	>350.0	$>9.96 \times 10^{-2}$
BV-V2	10.0	1.50	0.20	1.70	4.2	0.8	3.40×10^{-3}
	15.0	2.00	0.10	2.10	4.2	1.2	4.12×10^{-3}
	20.0	16.05	0.20	16.25	4.2	3.1	1.38×10^{-3}
BV-V3	10.0	7.30	0.70	8.00	4.2	5.0	4.51×10^{-3}
BV-V4	10.0	7.80	0.15	7.95	4.2	20.0	1.82×10^{-2}
	15.0	7.80	0.15	7.95	4.2	20.0	1.82×10^{-2}
	20.5	8.95	0.10	9.05	4.2	20.0	1.59×10^{-2}
BV-W1	10.0	8.85	0.10	8.95	4.2	37.4	3.01×10^{-2}
	15.5	15.50	0.25	15.75	4.2	48.0	2.20×10^{-2}
	20.0	20.00	0.10	20.10	4.2	73.0	2.62×10^{-2}
	25.0	25.00	0.21	25.21	4.2	21.1	6.04×10^{-3}
BV-W2	10.0	1.12	0.39	1.51	4.2	0.8	3.82×10^{-3}
	15.0	0.54	0.31	0.85	4.2	1.8	1.53×10^{-2}
	20.0	20.00	0.17	20.17	4.2	0.2	7.15×10^{-5}
	25.0	25.00	0.16	25.16	4.2	16.7	4.79×10^{-3}
BV-W3	10.0	8.19	0.16	8.35	4.2	0.1	8.64×10^{-5}
	20.0	14.17	0.24	14.41	4.2	0.1	5.01×10^{-5}
BV-X1	5.0	5.00	0.60	5.60	4.2	0.7	9.02×10^{-4}
	10.0	6.40	0.60	7.00	4.2	3.9	4.02×10^{-3}
	15.0	12.00	0.60	12.60	4.2	37.8	2.16×10^{-2}
	20.0	13.20	0.20	13.40	4.2	0.4	2.15×10^{-2}
BV-X2	10.0	10.00	0.50	10.50	4.2	1.8	1.24×10^{-3}
	15.0	15.00	0.50	15.50	4.2	2.5	1.16×10^{-3}
	20.0	8.45	0.50	8.95	4.2	0.2	1.61×10^{-4}
BV-X3	10.0	10.00	0.54	10.54	4.2	0.9	6.16×10^{-4}
	15.0	15.00	0.50	15.50	4.2	7.8	3.63×10^{-3}
	20.0	14.43	0.50	14.93	4.2	3.5	1.69×10^{-3}
BV-X4	5.0	1.30	0.15	1.45	4.2	0.4	1.99×10^{-3}
	10.0	0.95	0.15	1.10	4.2	0.4	2.62×10^{-3}
	15.0	7.98	0.15	8.13	4.2	1.0	8.87×10^{-4}

TABLE 2.3.2(2/2) BOREHOLE DRILLING QUANTITY AND SUMMARY OF PERMEABILITY TEST

BOREHOLE No.	DEPTH (m)	WATER LEVEL (m)	CASING (m)	HEAD H(m)	CASING DIA. R (cm)	QUANTITY Q(l/min)	PERMEABILITY K (cm/sec)
BV-Y1	10.0	10.00	0.23	10.23	4.2	0.2	1.41×10^{-4}
	15.0	1.12	0.22	1.34	4.2	0.2	1.08×10^{-3}
BV-Y2	10.0	0.95	0.32	1.27	4.2	2.4	1.36×10^{-2}
BV-Z1	5.0	4.25	0.20	4.45	4.2	1.1	1.78×10^{-3}
	10.0	8.17	0.20	8.37	4.2	1.1	9.48×10^{-4}
	15.0	11.15	0.20	11.35	4.2	1.0	6.36×10^{-4}
	20.0	12.55	0.60	13.15	4.2	1.0	5.49×10^{-4}
BV-Z2	5.0	5.00	0.40	5.40	4.2	1.5	2.00×10^{-3}
	10.0	8.91	0.50	9.41	4.2	2.4	1.84×10^{-3}
	15.0	10.32	0.50	10.82	4.2	4.2	2.80×10^{-3}

TABLE 2.4-1 PHYSICAL PROPERTIES OF CLAYEY MATERIALS

Sample No.	Specific gravity	Liquid limit (%)	Plastic limit (%)	Plastic index (%)	Moisture content (%)	Unit weight (KN/m³)
S/R-1	2.65	122.3	45.7	76.6	35.3	18.3
					37.7	18.3
					41.6	17.8
					35.6	19.1
S/R-2	2.56	88.9	54.8	34.1	39.2	19.4
					37.6	18.6
					41.7	18.2
					36.0	19.2
D/W-1	2.58	110.8	40.5	70.3	36.6	16.8
					42.7	17.9
					36.6	16.4
					37.6	18.6
					33.0	18.3
D/W-2	2.79	67.5	46.7	20.8	24.7	17.2
					23.6	18.2
					26.2	17.4

* S/R : sample from the soil removal area
  D/W : sample from the drainage well

TABLE 2.4-2 MECHANICAL PROPERTIES OF CLAYEY MATERIALS

Sample No.	UNIAXIAL COMPRESSIVE TEST Strength (kg/cm²)	TRIAXIAL COMPRESSIVE TEST Cohesion (t/m²)	Friction angle (deg.)
S/R-1	2.08	1.8	9.2
	2.35		
	2.50		
	1.02		
S/R-2	2.35	2.1	5.5
	1.85		
	2.50		
	3.12		
D/W-1	0.15	3.0	10.0
	0.24		
	0.65		
	0.90		
	0.95		
D/W-2	0.95	1.0	28.0
	1.15		
	0.90		

* S/R : samples from the soil removal area
D/W : samples from the drainage well

TABLE 3.2-1 DESIGN EARTH PRESSURE

DEPTH (m)	DESIGN EARTH PRESSURE q : (tf/m²)	DEPTH (m)	DESIGN EARTH PRESSURE q : (tf/m²)
0.00	0.00	10.00	9.00
0.50	0.45	10.50	9.45
1.00	0.90	11.00	9.90
1.50	1.35	11.50	10.35
2.00	1.80	12.00	10.80
2.50	2.25	12.50	11.25
3.00	2.70	13.00	11.70
3.50	3.15	13.50	12.15
4.00	3.60	14.00	12.60
4.50	4.05	14.50	13.05
5.00	4.50	15.00	13.50
5.50	4.95	15.50	13.95
6.00	5.40	16.00	14.40
6.50	5.85	16.50	14.85
7.00	6.30	17.00	15.30
7.50	6.75	17.50	15.75
8.00	7.20	18.00	16.20
8.50	7.65	18.50	16.65
9.00	8.10	19.00	17.10
9.50	8.55	19.50	17.55
10.00	9.00	20.00	18.00

TABLE 3.2-2 RQUIRED QUANTITY OF MATERIALS FOR THE WELLS

MATERIAL	SPECIFICATION	UNIT WEIGHT (kg)	DRAINAGE WELL		INTERMEDIATE WELL	
			QUANTITY	WEIGHT (kg)	QUANTITY	WEIGHT (kg)
Steel plate	P10, t2.7mm	27.43	77plates	2112.1	217plates	5952.3
	Perforated	27.43	210plates	5760.3	…	…
Stiffener ring	H-125, l=2747.5mm	65.4	24pcs	1569.6	12pcs	784.8
Stiffener	H-175, l=6000mm	241.2	4pcs	964.8	…	…
	l=4500mm	180.9	4pcs	723.6	…	…
Connecting plate						
for ring	330×125×12mm	3.89	48plates	186.7	24plates	93.4
for stiffener	340×175×12mm	5.6	8plates	44.8	…	…
- do -	340×140× 6mm	2.24	8plates	17.9	…	…
Bolt	M16×35mm	0.146	3528pcs	515.1	2758pcs	402.7
	M16×45mm	0.160	420pcs	67.2	210pcs	33.6
	M20×50mm	0.283	528pcs	149.5	192pcs	54.3
	Nut for M20	…	192pcs	…	96pcs	…
	U-bolt M16	1.07	48pcs	51.4	…	…
Ladder	A-type H=1500mm	37.8	2pcs	75.6	…	…
	B-type H=2000mm	48.5	6pcs	291.0	…	…
	C-type H=1000mm	27.5	3pcs	82.5	…	…
	Step	41.4	3pcs	124.2	…	…
	Vertical H=4000mm	61.0	…	…	3pcs	183.0
	Vertical H=2000mm	32.0	…	…	1pc	32.0
	Handrail H=1000mm	17.6	1pc	17.6	…	…
	Metal fixtures	2.5	39pcs	97.5	…	…
Cover	Dia.=3600mm	610.0	1pc	610.0	1pc	610.0
		TOTAL WEIGHT (kg) =		13461.4		8146.1

TABLE 3.4-1 QUANTITY AND SIZE OF EMERGENCY COUNTERWEIGHT EMBANKMENT

Counterweight	Length (m)	Width (m)	Height (m)	Volume (m^3)	Elevation (m)	Time
First	40	5-6	3	655	46-49	Jan. 1990
Second	34	6-4	3	485	49-52	May. 1990
Third	21	2-3	8	154	52-55	Jun. 1990

* The first counterweight was for the landslide movements since Oct. 1989.

* The Second and third counterweights were for the landslide movements since Feb. 1990.

TABLE 3.4-2 SUMMARY OF STABILITY ANALYSIS RESULTS FOR THE SMALL LANDSLIDE BELOW EXTENSOMETER, E-3

STAGE	c (kg/sq. cm)	phi (deg.)	A	B	C
0	1.00	14.89	1.00	1.103	1.224
I	1.00	14.89	1.094	1.103	1.224
I	1.00	13.05	1.00	1.016	1.134
I + II + III	1.00	13.05	1.046	1.119	1.134

c : cohesion of soil
phi : internal friction angle of soil
A : safety factor along the firstly assumed sliding surface
B : safety factor along the secondary assumed sliding surface
C : safety factor along the thirdly assumed sliding surface
I : at the completion of the first counterweight
I + II + III : at the completion of the first to the third counterweights

TABLE 4.1-1 PRINCIPAL FEATURES AND MAJOR WORK QUANTITIES

Item	Dimension of Structures	Quantities
I. Drainage Well	(total of No.1 to 6)	422 cu.m
1. Drainage Well	(sub-total of No.1 to 5)	322 cu.m
(1) DW-2(No.1)		0 cu.m
(2) DW-3(No.2)		0 cu.m
(3) DW-4(No.3)	Diam.=3.5m L=15m	142 cu.m
(4) DW-5(No.4)	Diam.=3.5m L=10m	90 cu.m
(5) DW-6(No.5)	Diam.=3.5m L=10m	90 cu.m
2. Intermediate Well		
(1) DW-2(No.6)	Diam.=3.5m L=11m (sub-total of No.6)	100 cu.m
3. Metal works		
(1) Liner plate	0.5m×1.57m×7pcs./ring	48 lin.m
(2) Ring stiffener	H-125×125	26 sets
(3) Vertical stiffener	H-175×175	4 sets
4. Horizontal boring		
(1) Water collection	Diam.=66mm L=50-60m	2,100 lin.m
(2) Water drainage	Diam.=116mm L=50m	200 lin.m
II. Horizontal Boring		
1. Horizontal Boring		
(1) Water collection	Diam.=66mm L=30-50m	1,670 lin.m
III. Piling		
1. Vertical boring (416 nos.)	(total of section 1 to 13)	9,376 lin.m
(1) Section 1	Diam.=350mm, L=13m×49 nos.	637 lin.m
(2) Section 2	Diam.=350mm, L=17m×18 nos.	306 lin.m
(3) Section 3	Diam.=350mm, L=21m×5 nos.	105 lin.m
(4) Section 4	Diam.=350mm, L=25m×5 nos.	125 lin.m
(5) Section 5	Diam.=350mm, L=29m×5 nos.	145 lin.m
(6) Section 6	Diam.=350mm, L=33m×9 nos.	297 lin.m
(7) Section 7	Diam.=350mm, L=37m×41 nos.	1,517 lin.m
(8) Section 8	Diam.=350mm, L=33m×20 nos.	660 lin.m
(9) Section 9	Diam.=350mm, L=25m×65 nos.	1,625 lin.m
(10) Section 10	Diam.=350mm, L=25m×56 nos.	1,400 lin.m
(11) Section 11	Diam.=350mm, L=21m×41 nos.	861 lin.m
(12) Section 12	Diam.=350mm, L=17m×66 nos.	1,122 lin.m
(13) Section 13	Diam.=350mm, L=16m×36 nos.	576 lin.m
2. Pile installation (416 nos.)	(total pile length)	8,996 lin.m
(1) Steel pile	Outer diam.=300mm, t=17mm×380 nos.	8,420 lin.m
(2) Steel pile	Outer diam.=300mm, t= 9mm× 36 nos.	576 lin.m
3. Plug works		
(1) Concrete filling	inside of piles for 416 nos.	661 cu.m
(2) Mortar filling	outside of piles for 416 nos.	254 cu.m
(3) Pile head plug	earth materials for 116 nos.	11 cu.m
(4) Pile head plug	crushed stone for 264 nos.	24 cu.m
(5) Pile head plug	asphalt for 264 nos.	3 ton

