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THE GOVERNMENT OF MAURITIUS

THE STUDY ON LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

FINAL REPORT

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NOVEMBER [illegible]

JAPAN INTERNATIONAL COOPERATION AGENCY

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SUPPORTING REPORT (I)(II)(III)

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SUPPORTING I

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

I.1 INTRODUCTION

I.1.1 General

The subject of this Study is a landslide which occurred on a lower slope of Signal mountain which faces Port Louis in the southwest of the city. The landslide occupies an area of about 400 m x 700 m in size. The location of the landslide is shown in Fig.I.1.1.

The Study on Landslide Protection Project in Port Louis was divided into two study periods. The first stage study was performed in the period from April 1989 to August 1989 to confirm the natural conditions of the area including La Butte landslide and to clarify the landslide mechanism. The main study in this period consisted of field investigations such as ground survey, geological investigation, installation of monitoring equipment, collection and analysis of monitoring data and so on.

Based on the all collected data, stability analysis on the concerned landslide was carried out, and planning of an experimental investigation.

In this study period installation of surface drainage channels was made as an emergency protection measure for preventing rain water from infiltrating into the ground.

This Supporting Report-I describes field investigation results, landslide mechanism of the landslide, stability analysis results, planning and designing of the experimental investigation and so on.

I.1.2 Field Investigations

Data collection by monitoring equipment was made from one (1) rainfall gauge, seven (7) tiltmeters, sixteen (16) extensometers, and eight (8) boreholes in which guide pipes for a borehole inclinometer were installed. Collection and analysis of monitoring data was continued from the beginning of this study period.

A contour map at 1/1000 in scale was prepared by ground survey. Five observation lines, along which most of the boreholes were drilled, were aligned to cross the landslide area for stability analysis north and south in the direction of maximum slope inclination of the landslide area. Leveling surveys along four lines following existing roads were conducted and clear subsidence of ground surface were confirmed at the upper part of landslide area.

Clayey materials of scree deposits were collected from the soil removal area and the drainage well, and the samples were supplied for laboratory tests. The clayey materials are classified as CH from their physical properties. The results of triaxial compressive tests indicate that mechanical properties of the clayey soil range from 1.0 t/m^2 to 3.0 t/m^2 in cohesion and from 5.5 deg. to 28.0 deg. in internal friction angle.

I.1.3 Collection and Analysis of Monitoring Record

The annual mean rainfall for 37 years from 1951 to 1987 was 960 mm, and about 79 % of this had been recorded in the rainy season from November to April. In recent years the annual rainfall which seems to have affected landslide movements has been as high as 1000 mm to 1340 mm.

Ten boreholes for 235 m were drilled for observation of the groundwater levels and eight boreholes for 195 m (8 holes) were drilled for installation of guide pipes for a borehole inclinometer. Drilled core samples were collected from the whole drilled depth of the boreholes for judgment of subsurface conditions especially for judgment of slide surfaces and bedrock surfaces in the landslide area.

Observation of groundwater levels was made in the boreholes drilled in this study and in the previous stage. Groundwater levels were gradually decreased after the rainy season from 1988 to 1990 and a large drawdown has been observed between July 1989 and the present. No significant recovery of groundwater levels was measured in the boreholes because rainfall was relatively small in the rainy season from 1989 to 1990.

Tilting of ground surface was measured by tiltmeters installed around the sliding area. There is no clear accumulating tendency in tiltmeter readings. However, a slight accumulating tendency is interpreted in the records from the tiltmeters installed in the eastern part of the landslide area.

Shearing displacement generally ceased after the 1988/9 rainy season. Slight displacements were observed in the boreholes inclinometer readings from measurements in the boreholes drilled in the eastern part of the landslide since the beginning of 1990 in the rainy season. The displacement is very small and landslide movement seems to be in a very quiet period for the whole landslide.

Readings of the borehole inclinometer in the borehole, BV-X1, indicate a clear shearing displacement of the borehole in shallow zones in November 1989. This shearing displacement was caused by movement of a small landslide that happened near extensometer E-3, at the eastern part of the soil removal work site where about 50000 m^3 of soil materials were removed for mitigation of the landslide movement as an emergency protection measure.

Extensometers are installed in tension zones, compression zones and in the central part of the landslide. Remarkable displacement of tension cracks was measured in the rainy season from 1988 to 1989. After the rainy season displacement of cracks ceased with the decrease in rainfalls. Distinct displacement was interpreted in the records obtained from extensometer, E-3. This displacement was accompanied by the small landslide happened in the vicinity of E-3 after the soil removal works carried out from October to November 1989.

I.1.4 Landslide Mechanism

The landslide that occurred at La Butte is classified as a colluvial landslide according to the general classification of landslides. Scree deposits constituting the slide masses of the landslide are composed of clayey soil with basaltic gravel. Very frequently developed slickensides are seen among the clayey soil, indicating that intensive stresses had been acting on the scree deposits.

Based on recovered core samples from borehole drilling, analysis of monitoring records, and site reconnaissance, geological profiles were prepared along the observation lines aligned by ground survey works. The landslide area is composed of scree deposits which overlie basaltic bedrocks. The thickness of the scree deposits is large in the western part and smaller in the eastern part. The depth to the slide surfaces is also large in the western part and smaller in the eastern part.

According to the monitoring data, landslide movement seems to have continued slightly in the eastern part of the landslide. No significant movements of the whole landslide were not observed during the study period because of the relatively small rainfall in the 1989/90 rainy season and also because of the effects of soil removal work carried out in the central part of the landslide area from October to November 1989. The landslide has been generally in a stable condition in this study period.

I.1.5 Installation of Surface Water Drains

Increasing groundwater levels in landslide area clearly reduces the stability of landslides. Therefore, decreasing of groundwater levels is one of the most important protection measures which can be taken to reduce landslide movement. Accordingly prevention of rainwater from infiltrating into the ground is considered to be an means of improving the stability of landslides.

From this point of view, laying out of surface water drains for 400 m was carried out at the initial stage of this study.

Sealing of open cracks was also conducted to prevent rainwater from infiltrating into the ground when laying out the surface water drains.

I.1.6 Preparation of Experimental Investigation

Based on the all data collected in the first stage and stability analysis results, planning of an experimental investigation was made for confirmation of the effectiveness of the drainage well for inhibiting landslide movement by decreasing the groundwater levels in the rainy season.

One drainage well of 20 m in depth with 1000 m groundwater collection boreholes (20 holes) were planned as an experimental investigation. For assuring smooth gravitational drainage of groundwater collected into the drainage well, one intermediate well is planned between the drainage well and the ground surface.

On the assumption of groundwater levels decreasing by about 2 m in the area surrounding the drainage well, stability analysis was made along the observation lines near the drainage well. The calculated results indicate about 6 % improvement in the safety factor by construction of the drainage well.

I.2 FIELD INVESTIGATION

I.2.1 Field Investigation

Field investigations carried out in this study were divided into two stages. The first stage was composed of installation of monitoring equipment, topographic survey, geological survey and soil mechanical tests. The investigations in the first stage were for clarifying the natural conditions in the landslide area which is about 400 m in length and about 700 m in width.

Following the investigation results in the first stage, the second stage investigation was planned and executed experimentally. The second stage investigation consisted of construction of a drainage well, an intermediate well and drilling of groundwater collection boreholes and drainage boreholes. The investigation in the second stage was for confirming the effects of the experimental investigations.

The core boring points were selected along five observation lines, Line-V, Line-W, Line-X, Line-Y and Line-Z aligned in the landslide area for effective stability analysis for the whole landslide area. The lines, along which stability analysis of landslide were made, were selected parallel to the main sliding direction, north and south, at the central part of the landslide. Both ends of the lines were extended outside the landslide area sufficiently for stability analysis. The results of stability analysis influenced the planning of the landslide protection measures in methods and quantities.

In this study five lines were selected on the base of the location of open cracks, movements of extensometers and tiltmeters, direction of landslide movement and so on. Among these five observation lines, Line-W and Line-Y, were considered to be the main observation lines for the concerned landslide. The alignment of observation lines and the location of the investigation site are shown in Fig.I.2.1.

I.2.2 Installation of Monitoring Equipment

Monitoring equipment, which consisted of one (1) rain gauge, seven (7) units of tiltmeters, sixteen (16) units of extensometers were installed in and around the landslide area. Measurement of borehole inclinometer and groundwater level indicator were also made in eight (8) and ten (10) drilled boreholes respectively. The location of the monitoring equipment is shown in Fig.I.2.1.

One rain gauge was installed in the yard of Ecole de la Montagne (the government elementary school) by previous JICA expert team in 1988. The rainfall data had been obtained at Line Barracks about 0.7 km northeast to the landslide area before installation of the additional rain gauge.

Seven units of tiltmeters were installed at the marginal parts of the landslide area by the previous JICA expert team. The purpose of the installation of tiltmeters was to confirm the boundary between the moving zone and the stable zone.

Sixteen (16) units of extensometers of which four units had been installed by the previous JICA expert team in 1988 and twelve units were installed by the study team at the initiating stage of this study period. Six units of them were installed at the crown part of the landslide for measurement of the tension movement; three units were at the toe portion of the landslide for measurement of compression movement; and seven units were aligned in a straight line beside Ecole de la Montagne for confirming the possible boundary between compression and tension zones of the landslide. One extensometer, E-16, installed at the crown portion was replaced to the western part of the extensometer, E-3, afterward for confirming movement of a small landslide. The extensometers installed in this study are summarized in Tables I.2.2-1 and I.2.2-2.

LOCATION OF EXTENSOMETERS

Extensometer	Location	Purpose
E-5	upper part of E-1	to confirm extension of

	sliding zones to upper part	
E-6 - E-12	lower part of E-4 between tension zone and compression zone	to confirm the boundary
E-13	about 70 m east of E-4	to confirm movement in eastern zone
E-14 - E-16	at upheaving zones	to confirm upheaving movement

Installation of guide pipes for a borehole inclinometer was made in eight boreholes drilled along the observation lines, V-line, X-line and Z-line. Measurement of the borehole inclinometer was made in these boreholes by inserting the borehole inclinometer into these boreholes.

Measurement of groundwater levels had been made in ten boreholes drilled along the observation lines, V-line, W-line, Y-line and Z-line for confirming groundwater levels through the rainy season and the dry season. The target depth of boreholes was decided to be sufficient for measurement of groundwater levels at the initial stage of the investigation.

I.2.3 Topographic Survey

The results obtained from the topographic survey carried out in this study are as follows:

a) Topographic profiles on a scale of 1 : 500

b) Contour map covering landslide area on a scale of 1 : 1000

(1) Establishment of traverse points

With regard to the preparation of topographic profiles and a contour map, several traverse points were established in and around the landslide area. For observation of ground surface movement generated by sliding movement four traverse lines and two fixed lines were established in the landslide area. The alignment of these observation lines are shown in Fig.I.2.3-1. Traversing survey was made in cooperation with the Ministry of Housing in this connection.

(2) Establishment of leveling points

There were existing two leveling points around the landslide area; however, three additional leveling points were established as follows in cooperation with the Ministry of Housing.

ELEVATION OF LEVELING POINTS

Existing	:	TBM 2	:	elevation 18.527 m
Existing	:	BM (9603)	:	6.434 m
New	:	KBM 1	:	29.008 m
New	:	KBM 2	:	16.510 m
New	:	KBM 3	:	7.028 m

(3) Leveling

Four observation lines, Line-1, Line-2, Line-3 and Line-4, were established in the period December 1987 to January 1988 for the leveling survey. The leveling survey was conducted in May 1989 and the results are summarized in topographic profiles, Fig.I.2.3-2 to Fig.I.2.3-4.

A further seven (7) lines of leveling survey were established in this period to compensate for survey points removed during soil removal operations. The locations of these lines are along each berm in the soil removal area and the surface drainage channel laid along the upper slope of extensometer, E-3. These locations were selected for surveying the ground surface displacement in the soil removal area and for confirmation of the possibility of landsliding if it was to extend to higher slopes. The layout of survey points and survey results are shown in Fig.I.2.3-5 and Fig.I.2.3-6.

As for the four survey lines, Line-1, Line-2, Line-3 and Line-4, covering the whole landslide area, significant subsidence had been surveyed in the section between 1/20 and 1/33 along Line-1, which is aligned at the crown portion of the landslide area.

Significant ground subsidence was observed along Line-1. The maximum amount of subsidence was about 50 cm at observation point, 1/21. Observation results are shown in Fig.I.2.3-2. Along observation Line-2 subsidence of about 2.5 cm was observed in the section between 2/9 and 2/21, and upheaval of about 12 cm was observed in the section between 2/30 and 2/39, which is shown in Fig.I.2.3-3. Along observation Line-3 upheaval of 10 cm was observed in the section between 3/13 and 3/26, and upheaval of about 3 cm was observed in the section between 3/28 and 3/33, which is shown in Fig.I.2.3-4. Large movements were observed along Line-4; however, these seem to be caused by other phenomena than displacement caused by landslide movement, and could not be interpreted, in any definite way.

However, leveling survey in the section between 1/19 and 1/27 along Line-1 could not be conducted after starting of the soil removal works in October 1989 because the survey points in this section had been destroyed by the soil removal works. Among the remaining survey points along Line-1, larger subsidences were measured on the survey point, 1/33. The subsidence amounts were about 30 cm in September 1989 and about 32 cm in the middle of May 1990.

No significant accumulation of subsidence or upheaval was no observed by the leveling survey along Line-2, Line-3 and Line-4. Upheaval movements of about 30 cm were seen in the section between 2/36 and 2/39 along Line-2. These survey points are aligned along Lime Street. The concerned section along this street was damaged intensively by dumping trucks during the soil removal works. Therefore, the movements in this section may have been caused by the soil removal works.

The additional seven survey lines were established in May 1990 and the leveling survey was conducted only twice in May and June 1990 in this study period. The height difference between the first and the second leveling survey works are shown in Fig.I.2.3-6. It is difficult to describe the ground surface movements in detail with survey results collected over such a short period in the dry season.

However, survey lines aligned in the upper part of the soil removal area seem to have a slight subsiding tendency; the line along the drainage channel at El.70 m to El.74 m subsided about 5 cm; the line along the berm at El.55 m subsided 2 cm to 4 cm; the line along the berm at El.52 m subsided 2 cm to 3 cm. Clear movements are not seen along the remaining survey lines.

As for the four survey lines, Line-1, Line-2, Line-3 and Line-4, covering the whole landslide area, significant subsidence had been surveyed in the section between 1/20 and 1/33 along Line-1, which is aligned at the head portion of the landslide area as mentioned in the Interim Report. The maximum amount of subsidence, exceeding 53 cm, was observed at survey point, 1/21, on Line-1 in September 1989.

However, the leveling survey in the section between 1/19 and 1/27 along Line-1 could not be conducted after the start of the soil removal works in October 1989 because the survey points in this section were destroyed by the soil removal works. The additional seven (7) lines of leveling survey were established in this period. The locations of these lines are along the berms in the soil removal area and the surface drainage channel laid on the upper slope of extensometer, E-3. These locations were selected for surveying the ground surface displacement in the soil removal area and for confirmation of whether the landslide would to extend to upper slopes or not. The layout of the survey points and survey results are shown in Figs.I.2.3-5 and Fig.I.2.3-6.

(4) Cross sectional survey

Cross sectional surveys were conducted along the lines, V, W, X, Y, Z, and the line through the drainage well and the intermediate well. These lines were extended into the stable area outside the landslide area sufficiently for eliminating landslide movements. The lines were extended to the upper portion above the elevation of 120 m and downwards for more than 30 m beyond the cross-sectional points with the leveling survey Line-4. Open cracks and topographic condition along these cross sectional lines were carefully observed and indicated in the topographic cross sections.

(5) Planimetric survey

Land use of the landslide area had been changed after completion of the existing 1:1000 scale topographic map prepared by the Ministry of Housing. Therefore, a new contour map at a scale of 1:1000 was prepared in this study.

(6) Traversing survey work

A traversing survey was repeated on the following points twice a year.

Survey points : T0, T1, T2, T3, F0, F1, F2, I/21, T/31

The elevations of the fixed points obtained in this study are as follows:

ELEVATION OF SURVEY POINT

POINT	ELEVATION (m)
T0	19.575
T1	18.478
T2	20.073

T3	21.613
F0	33.600
F1	33.216
F2	32.542

I.2.4 Geological Investigation

(1) Borehole drilling

In this study drilling of 18 boreholes along the selected five lines for stability analysis was planned for clarifying subsurface geological conditions. The locations of the boreholes were decided on the basis of analysis of existing data and field reconnaissance results. The borehole locations are shown in Fig.I.2.1 Core drilling was carried out in two phases, the phase-1 and the phase-2. The core drilling for 235 m (10 holes) was made in the phase-1 for groundwater level observation and for 195 m (8 holes) in the phase-2 were for measurement of borehole inclinometer. The quantity of drilling is summarized in Table I.2.4-1 and Table I.2.4-2.

Permeability tests by the open-end method was conducted in the zone of talus deposits, and Lugeon tests in the rock formations. Standard penetration tests (SPT) were performed in order to confirm the strength of subsurface loose materials. Pvc pipes were inserted into drilled boreholes after completion of drilling of each borehole.

1) Geological condition judged from recovered core samples

Judging from the recovered core samples, there are two kinds of materials rock formations and talus deposits or scree deposits. The bedrocks are composed of basaltic lavas and volcanic breccias, which intercalate basaltic lavas. The basaltic lavas are very hard in general, and are mostly recovered as fragmental cores because of development of joints except for partially recovered long cylindrical cores. The volcanic breccias composed of brownish volcanic ashes and volcanic boulders, which are angular to subangular. A summary of core log of the recovered core samples is shown Figs.I.2.4-1 and I.2.4-2.

Scree deposits are composed of basaltic gravels with clayey soil. They are classified into two types; predominant gravel materials with clayey soil and predominant clayey soil with gavel. There were some sections from which only gravel materials were recovered. These sections interpreted to be sections of clayey soil with gravel on the assumption that the clayey soil was washed out during drilling.

Dark brown clayey materials, which are observed among scree deposits, seem to be derived from weathered volcanic breccias and volcanic lavas. Generally, the clayey materials are sandy or with some contents of small gravel and are poor in moisture contents.

2) Judgment of sliding surfaces by observation of recovered core samples

There was no sign of sliding movement in the rock formations, but inclusion of clayey materials was observed frequently in scree deposits. Therefore, sliding surfaces are considered likely to be present in the zones of scree deposits. The judgment of sliding surfaces in this landslide is not very clear because the subsurface loose materials are composed of fragmental cores mostly, and clayey layers are intercalated with fragmental cores in many sections. The sliding surface seems not to be a simple plane but a combination.

The clayey materials seem to be derived from volcanic breccias. They appear to vary from highly cohesive to less cohesive soils. They are reddish brown sandy materials with fine gravel materials in general, and they are partly accompanied with slickensides. For judgment of the sliding surfaces, special attention was paid to the high cohesive clayey materials and clayey materials accompanied by slickensides. The sections of the clayey layers which seem to have characteristics of possible sliding surfaces are summarized as follows for each boreholes.

SECTIONS OF POSSIBLE SLIDING PLANE

BOREHOLE No.	DEPTH (m)
BV-V1	24.00
BV-V2	18.50
BV-V3	13.50
BV-V4	5.70
BV-W1	15.80
BV-W2	25.00
BV-W3	24.50
BV-X1	11.75
BV-X2	20.60

BV-X3	19.80
BV-X4	6.50
BV-Y1	4.20
BV-Y2	16.60
BV-Y3	---
BV-Z1	---
BV-Z2	10.60
BV-Z3	13.90
BV-Z4	3.30

3) Standard penetration test (SPT)

SPTs were conducted in the boreholes for confirming the strength of subsurface loose materials. It was possible to conduct the tests only in limited shallow zones, and the test was impossible in deeper parts in general because of inclusion of gravel materials.

4) Permeability test

Permeability of materials in the landslide area was examined by means of the open-end test in loose scree deposits and the Lugeon test in rock formations. The results of permeability tests are summarized in Table I.2.4-3 and Table I.2.4-4. The P-Q graph obtained from the Lugeon test is shown in Fig.I.2.4-3.

(2) Geology along observation lines for stability analysis

The lines for the stability analysis are aligned in the direction of maximum slope inclination of the landslide area. They are V-line, W-line, X-line, Y-line, and Z-line from the east to the west. The geological condition along these lines is summarized below. Alignment of these lines are shown in Fig.I.2.1 and geological profile along the observation lines are shown in Fig.I.2.4-4 to Fig.I.2.4-9.

1) V-line

V-line is aligned at the most western part of the landslide area. The depth to the basaltic rock formation is to be 15 m to 25 m. Thick talus deposits or scree deposits composed of basaltic gravel and boulder with clayey soil overlie the rock formation. The depth to the sliding surface from the ground surface is assumed to be about 20 m at maximum, being decreased gradually to the lower slope. The sliding surfaces are present in loose material zones.

