THE GOVERNMENT OF MAURITIUS
MINISTRY OF ENERGY, WATER RESOURCES AND POSTAL SERVICES
CENTRAL WATER AUTHORITY

THE DETAILED DESIGN ON THE PORT LOUIS WATER SUPPLY PROJECT IN MAURITIUS

FINAL REPORT (2)

DESIGN REPORT

APPENDIX
FIELD INVESTGATION REPORT AND PROBABLE FLOOD

FOR

LOT II : CIVIL WORKS(DAM AND APPURTENANT STRUCTURES INCLUDING CLOSURES OF DIVERSION TUNNEL)

MARCH 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

THE GOVERNMENT OF MAURITIUS
MINISTRY OF ENERGY, WATER RESOURCES AND POSTAL SERVICES
CENTRAL WATER AUTHORITY

THE DETAILED DESIGN ON THE PORT LOUIS WATER SUPPLY PROJECT IN MAURITIUS

FINAL REPORT (2)

DESIGN REPORT
APPENDIX
FIELD INVESTGATION REPORT
AND
PROBABLE FLOOD

FOR

LOT II : CIVIL WORKS(DAM AND APPURTENANT STRUCTURES INCLUDING CLOSURES OF DIVERSION TUNNEL)

MARCH 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

INVESTIGATIONS
FOR
CONSTRUCTION MATERIALS

CONTENTS

		Page
A1. INTRODUCTION		A-1
A2. EMBANKMENT MATERIALS		A-2
A2.1	Outline of Investigation	A-2
	A2.1.1 Field reconnaissance and test pitting	A-2
	A2.1.2 Laboratory test	A-2
A2.2	Field Reconnaissance and Test Pitting	A-3
	A2.2.1 Location of borrow and quarry sites	A-3
	A2.2.2 Test pitting	A-4
	A2.2.3 Available quantity	A-5
A2.3	Laboratory Tests for Earth Material	A-6
	A2.3.1 Purpose	A-6
	A2.3.2 Index properties	A-6
	A2.3.3 Engineering properties	A-7
A2.4	Laboratory Tests for Filter material	A-9
	A2.4.1 Purpose	A-9
	A2.4.2 Index properties	A-9
A2.5	Laboratory Tests for Rock Material	A-10
	A2.5.1 Purpose	A-10
	A2.5.2 Index properties	A-10
	A2.5.3 Engineering properties	A-11

		Page
A3.	CONCRETE	A-13
A3.1	Method of Investigation	A-13
A3.1.1	Tests for aggregates	A-13
A3.1.2	Tests for mix proportion of concrete	A-13
A3.2	Tests for Aggregates	A-13
A3.2.1	Purpose	A-13
A3.2.2	Test results	A-14
A3.3	Tests for Mix Proportion of Concrete	A-15
A3.3.1	Purpose	A-15
A3.3.2	Classes of concrete specified	A-15
A3.3.3	Concrete materials	A-16
A3.3.4	Design method of mix proportion of concrete	A-18
A3.3.5	Mix proportion of concrete for trial mixes	A-18
A3.3.6	Tests for selection of fine aggregate percentage	A-25
A3.3.7	Final mixes	A-25
A4.	CONCLUSION AND RECOMMENDATIONS	A-27
A4.1	Sources of Construction Materials	A-27
A4.2	Properties of Embankment Materials	A-28
A4.2.1	Earth material	A-28
A4.2.2	Filter material	A-29
A4.2.3	Rock material	A-31
A4.3	Properties of Concrete Aggregates	A-31
A4.4	Design Values of Embankment Materials	A-31
A4.5	Determination of Specified Concrete Mixes	A-32

A1. INTRODUCTION

Dam of the Port Louis Water Supply Project was planned to be located at downstream of the confluence of the Terre Rouge and Profonde rivers, about 6 to 8 km southwest of Port Louis city as shown on the Fig.A1-1.

According to the feasibility study performed in 1989, dam was planned as rockfill type dam. The dam with about 80 m in height require about 1,600,000 m^3 in total embankment volume. Spillway was planned as side channel type. The spillway with 90 m in overflow crest length require about 53,000 m^3 in total concrete volume.

The construction material investigation for this detailed design stage was carried out from May to October in 1990. The investigation aimed for clarifying physical properties, obtainable quantities, design values for embankment materials, i.e. earth, filter and rock, and determining specified concrete mixes for concrete structures in the Project.

This report presents results of the above construction material investigation for the Port Louis Water Supply Project.

A2. EMBANKMENT MATERIALS

A2.1 Outline of Investigation

The embankment material investigation for the Project consists of the field reconnaissance and test pitting, and the laboratory test.

The outline of each work is described hereunder.

A2.1.1 Field reconnaissance and test pitting

The field reconnaissance and test pitting were as follows:

(1) The field reconnaissance aimed to reconfirm the borrow/quarry areas for the embankment materials such as impervious core material, filter and rock materials.

(2) The test pitting was made at four (4) points in the proposed borrow area. All the test pits were 4 m in depth. It aimed to reconfirm the thickness of residual soil layer for impervious core material and to take samples for laboratory tests.

(3) Sampling of rock material for large scale direct shear test, filter material and concrete aggregates was carried out by blasting of fresh boulders at the proposed quarry site.

(4) Samples of rock material for laboratory test were taken from the boring cores at the proposed quarry site.

A2.1.2 Laboratory test

The laboratory tests were performed in accordance with the approved and standardized testing method such as ASTM (American Society for Testing and Materials), BS (British Standard) and Earth Manual of USBR, etc.

The laboratory test items for each material were as follows:

(1) For earth material, index property tests, i.e. specific gravity, grain-size analysis, consistency, moisture content and soluble salts determination tests, and engineering property tests, i.e. compaction, permeability, consolidation tests and triaxial shear (CU) test with measurement of pore-water pressure, were carried out to clarify the soil material index and to prepare the design values for the detailed design.

(2) For filter material, laboratory tests, i.e. specific gravity & absorption, soundness and abrasion tests, were carried out to clarify the availability for filter material.

(3) For rock material, laboratory tests, i.e. specific gravity & absorption, compressive strength and large scale direct shear tests, were carried out to confirm the rock properties and to prepare the design values for the detailed design.

A2.2 Field Reconnaissance and Test Pitting

A2.2.1 Location of borrow and quarry sites

In this project, the dam was planned to be constructed on Terre Rouge river, just below its confluence with Profonde river based on the feasibility study.

The investigation sites were selected based on the geographical and geological information. The location map for the potential sources of construction materials is shown on the Fig. A2-1.

(1) Borrow area

A brief reconnaissance for the earth material was carried out in the borrow area around the dam site. And more precise investigation followed for determining the location of the test pits for the earth material.

Finally the borrow area of the earth material for the dam was proposed to be located on the right bank of Terre Rouge river about 0.5 to 1.5 km upstream from proposed dam site, having sufficient amount of residual soil. Present condition of the borrow area is sugar cane field which is private land. After completion of construction of the impervious core zone on the construction stage, the land must be returned to the land owner on condition that it can be used for sugar cane field again. So the borrow area requires rather wide area, because top soil stripped for taking the earth material will be stored on the borrow area during construction of the dam and filled for plantation of sugar cane after the construction of the dam.

(2) Quarry site

The quarry site for filter, rock materials and concrete aggregates, etc. was selected at Mt.Ory on the feasibility study, which is located at about 1.0 km north of the dam site and is composed of very hard basaltic lava. Precise reconnaissance was carried out at three (3) alternative quarry sites on the basic design stage, taking into consideration the rock quality, available volume, environmental effect, necessary compensation, convenience and cost for hauling the material and so on.

Following three sites were found to be conceivable quarry sites for the project.

Quarry site plan (A)	:	The site is located at the northern part of Mt.Ory, where the altitude of the ridge of Mt.Ory is low.
Quarry site plan (B)	:	The site is located at the southern part of Mt.Ory. The motorway (Trunk Road of Mauritius) passes near the site (about 300 to 400 m away).
Quarry site plan (C)	:	This site is situated on the mountain slope of the Guibies Peak which is located at about 8.0 km northeast from the dam site.

As the quarry site for filter and rock materials, the Quarry site plan (B) was recommended. Main reasons are better quality of rock and shorter length of haul road than others.

A2.2.2 Test pitting

Four (4) test pits were dug to reconfirm the thickness of residual soil layer and to take samples for the laboratory test. These places are selected for test pitting in the proposed borrow area for the earth material as shown on Fig. A2-2.

A2.2.3 Available quantity

(1) Earth material

From the observation of test pits in the feasibility study and the geotechnical investigation in the detailed design, the soil profiles in the borrow area are described as follows:

Top soil	:	1 m in thickness, dark brown and reddish brown in color, silty clay and clayey silt of high plasticity, containing organic matter such as plant roots.
Residual soil	:	More than 3 to 4 m in thickness, reddish, grayish and dark brown in color, silty clay and clayey silt of high plasticity.

The groundwater level at the borrow area seems to be lower than the bottom of excavation in the construction stage.

The predominant stratum for the earth material in the borrow area can be expected to consist mainly of a residual soil originated from in situ weathered basalt, clayey silt of which thickness is more than 3 to 4 m. The residual soil overlain by the top soil with 1 m in thickness will be exclusively used for the impervious core zone.

The borrow area is roughly estimated at 420,000 m^2, and it is partly used for stockpile of the stripped top soil and for moisture adjustment of the earth material, etc. So the area purely available for the purpose of the borrow area is about 200,000 m^2. And thickness of the soil for the earth material at the borrow area is only about 2 m except top soil. The soil below this level shall not be removed in consideration of plantation in the borrow area after the construction of the dam. The available quantity of the earth material at the borrow area is approximately estimated at 400,000 m^3. The required quantity for the embankment of impervious core zone is estimated at about 270,000 m^3 in original ground condition. The available quantity meets requirement of the impervious core material of the dam.

(2) Concrete aggregates, filter and rock materials

Concrete aggregates, filter and rock materials will be exploited from depths of more than about 10 m below the ground surface after excavation of residual soil and weathered rock. It is hard and fresh rock of basalt and agglomerate, and it will be considered to be used for concrete aggregates, filter material and rock material.

The amount of required concrete aggregates, filter material and rock material are estimated at about 40,000 m^3, 100,000 m^3 and 1,150,000 m^3, respectively. To meet this requirement and other requirements such as metaling on haul road, the required amount of the rock material will be about 1,850,000 m^3. Therefore it will be necessary to develop the quarry site of which quantity is about 1,430,000 m^3 in consideration of bulking factor 1.3. Available quantity of rock materials at the quarry site is more than 1,430,000 m^3.

A2.3 Laboratory Tests for Earth Material

A2.3.1 Purpose

The purpose of laboratory tests for the earth material is to reconfirm and evaluate the precise engineering properties of the earth material. The results of the laboratory test will provide geotechnical information for the tender documents to be prepared in the detailed design.

The laboratory test items are shown in Table A2-1.

A2.3.2 Index properties

The test values of each sample are described in Table A2-2. The test results of index properties are summarized in Table A2-3.

The earth material will consist of reddish, grayish and dark brown residual soils, silty clay and clayey silt.

Specific gravity is 2.882 in average in the range from 2.862 to 2.900. These values are relatively high for the material originated from basalt and agglomerate.

Grain-size analysis of the earth material was carried out by sieve and hydrometer analyses. The gradation curves are shown on Fig. A2-3. Particle size of the earth material is very fine. The content of silt and clay is 91 % in average.

Liquid limit is 68.7 % in average in the range from 62.8 to 75.6 %, plastic limit is 47.7 % in average, ranging from 40.7 to 59.7 %. Plasticity index is 21.0 in average in the range from 15.9 to 24.5. Plasticity chart is shown on Fig. A2-4. The material is classified into MH in the unified soil classification. The earth material is silty soil with high plasticity.

Soluble salt is 175 parts/million in average in the range from 100 to 250 parts/million. The earth material is non-dispersive soil because of low soluble salts values. According to the study of J.L.Sherard, L.P.Dunnigan, and R.S.Decker in 1977, *"Some Engineering Problems with Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM STP 623"*, all tested fine-grained soils known to be derived from in situ weathering of igneous and metamorphic rocks are non-dispersive as well as all soils derived from limestone. The earth material is in situ weathered basalt and basalt is igneous rock. So the earth material will be non-dispersive soil.

A2.3.3 Engineering properties

Compaction properties, permeability, consolidation and triaxial shear strength tests were carried out for earth material.

(1) Compaction properties

Compaction tests were conducted with the compaction energy of 1 Ec (standard proctor energy). Standard compaction tests for maximum dry density and optimum moisture content of the earth material were carried out on 4 samples of which results are shown on Fig. A2-5. All the samples were prepared by carefully drying or wetting the soils with the natural moisture content to obtain the moisture content required for each point on the compaction curve.

As shown in Table A2-2, maximum dry density (γ_{dmax}) is 1.293 tf/m^3 in average in the range from 1.278 to 1.310 tf/m^3, optimum moisture content (OMC) is 40.9 % in average in the range from 40.5 to 42.0 %.

(2) Permeability

Permeability tests were carried out on 8 samples of remolded material. The conventional falling head method was employed on all specimen which were prepared to have the density of 95 % of γ_{dmax} under 1 Ec (maximum dry density of 1 Ec) with the moisture contents of dry and wet sides on the compaction curve in consideration that the quality control is usually so made that the dry density be more than 95 % of the maximum dry density. The following results were obtained from the laboratory test.

The results are shown in Table A2-2 and A2-4. The coefficient of permeability ranges from 4.8×10^{-7} to 5.9×10^{-6} cm/sec when the moisture contents of specimens were in dry side, but it was efficiently small in the range from 3.7×10^{-8} to 6.0×10^{-7} cm/sec when the moisture contents were in wet side. The coefficient of permeability was mostly less than 1×10^{-6} cm/sec, indicating the material had low permeability, being well suited for use in the impervious core zone.

(3) Consolidation

Four (4) samples were subject to confined one-dimensional consolidation tests. The tests were performed on the samples compacted to have the density of $\gamma_{dmax} \times 95$ % with moisture contents of wet side on the compaction curve after saturated. Specimens were subject to seven (7) load increments in the stress range from 0 to 16.3 kgf/cm^2.

The relation between consolidation pressure and void ratio of the earth material is shown on Fig. A2-6. Relations between consolidation pressure and coefficient of consolidation, and coefficient of volume change are shown on Fig. A2-7, A2-8.

The e - log p curves (Fig. A2-6) of compacted materials show low gradients for fine graded material and variation of them is small. So settlement of core zone in the embankment will not be large and differential settlement may not occur. And the coefficient of consolidation (c_v) which is an indicator to show the speed of consolidation is large, about 300 to 400 cm^2/day for fine materials. Settlement of embanked body of core zone will be almost completed during construction.

(4) Triaxial shear strength

Four (4) samples were subject to triaxial shear tests for CU test (consolidated undrained test with pore-water pressure measurement). The specimens for triaxial shear tests were prepared to have the density of $\gamma_{dmax} \times 95$ % under 1 Ec with the moisture contents of wet side on the compaction curve.

The results are shown in Table A2-2 and Mohr's circles of the samples are shown on Fig. A2-9. Cohesion (c') of CU (consolidated-undrained with pore-water pressure measurement) under the effective stress analysis is 0.05 kgf/cm^2 in average in the range from 0.00 to 0.08 kgf/cm^2 and internal friction angle (ϕ') under the effective stress analysis is 33 degrees on average, ranging from 33 to 34 degrees.

A2.4 Laboratory Tests for Filter Material

A2.4.1 Purpose

The purpose of laboratory tests for the filter material is to confirm and evaluate the precise engineering properties of the filter material used for the proposed rockfill dam. Precise index properties of the material which is the same material as concrete aggregates are described in section of tests for aggregates. The results of the laboratory test will provide geotechnical information for the tender documents to be prepared in the detailed design.

The laboratory test items are shown in Table A2-1.

A2.4.2 Index properties

The results of the tests are shown in Table A2-5. Specific gravity of the material on oven dry condition is 2.891 in average, ranging from 2.871 to 2.903. Specific gravity of the material on saturated surface dry condition is 2.913 in average in the range from 2.898 to 2.922. And absorption is 0.75 % in average ranging from 0.65 to 0.93 %.

Grain size of the material is very coarse as shown on Fig. A2-10. Uniformity coefficient of the material is relatively small, about 2.4. Crushing plant for the filter material will be required to make suitable filter materials.

Loss in weight of soundness test by use of sodium sulfate is 2.3 % in average in the range from 1.6 to 3.1 %. The results are sufficiently small.

Loss in weight of abrasion in the Los Angeles machine is 16.5 % in average, ranging from 15.9 to 16.9 %. The results are sufficiently small for filter material.

A2.5 Laboratory Tests for Rock Material

A2.5.1 Purpose

The purpose of laboratory tests for the rock material is to confirm and evaluate the precise index and engineering properties of the rock material used for the proposed rockfill dam. The results of the laboratory test will provide geotechnical information for the tender documents to be prepared in the detailed design.

The laboratory test items are specific gravity, absorption and compressive strength tests for specimens taken by core boring and large scale direct shear test for the blasted boulders of fresh rock.

The laboratory test items are shown in Table A2-1.

A2.5.2 Index properties

The results of the tests are shown in Table A2-6. Specific gravity of the rock material is 2.585 in average, ranging from 2.241 to 2.932. And absorption is 5.6 % in average in the range from 1.8 to 10.6 %. Especially, specific gravity and absorption of the fresh material deeper than 10 m from ground level are 2.834 in average ranging from 2.709 to 2.932, and 2.0 % in average in the range from 1.8 to 2.4 %, respectively.

Compressive strength of the rock material is 691 kgf/cm^2 in average ranging from 168 to 1408 kgf/cm^2. But compressive strength of the material deeper than 10 m from ground level is 1081 kgf/cm^2 ranging from 855 to 1408 kgf/cm^2.

A2.5.3 Engineering properties

Large scale direct shear test was performed for the purpose to determine design strength parameters of rock zone of the rockfill dam of the Project.

(1) Test conditions

The test was carried out by use of the blasted material less than 100 mm taken from the quarry site. The test was performed under the following conditions.

Size of specimen	;	1.25 m × 1.25 m × 0.75 m
Max. size of material	;	100 mm
Shear velocity	;	5 mm/min. (0.4 %/min.)
Confining pressure	;	0.98, 1.94, 3.21 kgf/cm^2
Drain condition	;	Consolidated-Drained condition (CD)
Material condition	;	Natural moisture content

Other testing conditions are described below.

(a) Material

The material used in the test was obtained at the quarry site. The material was taken by blasting of boulders of fresh rock.

(b) Gradation for the Test

Maximum grain size of the tested material is determined by smaller dimension between one tenth of specimen size on shear direction and one fifth of specimen height. Then, the tested material was under 100 mm. Grain-size distribution for the test was determined by excepting particles larger than required maximum size on the test: that is, the material was sifted by 100 mm sieve on the field. The gradation curve of the material is shown on Fig. A2-11. As seen, the gradation is not favorable, because fine material such as less than 10 mm scattered by blasting was not able to be taken. Uniformity coefficient of the material was 2.4. But at construction stage, blasting method will be the bench cut method, and fine material will also be included. So gradation of rock material at the construction stage will be better than the tested material, and uniformity coefficient of rock material will be larger than the tested material.

(c) Density for the Test

The field densities of actual dams in Japan range from 1.6 to 2.3 tf/m^3 and almost all data of densities cover more than 1.9 tf/m^3. And data of the field densities concentrate at about 2.1 tf/m^3. The densities of large scale direct shear tests for actual dams at design stage and construction stage in Japan range from 1.3 to 2.3 tf/m^3 and the densities mostly range from 1.5 to 2.1 tf/m^3. The density of the tested material was 1.625 tf/m^3. The value is relatively small because of the material compacted by man power and low uniformity coefficient.

(d) Testing apparatus

The testing apparatus is shown on Fig. A2-12. Dimensions of the testing apparatus are about 605 cm in length, 200 cm in width and 324 cm in height.

(2) Results of the test

Relations between strain and volume change, stress are shown on Fig. A2-13. Relation between vertical stress and shear stress is shown on Fig. A2-14.

The specimen is of low density, and dilatancy of the specimen shows negative values during shear. This is due to that the specimen was compacted by manpower, and compaction energy of the specimen was considered insufficient. The test results of actual dams under condition of low vertical stress indicate positive values. Density of the material compacted by heavy equipments and use of well graded material in field will be as large as 2.1 tf/m^3.

Cohesion (c_d) and internal friction angle (ϕ_d) of the tested material were 0.14 kgf/cm^2 and 42 degrees, respectively. Internal friction angle of the rock zone in field will be larger because heavy equipment is used for compaction.

A3. CONCRETE

A3.1 Method of Investigation

The concrete investigation for the Project consists of the tests for aggregates and mix proportion of concrete. The laboratory tests were performed in accordance with the approved and standardized testing method such as ASTM (American Society for Testing and Materials), BS (British Standard), etc. The laboratory tests were carried out in the Laboratory of University of Mauritius, School of Industrial Technology.

A3.1.1 Tests for aggregates

Laboratory tests, i.e. specific gravity & absorption, grain-size analysis, unit weight, soundness, abrasion and alkali aggregate reaction tests, were carried out to clarify the availability for fine and coarse aggregates of concrete.

Tested materials were obtained by blasting fresh boulders in the proposed quarry site and prior to commencement of the tests, those were screened by sieve of 100 mm.

A3.1.2 Tests for mix proportion of concrete

The tests for mix proportion of concrete were composed of concrete material tests, tests for selection of fine aggregate percentage (s/a), fresh concrete tests and concrete strength tests. Table A3-1 shows executed test items.

A3.2 Tests for Aggregates

A3.2.1 Purpose

The purpose of tests for aggregates is to confirm and evaluate the precise engineering properties of concrete aggregates. The results of the tests will provide informations for the tender documents to be prepared in the detailed design.

A3.2.2 Test results

The test results of aggregates are summarized in Table A3-2. The detailed test results of each sample are shown in Table A3-3.

Material to be used for concrete aggregates from the proposed quarry site is covered with top soil and thick weathered rock about 10 m in thickness. But the material below top soil and weathered rock is fresh and hard rock originated from basalt and agglomerate. The tested material for aggregates is typical fresh and hard basalt.

Specific gravity of the material on oven dry condition is 2.891 in average, ranging from 2.871 to 2.903. And specific gravity of the material on saturated surface dry condition is 2.913 in average in the range from 2.898 to 2.922. Specific gravity of coarse aggregates is generally more than 2.5 which is sufficiently high for concrete aggregates. Absorption is 0.75 % in average ranging from 0.65 to 0.93 %, which indicates that the material is favorably fresh in consideration of much less value than the allowable absorption of 3 %.

Grain sizes of the material all belong to gravel particles as shown on Fig. A3-1. Uniformity coefficient of the material is relatively small, about 2.4, showing the material is badly graded. Small bulk density values and large voids are also shown due to badly graded material as follows:
Bulk density of the material is 1463 kgf/m^3 in average in the range from 1428 to 1494 kgf/m^3. Voids of the material are 49.4 % in average, ranging from 48.3 to 50.3 %. The above results from insufficient crushing of the material due to no availability of the crushing plant in the investigation stage, implying that this problem is soluble.

Loss in weight of soundness test by use of sodium sulfate is 2.3 % in average in the range from 1.6 to 3.1 %. Limit of loss in the soundness test is generally 12 %. The results are sufficiently small.

Loss in weight of abrasion in the Los Angeles machine is 16.5 % in average, ranging from 15.9 to 16.9 %. Limit of loss in the abrasion test is generally 40 %. The results are sufficiently small for concrete aggregates.

Results of the alkali aggregate reaction by chemical method show 33 millimoles/liter in dissolved silica and 82 millimoles/liter in reduction in alkalinity as shown on Fig. A3-2, resulting in the innocuous range.