TABLE 4.4-1 STANDARD OF STEEL PILE

Outer × Thickness dia.	Sectional area	Unit weight
D × t (mm) (mm)	A (cm²)	W (kg/m)
300 × 8	73.4	57.6
300 × 9	82.3	64.6
300 × 10	91.1	71.5
300 × 11	99.9	78.4
300 × 12	108.6	85.2
300 × 13	117.2	92.0
300 × 14	125.8	98.7
300 × 15	134.3	105.4
300 × 16	142.8	112.1
300 × 17	151.1	118.6
300 × 18	159.5	125.2
300 × 19	167.7	131.7
300 × 20	175.9	138.1
300 × 21	184.1	144.5
300 × 22	192.1	150.8
300 × 23	200.2	157.1
300 × 24	208.1	163.4
300 × 25	216.0	169.5
300 × 26	223.8	175.7
300 × 27	231.6	181.8
300 × 28	239.3	187.8
300 × 29	246.9	193.8
300 × 30	254.5	199.8
300 × 31	262.0	205.7
300 × 32	269.4	211.5
300 × 33	276.8	217.3
300 × 34	284.1	223.0
300 × 35	291.4	228.7
300 × 36	298.6	234.4
300 × 37	305.7	240.0
300 × 38	312.8	245.5
300 × 39	319.8	251.0
300 × 40	326.7	256.5
300 × 42	340.4	267.2
300 × 44	353.9	277.8
300 × 46	367.1	288.1
300 × 48	380.0	298.3
300 × 50	392.7	308.3

TABLE 4.6-1 TOTAL PROJECT COST

Exchange Rate : 1.0US$=Rs.15.3=JYE146.0

Item	Foreign Portion (Rs.1,000)	Local Portion (Rs.1,000)	Total Amount (Rs.1,000)
A. Construction Cost			
A-1 General Item	33,218	6,048	39,266
A-2 Drainage Well	6,549	5,354	11,903
(1) Earth works	792	1,405	2,197
(2) Concrete works	17	116	133
(3) Metal works	1,586	352	1,938
(4) Safety facilities	186	76	262
(5) Water collection boring	3,423	2,412	5,835
(6) Water drainage boring	393	266	659
(7) Borehole protection	152	713	865
(8) Drainage channel	0	14	14
A-3 Horizontal Boring	1,978	1,995	3,973
(1) Water collection boring	1,897	1,450	3,347
(2) Borehole protection	64	518	582
(3) Drainage channel	17	27	44
A-4 Piling	136,566	15,930	152,496
(1) Earth works	54	78	132
(2) Vertical boring	101,929	11,769	113,698
(3) Pile installation	32,133	967	33,100
(4) Reset machinery	50	175	225
(5) Disposal works	609	180	789
(6) Plug works	1,791	2,761	4,552
Total of A	178,311	29,327	207,638
B. Administration Expense	0	6,275	6,275
C. Price Escalation	0	5,200	5,200
D. Physical Contingency	17,789	4,098	21,887
E. Engineering Service	16,500	2,400	18,900
F. Interest during Construction	6,900	5,500	12,400
Total	219,500	52,800	272,300

TABLE 4.6-2 ANNUAL DISBURSEMENT SCHEDULE (Financial Cost)

(Unit : Mil.Rs.)   Exchange Rate : 1.0US$＝Rs.15.3＝JY146.0

	Total			First year			second year		
	F/C	L/C	total	F/C	L/C	total	F/C	L/C	total
A. Construction Works									
A-1 General Item	33.2	6.0	39.2	20.3	3.6	23.9	12.9	2.4	15.3
A-2 Drainage Well	6.5	5.4	11.9	1.0	0.8	1.8	5.5	4.6	10.1
A-3 Horizontal Boring	2.0	2.0	4.0	0.7	0.7	1.4	1.3	1.3	2.6
A-4 Piling	136.6	15.9	152.5	17.1	1.9	19.0	119.5	14.0	133.5
Total of A	178.3	29.3	207.6	39.1	7.0	46.1	139.2	22.3	161.5
B. Administration Expense	0.0	6.3	6.3	0.0	2.3	2.3	0.0	4.0	4.0
Total of A to B	178.3	35.6	213.9	39.1	9.3	48.4	139.2	26.3	165.5
C. Price Escalation	0.0	5.2	5.2	0.0	0.7	0.7	0.0	4.5	4.5
Total of A to C	178.3	40.8	219.1	39.1	10.0	49.1	139.2	30.8	170.0
D. Physical Contingency	17.8	4.1	21.9	3.9	1.0	4.9	13.9	3.1	17.0
E. Engineering Service	16.5	2.4	18.9	6.9	1.0	7.9	9.6	1.4	11.0
Total of A to E	212.6	47.3	259.9	49.9	12.0	61.9	162.7	35.3	198.0
F. Interest during Construction	6.9	5.5	12.4	1.3	1.1	2.4	5.6	4.4	10.0
Grand Total	219.5	52.8	272.3	51.2	13.1	64.3	168.3	39.7	208.0

TABLE 4.6-3 ANNUAL DISBURSEMENT SCHEDULE (Economic Cost)

(Unit : Mil. Rs.) Exchange Rate : 1.00US$ = Rs. 15.3 = JY¥146.0

	Total			First year			second year		
	F/C	L/C	total	F/C	L/C	total	F/C	L/C	total
A. Construction Works									
A-1 General Item	33.2	6.0	39.2	20.3	3.6	23.9	12.9	2.4	15.3
A-2 Drainage Well	6.5	5.4	11.9	1.0	0.8	1.8	5.5	4.6	10.1
A-3 Horizontal Boring	2.0	2.0	4.0	0.7	0.7	1.4	1.3	1.3	2.6
A-4 Piling	136.6	15.8	152.4	17.1	1.9	19.0	119.5	13.9	133.4
Total of A	178.3	29.2	207.5	39.1	7.0	46.1	139.2	22.2	161.4
B. Administration Expense	0.0	6.3	6.3	0.0	2.3	2.3	0.0	4.0	4.0
Total of A to B	178.3	35.5	213.8	39.1	9.3	48.4	139.2	26.2	165.4
C. Price Escalation	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total of A to C	178.3	35.5	213.8	39.1	9.3	48.4	139.2	26.2	165.4
D. Physical Contingency	17.8	3.5	21.3	3.9	0.9	4.8	13.9	2.6	16.5
E. Engineering Service	16.5	2.4	18.9	6.9	1.0	7.9	9.6	1.4	11.0
Total of A to E	212.6	41.4	254.0	49.9	11.2	61.1	162.7	30.2	192.9
F. Interest during Construction	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Grand Total	212.6	41.4	254.0	49.9	11.2	61.1	162.7	30.2	192.9

TABLE 4.7-1 ANTICIPATED DAMAGE BY THE LANDSLIDE OCCURRENCE AT LA BUTTE

(Mill. Ra.)

ANTICIPATED DAMAGE ITEM	VALUE
1. Building and Properties	339.9
2. Traffic service	44.37
a. disruption of traffic service	(31.76)
b. treatment of debris deposits	(8.08)
c. road pavement	(4.53)
3. Water supply	17.64
4. Electric supply	1.12
5. Land value	-
6. Loss of human life	-
7. Regional economic activities	-
8. Increase in employment	-
9. Social psychological damage (each year excluding 1990)	14.08
TOTAL	417.11

TABLE 4.7-2 CASH FLOW OF THE PROJECT FOR THE LONG TERM PROTECTIVE MEASURES

	Social Discount Rate (r) r=0 %		r=10 %	
Year	Benefit	Cost	Benefit	Cost
1	0.00	3.60	0.00	3.60
2	14.08	165.70	12.80	150.64
3	14.08	84.40	11.64	69.75
4	417.11	0.00	313.38	0.00
5	14.08	0.00	9.62	0.00
6	14.08	0.00	8.74	0.00
7	14.08	0.00	7.95	0.00
8	14.08	0.00	7.23	0.00
9	14.08	0.00	6.57	0.00
10	14.08	0.00	5.97	0.00
11	14.08	0.00	5.43	0.00
12	14.08	0.00	4.93	0.00
13	14.08	0.00	4.49	0.00
14	14.08	0.00	4.08	0.00
15	14.08	0.00	3.71	0.00
16	14.08	0.00	3.37	0.00
17	14.08	0.00	3.06	0.00
18	14.08	0.00	2.79	0.00
19	14.08	0.00	2.53	0.00
20	14.08	0.00	2.30	0.00
21	14.08	0.00	2.09	0.00
22	14.08	0.00	1.90	0.00
23	14.08	0.00	1.73	0.00
24	14.08	0.00	1.57	0.00
25	14.08	0.00	1.43	0.00
26	14.08	0.00	1.30	0.00
27	14.08	0.00	1.18	0.00
28	14.08	0.00	1.07	0.00
29	14.08	0.00	0.98	0.00
30	14.08	0.00	0.89	0.00
31	14.08	0.00	0.81	0.00
32	14.08	0.00	0.73	0.00
33	14.08	0.00	0.67	0.00
34	14.08	0.00	0.61	0.00
35	14.08	0.00	0.55	0.00
	881.75	253.70	438.09	223.99

r=0 %	r=10 %
B= 881.75	B= 438.09
C= 253.70	C= 223.99
B−C= 628.05	B−C= 214.10

Net benfit : B−C =214.10

Cost benefit ratio : B/C =1.96

TABLE 4.7-3 ECONOMIC EVALUATION BY THREE METHODS

DISCOUNTING TECHNIQUE	VALUE
1. CBR	1.96
2. NPV	Rs. 214.10
3. IRR	47.7 %
Benefit = Mill. Rs. 438.09	
Cost = Mill. Rs. 223.99	