2) Line along the drainage well and the intermediate well

This line aligned north and south between V-line and W-line, connecting the drainage well and the intermediate well. Thick scree deposits underlie the area along this line to about 30 m in maximum thickness. The bedrock surface is approximately parallel to the ground surface. Clayey materials are assumed to overlie the bedrock surface at the toe portion of the landslide with about 5 m in thickness.

3) W-line

The geological conditions were judged from the core samples recovered in the first investigation stage and observation data from the borehole inclinometer. The maximum depth to the rock formation was assumed to be very deep about 40 m and the surface of the rock formation seems to be concaved around this line. The depth to the rock formation in the part above the abandoned reservoir for water supply is 25 m to 30 m. Brownish clayey soil overlies the rock formation in the section lower than 5 m to 6 m in elevation. This clayey layer seems to be extended to the north direction widely. The sliding surface was assumed to be at about 27 m, indicating maximum depth among other lines. The sliding surfaces were confirmed in the zone of scree deposits.

4) X-line

The line crosses the sliding area at the central part. The depth to the rock formation is more than 30 m at the deepest part, decreasing to the mountain side suddenly. The depth to the sliding surface is estimated to be about 20 m at deepest part. The inclination of the sliding plane is gentler than the sliding surface along W-line.

5) Y-line

The depth to the rock formation is about 22 m along this line. The depth to rock formation seems to decreased between X-line and Y-line. The maximum depth to the sliding surface is about 20 m, indicating almost the same depth as on X-line. The length of the landslide between the crown part and the toe portion is about 200 m along this line, being decreased from X-line. The shallower depth to the rock formation seems to relate to the smaller size of the landslide at this point.

6) Z-line

This line is at the most eastern part. The depth to the rock formation is about 15 m at the deepest part, indicating the shallowest depth among the selected lines. The assumed rock surface is estimated to be parallel to the topographic surface. The sliding surface is estimated at about 12 m in depth. The length of landslide decreases to about 150 m along this line.

I.2.5 Soil Laboratory Test

Soil laboratory tests were also conducted in the study for the purpose of confirming the physical and mechanical properties of the clayey materials in the landslide area.

(1) Sampling

The fundamental stability analysis of a landslide area has to be made by use of obtained mechanical properties of clayey materials along the sliding surfaces. Since the sliding surfaces are estimated to lie at 20 m to 30 m depth in the central part of the landslide area, sampling of clayey materials from the sliding surfaces seemed to be impossible. Sampling of clayey materials was made in the soil removal area and in the drainage well as the works progressed but only at a relatively shallow depth.

The undisturbed sampling of two series of samples, S/R-1 and S/R-2 which were composed of 5 each samples, was made in the central part of the soil removal area at depths of about 5 m and 7 m with the progress of soil excavation works. The undisturbed sampling for other two series, D/W-1 and D/W-2 which were composed of 4 samples and 5 samples, was made in the drainage well at the depth of 10.5 m and 17.0 m with the progress of excavation work of the drainage well.

Slickensides are developed very frequently in the collected clayey samples, and undisturbed sampling was rather difficult because samples were apt to be cracked along the developed slickensides. Though the samples were collected from relatively shallow zones, these samples seem to be intensively affected by sliding movements and the samples are considered to have similar properties to the clayey materials which would be expected along the slide surfaces. Therefore, the samples from the soil removal area and the drainage well were supplied for the laboratory tests instead of the clayey materials which may lie along the slide surfaces.

(2) Results of laboratory tests

According to the specification prepared by JICA study team, laboratory tests were performed, based on British Standards in general. The test items and quantities conducted were as follows:

TEST ITEM	QUANTITY
Physical Test:	
Specific gravity	4 times
Liquid limit and plastic limit	4 times
Particle size distribution	4 times
Moisture content	28 times
Bulk unit weight	28 times
Mechanical Test:	
Uniaxial compressive	16 times
Triaxial compressive (UU)	12 times

Physical tests were conducted on four (4) representative disturbed samples except for a moisture content test and a bulk unit weight test for undisturbed samples. When mechanical tests were performed the moisture content test and bulk unit weight test were conducted at the same time. On the other hand, mechanical tests were made on four series of undisturbed samples, S/R-1, S/R-2, D/W-1, and D/W-2, of which each series was collected from the same site.

The test results are shown in Tables I.2.5-1 and I.2.5-2. The physical test results may be summarized as follows:

PROPERTY	VALUE
Specific gravity	2.56 - 2.79
Liquid limit	67.5 - 122.3 %
Plastic limit	40.5 - 54.8 %
Plastic index	20.8 - 76.6 %
Moisture content	23.6 - 42.7 %
Unit weight	1.64 - 3.56 t/m^3

The particle size of the materials was analyzed as well by the laboratory test, indicating very fine distribution. The distribution of materials finer than 1.0 micron is more than 50 % for three samples out of four representative samples. The remaining sample shows the contents of materials finer than 1.0 micron is 27 %. The particle size of 50 % distribution of this sample is 15 micron, indicating very fine distribution. The particle size distribution curves as shown in Fig.I.2.5-1 to Fig.I.2.5-4.

The clayey materials tested were classified as CH according to the unified soil classification. Natural moisture content is in a relatively small range for clayey materials. This seems to be related to deeper groundwater levels when samples were collected.

The mechanical properties of the materials are summarizes hereunder. Large difference among the obtained values on the uniaxial compressive test seems to depend on the condition of samples. The test on the samples which had frequent slickensides were resulted to be very small values. After saturation of samples, triaxial compressive tests were performed under UU condition with measurement of pore pressure during the test. The obtained Mohr's circles are shown in Fig.I.2.5-5 to Fig.I.2.5-8.

PROPERTY	VALUE
Uniaxial Compression Test:	
Compressive strength	1.5 - 31.2 t/m^2
Triaxial Compression Test:	
Cohesion	1.0 - 3.0 t/m^2
Internal friction angle	5.5 - 28.0 deg.

(3) Strength of clayey materials for the stability analysis

In the stability analysis for the designing of experimental investigation, the following values were assumed for the clayey materials along the assumed slide surfaces.

Cohesion	1.0 t/m^2 (survey lines, V to Z)
Internal friction angle	6.0 deg. to 9.9 deg. (survey lines, V to Z)

The test results were obtained on the clayey materials collected from the soil removal work site and the excavation site of drainage well. The sites are underlain by loose talus deposits or scree deposits, and the materials seem not to be largely changed with depth as it is observed in the drainage well. The materials for the laboratory tests are not from the slide surfaces but from scree deposits above the assumed sliding surface. However, there are developments of frequent slickensides in the clayey materials, and the materials seemed to be stressed and disturbed intensively.

The obtained values from the laboratory tests range 1.0 t/m^2 to 3.0 t/m^2 (cohesion) and 5.5 deg. to 28.0 deg. (internal friction angle). There is no large difference between the obtained test values and the assumed values for the stability analysis. Taking the natural conditions and obtained test results into consideration, the stability analysis is considered to be reasonable with proper assumptions on the properties of clayey materials which are expected along the slide surfaces.

I.2.6 Collection and Analysis of Monitoring Records

Collection and analysis of records from monitoring equipment installed at and around the landslide area are inevitable tasks for clarifying landslide mechanisms. Accordingly continuous technical data collection has been made since the start of this study.

In addition to the existing monitoring equipment consisting of one (1) set of rain gauge, seven (7) sets of tiltmeters, four (4) sets of extensometers, and eight (8) sets of inclinometer guide pipes, several sets of monitoring equipment were installed for this study. They are one (1) set of rainfall gauge prepared by the previous JICA expert team, twelve (12) sets of extensometers, and guide pipes for a borehole inclinometer into eight boreholes for displacement measurement of the slide surface. Strainer pipes for groundwater level observation were inserted into ten (10) boreholes. The location map for the monitoring equipment is shown in Fig.I.2.1.

(1) Rainfall

The study area has an oceanic climate characterized by a rainy season from November to April and a dry season from May to October. The the rainy season have close relationship with landslide movements because groundwater levels will be increased by heavy rainfalls in the rainy season.

Rainfall records around the La Butte area are available in the period between 1951 and 1987. The records were obtained at Line Barracks at about 0.7 Km northeast to the landslide area. Mean annual rainfall is about 960 mm in the above period for 37 years. About 79 % of the total annual rainfall (or 755 mm in total rainfall) is recorded in the rainy season from November to April in the above mentioned period.

After the occurrence of significant landslide movements, one set of rainfall gauge was installed at Ecole de la Montagne (the elementary school), which is situated in the center of the landslide, at the end of 1987 in the preliminary investigation stage for this landslide. Collection of rainfall data from this rainfall gauge has been continued since January 1988 for confirmation of actual rainfall in the concerned area.

Monthly rainfalls since 1951 in and around La Butte area and monthly mean rainfalls in the period between 1951 and 1987 are summarized in Tables I.2.6-1 and I.2.6-2 and shown in Figs.I.2.6-1 and I.2.6-2.

Rainfall which seems to have affected to landslide movements ranges from an annual amount of 1000 mm (in 1986) to 1340 mm (in 1987) in the recent years. The annual rainfalls of 1000 mm and 1340 mm is equivalent to the proportion of 105 % and 152 % of the annual mean value recorded from 1951 to 1987. Periodical recurrence of heavy and light rainfall on an annual basis is seen in the records since 1986, although clear periodical returning is not seen in the long term from 1951 to 1986.

A heavy rainfall of about 1340 mm, which is equivalent to 177 % of the mean value in the rainy season from 1951 to 1987, was recorded in the rainy season from 1986 to 1987. It is considered that the occurrence of the sudden and significant landslide in 1987 was caused by this heavy rainfall. The total rainfall during the rainy season from 1987 to 1988 was only 500 mm, which is equivalent to 67 % of the mean value from 1951 to 1987. Though the rainfall amount was relatively small in and after this rainy season, landslide movement seems to have been continued.

However, a heavy rainfall more than 1300 mm, which is equivalent to 172 % of the mean rainfall in the rainy season from 1951 to 1987, was recorded in the rainy season from 1988 to 1989. This large amount of rainfall is believed to have triggered remarkable displacement on extensometer records.

On the other hand, the rainfall in the rainy season from 1989 to 1990 was only 590 mm, which is equivalent to 45 % of the rainfall in the rainy season from November 1988 to April 1989. Stable condition of the landslide movements on the whole in and after this rainy season seems to depend on this small rainfall and also the soil removal works of the emergency protection measures. The soil removal works are believed to be very effective on the stability of the main part of landslide area so far. The monthly mean rainfall in the rainy season from 1986 to 1990 is shown in Fig.I.2.6-2.

(2) Borehole inclinometer

In the preliminary investigation stage in 1987, eight boreholes were drilled and guide pipes for inclinometers were inserted into 5 boreholes. The measurement had been conducted in 3 boreholes since December 1987 and in 2 boreholes since October 1988, but the measurement had been halted since October 1988. The observation was started again in this study in order to clarify the landslide movements since 1988. The guide pipes for inclinometers were inserted in the boreholes, A002, A003, A005, A006, and A007, which are approximately in a line along the Line-W, for stability analysis. The results of recent reading of inclinometers are summarized below for each unit. The records of inclinometer reading are shown in Fig.I.2.6-3 to Fig.I.2.6-15.

1) A002 (borehole depth = 20.0 m)

Larger shearing deformation was interpreted on the record of this inclinometer between March and June 1988 and the deformation had a tendency of increasing. The deformation increased after the rainfall of 10 mm to 50 mm in March 1988. The rainfall in April 1988 was also 10 mm to 20 mm and the deformation was believed to continue. However, the details of the deformation are not clear because no measurement was carried out between April and May 1988. It was not possible to insert the inclinometer probe deeper than 3.0 m during the observation in June, 1989.

2) A003 (borehole depth = 19.0 m)

Shearing deformation was gradually found in the section between 11.0 m and 12.0 m after staring observation in December 1987 and the deformation increased in the period between February and March 1988. The depth of the sliding plane was estimated at 11.0 m. Irregular deformation was observed below 12.0 m on 13th June 1988. No measurement was possible in the section below 11.5 m on 22nd June 1988. The guide pipe was estimated to have been cut below 11.0 m because observation was possible only up to 11.0 m in May 1989.

3) A005 (borehole depth = 16.0 m)

There was no remarkable deformation after starting the measurement of A005 inclinometer. The guide pipes of A005 were estimated to have been moving horizontally with the movement of the land masses, because clear deformation was observed in A006 and A003 which are located at upper and lower parts of A005. The borehole depth of A005 is very shallow and the bottom of boreholes seems not to penetrate the assumed slide surface. According to the observation conducted in May 1989, the tendency of guide pipe displacement was in the direction of the lower slope of the landslide area.

4) A006 (borehole depth = 36.0 m)

Constant shearing deformation was interpreted since June, 1988, and the depth of the sliding plane was estimated at 26 m. The guide pipes were moving parallel to horizontal direction in the upper zone of the sliding surface. Constant deformation was measured in the period between June and September 1988, which coincides with the less rainfall season. It was not possible to carry out any observation deeper than 24.5 m in May 1989 but the guide pipes were moving to horizontal direction in the section above 24.5 m.

5) A007 (borehole depth = 52.0 m)

Larger deformation was interpreted in the period between July and September 1988. The deformation was identified at two parts about 12 m and from 22 m to 27 m in depth. Shearing deformation was observed at about 12 m, but the guide pipes seem to stand straight in the section above 12 m. On the other hand, the section between 22 m and 27 m was indicated to deform to the lower slope side with a total deformation amount of 47 mm. The sliding surface was estimated at 27 m in depth. The probe of the inclinometer could not be inserted deeper than 27 m in May 1989. The deformation was not observed with remarkable movement since September 1988.

In this study guide pipes for inclinometer measurement were installed into eight boreholes, BV-V2, BV-V4, BV-X1, BV-X3, BV-X4, BV-Z1, BV-Z2 and BV-Z3 at the beginning of the study period. Measurement of the inclinometer was made once a month as a general rule from the late August to the early October 1989 with the progress of the borehole drilling. Monthly and total displacements at certain depths in each borehole are summarized in Table I.2.6-3. Measurement results are shown in Fig.I.2.6-8 to Fig.I.2.6-15.

In the borehole X-1 located in the upper part of the soil removal area, sliding surface due to minor landslide was estimated at the depths about 8 m and about 2 m below the present ground surface. Measurement of the borehole BV-X1 had been stopped in the middle February 1990 because the prove of the inclinometer was not able to insert in the section deeper than 2 m.

Along X-line displacements which seem to indicate the slide surface are estimated at the depth between 21 m and 22 m in the borehole, BV-X3, and between 6.5 m and 8 m in the borehole, BV-X4. Along Z-line displacements are observed at the depth between 9.5 m and 11 m in the borehole, BV-Z2, and between 12.5 m and 13.5 m in the borehole BV-Z3. In the remaining three (3) boreholes, BV-V2, BV-V3 and BV-Z1, no remarkable displacement was observed since September 1989, although slight deformation was seen on the measurement records from these borehole.

(3) Tiltmeter

Seven units of tiltmeter were installed by the previous JICA expert team to confirm the sliding movement of the landslide area. The location of the tiltmeter is shown in Fig.I.2.1. The observation by tiltmeter has continued at the intervals of one week.

Inclination movements of the ground surface are measured by tiltmeters. The results of measurement are shown in Fig.I.2.6-16 to Fig.I.2.6-18. Monthly values of inclination, average rates of inclination on the daily base, and inclination directions in the monitoring period since August 1989, are shown in Table I.2.6-4 and Fig.I.2.6-19.

Inclination records of the ground surface show intermittent movements in general. Correlation between inclination movements and rainfalls in the rainy season is seen in the attached graphs showing the relation between inclination angle and rainfall.

Among seven tiltmeters, the records from T-4 and T-6 indicate accumulation tendency of the ground inclination during the observation period. Therefore, these two tiltmeters seem to be affected by the landslide movement. Remaining tiltmeters, T-1, T-2, T-3, T-5 and T-7, seem to be located outside of the landslide area because they show intermittent movements only in the limited short period.

(4) Extensometer

Measurements of four (4) extensometers, E-1 to E-4, which were installed in the preliminary investigation study stage, and twelve (12) extensometers, which were installed in this study period had been commenced in October 1988 and June 1989 respectively. Measurement of the extensometer, E-15, was stopped in March 1990 and the extensometer was shifted to the new location of E-17 at the western part of the extensometer, E-3, in order to confirm whether the minor landslide occurred near E-3 is going to extend to the western part or not.

Monthly and total displacements of extensometers are shown in Table I.2.6-5. Graphs of measuring results from seven extensometers including E-17, which are located at the crown portion of the landslide area, are shown in Fig.I.2.6-20. Measurement results from ten units of extensometers, which are located in the central part and at the toe portion of the landslide area, are graphed as shown in Fig.I.2.6-21.

Four units of extensometers, E-1 to E-4, were installed by the previous JICA expert team at the crown part of the landslide. The recording of movement has continued since October 1988. The observation results are shown in the correlation graphs between the extensometer record and the rainfall record. The graphs are shown in Fig.I.2.6-20 to Fig.I.2.6-21.

All extensometers have indicated active movements since the middle of December 1988 because of continuous rainfall from the end of late November to the beginning of December. Relatively intensive movement for more than 10 mm/day was recorded by the extensometer, E-4, installed in the eastern part of the area during the attacking of Cyclone Firinga. This cyclone brought rainfall of 275 mm/day on 23rd

January 1989. The movement decreased to less than 5 mm/day in the following days and an emergency case was avoided fortunately.

When Firinga attack Mauritius the extensometer E-1 installed in the western part of the landslide area recorded sudden compressive deformation. An increasing of the landslide area was feared accordingly, and tension cracks were observed at the top of E-1 location by visual observation. Therefore, one additional extensometer E-5 was installed crossing the observed tension cracks afterward. On the basis of the extensometer records, the movement of open cracks was confirmed to decrease after the visit of Firinga. However, the movement was increased again as a result of continuous rainfall in the period between the late March and the early April 1989.

During the rainy season from 1988 to 1989, maximum accumulated movements for 256 mm and 246 mm were recorded by the extensometers E-3 and E-4 which were installed in the central crown portion of the landslide area. On the other hand, the minimum movement of 35 mm was recorded by E-1 installed in the western part. The medium movement of 130 mm was recorded by E-2 in the eastern part of E-1. According to the extensometer records, the displacement of tension cracks and upheaval zones was very little or disappeared in this dry season. This seems to be directly related to the small rainfall in the dry season.

During the rainy season from 1988 to 1989, the extensometers, E-1, E-2, E-3 and E-4, recorded remarkable landslide movements with the displacement amount more than 250 mm in the total maximum value. On the contrary, remarkable displacements were not observed due to small rainfall in the rainy season from 1989 to 1990 and the completion of soil removal works except for the displacement records from the extensometer, E-3. The displacement recorded by the extensometer, E-3, was believed to be caused by a minor landslide triggered by the soil removal works and displacement amount of E-3 was 120 mm in total or 1.25 mm/day on an average in the period from the late October 1989 to the late January 1990.

For mitigation of this displacement counterweight embankment was planned and executed. After completion of the first counterweight on the berm at El.46 m in the soil removal area as an urgent protect measure for the minor landslide in late January 1990, displacement of E-3 was minimized to be 0.27 mm/day for about following one month. However, remarkable displacement reaching to 200 mm/month or 6.5 mm/day on an average was observed again in March 1990. The second counterweight was executed on the berm at El.49 m and El.52 m for mitigation of the small landslide. The displacement has been minimized to be less than 0.5 mm/day after completion of the second emergency counterweight at the early June 1990. No remarkable displacement is recorded by other extensometers since the dry season in 1989, although there are some minor displacements which seem to be affected by small rainfalls during the last rainy season.

(5) Groundwater level measurement

Groundwater levels have been measured in ten (10) boreholes, BV-V1, BV-V3, BV-W1, BV-W2, BV-W3, BV-X2, BV-Y1, BV-Y2, BV-Y3 and BV-Z4, which were drilled in this project, and in the boreholes, A003, A005, and A007, which were drilled in the previous study period. The fluctuation of groundwater levels had been reflecting the rainfalls in most of the boreholes. The measurement results of groundwater levels have been recorded as fluctuation of groundwater levels in the period after the completion of the borehole drilling since June 1989. The fluctuation of groundwater levels are shown in Fig.I.2.6-22.

Relatively high, immediately after the completion of drilling works in general, groundwater levels were lowered to the depths deeper than the bottom of boreholes since July 1989. In the rainy season from 1989 to 1990 groundwater levels were expected to rise to the level higher than the levels observed at the beginning of this study. Recovery of the groundwater levels, however, had not been observed until the end of field investigation of this study. This is believed to depend on the small amount of rainfall in the rainy season from 1989 to 1990. Groundwater level was not observed also in the drainage well, DW-1, constructed as an experimental investigation during the rainy season from 1989 to 1990.

Taking these indistinct conditions of groundwater levels into consideration, one additional observation borehole was drilled from the bottom of the drainage well, DW-1, Cleaning of the existing observation boreholes was carried out by flushing out slime by use of an air compressor in this study period. Exact groundwater levels in the holes, DW-1, W-3, X-2, Y-3 and Z-4 had been revealed by these works since the beginning of June 1990.