A3.3 Tests for Mix Proportion of Concrete

A3.3.1 Purpose

The purpose of tests for mix proportion of concrete is to decide the specified concrete mixes of several concrete classes used for concrete structures in the Project. The results of the tests will provide informations for the tender documents to be prepared in the detailed design.

A3.3.2 Classes of concrete specified

The concrete specified in the Project is classified into 6 classes, class A to class F, as shown in the Table A3-4.

Class A is used for slab, block out, pneumatically applied concrete and building, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class A are 20 mm, 210 kgf/cm^2 and 14 to 18 cm, respectively.

Class B is used for drain ditch, catch basin, drain inlet & outlet and precast concrete, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class B are 20 mm, 180 kgf/cm^2 and 12 to 14 cm, respectively.

Class C is used for reinforced concrete, tunnel lining and outside concrete of spillway, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class C are 40 mm, 180 kgf/cm^2 and 12 to 14 cm, respectively.

Class D is used for backfill, levelling and gravity wall, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class D are 40 mm, 140 kgf/cm^2 and 8 to 10 cm, respectively.

Class E is used for inside concrete of spillway, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class E are 80 mm, 120 kgf/cm^2 and 8 to 10 cm, respectively.

Class F is used for levelling concrete, etc. Maximum grain size of aggregates, required compressive strength and range of slump of class F are 40 mm, 100 kgf/cm^2 and 8 to 10 cm, respectively.

A3.3.3 Concrete materials

(1) Cement

Cement used in the concrete tests is the ordinary portland cement. The ordinary portland cement for the tests was obtained from Port Louis city. The selected ordinary portland cement is "Marine Cement" supplied by Mauritius Portland Cement Co., Ltd., Port Louis, Mauritius. The cement is imported from Kenya. Marine Cement only is the ordinary portland cement available in Mauritius.

Properties of the cement are shown in Table A3-5.

Specific gravity of the cement is relatively small compared with the usual ordinary portland cement. Initial and final setting times are reasonable values and soundness of the cement is good. The strength of cement is sufficient. Quality of the cement seems to be similar to high-early-strength portland cement.

(2) Water

Water was used one of quality similar to water obtained from Grand River North West river in the concrete tests.

(3) Aggregate

As the crushing plant was not available, the concrete aggregates produced in Mauritius were used for the tests for mix proportion of concrete in view that the above concrete aggregates were produced from the same basalts as those to be obtained from the proposed quarry site.

Grain size distribution of the aggregates in the concrete tests is shown on Fig. A3-3. Gradation of aggregates is summarized as follows:

Fine aggregate

Grain size (mm)	Percentage (%)
0.15 under	12
0.15 to 1.2	58
1.2 to 5.0	30

Coarse aggregate

Max. size (mm)	Grain size (mm)	Percentage (%)
20	5 to 10	37
	10 to 20	63
40	5 to 10	20
	10 to 20	32
	20 to 40	48
80	5 to 10	12
	10 to 20	22
	20 to 40	23
	40 to 80	33

Test results of the aggregates for the concrete tests are summarized in Table A3-6. Its details are shown in Table A3-7.

The results show that specific gravity and absorption of the aggregates are of slightly low quality compared with those obtained from the proposed quarry site described in Section A3.2. In view that both the materials are obtained from the same kind of basalts in Mauritius, the above difference is considered to be caused due to a difference in the degree of weathering (or freshness).

As mentioned, the tests for mix proportion of concrete are made by using the aggregates with a quality slightly lower than that from the proposed quarry site. However, the design of concrete mix proportion based on the above is considered justifiable in view that the design be made in a safety side to cope with possible variations of quality of the aggregates to be extracted from the proposed quarry site.

(4) Admixture

Admixture was used as water reducing agent, set-retarding agent and associated air entrained agent. The admixture used was MF603 which is prevailing in Mauritius. MF603 is produced by Multi Construction Chemicals (Pty) Ltd. Quantity of the admixture was 250 ml against cement weight 100 kg.

A3.3.4 Design method of mix proportion of concrete

Design of mix proportion of concrete is determined through following procedure.

Firstly, mix proportion of concrete is estimated for trial mixes by empirical values, etc. The trial mixes are tests for determining the optimum fine aggregate percentage. The tests for various fine aggregate percentages are carried out under the afore-mentioned mix proportion.

After that, the mix proportion of concrete is adjusted for final mixes based on the test results of fine aggregate percentage. The final mixes are the tests for determining the final design of concrete mix proportion, consisting of the fresh concrete tests and concrete strength tests on the mix proportion of concrete adjusted as mentioned above.

The final design of mix proportion of concrete is determined to satisfy the requirements based on relation between σ_{28} and C/W, which is cement water ratio, to be obtained from the results of concrete strength test.

A3.3.5 Mix proportion of concrete for trial mixes

Mix proportion of concrete was estimated for the trial mixes based on empirical values, etc. as follows:

(1) Determination of water cement ratio

Water cement ratio for obtaining the required strength was determined based on following procedure.

Target strength which is the strength required in laboratory test was taken at 1.19 times of the required strength in design in consideration of coefficient of variation of strength. The coefficient of variation of strength was assumed to be 20 % in consideration of usual quality control in field.

The determination of water cement ratio is referred to 2 methods in accordance with the JSCE specification which is the Japan Society of Civil Engineers specification. One is the method to determine based on an equation between σ_{28} and C/W. Another is the method to determine based on empirical value of water cement ratio for the required durability and watertightness.

The equation between σ_{28} and C/W is empirically given as follows:

$$\sigma_{28} = -195 + 300\ C/W$$

(the Japan Cement Association)

The following table shows W/C based on the empirical equation between σ_{28} and C/W, W/C based on the empirical values for the required durability and watertightness, and the selected W/C for the trial mixes. The smaller W/C value out of two methods is taken for W/C for the trial mixes.

Class of concrete	W/C by empirical equation (%)	W/C by empirical values for the required durability & watertightness (%)	Selected W/C (%)
A	67	60	60
B	73	65	65
C	73	55	55
D	83	65	65
E	89	65	65
F	96		75

W/C of class F was taken at 75 % which is the upper limit to avoid segregation of concrete materials and depression of concrete quality.

(2) Determination of air content

Air content was determined based on the standard relation between maximum grain size of aggregate, admixture and air content. Determined air content, which is the standard value for concrete with admixture which had air entrained agent, is as follows:

Class of concrete	Selected air content (%)
A	6.0 ± 1.0
B	6.0 ± 1.0
C	4.5 ± 1.0
D	4.5 ± 1.0
E	3.5 ± 1.0
F	4.5 ± 1.0

(3) Estimation of fine aggregate percentage

As mentioned, the trial mixes are the tests to determine the optimum fine aggregate percentage which is found through the tests to be carried out by changing the fine aggregate percentage.

Initial fine aggregate percentage for the trial mixes is determined on the basis of the standard given in JSCE specification as follows:

This method consists of two steps as follows.

<u>Step 1</u>

For concrete with and without admixture, initial values of fine aggregate percentage are determined with regard to various maximum sizes of aggregate from the table below.

Maximum size of aggregate (mm)	Fine aggregate percentage (%)		
	Concrete without admixture	Concrete with high-quality air-entraining agent	Concrete with high-quality water-reducing and air-entraining agent
15	49	46	47
20	45	42	43
25	41	37	38
40	36	33	34
50	33	30	31
80	31	28	29

Fine aggregate percentages in the table above are of concrete to have qualities and materials as follows:

(a) Sand with finess modulus of about 2.80,
(b) gravel of ordinary gradation as aggregate,
(c) water cement ratio of about 0.55, and
(d) slump of about 8 cm.

Step 2

Fine aggregate percentages derived at Step 1 are corrected according to manners shown on the table below, when materials to be used and/or quality of concrete differ from the conditions of Step 1.

No.	Item of difference	Manner of correction of fine aggregate percentage
1	Finess modulus of sand	Increase (decrease) by 0.5 for increase (decrease) of every 0.1in finess modulus of sand
2	Slump	No correction required
3	Air content	Decrease (increase) by 0.5 to 1.0 % for increase (decrease) of every 1.0 % in air content
4	Water cement ratio	Increase (decrease) by 1.0 % for increase (decrease) of every 0.05 of water cement ratio
5	Use of crushed stone	Increase by 3 to 5 % for use of crushed stone
6	Use of crushed sand	Increase by 2 to 3 % for use of crushed sand

Note : When crushed sand contains a quite large amount of powder, fine aggregate percentage is to be decreased by 1 to 2 %. Here, powder is defined as particles passing a 0.15 mm sieve.

The initial fine aggregate percentages determined through Step 1 and Step 2 are as follows:

Class of concrete	Selected fine aggregate percentage (%)
A	49
B	50
C	39
D	41
E	36
F	43

(4) Estimation of water content

Water content for the trial mixes is also determined on the basis of the standard value given in JSCE specification as follows:

This method consists of two steps as follows.

Step 1

For concrete with and without admixture, initial values of unit water content are determined with regard to various maximum sizes of aggregate from the table below.

Maximum size of aggregate (mm)	Unit water content (kgf/m^3) Concrete without admixture	Concrete with high-quality air-entraining agent	Concrete with high-quality water-reducing and air-entraining agent
15	190	170	160
20	185	165	155
25	175	155	145
40	165	145	135
50	155	135	125
80	140	120	110

Unit water contents in the table above are of concrete to have qualities and materials as follows:

(a) Sand with finess modulus of about 2.80,
(b) gravel of ordinary gradation as aggregate,
(c) water cement ratio of about 0.55, and
(d) slump of about 8 cm.

Step 2

Unit water contents derived at Step 1 are corrected according to manners shown on the table below, when materials to be used and/or quality of concrete differ from the conditions of Step 1.

No.	Item of difference	Manner of correction of unit water content
1	Finess modulus of sand	No correction required
2	Slump	Increase (decrease) by 1.2 % for increase (decrease) of every 1 cm in slump
3	Air content	Decrease (increase) by 3.0 % for increase (decrease) of every 1.0 % in air content
4	Water cement ratio	No correction required
5	Fine aggregate percentage	Increase (decrease) by 1.5 kgf/m^3 for increase (decrease) of every 1.0 % in fine aggregate percentage
6	Use of crushed stone	Increase by 9 to 15 kgf for use of crushed stone
7	Use of crushed sand	Increase by 6 to 9 kgf for use of crushed sand

The unit water contents determined through Step 1 and Step 2 are as follows:

Class of concrete	Selected water requirement (kgf/m^3)
A	190
B	184
C	163
D	157
E	131
F	157

(5) Mix proportion for trial mixes

Table A3-8 shows mix proportion of concrete determined for the trial mixes as explained. This design is only for trial mixes which are tests for selection of fine aggregate percentage. It is necessary to adjust the design through trial mixes and final mixes.

A3.3.6 Tests for selection of fine aggregate percentage

In order to select the optimum fine aggregate percentage (i.e., fine aggregate percentage which gives the minimum quantity of water satisfying required workability), the following tests were conducted.

The tests were conducted for maximum grain size of the aggregates, 80 mm, 40 mm and 20 mm, respectively, which is the main factor for the fine aggregate percentage. So selected concrete classes for the tests were A, C and E. Required mixing quantity was 25 to 45 l per one batch for the trial mixes.

Under the quantity of water, cement and admixture were constant, trial mixes were performed by changing the fine aggregate percentage by 1 to 2 % measuring slump and workability of concrete. The tests were continued until the optimum fine aggregate percentage is obtained.

Optimum fine aggregate percentage is found as follows:

Class of concrete	Maximum grain size (mm)	Optimum fine aggregate percentage (%)
A	20	55
C	40	44
E	80	45

The results of the tests show relatively high values compared with the initially assumed fine aggregate percentage. The shape of aggregates is deemed to effect on the result.

A3.3.7 Final mixes

The mix proportion of concrete was adjusted for the final mixes on the basis of the test results of fine aggregate percentage. Adjusted mix proportion for the final mixes is shown in Table A3-9. The final mixes which are the tests to determine the final design of mix proportion through the fresh concrete and concrete strength tests to be carried out on the mix proportion adjusted by the tests for selection of fine aggregate percentage were performed. Required mixing quantity was 100 to 135 l per one batch for the final mixes.

The test results are as follows:

(1) Fresh concrete tests

Properties of fresh concrete for 6 classes are shown in Table A3-10.

Fine aggregate percentage determined in the trial mixing is judged suitable to obtain workable concrete. Water content to obtain desirable consistency of concrete is relatively much, because the aggregates are made by crushing.

Unit weight of concrete for the mix proportion is sufficiently large values.

Very small air content and large bleeding values are shown. The reason for the above is considered to be insufficient effect of the used admixture. Although the used admixture contained the water reducing agent, set retarding agent and associated air entrained agent. It will be necessary to check and examine on the admixture for concrete in the construction stage.

(2) Concrete strength tests

The results of concrete strength tests are shown in Table A3-11.

Compressive strength of concrete of all classes satisfies both the required compressive strength and the target compressive strength. Compressive strength cured for 7 days is ranging 63 to 100 % against one for 28 days. Tensile strength of concrete is more than 10 % of compressive strength except class D cured for 28 days.

A4. CONCLUSION AND RECOMMENDATIONS

Based on the field investigation and laboratory test results, quality, available quantity and effective utilization of construction materials such as core, filter, rock materials and concrete aggregates were studied. The conclusion and recommendation are mentioned hereinafter.

A4.1 Sources of Construction Materials

The proposed borrow area for the earth material of the dam is located on the right bank of Terre Rouge river about 0.5 to 1.5 km upstream from the dam site.

The proposed quarry site for the filter, rock materials and concrete aggregates is located at the southern part of Mt.Ory. The mortorway (Trunk Road of Mauritius) passes near the site (about 300 to 400 m apart).

Required quantities of the embankment materials on embanked condition are shown below.

Earth material	:	240,000	m^3
Filter material	:	100,000	m^3
Rock material	:	1,150,000	m^3

And required quantity of concrete aggregates are shown below.

Concrete aggregates	:	40,000	m^3

Taking bulking factors of the materials into consideration, required quantities of the construction materials on original ground condition are shown below.

Earth material	:	270,000	m^3
Filter material	:	77,000	m^3
Rock material	:	885,000	m^3
Concrete aggregates	:	31,000	m^3

Available quantities of construction materials on original ground condition are shown below.

Earth material	:	400,000	m^3
Filter, rock and aggregates	:	1,430,000	m^3

Available quantities of construction materials meet required quantity of the materials.

A4.2 Properties of Embankment Materials

A4.2.1 Earth material

Variations of index and engineering properties in the test results are very small. Properties of the earth material in the borrow area may be homogeneous in the whole area.

The earth material is very fine material and classified into MH in the unified soil classification, which has high compressibility when compacted and saturated, and poor workability as a construction material according to Earth Manual of USBR. And cracks may occur on the embanked body when the material is compacted on moisture content of dry side against the optimum moisture content (OMC).

The natural moisture content ranges from -5.0 to +4.5 % against OMC and the coefficient of permeability of the compacted material is very small, less than 1×10^{-6} cm/sec in the laboratory. The coefficient of consolidation of the compacted material is relatively large.

The embankment of core material should be undertaken under the following conditions.

As the content of fine particles is more than 90 %, trafficability of the earth material will be remarkably bad during rainy seasons. For this reason, construction period of the earth material should be limited to only dry seasons, during which field moisture content will be easily adjusted in field, because the natural moisture content of the earth material will be close to its OMC.

For quality control, the earth material is usually compacted at dry density of more than 95 % of γ_{dmax} which is maximum dry density. Required range of moisture content to attain sufficient compaction effect or as dry density as more than 95 % of γ_{dmax}will be OMC to OMC + 3 % in field. This is because over compaction may occur and it will be impossible to obtain sufficient trafficability when the material is compacted at moisture content of wet side more than OMC + 3 %. And embankment of the fine-grained soils like the earth material used in the project tends to be stiff and brittle when compacted at a moisture content several percent below OMC whereas the compacted soils are capable of appreciable deformation without occurrence of crack when compacted at a few percent above OMC.

Otherwise it will be necessary to pay attention to design of downstream filter of the dam for controlling concentrated leaks etc. of the core zone made of very fine earth material in the project. And as the dam aims for water supply and the reservoir capacity is not so much, it is better to provide a conservative upstream filter for the dam retaining reservoir which is frequently lowered. The design of the upstream and downstream filters of the dam is described hereafter.

A4.2.2 Filter material

Variation of properties in the test results are very small. The test results of specific gravity, absorption, loss of soundness by use of sodium sulfate and abrasion in the Los Angeles machine showed reasonable values. But blasted material will not be suitable gradation, so that the filter material obtained by crushing process should be used.

Filters of the dam are separated into upstream and downstream filters. Recommended criteria of each filter are as follows:

Downstream filter works for preventing concentrated leaks etc. according to the study of J.L.Sherard, *"Trends and debatable aspects in embankment dam engineering," Water Power & Dam Construction, December 1984*. The study has confirmed that sands and gravelly sands with average D_{15} size of 0.5 mm or smaller are conservative filters for most fine-grained clays in nature, with d_{85} size of 0.03 mm or larger. Grain size of d_{85} of the earth material is 0.025 mm or larger, so that it is classified into most fine-grained clays. The conservative downstream filter is required in consideration of afore-mentioned matter. And double downstream filters are also required because gradations between the downstream filter and rock material are remarkably different. Recommendable grading curves for the downstream filters are shown on Fig. A4-1. The gradations of the recommendable filter materials are as follows:

Fine filter

Grain size (mm)	Percentage (%)
0.075	0
0.2	15
1.2	50
1.7	60
3.5	85
5.0	100

Coarse filter

Grain size (mm)	Percentage (%)
0.075	0
3.4	15
5.0	20
24.5	50
36.0	60
70.0	85
100.0	100

Upstream filter works for preventing the earth material from migrating in the upstream direction into the voids in the upstream rock zone following reservoir drawdown according to the study of J.L.Sherard, *"Trends and debatable aspects in embankment dam engineering," Water Power & Dam Construction, December 1984*. But the study has confirmed that there is no tendency for consolidated core zone simply to migrate in the upstream direction by falling under gravity action and that the amount of water which seeps out of the core zone in the upstream direction following drawdown is very low. So the upstream filter is considered a non-critical filter because it is never called upon to act to control a concentrated leak. But as the dam aims for water supply and the reservoir capacity is not so much, meaning the dam retaining reservoir is frequently lowered, it is better to provide a conservative upstream filter. Thus, the same filter as the downstream one is recommended for the upstream filter.

A4.2.3 Rock material

Variation of properties in the test results of fresh rock is small. The test results of specific gravity and absorption, and compressive strength showed reasonable values.

The rock material should be obtained by blasting such as bench cut method in the quarry site. Maximum grain size of the rock material will be about 1.00 m. Gradation of the rock material will become as shown on Fig.A4-1. Blasting method of the rock material will be determined based on blasting tests in the quarry site.

A4.3 Properties of Concrete Aggregates

Variation of properties in the test results is very small. The test results of specific gravity, absorption, loss of soundness by use of sodium sulfate and abrasion in the Los Angeles machine showed reasonable values. And from the result of the alkali aggregate reaction test by the chemical method the aggregates were considered innocuous. But the blasted material will not be suitable in gradation, so that the concrete aggregates obtained by crushing process should be used.

A4.4 Design Values of Embankment Materials

Design values of embankment materials consist of physical property parameters, coefficient of permeability and shear strength parameter. Recommendable design values are shown in Table A4-1.

Physical property parameters are specific gravity, natural moisture content for the earth material, water absorption for filter and rock materials, dry density, wet density, saturated density and submerged density. The design values of physical property parameters are either mean values of the test results or values calculated from other parameters.

The coefficients of permeability for earth materials were determined based on permeability test. The coefficient of permeability of the earth material in the laboratory test was less than 1×10^{-6} cm/sec, but the laboratory test was conducted under the condition of one-dimentional seepage, and the design coefficient of permeability of the earth material in the field was determined in consideration of three-dimmentional seepage. For filter and rock materials, the design coefficient of permeability was based on Creager's values which provide coefficients in accordance with the gradation.

The shear strength parameters such as cohesion and internal friction angle for earth materials were determined from triaxial shear tests under consolidated-undrained condition with pore-water measurement while the shear strength parameters for filter and rock materials were determined from a large scale direct shear test under consolidated-drained condition. All the shear strength parameters were established for effective stress analysis. The cohesion (c_d) and internal friction angle (ϕ_d) of filter and rock materials which are obtained under consolidated-drained condition are considered as the same as c' and ϕ' of effective stress analysis. The design values of cohesion and internal friction angle shown in Table A4-1 are estimates at a conservative side.

A4.5 Determination of Specified Concrete Mixes

Relation between cement water ratio (C/W) and compressive strength cured for 28 days (σ_{28}) was taken by the least square method as shown on Fig. A4-2. Equations obtained in the tests are divided by maximum grain size as follows:

Maximum grain size 20 mm

$$\sigma_{28} = - 96 + 238 \ C/W$$

Maximum grain size 40 mm

$$\sigma_{28} = - 108 + 269 \ C/W$$

Specified concrete mixes decided by the C/W - σ relation obtained from the tests are shown in the Table A4-2. In determining of the specified concrete mixes, adjustment factor against required compressive strength of concrete was taken at 1.19. Further, it is noted that the upper limit of W/C value for freeze-thaw resistance is taken into consideration in determining W/C value in Table A4-2.