CBR : cost-benefit ratio method

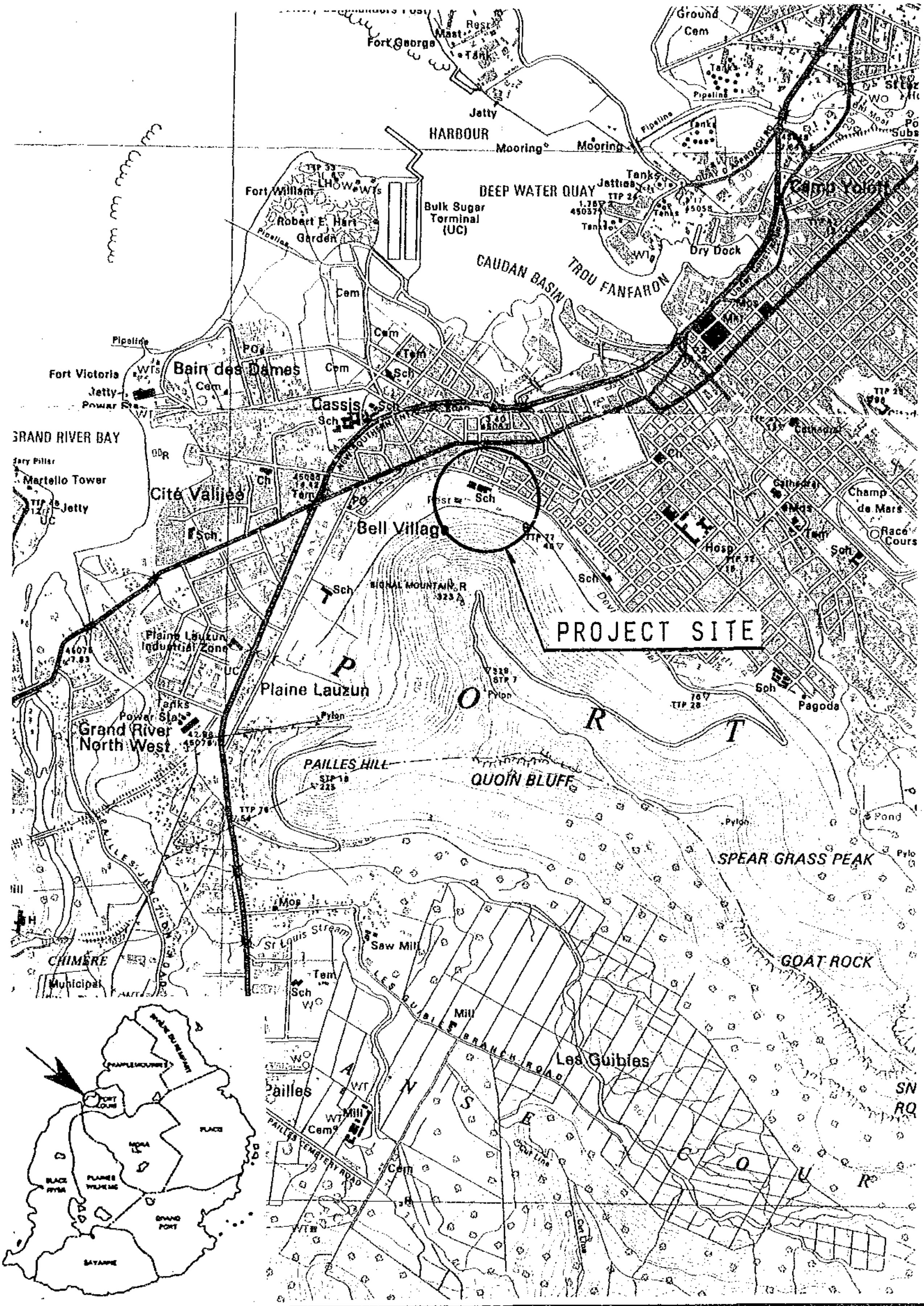
NPV : net present value method

IRR : internal-rate-of-return method

Note : CBR and NPV are measured by principal interest rate of r = 10 % per annum.

FIGURES

Fig. 1.1-1



LOCATION MAP OF THE PROJECT SITE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	1.1-2

Photo 1.
Slip scarps and open cracks at crown part of the landslide

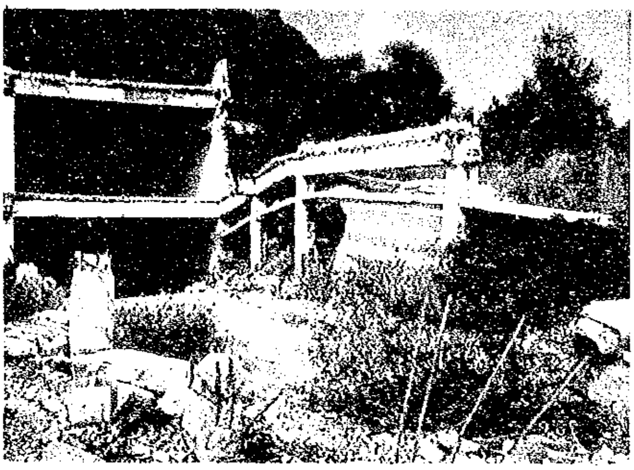


Photo 2.
Demolished building in the landslide

Photo 3.
Demolished mosque in the landslide

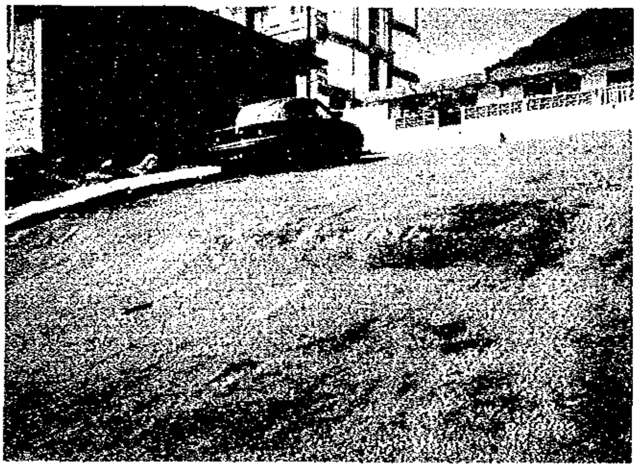


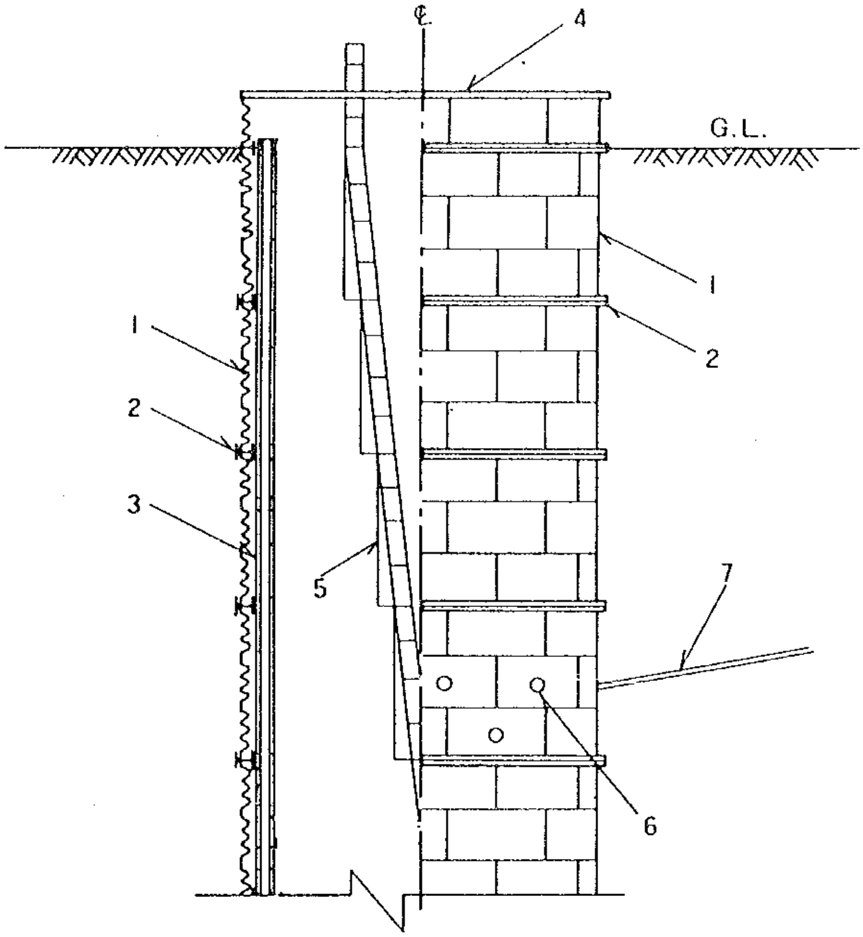
Photo 4.
Upheavals crossing a road at northern toe portion of the landslide

Photo 5.
Damaged structures in the landslide area

Photo 6.
Damaged house in the landslide area

PHOTOGRAPHS OF DAMAGES BY THE LANDSLIDE	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 1.4-1



1. AN-TYPE LINER PLATE
2. STIFFENER RING
3. VERTICAL STIFFENER
4. COVER(EXPANDED METAL)
5. LADDER
6. WATER COLLECTION HOLES
7. GROUNDWATER COLLECTION BOREHOLE

STRUCTURE OF DRAINAGE WELL	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
	JAPAN INTERNATIONAL COOPERATION AGENCY

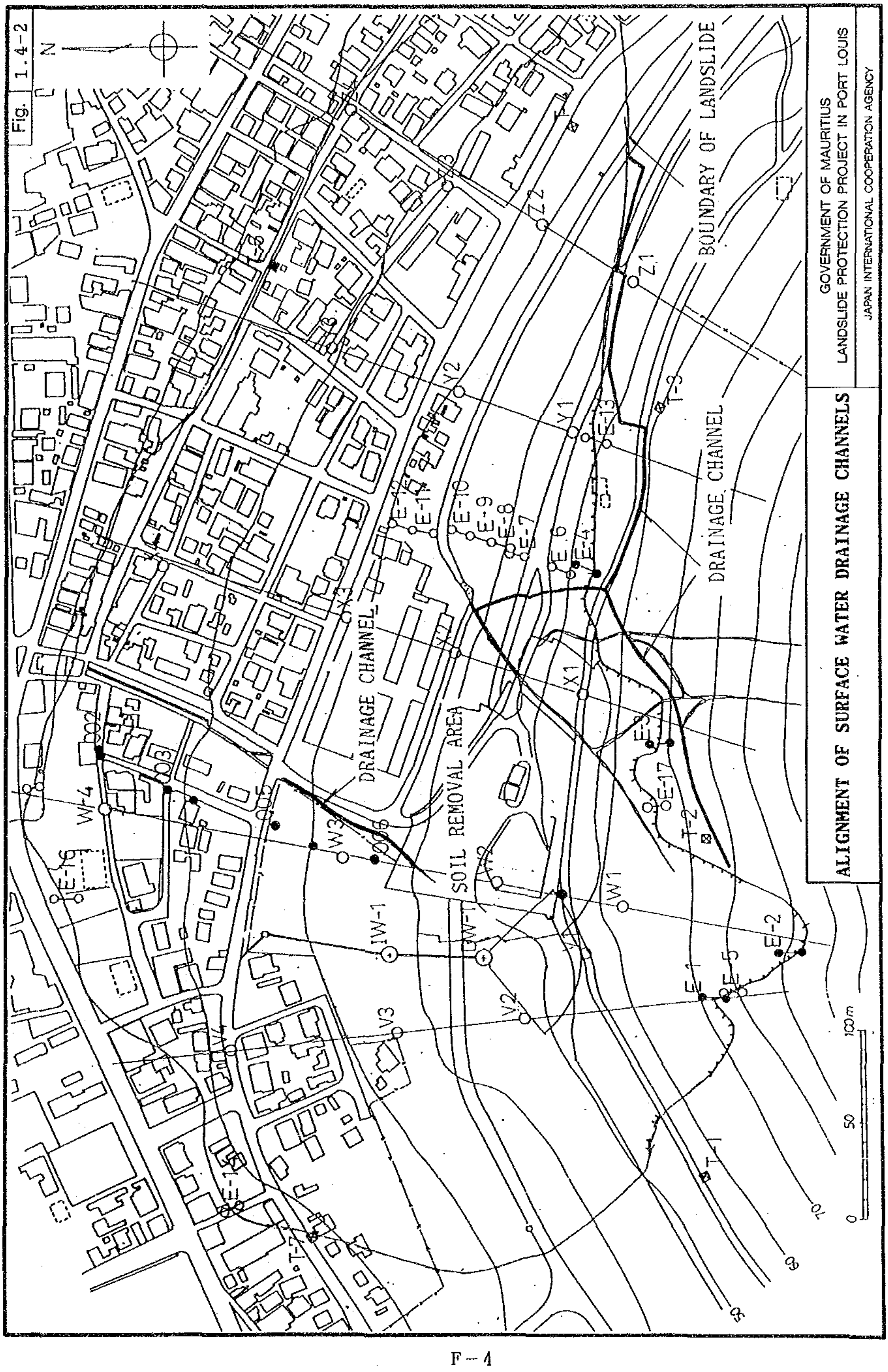
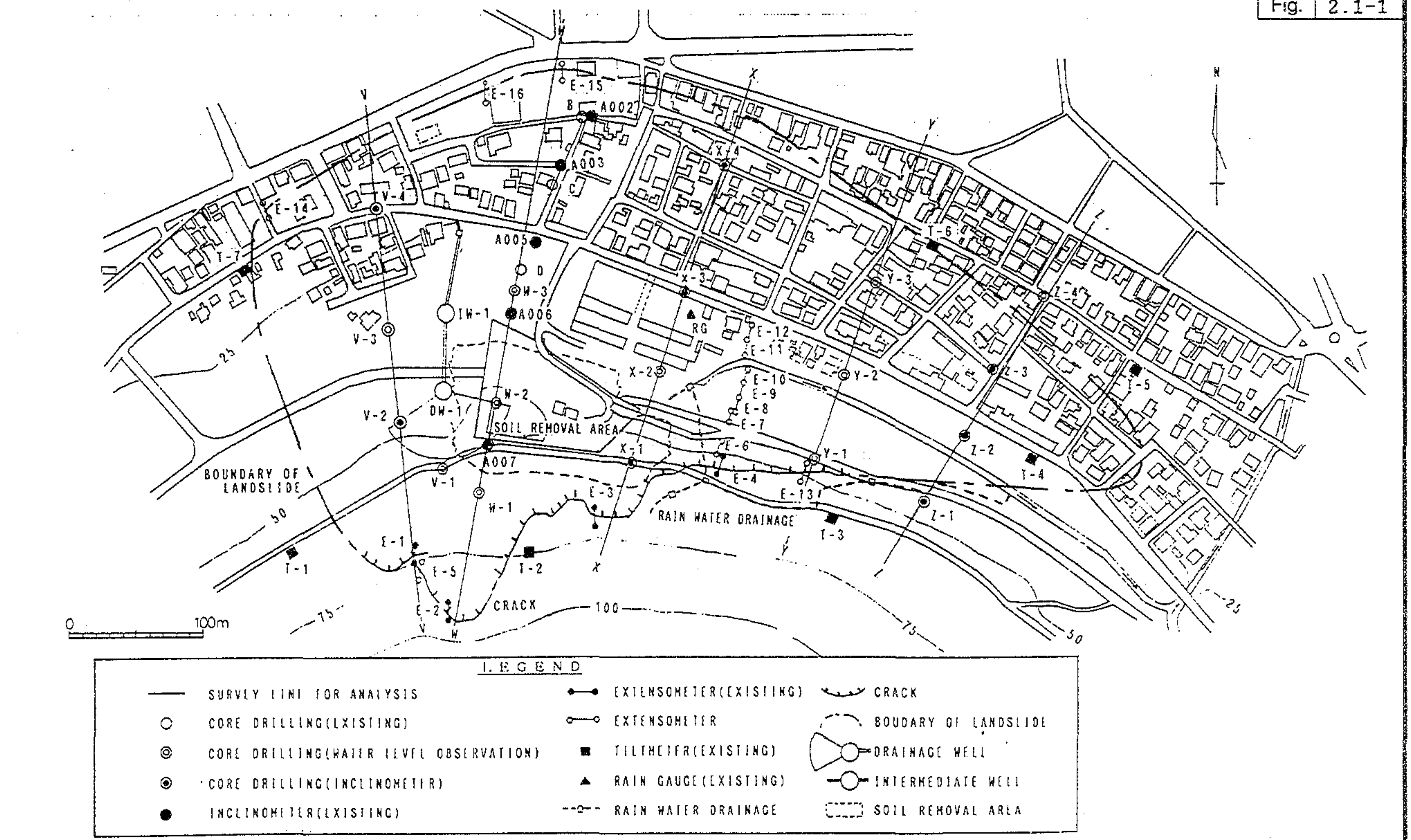