In the western part of the landslide area, significant drawdown of the groundwater levels had been measured in this study period. The groundwater levels in this part had been lowered to the levels deeper than the sliding surface as they had been observed in the boreholes, DW-1 and BV-W3. On the contrary, remarkable drawdown of the groundwater levels had not been observed in the eastern part as they had been observed in the boreholes, BV-X2 and BV-Y3. Based on the variations of groundwater levels mentioned above, it seems to be able to conclude that drainage of groundwater by drainage wells seems to be more effective in the eastern part than the western part of the landslide area.

(6) Resistivity logging of groundwater in borehole

Resistivity logging of groundwater was made in boreholes for confirming possible layers of groundwater flows by measuring resistivities of the groundwater by dissolving electrolytic materials in groundwater. From the changes of electric resistivities, groundwater flow layers and their depths can be interpreted.

In this study electric sounding was conducted in the boreholes, BV-W2, BV-W3, BV-W4 and BV-Z4, with the observation depth at every 0.25 m or 0.50 m interval below the groundwater level. Intervals of observation period were 5, 10, 15, 30, 60, 120 and 180 minutes. The measurement results are shown in Fig.I.2.6-23 and fig.I.2.6-24. Among drilled boreholes in which electric sounding was carried out, groundwater flow layers was confirmed at the depth of 19.5 m, 19.25 m and 10.75 m in BV-W2, BV-W3 and BV-W4 respectively. Minor variations of resistivities were measured at 17.25 m in BV-W3 and near groundwater level in BV-Z4. However, these variations were so small that the groundwater flow layers were not distinguished.

The equipment for resistivity logging of groundwater in boreholes has as follows:

Digital GWC tester : Model KCM-200B

I.2.7 Landslide Mechanism

The landslide that happened in La Butte area was a colluvial landslide according to the general classification of the landslide types as shown below.

1) Bedrock landslide; composed of fresh rocks (in stage of youth)
2) Weathered rock landslide; composed of weathered rocks or debris including big boulder (in stage of adolescence)
3) Colluvial landslide; composed of earth including gravel (in stage of maturity)
4) Clayey soil landslide; composed of clay or clayey soil including gravel (in stage of old age)

Very thick scree deposits composed of clayey soil with basaltic boulders underlie the landslide area. According to the visual observation of scree deposits at the soil removal area and in the drainage well, there are frequently developed slickensides in clayey materials composing the scree deposits. Development of slickensides in the clayey materials seems to imply that strong stress had been acting on the materials for a long period preceding to happening of the landslide movement.

One additional vertical borehole was drilled from the bottom of the drainage well for 20 m in depth after completion of the intermediate well and drainage boreholes. This borehole was drilled for confirmation of the depth of groundwater levels and bedrock surface at and around the drainage well because groundwater was not collected from groundwater collection boreholes drilled for 1000 m in total length from the drainage well to the mountain side.

In this borehole the groundwater level was confirmed at about -12.0 m (El.15 m) below the bottom of the drainage well. The materials encountered during the drilling of this borehole are scree deposits composed of gravel with clayey soil until about 9.4 m below the bottom of the drainage well. Weathered basalts were confirmed in the section between 9.4 m and 15.0 m and fresh basalts were confirmed below 15.0 m. The general condition of slide surfaces, bedrock surfaces, and groundwater levels in the landslide area is summarized hereunder. Distribution of slide surfaces, bedrock surfaces in the landslide area are shown in Fig.I.2.7-1.

(1) Slide surface

The geological profiles along V-line, W-line, X-line, Y-line, Z-line, and the line through the drainage well and the intermediate well is attached to this report are shown in Fig.I.2.4-4 to Fig.I.2.4-9.

Depth to slide surfaces is deeper at the main part of the landslide area which coincides with the central part to the eastern part of the landslide. The length of the landslide in this part is longer than the remaining part. The depth to slide surfaces is judged from drilled core samples and the records obtained from the borehole inclinometer. The features of slide surfaces shown in the geological profiles are summarized as follows:

V-line (refer to Fig.I.2.4-4) : The slide surface forms smoothly curved line, declining to the lower part, that is, to the northern part of the landslide area. The maximum depth to the slide surface from the ground surface is about 20 m at the part near the borehole, BV-V2, in the central to upper parts of the landslide, that is, to the southern part of the landslide. The depth to slide surface is confirmed by measurement of borehole inclinometer in the boreholes, BV-V2 and BV-V4. The slide surface is assumed to lie on the bedrock surface near the borehole, BV-V3.

Line through the drainage well and the intermediate well (refer to Fig.I.2.4-5): The slide surface declines steeply from the upper part to the central part of the landslide, and it keeps almost horizontal to the lower part of the landslide. The slide surface is assumed to rise up to the ground surface at the tension zone of the landslide. The maximum depth to the slide surface is about 25 m near the drainage well. The slide surface is assumed to shear through scree deposits at about 25 m in depth from the central to the lower parts of the landslide.

W-line (refer to Fig.I.2.4-6): There is a concaved shape of slide surface at the central part of the landslide along this line. The slide surface declines from the open-cracks at the upper part of the landslide to the borehole, BV-W3. From the borehole, BV-W3, the slide surface inclines gently to the lower part of the landslide until the tension zone. The maximum depth to the slide surface is assumed to be about 25 m at around the borehole, BV-W3. The slide surface is assumed to shear through scree deposits for the full extent of this line.

X-line (refer to Fig.I.2.4-7) : The slide surface declines gently and smoothly from the open-cracks at the upper part to the lower part of the landslide. The maximum depth to the slide surface is about 20 m at the central part. The slide surface seems to shear dark grayish clayey layers at the lower part of the landslide at and around the borehole, BV-X4.

Y-line (refer to Fig.I.2.4-8): The slide surface declines from the open-cracks at the upper part to the central part of the landslide. The slide surface is almost horizontal in the central to the lower parts. The slide surface turns upward slightly to the ground surface at the compression zone. The slide surface is estimated to lie on bedrock surface at the central part of the landslide. The maximum depth to the slide surface is about 17 m near the borehole, BV-Y2.

Z-line (refer to Fig.I.2.4-9): Gently curved line is estimated as the slide surface along this line. The depth to shearing zones are confirmed by measurement of the borehole inclinometer. The slide surface is assumed to shear through loose scree deposits. The maximum depth to the slide surface is about 15 m.

The slide surface is assumed to shear through loose scree deposits in the zone above bedrock surfaces in general. However, the slide surfaces seem to close to bedrock surfaces partly especially at the parts where bedrock surface has convex shapes. The depth to slide surfaces is deeper at the central to the western parts than the eastern part of the landslide area.

(2) Bedrock surface

The depth to bedrock surfaces is confirmed from recovered drilling core samples in the Stage-1 study. Borehole drilled from the bottom of the drainage well adds supplemental information on the depth to bedrock surface. General features of the bedrock surfaces are summarized below along each geological profile.

V-line : There are smoothly curved two steps of the bedrock surface. The upper step declines from the part near open cracks at the upper part of the landslide to the borehole, BV-V3. The elevation of the bedrock surface is about 20 m in the boreholes, BV-V3. The lower step declines again from the borehole, BV-V3, to the borehole, BV-V4 at the lower part of the landslide. The elevation of bedrock surface is about 0 m at this part. The bedrock surfaces are overlain by dark grayish clayey layers at the lower part of the landslide. The maximum depth to the bedrock surface is about 25 m near the borehole, BV-V1, drilled at the upper part of the landslide.

Line through the drainage well and the intermediate well : The bedrock surface along this line is smooth from the upper part to the lower part of the landslide. The depth to bedrock surface is 25 m to 30 m in general except for the lower part. The depth at the lower part is reduced to be about 10 m because the ground surface declines to the lower part of the landslide.

W-line : There are smoothly curved two steps: one is from the open-cracks to the borehole, BV-W2: the other one is from the borehole, BV-W2, to the lower part of the landslide. A concaved shape of bedrock surface is assumed in the central part of the landslide. The maximum depth to the bedrock surface is assumed to be about 40 m in the central part of the landslide. Clayey layer overlies the bedrock surfaces for about 5 m in the lower part of the landslide.

X-line : A smoothly curved line of bedrock surface is assumed along this line. The bedrock surface shows a slightly concaved shape in the central part of the landslide. The maximum depth to the bedrock surface is about 30 m in the central part of the landslide. The bedrock surfaces are overlain by clayey layer for about 5 m at the lower part of landslide.

Y-line : The bedrock surface declines from the open-cracks at the upper part of the landslide to the central part, forming a smooth curve. The bedrock surface declines again slightly from the central part to the lower part (from about El.10 m to about El.0 m). The maximum depth to the bedrock surface is about 12 m in the central part.

Z-line : The bedrock surface is assumed to curve smoothly from the upper part to the lower part, being accompanied by a slightly concaved shape in the central part. Generally the depth to bedrock surfaces is shallow along this line. The maximum depth to the bedrock surface is about 20 m in the central part. However, the depth is decreased to be less than 10 m at upper part and at the lower part. Thin clayey layer overlies the bedrock surface at the lower part.

The shape of bedrock surfaces is assumed to be smoothly curved lines in general. Several concaved shapes are assumed on bedrock surfaces along several lines. The minimum elevation of bedrock surface is almost El.0 m at the lower part of the landslide. Clayey layers are confirmed to overlie the bedrock surfaces at the lower part.

(3) Groundwater level

In the Stage-1 study 18 boreholes were drilled in the landslide area for observation of groundwater levels and measurement of borehole inclinometer. However, groundwater levels had been not able to measure in several boreholes after July 1989 because of remarkable drawdown of the groundwater levels in those boreholes. Rainfalls in the last rainy season from November 1989 to April 1990 were relatively small, and clear recovery of the groundwater levels was not confirmed eventually.

Groundwater levels in landslide areas are one of the important factor for stability of landslide movement because higher groundwater level is believed to accelerate landslide movement in general. In this study period cleaning of boreholes was tried several times in the boreholes in which slims were sedimented during drawdown of groundwater level. Through this treatment groundwater levels has been able to measure in the boreholes, BV-W2, BV-W3, and BV-X2.

In addition to these boreholes groundwater measurement, is able to conduct in the additional vertical borehole from the bottom of the drainage well. Maximum and minimum groundwater levels are shown in the geological profiles, Fig.I.2.4-4 to Fig.I.2.4-9. The higher groundwater levels indicate the groundwater levels measured in June

1989, and the lower groundwater levels indicates the groundwater levels measured in June 1990. The features on the groundwater levels are summarized hereunder for each geological profile. Measurement results of groundwater levels are shown in Fig.I.2.6-22.

V-line : Groundwater level lines are assumed to decline from the upper to the lower parts of the landslide smoothly. The groundwater level was measured in the boreholes, BV-V1, BV-V2, BV-V3 and BV-V4, in June 1989. The groundwater levels was higher in June 1989 after heavy rainfalls in the rainy season from November 1988 to April 1989. The high groundwater level ranges from El.10 m to El.30 m in the section between BV-V4 and BV-V1. On the other hand, the low groundwater level ranges from 25 m to 10 m, implying no large difference between the high and the low groundwater levels in the lower part of the landslide.

Line through the drainage well and the intermediate well : Both high and low groundwater levels are assumed to decline very gently from the upper part to the lower part of the landslide. The groundwater level lines are almost horizontal from the central part to the lower part. Though the high groundwater level ranges from El.10 m to El.30 m, the low groundwater level ranges from El.10 m to El.25 m. Difference of about 5 m is assumed between the high groundwater level in the borehole, BV-V1 and the low groundwater level in the vertical borehole drilled from the bottom of the drainage well.

W-line : Both high and low groundwater levels seem to remain mostly in scree deposits because the depth to bedrock surfaces is very large along this line. The groundwater levels seem not to drawdown into the bedrock zones accordingly. The high groundwater level declines gently from the upper part to the central part of the landslide. The groundwater level declines again from the central part to the lower part. The groundwater level line is assumed to form a convex shape at the lower part of the landslide from the measurements results.

Low groundwater level line shows a concaved shape in the central part of the landslide. The minimum groundwater level is about 2 m in the borehole, BV-W3. Judging from the forms of bedrock surface and groundwater levels surrounding this part, groundwater seems to drain to the northeastern direction, not directly to the north direction along W-line. A large gap between the high and the low groundwater levels is assumed at and around the borehole, BV-W3.

After flushing out slims from the boreholes, BV-W2 and BV-W3, fluctuation of the groundwater levels corresponding to rainfalls are able to interpret on the rainfall records in these boreholes. The fluctuation of the groundwater levels in these boreholes are shown in Fig.I.2.6-22. The maximum groundwater level is at about 40 m and the minimum is at about 10 m along this line on the lower groundwater level.

X-line : Except for the upper part of the landslide the groundwater level seems to remain in the scree deposits. The groundwater level is assumed to be below bedrock surfaces at and around the borehole, BV-X1 at the upper part of the landslide. The groundwater level lines decline almost parallel to the inclination of the ground surface. There is no large difference between the high and the low groundwater levels. The maximum difference between the high and low groundwater levels is estimated to be less than 5 m. The maximum groundwater level is at about El.45m and the minimum is less than El.5 m along this line on the lower groundwater level.

Y-line : Groundwater level lines show waving forms along this line, implying that the groundwater does not drain straight to the northern direction along this line. The groundwater levels seem to remain in bedrock zone in the central and the upper parts of the landslide. The groundwater level lines seem to rise up from the central part to the lower part and decrease to the lower outside of the landslide area.

The maximum groundwater level is about El.25 m and the minimum groundwater level is El.5 m along this line on the lower groundwater level. The difference between the high and the low groundwater levels is assumed to be 5 m to 10 m in general. The immediate response of groundwater levels to rainfalls is able to interpret on the fluctuation of groundwater level in the borehole, BV-Y1.

Z-line : Gently declined high and low groundwater level lines are assumed almost in parallel with small height difference less than 5 m. The groundwater levels are assumed; in the bedrock zones in the upper part; in scree deposits in the central part; and on the bedrock surfaces in the lower part. The maximum groundwater level is at about El.25 m but the minimum groundwater level is at about El.5 m along this line on the lower groundwater level.

(4) Landslide movement

Since there was no heavy rainfall in the surrounding area of the landslide during the last rainy season, no significant landslide movements on the whole landslide area were observed on the monitoring records from the extensometers, the borehole inclinometer and the tiltmeters. The present landslide condition is considered to be in a tentatively calm period after the occurrence of an initial remarkable landslide movements in 1987 and the occurrence of intensive movements in the rainy season between November 1988 and April 1989. Landslide movements are interpreted from the monitoring records in this study period are summarized below.

Extensometer: Sixteen (16) units of extensometers had been installed in the landslide prone area in total. The locations and displacement records of the extensometers are shown in Fig.I.2.1 and Fig.I.2.6-20 to Fig.I.2.6-21. Among these sixteen units of extensometers tension movements are recorded by two units of extensometers, E-3 and E-17.

The remarkable displacement recorded by E-3 was caused by the small landslide triggered by the soil removal works at the southeastern part of the soil removal work site. The maximum displacement was reached to be about 20 mm/day or more in March 1990. This small landslide has a size of about 60 m in length and about 40 m in width. The movement of this landslide is stabilized by emergency counter weight embankments so far. Determent works using steel piles will be required as a long term protection measure because the movement of this small landslide might be accelerated again by heavy rainfalls in the future.

The extensometer, E-17, was transferred to the western part of the extensometer, E-3, from the lower central margin of the landslide area for the purpose of confirming the possibility of the landslide to extend to the western direction. There were some tension movements on the extensometer records immediately after transferring to the new location. However, the records without displacement have been obtained since the beginning of May 1990.

Small tension movements were tentatively recorded on the records from the extensometers, E-8 and E-9, in the period from the end of April to the beginning of May 1990. However, these movements were diminished after the period and no movement has been observed so far.

Slight compressive movements were observed on the records of the extensometers, E-1, E-2, E-4, E-5 and E-13 in the period from February to May 1990. These movements have been ceased after the middle of May 1990. There is no any remarkable displacements on the records from the remaining extensometers installed in the landslide area.

Borehole inclinometer: Shearing displacements are seen on the records obtained from the measurement of the borehole inclinometer. The measurement had been impossible in the borehole, BV-X1, because intensive displacement caused shearing on the guide pipe of the borehole inclinometer in the previous study period. The measurement has been continued in the remaining seven boreholes, BV-V2, BV-V4, BV-X3, BV-X4, BV-Z1, BV-Z2 and BV-Z3. Among the records obtained from these boreholes relatively clear displacement has been interpreted in the boreholes, BV-X3, BV-X4, BV-Z2 and BV-Z3 from February 1990.

These boreholes were drilled in the eastern part of the landslide prone area. The obtained records seem to imply shearing displacement has been slightly continued in the eastern part of the landslide, although clear movement is not observed visually in this area. The depths of shearing planes are estimated at about 21.5 m, 8.0 m, 11.0 m, 13.5 m in the boreholes, BV-X3, BV-X4, BV-Z2 and BV-Z4.

On the other hand, any displacement has not been observed on the records from the borehole, BV-Z1. From this fact the borehole, BV-Z1, is considered to drill at the outside of the landslide area. There seems no clear development of shearing displacement in the boreholes, BV-V2 and BV-V4.

Tiltmeter: Accumulated tendency is observed on the measurement results from the tiltmeters, T-4 and T-6. The largest tilting amount has reached to be about 2200 sec. in N-S direction on the record obtained from T-6, which is installed on the northeastern margin of the landslide, during the observation period of about 11 months from August 1989 to June 1990. The records obtained from the tiltmeters, T-4, in the eastern part of the landslide have reached to the total amount to be about 1800 sec. in N-S direction in the same period. The tilting amount of Tiltmeter, T-5, had reached to be about 1200 sec. in E-W direction.

The records of tiltmeter, T-1, which situates in the western part of the landslide area, indicates the tilting amount of 800 sec. in N-S direction in the observation period for about 11 months. The remaining tiltmeters, T-2 and T-7, have no accumulation tendency in the tilting movement. From these records the landslide movement seems to have been continued rather in the eastern part than western part of the landslide area.

According to the obtained monitoring records, the landslide movement seems to be possible to conclude that slight movements have been continued in the eastern part of the landslide. However, the landslide movement has been in a stable condition on the whole in this study period.

Visual observation of the landslide area especially on the open cracks happened on the upper margin of the landslide or cracks occurred on the drainage channel installed in the soil removal area was not able to distinguish any remarkable landslide movement in this period. Leveling survey works in the soil removal area and in the surrounding area had been started additionally in this period, and survey results for only two times have been obtained in this period. However, it is difficult to discuss the relation between the survey results and the landslide movements from the obtained insufficient survey results so far.

The data collection from the monitoring equipment such as the rainfall gauge, the extensometer, the tiltmeter, and the borehole inclinometer and the measurement of groundwater levels, and leveling survey are strongly recommended to continue after the completion of the field investigation by JICA study team for the analysis and prediction of the landslide movements.

I.2.8 Installation of Surface Water Drains

One of the important factors for accelerating landslide movement is assumed to be increasing of groundwater level in the landslide area. Shallow groundwater seem to be supplied from infiltration of rainfall in general. The reason why landslides occurred after continuous heavy rainfalls or cyclones is that the landslide movement was accelerated by increasing of pore pressure among the materials in the landslide area by saturation of groundwater.

When a landslide is occurred, tension cracks take place at the crown parts of landslide generally, and rain water is apt to infiltrate into the ground. On the other hand, sliding surfaces are impervious generally and the zones above the sliding surfaces is considered to act as aquifers. This assumed mechanism seems to accelerate the sliding movement after heavy rainfall. Therefore, the measures for preventing rain water from infiltrating into the ground are important at the initial stage of a landslide occurrence.

From this technical viewpoint drain channels were laid out as shown in Fig.I.2.8. The drainage channels are aligned parallel to the open cracks which were observed at the crown part of the landslide. The total length of the drainage channel is 400 m. Four pits and three ground sills were provided on the route of drainage channel, in order to reduce the energy of running water and prevent erosion. The surface of drainage channels was protected by wire meshes with gunite-shooting.

Open cracks were initially planned to be covered by vinyl sheets in order to prevent rain water infiltration. However, the open cracks were sealed by clayey soil under compaction because vinyl sheets seem to be damaged easily by fire of bush cleaning.

I.3 PREPARATION FOR EXPERIMENTAL INVESTIGATION

For mitigating the landslide movement, the most effective measures were considered to include construction of a drainage well, installation of drainage channels along open cracks at the crown part of the landslide, and soil removal from the upper slope of the landslide. The effectiveness of the drainage well and soil removal works conducted on this study will be judged from the result of stability analysis of the landslide area. The result of stability analysis of the landslide area and the plan for urgent protection measures are described in the followings.

I.3.1 Stability analysis for experimental investigation

The stability analysis was made for the assumption of strength of the materials along the sliding surfaces at first. The strength of the materials is indicated by internal friction angle and cohesion. For the designing of the protection measures, the assumed values for the material strength were adopted to the stability analysis for confirming the appropriateness of the protection measures afterward.