TABLES

Table A2-1 TEST ITEMS FOR EMBANKMENT MATERIALS

Item	Standard
1. Earth material	
(1) Specific gravity of soils	ASTM D854-58
(2) Moisture content of soils	ASTM D2216-80
(3) Grain-size analysis of soils	ASTM D422-63
(4) Liquid limit of soils	ASTM D423-66
(5) Plastic limit of soils	ASTM D424-59
(6) Soluble salts determination of soils	EM E-8
(7) Moisture-density relations of soils	ASTM D698-78
(8) Permeability of soils	ASTM D2434-68
(9) Triaxial shear of soils	EM E-17
(10) Consolidation of soils	ASTM D2435-70
2. Filter material	
(1) Specific gravity and absorption of coarse aggregate	ASTM C127-84
(2) Grain-size analysis of fine and coarse aggregate	ASTM C136-84a
(3) Soundness of aggregate by use of sodium sulfate	ASTM C88-83
(4) Resistance to degradation of small-size coarse aggregate abrasion and impact in the Los Angeles machine	ASTM C131-81
3. Rock material	
(1) Specific gravity and absorption of coarse aggregate	ASTM C127-84
(2) Compressive strength of intact rock core specimens	ASTM D2938-86
(3) Large scale direct shear test	
(4) Bleeding of concrete	ASTM C232-87

Table A2-2 TEST RESULTS OF EARTH MATERIAL

ITEM				EM1-1	EM1-2	EM2-1	EM2-2	EM3-1	EM3-2	EM4-1	EM4-2
Specific gravity		Gs		2.862	2.875	2.885	2.879	2.882	2.900	2.884	2.891
	Gravel		(%)	0	34	0	0	0	0	0	0
Gradation	Sand		(%)	5	13	4	4	1	2	5	7
	Silt		(%)	49	29	45	50	42	43	37	59
	Clay		(%)	46	24	51	46	57	55	58	34
Liquid limit		LL	(%)	70.2	63.7	68.0	62.8	75.6	69.0	72.2	68.0
Plastic limit		PL	(%)	45.7	46.3	45.8	40.7	59.7	48.2	48.2	46.6
Plasticity index		PI		24.5	17.4	22.2	22.1	15.9	20.8	24.0	21.4
Unified soil classification				MH	MH	MH	MH	MH	MH	MH	MH
Soluble salt			(part/million)		150	250			100	200	
Natural moisture content			(%)	42.8	45.0	41.6	37.0	41.8	36.0	38.2	37.0
Optimum moisture content		OMC	(%)		40.5	40.6			40.6	42.0	
Maxmum dry density		DDmax	(tf/m3)		1.297	1.306			1.31	1.278	
Coefficient of permeability		k	(cm/sec)								
	dry side				5.9E-06	7.6E-07			3.3E-06	4.8E-07	
	wet side				3.7E-08	7.6E-08			6.0E-07	1.0E-07	
Triaxial shear (effective stress)		CU condition									
Cohesion		c'	(kgf/cm2)		0.07	0.00			0.04	0.08	
Internal friction angle			(degrees)		34	33			33	33	

Table A2-3 SUMMARY OF TEST RESULTS OF EARTH MATERIAL

Item			Range	Average
Specific gravity	Gs		2.862 - 2.900	2.882
	Gravel	(%)	0 - 34	4
Gradation	Sand	(%)	1 - 13	5
	Silt	(%)	29 - 59	44
	Clay	(%)	24 - 58	47
Consistency	LL	(%)	62.8 - 75.6	68.7
	PL	(%)	40.7 - 59.7	47.7
	PI		15.9 - 24.5	21.0
Unified soil classification			MH	MH
Natural moisture content	wn	(%)	36.0 - 45.0	39.9
Soluble salt determination		(parts/million)	100 - 250	175

Table A2-4 PERMEABILITY OF EARTH MATERIAL

Sample		Dry density (tf/m^3)	Moisture content (%)	Coefficient of permeability (cm/sec)
EM1-2	(Dry side)	1.215	34.5	5.9×10^{-6}
	(Wet side)	1.215	44.1	3.7×10^{-8}
EM2-1	(Dry side)	1.241	36.0	7.6×10^{-7}
	(Wet side)	1.241	47.5	7.6×10^{-8}
EM3-2	(Dry side)	1.244	34.2	3.3×10^{-6}
	(Wet side)	1.244	46.5	6.0×10^{-7}
EM4-1	(Dry side)	1.214	37.9	4.8×10^{-7}
	(Wet side)	1.214	47.6	1.0×10^{-7}

Table A2-5 PROPERTIES OF FILTER MATERIAL

Item			A-1	A-2	A-3	A-4	A-5	A-6
Specific gravity	(Oven dry)		2.871	2.894	2.892	2.892	2.903	2.893
	(Saturated surface dry)		2.898	2.913	2.912	2.915	2.922	2.915
Water absorption	(%)		0.93	0.65	0.70	0.81	0.65	0.77
Gradation	Gravel	(%)	100	100	100	100	100	100
	Sand	(%)	0	0	0	0	0	0
	Silt	(%)	0	0	0	0	0	0
	Clay	(%)	0	0	0	0	0	0
Soundness		(%)	1.9	2.3	3.1	1.6	2.9	1.7
Abrasion		(%)	16.9	16.4	16.7	16.5	15.9	16.4

Table A2-6 PROPERTIES OF ROCK CORE MATERIAL

Item	R-1	R-2	R-3	R-4	R-5	R-6
Type of rock*	F	O	O	F	F	O
Hole	JQ-1	JQ-1	JQ-1	JQ-2	JQ-2	JQ-2
Depth (GL- m)	9.8	12.7	14.6	7.6	9.1	13.7
Unit weight (tf/m^3)	2.321	2.760	2.846	2.381	2.578	2.912
Natural moisture content (%)	7.4	1.2	1.0	4.8	5.6	0.7
Specific gravity	2.241	2.709	2.860	2.289	2.478	2.932
Water absorption (%)	10.3	1.8	1.8	10.6	6.8	2.4
Compressive strength (kgf/cm^2)	168	980	1408	216	517	855

* Type of rocks
F : Flow Breccia
O : Old Lava (Felty)

Table A3-1 TEST ITEMS FOR MIX PROPORTION OF CONCRETE

Item	Standard
1. Concrete material tests	
1.1 Cement tests	
(1) Specific gravity	BS4550 Part3
(2) Setting time	BS4550 Part3
(3) Soundness	BS4550 Part3
(4) Strength	BS4550 Part3
1.2 Aggregate tests	
(1) Specific gravity and absorption of coarse aggregate	ASTM C127-84
(2) Specific gravity and absorption of fine aggregate	ASTM C128-84
(3) Unit weight and voids in aggregate	ASTM C29-87
2. Tests for selection of fine aggregate percentage	
(1) Making and curing concrete test specimens in the laboratory	ASTM C192-81
(2) Slump of portland cement concrete	ASTM C134-78
(3) Unit weight, yield, and air content(gravimetric) of concrete	ASTM C138-81
3. Fresh concrete tests	
(1) Making and curing concrete test specimens in the laboratory	ASTM C192-81
(2) Slump of portland cement concrete	ASTM C134-78
(3) Unit weight, yield, and air content(gravimetric) of concrete	ASTM C138-81
(4) Bleeding of concrete	ASTM C232-87
4. Concrete strength tests	
(1) Compressive strength of cylindrical concrete specimens	ASTM C39-86
(2) Splitting tensile strength of cylindrical specimens	ASTM C496-86

Table A3-2 SUMMARY OF TEST RESULTS OF AGGREGATES

Item			Range			Average
Specific gravity	Oven dry		2.871	-	2.903	2.891
Saturated surface dry			2.898	-	2.922	2.913
Water absorption	(%)		0.65	-	0.93	0.75
	Gravel	(%)		100		100
Gradation	Sand	(%)		0		0
	Silt	(%)		0		0
	Clay	(%)		0		0
Bulk Density	(kgf/m^3)		1428	-	1494	1463
Voids in aggregate	(%)		48.3	-	50.3	49.4
Soundness	(%)		1.6	-	3.1	2.3
Abrasion	(%)		15.9	-	16.9	16.5

Table A3-3 TEST RESULTS OF AGGREGATES

Item			A-1	A-2	A-3	A-4	A-5	A-6
Specific gravity	(Oven dry)		2.871	2.894	2.892	2.892	2.903	2.893
(Saturated surface dry)			2.898	2.913	2.912	2.915	2.922	2.915
Water absorption	(%)		0.93	0.65	0.70	0.81	0.65	0.77
	Gravel	(%)	100	100	100	100	100	100
Gradation	Sand	(%)	0	0	0	0	0	0
	Silt	(%)	0	0	0	0	0	0
	Clay	(%)	0	0	0	0	0	0
Bulk density	(kgf/m^3)		1428	1478	1494	1489	1445	1445
Voids in aggregate	(%)		50.3	48.9	48.3	48.5	50.2	50.1
Soundness	(%)		1.9	2.3	3.1	1.6	2.9	1.7
Abrasion	(%)		16.9	16.4	16.7	16.5	15.9	16.4

Table A3-4 CLASS OF CONCRETE SPECIFIED

Class of Concrete	Maximum Grain Size (mm)	Required Compressive Strength (kgf/cm^2)	Range of Slump (cm)	Structures
A	20	210	14 - 18	Slab. Block out. Pneumatically applied concrete, Building
B	20	180	12 - 14	Drain ditch, Catch basin, Drain inlet & outlet, Precast concrete.
C	40	180	12 - 14	Reinforced concrete, Tunnel lining, Spillway (outside).
D	40	140	8 - 10	Backfill, Levelling, Gravity wall.
E	80	120	8 - 10	Spillway (inside).
F	40	100	8 - 10	Levelling concrete

Table A3-5 PROPERTIES OF CEMENT IN CONCRETE TESTS

Item			Result
Specific gravity			3.13
Setting time			
	Initial	(min)	176
	Final	(min)	290
Consistency		(%)	29
Soundness			Good
Strength	3 days	(kgf/cm^2)	232.5
	28 days	(kgf/cm^2)	505.8

Table A3-6 PROPERTIES OF AGGREGATES IN CONCRETE TESTS

Item		Fine	Max.20	Max.40	Max.80
Specific gravity	(Oven Dry)	2.96	2.75	2.71	2.77
	(Saturated Surface Dry)	3.00	2.81	2.77	2.82
Absorption	(%)	1.25	2.14	1.88	1.41
Unit weight	(kgf/m^3)	1906	1517	1514	1677
Voids	(%)	36.5	46.2	45.3	40.5

Table A3-7 TEST RESULTS OF AGGREGATES IN CONCRETE TESTS

Item		Grain size	Result
Specific gravity	(Oven Dry)	80 - 40	2.913
		40 - 30	2.732
		30 - 20	2.599
		20 - 14	2.732
		14 - 10	2.741
		10 - 6	2.754
		6 - 5	2.797
		5 - 0	2.964
	(Saturated Surface Dry)	80 - 40	2.927
		40 - 30	2.768
		30 - 20	2.684
		20 - 14	2.785
		14 - 10	2.801
		10 - 6	2.820
		6 - 5	2.852
		5 - 0	3.001
Water absorption	(%)	80 - 40	0.47
		40 - 30	1.31
		30 - 20	1.88
		20 - 14	1.94
		14 - 10	2.19
		10 - 6	2.47
		6 - 5	1.97
		5 - 0	1.25
Unit weight	(kgf/m^3)	Max.80	1677
		Max.40	1514
		Max.20	1517
		less than 5	1906
Voids	(%)	Max.80	40.5
		Max.40	45.3
		Max.20	46.2
		less than 5	36.5

Table A3-8 DESIGN OF MIX PROPORTION FOR TRAIL MIXES

Class of cement	Maximum grain size of aggregate (mm)	Slump (cm)	Air content (%)	Water cement ratio W/C (%)	Fine aggregate percentage s/a (%)	Unit Quantity (cubic meter)							
						Water W (kg)	Cement C (kg)	Fine aggregate S (kg)	Coarse aggregate 80~40 (kg)	40~20 (kg)	20~10 (kg)	10~5 (kg)	Admixture (ml)
A	20	14 ~ 18	6.0	60	49	190	317	954	0	0	586	344	793
B	20	12 ~ 14	6.0	65	50	184	283	998	0	0	589	346	708
C	40	12 ~ 14	4.5	55	39	163	296	816	0	566	377	236	740
D	40	8 ~ 10	4.5	65	41	157	242	886	0	565	377	236	605
E	80	8 ~ 10	3.5	65	36	131	202	831	458	458	306	167	505
F	40	8 ~ 10	4.5	75	43	157	209	943	0	554	254	231	523

Specific Gravity of Fine Aggregate = 3.00

Specific Gravity of Coarse Aggregate (Max. 20mm) = 2.81

Specific Gravity of Coarse Aggregate (Max. 40mm) = 2.77

Specific Gravity of Coarse Aggregate (Max. 80mm) = 2.82

Specific Gravity of Cement = 3.13

Table A3-9 MIX PROPORTION FOR FINAL MIXES

Class of cement	Maximum grain size of aggregate (mm)	Slump (cm)	Air content (%)	Water cement ratio W/C (%)	Fine aggregate percentage s/a (%)	Unit Quantity (cubic meter)							
						Water W (kg)	Cement C (kg)	Fine aggregate S (kg)	Coarse aggregate 80~40 (kg)	Coarse aggregate 40~20 (kg)	Coarse aggregate 20~10 (kg)	Coarse aggregate 10~5 (kg)	Admixture (ml)
A	20	14 ~ 18	6.0	60	55	207	345	1028	0	0	496	291	863
B	20	12 ~ 14	6.0	65	56	200	308	1078	0	0	500	294	770
C	40	12 ~ 14	4.5	55	44	177	322	891	0	503	335	209	805
D	40	8 ~ 10	4.5	65	46	170	262	968	0	504	336	210	655
E	80	8 ~ 10	3.5	65	38	143	220	857	434	434	289	158	550
F	40	8 ~ 10	4.5	75	45	170	227	962	0	521	239	217	568

Specific Gravity of Fine Aggregate = 3.00

Specific Gravity of Coarse Aggregate (Max. 20mm) = 2.81

Specific Gravity of Coarse Aggregate (Max. 40mm) = 2.77

Specific Gravity of Coarse Aggregate (Max. 80mm) = 2.82

Specific Gravity of Cement = 3.13

Table A3-10 PROPERTIES OF FRESH CONCRETE

Class of concrete	Slump (cm)	Unit weight (kgf/m^3)	Air content (%)	Bleeding Quantity (cm^3/cm^2)	Bleeding Ratio (%)
A	17	2505	0.44	0.26	4.4
B	14	2520	0.39	0.29	4.9
C	14	2540	0.47	0.17	3.5
D	8	2550	0.58	0.29	5.3
E	9	2604	1.33	0.24	5.9
F	11	2545	1.55	0.45	9.2

Table A3-11 RESULTS OF CONCRETE STRENGTH TESTS

Class of concrete	Curing time (days)	Unit weight (kgf/m^3)	Compressive strength (kgf/cm^2)	Tensile strength (kgf/cm^2)
A	7	2487	214.1	30.6
	28	2475	340.6	36.7
B	7	2510	228.4	25.5
	28	2495	305.9	33.7
C	7	2514	301.8	37.7
	28	2540	325.3	34.7
	91		388.5	
D	7	2530	223.3	27.5
	28	2542	295.7	28.6
E	7	2598	222.3	29.6
	28	2590	240.7	34.7
	91		231.5	
F	7	2543	206.0	28.6
	28	2525	206.0	26.5

Table A4 - 1 DESIGN VALUES OF EMBANKMENT MATERIALS

Item		Earth	Filter Fine	Filter Coarse	Rock
Specific gravity		2.88			
(Oven dry condition)			2.87	2.87	2.87
Natural moisture content	(%)	40.0			
Water absorption	(%)		2.0	2.0	2.0
Dry density	(tf/m^3)	1.23	1.90	2.00	2.10
Wet density	(tf/m^3)	1.72	1.94	2.04	2.14
Saturated density	(tf/m^3)	1.80			
Submerged density	(tf/m^3)	0.80	1.23	1.30	1.37
Coefficient of permeability	(cm/sec)	$1x10^{-5}$	$1x10^{-3}$	$1x10^{-2}$	$1x10^{-1}$
Strength parameter (effective stress analysis)					
Cohesion c'	(tf/m^2)	0.0	0.0	0.0	0.0
Internal friction angle	(degree)	30.0	36.0	38.0	40.0

Table A4-2 MIX PROPORTION OF CONCRETE SPECIFIED

Class of cement	Maximum grain size of aggregate (mm)	Slump (cm)	Air content (%)	Water cement ratio W/C (%)	Fine aggregate percentage s/a (%)	Unit Quantity (cubic meter)							
						Water W (kg)	Cement C (kg)	Fine aggregate S (kg)	Coarse aggregate				Admixture (ml)
									80~40 (kg)	40~20 (kg)	20~10 (kg)	10~5 (kg)	
A	20	14 ~ 18	6.0	60	55	207	345	1028	0	0	496	291	863
B	20	12 ~ 14	6.0	65	56	200	308	1078	0	0	500	294	770
C	40	12 ~ 14	4.5	55	44	177	322	891	0	503	335	209	805
D	40	8 ~ 10	4.5	65	46	170	262	968	0	504	336	210	655
E	80	8 ~ 10	3.5	65	38	143	220	857	434	434	289	158	550
F	40	8 ~ 10	4.5	75	45	170	227	962	0	521	239	217	568

Specific Gravity of Fine Aggregate = 3.00

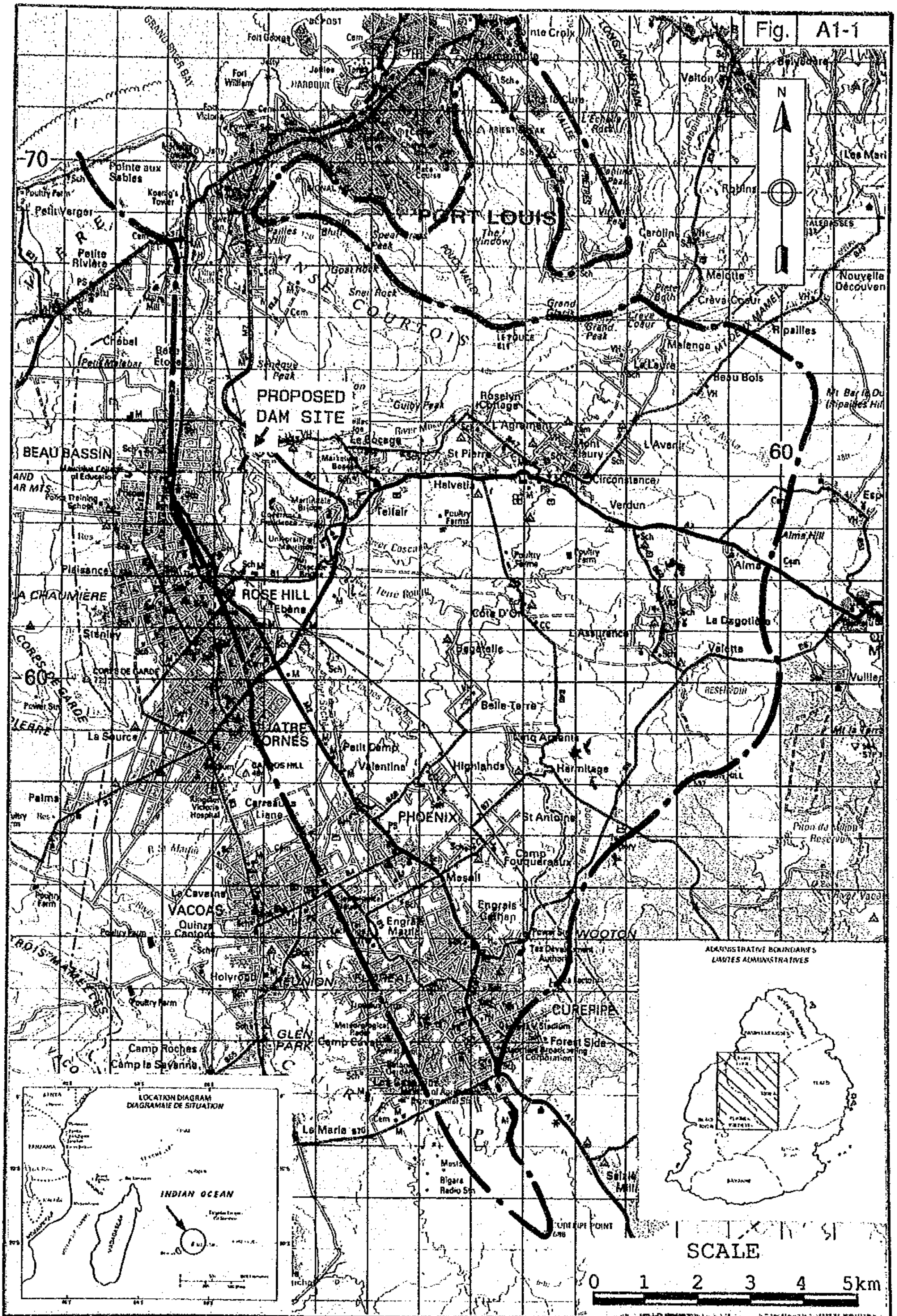
Specific Gravity of Coarse Aggregate (Max. 20mm) = 2.81

Specific Gravity of Coarse Aggregate (Max. 40mm) = 2.77

Specific Gravity of Coarse Aggregate (Max. 80mm) = 2.82

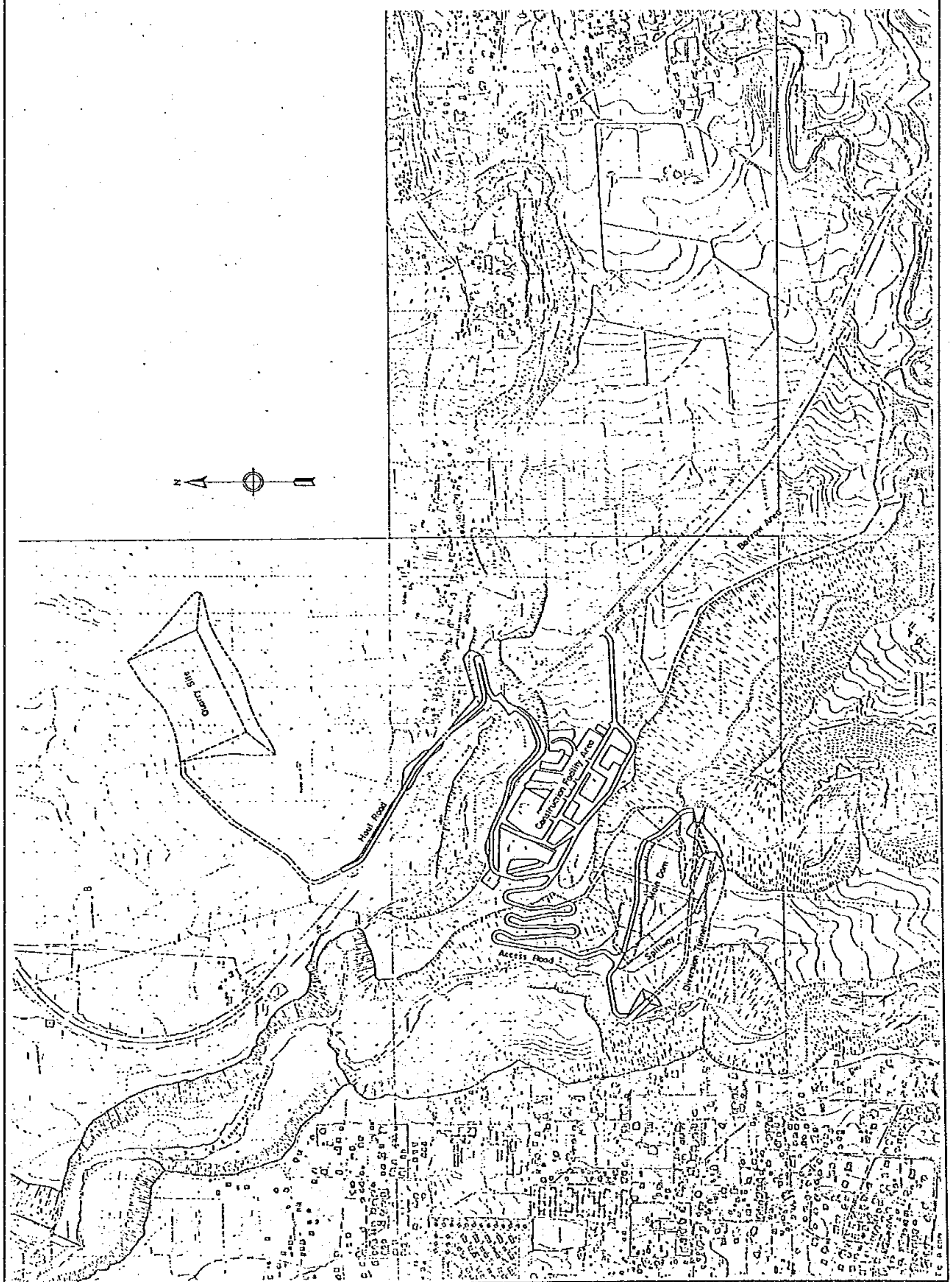
Specific Gravity of Cement = 3.13

FIGURES

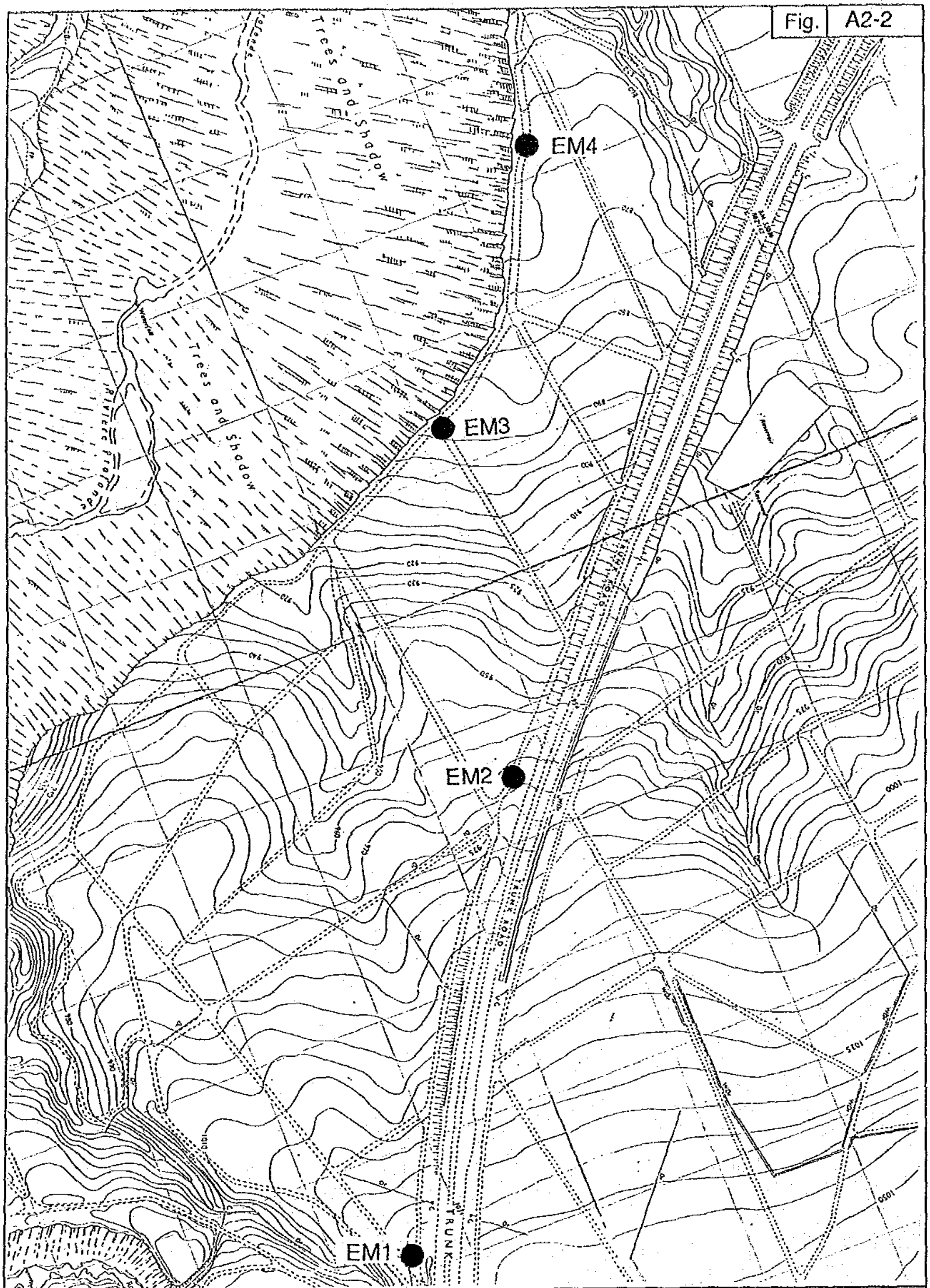


GENERAL LOCATION MAP OF PROJECT AREA	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-1