Fig. 1.4-2
N
BOUNDARY OF LANDSLIDE
DRAINAGE CHANNEL
DRAINAGE CHANNEL
SOIL REMOVAL AREA
0
50
100m
ALIGNMENT OF SURFACE WATER DRAINAGE CHANNELS
GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.1-1

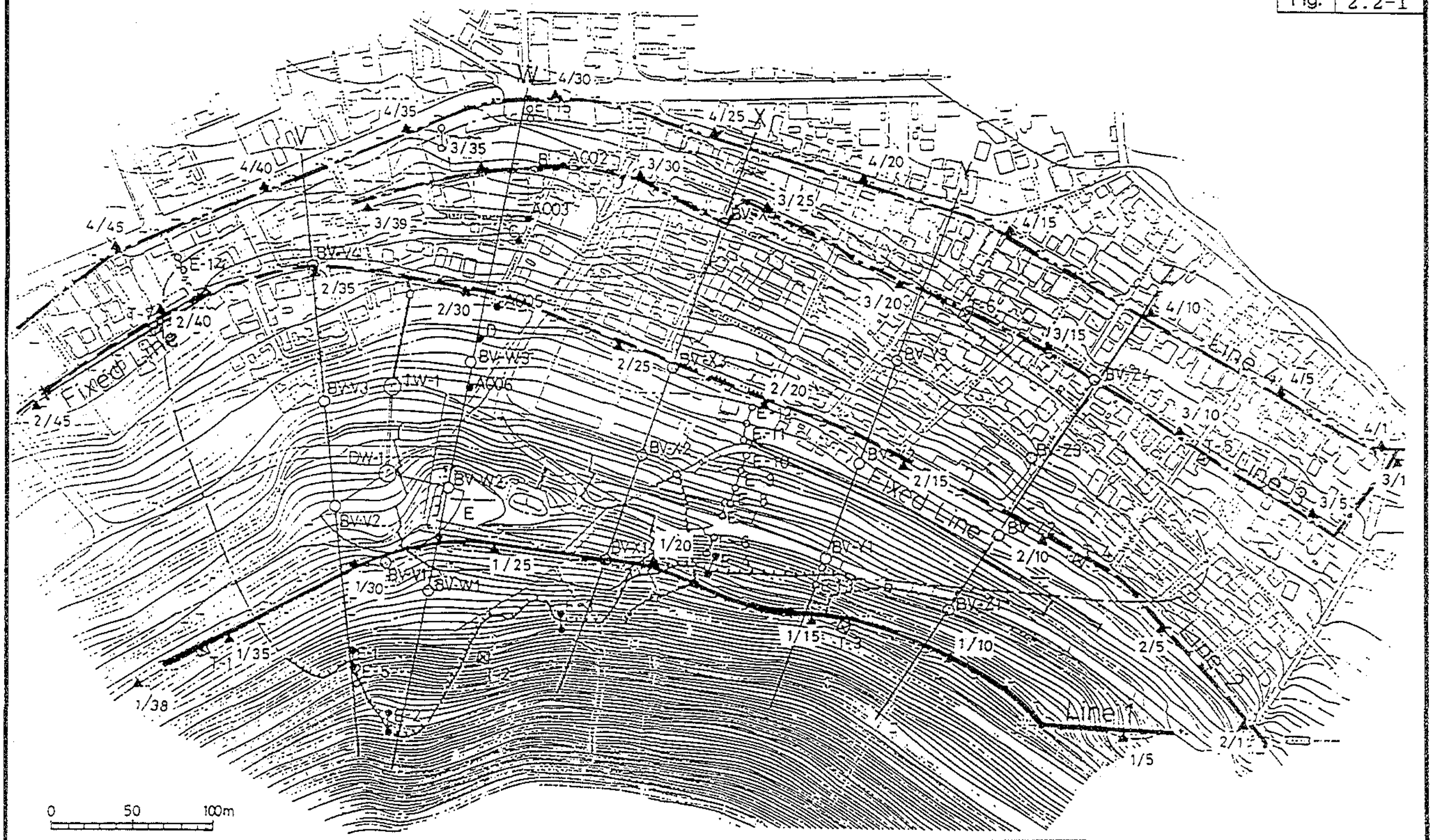


LOCATION MAP OF THE INVESTIGATION SITE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.2-1