Five lines, Line-V, Line-W, Line-X, Line-Y, and Line-Z, were selected for stability analysis. Among these lines Line-W is the main line of the western block of the landslide and Line-Y is the main line of the eastern block. In this study the slice method, which is the most popular for landslide stability analysis, was adopted, with unit weight of 1.8 t/m^3 taken as the ground mass of the landslide area. The equation of the slice method is shown below.

$$F.s = (\mathrm{Tan}\ a \times (N-p) + c \times L) \ / \ T$$

where, F.s : safety factor

N : force along normal line of each slice (t/m)

T : force along tangential line of each slice (t/m)

p : pore pressure for each slice (t/m)

L : length of slide surface (m)

a : internal friction angle for materials along sliding surface (degree)

c : cohesion of materials along sliding surface (t/m^2)

According to the investigation results of this study, profiles for the stability analysis were prepared as shown in Fig.I.3.1-1 to Fig.I.3.1-5. In the stability analysis the factors except the internal friction angle (a) and the cohesion (c) of the materials along the sliding planes are able to be calculated by using topographic profiles with the assumption of the groundwater levels and the unit weight of 1.8 t/m^3 for the materials.

On the basis of the data obtained after the rainy season 1989 and in dry season in this study, the groundwater levels indicated in the profiles were assumed as the high water and the low water levels. Under this condition the safety factor is assumed as 1.0 (F.s=1.0), and the values for each factor were calculated as follows:

VALUES FOR STABILITY ANALYSIS

Line	T	N	L	P	area of section(m^2)
V	975.12	5454.39	279.55	1233.20	3118.56
W	1688.63	9980.60	334.57	2050.74	5713.92
X	1270.23	7411.98	320.91	1597.67	4308.09
Y	683.2	4213.5	206.2	1371.1	1961.5
Z	494.2	2979.9	188.1	1042.7	919.56

The cohesion (c) of clayey materials along the slide surface is estimated as increasing the rate of 1.0 t/m^2 for every 10 m of the thickness of the sliding ground mass in general. More than 2.0 t/m^2 was estimated in the upper sliding area where the depth to the slide surface is more than 20 m. On the other hand, cohesion (c) of the clayey soil along the slide surface of this area was confirmed to be c = 0.0 t/m^2 by the previous study. Therefore, the cohesion of c = 1.0 t/m^2 was chosen as the average of the above two values for this stability analysis. The internal friction angle was interpreted from the correlation graph between the cohesion and the internal friction angle, which is based on the calculated values mentioned above, using the equation for the slice method. The correlation graph is shown in Fig.I.3.1-6. The interpreted internal friction angles are summarized below for each line.

PHYSICAL PROPERTY OF THE MATERIAL ALONG SLIDING SURFACE

Line	Cohesion (t/m2)	Internal friction angle (degree)	Tan(a)
V	1.0	9.4	0.165
W	1.0	9.7	0.171
X	1.0	9.26	0.163
Y	1.0	9.53	0.168
Z	1.0	8.98	0.158

I.3.2 Establishment of urgent protection measures

Urgent protection measures against landslides include mitigating the sliding movement, minimizing the possible damages and providing the necessary minimum countermeasures. The protection measures for landslides are classified into prevention works and determent works, and they are subdivided as below.

LANDSLIDE CONTROL WORKS

Prevention Works:
- Surface drainage works
  - Subsurface drainage works
    - Shallow groundwater drainage works (Interceptor drains, Horizontal drainage works, Interceptor trench)
    - Deep groundwater drainage works (Horizontal drainage works, Drainage well works, Drainage tunnel works)
  - Soil removal works
  - Counterweight fills

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

The prevention works are expected to mitigate sliding movements by changing natural conditions with draining groundwater, reducing weight of ground mass, preventing infiltration of surface water and so on. On the contrarily, the determent works are expected to prevent the movement mechanically with providing piles and anchors. Taking the size of the concerned landslide area, suitability of work, and economic view points of the works into consideration, protection measures such as laying out drainage channels, construction of drainage wells, and soil removal works were selected as the urgent protection measures for securing the proposed safety factor more than 1.0. The urgent protection measures for the landslide movement were carried out in the second stage of this study.

(1) Drainage channel works

Drainage channels are expected to prevent the surface water from infiltrating into the ground through open cracks. These drainage channels were provided for about 400 m along open cracks at the crown part of the landslide as shown in Fig.I.2.8.

(2) Drainage well

In general, drainage wells are planned where groundwater is able to be drained concentrically from the predominant aquifers and where the length of horizontal drainage boreholes from the ground surface is expected to be very long because of the topographic condition. In this study the drainage well is planned as an experimental investigation for urgent protection measures.

The location of the drainage well has to be constructed in stable ground masses where the depth to the sliding surface is great and at the place where construction work will be carried out easily. Moreover, the location of the drainage well will be required to be at the point where groundwater collection will be expected to be favorable. The depth of the drainage well has to be shallower than the depth to the sliding surface in an active landslide area because damages caused by sliding movements are able to be avoided.

Drainage of collected groundwater from the drainage well has to be done gravitationally as a general rule, and the drainage borehole to the ground surface will be required consequently. When the length of the drainage boreholes is too long, intermediate wells will be required on the way to the ground surface.

Taking the above mentioned condition into consideration, one drainage well, one intermediate well, groundwater collection boreholes for 1000 m (20 holes), and drainage boreholes for 120 m (2 holes) were planned as urgent protection measures for the experimental investigation.

(3) Soil removal works

Soil removal from the landslide area is made in order to reduce the weight of sliding masses from the upper part of the landslide area and ensure safety of the landslide. Soil removal will be planned in the cases of landslide where ground mass is thick at the upper part, the possibility of occurring new landslides will be reduced by soil removal, and the soil removal work seems to be performed easily. The soil removal is planned in combination with other protection measures.

Soil removal for about 48000 m^3 from the area around the abandoned reservoir for water supply was planned for one of the experimental investigation in the second stage study. The area of soil removal works is shown in Fig.I.2.1.

I.3.3 Design concept for drainage well and intermediate well

For the confirmation of the effectiveness of the construction of a drainage well in the landslide area, stability analysis was made along the Line-W which is the main line of the western part of the landslide. The strength of the materials along the sliding surfaces are calculated in Supporting Report 2, being obtained to be 9.7 deg. for the internal friction angle and 1.0 t/m^3 for the cohesion. The groundwater level at the occurrence of landslide movement was assumed as indicated in the profile for the stability along the Line-W. The unit weight of 1.8 t/m^3 was applied for the soil mass of the landslide.

For the calculation of safety factor (F.s) of the landslide area, the equation of slice method was adopted.

$$F.s = (\text{Tan } a \ (N-p) + c \times L) \ / \ T$$

where, F.s : safety factor
- N : force along normal line at each slice (t/m)
- T : force along tangential line at each slice (t/m)
- p : pore pressure for each slice (t/m)
- L : length of slice surface (m)
- a : internal friction angle for materials along sliding surface (deg.)
- c : cohesion of materials along sliding plane (t/m^2)

The factors except for the internal friction angle and cohesion of the materials along the sliding surface were calculated by accumulating each divided slice from the profile along the Line-W. The values of the factors of each slice are summarized below.

VALUE OF EACH SLICE

Slice No.	T	N	L	p	Area of slice(m^2)
1 - 9	-32.29	546.89	51.24	146.26	304.62
9 - 15	0.00	3007.32	97.50	774.75	1670.74
15 - 19	129.79	1240.93	27.90	262.89	694.39
19 - 26	742.11	3533.83	77.96	664.76	2006.06

26 - 33	295.09	756.01	20.23	155.39	453.07
33 - 38	553.93	895.62	59.74	46.69	585.04
Total	1688.63	9980.60	334.57	2050.74	5713.92

The safety factors before and after the construction of the drainage well are calculated as follows by using the above mentioned equation.

1) before construction of the drainage well

F.s = (0.171 x (9980.60 - 2050.74) + 1.0 x 334.57) / 1688.63
=1.00

2) after construction of the drainage well

F.s = (0.171 x (9980.60 - 1510.56) + 1.0 x 334.57) / 1688.63
= 1.061

According to the calculation results, the improvement of the safety factor after the construction of the drainage well is expected for more than 6 %.

On the basis of investigation results in the first stage of this study, the landslide movement was confirmed to be accelerated in the rainy season as shown in Fig.I.2.6-20. There is no room for doubt that the landslide was activated by increasing groundwater level. Drainage of groundwater from the landslide area is concluded to be effective for securing the stability of the landslide area accordingly. In this study construction of drainage well and drilling of groundwater collection boreholes and drainage boreholes were planned as the experimental investigation for the main works of the second stage.

On the basis of the stability analysis confirming sufficient effects for the stability of landslide, the preparation was made for the basic design of the drainage well and the intermediate well. The design works had made on the structure of wells with assuring sufficient strength for the natural condition surrounding the wells. In this study, examination of the local contractor was also carried out on the ability for experimental investigation and ability for supplying required materials for the experimental investigation. The drainage well is shown in Fig.I.3.3

I.3.4 Dimension of the wells

Regarding the structure of the drainage well and the intermediate well, the wells of 3.5 m in diameter with using steel liner plates were selected with reference to examples performed in Japan at sites with almost the same natural conditions with regard to topography and geology. The depth of the drainage well was designed to be 20 m, judging from the core drilling results which were obtained in the first stage investigation. The detail structure of the drainage well and the intermediate well are mentioned in Supporting Report (II).

The location of the drainage well was selected near the abandoned reservoir for the water supply to Port Louis city after paying attention to the following points. The location of main structure for the experimental investigation is shown in Fig.I.3.4-1.

a. Depth to sliding surface is great, and groundwater seems to be concentrated into the surrounding area of the proposed well location.

b. Concave topography in which surface water is easily concentrated, and the depth of drainage well will be minimized.

c. Landslide movement is relatively small and field activity of the site will be safe.

d. Ground is stable and transportation of materials is easy.

I.3.5 Outline of specification for the wells

The main specifications of the wells are as follows:

a. For the construction of drainage well, perforated (with strainer), steel liner plates shall be used for protection against collapses of the well wall.

b. Liner plates shall be required to be corrosion-proof types, which are galvanized plates. Stiffeners have to be galvanized also.

c. The first section of 50 cm liner plate shall be assembled above the ground surface for prevention of mud flowing into wells.

d. Steel covers shall be provided on the top of the wells in order to prevent materials from falling into the wells.

e. Ladder shall be provided in the wells for the future inspection.

f. Groundwater collection boreholes of 66 mm in diameter shall be drilled for 1000 m (20 holes) divided into three steps.

g. PVC pipes of 40 mm in diameter which are perforated shall be inserted into groundwater collection boreholes.

h. Drainage of collected water into the drainage well to the ground surface shall be gravitational. However, dewatering by pumps shall be made until completion of the drainage boreholes, which penetrate the sections between the drainage well and the ground surface through the intermediate well.

i. The outer diameter of drainage boreholes shall be 116 mm. Steel pipes of 101.6 mm in outer diameter shall be inserted after completion of the drilling.

j. The length of drainage borehole will be more than 100 m from the condition of the surface slope inclination at this part. The intermediate well shall be provided on the way to the ground surface at about 50 m from the drainage well. The length between the intermediate well and the ground surface shall be about 70 m also.

k. The diameter of the intermediate well shall be also 3.5 m, and the depth shall be 15 m. The intermediate well shall be protected by steel liner plates also but perforation of the liner plates shall not be required.

Based on the above mentioned specifications, designing of the well structures was made with reference to technical standards, design examples and guidelines, which have been used effectively by Japanese government offices and other organizations.

TABLES

TABLE I .2.2-1(1/2) LOCATION OF INVESTIGATION SITES

SITES	X (E)	Y (N)	Z (m:elevation)
BV-V1	996.025	1003.297	55.40
BV-V2	995.992	1003.333	48.90
BV-V3	995.964	1003.402	36.55
BV-V4	995.974	1003.490	22.71
BV-W1	996.053	1003.280	62.48
BV-W2	996.064	1003.345	43.08
BV-W3	996.077	1003.430	32.11
BV-X1	996.165	1003.303	51.95
BV-X2	996.187	1003.371	37.95
BV-X3	996.206	1003.429	29.01
BV-X4	996.235	1003.523	15.15
BV-Y1	996.304	1003.305	44.43
BV-Y2	996.325	1003.369	29.49
BV-Y3	996.325	1003.435	19.77
BV-Z1	996.384	1003.275	45.73
BV-Z2	996.414	1003.323	29.04
BV-Z3	996.433	1003.373	21.64
BV-Z4	996.474	1003.427	12.30
DW-1	996.024	1003.355	45.50
IW-1	996.026	1003.408	39.00
OUTLET	996.035	1003.469	24.15
E-1	996.004	1003.229	75.00
E-2	996.029	1003.188	92.00
E-3	996.140	1003.256	70.00
E-4	996.232	1003.296	55.50
E-5	996.009	1003.216	81.50
E-6	996.231	1003.308	45.00

TABLE I .2.2-2(2/2) LOCATION OF INVESTIGATION SITES

SITES	X (E)	Y (N)	Z (m:elevation)
E-7	996.239	1003.331	44.50
E-8	996.242	1003.341	41.00
E-9	996.245	1003.350	38.50
E-10	996.250	1003.362	36.00
E-11	996.250	1003.382	32.50
E-12	996.252	1003.393	31.00
E-13	996.304	1003.305	50.50
E-14	995.898	1003.486	17.50
E-15	996.113	1003.590	10.00
E-16	996.055	1003.570	13.00
E-17	996.105	1003.260	66.50
T-1	995.909	1003.238	56.50
T-2	996.090	1003.237	73.50
T-3	996.317	1003.262	59.50
T-4	996.466	1003.307	27.50
T-5	996.542	1003.376	13.50
T-6	996.390	1003.467	12.50
T-7	995.877	1003.447	18.50

* Coordinates are interpreted from topographic map (1:1000) prepared in July 1989.
* Elevations of boreholes and wells are surveyed by leveling.
* Elevations of extensometers and tiltmeters are interpreted from topographic map (1:1000).

TABLE I .2.4-1 INVESTIGATION QUANTITY IN PHASE-1, 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 I .2.4-2 INVESTIGATION QUANTITY IN PHASE-2, FIRST STAGE

PHASE	BOREHOLE No.	DEPTH (m)	PERMEABILITY TEST OPEN-END	IUGEON	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 Ⅰ.2.4-3(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	3.04x10-3
	15.0	15.00	0.00	15.00	4.2	72.7	3.50x10-2
	20.0	20.00	0.12	20.12	4.2	>350.0	>1.26x10-1
	25.0	25.00	0.35	25.35	4.2	>350.0	>9.96x10-2
BV-V2	10.0	1.50	0.20	1.70	4.2	0.8	3.40x10-3
	15.0	2.00	0.10	2.10	4.2	1.2	4.12x10-3
	20.0	16.05	0.20	16.25	4.2	3.1	1.38x10-3
BV-V3	10.0	7.30	0.70	8.00	4.2	5.0	4.51x10-3
BV-V4	10.0	7.80	0.15	7.95	4.2	20.0	1.82x10-2
	15.0	7.80	0.15	7.95	4.2	20.0	1.82x10-2
	20.5	8.95	0.10	9.05	4.2	20.0	1.59x10-2
BV-W1	10.0	8.85	0.10	8.95	4.2	37.4	3.01x10-2
	15.5	15.50	0.25	15.75	4.2	48.0	2.20x10-2
	20.0	20.00	0.10	20.10	4.2	73.0	2.62x10-2
	25.0	25.00	0.21	25.21	4.2	21.1	6.04x10-3
BV-W2	10.0	1.12	0.39	1.51	4.2	0.8	3.82x10-3
	15.0	0.54	0.31	0.85	4.2	1.8	1.53x10-2
	20.0	20.00	0.17	20.17	4.2	0.2	7.15x10-5
	25.0	25.00	0.16	25.16	4.2	16.7	4.79x10-3
BV-W3	10.0	8.19	0.16	8.35	4.2	0.1	8.64x10-5
	20.0	14.17	0.24	14.41	4.2	0.1	5.01x10-5
BV-X1	5.0	5.00	0.60	5.60	4.2	0.7	9.02x10-4
	10.0	6.40	0.60	7.00	4.2	3.9	4.02x10-3
	15.0	12.00	0.60	12.60	4.2	37.8	2.16x10-2
	20.0	13.20	0.20	13.40	4.2	0.4	2.15x10-2
BV-X2	10.0	10.00	0.50	10.50	4.2	1.8	1.24x10-3
	15.0	15.00	0.50	15.50	4.2	2.5	1.16x10-3
	20.0	8.45	0.50	8.95	4.2	0.2	1.61x10-4
BV-X3	10.0	10.00	0.54	10.54	4.2	0.9	6.16x10-4
	15.0	15.00	0.50	15.50	4.2	7.8	3.63x10-3
	20.0	14.43	0.50	14.93	4.2	3.5	1.69x10-3
BV-X4	5.0	1.30	0.15	1.45	4.2	0.4	1.99x10-3
	10.0	0.95	0.15	1.10	4.2	0.4	2.62x10-3
	15.0	7.98	0.15	8.13	4.2	1.0	8.87x10-4

TABLE I .2.4-3(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	1.41x10-4
	15.0	1.12	0.22	1.34	4.2	0.2	1.08x10-3
BV-Y2	10.0	0.95	0.32	1.27	4.2	2.4	1.36x10-2
BV-Z1	5.0	4.25	0.20	4.45	4.2	1.1	1.78x10-3
	10.0	8.17	0.20	8.37	4.2	1.1	9.48x10-4
	15.0	11.15	0.20	11.35	4.2	1.0	6.36x10-4
	20.0	12.55	0.60	13.15	4.2	1.0	5.49x10-4
BV-Z2	5.0	5.00	0.40	5.40	4.2	1.5	2.00x10-3
	10.0	8.91	0.50	9.41	4.2	2.4	1.84x10-3
	15.0	10.32	0.50	10.82	4.2	4.2	2.80x10-3

TABLE I .2.4-4 SUMMARY OF PERMEABILITY TEST RESULTS(LUGEON TEST)

BOREHOLE No.	TEST SECTION (m)	YIELDED PRESSURE (kg/cm²)	TOTAL PRESSURE (kg/cm²)	WATER LOSS (l/min)	LUGEON UNIT
BV-W2	27.76 - 30.00	1.4	3.32	15.3	
		2.8	4.70	27.4	
		3.5	5.39	31.7	
		4.2	6.07	42.5	95.0
		3.5	5.39	29.6	
		2.8	4.72	29.6	
		1.4	3.33	9.3	

TABLE I .2.5-1 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 I .2.5-2 MECHANICAL PROPERTIES OF CLAYEY MATERIALS

Sample No.	UNIAXIAL COMPRESSIVE TEST	TRIAXIAL COMPRESSIVE TEST	
	Strength (kg/cm²)	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 I .2.6-1 MONTHLY RAINFALL FROM 1951 TO 1987

JAN.	FEB.	MAR.	APR.	MAY.	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.	TOTAL
272	80	164	98	20	53	52	10	10	19	54	18	850
31	91	186	213	94	61	160	70	91	72	180	262	1511
196	189	86	47	306	107	33	32	11	6	79	109	1201
34	134	414	46	54	90	51	34	8	8	79	109	1061
153	439	400	65	143	54	14	50	19	17	21	136	1511
110	224	98	67	68	34	9	14	8	41	71	119	863
195	111	191	96	24	12	47	8	43	3	42	120	892
200	66	734	211	6	7	81	9	16	5	15	52	1402
80	148	121	17	13	4	20	47	1	8	140	60	659
280	372	260	0	0	51	34	0	70	1	2	51	1121
41	66	153	125	37	145	61	32	6	12	16	508	1202
99	110	255	26	26	16	1	6	19	26	23	61	668
121	112	28	53	19	25	47	1	1	17	36	6	466
334	82	102	52	18	8	8	27	18	4	0	73	726
213	51	12	151	6	5	46	71	79	44	67	19	764
228	55	80	38	14	50	47	8	36	4	18	193	771
167	11	229	116	27	3	31	24	25	35	138	134	940
5	437	310	32	21	29	37	3	20	12	5	33	944
94	133	206	83	78	46	61	21	14	0	18	409	1163
229	78	301	12	12	23	17	29	21	5	10	4	741
76	205	31	198	185	6	16	19	2	2	144	28	912
57	347	66	61	15	18	32	215	9	120	27	27	994
130	104	192	33	80	30	33	16	11	6	1	49	685
90	166	112	34	29	19	13	40	28	5	1	110	647
55	330	56	32	18	20	15	15	20	7	32	28	628
105	148	41	117	146	44	25	75	12	45	51	126	935
194	130	41	200	77	33	63	25	19	34	14	89	919
104	51	70	365	27	47	67	8	61	29	62	45	936
146	161	94	95	30	16	18	54	9	3	29	432	1087
717	77	330	173	34	25	15	3	19	35	23	47	1498
87	48	111	192	19	28	10	18	14	10	28	156	721
152	0	76	92	133	23	48	65	31	79	123	69	891
119	80	19	23	11	5	8	9	7	7	18	24	330
200	131	12	22	10	16	14	12	62	2	32	89	602
477	641	134	34	18	73	42	30	19	23	36	261	1788
86	363	98	26	183	3	6	85	4	49	11	89	1003
185	511	151	391	50	9	32	16	0	102	6	0	1453
164	175	161	98	55	33	36	32	23	24	45	112	958
75	71	206	146	27	16	50	4	17	14	108	330	1064
388	198	215	62	29	83	23	36	121	37	18	124	1334
47	208	84	110	30	41	—	—	—	—	—	—	520

s : 1951 1987 Measured at Line Barracks Station about
0.7 km northeast from La Butte Area
1988 Date Measured at La Butte Station.

TABLE I .2.6-2 MONTHLY RAINFALL IN RAINY SEASON FROM 1951 TO 1987

YEAR	NOV.	DEC.	JAN.	FEB.	MAR.	APR.	TOTAL
1951/52	54	18	31	91	186	213	593
1952/53	180	262	196	189	86	47	960
1953/54	79	109	34	134	414	46	816
1954/55	79	109	153	439	400	65	1245
1955/56	21	136	110	224	98	67	656
1956/57	71	119	195	111	191	96	783
1957/58	42	120	200	66	734	211	1373
1958/59	15	52	80	148	121	217	633
1959/60	140	60	280	372	260	0	1112
1960/61	2	51	41	66	153	125	438
1961/62	16	508	99	110	255	26	1014
1962/63	23	61	121	112	28	53	398
1963/64	36	6	334	82	102	52	612
1964/65	0	73	213	51	12	151	500
1965/66	67	19	228	55	80	38	487
1966/67	18	193	167	11	229	116	734
1967/68	138	134	5	437	310	32	1056
1968/69	5	33	94	133	206	83	554
1969/70	18	409	229	78	301	12	1047
1970/71	10	4	76	205	31	198	524
1971/72	144	28	57	347	66	61	703
1972/73	27	27	130	104	192	33	513
1973/74	1	49	90	166	112	34	452
1974/75	1	110	55	330	56	32	584
1975/76	32	28	105	148	41	117	471
1976/77	51	126	194	130	41	200	742
1977/78	14	89	104	51	70	365	693
1978/79	62	45	146	161	94	95	603
1979/80	29	432	717	77	330	173	1758
1980/81	23	47	87	48	111	192	508
1981/82	28	156	152	0	76	92	504
1982/83	123	69	119	80	19	23	433
1983/84	18	24	200	131	12	22	407
1984/85	32	89	477	641	134	34	1407
1985/86	36	261	86	363	98	26	870
1986/87	11	89	185	511	151	391	1338
Mean	46	115	161	178	161	98	11385
1987/88	6	0	75	71	206	146	504
1988/89	108	330	388	198	215	62	1301
1989/90	18	124	47	208	84	110	591

Remarks : 1951 - 1987 Measured at Line Barracks Station about 0.7 km northeast from La Butte Area.
1988 - Date Measured at La Butte Station

TABLE I .2.6-3 TOTAL MONTHLY DISPLACEMENT OF BOREHOLE INCLINOMETER

Unit :
Displacement : cm
Average : cm/month
Depth : m

Month (Depth)	BV-V2 (2.0m)	BV-V4 (2.5m)	BV-X1 (5.0m)	BV-X3 (2.5m)	BV-X4 (3.5m)	BV-Z1 (1.0m)	BV-Z2 (3.0m)	BV-Z3 (1.0m)
Sep. 89			0.504					
Oct.	0.315			0.275		0.230	0.080	0.385
Nov.	0.281		4.120	-1.160	0.004	-0.122	-0.068	-0.411
Dec.	0.106		-1.998	0.675	0.019	0.097	0.181	0.205
Jan. 90	-0.131	-0.701	0.131	0.458	-0.013	0.009	-0.069	0.002
Feb.	0.091	0.112	0.372	-0.214	0.236	0.082	0.423	0.223
Mar.	0.182	0.086		0.688	0.369	0.026	0.363	0.426
Apr.	-0.087	-0.204		-0.108	0.017	-0.101	0.029	-0.133
May.	-0.152	0.160		0.124	0.011	-0.042	0.069	0.160
Jun.	0.336	0.023		-0.020	0.007	-0.025	0.042	
Total	0.941	-0.524	3.729	0.396	0.623	0.186	0.983	0.899
Max.	0.336	-0.701	4.120	-1.160	0.369	0.230	0.423	0.426
Min.	-0.087	0.023	0.131	-0.020	0.004	0.009	0.029	0.002
Average	0.104	0.065	0.621	0.044	0.077	0.020	0.109	0.099

TABLE I .2.6-4 TOTAL MONTHLY DISPLACEMENT OF TILTMETER

Unit : Second

Month	T - 1		T - 2		T - 3		T - 4		T - 5		T - 6		T - 7	
	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
Oct. 88	—	—	7	-30	8	3	—	—	-9	-34	53	-41	-0	-0
Nov.	-28	12	31	15	36	83	-17	2	37	6	67	-35	-19	228
Dec.	65	219	27	173	625	52	87	-38	-96	233	188	34	-159	-142
Jan. 89	233	-59	—	—	335	-59	-22	-147	-38	-146	78	-8	26	-30
Feb.	209	404	119	60	173	217	185	-180	44	-98	-213	315	—	—
Mar.	155	177	-221	-58	-42	-255	173	408	194	345	265	256	107	-63
Apr.	-32	-58	2	28	131	119	254	95	47	-107	-168	373	43	-42
May.	16	-59	-55	154	-102	-115	210	166	332	-369	-440	230	-1	56
Jun.	-88	-66	-303	115	3	377	69	292	230	-361	156	292	10	-25
Jul.	57	-145	-178	-16	-32	-12	18	-11	255	-220	21	23	0	21
Aug.	48	-51	-164	8	-46	-35	2	70	183	-1	-17	4	17	24
Sep.	105	76	78	7	200	60	79	-56	602	343	24	-218	43	-42
Oct.	27	0	-26	-8	130	85	183	138	-182	253	102	243	-33	-12
Nov.	7	-20	-154	-3	-81	-80	41	37	47	227	301	-10	227	-6
Dec.	63	60	20	25	-24	-40	314	-66	42	47	45	45	102	-20
Jan. 90	176	-37	75	96	-38	-13	21	-162	33	8	324	178	-95	-17
Feb.	128	1	465	54	-549	19	-287	-347	27	-15	239	-313	68	-33
Mar.	189	1	-521	-27	879	-65	203	15	-12	8	405	-390	56	-65
Apr.	5	-60	-205	-15	-71	51	388	10	-35	-53	480	-50	-2	-20
May.	19	-25	252	-18	436	-73	73	60	-109	192	300	-78	7	15
Jun.	33	-36	-131	-25	-43	-26	-19	36	-86	323	25	93	10	34
Total	1387	-104	-882	535	1928	353	1781	322	1506	581	2235	943	407	-139
Max.	233	404	-521	173	879	377	-287	408	602	-369	480	-390	227	228
Mini.	5	1	2	7	3	3	2	2	-9	-1	-17	4	0	-6
Direction	N4. 30W		S31. 30E		N10. 36E		N10. 19E		N21. 06E		N22. 87E		N19. 04W	
Daily Average (Sec/day)	2.31		168		3.22		3.04		2.65		3.98		0.71	

TABLE I .2.6-5 TOTAL MONTHLY DISPLACEMENT OF EXTENSOMETER

Unit : mm

Month	E-1	E-2	E-3	E-4	E-5	E-6	E-7	E-8	E-9	E-10	E-11	E-12	E-13	E-14	E-15	E-16	E-17
Oct.88	0.0	0.4	3.2	3.2													
Nov.	0.0	2.2	2.3	2.3													
Dec.	-3.0	19.0	27.2	27.2													
Jan.89	-20.9	25.4	52.8	45.5													
Feb.	7.8	42.8	56	56.8													
Mar.	17.3	20.7	48.1	52.0													
Apr.	30.6	19.1	56.5	48.8													
May.	5.5	1.4	11	1.3													
Jun.	6.9	0.4	0.5	0.8	0.0	0.0	0.8	-0.8	0.4	0.0	0.6	0.0	0.0	0.0	0.0	0.0	
Jul.	3.1	0.4	3.4	1.6	0.0	0.0	-0.8	0.0	-1.7	0.4	0.0	-0.2	0.0	0.2	-0.3	0.0	
Aug.	2.5	0.0	1.2	0.4	0.0	0.0	0.6	0.0	-0.9	0.4	0.0	0.2	0.0	0.4	-0.5	0.4	
Sep.	0.5	2.8	-1.4	1.8	-0.5	1.2	1.0	-0.3	9.0	-1.8	-1.2	-0.4	4.2	0.7	0.2	-0.8	
Oct.	-0.2	1.2	4.7	0.0	-0.6	0.0	0.6	0.4	-0.3	-0.8	0.2	0.0	0.2	0.4	0.0	1.4	
Nov.	0.3	0.4	34.4	0.0	0.0	0.0	-0.6	-0.1	-1.5	-1.8	0.0	0.8	0.0	0.7	0.0	1.4	
Dec.	0.3	2.4	48.6	-0.5	-3.2	0.2	-0.6	-5.6	-1.6	1.5	-0.3	0.3	2.9	1.6	0.4	1.3	
Jan.90	0.0	-2.0	31.8	0.2	-0.6	-1.5	-1.0	-0.4	-2.4	0.2	-1.5	-0.4	0.1	-0.1		0.2	
Feb.	-2.1	2.3	17.7	-2.1	-3.3	1.3	1.0	-6.1	7.4	2.1	0.1	-0.2	1.4	0.1		-1.1	
Mar.	-8.5	-0.7	201.9	-0.3	0.5	0.0	-0.7	-0.8	-7.9	0.0	0.8	-0.9	-1.8	0.7		-0.6	6.6
Apr.	-0.2	-0.7	49.7	-2.2	-1.9	0.0	0.0	1.3	7.3	-2.5	-0.6	0.0	-2.4	-0.3		-2.0	19.5
May.	-0.8	-3.5	28.9	0.7	-1.1	-1.4	-1.4	-3.7	-5.4	0.8	0.7	0.2	-1.7	1.3		0.6	-1.4
Jun.	0.7	-0.7	16.4	0.8	-0.1	-0.7	-0.3	0.1	-1.8	0.4	-1.0	-0.8	0.5	0.0		0.1	3.1
Total	38.8	133.3	694.9	247.2	-10.8	-0.9	-1.4	16.0	0.6	-1.1	-3.4	-1.8	3.4	5.7	-0.2	-0.7	27.8
Max.	30.6	42.8	201.9	56.8	-3.2	-1.5	-1.4	-6.1	9.0	-2.5	-1.5	-0.9	4.2	1.6	-0.5	-2	19.5
Min.	0	0	0.5	0	0	0	0	0	-0.3	0	0	0	0	0	0	0	-1.4
Averag	1.9	0.03	33.09	11.77	-0.83	-0.07	-0.11	-1.23	0.05	-0.08	-0.26	-0.14	0.26	0.44	-0.03	-0.05	6.95

TABLE I .3.3-1 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

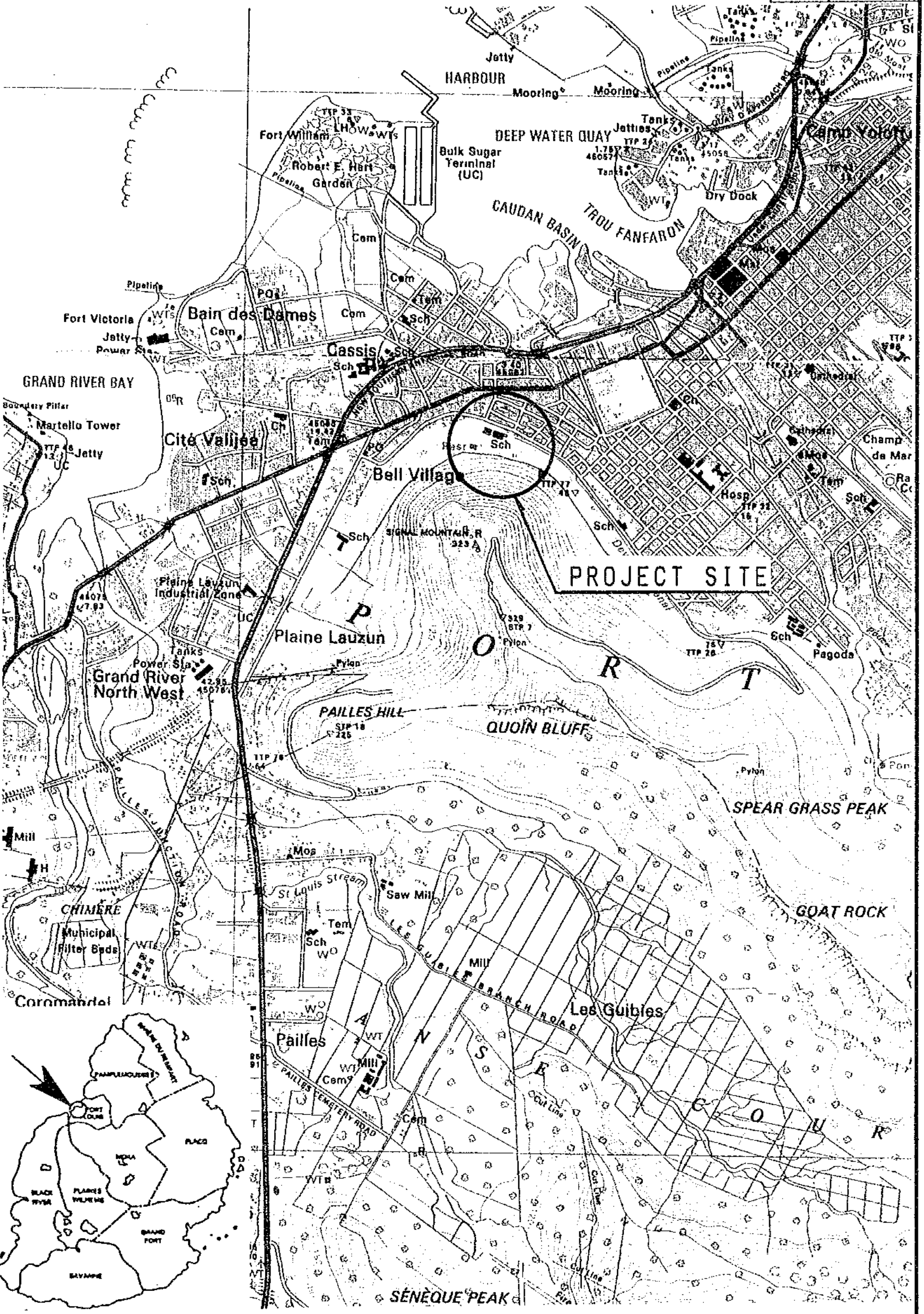
TABLE I .3.3-2 ALLOWABLE EXTERNAL PRESSURE

THICKNESS (mm)	STIFFENER RING SIZE (mm)	STIFFENER RING INTERVAL (m)	ALLOWABLE PRESSURE FOR BUCKLING (tf/m²)	ALLOWABLE PRESSURE FOR COMPRESSION (I) (tf/m²)	ALLOWABLE PRESSURE FOR COMPRESSION (II) (tf/m²)
1. Without perforation					
	....	....	11.05	40.90	9.25
	H-125	2.0	44.24	56.48	24.25
2.7	H-125	1.5	55.30	61.68	28.44
	H-125	1.0	77.43	72.07	36.38
	H-125	0.5	143.81	103.25	58.70
	....	....	13.13	48.47	10.96
	H-125	2.0	46.32	64.05	26.31
3.2	H-125	1.5	57.38	69.25	30.62
	H-125	1.0	79.51	79.64	38.76
	H-125	0.5	145.89	110.82	61.45
2. With perforation					
	....	....	8.84	40.90	8.22
	H-125	2.0	42.03	56.48	23.94
2.7	H-125	1.5	53.09	61.68	28.18
	H-125	1.0	75.22	72.07	36.18
	H-125	0.5	131.59	103.25	58.57
	....	....	10.51	48.47	9.74
	H-125	2.0	43.70	64.05	25.90
3.2	H-125	1.5	54.76	69.25	30.28
	H-125	1.0	76.89	79.64	38.50
	H-125	0.5	143.26	110.82	61.28

* (I) : in the case that only compressive strength is considered.
(II) : in the case that bending moment os considered.

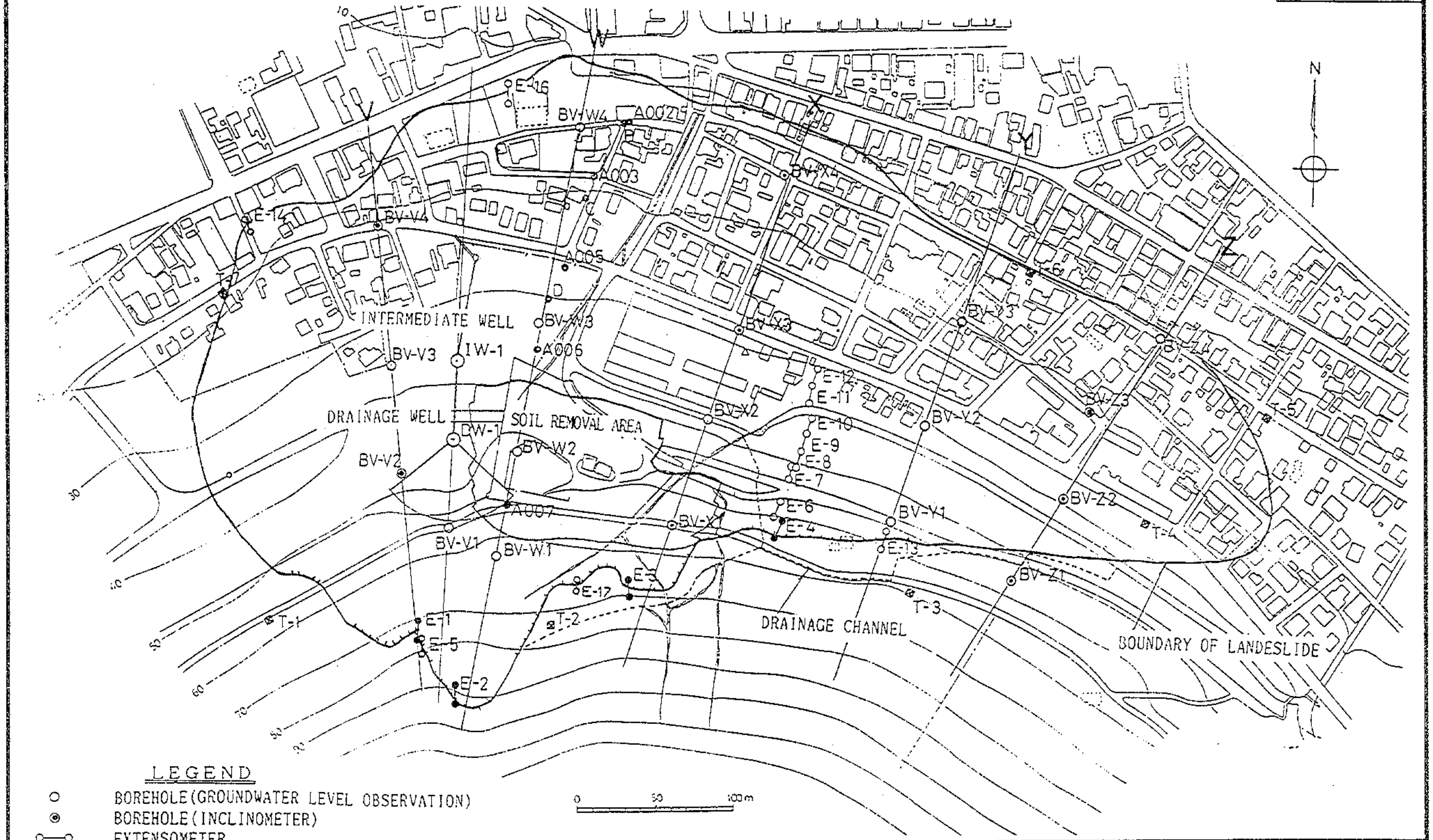
FIGURES

Fig. I.1.1



LOCATION MAP OF THE PROJECT SITE	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.1

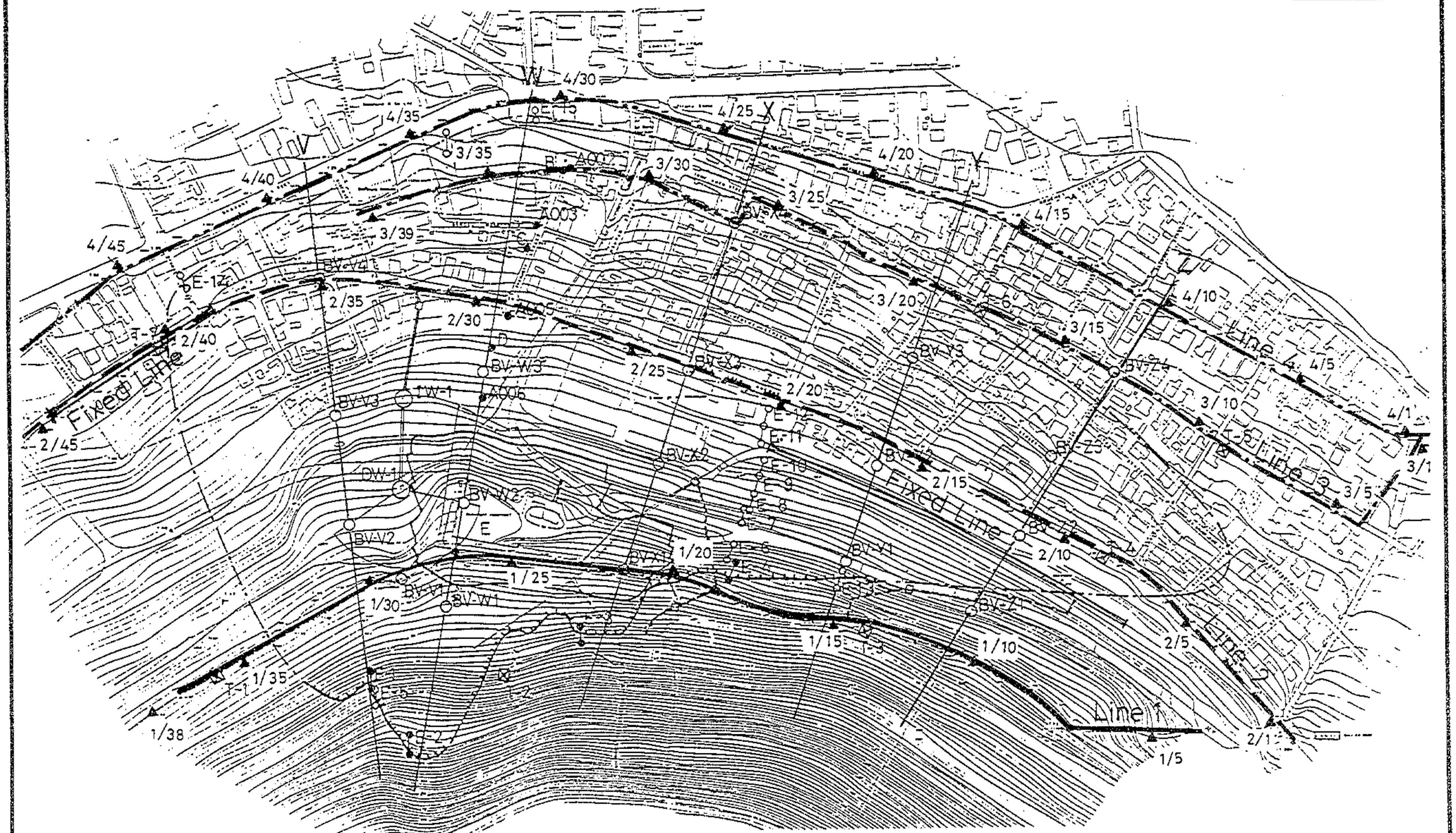


LOCATION MAP FOR INVESTIGATION SITE AND MONITORING EQUIPMENT

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.3-1

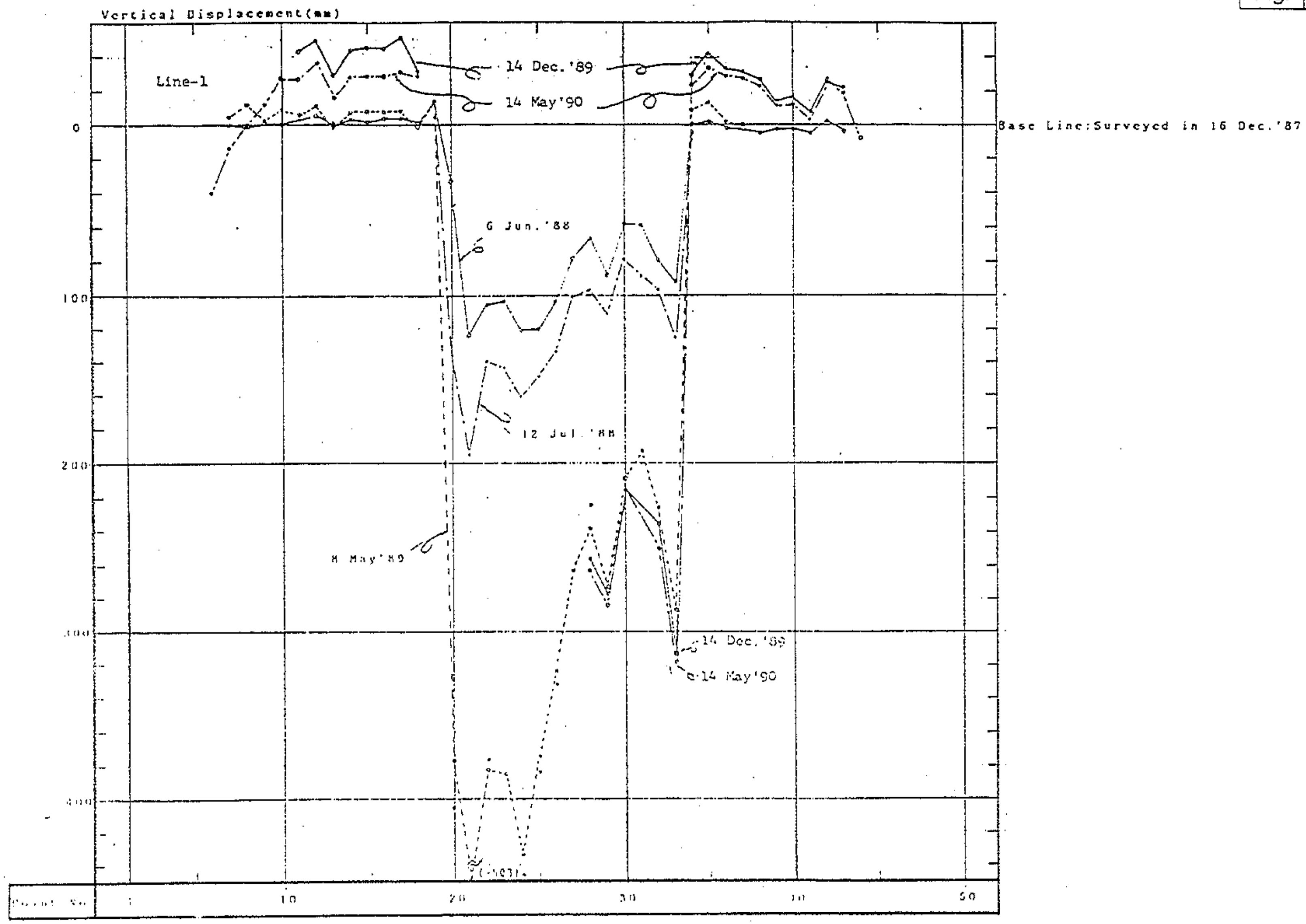


LOCATION MAP FOR GROUND SURVEY

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.3-2

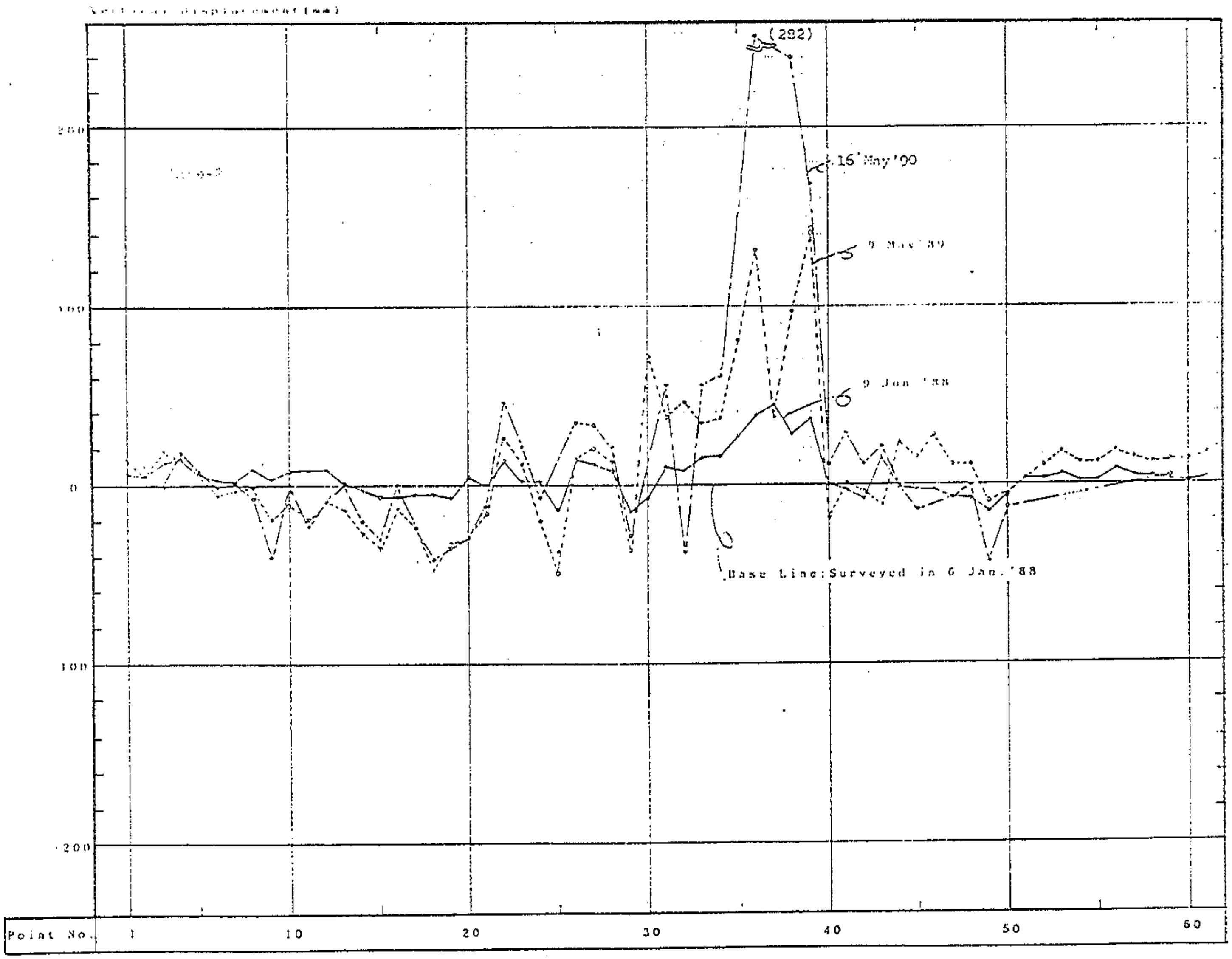


PROFILE OF THE LEVELING SURVEY
ALONG LINE-1

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

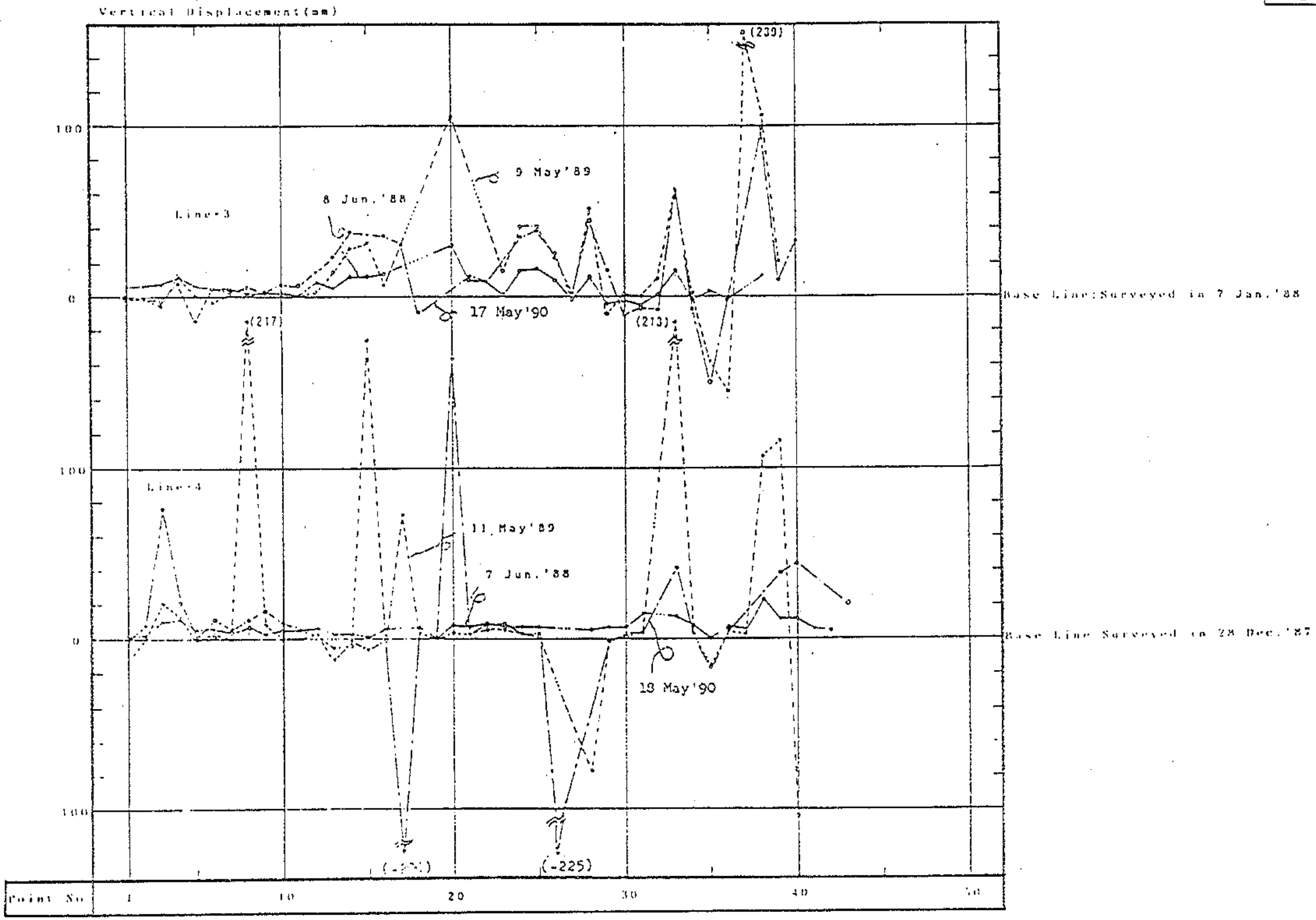
Fig. I.2.3-3



PROFILE OF THE LEVELING SURVEY ALONG LINE-2

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

Fig. I.2.3-4

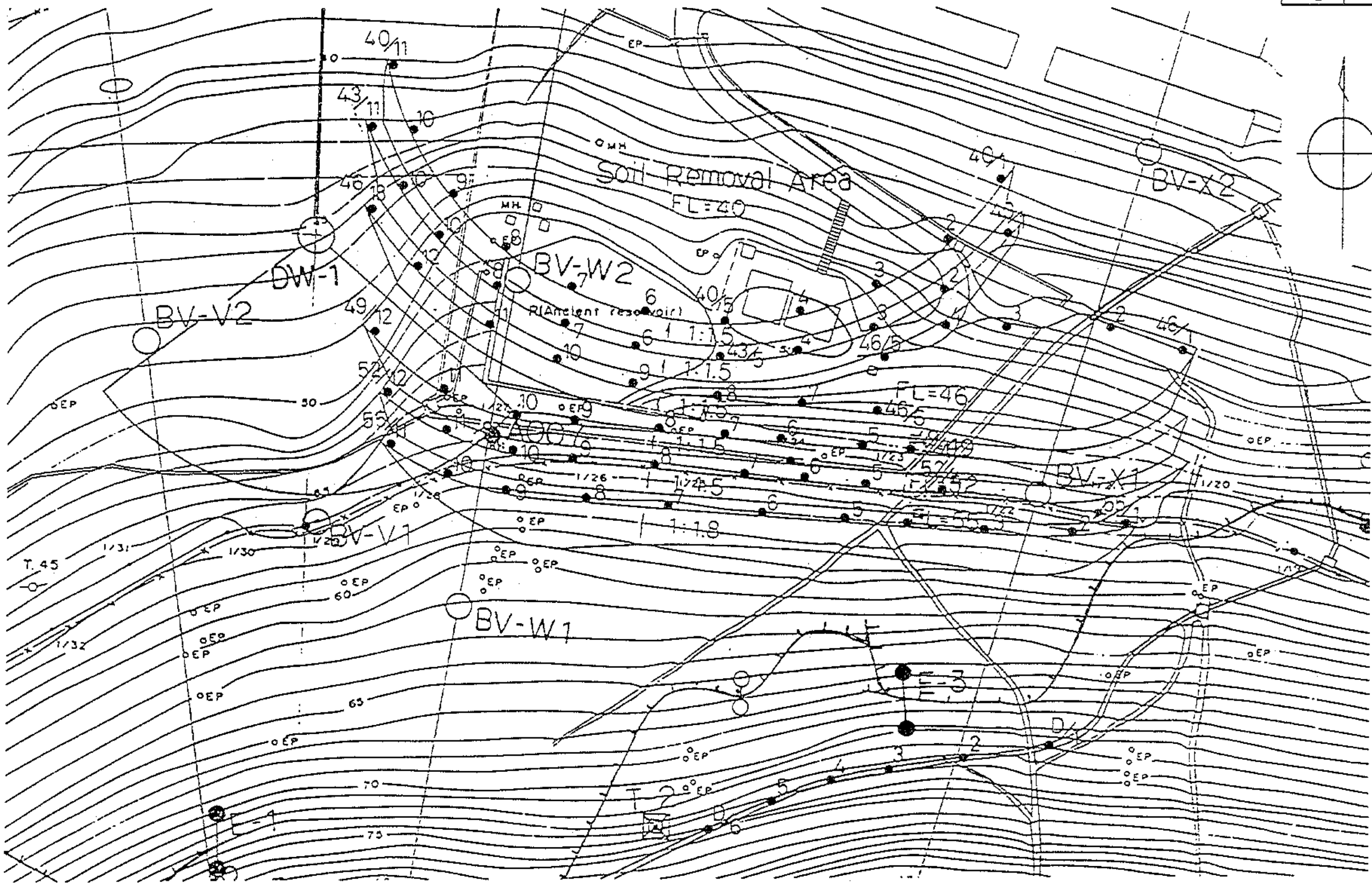


PROFILE OF THE LEVELING SURVEY ALONG LINE-3 AND LINE-4

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