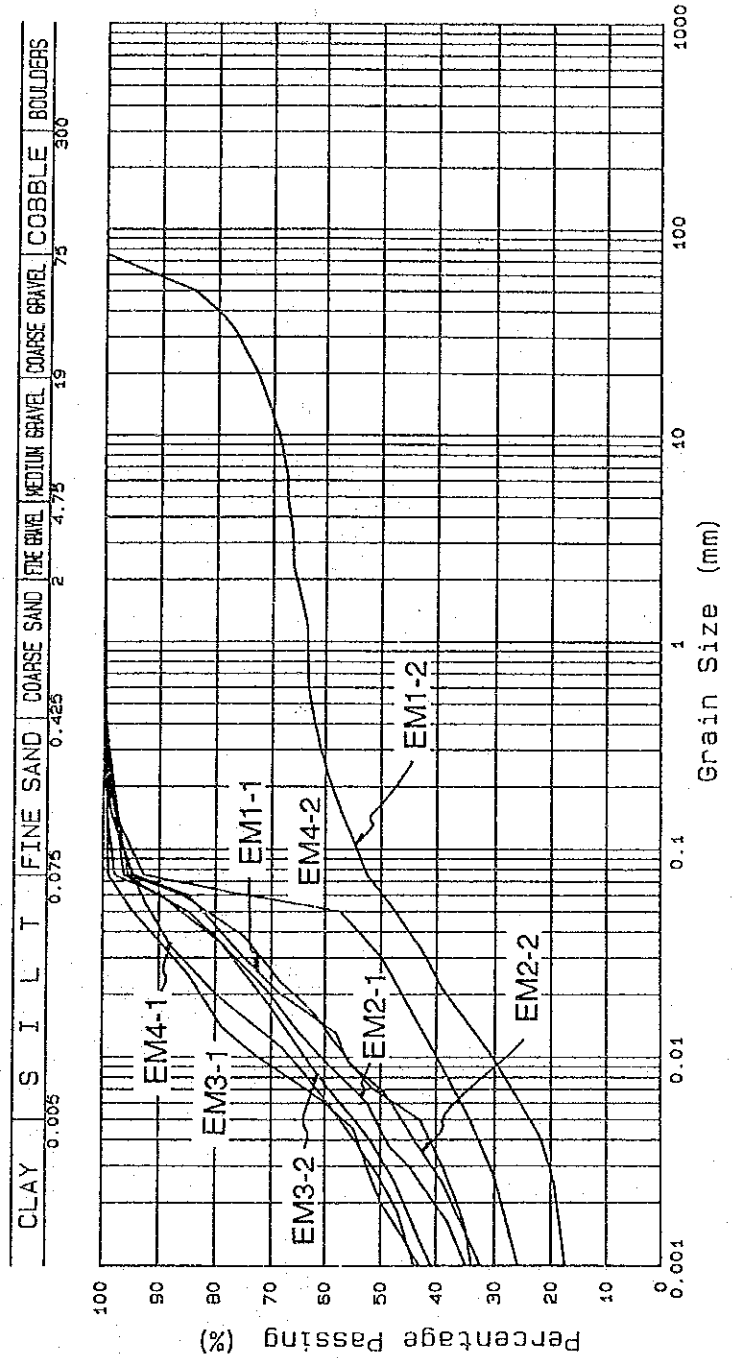


GENERAL PROJECT LAYOUT	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY



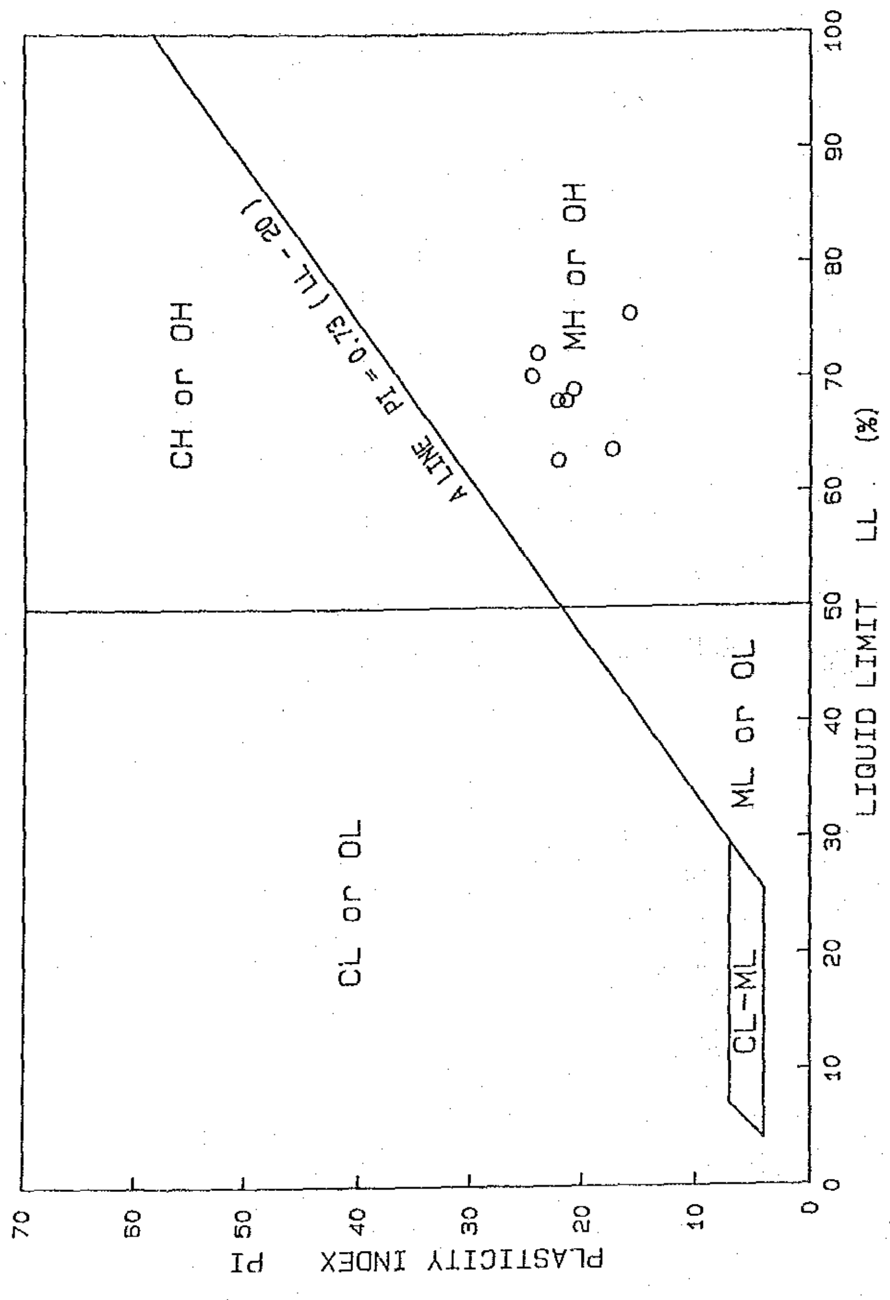
SAMPLING LOCATIONS IN BORROW AREA	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A2-3



GRADING CURVES OF EARTH MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-4

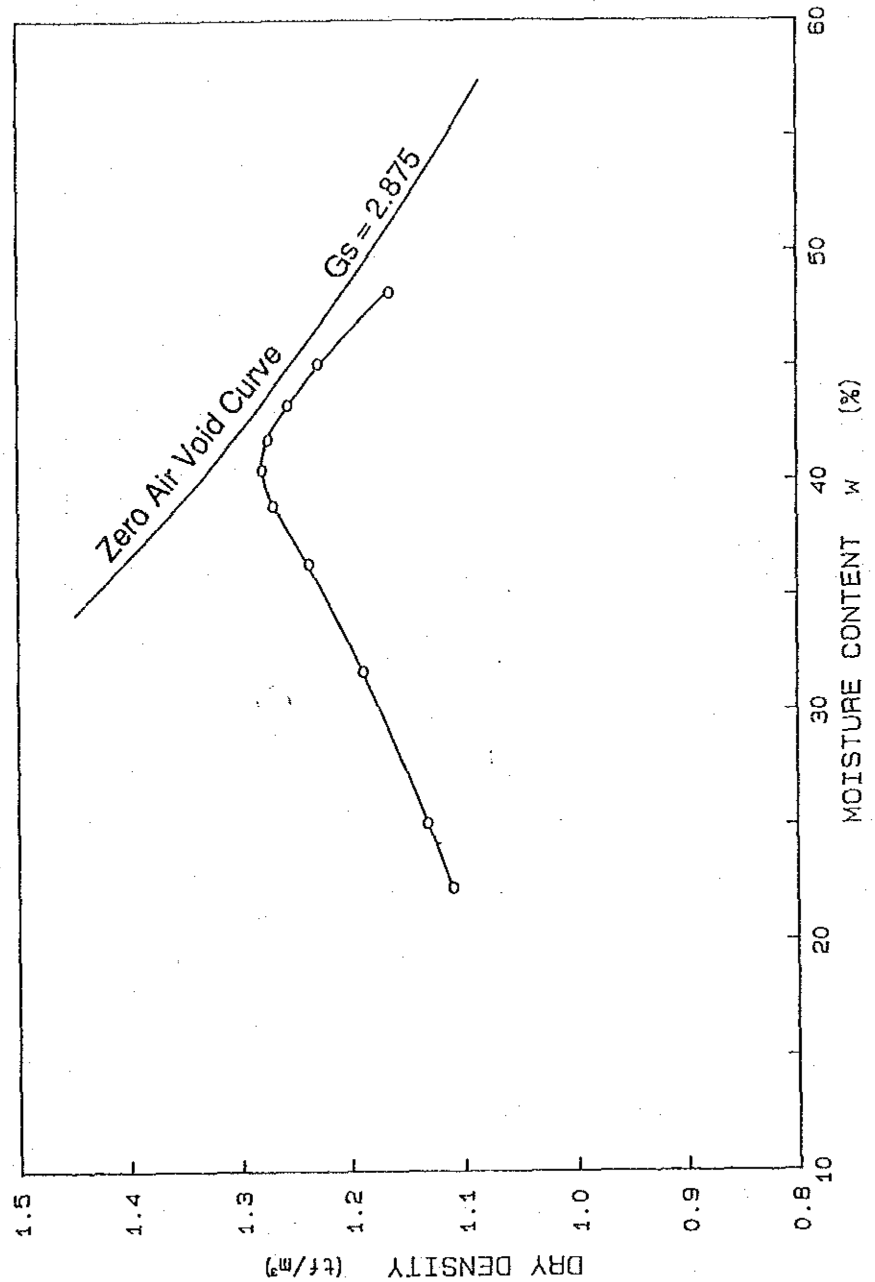


PLASTICITY CHART OF EARTH MATERIALS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

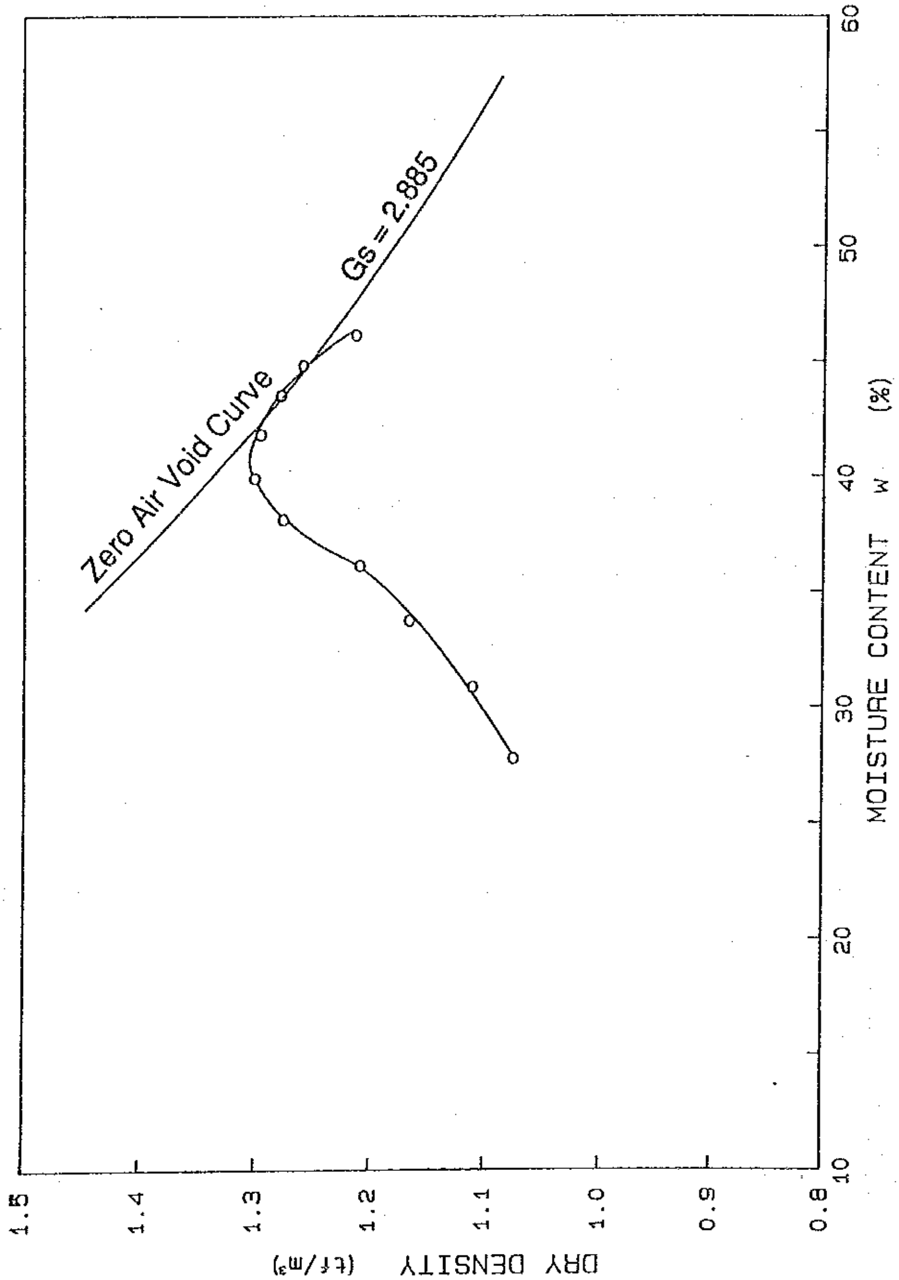
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-5 (1)



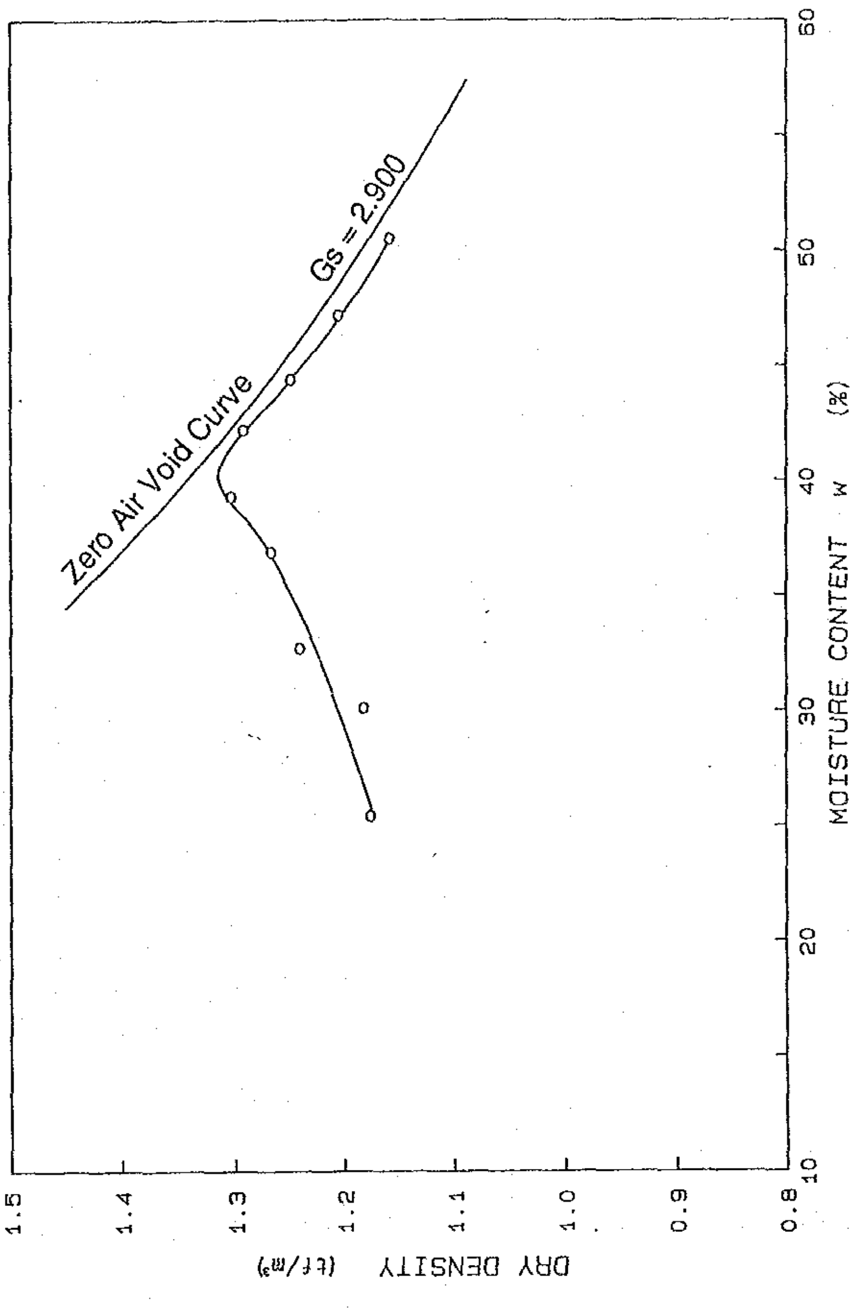
COMPATION CURVE OF EARTH MATERIAL (EM1-2)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-5 (2)



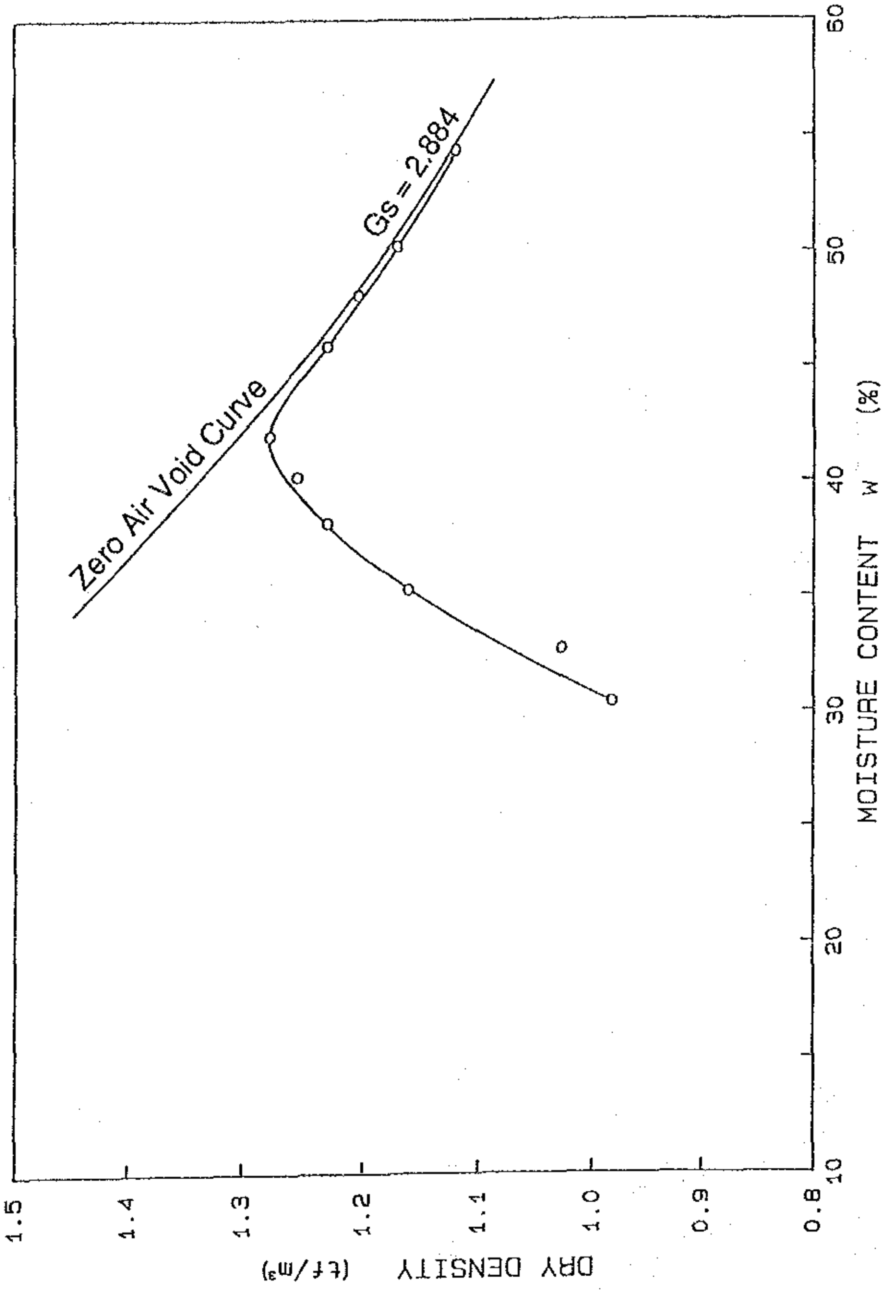
COMPATION CURVE OF EARTH MATERIAL (EM2-1)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-5 (3)