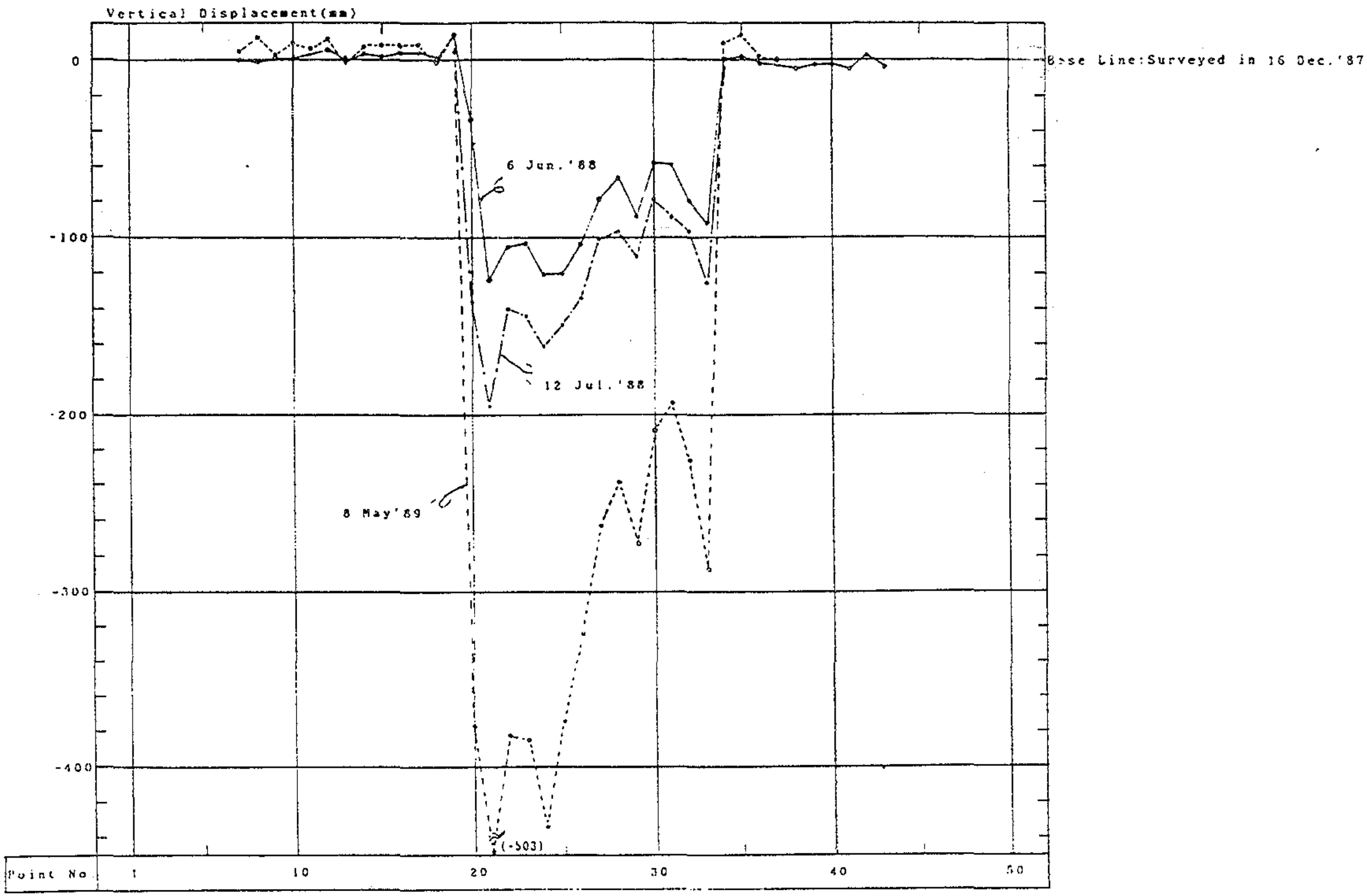


LOCATION MAP FOR THE GROUND SURVEY

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.2-2

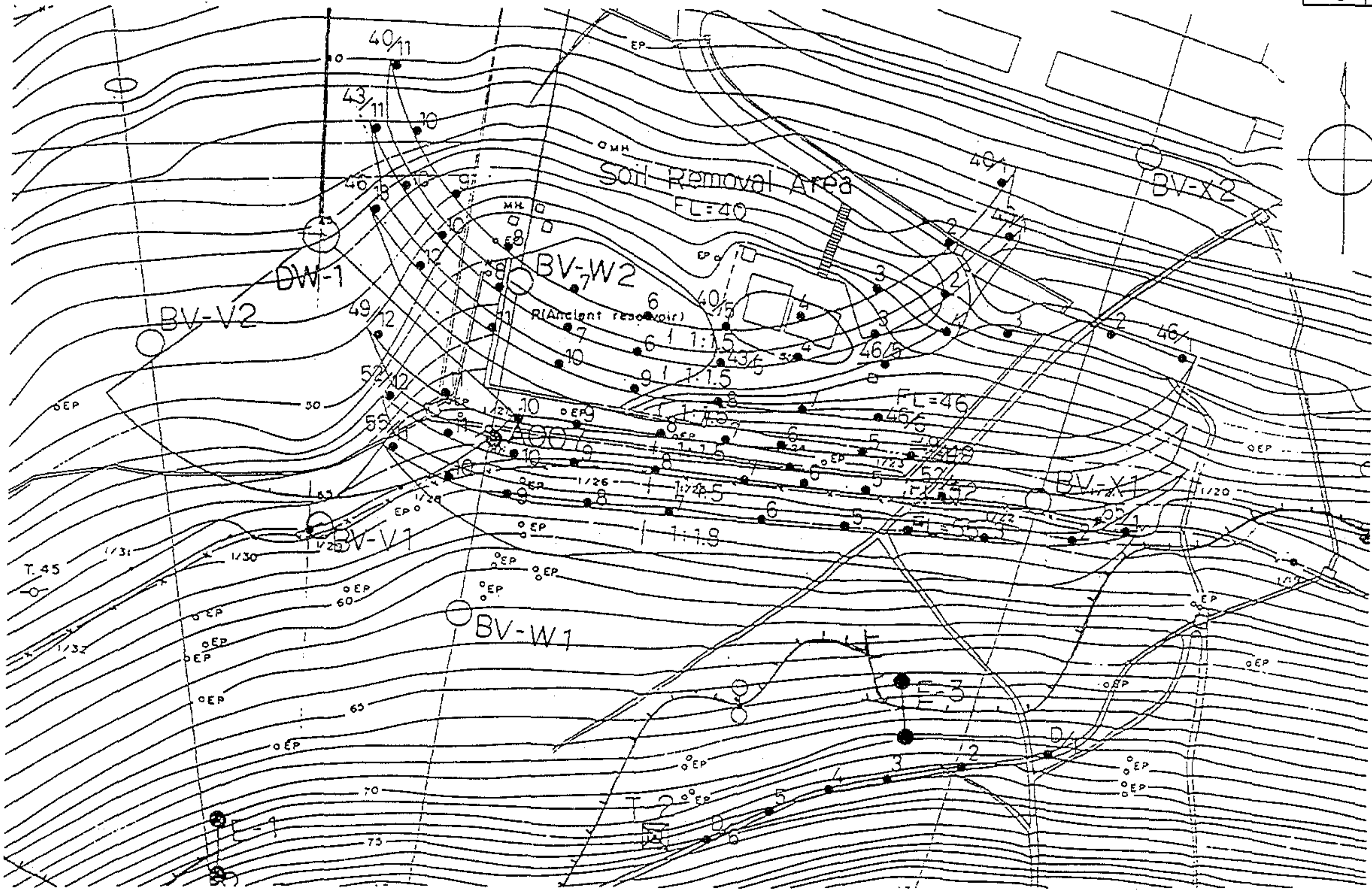


LEVELING SURVEY RESULT ALONG LINE-1

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

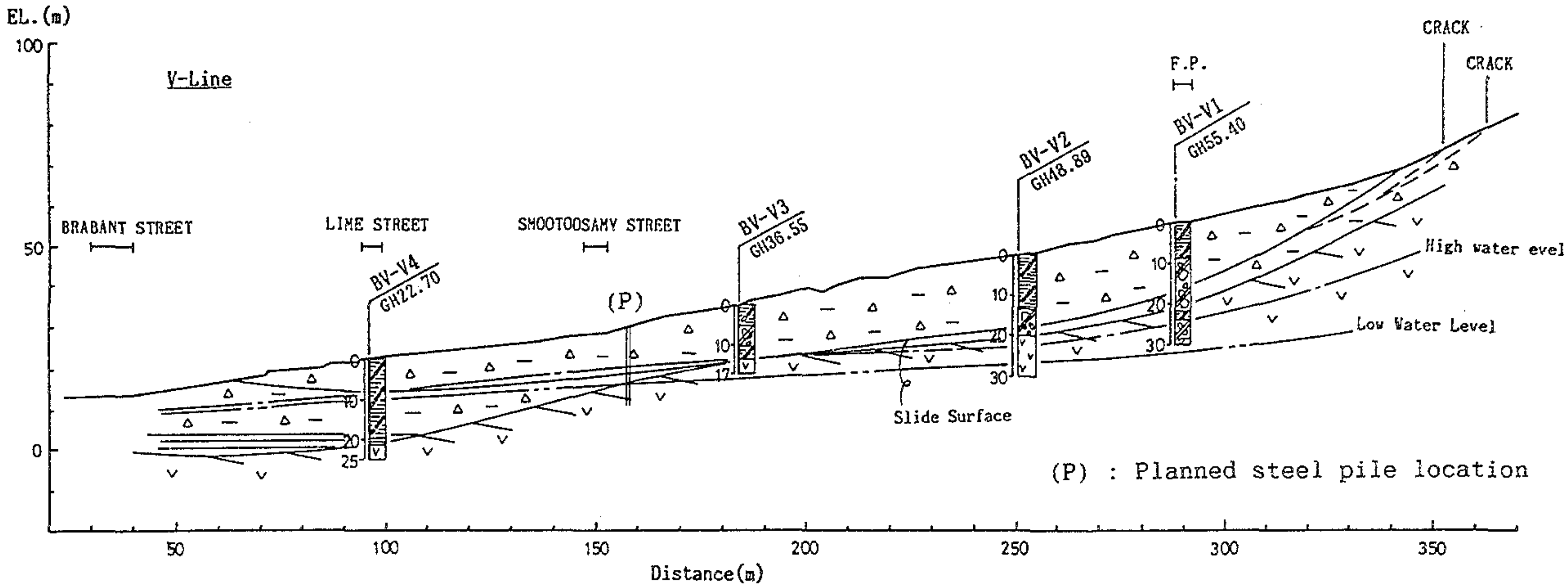
Fig. 2.2-3



LAYOUT OF LEVELING SURVEY POINTS
AT THE SOIL REMOVAL AREA

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
JAPAN INTERNATIONAL COOPERATION AGENCY

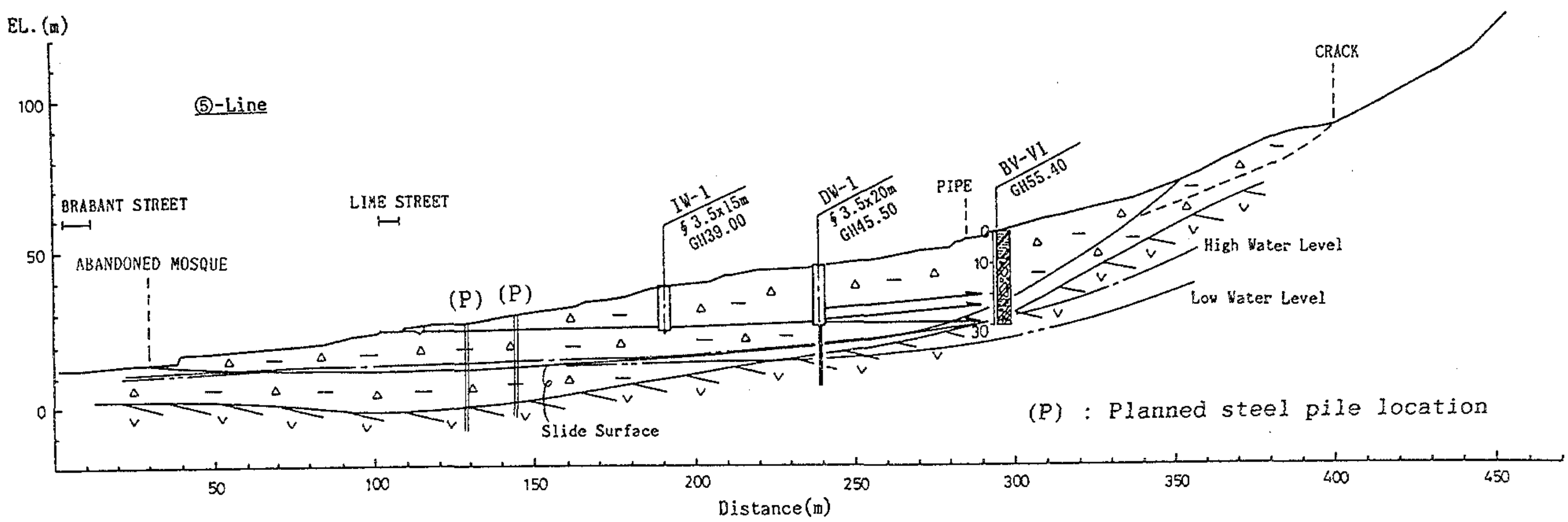
Fig. 2.3-1



GEOLOGICAL PROFILE ALONG V-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.3-2

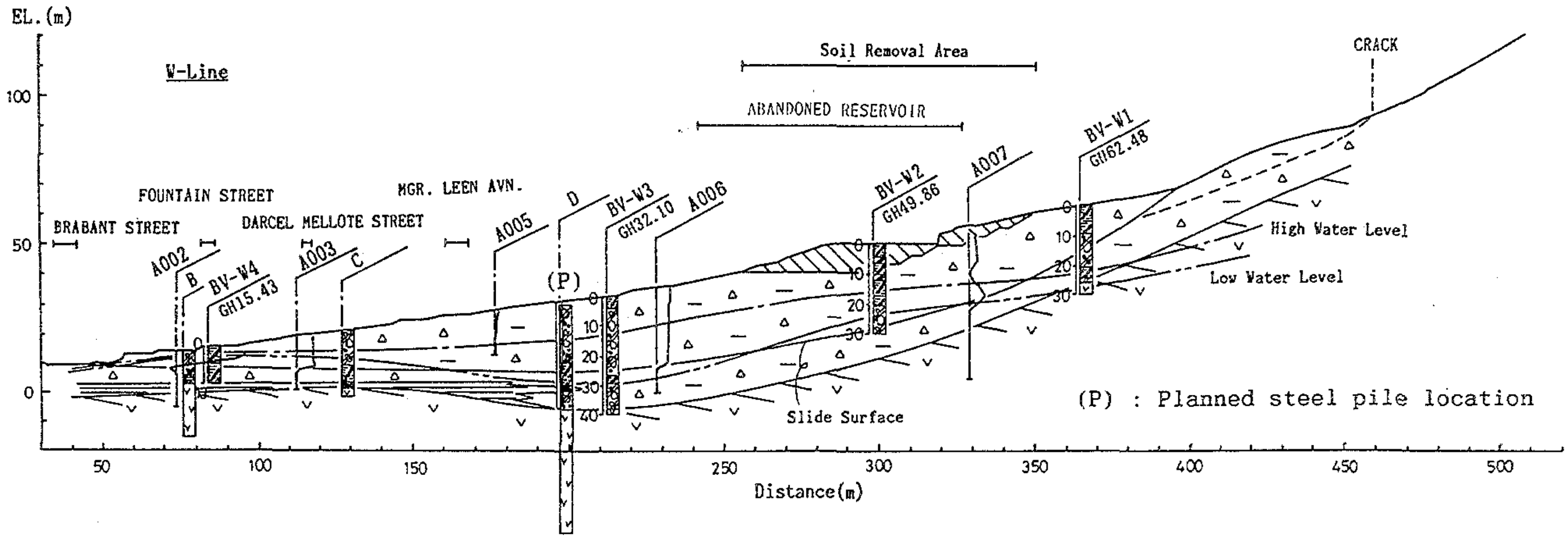


GEOLOGICAL PROFILE ALONG THE LINE THROUGH THE DRAINAGE WELL AND THE INTERMEDIATE WELL

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

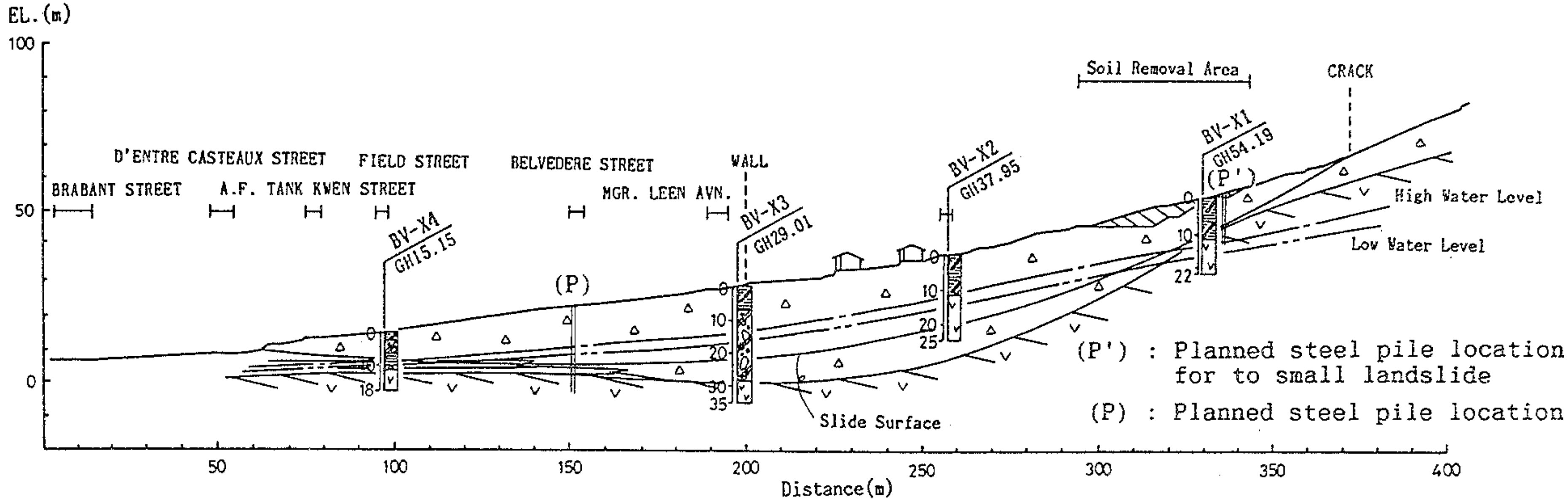
Fig. 2.3-3



GEOLOGICAL PROFILE ALONG W-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.3-4