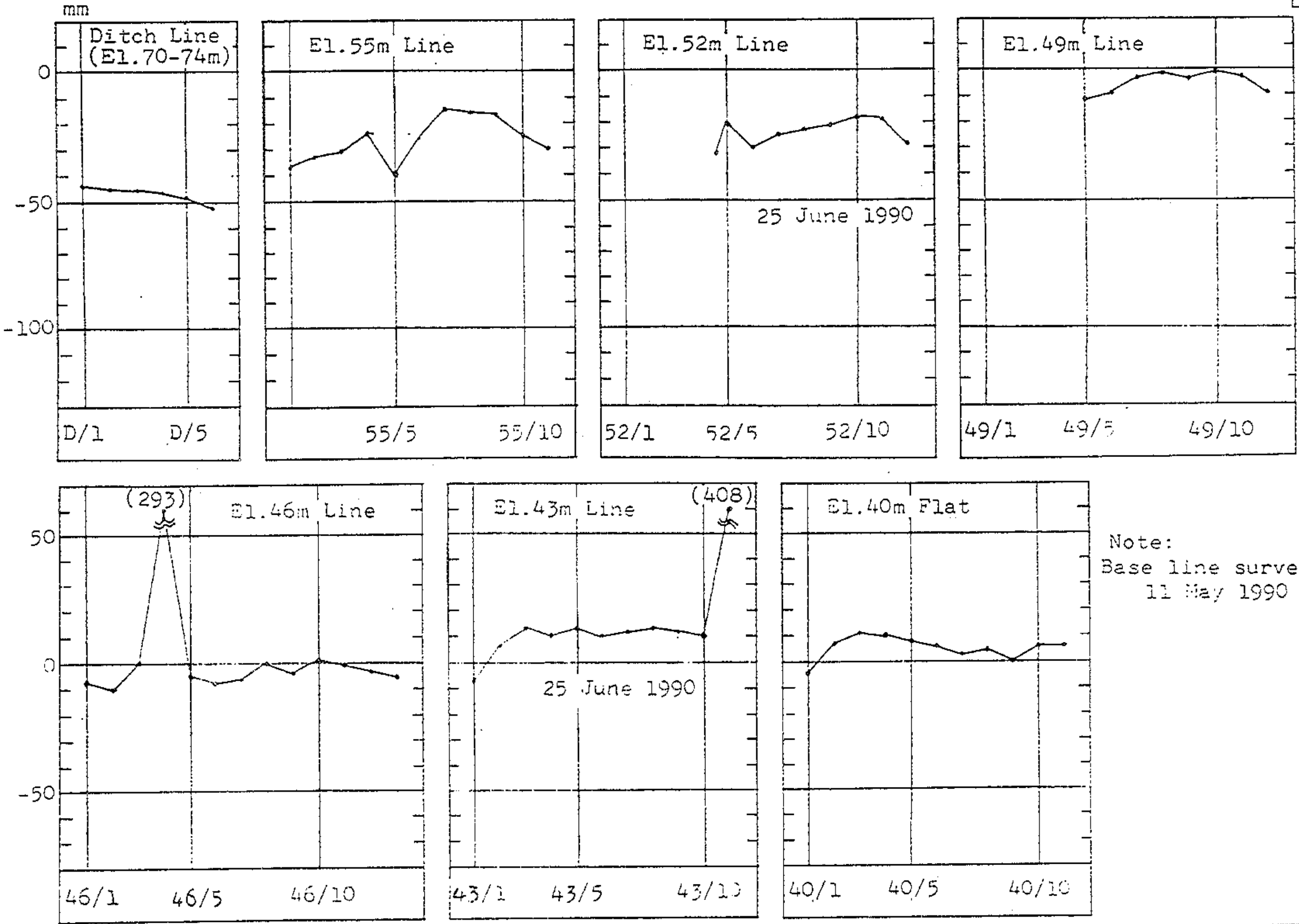
Fig. I.2.3–5



LAYOUT OF LEVELING SURVEY POINTS
AT THE SOIL REMOVAL AREA

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

Fig. I.2.3-6



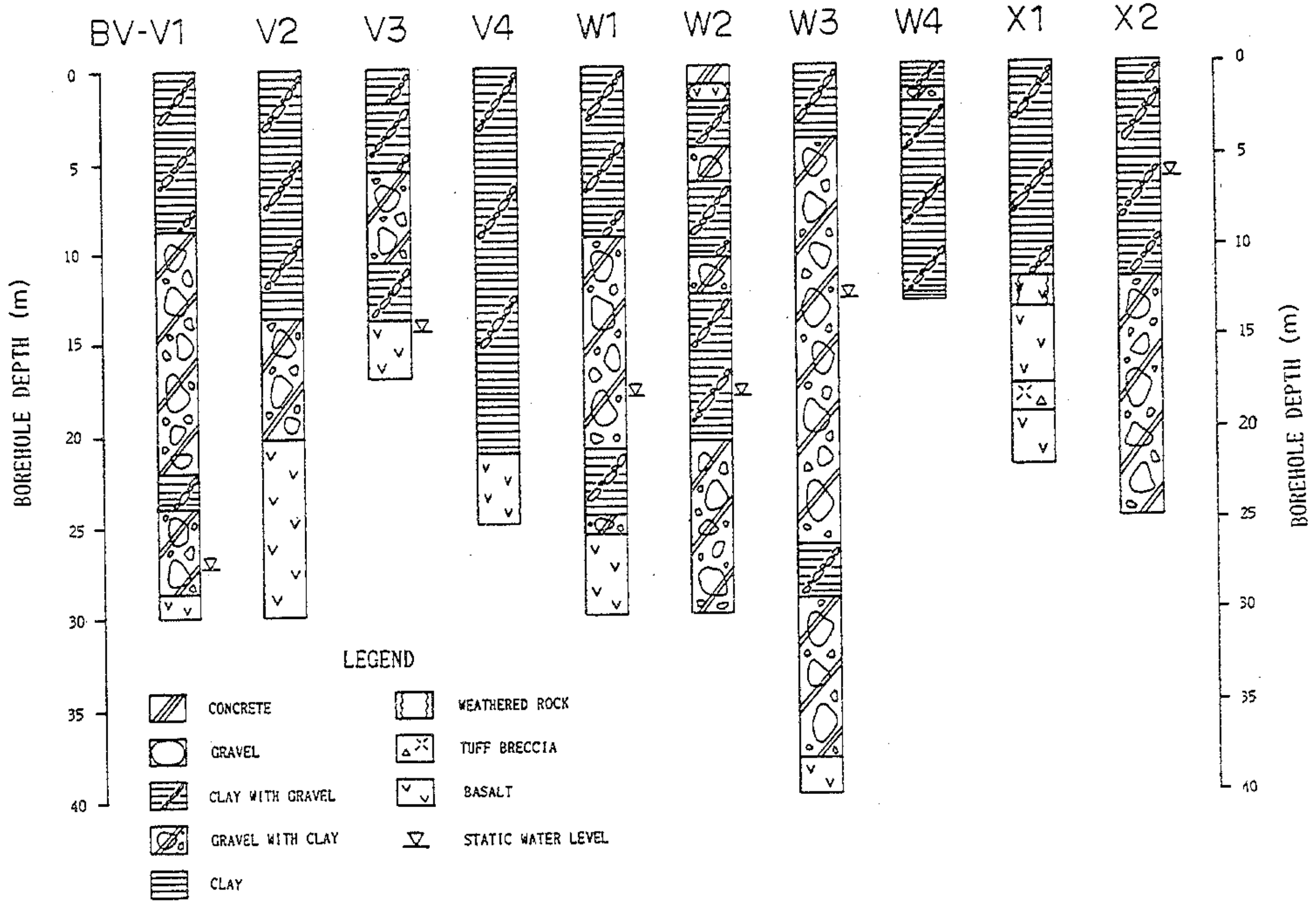
Note:
Base line survey:
11 May 1990

LEVELING SURVEY RESULTS AT THE SOIL REMOVAL AREA

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.4–1

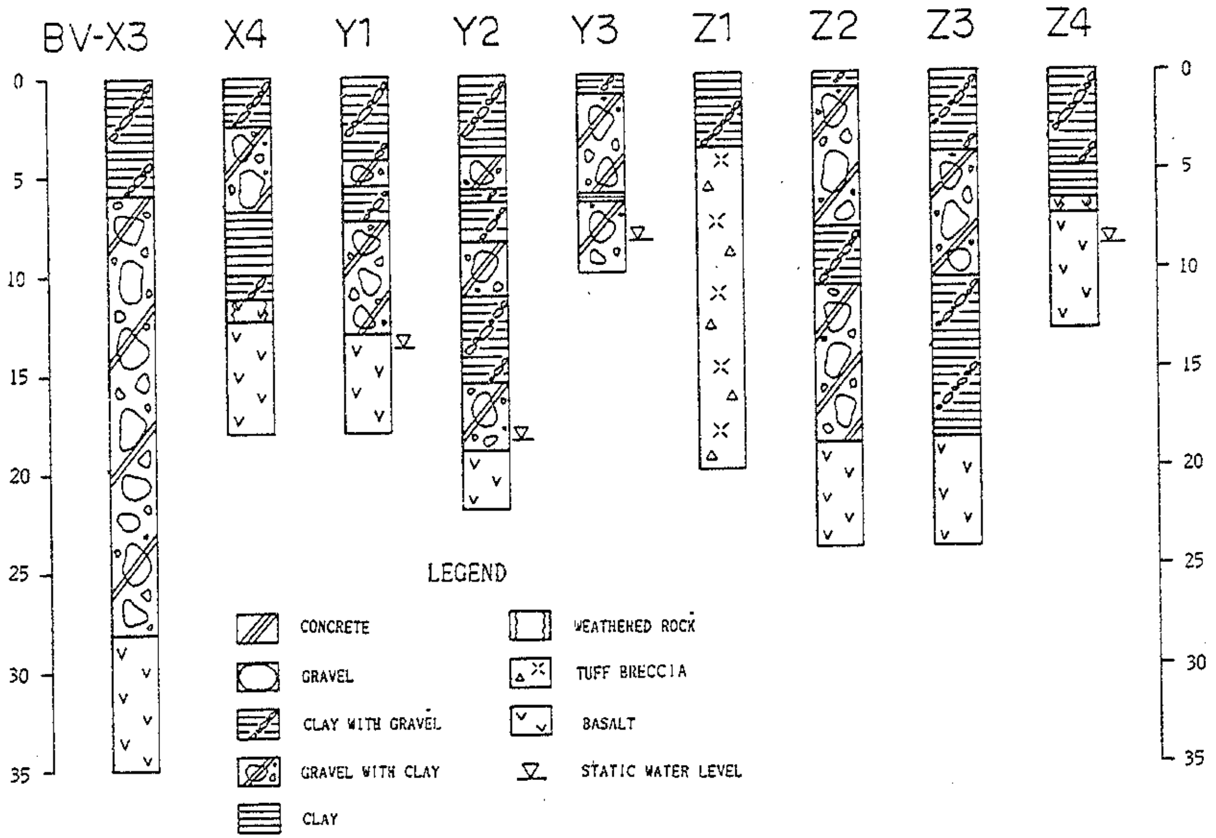


CORE LOGS OF RECOVERED SAMPLES (BV-V1～BV-X2)

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.4–2

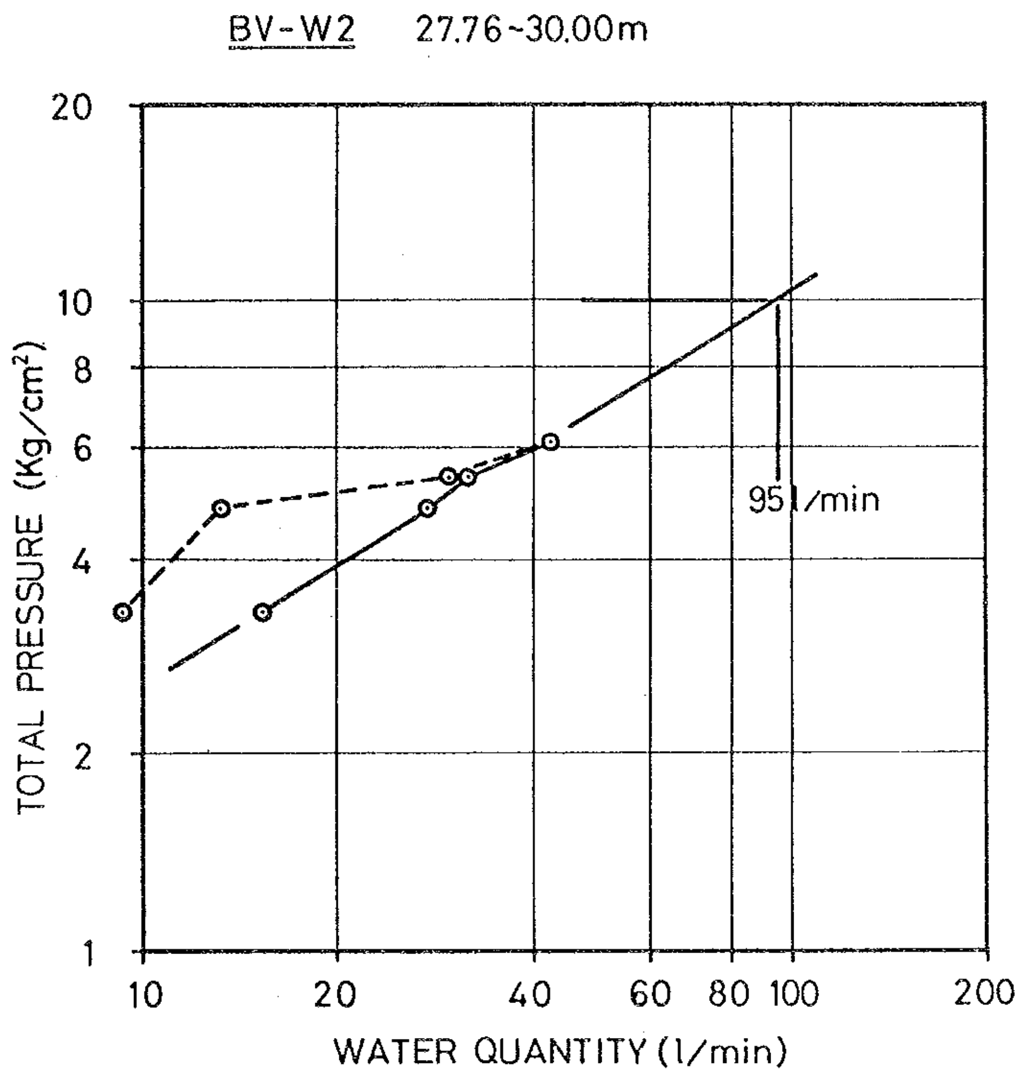


CORE LOGS OF RECOVERED SAMPLES (BV-X3～BV-Z4)

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

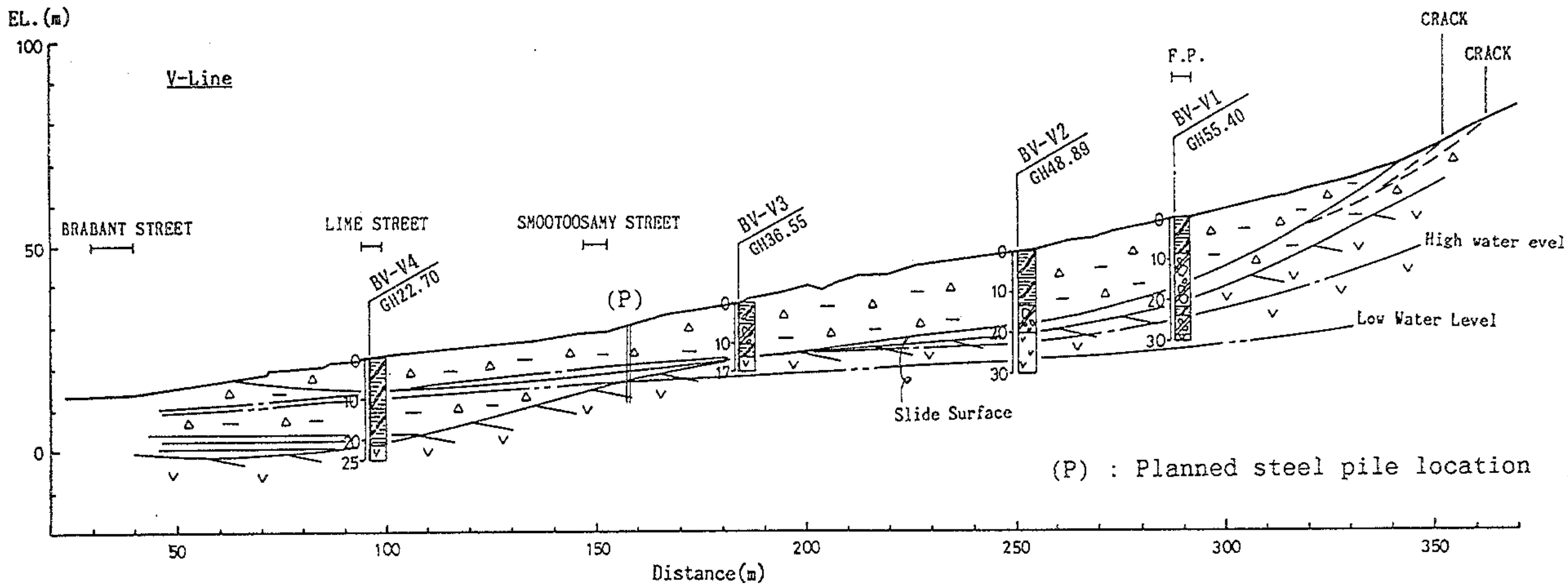
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.4-3



P-Q GRAPH	GOVERNMENT OF MAURITIUS LANDSLIDE PROTECTION PROJECT IN PORT LOUIS JAPAN INTERNATIONAL COOPERATION AGENCY

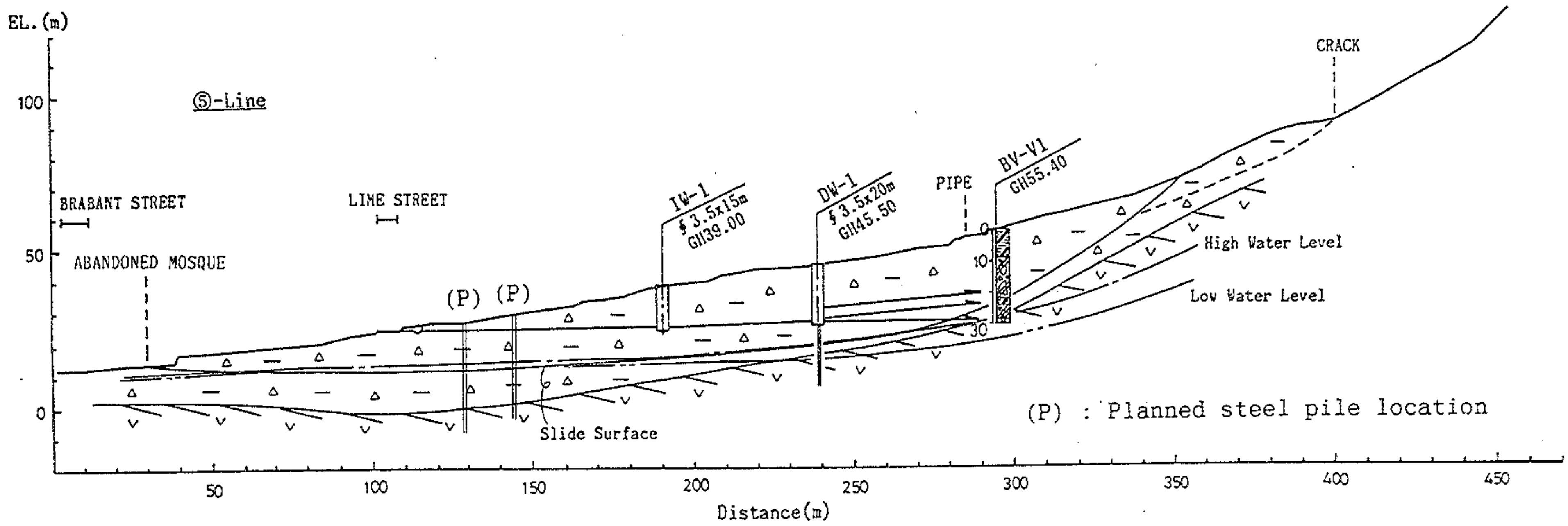
Fig. I.2.4-4



GEOLOGICAL PROFILE ALONG V-LINE

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

Fig. I.2.4-5

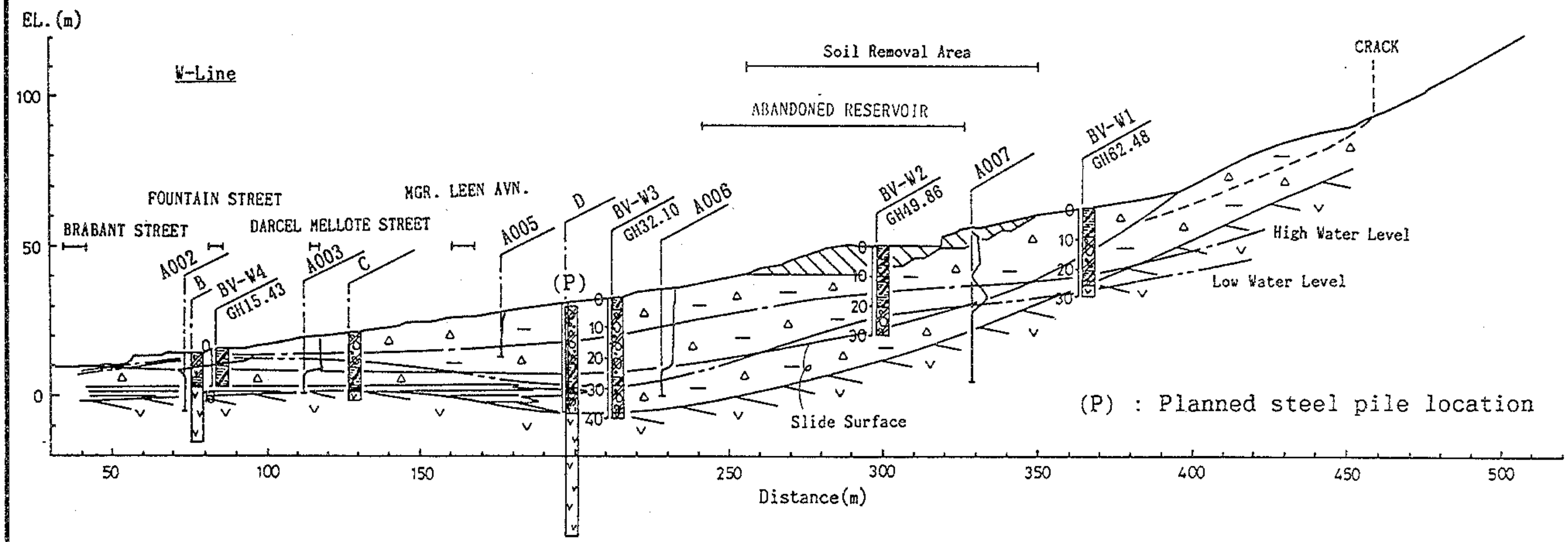


GEOLOGICAL PROFILE ALONG THE LINE THROUGH THE DRAINAGE WELL AND THE INTERMEDIATE WELL

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

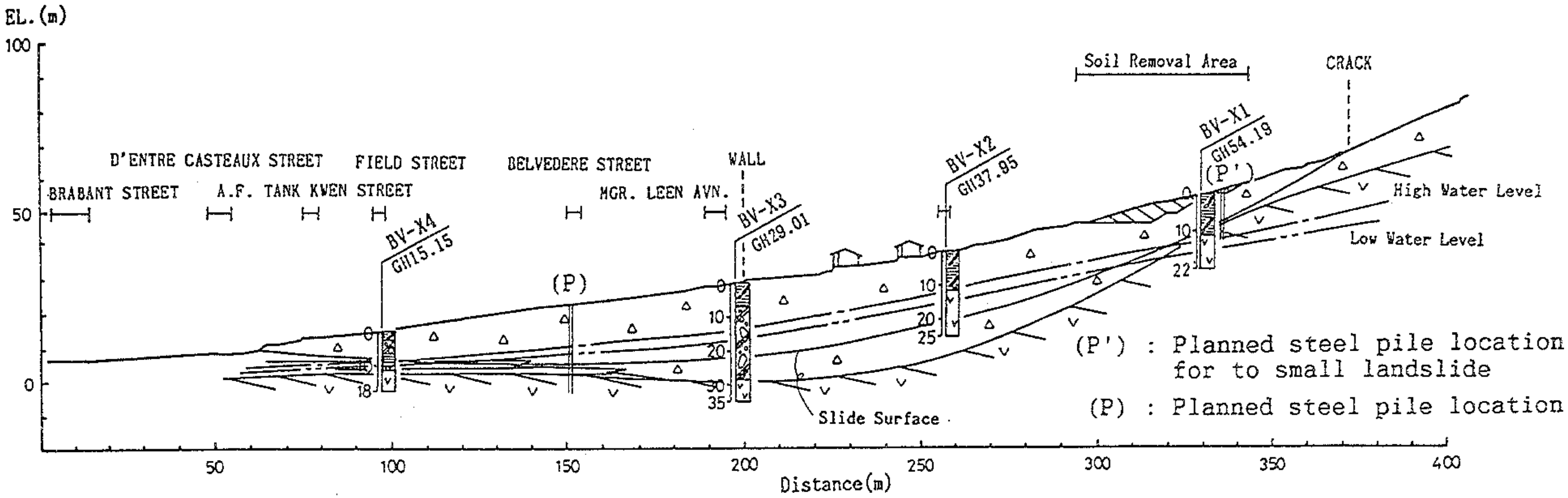
Fig. I.2.4-6



GEOLOGICAL PROFILE ALONG W-LINE

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

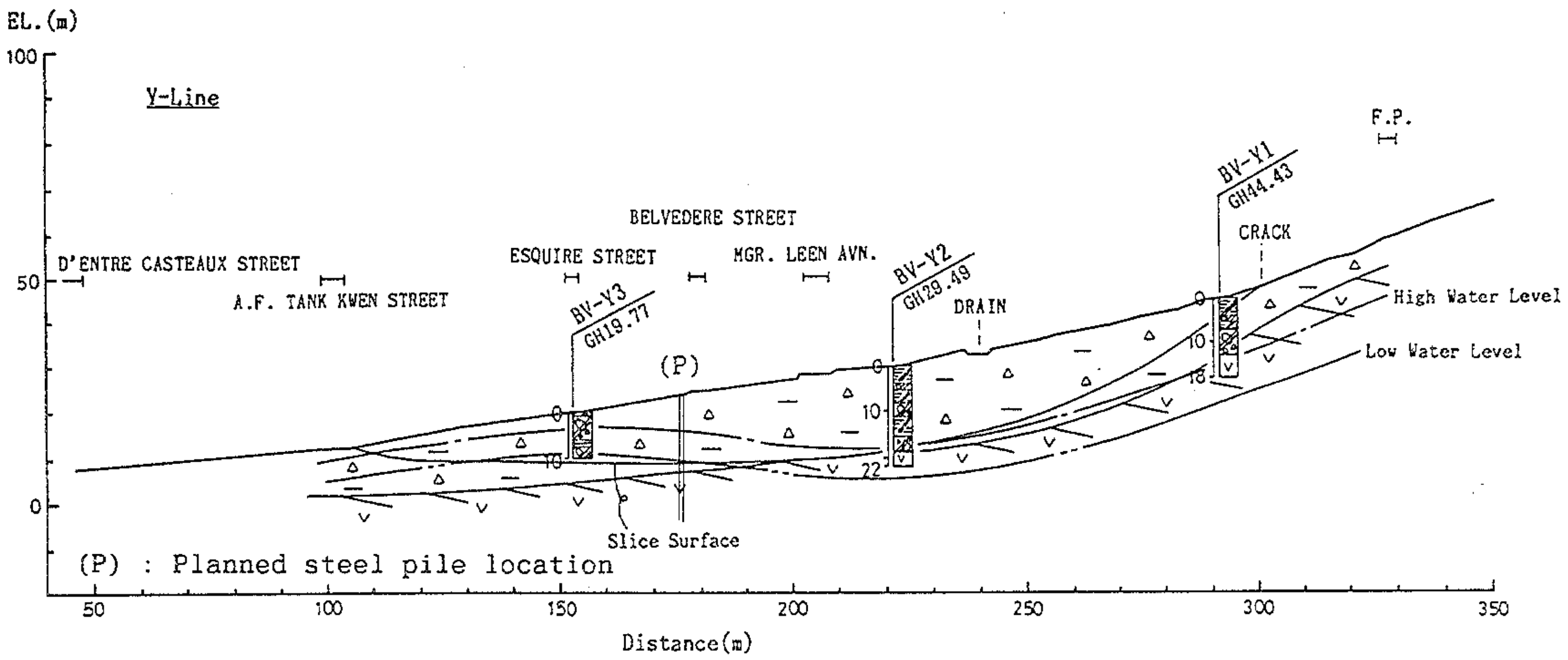
Fig. I.2.4–7



GEOLOGICAL PROFILE ALONG X-LINE

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

Fig. I.2.4-8

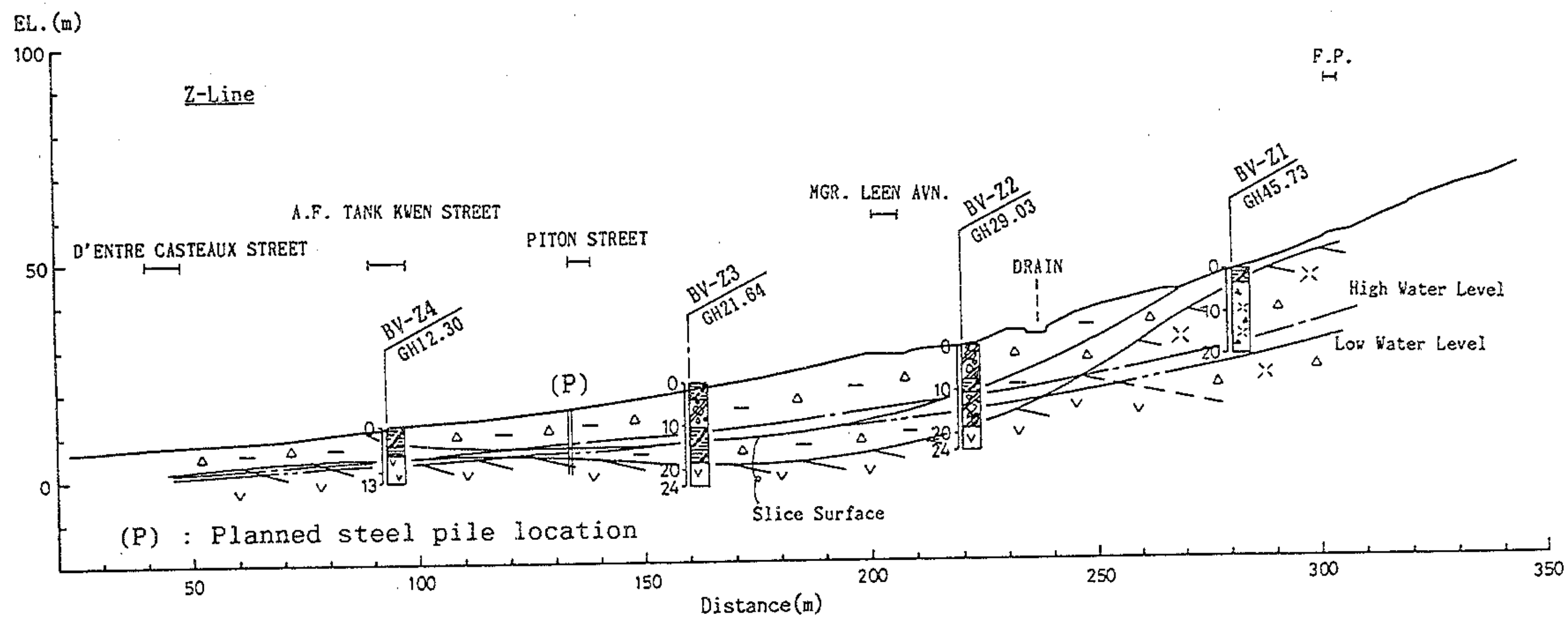


GEOLOGICAL PROFILE ALONG Y-LINE

GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

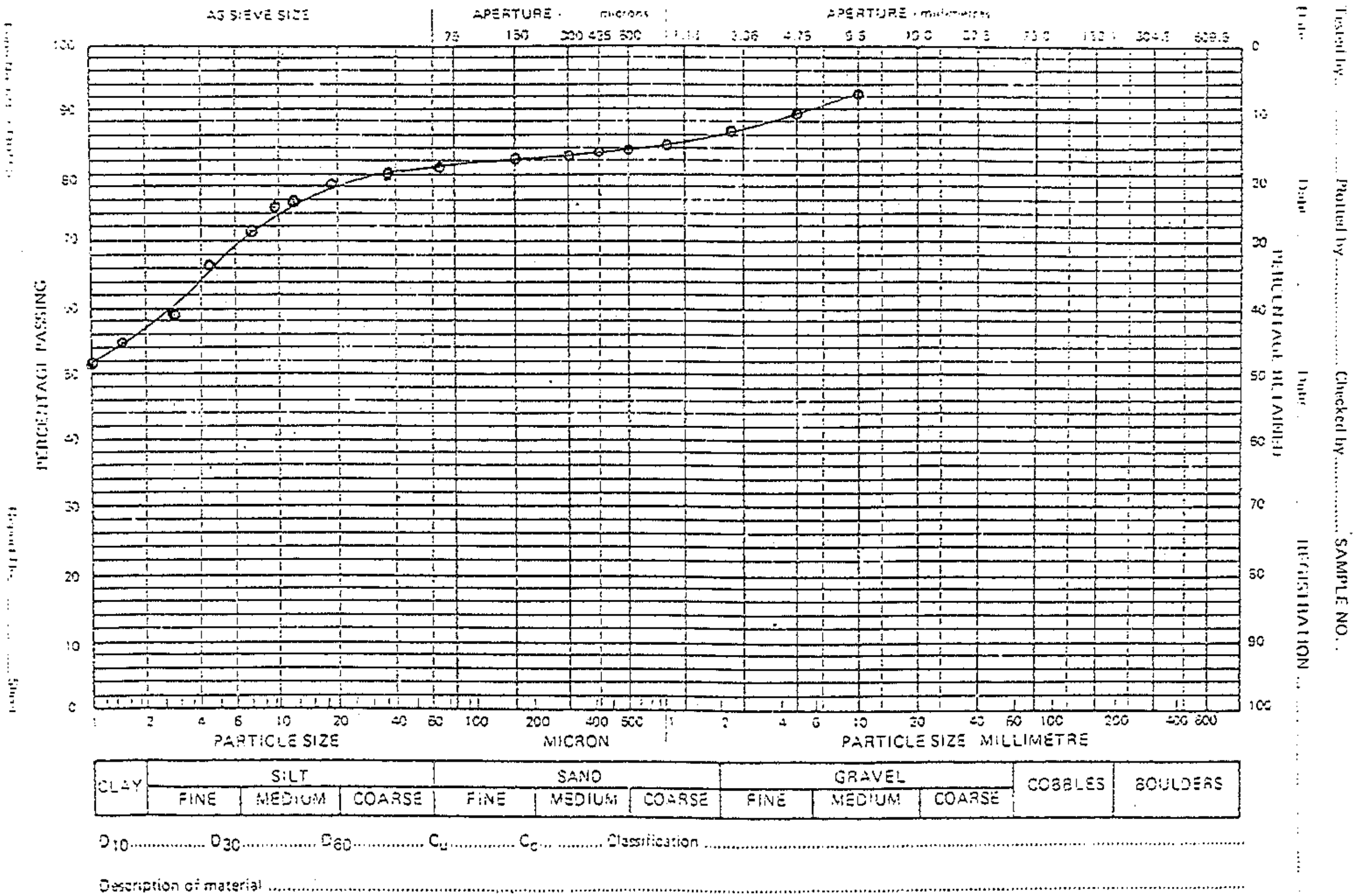
Fig. I.2.4-9



GEOLOGICAL PROFILE ALONG Z-LINE

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

Fig. I.2.5–1



PARTICLE SIZE DISTRIBUTION CHART, S/R-1

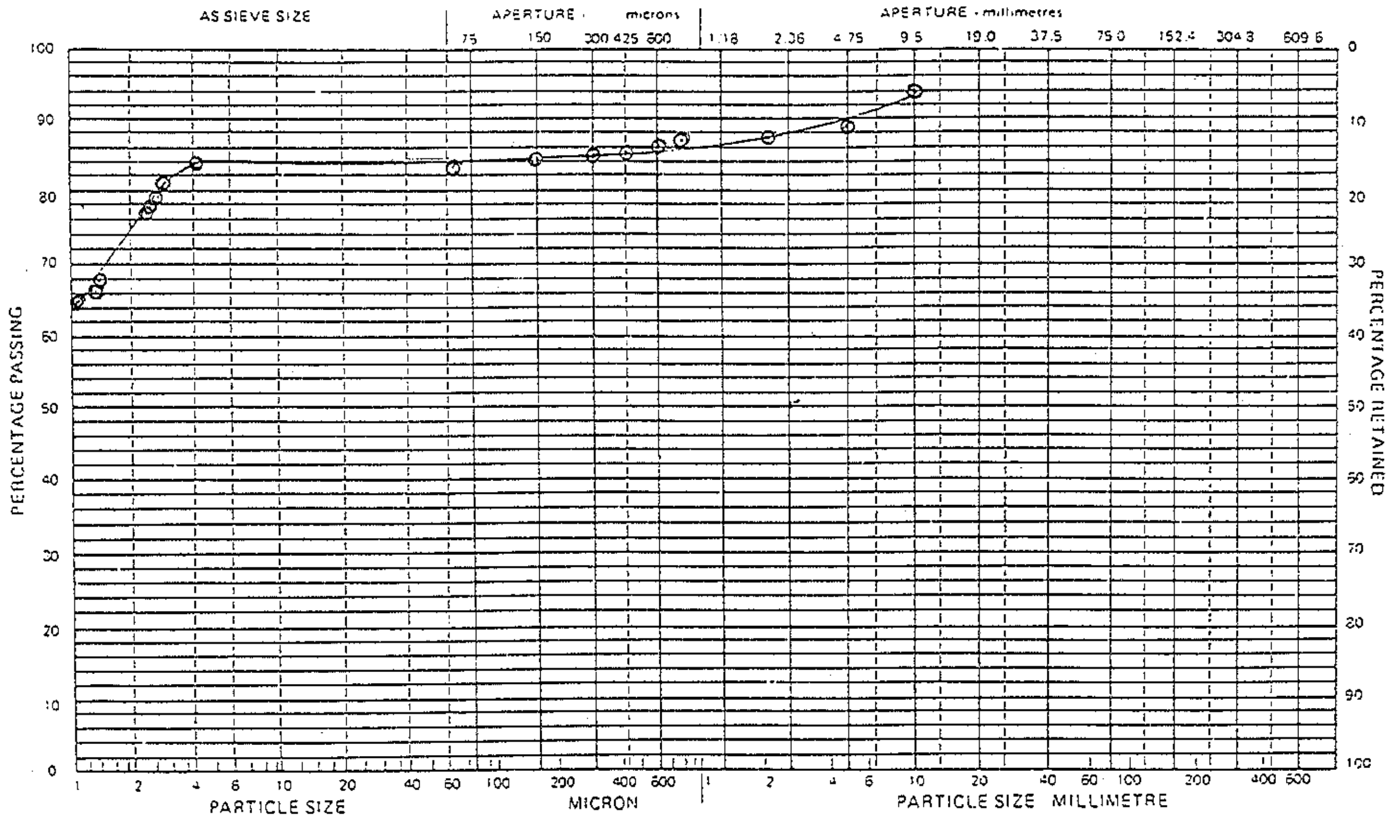
GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.5-2

Tested by.. .... Plotted by. .... Checked by.............. SAMPLE NO. ..

Date...... .... Date........................ Date...................... REGISTRATION ....



CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

D_{10}............... D_{30}............... D_{60}.............. C_u............. C_c............ Classification ....

Description of material ....

PARTICLE SIZE DISTRIBUTION CHART, S/R-2

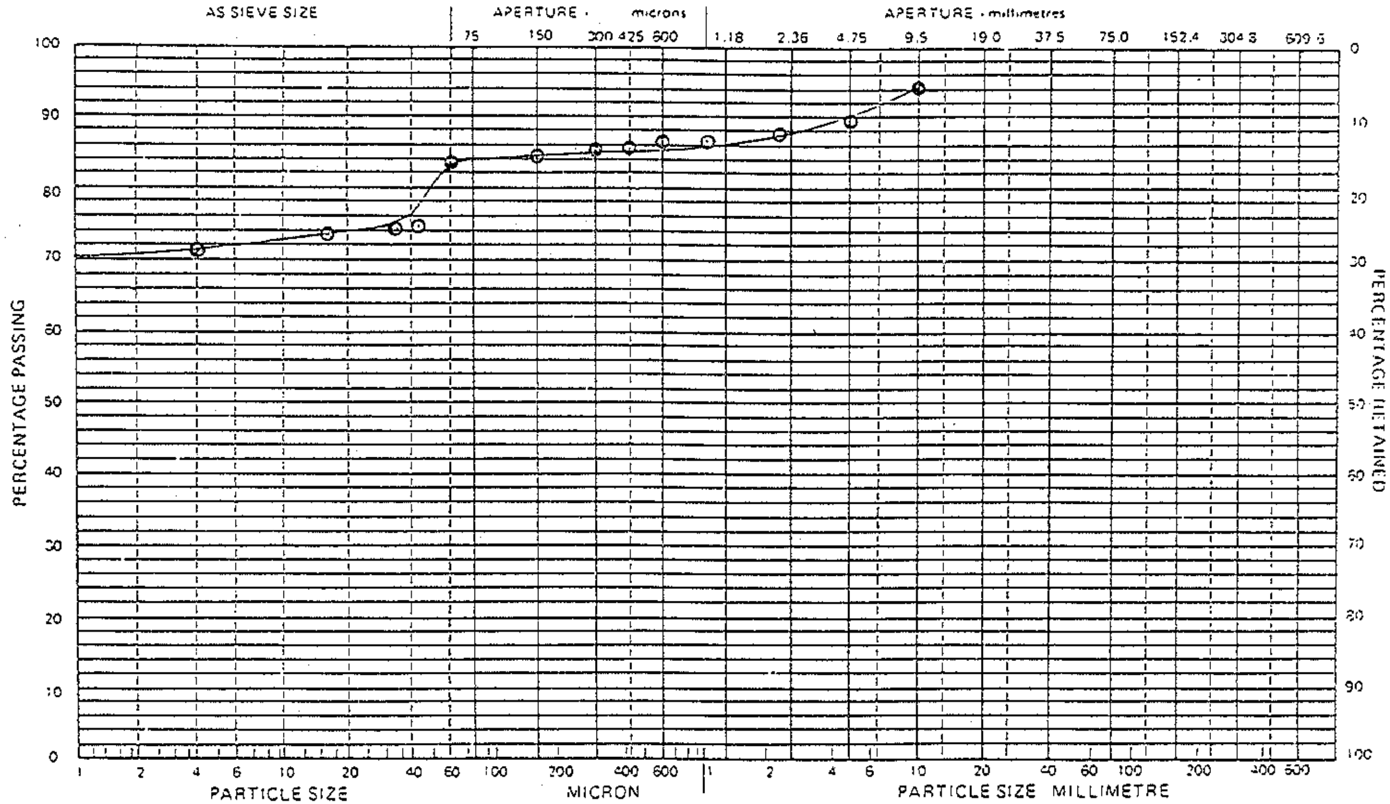
GOVERNMENT OF MAURITIUS
LANDSLIDE PROTECTION PROJECT IN PORT LOUIS

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. I.2.5-3

Tested by.................. Plotted by.................. Checked by.................. SAMPLE NO. ...

Date. Jan. 90 .... Date.................. Date.................. REGISTRATION ....



CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

D10.............. D30.............. D60.............. Cu.............. Cc.............. Classification ....

Description of material ....

Form No. 6/127 (10/74)   Report No. .... Sheet ....

PARTICLE SIZE DISTRIBUTION CHART, D/W-1

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