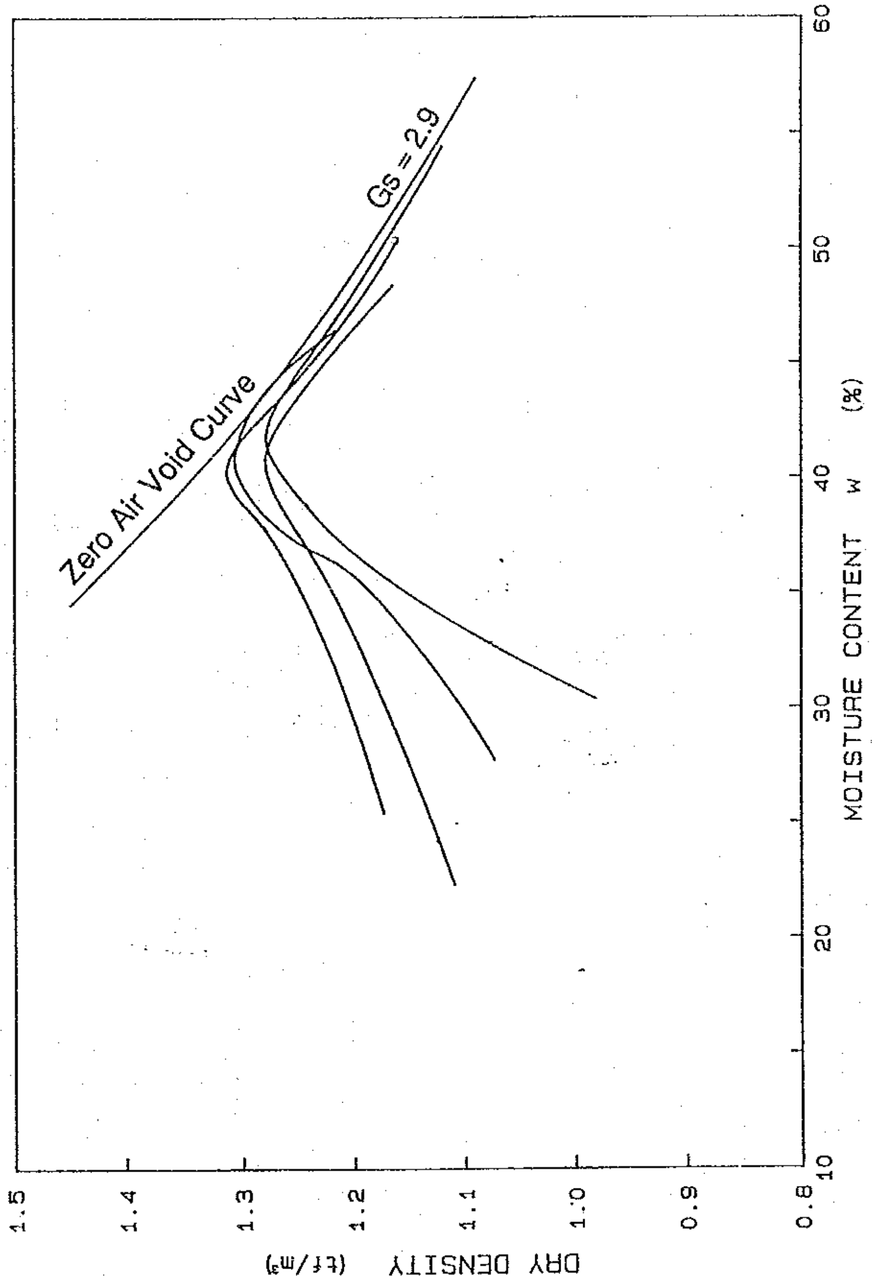
COMPATION CURVE OF EARTH MATERIAL (EM3-2)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-5 (4)



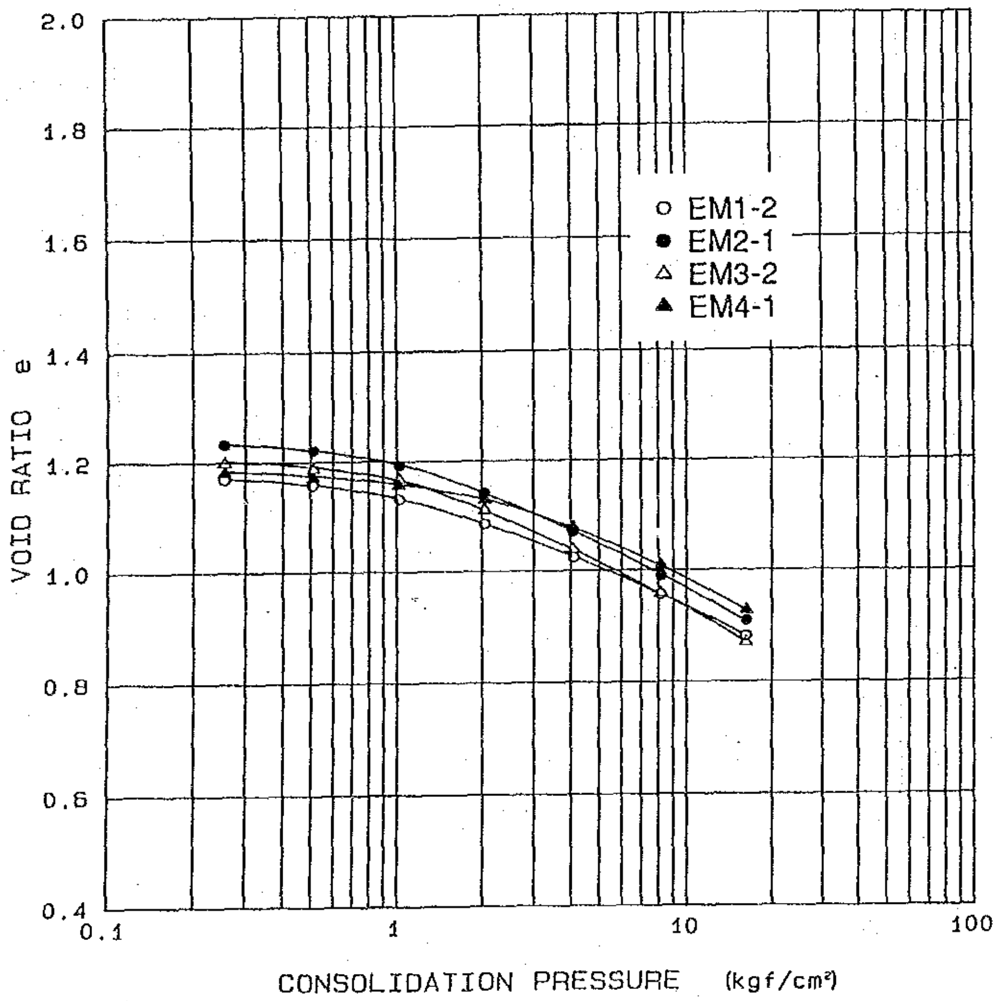
COMPATION CURVE OF EARTH MATERIAL (EM4-1)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-5 (5)



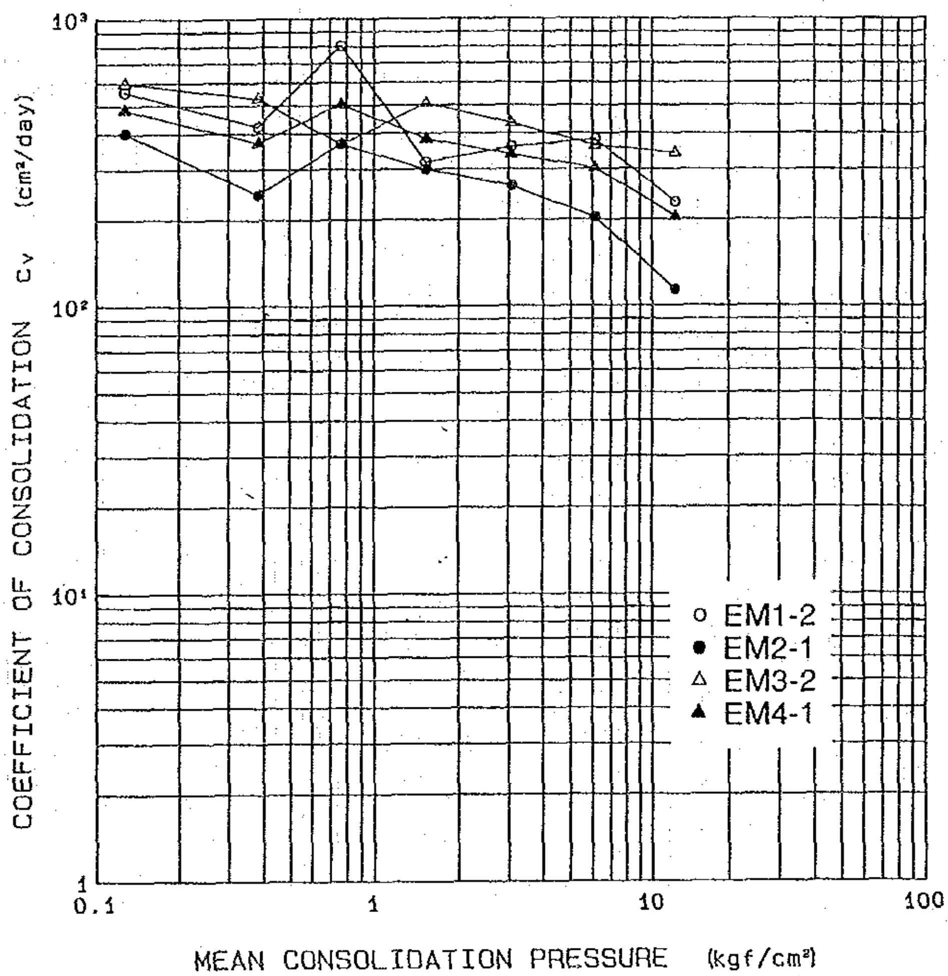
COMPATION CURVE OF EARTH MATERIALS (ALL MATERIALS)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-6

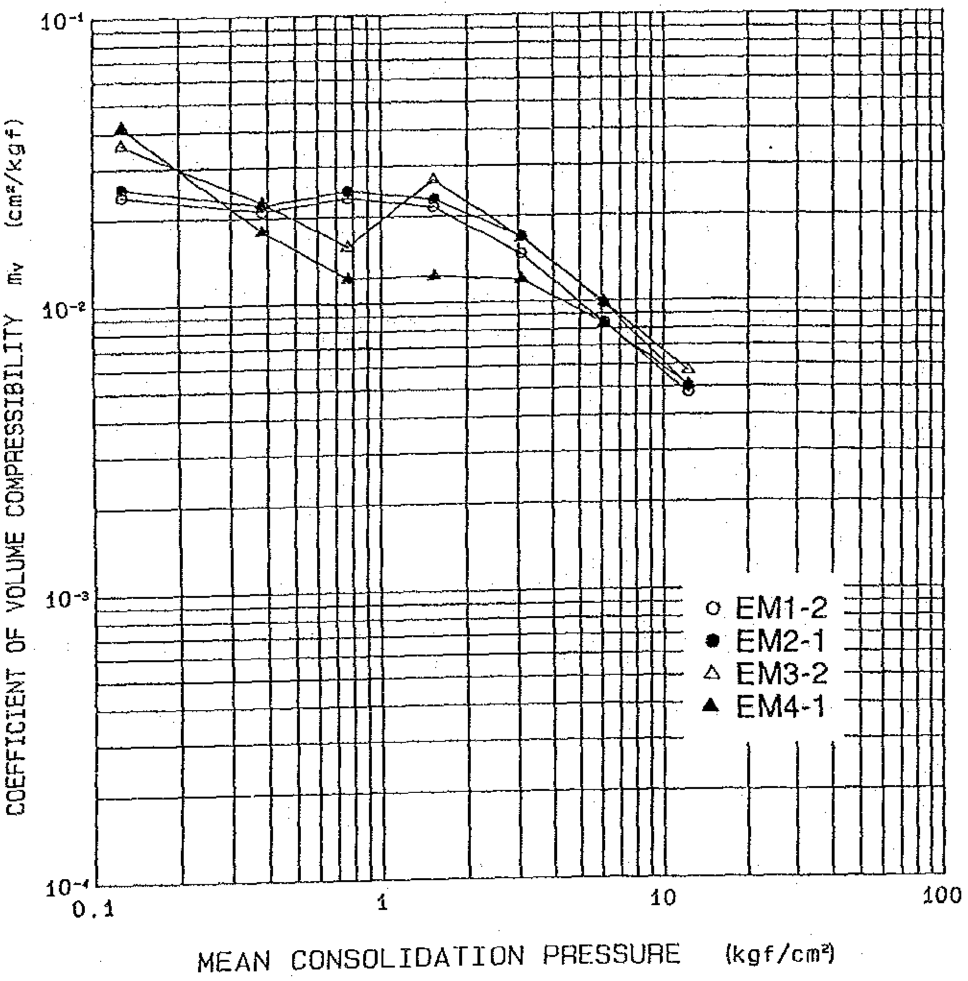


RELATION BETWEEN CONSOLIDATION PRESSURE AND VOID RATIO OF EARTH MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A2-7

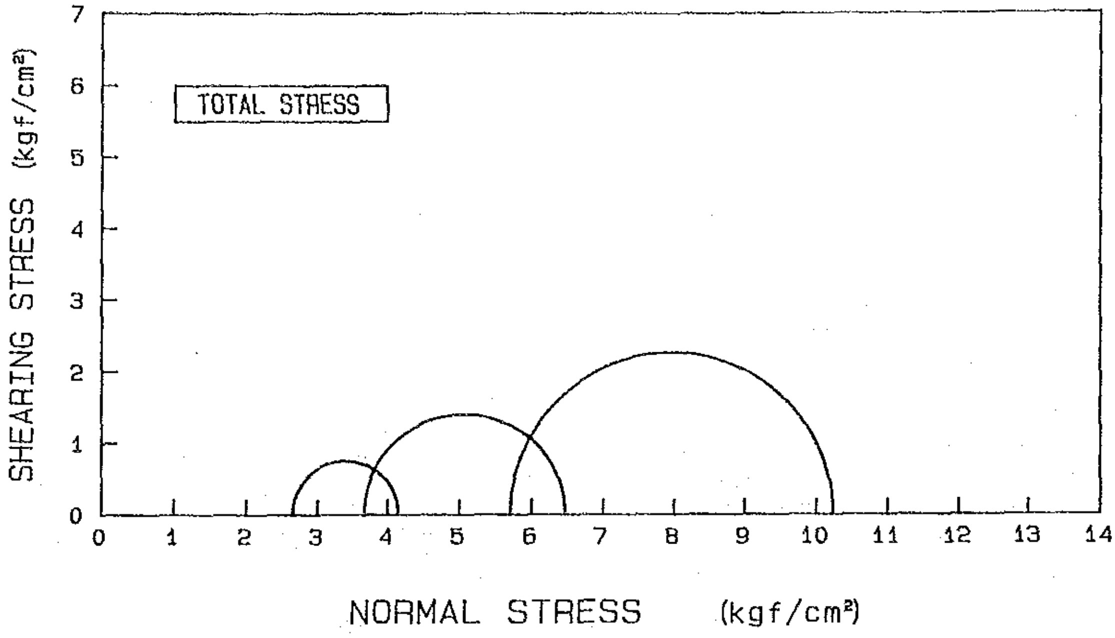


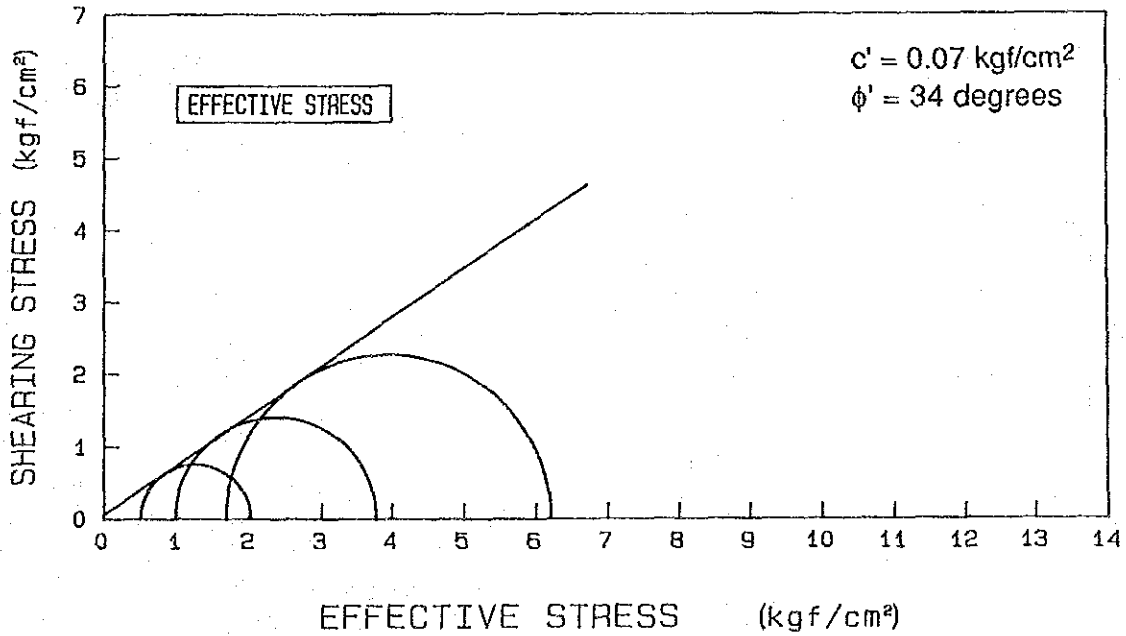
RELATION BETWEEN MEAN CONSOLIDATION PRESSURE AND COEFFICIENT OF CONSOLIDATION OF EARTH MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY



RELATION BETWEEN MEAN CONSOLIDATION PRESSURE AND COEFFICIENT OF VOLUME COMPRESSIBILITY OF EARTH MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

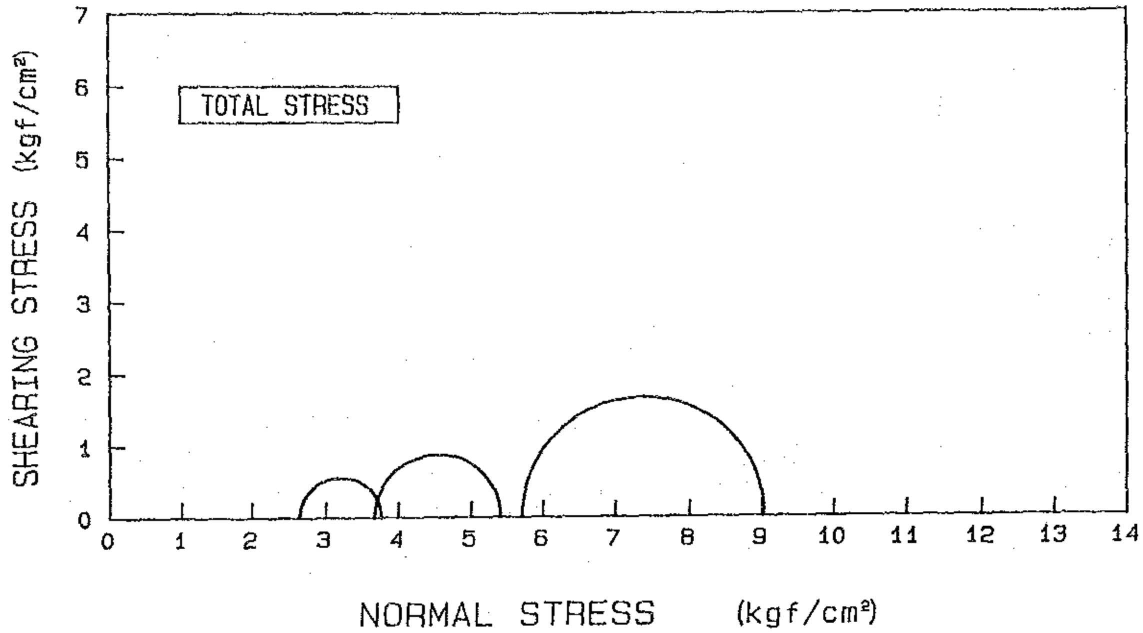
Fig. A2-9 (1)

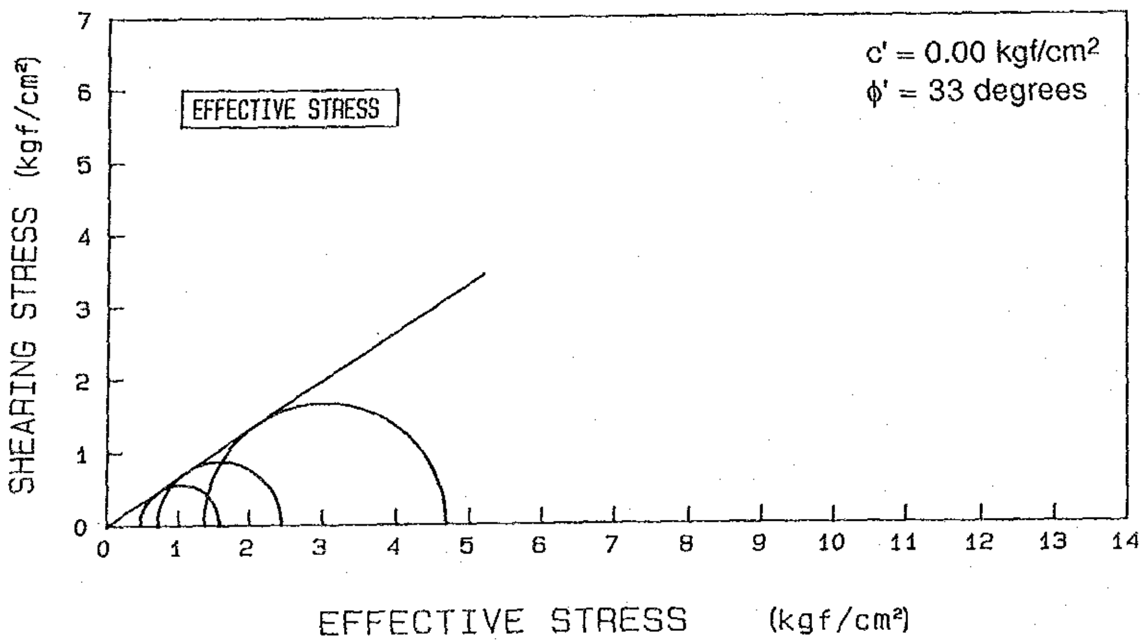




MOHR'S CIRCLES OF EARTH MATERIAL (EM1-2)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

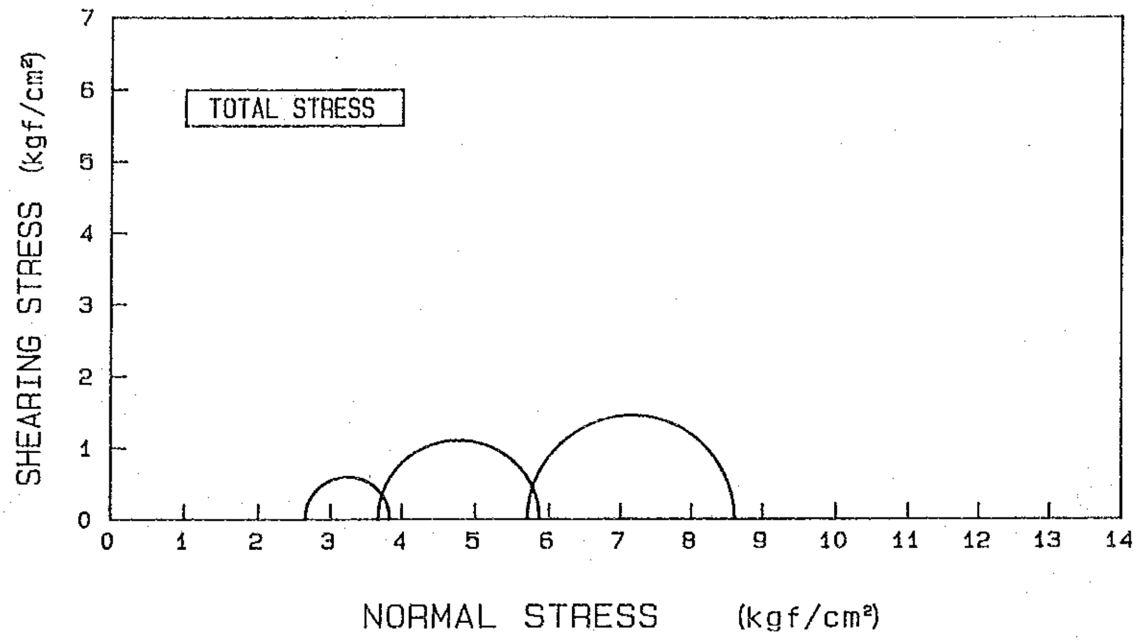
Fig. A2-9 (2)

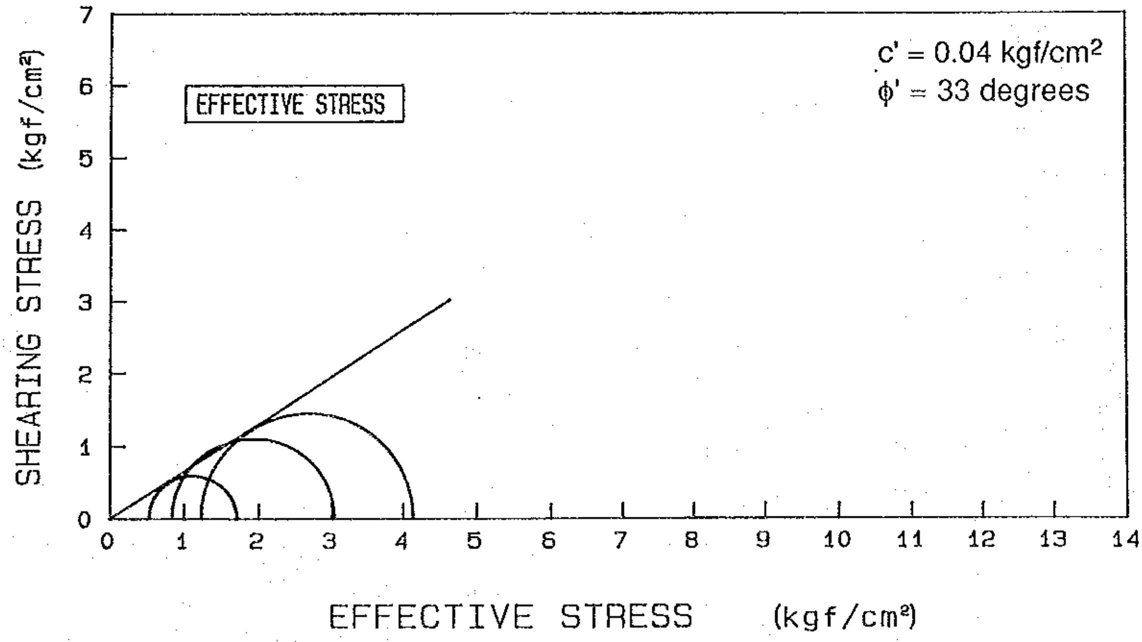




MOHR'S CIRCLES OF EARTH MATERIAL (EM2-1)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

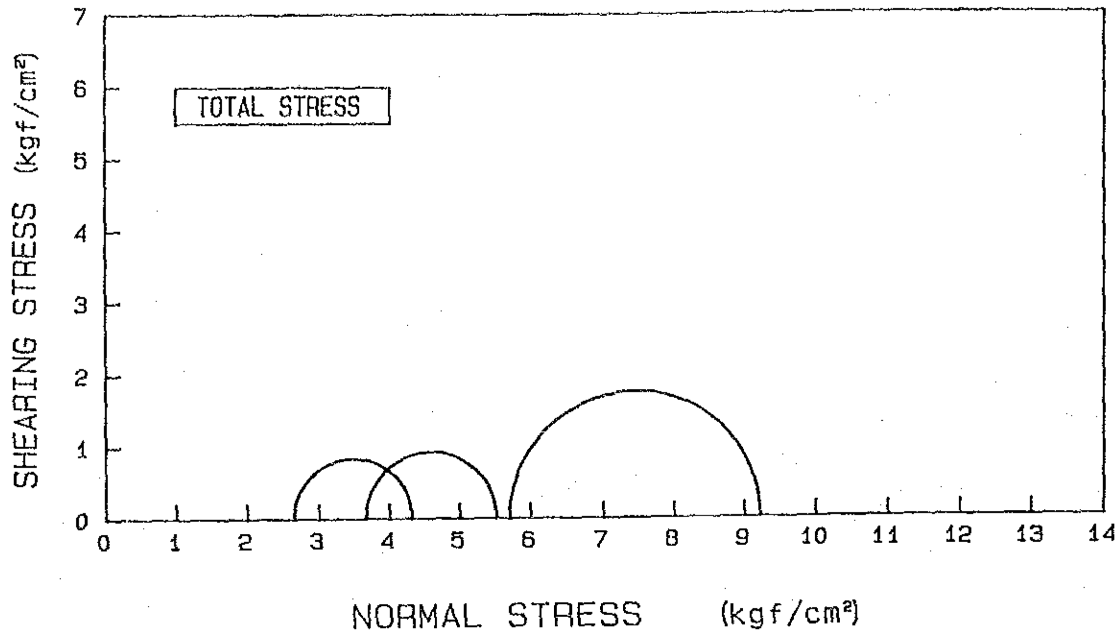
Fig. A2-9 (3)

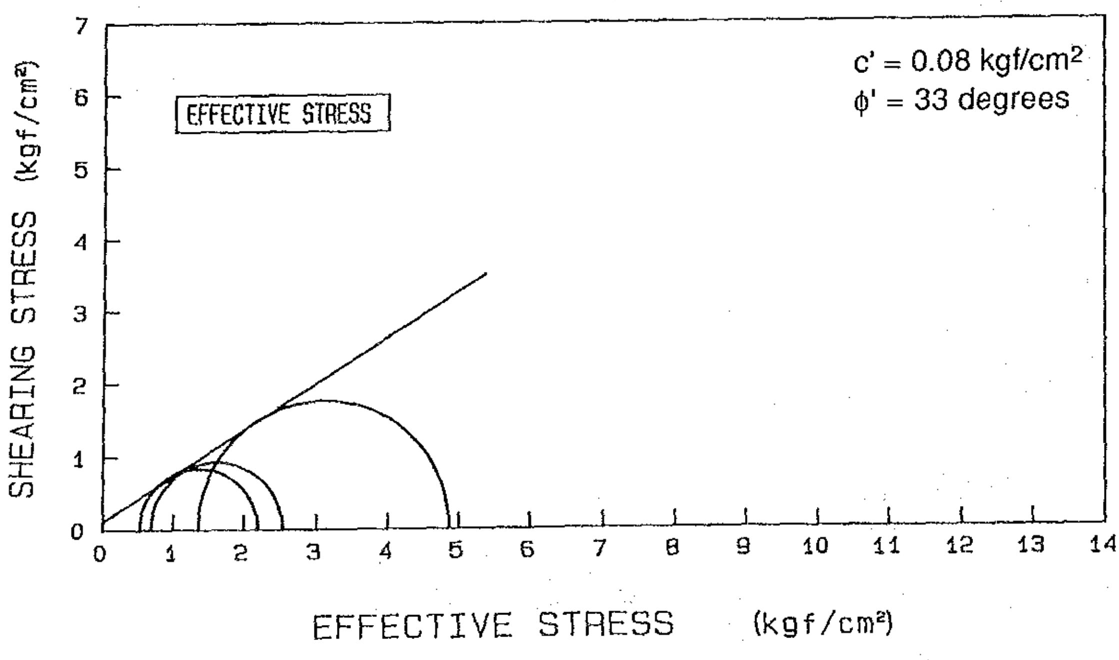




MOHR'S CIRCLES OF EARTH MATERIAL (EM3-2)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

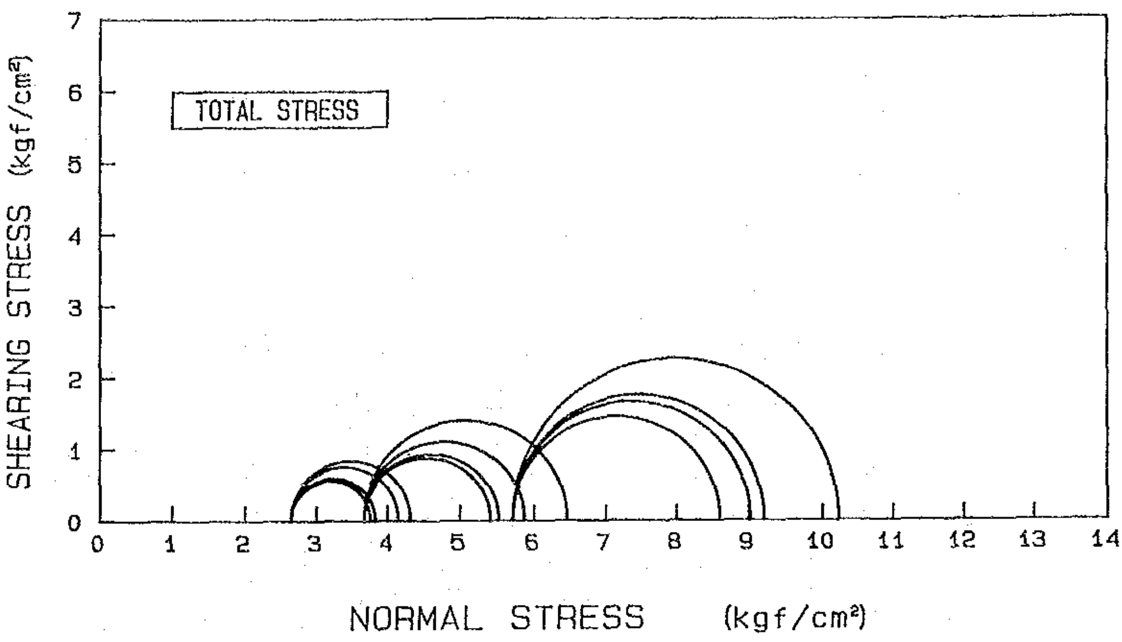
Fig. A2-9 (4)

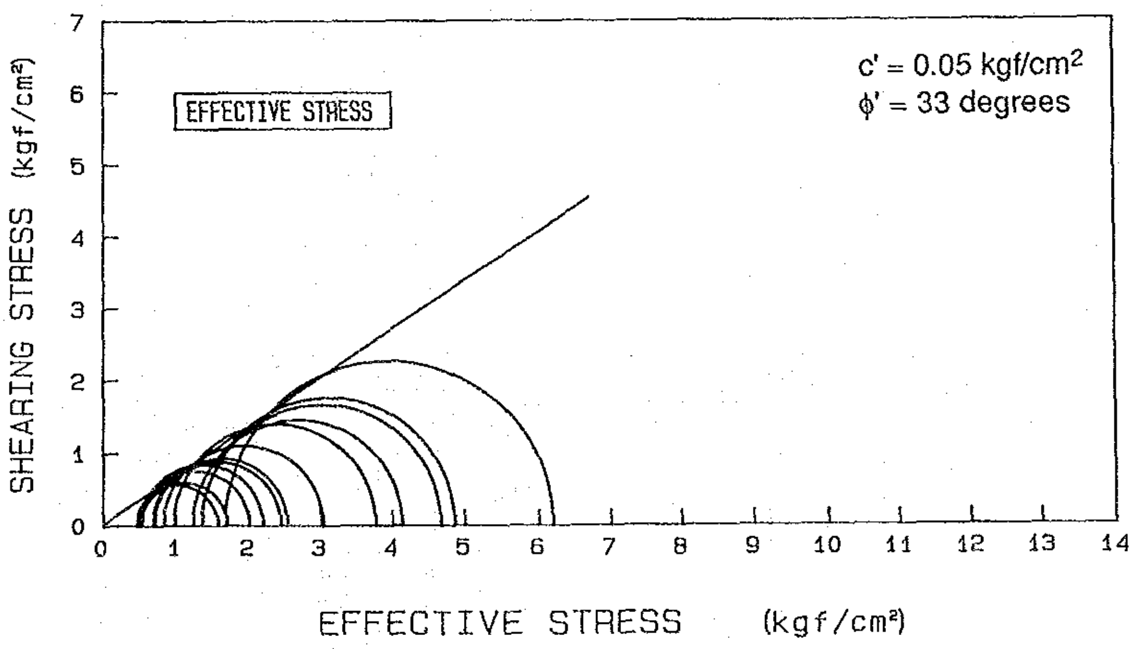




MOHR'S CIRCLES OF EARTH MATERIAL (EM4-1)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

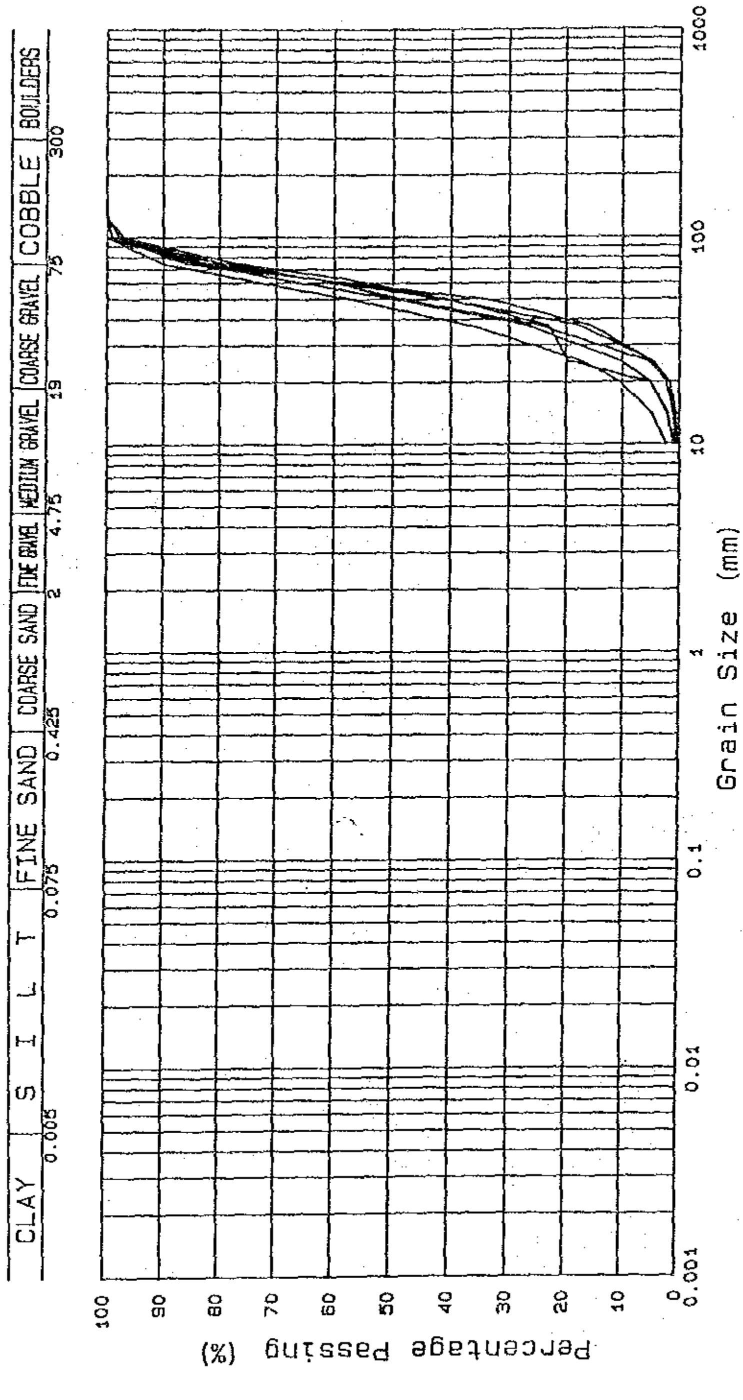
Fig. A2-9 (5)





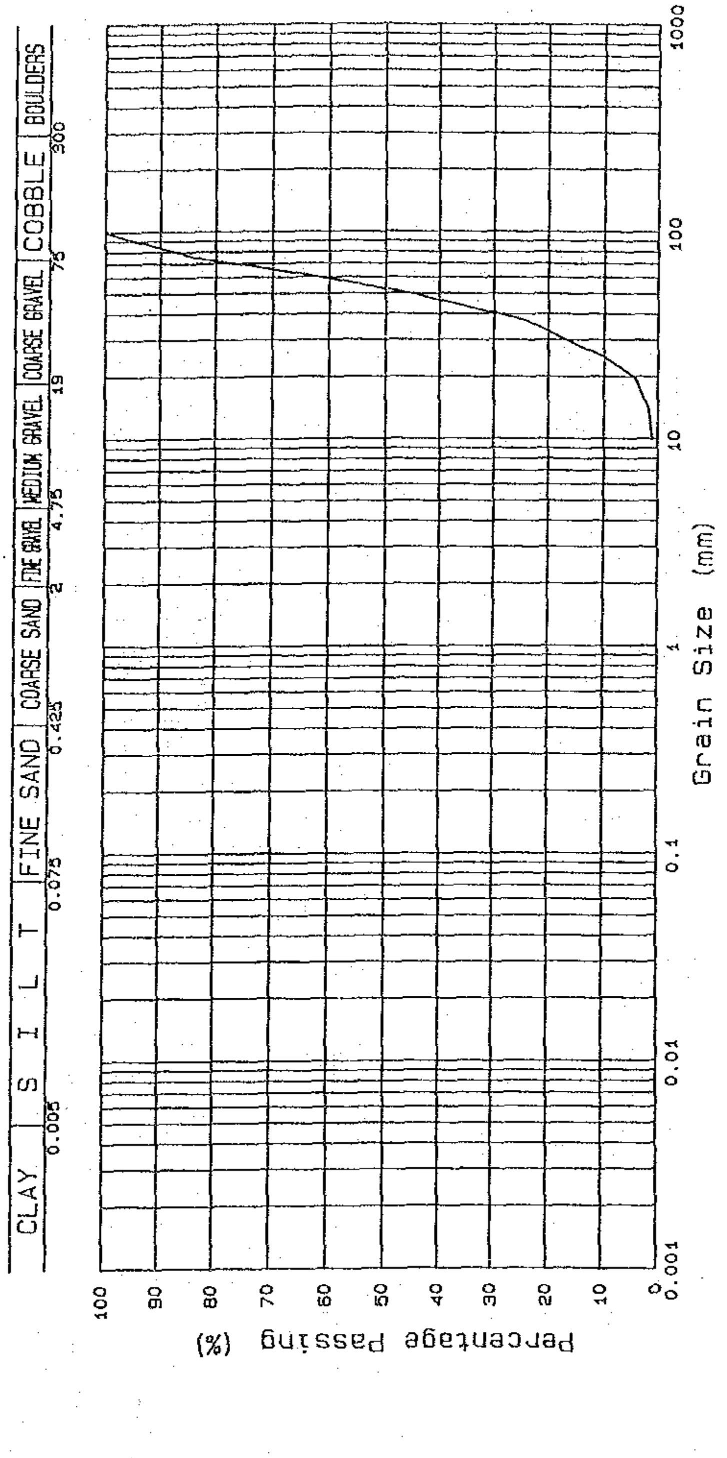
MOHR'S CIRCLES OF EARTH MATERIAL (ALL MATERIALS)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A2-10



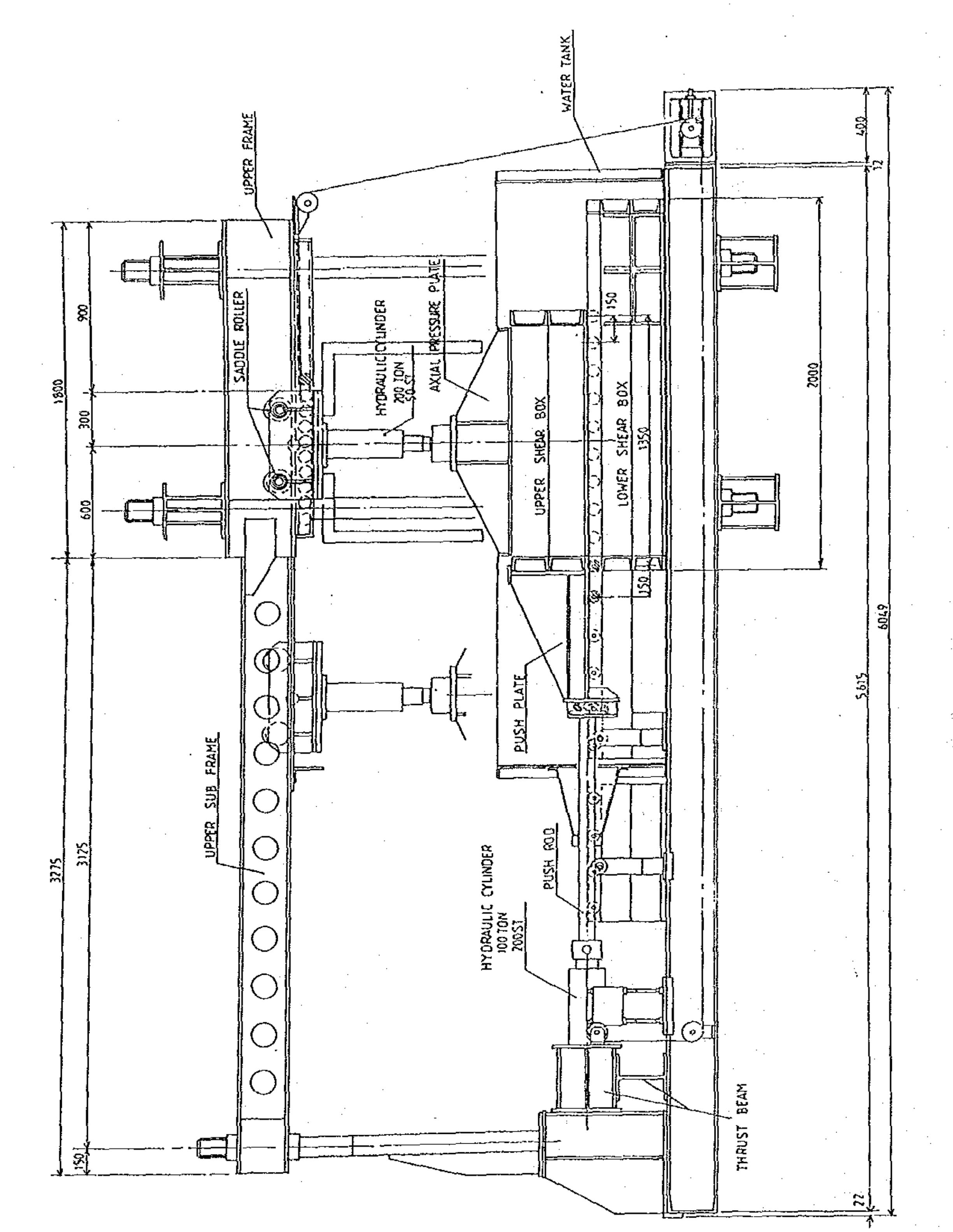
GRADING CURVES OF FILTER MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A2-11



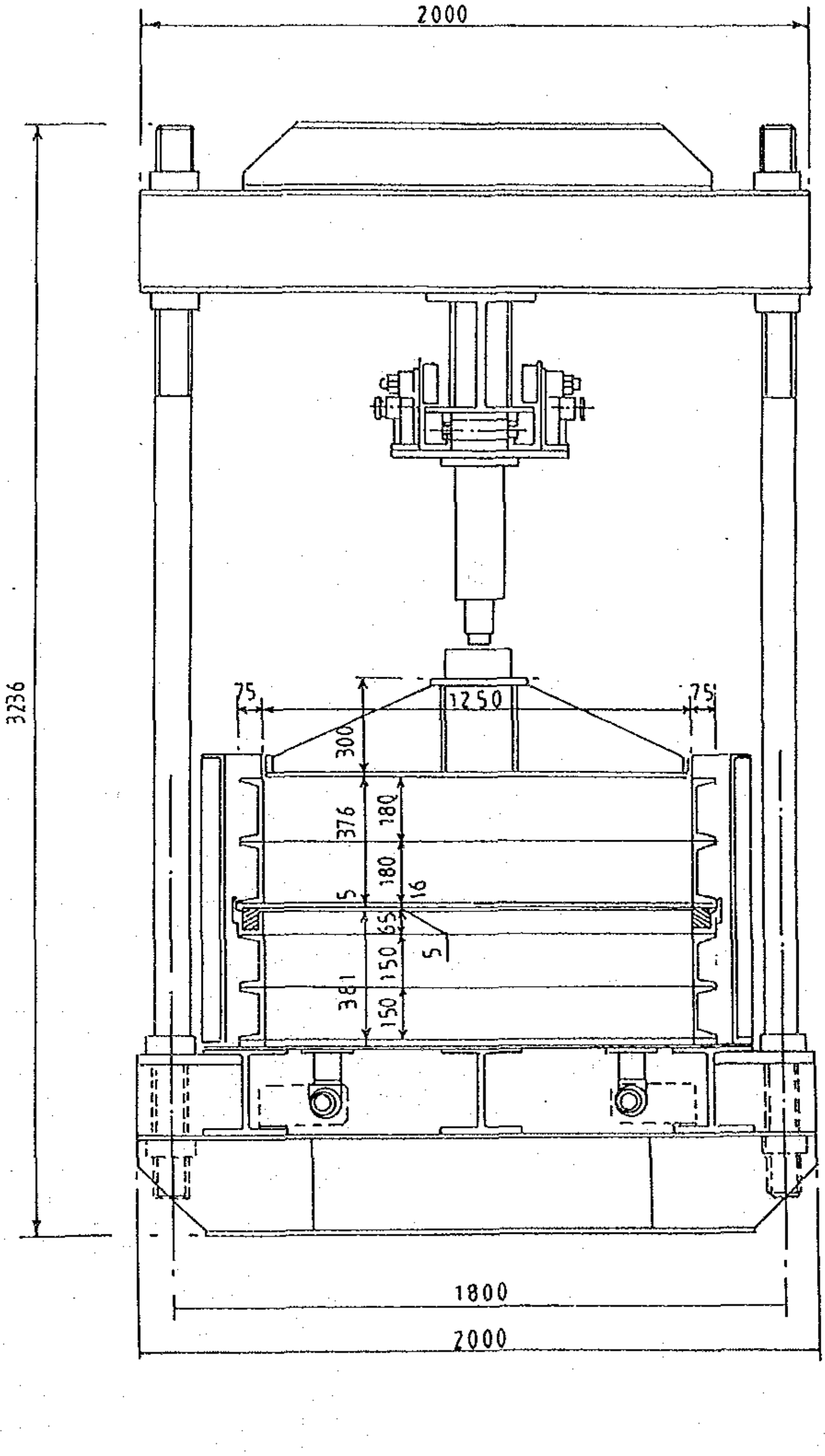
GRADING CURVE OF TESTED MATERIAL OF LARGE SCALE DIRECT SHEAR TEST	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-12 (1)



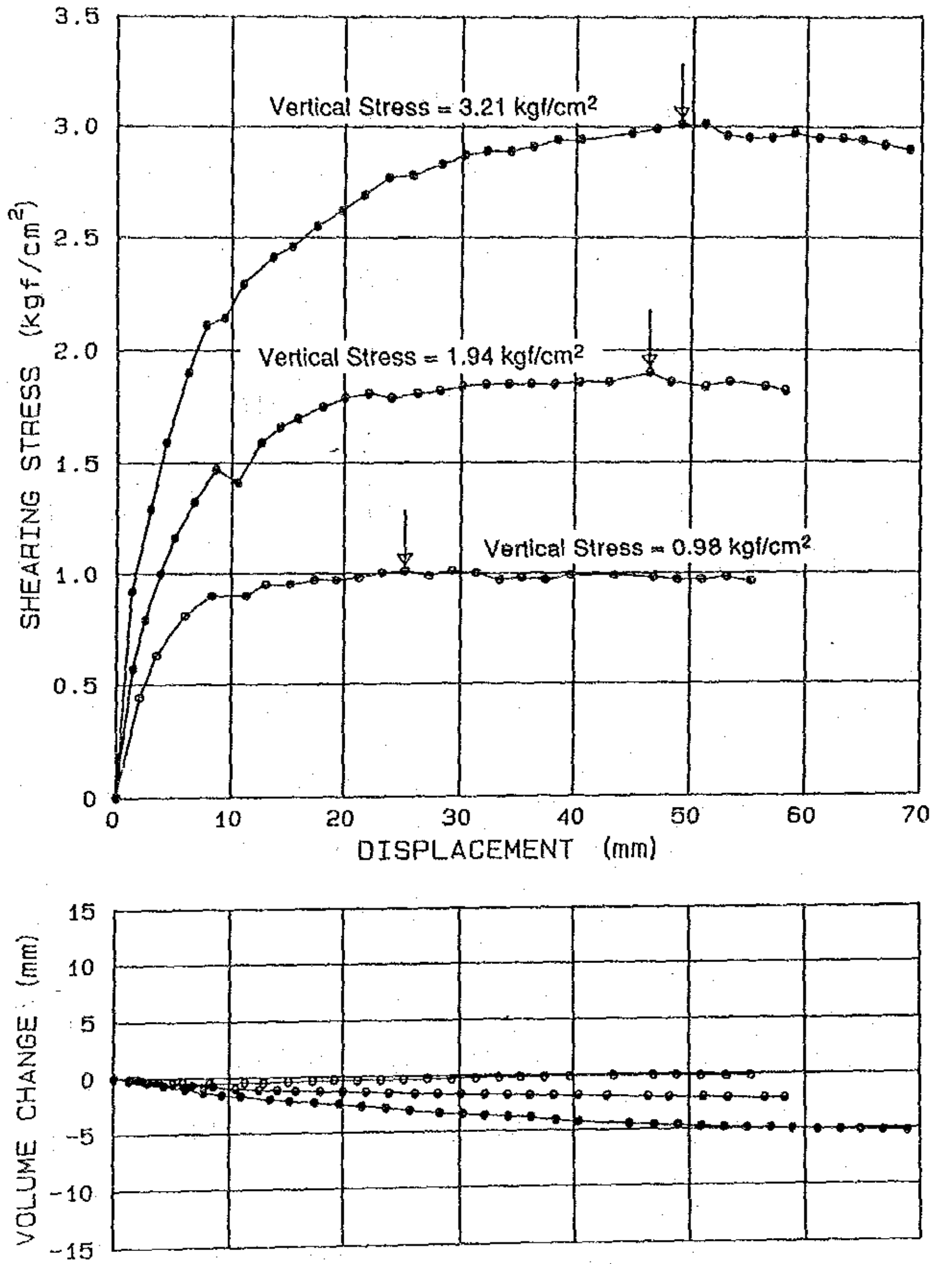
TESTING APPARATUS OF LARGE SCALE DIRECT SHEAR TEST (1)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-12 (2)



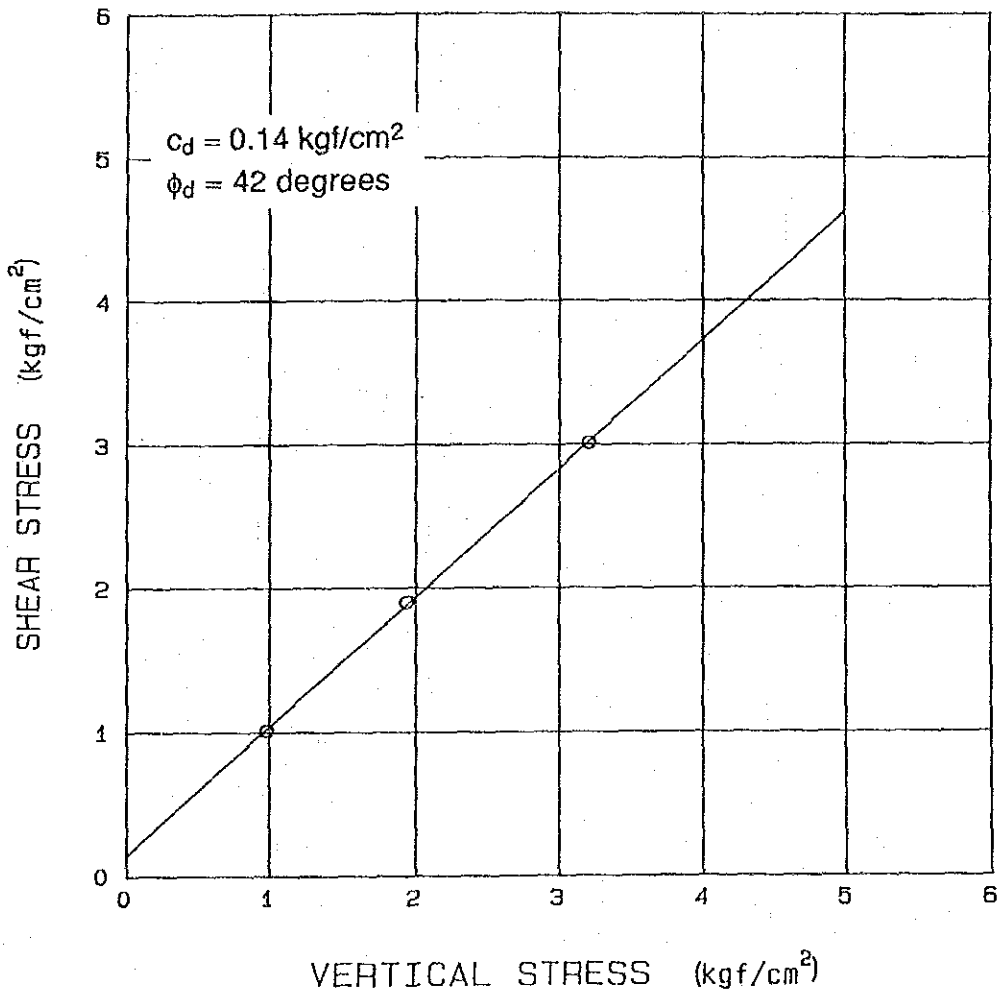
TESTING APPARATUS OF LARGE SCALE DIRECT SHEAR TEST (2)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-13



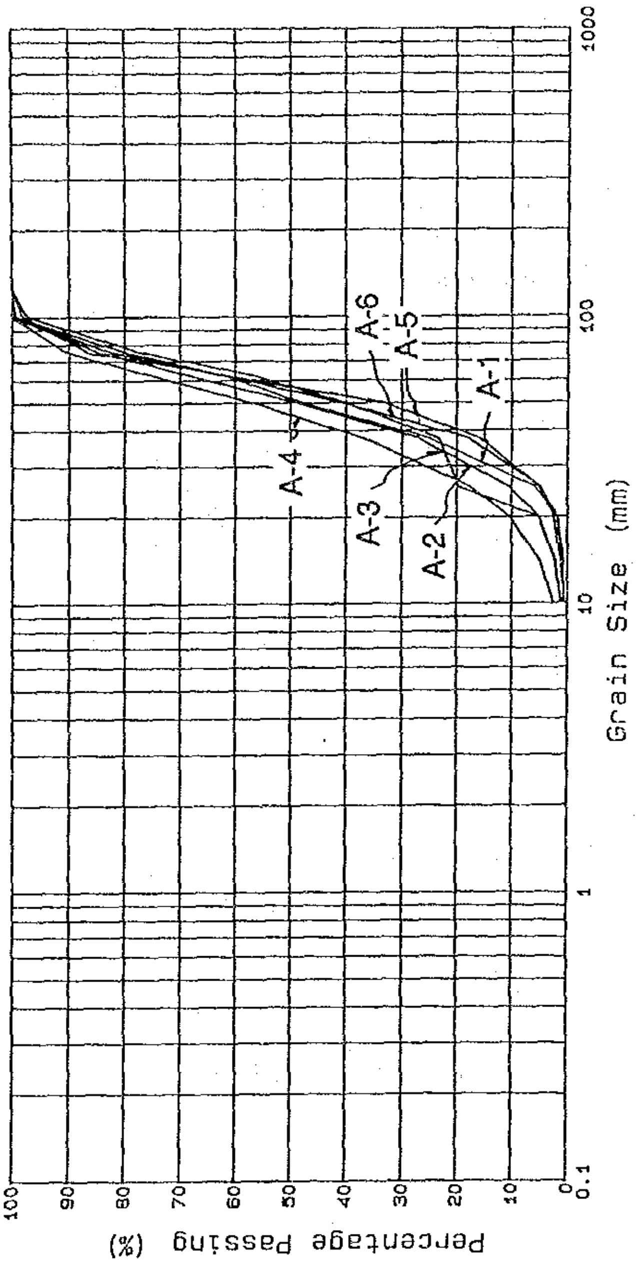
RELATION BETWEEN DISPLACEMENT, AND SHEARING STRESS AND VOLUME CHANGE	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A2-14



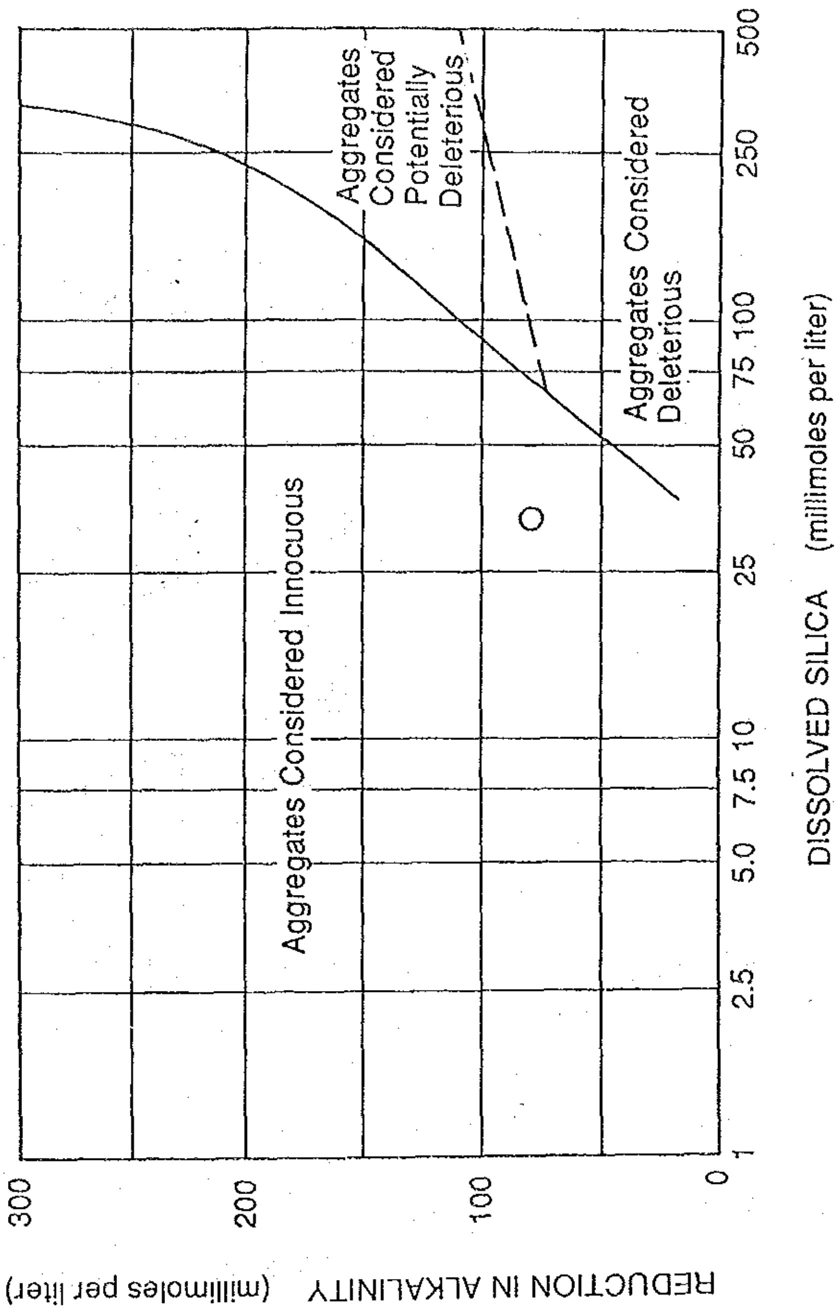
RELATION BETWEEN VERTICAL STRESS AND SHEAR STRESS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A3-1



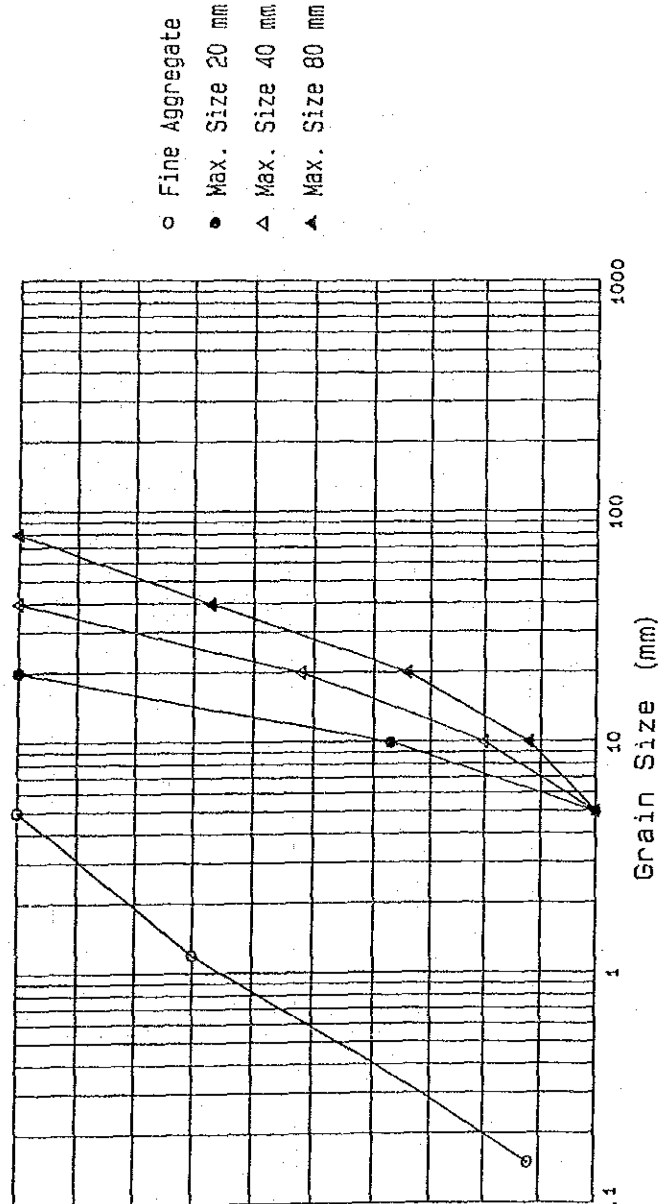
GRADING CURVES OF CONCRETE AGGREGATES	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT	
	JAPAN INTERNATIONAL COOPERATION AGENCY	

Fig. A3-2



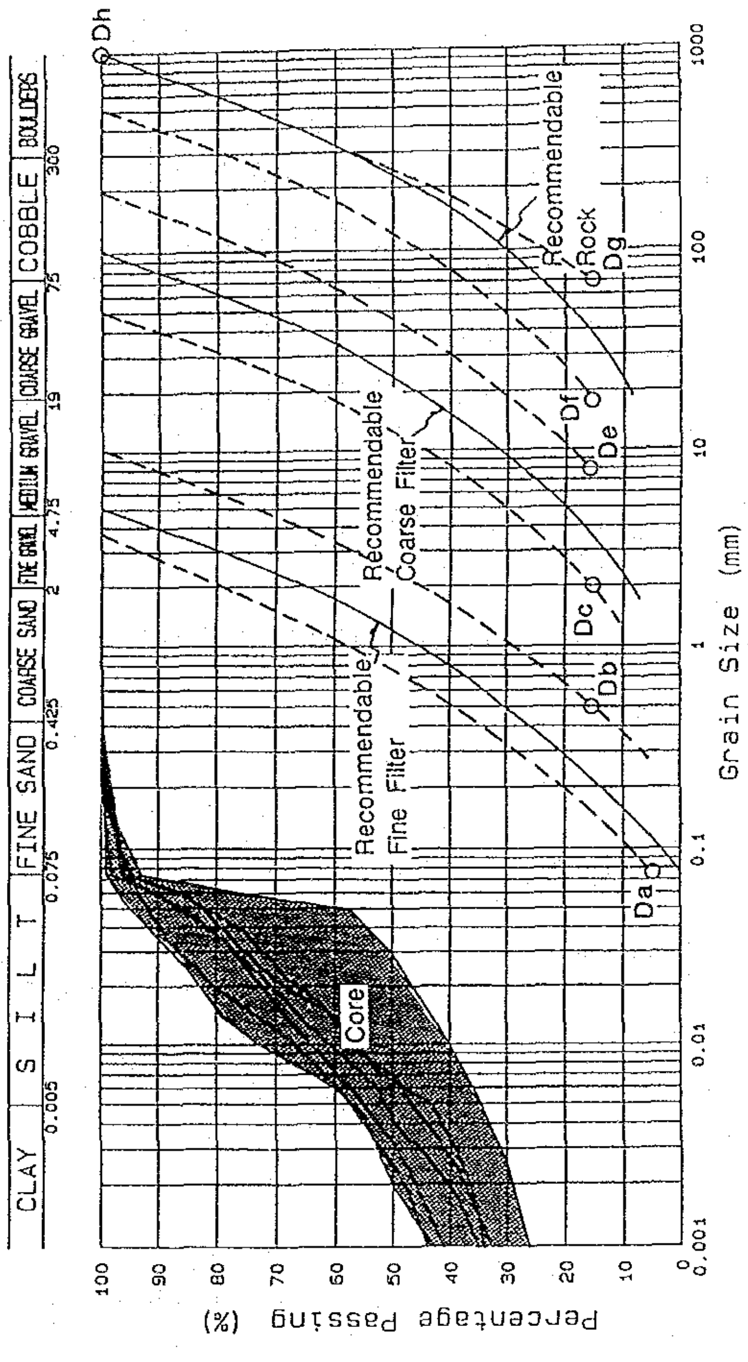
RELATION BETWEEN DISSOLVED SILICA AND REDUCTION IN ALKALINITY	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A3-3



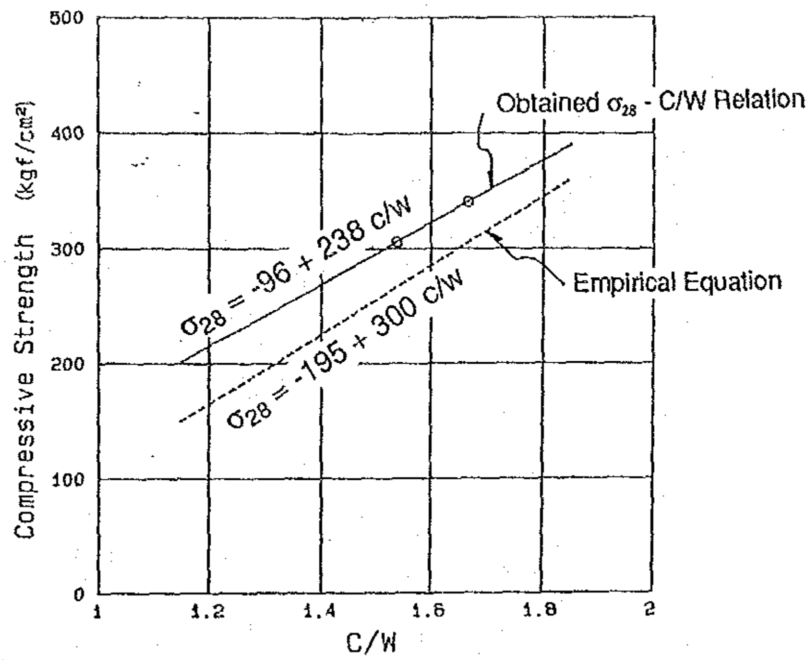
GRADING CURVES OF CONCRETE AGGREGATES FOR MIX PROPORTION TEST	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig.	A4-1

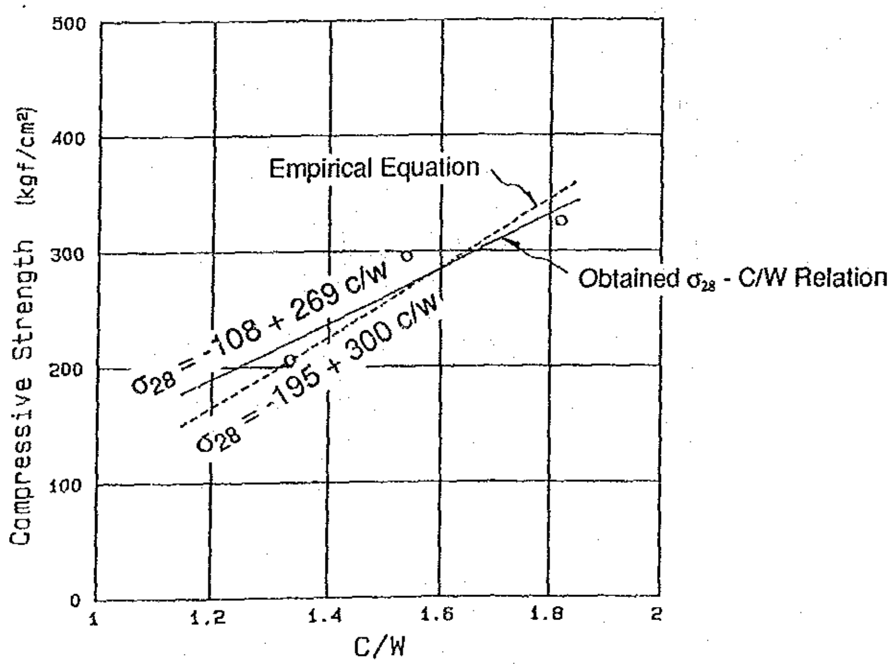


GRADING CURVES OF RECOMMENDABLE FILTER AND ROCK MATERIALS	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. A4-2



For Maximum Grain Size 20 mm



For Maximum Grain Size 40 mm

RELATION BETWEEN CEMENT WATER RATIO AND COMPRESSIVE STRENGTH	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

GEOLOGICAL INVESTIGATIONS

CONTENTS

Page

Introduction B-1

B1 Regional Geology B-2

B2 Site Geology B-2

B3 Geological Investigations B-4

B3.1 Concept of Geological Investigations B-4

B3.2 Geological Mapping B-6

B3.3 Core Drilling B-7

B3.4 Test Grouting B-7

B3.5 Test Adits B-8

B3.6 In-situ Rock Test B-10

B3.6.1 Location and Quantity of Test B-10

B3.6.2 Plate Loading Test B-10

B3.6.3 Shear Test B-12

B4 Foundation Engineering B-13

B5 Geology of Quarry Site B-15

PORT LOUIS WATER SUPPLY PROJECT REPORT ON GEOLOGICAL INVESTIGATION

Introduction

In the period from June 1990 to March 1991, a series of geological investigation has been performed for detailed design of the 80 metre high dam on the river Terre Rouge which was contemplated for the purpose of water supply to the city of Port Louis. Core drilling at thirteen locations, totalling 1069 metres in length, grouting test with four holes drilled to the depth of 60 metres respectively and four exploratory adits of 420 metres in total length have been completed in this period. Borehole Lugeon tests for evaluation of seepage potential of the bedrock have been made in the core drilling holes, and in-situ rock mechanics tests for shear strength and deformation modulus of rock have been carried out in the adits.

Further, 60 metres of core drilling have been performed in the quarry site, and other 120 metres have been done for foundation of a water treatment plant at Pailles.

These investigations have been planned referring to and to succeed the previous investigations made in the stage of the Feasibility Study of the Project. Geological maps and profiles of the previous investigation have been reviewed and revised in the light of new findings in the recent surface geological survey, core drilling and aditting. The drilling logs of the previous geological investigation are contained in an appendix to this report.

These investigations have revealed a portion of low piezometric head of groundwater in the plateau on the left bank of the dam site, and additional core drillings at three locations to the total length of 450 metres have been carried out from May to July 1991.

B1. Regional Geology

According to E.S.W. Simpson (1951), geology of the volcanic island of Mauritius is divided into two series of Older Volcanics and Younger Volcanics, and the latter is further divided into two phases, that is, Early Lavas and Late Lavas.

The Older Volcanic Series is alternation of basalt lavas and agglomerates, now composing steep dissected hills and mountains which are remnants of a large shield volcano of the ancient Mauritius. The Younger Volcanic Series, comprising basalt lavas and pyroclastic rocks, forms plateaus gently inclined toward the coasts. Early Lavas, a member of the Younger Volcanic Series, crops out only in the southwestern part of the island, while Late Lavas, the younger volcanic products originating in craters located along a lineation of NNE to SSW through the centre of the island, covers a large part of the island. The Late Lavas are largely composed of doleritic basalt, which are highly vesicular in parts, according to Simpson.

B2. Site Geology

The river Terre Rouge for the dam and reservoir forms a gorge dissected in the Lavas. Top of the lava plateau is at elevation 240 metres to 250 metres in the contemplated dam site, and the river bed is at elevation 120 metres. At the dam site, the river flows westward through a 30 metre wide channel. The river bank rises with steep slopes of 1 vertical to 1 or 1.25 horizontal up to the flat tops of the plateau. A 40 metre wide terrace is formed at elevation 140 to 150 metres on the left bank.

A gorge of Plains Wilhems joins the Terre Rouge from the left bank side at about 350 metres downstream of the dam site. With the Terre Rouge on the east to north and the Plains Wilhems on the west, the plateau on the left bank is as narrow as 400 metres to even 250 metres in the narrowest portion at the contemplated high water level of the reservoir, i.e. elevation 189 metres. When the reservoir is built up in the Terre Rouge valley, this thin plateau has to form a water-tight barrier against the empty Plains Wilhems. The bottoms of these two valleys are nearly at the same level on cross sections right angle to the axis of the plateau.

The bedrock is composed of a number of basalt lava flows, with thicknesses varying from a few to more than 10 metres, which are stratified sub-horizontal. The basalt is generally characterized by coarse plagioclase phenocrysts of about one millimetre in size among basic minerals, and hard and solid if it is fresh. It is often the case that the basalt is vesicular bearing many pores near the upper and lower boundaries of each flow. It is also frequently observed that the basalt is intensively weathered at the top into soft deteriorated rocks and dense or compact residual soil, namely "hard clay" in this report. Sometimes, signs of volcanic ash and old surface soils are seen in the hard clay. A sort of auto brecciated lavas, mingled with soil or volcanic ash are also observed in the boundary zones between two lava flows, as called flow breccias in this report.

A unit of lava flow is, accordingly, represented by a schematic sequence, in ascending order, of flow breccia, vesicular basalt, non-vesicular massive basalt, weathered vesicular basalt, highly weathered basalt and hard clay, though all the real lava flows do not always include all these members in their cycles.

Stratigraphic correlations, with the boundary features as marker beds, among all the drilling core samples of the dam site indicates wide horizontal development and continuity of each flow, though some of thin basalt beds may pinch out and do not endure for a long distance.

Encountered in lower horizons, or in levels lower than elevation 140 metres, are a 20 metre thick porphyritic basalt with relatively large plagioclase phenocrysts of 2 to 3 millimetres and the underlying cycles of fine grained basalt flows. The latter shows an aphanitic texture, that has too fine grained component minerals to be visible to naked eyes and appears to have no discrete crystalline units. Hence the name of glassy basalt in this report. It is also characterized by inclusion of fluorite and opal in pores.

Considering dominant existence of the similar sort of aphanitic basalt in Mount Ory, a contemplated quarry site, which obviously consists of the Older Volcanic Series for its geomorphological situation, the "glassy basalt" below the river bed of the dam site is deemed to fall under the same old series. In the other hand, the coarse basalts forming the dam abutments may be classified into the Late Lavas in the Younger Volcanic Series.

The above classification of the dam site geology in terms of the theoretical stratigraphy of Mauritius, however, may not be well proved. For the engineering geological purpose of this report, a tentative terminology of Old Lavas and Young Lavas will be applied.

A layer of intermingled basalt and hard clay, named the pyroclastic flow, which has been encountered by a few drill holes near the level of the river bed, is deemed to mark the base of the Young Lavas. An unconformity plane as the boundary of the Old Lavas and the Young Lavas is located between the pyroclastic flow and the underlying porphyritic basalt.

Accordingly, the stratigraphic sequence of the dam site is schematically tabulated as shown below:

Volcano-stratigraphic Sequence of Basalt Lava Flow at the Dam Site

Classification	Strata		Thickness (m)
	Young Lava 4	(YL4)	40
	Young Lava 3	(YL3)	20
Young Lavas	Young Lava 2	(YL2)	20
	Young Lava 1	(YL1)	40
	Pyroclastic Flow	(YL0)	0 - 10
	Unconformity		
	Old Lava 3 (porphylitic basalt)	(OL3)	5 - 20
Old Lavas	Old Lava 2 (glassy basalt 2)	(OL2)	20
	Old Lava 1 (glassy basalt 1)	(OL1)	?

No major fault has been found in the dam site and the reservoir area.

With the sub-horizontal bedding of the lava flows, the stratigraphy of the reservoir area is essentially similar to that of the dam site. Its major part is situated in the Young Lavas, because of the raised river bed higher than in the dam site.

B3. Geological Investigation

B3.1 Concept of Geological Investigations

Based on the geological investigation in the stage of Feasibility Study, including seismic refraction prospecting on 22 exploration lines with total length of 4,025 metres and core drilling at 17 locations with total length of 1,280 metres, it was clarified that a site selected on the river Terre Rouge, a few hundred metres upstream from the confluence of the river Plains Wilhems, is geotechnically viable for construction of a 80 metre high rockfill dam. The bedrock of basalt flows are sufficiently strong, even though frequently intercalating soft rock layers, and not highly pervious in general.

Since the period of Feasibility Study, comments and questions were issued through Central Water Authority on the geotechnical aspect of this dam site. A major question was on possibility of existence of such large cavities as lava tunnel which might provide significant passages for water leakage from the reservoir.

Existence of lava tunnel, which might be formed by rather accidental outflow of a part of still fluid lava in the course of its cooling and hardening, however, is not easy to prove or disprove, unless any lava tunnel be encountered by drilling. No geophysical exploration is capable to distinguish it with reliable clarity when it exists deep under the ground surface. Development of the groundwater table, or piezometric head, however, could be an index of any privileged water passage. So far as the boreholes drilled for the Feasibility Study are concerned, the groundwater levels were observed generally high in boreholes drilled to the depth of 120 metres from the tops of the plateaus on both banks, or to the level approximately similar to the river bed. This was deemed to deny the possible existence of lava tunnel.

Nevertheless, the question on lava tunnel was still the matter to be examined further, especially in connection with the thin plateau on the left bank of the dam site, which might provide leakage paths from the reservoir to the gorge of the Plains Wilhems even without lava tunnel. Geological investigations for the detailed design were planned with this question in view, with deep drilling at five locations from the top of the left bank plateau and the test adit for in-situ eye-observation of sub-surface geology.

This time, the drilling JD-12 on the left bank plateau at approximately 300 metres from the contemplated dam wing has recorded gradual draw-down of groundwater level along with the progress of the drilling until it reached to a level around elevation 137 metres when the 120 metre deep drilling completed. This local deep groundwater level appeared to be a part of a valley of groundwater observed also by JD-2 and JD-13. This could be an indication of local highly pervious zone running from the Terre Rouge valley of the reservoir to the Plains Wilhems valley. It was judged, however, from the slow drawdown of the water that this pervious zone could not be such a large opening as lava tunnel or lava tube, but only a local cracky or fractured zone.

At the same time, three additional drilling, JD-101, JD-102 and JD-103, have been carried out within the distance of 25 metres from JD-12, and confirmed the low piezometric head in the deep portion and the necessity of the rim grout curtain.
As a countermeasure to this, a design of a rim grout curtain of approximately 400 metres in the stretch into the left bank has been contemplated.

The length (lateral extent) of the grout curtain has been determined so that the curtain may intercept seepages in a zone where the river bed of the Terre Rouge upstream of the dam site, which runs parallel with that grout curtain, is not higher than elevation 160 metres. As the full supply level of the reservoir being at elevation 189 metres, there can be seepage from a 30 metre deep reservoir through under a 300 metre wide plateau without any cut-off work outside the extent of the rim grout curtain. The seepage within the range of 100 metres outside the rim curtain is estimated at only 0.3 liters every second, little enough to be acceptable, assuming 1 exp.-4 cm/sec of permeability coefficient and 30 metres for the range of seepage in depth. The hydraulic gradient is estimated to be 1/10 (30m/300m). The length of the rim grout curtain is, therefore, sufficient.

The depth of the rim grout holes, 60 metres and 40 metres, has been determined to satisfy the conditions :
(1) to reach the level of the nearest river bed or deeper
and ;
(2) To be deeper than the level of groundwater, actually observed and conceivable from the observations, including the part of groundwater table depression around the test hole J-12.

The grout hole spacing of 2 metres near the dam/spillway and 3 metres in the most part of the rim curtain has been determined empirically. The length of seepage paths through the plateau is larger (300m) than that under the dam body (approximately 60 m), and reverse is the hydraulic gradient. Requirement for the permeability in the rim can be so strict as in the dam site. The purpose of the rim grouting is to block outstandingly large and continuous water leakage paths, if any. The grout hole arrangement at 3 metre intervals is deemed appropriate and sufficient.

In the scene of the construction work, the grouting will be performed by a split-spacing method, in which the holes will be initially drilled and grouted at larger intervals (e.g. 6m) to examine by the grout-takes whether it is necessary to put other holes at the locations of 3 metre intervals. In this procedure, the grouting work quantity will be saved in zones appreciably watertight and may be added where it is necessary. In this context, the designed quantity will be sufficient with due spare.

B3.2 Geological Mapping

Ground surface geological mapping has been performed to produce a geological map of the dam site to the scale of 1/2000 and a geological map of the reservoir area to the scale of

1/2500. Information from core drilling and exploratory aditting have also been incorporated for completion of the geological maps.

B3.3 Core Drilling

Core drilling at sixteen locations, totalling to 1519 metres in length has been performed in the dam site and its left wing. Water level in each drill hole has been recorded every morning before commencement of the day's drilling work. Lugeon tests have been made by 5 metre stages in parts of the drill holes lower than the contemplated high water level of the reservoir, that is, elevation 189 metres.

In the quarry site at Mount Ory, core drilling has been made at three locations for total length of 60 metres.

Other core drillings have been carried out at six locations, totalling to 120 metres of length, for a water treatment plant at Pailles.

Details of the drilling quantity are shown in the attached List of Drilling, and the locations of the drilling are in the Location Map.

Results of the drillings are presented in Drill Logs attached to this report. While the bedrock forms an alternation of hard basalts and soft rocks, including flow breccias and hard clay, the core recovery has been generally high, indicating solidity or compactness of the soft rock.

Results of the Lugeon tests are presented in Record of Water Pressure Test attached to this report. In the dam site and the thin plateau on the left bank, Lugeon values observed have been not more than 40 in 93 percent of all the 116 sections tested, not more than 20 in 73 percent and not more than 10 in 52 percent. This indicates that the bedrock is not very highly pervious, and can be treated by ordinary cement grouting. It is also shown that the boundary zones between different lava flows are not especially pervious compared with other parts.

B3.4 Test Grouting

Test grouting has been carried out on the left bank terrace at elevation 146 metres. Three grout holes, GT-1, GT-2 and GT-3, have been drilled at positions of three corners of a regular triangle with the side length of 2.0 metres. The grout holes have been Lugeon-tested and grouted by 5 metre stages to the depth of 20 metres, and by 10 metre stages in the depth from 20 metres to 60 metres, in descending order. Then, a check hole GT-4 has been drilled at the centre of the triangle for Lugeon test and grouting to examine the effect of the foregoing grouting through the three grout holes. The results are shown in the attached Figure.

A very high Lugeon value of 103 and a large grout-take of 257 kg cement every metre have been registered in the third stage between 10 metres and 15 metres of depth in the hole GT-2, if not to mention even higher Lugeon values in the second stages which are partly through loose rocks near the ground surface. Other large grout-takes recorded have been 53 kg cement/m for a 17 Lugeon section and 49 kg cement/m for a 24 Lugeon section, both in the hole GT-1. The bottom stage between 50 metres and 60 metres, covering a boundary zone of lava flows has registered only one Lugeon and no grout-take.

The check hole GT-4 has shown Lugeon units less than 4.0 and very little grout take, providing a good effect of the grouting.

The result indicates that grouting through holes arranged on two lanes and at 2 metre intervals on each lane can be effective to improve permeability of the bedrock. The grouting pressure for this test has been:

Depth (m)	Maximum Pressure
0 - 5	-
5 - 10	2.0
10 - 15	4.0
15 - 20	6.0
more than 20	10.0

The conclusion can be applicable both for the curtain grouting and the blanket or consolidation grouting.

B3.5 Test Adits

Four test adits have been driven to abutments of the dam site; two on the right abutment and the other two on the left abutment, aiming at soft rocks in the boundary zone which could form weakest planes in the dam foundation. Exploration of privileged leakage paths, if any, has been one of the purposes of the aditting. The adits have been excavated in the direction parallel to the dam axis with branches right angle to it.

Quantitative details of the test adits are as shown in the attached List of Geological Investigations. The geological conditions observed in the adits are presented in Geological Sketchmaps of Test Adit.

Outlines of the adit geology are as described below:

- Adit No. 1 (El. 180 m, left bank)

  After passing through talus deposit in the initial 17 metre section, the adit follows a highly weathered rock zone at the top of Young Lava 2. The hard clay or poorly consolidated tuffaceous material in the upper horizon is encountered at the level of the adit at places because of gentle undulation of the sub-horizontally bedded lavas. So does the moderately weathered basalt underlying, too. As a whole, the adit lies in a same bed without significant variation from geotechnical point of view.

- Adit No. 2 (El. 150 m, left bank)

  This adit has been driven through another highly weathered rock zone at the top of Young Lava 1. Due to the gentle undulation of the weathered rock zone, less weathered hard rocks appear in the section further than 55 metres from the portal.

- Adit No. 3 (El. 140 m, right bank)

  This adit could not enter a lava flow boundary that had been aimed, and, instead, entered into the moderately to slightly weathered basalt. The intensive weathering in the initial 20 metre section is deemed to have developed from the present slope surface. The encounter at the station 64 metres with a hard clay bed rising up to the level of the adit is due to undulation of the sub-horizontal bed, reflecting a vertical geological change. Slow water seepage is wide-spread all over the adit.

- Adit No. 4 (El. 160 m, right bank)

  This adit constantly follows a boundary of hard rock and underlying highly weathered rock. This condition covers consistently almost entire section of the adit.

Discontinuity planes observed in the adits are joints and small shear planes with slickenside of inconsistent orientations. The joints are occasionally open within 10 millimetres, filled up with clayey material, but generally closed. Minor water seepages are seen in some of the Joints. The slickensides seem to be more reflecting movements inside the hard clay under the process of consolidation than any long stretched dislocations due to tectonic movement.

3.6 In-situ Rock Test

3.6.1 Location and Quantity of Test

In-situ rock mechanics tests for deformability and shear strength have been performed in the test adits. For these tests, a branch adit has been excavated at right angle to the main adit in each of Adit No. 1 to No. 3. In Adit No. 4, the tests have been done in the main adit. Allocation of test blocks and testing spots, with general geological sketches, are shown in Figures B-82 & B-83.

Items and quantities of the tests in each adit are as listed below:

Adit No.	Plate Loading Test	Shear Test
1	2 (vertical loading)	1 set (3 test blocks)
2	1 (horizontal loading)	1 set (3 test blocks)
3	1 (vertical loading)	1 set (3 test blocks)
4	2 (vertical loading) 2 (horizontal loading)	1 set (3 test blocks)

B3.6.2 Plate Loading Test

Plate loading tests that have been made in the adits are aimed at obtaining moduli of elasticity (Young's modulus) and deformation of rock, by observation of its deformation under increasing load within the limit not to reach collapse of the rock.

The test is based upon a formula presented below:

$$Et \text{ (or } D) = (1 - r^2)/2a \times dP/dw$$

where,

Et	:	Modulus of elasticity (Young's modulus)	kg/cm^2
D	:	Modulus of deformation	kg/cm^2
r	:	Poisson's ratio	

a	:	Radius of loading plate	cm
dP	:	Increment of load	kg
dw	:	Displacement caused by dP	cm

As shown in Figures of Loading Pattern, loading has been made in cycles of increasing and decreasing. The maximum load has varied from 7 tons to more than 30 tons, depending on apparent strength of rocks to be tested.

For calculating Et, a proportion between the increased load and displacement in a single loading cycle is taken for dP/dw, whilst D is calculated with dP/dw taken from a proportion of load and displacement through initial several cycles where the peak load has been raised cycle by cycle.

Load has been applied on a disc-type steel loading plate of 30 centimetres in diameter by use of a hydraulic test jack with the maximum loading capacity of 200 tons. Displacement has been observed with dial gauges and electric recorders to an accuracy of 1/1000 millimetres.

Results of the plate loading tests are as follows:

Adit No.	Test No.		Geology	Modulus	
				Elasticity Et (kg/cm^2)	Deformation D (kg/cm^2)
1	1	(V)	Highly weathered basalt	1,300	1,200
	2	(V)	Highly weathered basalt	1,500	1,200
2	1	(H)	Highly weathered basalt	870	540
3	1	(V)	Hard clay (red brown)	3,900	2,700
4	FB1	(H)	Highly weathered basalt	3,800	1,100
	FB2	(H)	Highly weathered basalt	2,000	730
	VB1	(V)	Slightly weathered basalt	57,000	40,000
	VB2	(V)	Slightly weathered basalt	53,000	32,000

Note: V: Vertical loading H: Horizontal loading
The highly weathered basalts are deteriorated into a condition of compact sandy clay.
The hard clay is consolidated to a condition of mudstone or soft shale.