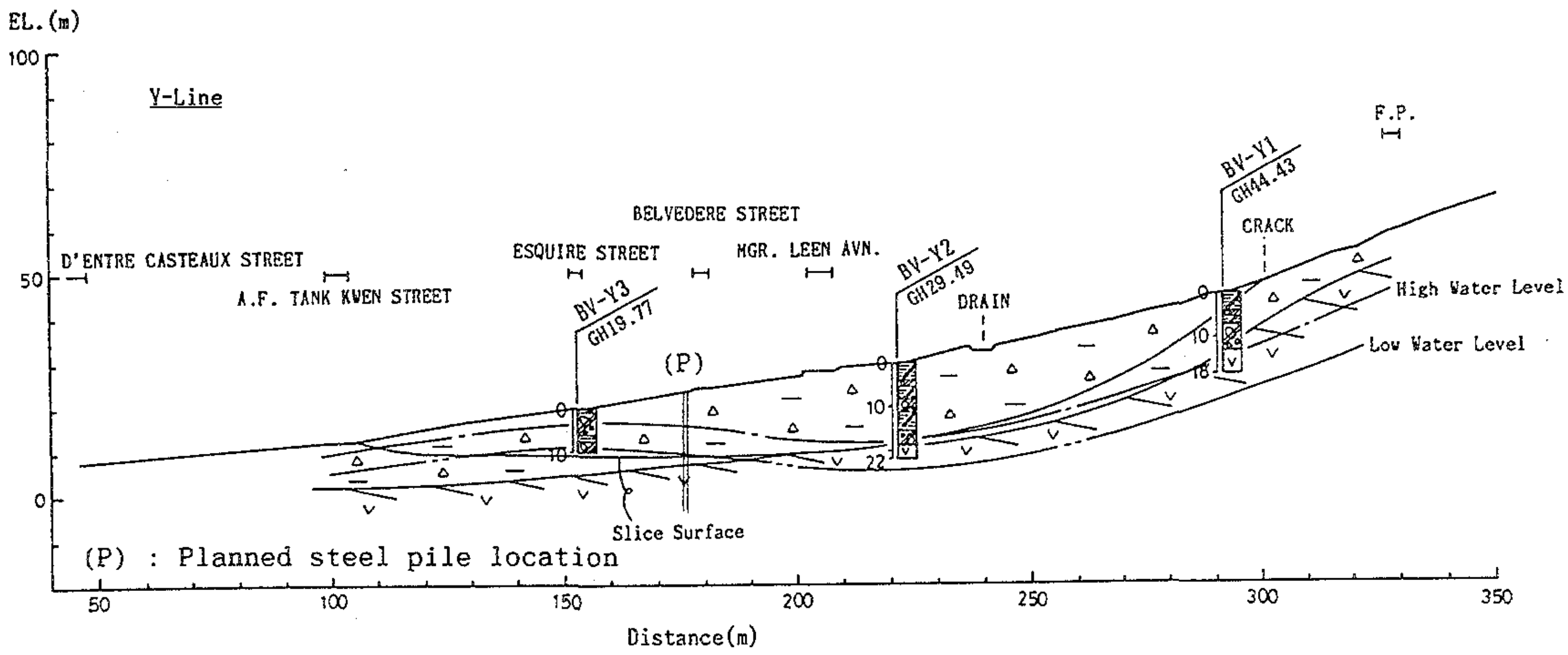


GEOLOGICAL PROFILE ALONG X-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

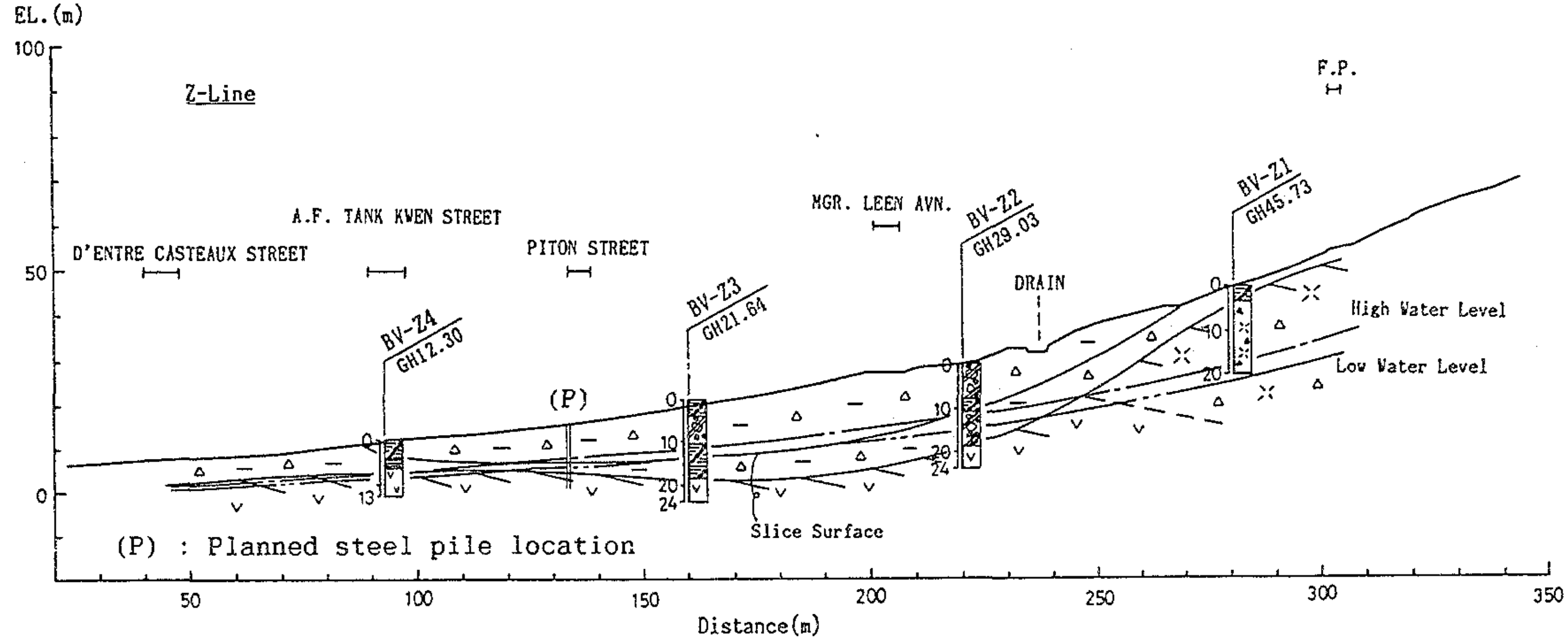
Fig. 2.3-5



GEOLOGICAL PROFILE ALONG Y-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.3-6

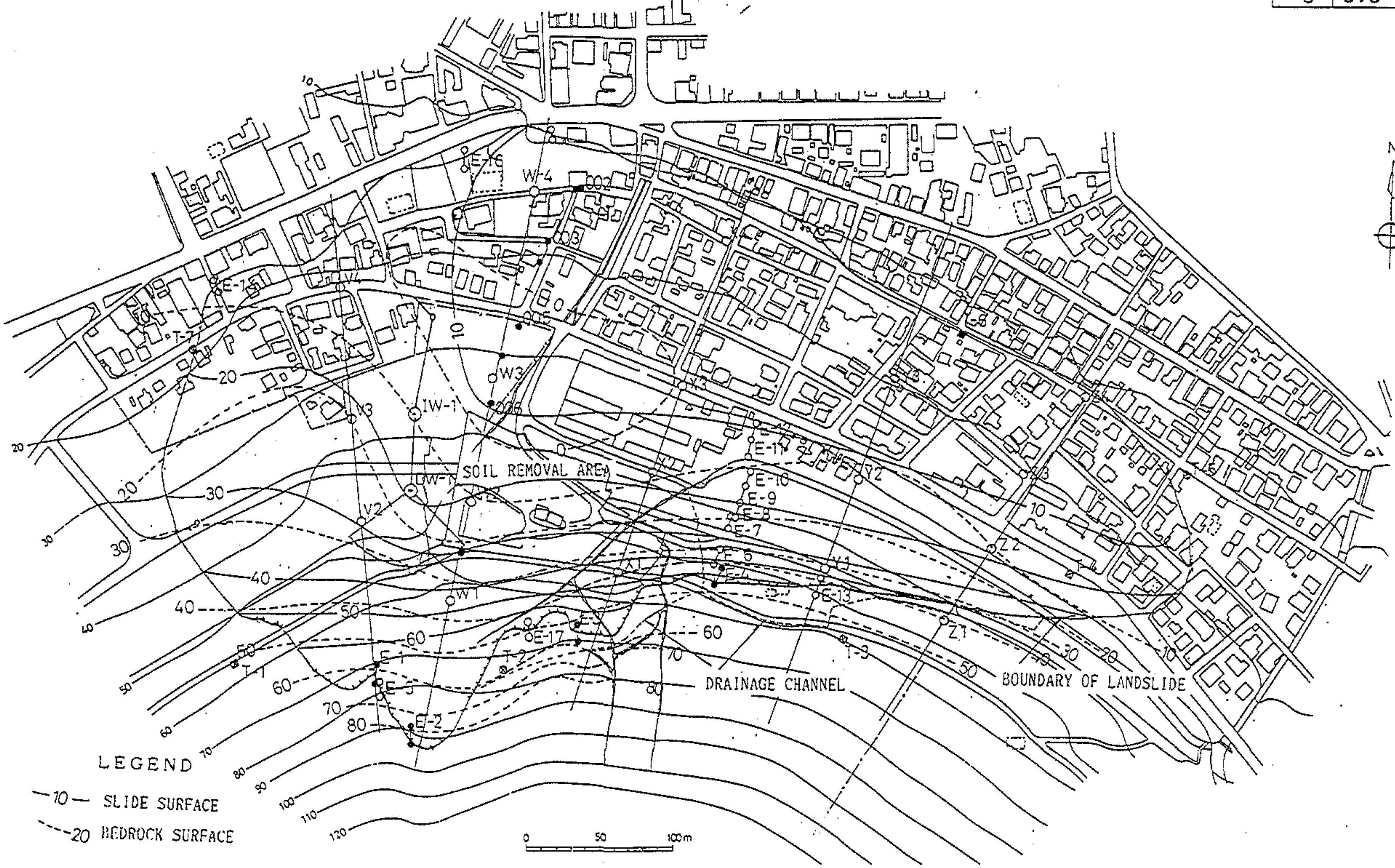


GEOLOGICAL PROFILE ALONG Z-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.3-7

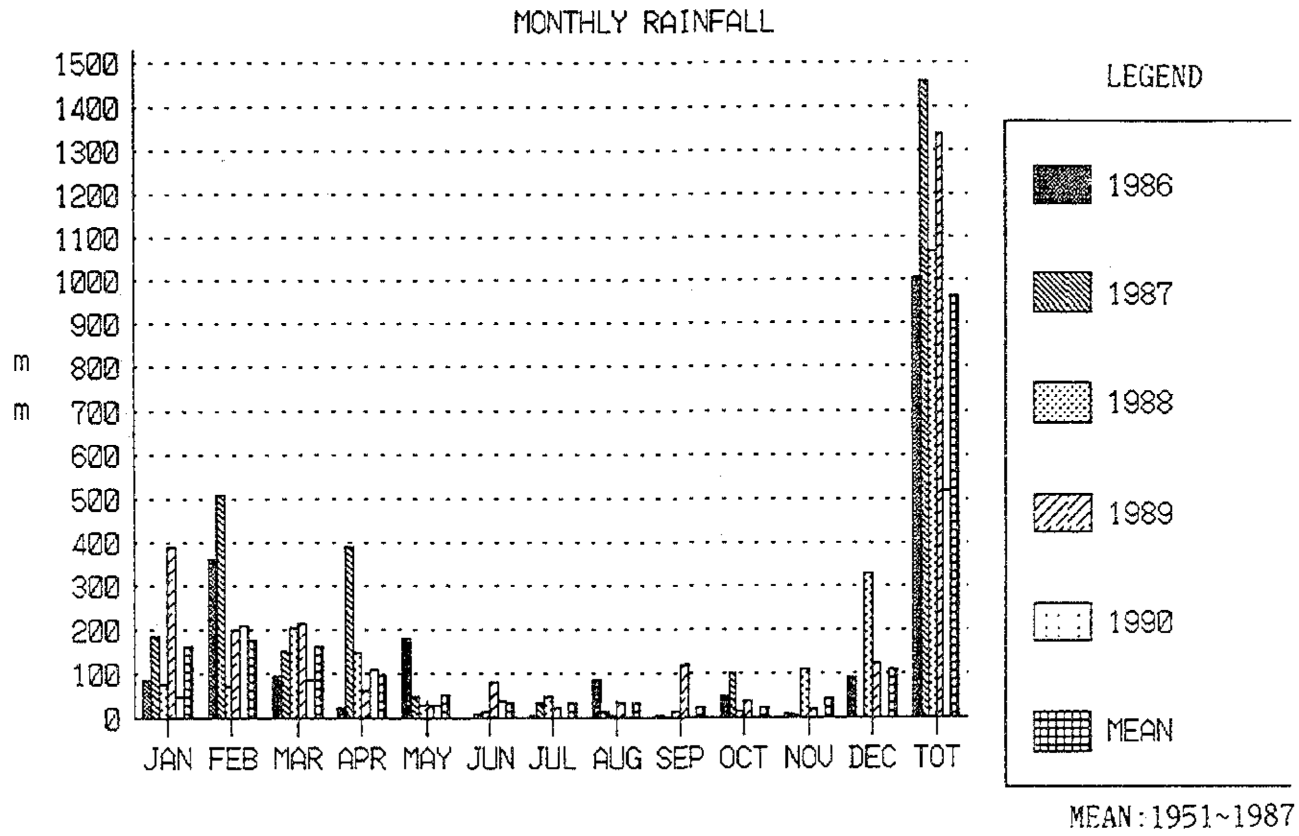


CONTOUR MAP OF ASSUMED SLIDE SURFACE AND BEDROCK SURFACE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.5-1

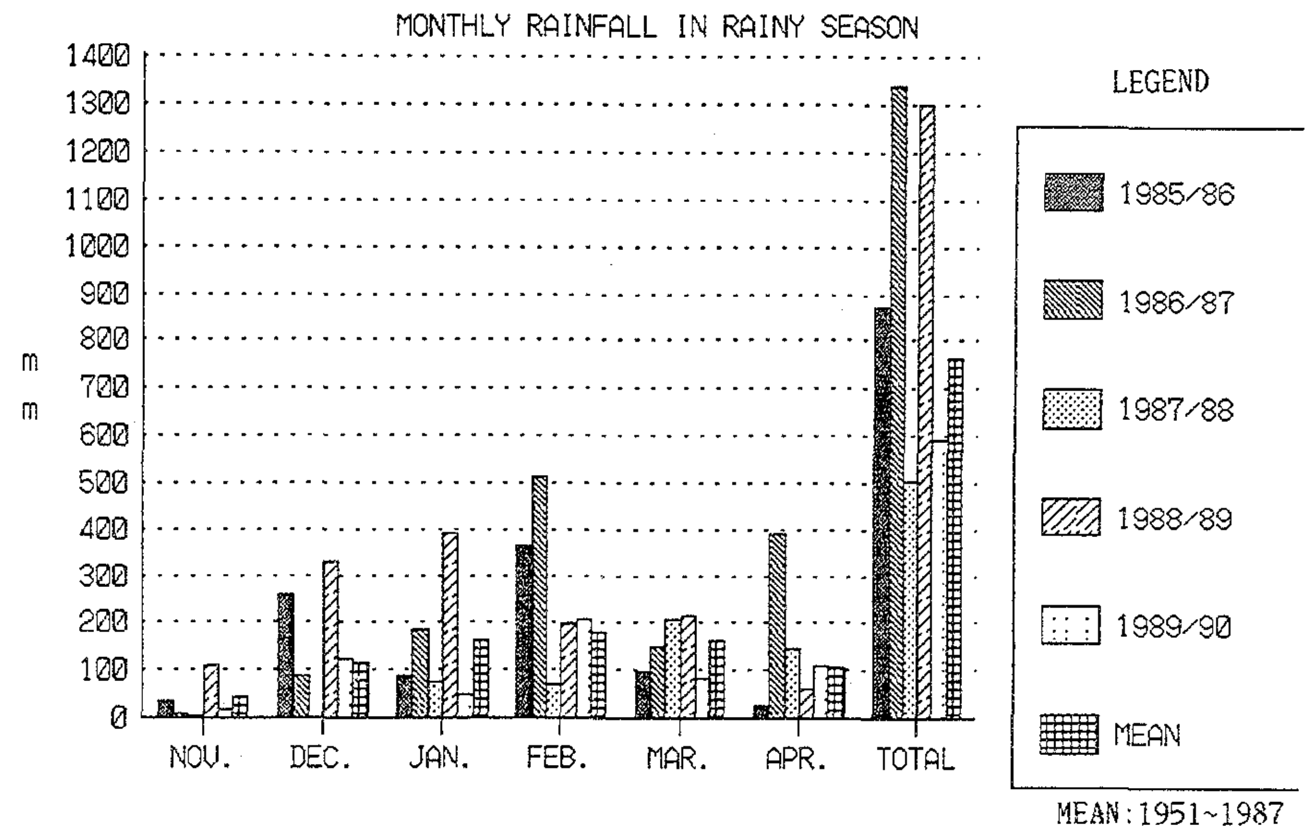


MONTHLY RAINFALL FROM 1986 TO 1990

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.5-2

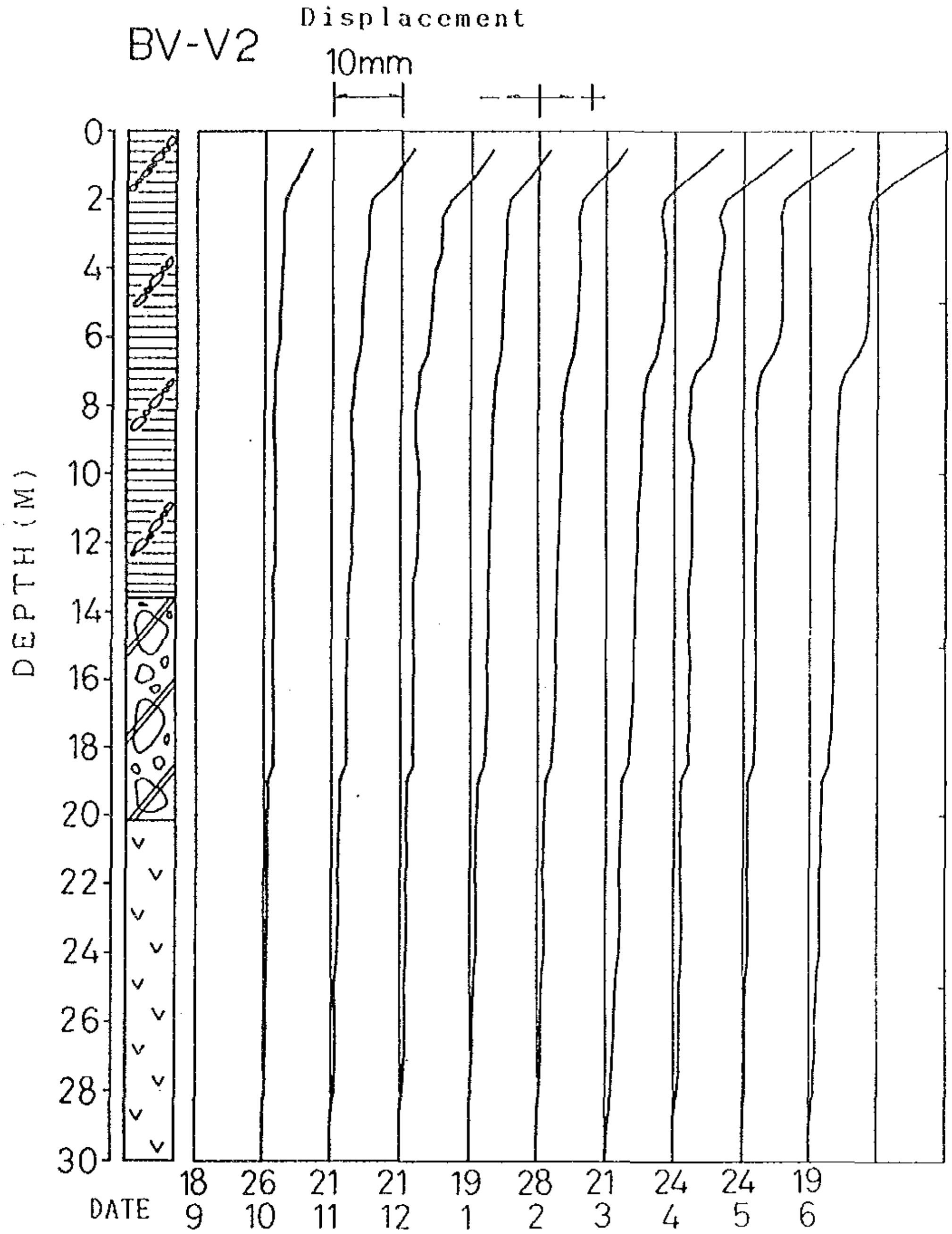


MONTHLY RAINFALL IN RAINY SEASON FROM 1985 TO 1990

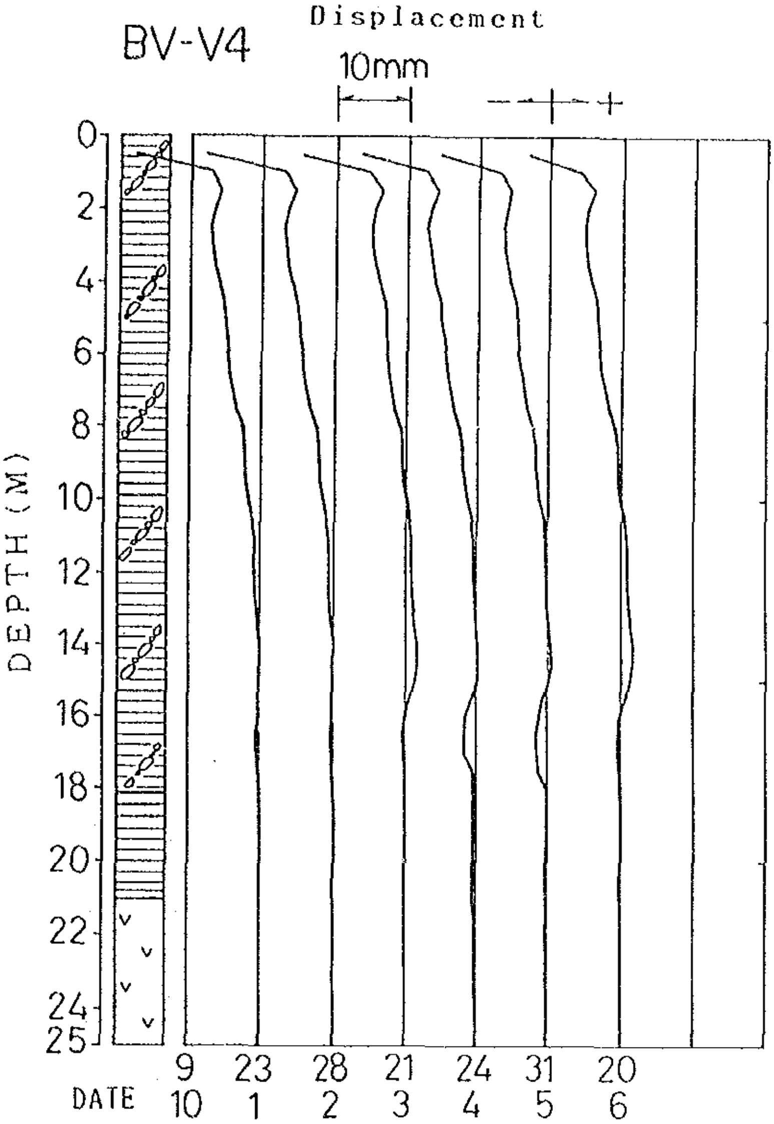
GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

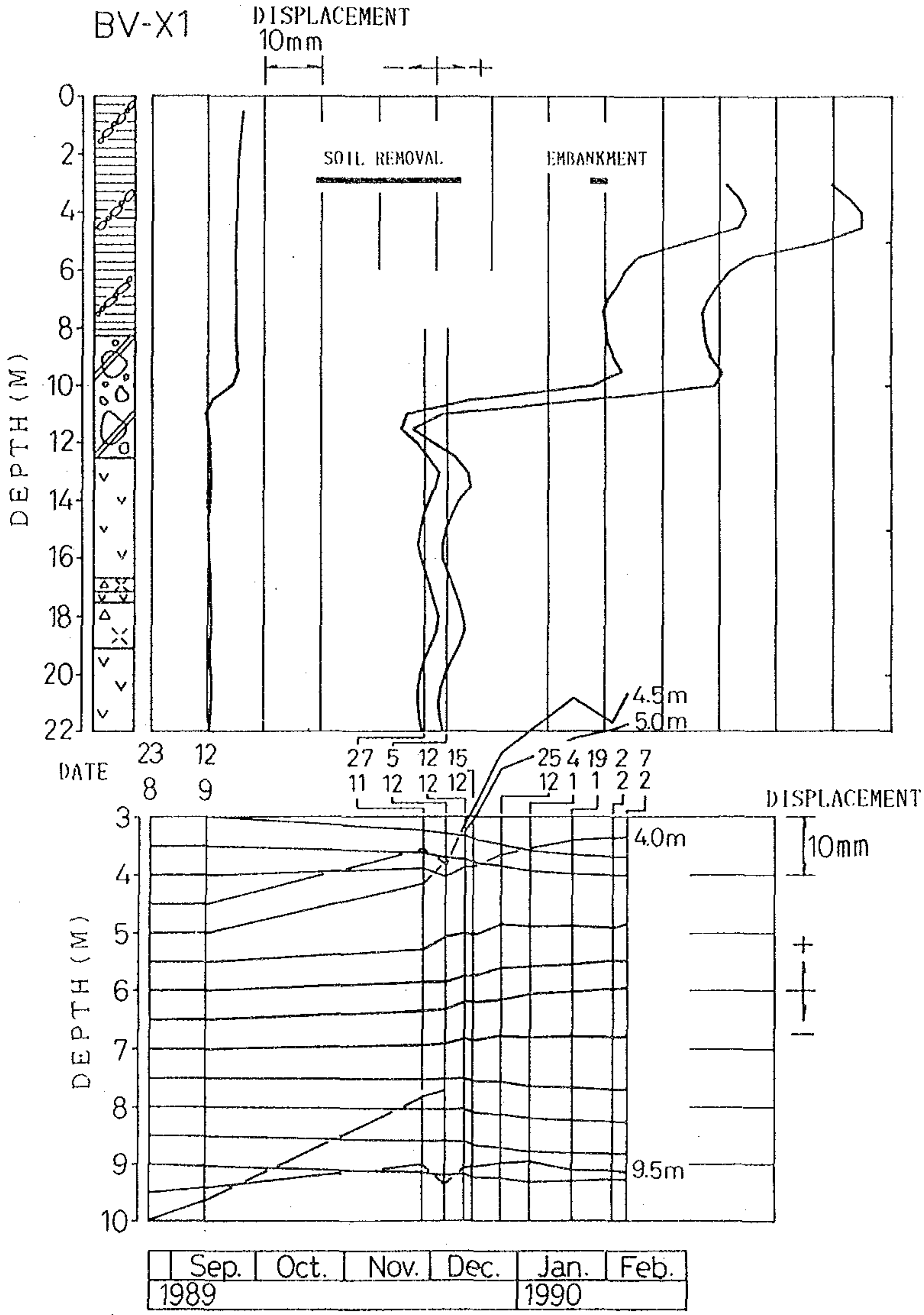
Fig. 2.5-3



DISPLACEMENT OF BOREHOLE INCLINOMETER IN BOREHOLE, BV-V2	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY



DISPLACEMENT OF BOREHOLE INCLINOMETER IN BOREHOLE, BV-V4	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY

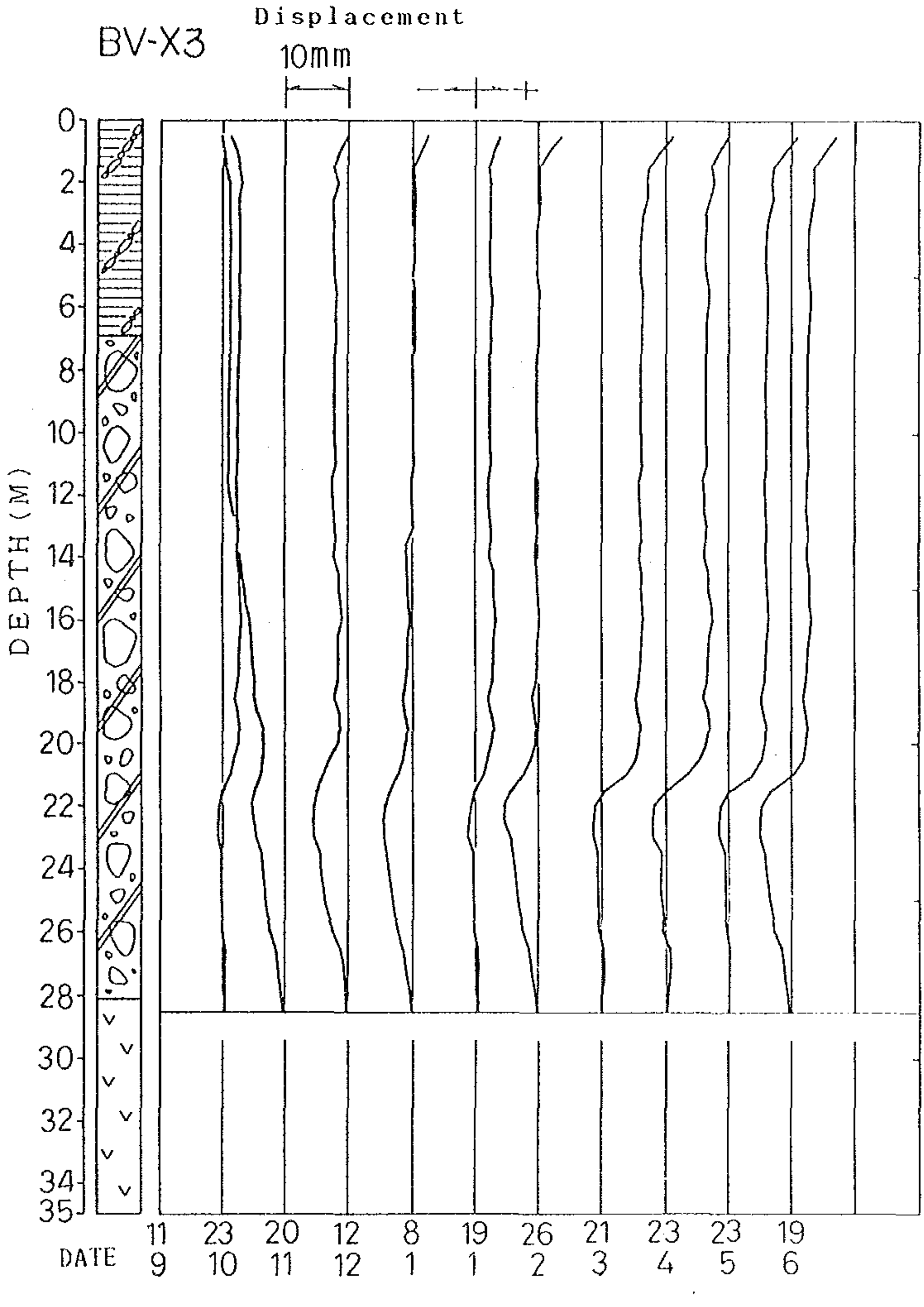


DISPLACEMENT OF BOREHOLE INCLINOMETER IN BOREHOLE, BV-X1

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.5-6

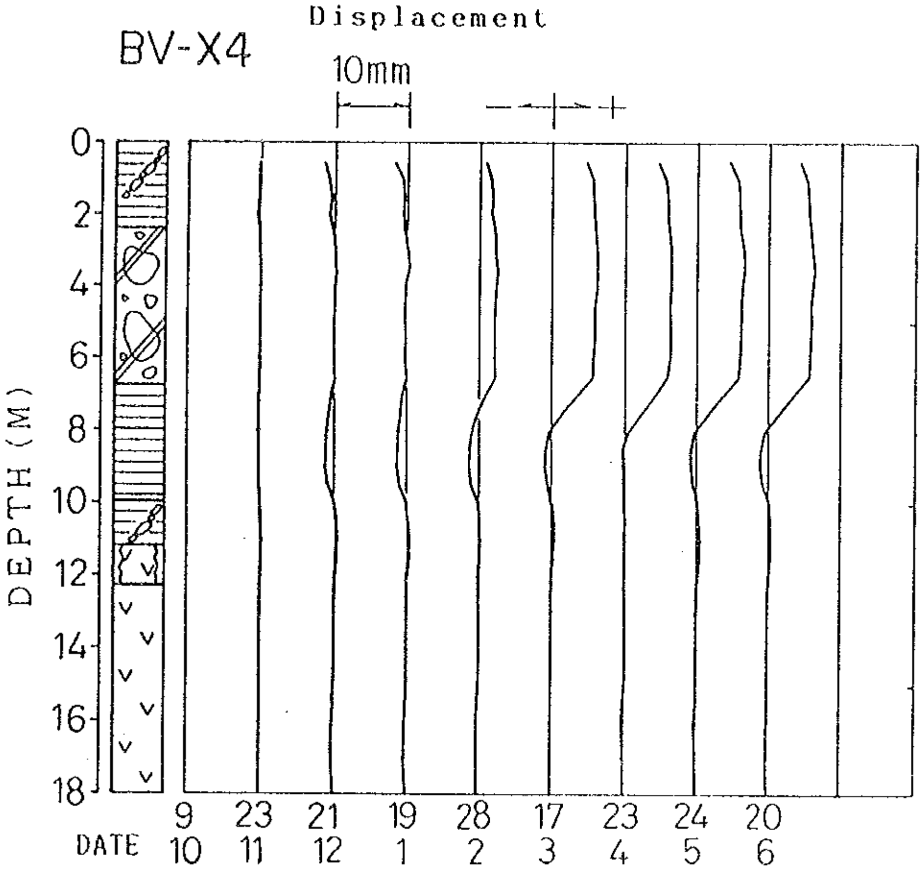


DISPLACEMENT OF BOREHOLE INCLINOMETER IN BOREHOLE, BV-X3

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.5-7



DISPLACEMENT OF BOREHOLE INCLINOMETER IN BOREHOLE, BV-X4	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY