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THE REPUBLIC OF PARAGUAY

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY

SUPPORTING REPORT

1. HYDROLOGY
2. GEOLOGY
3. INUNDATION AND FLOOD DAMAGE
4. URBAN PLANNING
5. RIVER AND DRAINAGE PLANNING
6. IMPLEMENTATION SCHEDULE AND COST ESTIMATE
7. PROJECT EVALUATION
8. ORGANIZATION AND MANAGEMENT

JANUARY 1987

JAPAN INTERNATIONAL COOPERATION AGENCY

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1. HYDROLOGY

SUPPORTING REPORT
ON
HYDROLOGY

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SUPPORTING REPORT
ON
HYDROLOGY

1. General

In accordance with the Scope of Works, this project's hydrological study basically covers the following items:

(1) Meteorological Study;
(2) Rainfall Analysis;
(3) Runoff Analysis; and
(4) Probable Discharge Calculation.

One of the purposes of the meteorological study is to identify the meteorological characteristics in the study area for the rainfall and runoff analyses. The ultimate purpose of the rainfall analysis is to obtain the necessary data for estimating runoff discharge. In this project, the data required are the rainfall intensity-duration curves under different probabilities to be converted into runoff discharge by means of the runoff calculation method. The model hyetograph used to provide the discharge hydrograph is also studied in the rainfall analysis.

The runoff analysis is meant to estimate the runoff discharge through the study on runoff elements such as runoff coefficient, concentration time, storage and infiltration capacity of the study area, and so on. The probable discharge, among the results of the hydrological study obtained through the rainfall and runoff analyses, is essential for the formulation of a suitable storm water control plan.

Aside from the above study, several hydrological equipment have been installed to collect supplemental hydrological data and infiltration tests were conducted to know the permeability of the ground in the study area. The results are shown and in ANNEXES 1, 2 and 3.

2. Previous Study

The construction of storm water drainage system in Asuncion City started in 1975. Prior to the constructions works, several studies including hydrological study were made and the results are compiled in the following reports:

- Prevision de Lluvias Criticas Para la Ciudad de Asuncion by Engr. Nasser, 1957.
- Master Plan Study for a Storm Sewer System in Asuncion City, 1967.
- Feasibility Study on Storm Drainage Project (Second Stage), 1975.

The hydrological study results of the above reports are summarized as follows:

<u>Report by Engr. Nasser in 1957</u>

This report was prepared by Engr. Nasser solely to provide the rainfall intensity-duration curve in Asuncion City. The study was made in the following procedure:

- Rainfall data at Armada Station from 1928 to 1944 were collected in the form of recording charts.
- Annual maximum rainfalls of short duration such as 5 min., 10 min., 15 min., etc., were obtained from the recording charts.
- The probable rainfall intensities of 2, 5 and 10-year return periods for each duration were then obtained from the probable paper where the annual maximum rainfalls were plotted in the order of magnitude of the rainfall intensity.
- By applying the least square method to the relation between the probable rainfall intensity and the duration such as 5 min., 10 min., 15 min., etc., the equation for the rainfall intensity-duration curve was formulated for the said return periods.

Report on Master Plan Study in 1967

In the master plan study, both rainfall analysis and runoff analysis covering an area of about 3,400 ha in Asuncion City were made. Although the study procedure was not mentioned in detail, the following results were obtained:

(1) Rainfall Analysis

In this study, the main purpose of the rainfall analysis was to plot the rainfall intensity-duration curve in the same procedure made by Engr. Nasser, but the rainfall data from 1929 to 1966 were applied to the analysis. As a result, the rainfall intensity-duration curve for 2 and 15-year return period probabilities are compiled in the report (refer to Fig. 1-8.).

(2) Runoff Analysis

For the analysis, the study area of about 3,400 ha was divided into 29 basins. Runoff coefficient and concentration time in each basin were analyzed on the basis of the observation data obtained during the study period. The results show that the runoff coefficients of most of the basins range between 0.6 and 0.7, while the concentration time is between 15 minutes and 20 minutes.

Since the rainfall intensities between 80 mm/h and 100 mm/h are given by the rainfall intensity-duration curve of 2-year return period, the runoff discharge of the return period calculated by the rational formula amounts to the specific discharge of more than 10 $m^3/s/km^2$.

Report on Feasibility Study in 1975

The feasibility study on an area of 510 ha selected from the area covered by the master plan study of 1967 also included both rainfall analysis and runoff analysis, as follows:

(1) Rainfall Analysis

As in the rainfall analysis under the master plan study of 1967, the feasibility study focussed on the creation of a rainfall intensity-duration curve. The newly created curve was based on the rainfall data from 1929 to 1944. The data from 1945 which have been included in the master plan study were excluded because of poor reliability.

The procedure was similar to that made by Engr. Nasser, i.e., the selection of annual maximum rainfalls of short duration, the estimation of the probable rainfall for several return periods, and the formulation of the equation for the rainfall intensity-duration curve. The difference lies in the way of estimation of the probable rainfall; the logarithmic paper was used in this method while Engr. Nasser utilized the probable paper. (Refer to Fig. 1-8.)

(2) Runoff Analysis

In this study, the rational formula was applied to the calculation of the design discharge for the planning of the drainage pipe for the 510-ha area. In this connection, the runoff analysis was performed to obtain a suitable value for the formula; namely, runoff coefficient and concentration time. The following runoff coefficients were applied to the calculation:

- Central Commercial Zone : 0.70
- Residential Area and Minor Commercial Zone : 0.60

As for the concentration time (Tc), it was further classified into the initial concentration time (Tci) and the time of running in a drainage channel (Tcr), as presented in the following formula:

$$Tc = Tci + Tcr$$

Ten (10) minutes was applied as initial concentration time and the following equation was used in the estimation of the time of running in a drainage channel:

$$Tcr = \frac{L}{60V}$$

where, L : Length of a channel or a pipe (m)
V : Velocity (m/sec)
Tcr : Time of running in a drainage channel (min)

Probable discharge was calculated through the foregoing studies. The results show that the specific discharge of the probable discharge thus obtained in the area with a 5-year return period amounts to about 20 $m^3/s/km^2$.

3. Meteorology

3.1 Observatory

At present, there exists in the study area only one meteorological station, Airport Station, that started observation in 1971. The observation items of this station include rainfall, temperature, humidity, wind direction, wind velocity, atmospheric pressure, and so on.

Meteorological data had also been collected during specific periods at other three stations: Armada Station from 1929 to 1964, Sajonia Station from 1965 to 1976, and San Lorenzo Station from 1957 to 1980.

As for the observation data of short duration rainfall which are essential for estimating runoff discharge, data from all the stations are available. It was noted, however, that although there are two kinds of recording charts for the gauge with a daily roll and a weekly roll as shown in Fig. 1-1, rainfall intensity for short duration is hardly obtainable from the recording chart of a weekly roll because of the short interval of a recording paper for one hour. Therefore, data on the intensity for short duration rainfall were extracted from the available data as follows: Armada Station from 1929 to 1944, Sajonia Station from 1965 to 1973, and San Lorenzo Station from 1972 to 1981.

The recording periods of these stations and their location are shown in Fig. 1-1 and Fig. 1-2.

3.2 Meteorological Characteristics

The study area is climatically situated in the subtropical zone, and a year is officially divided into four seasons: Spring from September 21 to December 21, Summer from December 22 to March 20, Autumn from March 21 to June 20, and Winter from June 21 to September 20.

According to the observation data at the Airport Station, for the recent five years the average monthly temperature ranged from 17°C to 28°C and the maximum temperature during these five years was 39°C, while the minimum marked 0°C. (Refer to Table 1-1 and Fig. 1-3.)

Average humidity which fluctuates between 60% and 80% shows a low value in September-October and a relatively high value in May-June. Although wind velocity in the study area averages at around 5.0 km/hr without showing any marked fluctuation by season or direction, wind directions are predominantly North, East and South throughout the year. (Refer to Tables 1-2 and 1-3 and to Figs. 1-3 and 1-4.)

Evaporation at San Lorenzo from 1976 to 1980 shows a mild fluctuation within the range between 2.0 mm/day and 3.5 mm/day throughout the year. (Refer to Table 1-4 and Fig. 1-3.)

The annual average rainfall in the study area is approximately 1,400 mm. Light rainfall usually occurs from June to September and heavy rainfall from November to April, though no pronounced wet-dry cycle can be seen. (Refer to Table 1-5 and Fig. 1-5.)

Rainfall is basically caused by the mingling of hot humid winds from the North Brazilian Grosso, famous for its high humidity, and the cool dry winds from the south. Besides, the local rainfall is sometimes brought by a cumulonimbus in hot seasons.

4. Rainfall Analysis

4.1 Rainfall Intensity-Duration Curve

Study Procedure

As described in the preceding section, the rainfall analysis in this study focussed on the provision of the rainfall intensity-duration curve. As in previous studies, the analysis was done in the following procedure:

- Selection of annual maximum rainfall intensity for short duration rainfall.
- Estimation of the probable rainfall for short duration rainfall.
- Derivation of the equation for the rainfall intensity-duration curve.

Selection of Annual Maximum Rainfall

As for the available data for the selection of annual maximum rainfall for short duration rainfall such as 5, 10, 15, 20 minutes, etc., it is basically required that rainfall data are recorded by an automatic rainfall gauge with a daily roll as aforementioned. Based on the rainfall observation records, the stations having the pertinent data are as follows:

- Armada Station : 1929 to 1944
- Sajonia Station : 1965 to 1973
- San Lorenzo Station : 1972 to 1981

Since none of these stations can individually supply sufficient data in a coherent manner, their combined data were used in the selection of the annual maximum rainfall data due to the following reasons:

- Correlation among the daily rainfall records of these stations are considerably high in case of big storm rainfalls, as verified from the fact that the recording of annual maximum rainfall in these stations often occurred on the same day.

- It is always desirable that the recording period for the data concerned be sufficiently long from the statistical point of view.

Table 1-6 shows the annual maximum rainfall selected from the said combined data.

Probable Rainfall

Probable rainfall can be obtained by applying Gumbel Formula to the selected annual maximum rainfall. The formula is expressed by the following equation:

$$P_x = 1 - \exp(-e^{-y})$$

$$x = x_o + \frac{1}{a}(y)$$

where;

P_x	:	Probability of excedance
x	:	Probable rainfall (mm)
y	:	Standard extreme value
x_o, a	:	Constants based on the rainfall data

Table 1-7 and Fig. 1-6 show the probable rainfall thus obtained.

Rainfall Intensity-Duration Curve

The following equation is generally used to express the rainfall intensity-duration curve:

$$R = \frac{a}{t + b}$$

where;

R	:	Rainfall intensity (mm/hr)
t	:	Rainfall duration (min)
a, b	:	Constants derived from the rainfall data

Here, constants a and b can be obtained by the least square method expressed in the following equations:

$$a = \frac{(R)(R^2 \times t) - (R^2)(R \times t)}{(R)^2 - N(R^2)}$$

$$b = \frac{N(R^2 \times t) - (R)(R \times t)}{(R)^2 - N(R^2)}$$

where;

R	:	Rainfall intensity (mm/hr)
t	:	Rainfall duration (min)
N	:	Number of rainfall intensity data
(R)	:	ΣR
(R2)	:	ΣR^2
(R x t)	:	ΣRt
(R^2 x t)	:	$\Sigma R^2 t$

The rainfall intensity-duration curves for several return periods were obtained by applying the above equations to the probable rainfalls. (Refer to Table 1-8 and Fig. 1-7.)

Fig. 1-8 shows the comparison of the rainfall intensity-duration curves obtained in this study and in previous studies for reference.

4.2 Model Hyetograph

A model hyetograph which is necessary for calculating a discharge hydrograph was provided under the following conditions:

- The model hyetograph is composed of 5 min. rainfalls.
- The model hyetograph is set up with the rainfall duration of 24 hours.

Calculation of 5-Minute Rainfall

Rainfall of R(5), R(10), R(15), R(20)...R(t), R(t+5) for the duration time of 5, 10, 15 and 20 minutes are given by the equation for rainfall duration-intensity curve.

Rainfall of every five minutes (r_n) are given by the difference between $R(t+1) - R(t)$, as follows:

$$R(t) = \frac{a}{t + b} \times \frac{t}{60}$$

$$r_n = \frac{a}{t(n+1) + b} \times \frac{t(n+1)}{60} - \frac{a}{t(n) + b} \times \frac{t(n)}{60}$$

[n = 1, 2, 3, ...]
[t(1) = 5; t(2) = 10; t(3) = 15; t(4) = 20 min...t(n) = n x 5 min]

where;

a, b	:	Constants of Rainfall Duration-Intensity Curve Equation
t(n), t(n+1)	:	Rainfall duration with 5 min. intervals
r_n	:	Rainfall for every 5 min.
R(t)	:	Rainfall during (t) min.

<u>Arrangement of 5-Minute Rainfall</u>

Arrangement of 5-min. rainfall is made by the following procedure:

- Rainfall number one (r_1) is placed at the center of the model hyetograph.

- Odd-numbered rainfalls (r_3, r_5, r_7, etc.) are placed in sequence to the left of center rainfall (r_1).

- Even-numbered rainfalls are placed in sequence to the right of center rainfall (r_1).

Fig. 1-9 shows the obtained model hyetographs with the return periods of 3 and 10 years for reference.

5. Runoff Analysis

5.1 Contents of the Study

In connection with the purpose of this project, runoff calculation in this study was conducted in the following cases:

<u>For the Basic and the Master Plan Study</u>

(1) Runoff calculation for probable discharge under past and present river basin condition;

(2) Runoff calculation for probable discharge under future river basin condition; and

(3) Runoff calculation for probable discharge in case detention facilities are provided.

<u>For the First Stage Project</u>

(1) Runoff calculation for probable discharge for subdivided basin in Mburicao and Itay river basins.

(2) Runoff calculation to estimate the storage capacity of the proposed retarding basin.

In this regard, the following studies on runoff analysis were made for calculating the probable discharge of the aforesaid cases:

- Study of runoff calculation method;
- Study of runoff elements such as runoff coefficient, concentration time, etc.;
- Study of effect of storage facilities for runoff discharge; and
- Study of effect of infiltration facilities for runoff discharge.

5.2 Observed Data on Runoff Analysis

Since the observed data on runoff analysis in the study area are hardly obtained because of poor runoff observation network, additional hydrological runoff stations consisting of one automatic rainfall gauge, two automatic water level gauges and eleven manual water level gauges were installed during the basic and master plan study period in the study area (refer to Annex 1). As a result, the hydrological data on several storms were obtained, as shown in Fig. 1-10 and Table 1-9.

5.3 Target Area for Runoff Analysis

The study area covering about 415 km^2 was divided into 31 basins which were further classified into two groups: one having specific river channels of 20 basins and the other having no specific river channels of 11 basins.

Runoff discharge in basins having no specific river channels are not necessary to be identified basinwisely, but it is necessary to calculate the runoff discharge basinwisely in other basins to study the suitable storm water control facilities. In this connection, the runoff analysis for the basic and master plan study was focussed on the basins with river channels to estimate the probable discharge. Fig. 1-11 shows the runoff calculation model for the 20 basins with river channels which were further subdivided into subbasins in connection with the storm water control plan.

As for the First Stage Project, the study area was narrowed down from the 20 basins to Maburicao and Itay river basins. In this connection, the runoff calculation model for these two basins was prepared anew for the newly subdivided basins in the feasibility study. (Refer to Fig. 1-14.)

5.4 Calculation Method for Runoff Discharge

There are several methods to calculate runoff discharge such as the rational formula, the unit hydrograph method, and the storage function method. The rational formula is generally used for a small river basin especially when few runoff discharge data are available. Since the rational formula can provide only peak discharge, the unit hydrograph method or the storage function method, though they require adequate observed data to justify their use, is applied when a flood hydrograph is needed to evaluate the regulation effect of flood control facilities such as retarding basin, detention facilities, etc.

In this study, a peak runoff discharge together with the flood hydrograph needs to be provided for the study of the effect of retarding basin and detention facilities. The unit hydrograph method was used to convert the model hyetograph into the flood hydrograph, and since a unit hydrograph was hardly obtainable due to the insufficient discharge data, the following equation based on the rational formula was applied to draw the unit hydrograph:

$$Q_t = K \frac{1}{tc} \int_{t-tc}^{t} r_t \times dt$$

$$K = 1/3.6 \times f \times A$$

where,

Q_t	:	Runoff discharge at the time of t (m^3/s)
r_t	:	Rainfall depth at the time of t (mm/hr)
tc	:	Concentration Time (min)
f	:	Runoff Coefficient
A	:	Catchment Area

In case detention facilities and retarding basin are provided, the said flood hydrograph which corresponds to the inflow discharge for the facilities and the basin is regulated and converted to outflow discharge. The storage function method, which is reliable to assess the effect of the detention facilities and the retarding basin, is applied to the calculation of the outflow discharge converted from the inflow discharge. This method is expressed by the following two equations, and the outflow discharge at the voluntary time (t) is given by the combination of these equations.

Continuity Equation : $Q_i - Q_o = \frac{ds}{dt}$

Dynamic Equation
(Storage Function) : $S = K \times Q_o^p$

where,

Q_i	:	Inflow Discharge to the objective stretch (m^3/s)
Q_o	:	Outflow Discharge from the objective stretch (m^3/s)
S	:	Storage Volume in the objective stretch (m^3/s)
K, p	:	Constants based on the relation between storage volume and outflow discharge in the objective stretch

5.5 Runoff Coefficient

Runoff Coefficient Based on the Observed Data

In case sufficient observed data can be obtainable, runoff coefficient can be derived from the relationship between the maximum rainfall intensity within the time of concentration and the maximum discharge on the basis of the following rational formula:

$Q = 1/3.6 \times f \times R \times A$

$f = 3.6 \times Q/R \times A$

where,

Q	:	Maximum Discharge (m^3/s)
R	:	Maximum Rainfall Intensity (mm/hr)
A	:	Catchment Area (km^2)
f	:	Runoff Coefficient

On the basis of the observed data in Mburicao and Itay river basins, the runoff coefficient was calculated at about 0.5 and 0.2, respectively. (Refer to Table 1-9.)

Runoff Coefficient Based on the Commonly Used Value

According to the study results concerning the land use pattern, the study area is basically composed of the following land categories; namely, commercial area, high density residential area, medium density residential area, low density residential area, park, public facility, industrial area, farmland, and unused land. The data on runoff coefficient are not sufficient in the study area, so that the coefficients that are commonly used with land use factors such as roof, road, interspace, park, etc., were applied to this study (refer to Table 1-10). Thereupon, the runoff coefficients for land use categories of commercial area, the three types of residential areas, industrial area and public facilities where runoff coefficient may vary widely due to the ratio of each land use factor were estimated on the basis of the factorwise proportion prevailing in several sample areas. (Refer to Table 1-11.)

The average runoff coefficient of the basin was calculated by summing up the runoff coefficients of all the areas according to their land use pattern. The equation used for this purpose is as follows:

$$f = \frac{(A_1 \times f_1) + (A_2 \times f_2) + (A_3 \times f_3) + \ldots .(A_8 \times f_8)}{A}$$

where,

f	:	Runoff Coefficient of the basin
A	:	Area of the basin
$A_1 \ldots A_8$	:	Area of each land use category in the basin
$f_1 \ldots f_8$	:	Runoff Coefficient of each land use category

The runoff coefficient thus obtained under the present land use conditions is shown in Table 1-12.

Adequacy of the Estimated Runoff Coefficient

Comparison between the runoff coefficients based on the observed data and the commonly used value shows that both values for Mburicao River Basin relatively coincide with each other, while those for Itay River

Basin show a considerable difference. This difference is attibuted to the fact that the observed runoff discharge in Itay River Basin is much less, due to the flood regulation effects of the wide inundation area in this basin, than the value estimated on the assumption that the river channel can confine all the runoff discharge without inundation.

The runoff coefficient obtained in Mburicao River Basin which is more reliable to examine the accuracy has justified the runoff coefficients based on the commonly used value in estimating runoff discharge.

Runoff Coefficients Under the Past and the Future Land Use Conditions

As mentioned in the Urban Planning Sector, the study area has been rapidly developing; thus, resulting in the change of land use pattern. Under this condition, the study on land use pattern were made for three cases: Past (1965), Present (1984), and Future (2005). According to this study on land use pattern, the runoff coefficients were also calculated for the past and future land use conditions. (Refer to Table 1-12.)

5.6 Concentration Time

Concentration Time Based on the Observed Data

Concentration Time (Tc) can be generally detected from the time difference (Tg) between the peaks of hyetograph and hydrograph, i.e., Tc = 2Tg. Based on the observed data, the time difference ranges from zero (0) to sixty (60) minutes as shown in Table 1-9 and Fig. 1-10. This may be due to the transition of the hyetal region which may sometimes bring about a considerable difference in time of the peaks of hyetograph, depending on the location of the basin.

From the above condition, concentration time is hardly detected on the basis of the observed data and it is desirable to collect more data in the future, to find the accurate value.

Concentration Time Based on Experimental Measures

Among the experimental measures for the calculation of concentration time (Tc), the way of accumulating the time of initial concentration (Tci) and the time of running in a drainage channel and pipe (Tcr) is commonly used. The former, Tci, is the time needed for storm water to reach an inlet of the terminal of the drainage facility and the latter, Tcr, to reach the outlet of the basin after running through the drainage channel and pipe. Concentration time (Tc) can then be summed up in the following formula:

$$Tc = Tci + Tcr$$

Concentration time, however, is subject to changes in rainfall intensity, and the aforementioned measure can hardly express such changes. In this study, therefore, the following equation is applied to estimate the concentration time in which the change in time is taken into account:

$$Tc = c \times A^{0.22} \times r_e^{-0.35}$$

where,

Tc	:	Concentration Time (min)
A	:	Catchment Area of basin (km^2)
r_e	:	Average effective rainfall during concentration time (mm/h)
c	:	Coefficient

In the foregoing equation, the average effective rainfall during the time of concentration is calculated by trial and error using assumed concentration time that should coincide with the concentration time calculated by the equation.

As for coefficient "c", it is desirable to confirm an adequate value based on the observed rainfall and discharge data. In case a few observed data is available, the values from 60 to 120 are generally applied to urban areas, depending on the development stages.

In this study, the following coefficient values were applied considering the development condition of the target area:

- Past Condition	:	120
- Present Condition	:	100 or 120
- Future Condition without Drainage Facility	:	100
- Future Condition with Drainage Facility	:	80

The concentration time calculated by the above equation are shown in Table 1-14.

5.7 Effect of Storage Facilities

In this subsection, the effect of one unit of storage facilities is studied.

Study Procedure

As mentioned elsewhere in the sector of River Channel and Drainage Planning of this Supporting Report, among several types of storage facilities, two types are proposed as applicable; namely, storage in public compounds consisting of parks and public facilities and in house lots. In this study, the effect of one unit of these two types of facilities are investigated and the results are used to calculate the probable discharge in the case of providing facilities in each basin.

Storage facilities contribute in regulating the flood discharge by converting the high peak discharge with a sharp hydrograph from the basin to the low peak discharge with a gentle hydrograph due to the storage function. The effect of the storage facilities, if they are provided in the basin, can be known through the comparison of the peak discharge with and without the facilities.

The study was made through the procedure outlined hereunder:

- Study of the dimension of the storage facilities.

- Study of the relation between the storage volume and the outflow discharge (storage function).

- Calculation of the outflow discharge hydrograph.

As for the inflow discharge hydrograph into storage facilities, the calculation results converted from the model hyetograph by the unit hydrograph method are used, as mentioned before.

Study on the Dimension of Storage Facilities

The dimension of the storage facilities is governed by the area, allowable water depth and shape of the outlet.

(1) Storage in Public Compounds

When a public compound is considered for use as storage facilities, the area will be dug up to some depth. Since public compounds have their own specific facilities, it is difficult to utilize all of their areas as storage facilities area. Therefore, it is assumed that 50% of the area is available for the area of the storage facilities.

According to the survey results of the urban planning, the area of public compounds is about one hectare on an average, and the available area for the storage facilities will be 0.5 ha. The allowable water depth of 30 cm is applied to the facilities to avoid accidents to promenaders and/or children caused by deep inundation water in these areas.

As for the shape of the outlet, one orifice of 30 cm deep by 30 cm wide will be provided.

(2) Storage in House Lots

As storage in house lots, it is proposed that a water tank with a small outlet hole to collect rainfall runoff from rooftops be provided in a garden. Although the dimension of the tank may be freely selected, two cases of water tanks are proposed in this study for convenience: one of 1.0 m^3 capacity and the other, 2.0 m^3.

The size of the outlet which is decided on the basis of inflow discharge into the tank and the available capacity of the tank has to be changed in view of the probable discharge applied. As a result, the diameter of from 2.0 cm to 3.0 cm will be employed for the probable discharge of from 3 to 10-year return periods.

Study on the Relation Between Storage Volume and Outflow Discharge of Storage Facilities

For certain water depths of the storage facilities, the relation between storage volume and outflow discharge is given in the following equations:

$$V = A \times h \qquad \text{Eq. 1}$$

$$Q = a \times c \times \sqrt{2g \times h} \qquad \text{Eq. 2}$$

where,

V	:	Storage Volume (m^3)
A	:	Available storage area (m^2)
h	:	water depth of detention facility (m)
Q	:	Outflow discharge from outlet (m^3/s)
a	:	Area of outlet (m^2)
c	:	Coefficient of orifice (0.6)
g	:	Acceleration of gravity (9.8 m/s^2)

By substituting Eq. 2 to Eq. 1, the following equation can be obtained:

$$Q^2 = a^2 \times c^2 \times 2g \times \frac{V}{A} \qquad \text{Eq. 3}$$

or,

$$V = \frac{A}{a^2 \times c^2 \times 2g} \times Q^2$$

On the other hand, the storage function method expresses the above relation by the following equation:

$$S = KQ^{p} \qquad \text{Eq. 4}$$

where,

S	:	Storage Volume (= V/3600; m^3/s)
Q	:	Outflow discharge (m^3/s)
K, p	:	Constants of storage function

From Eq. 3 and Eq. 4, the constants can be derived in the following equations:

$$K = \frac{A}{a^2 \times c^2 \times 2g \times 3600}$$

$$p = 2.0$$

(1) The Value of "K" of Storage Facilities in Public Compounds

In the above equation, the K value is calculated by substituting the dimensions of the facility; namely, A = 5000 m^2, a = 0.09 m^2, and c = 0.6, as follows:

$$K = \frac{5000}{0.09^2 \times 0.6^2 \times 2 \times 9.8 \times 3600} = 24$$

(2) The Value of "K" of Storage Facilities in House Lots

Similarly, the K value of storage facilities in house lots is calculated on the basis of the said dimensions in case of outlet diameter of 2 cm: A = 1.0 m^2, a = $0.01^2 \times 3.14$ m^2, and c = 0.6.

$$K = \frac{1}{(0.01 \times 0.01 \times 3.14)^2 \times 0.6^2 \times 2 \times 9.8 \times 3600} = 40$$

Calculation of Outflow Discharge Hydrograph

The outflow discharge hydrograph is calculated by the storage function method which is applied to the inflow discharge hydrograph given by the unit hydrograph method beforehand. As the result, the effect of one unit of the storage facilities is described as follows:

(1) Effect of Storage Facilities in Public Compounds

The effect of one unit of storage facilities in public compounds can be known by the difference in the peak discharges between the inflow and outflow hydrographs for the storm water falling inside of the facility, as tabulated in the following:

Return Period	Peak Discharge (m^3/s) Inflow	Outflow	Regulation Effect (m^3/s)	Storage Water Stage (cm)
3-year	0.38	0.07	0.31	8.0
10-year	0.51	0.09	0.42	14.0

In this case, only 8.0 cm and 14.0 cm out of 30 cm of allowable depth corresponding to the storage capacity of 27% and 47%, respectively, are used for 3 and 10-year return periods and, therefore, it is possible to collect the storm water not only inside of the area of the facilities but also that of the surrounding area in order to use the capacities of the storage facilities effectively.

Several cases of calculation were made assuming the area to collect storm water, so that the storage water stage may coincide with the allowable depth of the facilities. The possible maximum catchment area of storage facilities is presented in Table 1-13.

Eventually, the maximum inflow discharge of 1.0 m^3/s can be regulated to the maximum outflow discharge of 0.13 m^3/s by using the full storage capacity regardless of the probability. From the above, the regulation effect of one unit of the facilities is regarded as 0.87 m^3/s. (Refer to Fig. 1-12.)

In this connection, when the regulation effect of 0.87 m^3/s is evaluated in the form of rainfall depth for the area of 2.8 ha which is the possible maximum catchment area in case of 3-year return period, this effect corresponds to 110 mm/hr.

(2) Effect of Storage Facilities in House Lots

Since a house in a lot has a roof with the area of 100 m^2 on an average, it is assumed that one unit of the storage facilities in a house lot, which is designed to regulate the storm water from the roof, should collect the storm water for the area.

For this inflow discharge, the effect of the storage facilities with a full storage capacity is expected as follows:

Return Period	Size of Facility (m^3)	Maximum Discharge (m^3/s)		Regulation Effect (m^3/s)
		Inflow	Outflow	
3-year	1	0.0038	0.0026	0.0012
10-year	1	0.0051	0.0039	0.0012
3-year	2	0.0038	0.0018	0.0020
10-year	2	0.0051	0.0031	0.0020

The regulated discharges of 0.0012 m^3/s and 0.0020 m^3/s correspond to 43 mm/h and 72 mm/h in a form of rainfall depth for the area of 100 m^2. (Refer to Fig. 1-13.)

5.8 Effect of Infiltration Facilities

As in the study on storage facilities, the effect of one unit of infiltration facilities is studied and the result is used to know the total effect of this facility for the formulation of storm water control in each basin. Prior to this study, field investigation was performed to know the infiltation capacity of the study area. It was confirmed that the coefficient of infiltration capacity (a_p) in case of using trench-type facilities would be 0.76 ltr/hr which is used in the following equation:

$$Q = 10^3 \times a_p \times Hd \times A_p$$

where,

Q : Infiltration discharge (ltr/hr)

a_p : Coefficient of infiltration capacity (ltr/hr)

H : Water depth (m)

A_p : Bottom area of infiltration facilities (m^2)

Based on the results, the effect of the infiltration facilities on the runoff discharge is studied under the following conditions:

- The trench-type of infiltration facilities is employed in this study.

- Trench will be provided only for residential areas to collect storm water from rooftops, since water from areas such as roads, gardens, parking areas, etc., may remarkably deteriorate the function of an infiltration facility due to trash flowing with the storm water.

- Trench lengths of 10 m and 20 m to be provided to each house are employed and the height of 0.6 m and width of 0.6 m are adapted to the trench size, considering the previously executed system in other countries.

Under this condition, the infiltration capacity of the trench per one meter of the length is given by the following equation:

$$
\begin{aligned}
Q &= 10^3 \times a_p \times Hd \times A_p \\
&= 10^3 \times 0.76 \times 0.6 \times 0.6 \times 1.0 \text{ m} \\
&= 274 \text{ ltr/hr/m}
\end{aligned}
$$

The effect of 10 m long infiltration facilities for the storm water falling on a 100-m^2 roof of one private house is calculated as follows:

Return Period	Maximum Discharge (m^3/s)		Regulation Effect (m^3/s)
	Inflow	Outflow	
3-year	0.00381	0.00305	0.00076
10-year	0.00510	0.00434	0.00076

The regulation effect of 0.0008 m^3/s coincides with 27 mm/hr in a form of rainfall depth for the area of 100 m^2. Similarly, a

20-m long trench has the regulation effect of 0.0015 m^3/s (54 mm/hr).

6. Probable Discharge for the Basic and the Master Plan Study

Probable discharge for the basic and the master plan study covering the planning area was calculated on the basis of the rainfall and runoff analyses described in the foregoing section. As to the probability of calculation, 1.1, 2, 3, 5 and 10-year return periods were applied in relation to the project scale to be employed for the study of storm water control facility. Although the master plan is formulated at the target year of 2005, the probable discharge was calculated including the cases under the past and the present river conditions for reference.

6.1 Discharge Under Past and Present River Condition

Probable discharges under the past and present river conditions were calculated on the basis of the land use pattern in 1965 and as of 1984. Table 1-14 shows the calculation results for each basin as Case 1 and Case 2.

According to the results, the specific discharge of the probable discharge in case of 3-year return period amounts to between 5 and 20 $m^3/s/km^2$ and during the 20 years from 1965 to 1984, the runoff discharge has increased the range from 20% to 50% because of the city development.

6.2 Discharge in the Future River Condition

Probable discharge in the future river condition was calculated on the basis of the presumed land use pattern in the year 2005.

As for the provision of a storm water drainage facilities which may affect the shortening concentration time of storm water resulting in increment of peak discharge, the following two cases were applied: with and without storm water drainage facilities. Table 1-14 shows the probable discharge for each basin as Case 3 and Case 4.

According to the results, the specific discharges of the probable discharge in case of 3-year return period are in the range of between 8 $m^3/s/km^2$ and 23 $m^3/s/km^2$ and, within the 20 years from 1984 to 2005, the runoff discharge is expected to increase by about 50% of those under the present condition, which is emphasized with the increments of discharge in the Mburicao, Itay and Lambare river basins.

6.3 Discharge With Storage Facilities

The probable discharge in case of providing storage facilities was calculated on the basis of the future river basin condition in the year 2005, but in two cases: with and without storm drainage facilities.

Storage in Public Compounds

According to the study results on future land use pattern, about 10% of the area of each basin is expected to be used for public compounds. However, the area of public compounds are not always in the topographically suitable location for the use of storage facilities. The runoff calculation, therefore, is made in two assumptions: (1) the area of 2.5% of each basin, and (2) the area of 5% are available for the use of these storage facilities. In this connection, the effect of one unit of these storage facilities which was studied in the preceding section is reflected in proportion to the area provided for the storage facilities.

As a result, about 10% of the peak discharge is regulated on an average compared with the discharge without facilities in case of providing storage facilities to the area of 2.5% of each basin. Tables 1-15 and 1-16 (Case 2 and Case 3) show the runoff discharge with storage facilities. The runoff discharge without storage facilities is also shown in the table as Case 1 for reference.

Storage in House Lots

In case of the calculation for storage in house lots, it is assumed that the storage facilities are provided to all residential houses in the basin. The number of houses was based on the study result of the Urban Planning which reports that the number of houses in the residential area of one (1) hectare is 20 in high density residential area, 14 in medium density residential area, and 8 in low density residential area. The effect of the storage facilities which was studied for one house in the preceding section is taken into account in accordance with the number of houses in the basin.

Tables 1-15 and 1-16 show the probable discharge with the storage facilities in house lots as Case 4 and Case 5. According to the results, the peak discharge in case of providing facilities of 1.0 m^3 and 2.0 m^3 are lower by about 10% and 20% on an average, respectively, of that without facilities.

6.4 Discharge With Infiltration Facilities

As in the study cases of the storage facilities, the calculation of probable discharge of the infiltration facilities is based on the following conditions:

- As the river basin condition, the future conditiion in the year of 2005 is applied.

- Regarding the provision of storm drainage facilities, calculation of two cases will be made: with and without the facilities.

- As for the length of infiltration facilities of trench type, two cases are employed: 10 m long and 20 m long trenches; and the facilities are provided to all houses in the residential area of the basin.

Tables 1-15 and 1-16 (Case 6 and Case 7) show the probable discharge with the provision of infiltration facilities, the length of which is based on the number of houses in the basin. According to the results, the facilities will contribute to the regulation of peak discharge of about 5% and 10% in case of a 10-m long trench and a 20-m long trench, respectively.

7. Probable Discharge for the First Stage Project

7.1 Probable Discharge for Subbasins

The probable discharge for the First Stage Project was calculated, narrowing down the study area to Itay and Mburicao river basins, which were further divided into subbasins as shown in Figs. 1-14 and 1-15.

The probable discharges for the sub-basins were calculated based on a 3-year return period in accordance with the project scale for the First Stage Project under the present land use pattern in 1984 and the future land use pattern in 1995 and 2005. Table 1-17 shows the calculated probable discharges in each subbasin.

7.2 Storage Capacity of Retarding Basin

Retarding Basin in the Downstream of Aviadores del Chaco Avenue

In this study, a retarding basin is proposed in the downstream of Aviadores del Chaco Avenue or at the terminal point of the study area to absorb the incremental runoff discharge due to future land development and river improvement works in the upper basin and thus prevent augmentation of flood damage in the downstream reaches which had been excluded from the study area.

The runoff volume to be stored in the proposed retarding basin is calculated at about 350,000 m^3 from the difference of the runoff discharges detected from the beginning to the crossing point of the presumed design hydrograph and the runoff hydrograph prepared at the terminal point of the study area under the land use patterns of 1995 with and without river improvement conditions.

Retarding Basin in the Middle Reaches

Two (2) retarding basins, as alternative plan of the river improvement works, are proposed in this study in the middle reaches of Itay River. The effects of these basins are calculated by using the storage function method. The relationships among the inflow and outflow discharges and the storage capacity, which was calculated under the future land use patten in 1995, are tabulated below:

Retarding Basin *	Inflow Discharge (m^3/s)	Outflow Discharge (m^3/s)	Storage Capacity (m^3)
1	44	24	50,000
2	60	49	30,000

* Refer to River and Drainage Planning Sector.

TABLES

Table 1-1 TEMPERATURE (AIRPORT STATION)

(Unit: °C)

Year	Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1980	Mean	26.8	26.6	27.7	25.1	21.3	17.1	16.9	19.5	18.4	23.2	23.5	26.5
	Max.	36.8	35.4	37.0	35.0	31.8	30.2	31.0	32.4	33.6	38.6	34.4	36.4
	Min.	12.4	18.0	19.6	8.2	6.8	2.8	1.0	5.0	3.6	10.6	12.0	16.6
1981	Mean	27.3	27.2	25.2	23.4	23.1	16.1	17.1	21.3	19.2	23.3	25.5	25.8
	Max.	36.2	35.5	35.4	36.4	33.4	29.0	32.2	34.8	37.4	36.8	37.0	35.4
	Min.	17.8	21.4	12.2	10.0	11.0	1.2	0.4	8.0	4.4	8.6	13.4	16.8
1982	Mean	27.3	25.9	25.3	23.2	20.4	18.2	19.8	21.0	21.8	23.0	24.2	25.6
	Max.	37.2	36.2	35.2	32.8	32.4	30.0	30.0	33.4	33.2	36.0	36.0	34.3
	Min.	16.1	17.8	11.2	12.4	6.6	4.0	4.4	9.0	11.0	10.0	15.2	14.0
1983	Mean	27.6	26.9	25.0	22.8	20.4	15.0	16.2	18.2	18.5	23.4	23.9	27.5
	Max.	36.9	35.8	34.0	33.8	32.0	29.6	29.1	33.0	33.8	36.6	35.4	37.2
	Min.	20.0	16.6	12.6	11.9	8.4	4.4	5.2	5.2	5.6	8.4	11.4	18.0
1984	Mean	27.8	28.5	26.0	20.8	21.8	16.8	18.7	16.0	21.7	26.0	25.5	24.9
	Max.	38.4	38.4	36.0	32.6	32.5	29.4	31.1	30.4	35.8	37.4	35.2	35.2
	Min.	21.0	15.8	16.6	8.0	4.8	3.6	1.5	0.0	6.6	16.0	15.2	15.4
1980 to 1984	Ave.	27.36	27.02	25.84	23.06	21.4	16.64	17.74	19.2	19.92	23.78	24.56	26.06
	Max.	38.4	38.4	37.0	36.4	33.4	30.2	32.0	34.8	37.4	38.6	37.0	37.2
	Min.	12.4	15.8	11.2	8.0	4.8	1.2	0.4	0.0	3.6	8.4	11.4	14.0

Table 1-2 HUMIDITY (AIRPORT STATION)

(Unit: °C)

Year	Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1980	Mean	63.0	73.0	70.0	66.0	78.0	72.0	69.0	73.0	62.0	68.0	72.0	70.0
	Max.	98.0	98.0	98.0	96.0	98.0	98.0	96.0	100.0	96.0	98.0	100.0	98.0
	Min.	20.0	33.0	28.0	28.0	32.0	31.0	24.0	30.0	22.0	28.0	34.0	28.0
1981	Mean	75.0	75.0	72.0	71.0	72.0	75.0	61.0	62.0	69.0	57.0	67.0	72.0
	Max.	97.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	96.0	98.0	100.0
	Min.	43.0	44.0	33.0	33.0	38.0	30.0	26.0	16.0	27.0	20.0	22.0	39.0
1982	Mean	62.0	74.0	76.0	73.0	74.0	82.0	76.0	77.0	75.0	71.0	80.0	76.0
	Max.	96.0	98.0	96.0	96.0	98.0	98.0	98.0	98.0	98.0	98.0	100.0	98.0
	Min.	27.0	37.0	43.0	38.0	41.0	44.0	45.0	48.0	43.0	35.0	41.0	36.0
1983	Mean	76.0	78.0	75.0	82.0	85.0	81.0	84.0	70.0	68.0	70.0	71.0	66.0
	Max.	97.0	98.0	96.0	98.0	98.0	100.0	98.0	97.0	98.0	98.0	98.0	96.0
	Min.	44.0	51.0	40.0	47.0	53.0	48.0	47.0	43.0	24.0	33.0	35.0	29.0
1984	Mean	75.0	70.0	77.0	81.0	75.0	81.0	69.0	71.0	57.0	68.0	74.0	69.0
	Max.	98.0	98.0	98.0	98.0	97.0	98.0	98.0	98.0	96.0	98.0	98.0	96.0
	Min.	43.0	40.0	34.0	41.0	40.0	43.0	36.0	20.0	19.0	34.0	33.0	27.0
1980 to 1984	Ave.	70.2	74.0	74.0	74.6	76.8	78.2	71.8	70.6	66.2	66.8	72.8	70.6
	Max.	98.0	98.0	98.0	98.0	98.0	100.0	98.0	100.0	98.0	98.0	100.0	100.0
	Min.	20.0	33.0	28.0	28.0	32.0	30.0	24.0	16.0	19.0	20.0	22.0	27.0

Table 1-3 WIND VELOCITY (AIRPORT STATION)

(Unit: km/hr)

Wind Direction	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
NORTH (N)	6.6	7.4	7.0	7.2	7.4	9.2	9.6	8.6	8.0	8.8	7.2	6.0
NORTHEAST (NE)	5.6	5.4	5.2	6.0	7.0	9.0	8.8	7.4	8.8	7.6	5.4	5.0
EAST (E)	4.0	3.8	4.6	4.8	5.0	4.6	5.4	4.2	5.0	5.2	4.4	3.6
SOUTHEAST (SE)	5.0	6.2	4.2	4.2	4.4	4.4	4.2	4.4	4.4	5.8	5.0	4.8
SOUTH (S)	6.0	7.4	6.6	6.0	5.8	6.2	8.6	7.2	9.0	8.2	7.8	6.6
SOUTHWEST (SW)	7.6	5.4	4.2	5.6	5.8	8.6	6.2	7.4	8.6	3.4	7.2	7.6
WEST (W)	2.0	3.4	1.6	3.2	1.6	4.4	6.4	4.6	3.2	1.4	6.6	5.6
NORTHWEST (NW)	4.2	7.0	4.4	1.6	4.4	2.4	5.4	6.0	5.6	5.6	1.6	4.4

Table 1-4 EVAPOLATION (SAN LORENZO STATION)

(Unit: mm)

Year	Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1976	Mean	2.6	2.9	3.2	2.0	1.7	2.5	2.9	3.0	2.4	3.9	3.6	4.5
1977	Mean	2.5	3.8	2.6	2.4	1.6	1.9	3.7	2.3	4.0	4.5	2.7	2.8
1978	Mean	3.6	3.1	3.7	3.0	2.7	3.3	2.5	3.1	3.0	3.0	2.8	3.9
1979	Mean	4.6	3.2	2.7	1.4	1.5	1.9	2.5	2.4	2.9	2.7	3.0	2.7
1980	Mean	3.2	2.5	2.9	3.7	1.8	2.0	2.7	2.1	3.1	2.9	2.1	2.4
1976 to 1980	Mean	3.3	3.1	3.0	2.5	1.9	2.3	2.9	2.6	3.1	3.4	2.8	3.3

Table 1-5 MONTHLY RAINFALL (AIRPORT STATION)

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1971	372.0	167.2	212.5	65.5	124.7	79.6	103.3	110.3	42.3	119.0	84.6	126.9	1,608.5
1972	123.1	171.1	138.1	160.4	44.2	229.0	34.2	74.8	46.4	70.9	337.3	179.3	1,608.8
1973	83.8	83.2	159.9	76.3	52.8	85.5	24.1	75.8	44.4	252.4	136.6	141.6	1,216.4
1974	156.8	174.6	185.3	70.7	247.0	16.6	9.9	157.3	10.9	238.6	158.8	162.0	1,588.5
1975	80.6	57.0	224.2	227.4	101.5	77.2	69.4	59.7	163.1	80.0	180.5	207.9	1,528.5
1976	247.7	33.0	58.0	132.2	27.8	23.9	1.9	40.5	63.6	86.4	53.9	41.4	810.3
1977	143.1	42.0	107.3	41.3	127.1	148.9	23.0	37.3	18.5	55.1	191.9	170.5	1,106.0
1978	93.6	146.0	30.8	32.9	9.2	57.0	128.3	20.7	94.8	297.7	116.5	51.7	1,079.2
1979	52.3	137.8	66.3	231.4	221.0	16.0	23.9	222.6	138.1	94.5	269.7	91.7	1,565.3
1980	220.8	138.0	106.0	118.6	203.2	64.6	6.5	131.9	75.3	76.9	194.0	89.2	1,425.0
1981	147.2	253.0	73.1	146.3	56.6	45.4	7.2	44.6	48.2	17.7	172.7	197.5	1,209.5
1982	45.7	140.5	56.7	78.4	30.9	144.5	18.1	54.8	144.8	77.0	419.1	218.0	1,421.5
1983	134.3	248.3	110.9	261.5	270.6	31.7	103.2	21.0	112.8	74.0	96.4	31.6	1,496.5
Average	146.2	137.8	117.6	126.4	116.7	78.5	42.1	80.9	77.2	118.5	185.5	131.5	1,358.9

Table 1-6(1/3) ANNUAL MAXIMUM RAINFALL

(Unit: mm)

Year	Duration (min) 5	10	15	20	30	45	60	120	180	360	720
1929	10.8	20.0	30.0	40.0	49.5	58.5	63.0	71.3	-	-	-
1930	8.0	15.0	28.0	35.0	44.0	50.0	53.7	64.6	-	-	-
1931	15.0	20.0	29.0	38.2	47.2	55.6	57.3	72.6	-	-	-
1932	16.0	31.6	47.4	63.2	65.0	78.5	83.1	85.6	-	-	-
1933	15.0	24.7	35.5	47.3	71.0	78.0	79.3	82.8	-	-	-
1934	14.0	18.0	26.0	30.0	31.4	42.0	58.5	58.5	-	-	-
1935	15.0	24.0	35.0	40.0	41.0	42.0	43.0	54.8	-	-	-
1936	13.4	23.0	26.0	30.0	32.0	45.0	46.0	46.0	-	-	-
1937	13.8	22.0	26.0	35.0	53.0	79.0	105.0	131.0	-	-	-
1938	14.0	20.0	22.0	33.0	43.0	53.0	60.0	66.0	-	-	-
1939	19.0	30.0	40.0	50.0	60.0	77.0	80.0	118.2	-	-	-
1940	18.4	27.0	30.0	38.0	38.3	49.3	55.4	55.4	-	-	-
1941	19.0	27.0	40.0	42.0	60.0	67.0	69.7	69.7	-	-	-
1942	18.3	27.0	31.0	39.0	40.0	48.2	52.0	64.0	-	-	-
1943	19.7	24.0	32.0	39.2	43.2	55.0	68.0	72.0	-	-	-
1944	15.0	20.0	28.0	34.0	35.0	40.0	43.0	43.0	-	-	-
1945 to 1965	-- No Data --										
1966	8.0	14.0	16.0	18.0	27.0	29.0	37.0	47.0	52.0	65.0	65.0
1967	10.0	18.0	24.0	32.0	36.0	42.0	45.0	72.0	99.0	150.0	240.0
1968	7.0	11.0	15.0	20.0	30.0	38.0	47.0	50.0	58.0	63.0	63.0
1969	10.0	20.0	30.0	40.0	53.0	83.0	98.0	125.0	126.0	126.0	126.0
1970	7.0	9.0	12.0	16.0	20.0	31.0	33.0	35.0	35.0	40.0	62.0

Table 1-6(2/3) ANNUAL MAXIMUM RAINFALL

(Unit: mm)

Year	Duration (min) 5	10	15	20	30	45	60	120	180	360	720
1971	23.0	33.0	38.0	40.0	43.0	47.0	50.0	60.0	76.0	91.0	99.0
1972	9.0	16.0	19.0	21.0	22.0	30.0	31.0	57.0	68.0	78.0	82.0
1973	16.0	28.0	33.0	35.0	36.0	36.0	36.0	49.0	52.0	72.0	73.0
1974	9.0	15.0	19.0	23.0	34.0	37.0	37.0	38.0	39.0	45.0	74.0
1975	6.0	9.0	11.0	13.0	19.0	22.0	24.0	32.0	48.0	74.0	110.0
1976	-- No Data --										
1977	12.0	15.0	19.0	22.0	25.0	34.0	37.0	50.0	61.0	75.0	75.0
1978	9.0	13.0	18.0	21.0	28.0	38.0	48.0	53.0	54.0	64.0	71.0
1979	11.0	17.0	31.0	34.0	40.0	60.0	80.0	101.0	102.0	102.0	102.0
1980	8.0	16.0	20.0	24.0	32.0	47.0	53.0	61.0	68.0	69.0	72.0

Table 1-6 (3/3). ANNUAL MAXIMUM RAINFALL

(Unit: mm)

Year	Armada/Sajonia /1		Aeropuerto		San Lorenzo	
	Date	Rainfall	Date	Rainfall	Date	Rainfall
1950	Dec. 02	95.7				
1951	Feb. 20	164.8				
1952	Feb. 24	133.4				
1953	May 01	190.0				
1954	May 28	188.9				
1955	May 13	76.4				
1956	Jan. 04	93.2				
1957	Jan. 23	87.3			Jan. 8	103.1
1958	Feb. 16	118.1			Feb. 16	127.0
1959	Dec. 15	131.8			Feb. 6	172.2
1960	Nov. 03	93.0			Nov. 3	141.2
1961	Mar. 25	83.1			Nov. 9	129.9
1962	Jan. 23	95.4			Jan. 23	96.2
1963	Dec. 17	87.5			Dec. 17	84.3
1964	Mar. 28	135.6			Mar. 22	153.1
1965	Jan. 15	138.8			Feb. 25	117.4
1966	Mar. 31	194.5			Oct. 22	64.2
1967	Feb. 16	238.6			Feb. 15	118.3
1968	Jan. 01	63.3			Sep. 27	78.7
1969	Nov. 01	126.7	Jan. 07	89.6	Jan. 07	134.3
1970	Mar. 15	62.7	May 18	97.7	Mar. 15	77.5
1971	Jan. 11	100.9	Jan. 07	72.2	Jan. 07	100.7
1972	Nov. 12	122.5	Nov. 29	103.2	Mar. 09	92.0
1973	Mar. 07	71.5	Oct. 05	68.2	Mar. 07	93.8
1974	Oct. 15	155.2	Oct. 15	136.8	Oct. 15	92.5
1975	Apr. 07	105.8	Apr. 07	91.3	Apr. 07	117.4
1976	Jan. 05	50.0	Jan. 30	74.1	Jan. 30	58.2
1977			June 19	78.8	Dec. 06	75.0
1978			Oct. 31	97.0	Dec. 19	74.4
1979			May 22	87.4	Apr. 12	100.7
1980			Jan. 23	100.0	Jan. 23	113.3
1981			Feb. 12	88.0		
1982			Nov. 19	193.7		
1983			Feb. 15	126.4		
1984			Nov. 02	115.3		

/1 Data from 1950 to 1964 are of Armada.

Table 1-7. PROBABLE RAINFALL INTENSITY

(Unit: mm/hr)

Time (min)	Return Period (Year)				
	1.1	2	3	5	10
5	88.7	147.6	173.2	201.6	237.6
10	73.1	115.8	136.0	154.2	180.0
15	64.3	102.8	119.5	138.0	161.6
20	58.1	94.2	110.0	127.5	149.4
30	47.1	76.0	88.4	102.4	119.8
45	38.7	62.9	73.5	85.2	99.9
60	31.0	52.8	62.2	72.7	85.8
120	17.5	31.2	37.1	43.8	52.0
180	10.9	21.2	25.6	30.5	36.7
360	6.7	12.6	15.1	18.0	21.5
720	2.7	7.3	9.3	11.5	14.2

Table 1-8. RAINFALL INTENSITY DURATION CURVE

(Unit: mm/hr)

Time (min)	Return Period (Year) 1.1	2	3	5	10
5	88.5	131.6	152.2	174.9	204.1
10	74.3	116.9	135.8	156.8	183.4
15	65.1	105.1	122.6	142.0	166.5
20	57.8	95.4	111.7	129.8	152.5
30	47.5	80.6	94.9	110.7	130.5
45	37.4	65.4	77.4	90.7	107.3
60	30.8	55.0	65.4	76.9	91.1
120	18.1	33.7	40.3	47.7	56.8
180	12.8	24.2	29.1	34.6	41.3
360	6.8	13.2	15.9	18.9	22.7
720	3.5	6.9	8.3	9.9	11.9
A =	2,630.0	5,200.0	6,300.0	7,540.0	9,060.0
B =	25.4	34.5	36.4	38.1	39.4

$$R = \frac{A}{T + B}$$

where; A, B = Coefficient
T = Duration (min)
R = Rainfall intensity (mm/hr)

Note: The coefficients were derived from the probable rainfall intensities presented in Table 4-4.

Table 1-9 STORM RECORDS IN 1986

Date	R (mm/hr)	Mburicao River Basin H (m)	Q (m3/s)	Tg (min)	F	Itay River Basin H (m)	Q (m3/s)	Tg (min)	F
Jan.23	35	1.30	46	50	0.35	0.82	29	45	0.19
Jan.30	34.5	1.44	56	60	0.43	0.73	24	30	0.46
Mar.11	45	0.86	20	60	0.12	0.73	24	60	0.12
Apr.3	40	1.65	74	10	0.49	0.84	29	40	0.16
Apr.6	26.5	1.23	41	25	0.41	0.70	22	10	0.19
Apr.7	61.5	2.05	114	25	0.49	0.90	32	15	0.12
Apr.14	38.5	1.25	42	0	0.29	0.72	23	0	0.12
Apr.20	32.0	1.61	70	10	0.58	0.53	15	10	0.11
Average				30	0.40			25	0.19

NOTE
R : Rainfall intensity
H : Maximum water stage
Q : Runoff discharge
Tg: Time difference between peaks in hyetograph and hydrograph
F : Run-off Coefficient

Table 1-10. RUNOFF COEFFICIENT BY LAND USE FACTOR

Land Use Factor	Runoff Coefficient
Roof	0.90
Interspace (Hard) /1	0.80
Interspace (Soft) /2	0.20
Parking Area	0.80
Road	0.85
Park	0.20
Farmland	0.30
Vacant Land	0.30

Note:

/1 : Impermeable areas due to the pavement.
/2 : Permeable areas where rainfall may infiltrate to some degree.

Source: Design Criteria of Sewerage Facilities of Japan, American Society of Civil Engineers, Etc.

Table 1-11. RUNOFF COEFFICIENTS BY LAND USE CATEGORY

Area		Proportion of Land Use Factor (%) Roof	Interspace (Hard)	Interspace (Soft)	Road	Runoff Coefficient
Commercial		54.5	12.9	7.0	25.5	0.80
Residential Area	High Density	36.3	21.2	16.4	26.1	0.75
	Medium Density	19.1	26.8	30.0	23.9	0.65
	Low Density	11.2	6.4	59.6	22.8	0.45
Industrial Area		15.1	13.9	55.7	15.3	0.50
Public Utilization		7.2	4.7	88.1	-	0.30

Note: The runoff coefficients for each land use factor are as follows; roof: 0.9, Interspace (Hard): 0.8, Interspace (Soft): 0.2, and Road: 0.85.

Table 1-12. INCREMENT OF RUNOFF COEFFICIENT BY URBANIZATION

River Basin	Name of Basin	Runoff Coefficient		
		1965	1984	2005
B-1	Varadero	0.54	0.62	0.66
B-2	Jardin	0.65	0.68	0.68
B-3	Centro	0.59	0.62	0.67
B-4	Jaen	0.57	0.65	0.67
B-5	Tacumbu	0.48	0.53	0.53
B-6	Salamanca	0.52	0.57	0.58
B-7	Zanja Moroti	0.63	0.64	0.65
B-8	Ferreira	0.50	0.63	0.66
B-9	Villa Universitaria	0.32	0.44	0.60
B-10	Las Mercedes	0.48	0.59	0.62
B-11	Mariscal Lopez	0.56	0.62	0.63
B-12	Bella Vista	0.50	0.63	0.65
B-13	Tablada	0.45	0.62	0.63
B-14	Mburicao	0.42	0.50	0.57
B-15	Ycua Carrillo	0.37	0.44	0.63
B-16	Santa Rosa	0.33	0.41	0.56
B-17	Tres Puentes Cue	0.30	0.35	0.41
B-18	Itay	0.33	0.41	0.50
B-19	Lambare	0.36	0.51	0.67
B-20	Valle Apua	0.30	0.38	0.40
B-21	Villa Elisa	0.31	0.40	0.51
B-22	Nemby	0.30	0.36	0.44
B-23	San Lorenzo	0.30	0.32	0.35
B-24	Tayazuape	0.30	0.32	0.35
B-26	Zeballos Cue	0.32	0.35	0.41
B-27	Paso Cai	0.32	0.34	0.52

Table 1-13. POSSIBLE MAXIMUM CATCHMENT AREA OF STORAGE FACILITIES

Return Period	Catchment Area (ha)	Maximum Discharge (m^3/s)		Storage Water Depth (cm)	Possible Maximum Catchment (ha)
		Inflow	Outflow		
2-Year	1.0	0.33	0.065	7	3.0
	3.0	0.99	0.124	27	
	4.0	1.32	0.146	37	
3-Year	1.0	0.38	0.07	8	2.8
	2.0	0.76	0.11	10	
	3.0	1.14	0.15	31	
5-Year	1.0	0.44	0.08	11	2.3
	2.0	0.88	0.12	25	
	2.5	1.09	0.14	32	
10-Year	1.0	0.51	0.088	14	2.0
	1.5	0.77	0.11	22	
	2.0	1.02	0.13	30	

Table 1-14(1/2) PROBABLE DISCHARGE (WITHOUT DETENTION FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	CASE NO.	RUNOFF COEFFICIENT	PROBABLE DISCHARGE									
				1.1-yr		2-yr		3-yr		5-yr		10-yr	
				Tc	Q	Tc	Q	Tc	Q	Tc	Q	Tc	Q
				min	m^3/s	min	m^3/s	min	m^3/s	min	m^3/s	min	m^3/s
B-2	0.6	1	0.65	35	4.7	25	9.5	25	11.1	25	12.9	20	16.5
Jardin		2	0.68	25	5.9	20	10.8	20	12.7	20	14.7	15	18.9
		3	0.68	25	5.9	20	10.8	20	12.7	20	14.7	15	18.9
		4	0.68	20	6.6	15	11.9	15	13.9	15	16.1	10	20.5
B-4	2.47	1	0.57	50	13.6	40	27.3	40	32.2	35	40.3	35	47.6
Jaen		2	0.65	40	17.9	30	36.0	30	42.3	25	53.3	25	62.7
		3	0.67	40	18.5	30	37.1	30	43.6	25	54.9	25	64.7
		4	0.67	30	21.8	25	40.2	20	47.6	20	59.7	20	70.1
B-6	1.17	1	0.52	45	6.3	35	12.6	30	16.0	30	18.7	30	22.1
Salamanca		2	0.57	35	8.1	25	16.2	25	19.0	25	22.1	20	28.3
		3	0.58	35	8.2	25	16.5	25	19.3	25	22.5	20	28.8
		4	0.58	25	9.8	20	18.0	20	21.1	20	24.5	15	31.4
B-7	1.18	1	0.63	40	8.3	30	16.6	30	19.6	30	22.9	25	29.1
Zanja Moroti		2	0.64	30	10.0	25	18.3	25	21.5	20	27.2	20	32.0
		3	0.65	30	10.1	25	18.6	25	21.9	20	27.6	20	32.5
		4	0.65	25	11.1	20	20.3	20	23.8	15	30.3	15	35.5
B-8	4.0	1	0.5	65	16.2	50	34.2	45	43.0	40	53.6	40	63.4
Ferreira		2	0.63	45	26.2	35	52.4	35	61.8	30	77.5	30	91.4
		3	0.66	45	27.4	35	54.9	30	64.7	30	81.2	30	95.7
		4	0.66	35	31.9	25	64.1	25	75.2	25	93.3	20	111.9
B-10	2.12	1	0.48	55	9.2	40	19.7	40	23.3	35	29.2	35	34.4
Las Mercedes		2	0.59	40	14.0	30	28.0	30	33.0	25	41.5	25	48.9
		3	0.62	40	14.7	30	29.4	30	34.6	25	43.6	25	51.4
		4	0.62	30	17.3	25	31.9	20	37.2	20	47.4	20	55.7
B-12	0.75	1	0.5	40	4.2	30	8.4	30	9.9	25	12.4	25	14.7
Bella Vista		2	0.63	30	6.2	25	11.5	20	14.7	20	17.0	20	20.0
		3	0.65	30	6.4	20	11.8	20	15.1	20	17.6	20	20.7
		4	0.65	20	7.8	15	13.3	15	16.6	15	19.2	15	22.6
B-14	16.45	1	0.42	105	38.7	80	87.2	75	108.5	70	133.9	65	166.5
Mburicao		2	0.5	75	59.8	55	132.7	55	157.5	50	195.5	45	245.3
		3	0.57	70	71.8	55	151.3	50	189.9	45	236.3	45	279.6
		4	0.57	55	85.2	40	181.8	40	214.8	35	268.7	35	326.5
B-15	4.01	1	0.37	75	10.8	55	23.9	50	30.1	50	35.3	45	44.2
Ycua Carrillo		2	0.44	55	16.0	40	34.2	40	40.4	35	50.6	35	59.7
		3	0.63	45	26.2	35	52.5	35	61.9	30	77.7	30	91.6
		4	0.63	35	30.6	25	61.3	25	72.0	25	83.9	20	107.0
B-16	3.13	1	0.33	70	7.9	55	16.7	50	20.9	45	26.0	45	30.8
Santa Rosa		2	0.41	50	12.4	40	24.9	35	31.5	35	36.8	30	46.5
		3	0.56	45	18.2	35	36.4	30	46.2	30	53.9	30	63.6
		4	0.56	35	21.2	25	42.6	25	50.0	25	58.2	20	74.3
B-17	6.8	1	0.3	95	12.4	70	28.2	65	35.2	60	43.6	55	54.4
Tres Puentes		2	0.35	70	18.2	55	38.4	50	48.2	45	60.0	40	75.4
		3	0.41	65	22.5	50	47.7	45	59.9	45	73.8	40	88.4
		4	0.41	50	27.0	35	57.9	35	68.3	30	85.7	30	105.7
B-18	54.55	1	0.33	170	67.3	125	163.0	115	208.1	105	263.5	100	325.0
Itay		2	0.41	120	112.4	90	259.5	80	336.3	75	414.2	70	514.5
		3	0.5	110	147.2	80	344.0	75	428.5	70	528.5	65	657.5
		4	0.5	80	189.1	60	416.9	55	522.2	50	648.4	50	767.8

NOTE

Tc : Concentration Time (min)
Q : Run off Discharge (m3/s)

Case 1 : Under past river basin condition in 1965.
Case 2 : Under present river basin condition in 1984.
Case 3 : Under future river basin condition in 2005, without provision of drainage facilities.
Case 4 : Under future river basin condition in 2005, with provision of drainage facilities.

Table 1-14(2/2) PROBABLE DISCHARGE (WITHOUT DETENTION FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	CASE NO.	RUNOFF COEFFICIENT	PROBABLE DISCHARGE 1.1-yr Tc	1.1-yr Q	2-yr Tc	2-yr Q	3-yr Tc	3-yr Q	5-yr Tc	5-yr Q	10-yr Tc	10-yr Q
				min	m^3/s	min	m^3/s	min	m^3/s	min	m^3/s	min	m^3/s
B-19 Lambare	25.66	1	0.36	130	43.4	95	103.0	90	127.9	80	163.8	75	203.2
		2	0.51	85	86.6	65	190.0	60	237.6	55	294.4	50	368.4
		3	0.67	75	125.1	55	277.4	55	329.2	50	408.7	45	512.6
		4	0.67	55	156.2	45	312.4	40	393.8	40	461.1	35	581.5
B-21 Villa Elisa	11.53	1	0.31	10	19.3	80	45.1	75	56.1	70	69.3	65	86.2
		2	0.4	75	33.6	55	74.4	55	88.3	50	109.6	45	137.5
		3	0.51	65	47.5	50	100.5	50	119.1	45	148.2	40	186.4
		4	0.51	50	57.0	40	114.0	35	144.1	35	168.5	30	213.2
B-22 Nemby	5.58	1	0.3	90	10.6	65	24.3	60	30.4	60	35.7	55	44.6
		2	0.36	80	13.9	60	30.7	55	38.5	55	45.2	50	56.5
		3	0.44	60	21.0	45	44.6	40	56.2	40	65.8	35	83.1
		4	0.44	45	25.5	35	51.0	30	64.7	30	75.5	30	91.2
B-23 San Lorenzo	33.69	1	0.32	150	44.9	110	107.8	100	138.3	95	169.6	85	218.1
		2	0.37	140	55.1	105	129.1	95	166.0	90	203.8	80	262.7
		3	0.4	105	75.5	80	170.0	70	221.6	65	273.8	60	341.2
		4	0.4	75	98.1	60	206.0	55	258.0	50	320.4	45	401.8
B-24 Tayazuape	30.13	1	0.3	150	37.6	110	90.4	100	116.0	95	142.2	85	182.9
		2	0.32	145	41.3	105	99.8	100	123.7	90	157.6	85	195.1
		3	0.35	105	59.1	80	128.5	75	165.7	70	204.3	65	254.2
		4	0.35	80	73.1	60	161.2	55	201.9	50	247.7	50	296.9
B-26 Zeballos Cue	0.96	1	0.32	50	3.0	40	6.0	35	7.5	35	8.8	35	10.4
		2	0.35	50	3.3	40	6.5	35	8.2	35	9.6	30	11.7
		3	0.41	35	4.8	30	8.8	25	11.2	25	13.1	25	15.4
		4	0.41	30	5.2	20	10.4	20	12.2	20	14.2	20	16.7
B-27 Paso Cai	5.49	1	0.32	85	11.6	65	25.5	60	31.9	55	39.5	50	49.5
		2	0.34	85	12.4	65	27.1	60	33.9	55	42.0	50	52.5
		3	0.52	55	25.9	40	55.4	40	65.4	35	81.8	35	96.6
		4	0.52	40	31.9	30	63.9	30	75.2	30	87.8	25	111.6

NOTE
Tc : Concentration Time (min)
Q : Run off Discharge (m3/s)

Table 1-15(1/3) PROBABLE DISCHARGE (WITH DETENTION FACILITIES AND WITHOUT DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m^3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-2 Jardin	0.6	0.68	1	5.9	10.8	12.7	14.7	18.9
			2	5.1	9.9	11.7	13.8	16.3
			3	3.9	8.9	10.8	12.9	15.4
			4	4.8	9.7	11.6	13.6	17.8
			5	4.0	8.9	10.8	12.8	17.0
			6	5.1	10.0	11.9	13.9	16.5
			7	3.8	9.3	11.1	13.1	15.7
B-4 Jaen	2.47	0.67	1	18.5	37.1	43.6	54.9	64.7
			2	16.1	33.9	40.5	51.5	61.1
			3	13.6	30.7	37.3	44.6	57.6
			4	15.5	34.1	40.6	51.9	61.7
			5	13.4	31.9	38.5	49.8	59.5
			6	16.4	34.9	41.5	52.8	62.5
			7	12.9	32.8	39.3	50.7	60.4
B-6 Salamanca	1.17	0.58	1	8.2	16.5	19.3	22.5	28.8
			2	6.9	14.8	17.7	20.9	26.9
			3	5.7	12.2	16.1	19.3	25.1
			4	6.9	15.2	18.1	21.3	27.5
			5	6.0	14.3	17.2	20.4	26.6
			6	7.3	15.6	18.4	21.6	27.9
			7	6.4	13.4	17.5	20.7	27.0
B-7 Zanja Moroti	1.18	0.65	1	10.1	18.6	21.9	27.6	32.5
			2	8.0	16.9	20.2	23.8	30.6
			3	6.7	15.3	18.5	22.2	28.8
			4	8.5	17.0	20.2	26.0	30.8
			5	6.4	15.8	19.0	24.8	29.7
			6	8.1	17.4	20.7	24.3	31.3
			7	6.9	16.3	19.5	23.1	30.1
B-8 Ferreira	4.0	0.66	1	27.4	54.9	64.7	81.2	95.7
			2	23.8	50.2	60.1	76.1	90.5
			3	19.1	45.5	55.4	71.1	85.3
			4	22.4	49.9	64.6	76.2	90.7
			5	18.8	46.3	61.0	72.6	87.2
			6	23.8	51.3	61.1	77.6	92.2
			7	18.4	47.7	57.6	74.0	88.6
B-10 Las Mercedes	2.12	0.62	1	14.7	29.4	34.6	43.6	51.4
			2	12.6	26.7	31.9	40.7	48.3
			3	10.5	24.0	29.2	37.7	45.3
			4	12.2	26.9	32.1	41.1	48.8
			5	10.4	25.1	30.3	39.3	47.0
			6	12.9	27.6	32.8	41.8	49.6
			7	10.0	25.8	31.0	40.0	47.8
B-12 Bella Vista	0.75	0.65	1	6.4	11.8	15.1	17.6	20.7
			2	5.5	10.8	14.0	16.4	19.5
			3	4.6	9.7	12.8	15.3	18.3
			4	5.5	11.9	14.2	16.6	19.7
			5	4.8	11.3	13.5	15.9	19.0
			6	5.7	11.1	14.4	16.9	20.0
			7	5.0	10.4	13.7	16.2	19.3
B-14 Mburicao	16.45	0.57	1	71.8	151.3	189.9	236.3	279.6
			2	59.6	137.6	175.0	208.0	262.9
			3	49.0	123.8	152.0	193.1	246.1
			4	56.1	135.6	174.2	220.6	263.9
			5	41.3	124.4	163.0	209.4	252.7
			6	53.8	140.1	178.7	211.7	268.4
			7	39.6	120.9	157.1	200.5	257.2

Note:

Case 1 : Without detention facilities.
Case 2 : Storage facilities in public compound utilizing 2.5% of river basin area.
Case 3 : Storage facilities in public compound utilizing 5% of river basin area.
Case 4 : Storage facilities in house lots with capacity of 1.0 m^3.
Case 5 : Storage facilities in house lots with capacity of 2.0 m^3.
Case 6 : Infiltration facilities with the length of 10 m.
Case 7 : Infiltration facilities with the length of 20 m.

Table 1-15(2/3) PROBABLE DISCHARGE (WITH DETENTION FACILITIES AND WITHOUT DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m^3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-15 Ycua Carrillo	4.01	0.63	1	26.2	52.5	61.9	77.7	91.6
			2	22.7	47.8	57.3	72.6	86.4
			3	18.0	43.1	52.6	67.5	81.1
			4	21.0	47.3	56.7	72.5	86.4
			5	17.3	43.6	53.0	68.8	82.7
			6	20.8	48.8	58.2	74.0	87.9
			7	15.6	41.6	54.5	70.3	84.2
B-16 Santa Rosa	3.13	0.56	1	18.2	36.4	46.2	53.9	63.6
			2	15.4	32.8	39.3	49.9	59.5
			3	11.9	29.1	35.7	46.0	55.4
			4	14.3	32.6	42.3	50.0	59.7
			5	11.6	29.8	39.6	47.3	56.9
			6	14.2	33.7	40.2	51.1	60.8
			7	10.3	30.9	37.4	48.4	58.0
B-17 Tres Puentes Cue	6.8	0.41	1	22.5	47.7	59.9	73.8	88.4
			2	18.3	41.5	53.3	63.6	80.9
			3	13.6	35.3	44.2	57.0	73.4
			4	19.2	44.3	56.6	66.9	85.0
			5	16.8	41.9	54.2	64.5	82.6
			6	20.1	45.3	57.5	67.9	86.0
			7	16.5	42.8	55.1	65.5	83.6
B-18 Itay	54.55	0.5	1	147.2	344.0	428.5	528.5	657.5
			2	126.4	299.9	394.1	491.5	616.0
			3	108.4	270.2	345.9	454.5	574.6
			4	102.4	299.4	383.7	483.7	612.8
			5	70.5	267.4	351.8	451.8	580.8
			6	100.5	297.7	378.1	496.5	625.5
			7	64.2	252.5	346.2	441.2	593.6
B-19 Lambare	25.66	0.67	1	125.1	277.4	329.2	408.7	512.6
			2	106.7	242.9	307.7	385.5	486.6
			3	94.2	223.0	286.2	362.3	435.6
			4	89.4	241.8	293.5	373.1	477.0
			5	58.0	216.3	268.0	347.6	451.5
			6	88.3	237.3	303.7	383.2	487.2
			7	57.9	198.6	278.2	357.8	433.0
B-21 Villa Elisa	11.53	0.51	1	47.5	100.5	119.1	148.2	186.4
			2	38.4	85.3	108.7	136.9	173.7
			3	30.7	75.6	98.2	125.6	151.9
			4	34.0	89.5	108.1	137.2	175.3
			5	26.1	81.6	100.2	129.3	167.4
			6	34.9	87.0	111.2	140.3	178.5
			7	25.0	79.1	103.3	132.4	159.6
B-22 Nemby	5.58	0.44	1	21.0	44.6	56.2	65.8	83.1
			2	17.2	39.1	47.3	59.9	76.4
			3	12.9	31.9	41.8	54.0	65.5
			4	17.6	41.2	52.8	62.4	79.6
			5	15.2	38.8	50.4	60.0	77.2
			6	18.6	42.2	50.3	63.4	80.6
			7	15.0	39.7	47.9	61.0	78.2
B-23 San Lorenzo	33.69	0.4	1	75.5	170.0	221.6	273.8	341.2
			2	62.4	150.4	190.5	238.3	299.3
			3	51.0	126.1	169.3	215.4	273.7
			4	61.1	155.6	207.2	259.3	326.7
			5	50.7	145.2	196.9	249.0	316.4
			6	62.4	159.7	201.4	250.8	314.5
			7	44.8	142.3	191.1	240.5	304.2

Table 1-15(3/3) PROBABLE DISCHARGE (WITH DETENTION FACILITIES AND WITHOUT DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-24 Tayazuape	30.13	0.35	1	59.1	128.5	165.7	204.3	254.2
			2	46.3	111.0	146.7	183.9	231.3
			3	37.1	94.6	123.1	163.5	208.4
			4	51.6	127.7	160.3	199.0	248.9
			5	47.8	123.9	156.5	195.2	245.1
			6	53.1	129.2	161.9	200.5	250.4
			7	47.3	119.9	158.1	196.7	246.6
B-26 Zeballos Cue	0.96	0.41	1	4.8	8.8	11.2	13.1	15.4
			2	3.5	7.6	9.1	11.7	14.0
			3	2.5	6.3	7.9	10.4	12.6
			4	4.5	8.5	10.9	12.8	15.1
			5	4.2	8.3	10.7	12.6	14.9
			6	4.5	8.6	11.0	12.9	15.2
			7	4.0	8.4	9.9	12.6	15.0
B-27 Paso Caí	5.49	0.52	1	25.9	55.4	65.4	81.8	96.6
			2	21.9	46.5	59.5	75.4	90.0
			3	17.0	41.1	53.7	64.9	83.4
			4	21.2	50.6	60.7	77.1	91.9
			5	17.9	47.3	57.3	73.7	88.5
			6	22.6	48.5	62.0	78.4	93.2
			7	17.7	45.1	58.7	69.8	89.8

Table 1-16(1/3) PROBABLE DISCHARGE (WITH DETENTION AND DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-2 Jardin	0.6	0.68	1	6.6	11.9	13.9	16.1	20.5
			2	5.6	10.8	12.8	15.1	17.8
			3	4.7	9.8	11.8	14.0	16.8
			4	5.5	10.8	12.8	15.0	17.8
			5	4.7	10.0	12.0	14.2	17.0
			6	5.8	11.1	13.1	15.3	18.1
			7	5.0	10.3	12.3	14.5	17.3
B-4 Jaen	2.47	0.67	1	21.8	40.2	47.6	59.7	70.1
			2	18.8	36.7	47.5	55.9	66.2
			3	15.8	33.1	40.2	52.0	62.3
			4	18.8	37.2	48.4	56.7	67.1
			5	16.7	35.1	46.2	54.5	65.0
			6	19.7	38.0	49.2	57.5	68.0
			7	17.6	35.9	42.9	55.4	65.8
B-6 Salamanca	1.17	0.58	1	9.8	18.0	21.1	24.5	31.4
			2	8.2	16.1	19.2	22.7	29.3
			3	6.6	14.3	17.4	20.9	27.3
			4	8.6	16.7	19.8	23.2	30.1
			5	7.7	15.8	18.9	22.3	29.2
			6	8.9	17.1	20.2	23.6	30.5
			7	7.2	16.2	19.3	22.7	29.6
B-7 Zanja Moroti	1.18	0.65	1	11.1	20.3	23.8	30.3	35.5
			2	9.5	18.5	22.0	25.8	33.4
			3	7.9	16.6	20.1	24.0	31.3
			4	9.5	18.7	22.1	28.6	33.8
			5	8.3	17.5	21.0	27.4	32.6
			6	9.9	19.1	22.6	26.5	34.3
			7	8.8	18.0	21.4	25.3	33.1
B-8 Ferreira	4.0	0.66	1	31.9	64.1	75.2	93.3	111.9
			2	27.6	58.4	69.6	82.1	105.5
			3	23.2	48.8	64.0	76.5	91.7
			4	26.9	59.1	70.2	82.6	106.8
			5	23.4	55.5	66.7	79.0	103.3
			6	28.4	60.5	71.7	84.1	108.3
			7	24.8	52.0	68.1	80.5	104.7
B-10 Las Mercedes	2.12	0.62	1	17.3	31.9	37.2	47.4	55.7
			2	14.8	28.9	37.5	44.1	52.3
			3	12.2	25.9	31.5	40.9	49.0
			4	14.8	29.4	38.3	44.9	53.2
			5	13.0	27.6	36.5	43.1	51.4
			6	15.5	30.1	39.0	45.6	53.9
			7	13.7	28.3	37.2	43.8	52.1
B-12 Bella Vista	0.75	0.65	1	7.8	13.3	16.6	19.2	22.6
			2	6.7	11.7	15.3	17.9	21.2
			3	5.0	10.5	14.0	16.7	19.9
			4	6.9	13.3	15.6	18.3	21.6
			5	6.2	12.6	14.9	17.6	20.1
			6	7.2	12.2	15.9	18.5	21.9
			7	5.7	11.5	15.2	17.8	21.2
B-14 Mburicao	16.45	0.57	1	85.2	181.8	214.8	268.7	326.5
			2	73.1	164.2	197.2	249.6	297.5
			3	58.0	138.0	179.7	230.6	277.8
			4	69.5	166.1	199.1	253.0	301.5
			5	58.3	154.9	187.9	241.8	290.3
			6	74.0	170.6	203.6	257.4	306.0
			7	57.8	148.0	192.4	246.2	294.8

Note:

Case 1 : Without detention facilities.
Case 2 : Storage facilities in public compound utilizing 2.5% of river basin area.
Case 3 : Storage facilities in public compound utilizing 5% of river basin area.
Case 4 : Storage facilities in house lots with capacity of 1.0 m^3.
Case 5 : Storage facilities in house lots with capacity of 2.0 m^3.
Case 6 : Infiltration facilities with the length of 10 m.
Case 7 : Infiltration facilities with the length of 20 m.

Table 1-16(2/3) PROBABLE DISCHARGE (WITH DETENTION AND DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-15 Ycua Carrillo	4.01	0.63	1	30.6	61.3	72.0	83.9	107.0
			2	26.2	51.4	66.4	78.3	93.0
			3	21.8	46.3	60.7	72.7	87.2
			4	25.4	56.1	66.8	78.7	101.8
			5	21.7	52.4	63.1	75.0	98.1
			6	26.9	52.9	68.3	80.1	95.0
			7	20.8	49.2	64.6	76.4	91.3
B-16 Santa Rosa	3.13	0.56	1	21.2	42.6	50.0	58.2	74.3
			2	17.8	38.1	45.6	53.8	69.3
			3	14.4	31.2	41.2	49.5	59.5
			4	17.3	38.7	46.1	54.3	70.4
			5	14.6	35.9	43.3	51.6	67.6
			6	18.4	39.8	47.2	55.4	71.5
			7	14.1	33.7	44.4	52.7	63.0
B-17 Tres Puentes Cue	6.8	0.41	1	27.0	57.9	68.3	85.7	105.7
			2	21.5	46.8	60.4	72.0	92.2
			3	16.0	39.5	52.5	64.1	83.3
			4	23.6	54.6	65.0	82.4	97.7
			5	21.2	52.2	62.6	80.0	95.3
			6	24.6	51.6	65.9	77.5	98.7
			7	22.2	49.2	63.5	75.1	96.3
B-18 Itay	54.55	0.5	1	189.1	416.9	522.2	648.4	767.8
			2	156.0	374.6	476.5	567.8	716.4
			3	127.3	317.4	410.3	522.0	665.1
			4	144.3	372.2	477.5	603.7	723.1
			5	112.4	340.2	445.5	571.7	691.1
			6	140.7	364.0	463.2	581.6	735.9
			7	101.6	332.1	431.2	549.7	703.9
B-19 Lambare	25.66	0.67	1	156.2	312.4	393.8	461.1	581.5
			2	129.7	287.1	366.4	433.8	550.8
			3	112.4	261.9	339.1	406.5	520.1
			4	120.6	276.7	358.1	425.4	545.9
			5	85.9	251.2	332.7	399.9	520.4
			6	121.6	286.9	368.3	435.6	556.1
			7	88.0	261.4	318.7	410.1	530.6
B-21 Villa Elisa	11.53	0.51	1	57.0	114.0	144.1	168.5	213.2
			2	47.7	101.7	130.7	155.1	198.2
			3	36.4	89.3	110.1	141.8	171.3
			4	45.9	103.0	133.1	157.4	202.2
			5	38.0	95.1	125.2	149.5	194.3
			6	45.5	106.1	136.2	160.6	205.3
			7	34.4	98.2	118.9	152.7	183.1
B-22 Nemby	5.58	0.44	1	25.5	51.0	64.7	75.5	91.2
			2	20.5	44.5	53.7	68.4	81.7
			3	14.8	38.0	47.2	61.4	74.4
			4	22.1	47.6	61.3	72.1	85.6
			5	19.6	45.2	58.9	69.7	83.2
			6	23.0	48.6	62.3	73.1	86.6
			7	18.9	46.2	55.3	70.6	84.2
B-23 San Lorenzo	33.69	0.4	1	98.1	206.0	258.0	320.4	401.8
			2	77.0	179.9	229.8	289.9	347.6
			3	58.9	153.7	201.5	246.6	315.9
			4	83.6	191.5	243.6	305.9	387.4
			5	68.6	181.2	233.3	295.6	377.1
			6	83.1	195.7	247.7	310.1	391.5
			7	68.5	185.3	237.4	299.7	358.7

Table 1-16(3/3) PROBABLE DISCHARGE (WITH DETENTION AND DRAINAGE FACILITIES)

RIVER BASIN	CATCHMENT AREA (km^2)	RUNOFF COEFFICIENT	CASE NO.	PROBABLE DISCHARGE (m^3/s) 1.1-yr	2-yr	3-yr	5-yr	10-yr
B-24 Tayazuape	30.13	0.35	1	73.1	161.2	201.9	247.7	296.9
			2	56.2	137.8	176.7	212.0	268.5
			3	42.7	109.7	144.6	186.7	240.1
			4	67.8	155.9	196.6	245.4	291.5
			5	64.0	152.1	192.8	241.6	287.7
			6	69.3	157.4	198.1	246.9	293.1
			7	62.2	153.6	194.3	229.6	289.3
B-26 Zeballos Cue	0.96	0.41	1	5.2	10.4	12.2	14.2	16.7
			2	4.0	8.2	10.7	12.7	15.2
			3	2.9	6.8	9.2	11.2	13.6
			4	4.9	10.1	11.9	13.9	16.4
			5	4.7	9.9	11.7	13.7	16.2
			6	5.0	10.2	12.0	14.0	16.5
			7	4.8	9.1	11.8	13.8	16.2
B-27 Paso Cai	5.49	0.52	1	31.9	63.9	75.2	87.8	111.6
			2	26.5	52.9	68.2	80.8	103.7
			3	19.9	46.5	61.2	73.9	95.8
			4	27.2	59.2	70.5	83.1	106.9
			5	23.8	55.9	67.2	79.7	103.5
			6	28.5	56.0	71.9	84.4	108.2
			7	22.9	52.6	68.5	81.1	104.8

Table 1-17 RUNOFF DISCHARGE OF 3-YEAR RETURN PERIOD FOR MBURICAO AND ITAY RIVER BASINS

Reference Point	Catchment Area (km2)	Landuse Patterns								
		Present (1984)			Future (1995)			Future (2005)		
		F	Tc	Q	F	Tc	Q	F	Tc	Q
(Mburicao River)										
1	4.22	0.56	35	58	0.58	35	60	0.60	35	62
2	6.04	0.55	40	76	0.56	40	77	0.56	40	78
3	3.63	0.44	35	39	0.52	35	46	0.59	35	53
4	11.97	0.52	50	126	0.55	45	138	0.58	45	149
5	1.72	0.39	30	18	0.46	30	21	0.53	30	24
6-1	14.28	0.50	50	145	0.54	50	155	0.57	50	165
6-2	0.62	0.50	20	10	0.54	20	11	0.57	20	11
6-3	16.45	0.50	55	158	0.54	50	174	0.57	50	190
(Itay River)										
1-1	9.63	0.46	50	90	0.55	45	115	0.63	40	139
1-2	13.71	0.46	55	121	0.55	50	154	0.63	45	186
2-1	15.98	0.45	55	138	0.53	50	166	0.60	50	194
2-2	12.62	0.38	55	92	0.43	50	108	0.48	50	123
2-3	28.70	0.42	65	208	0.49	60	250	0.56	60	292
3-1	1.27	0.38	30	13	0.40	30	14	0.42	30	14
3-2	3.33	0.38	40	29	0.40	35	32	0.42	35	34
3-3	3.05	0.38	40	27	0.40	35	29	0.42	35	31
4-1	1.56	0.39	30	16	0.50	25	22	0.60	25	27
4-2	2.72	0.39	35	26	0.50	30	35	0.60	30	43
4-3	0.64	0.39	25	7	0.50	20	10	0.60	20	12
4-4	4.01	0.39	40	36	0.50	35	48	0.60	35	59
5	7.35	0.38	50	57	0.40	45	61	0.41	45	65
6-1	4.59	0.43	40	45	0.47	40	49	0.50	40	53
6-1D	33.29	0.42	70	230	0.49	65	281	0.55	60	332

NOTE
F : Runoff Coefficient
Tc: Time of Concentration (min)
Q : Runoff Discharge (m^3/s)

FIGURES

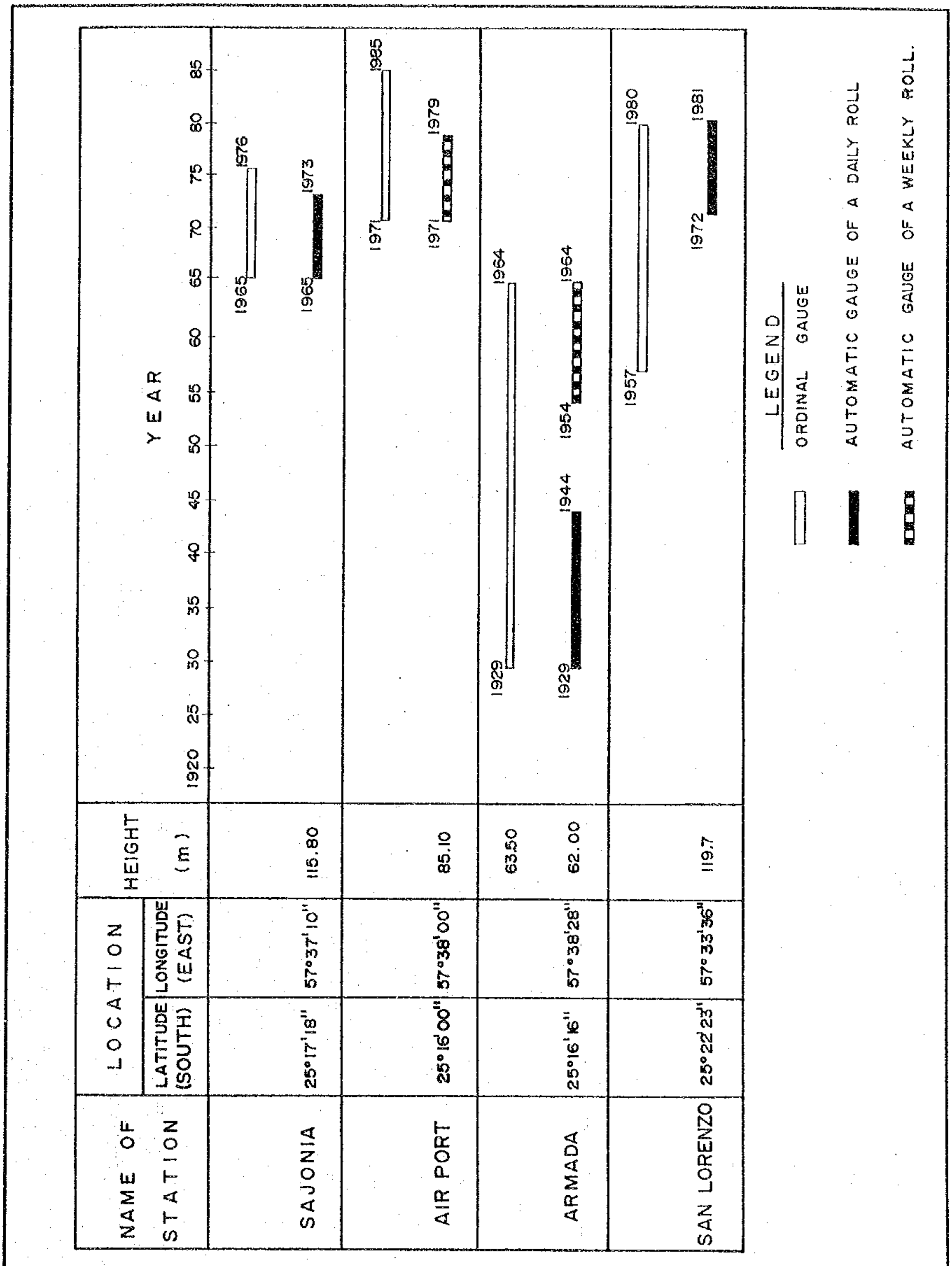


Fig. 1-1. METEOROLOGICAL OBSERVATION RECORDS

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

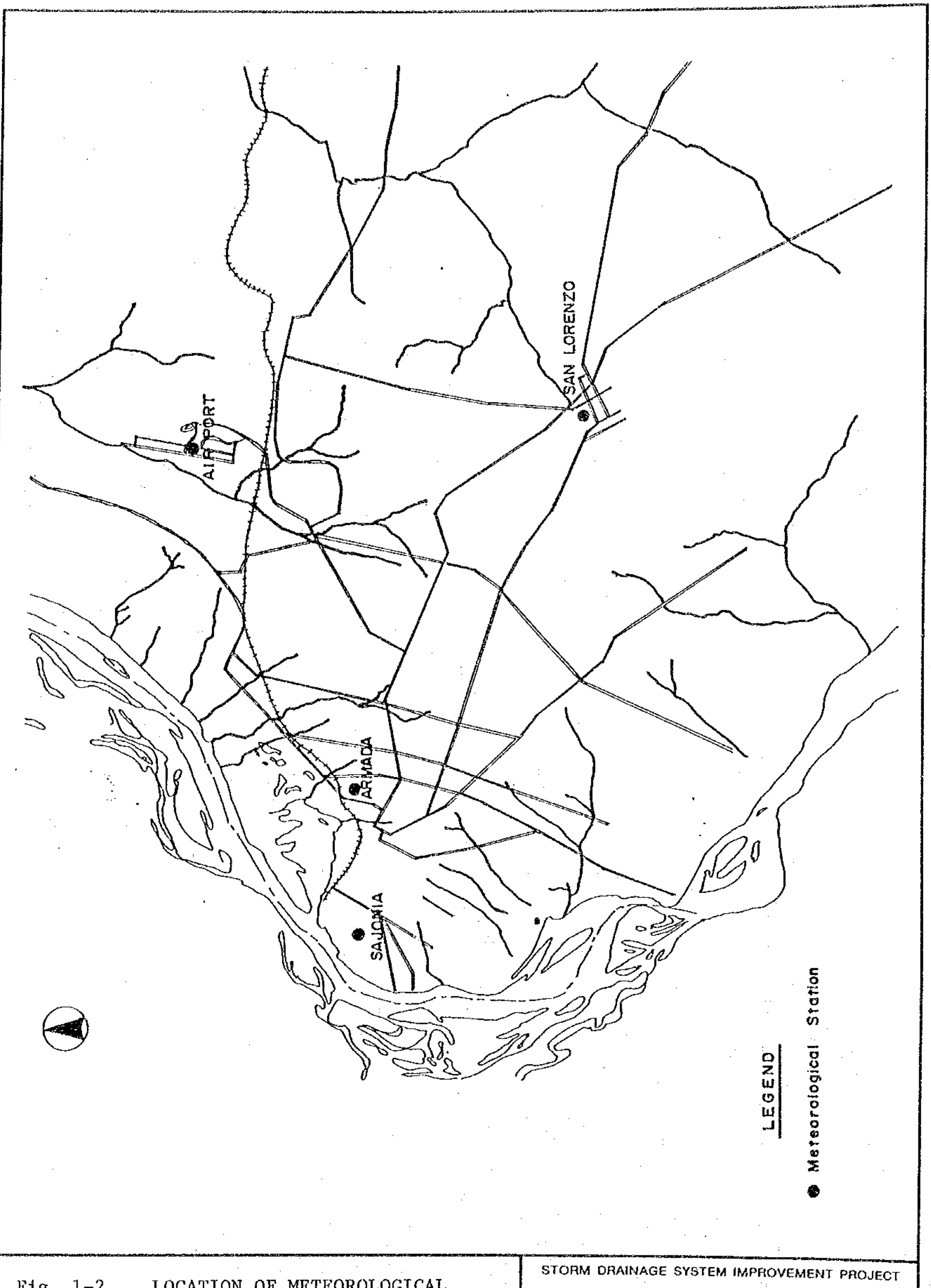


Fig. 1-2. LOCATION OF METEOROLOGICAL STATIONS

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

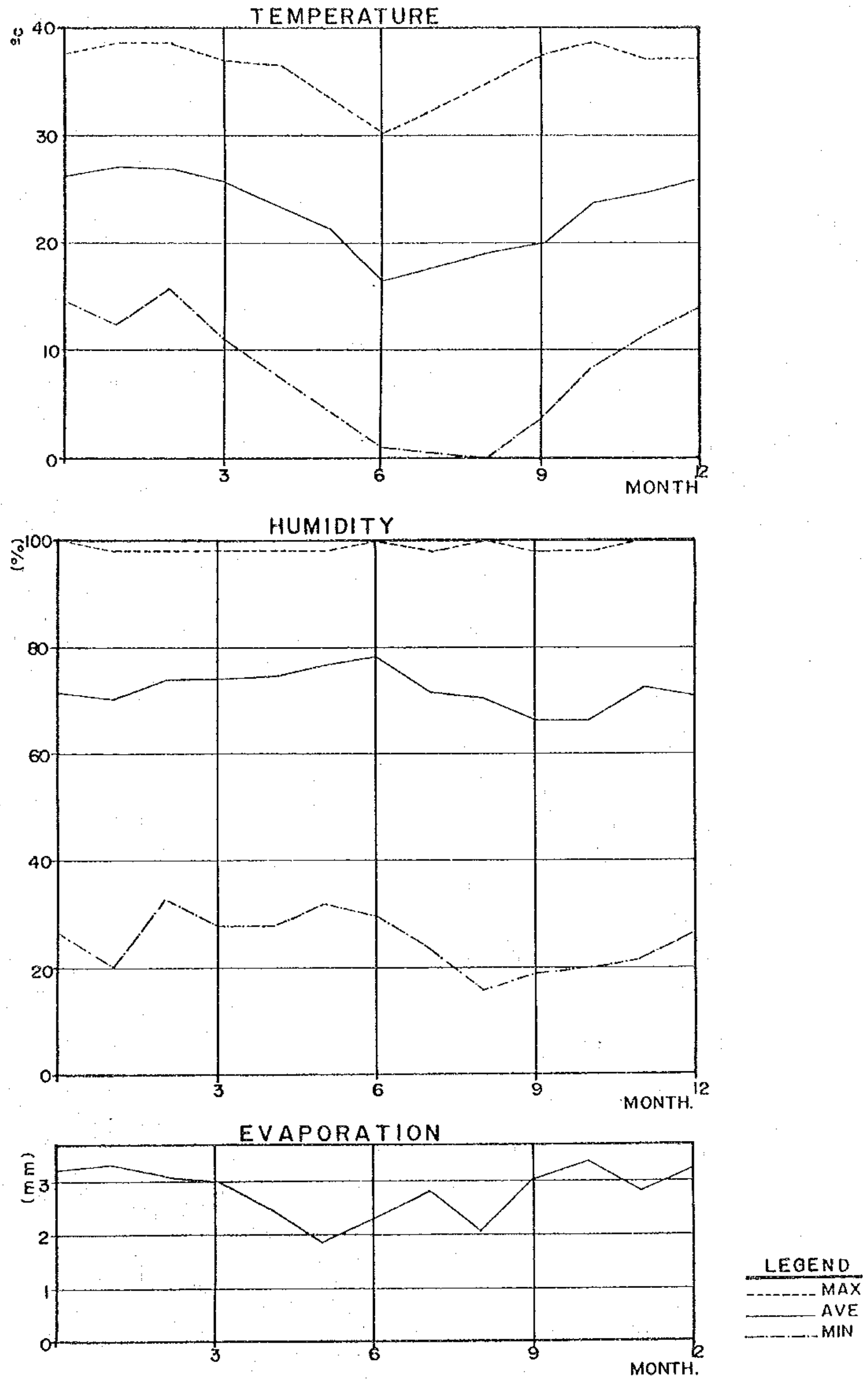
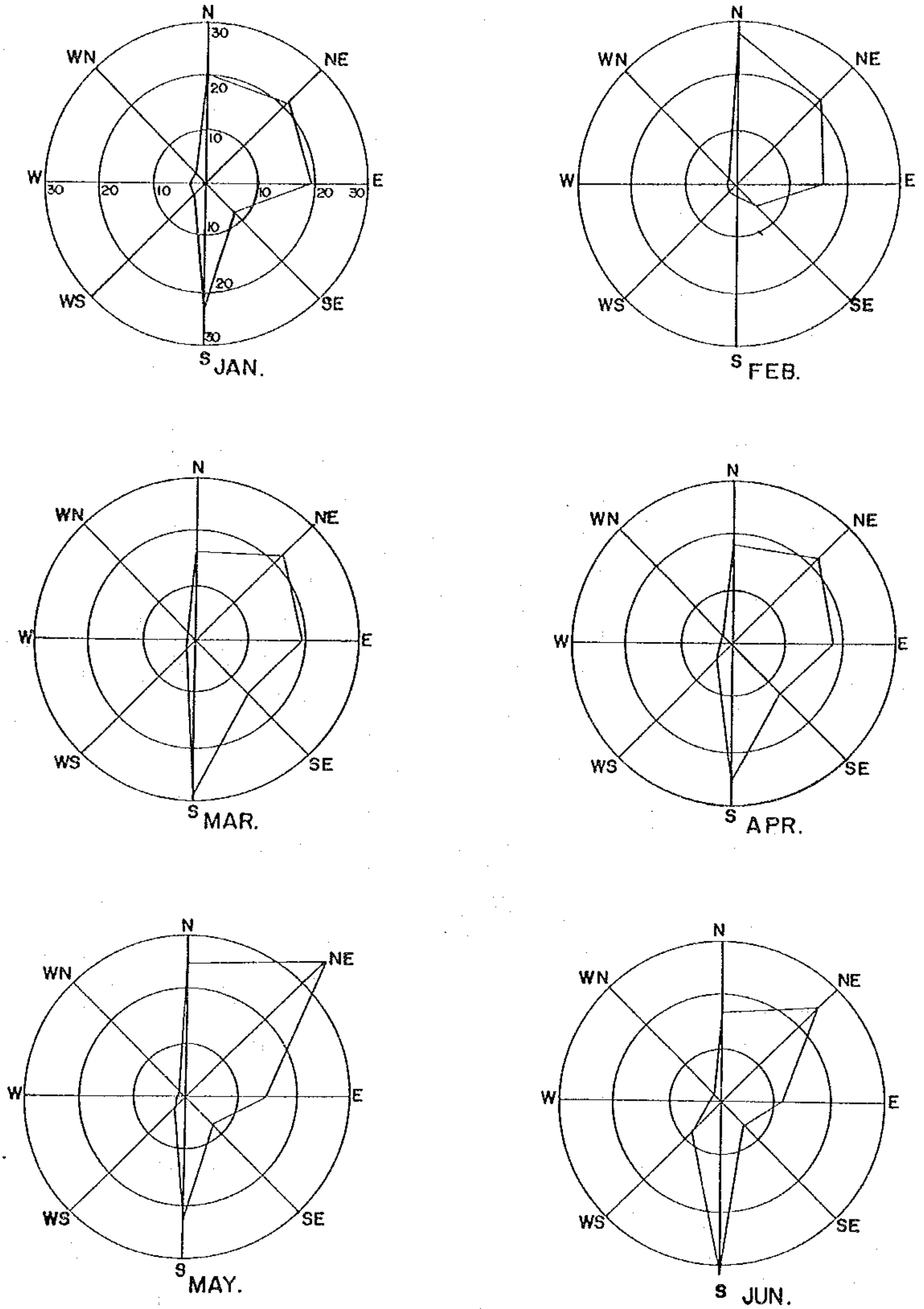


Fig. 1-3. MONTHLY VARIATION OF TEMPERATURE, HUMIDITY AND EVAPORATION

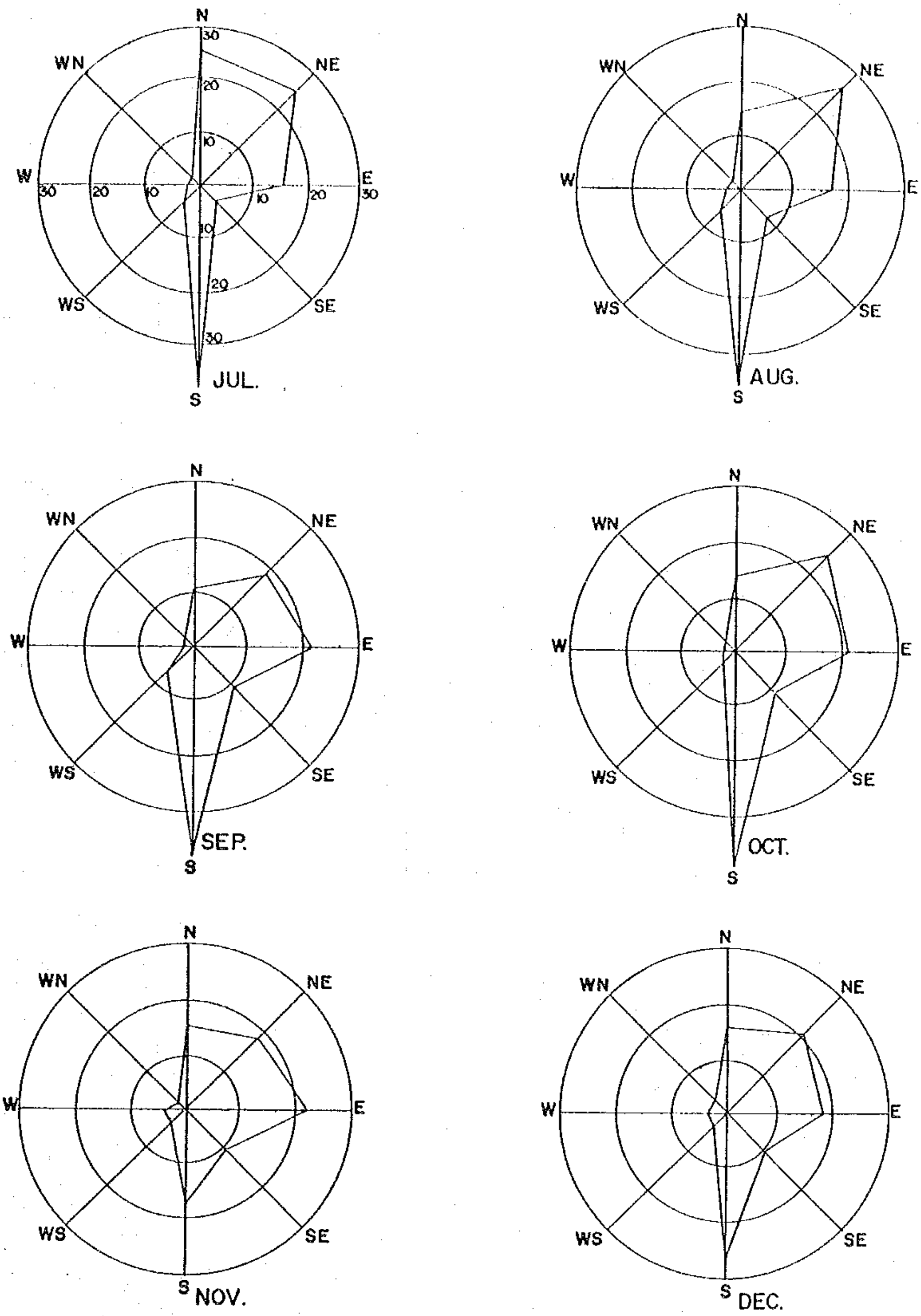
STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY



NOTE : NUMBERS OF FIG. SHOW THE FREQUENCY OF OCCURRENCE OF WIND DIRECTION.

Fig. 1-4. (1/2) WIND DIRECTION	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY



NOTE: NUMBERS OF FIG. SHOW THE FREQUENCY OF OCCURRENCE OF WIND DIRECTION.

Fig. 1-4. (2/2) WIND DIRECTION	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

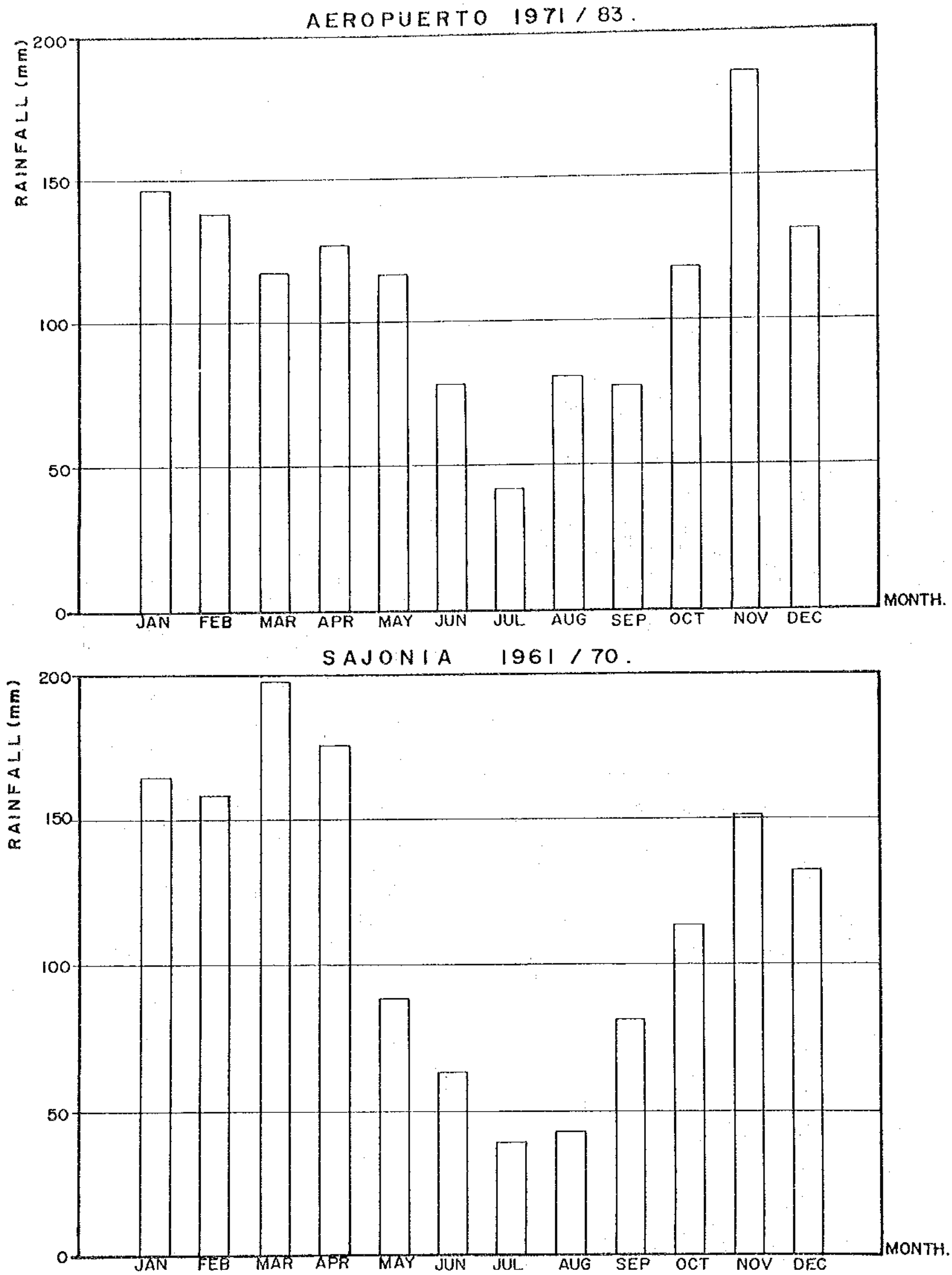


Fig. 1-5. (1/2) MEAN MONTHLY RAINFALL	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

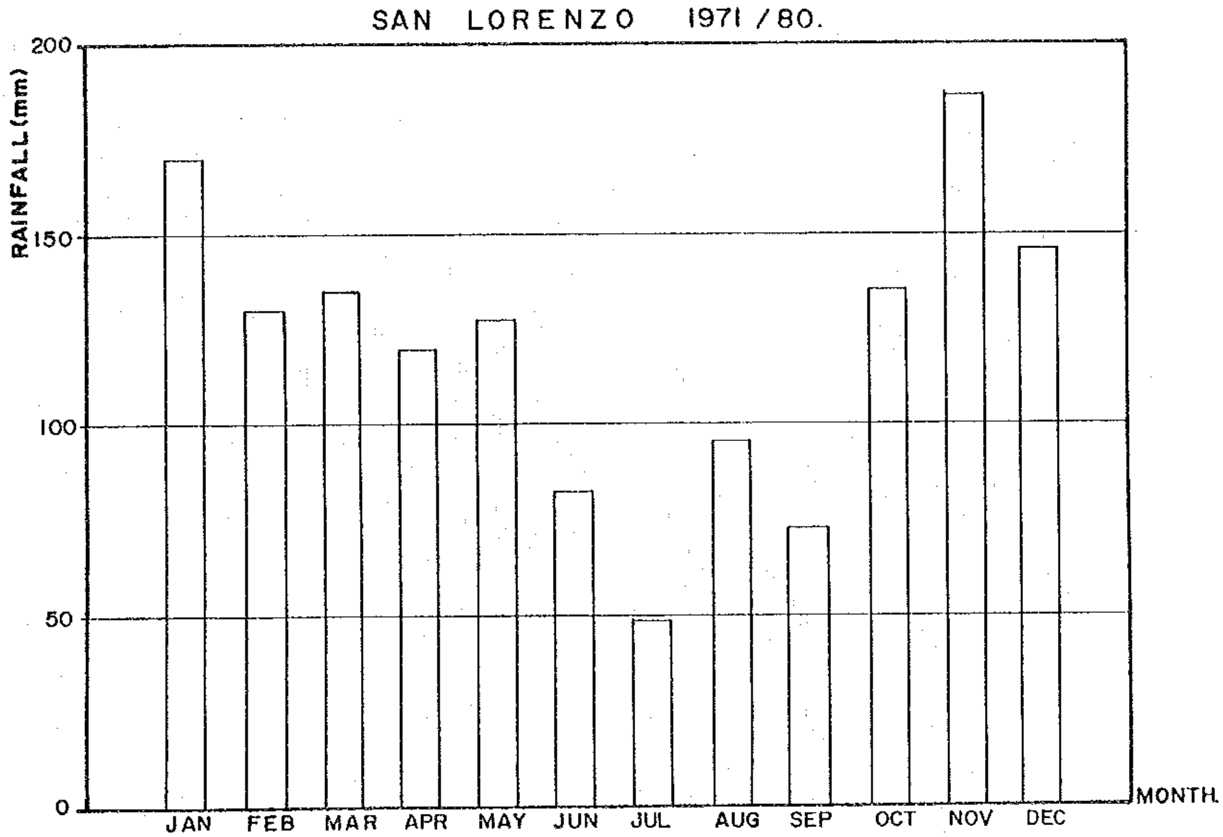


Fig. 1-5. (2/2) MEAN MONTHLY RAINFALL	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

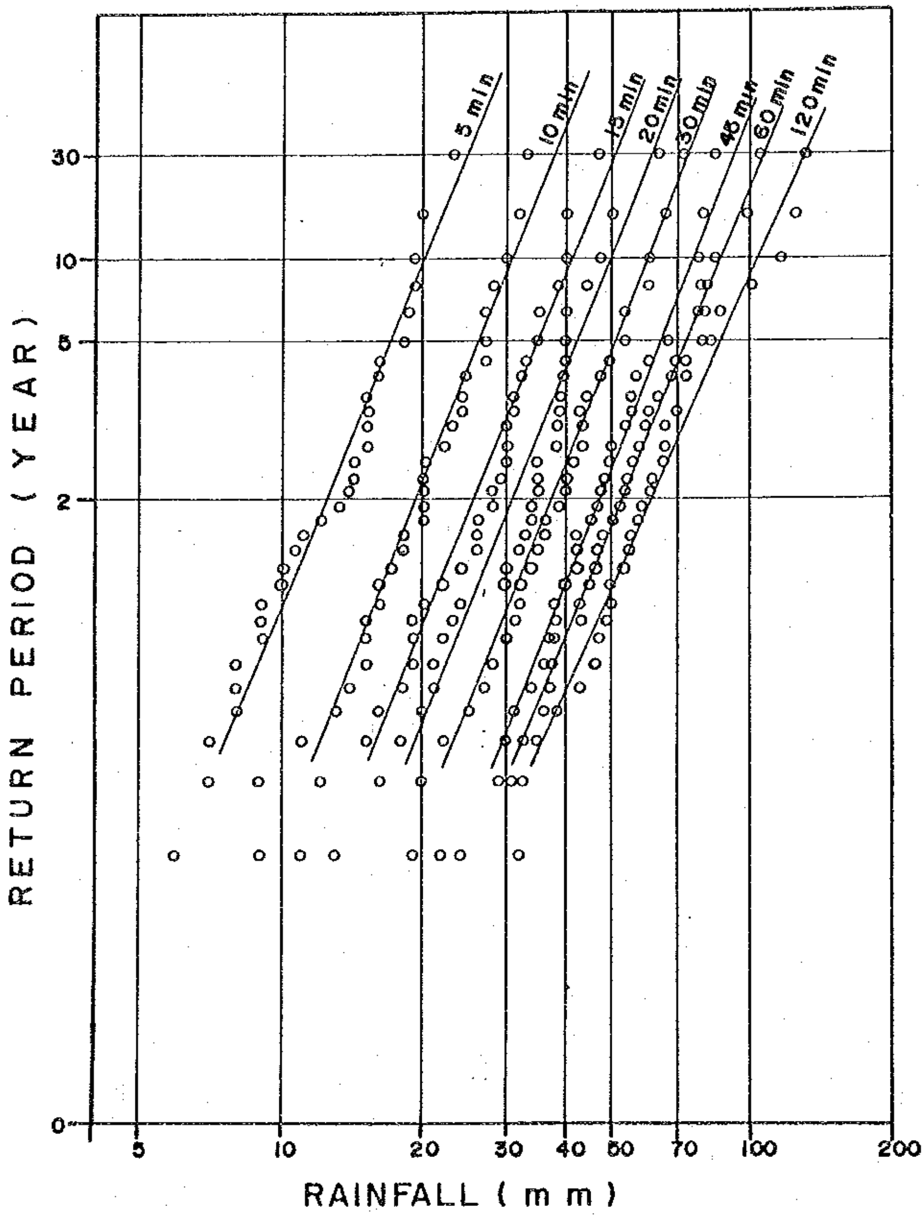


Fig. 1-6. (1/2) PROBABLE RAINFALL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

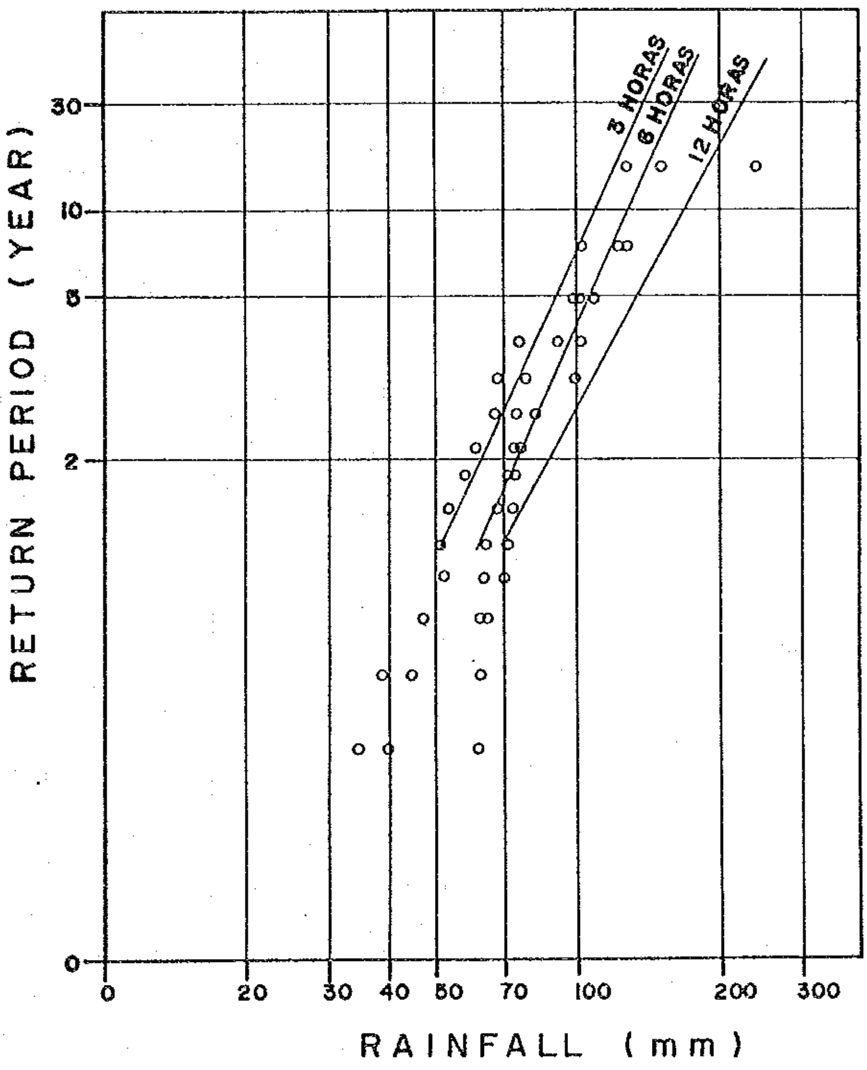


Fig. 1-6. (2/2) PROBABLE RAINFALL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

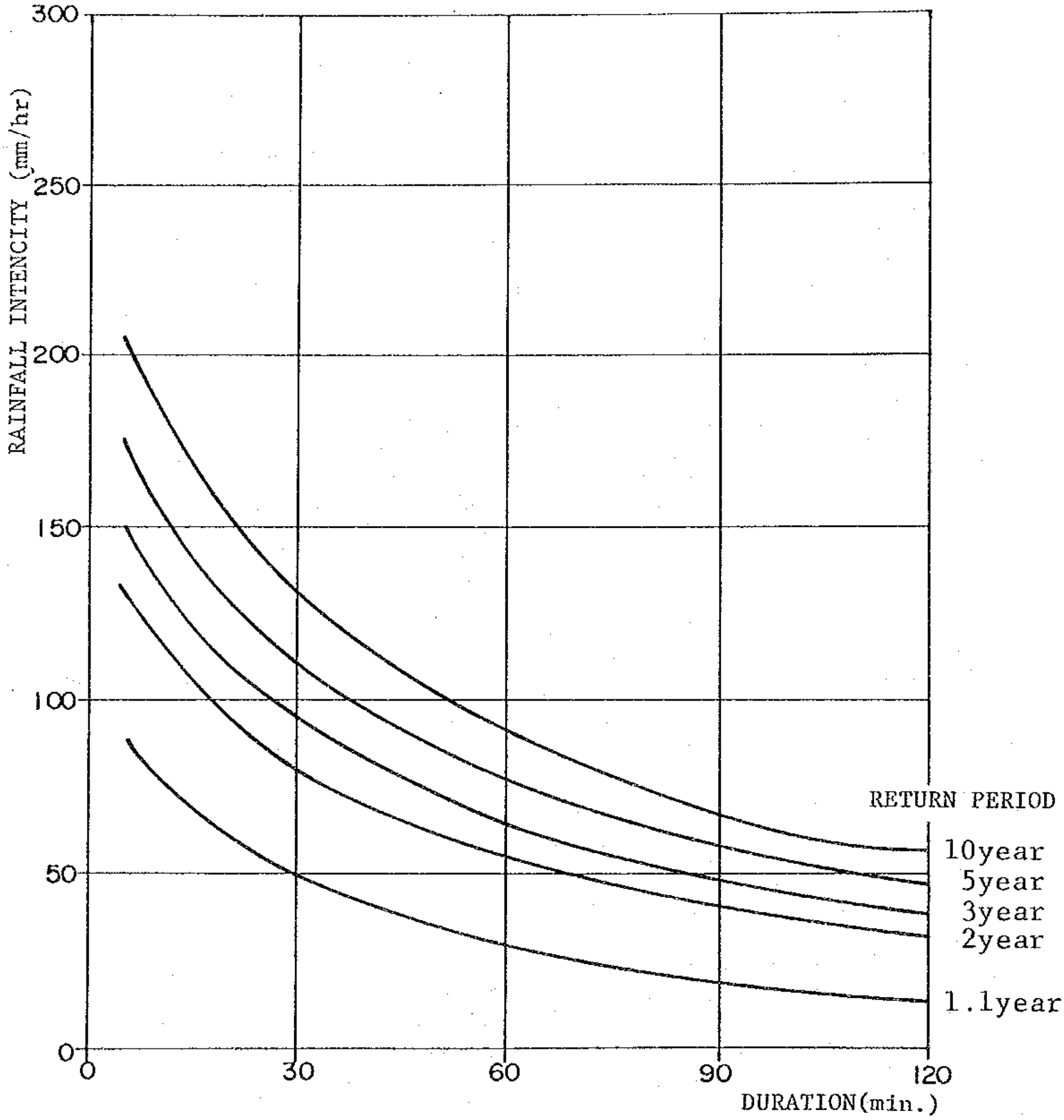


Fig. 1-7. RAINFALL INTENSITY-DURATION CURVE

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

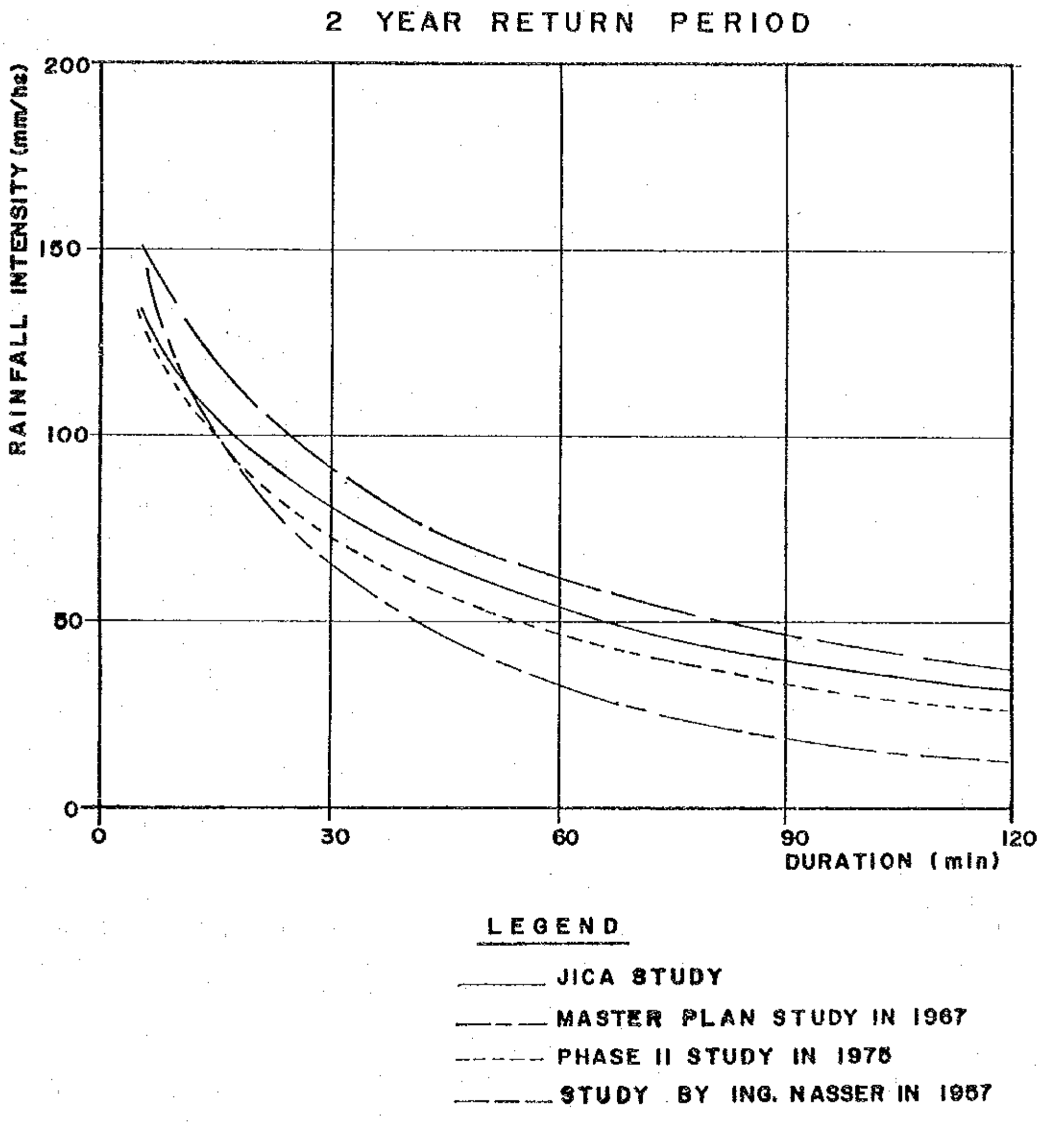


Fig. 1-8. (1/2) COMPARISON OF RAINFALL INTENSITY DURATION CURVE	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

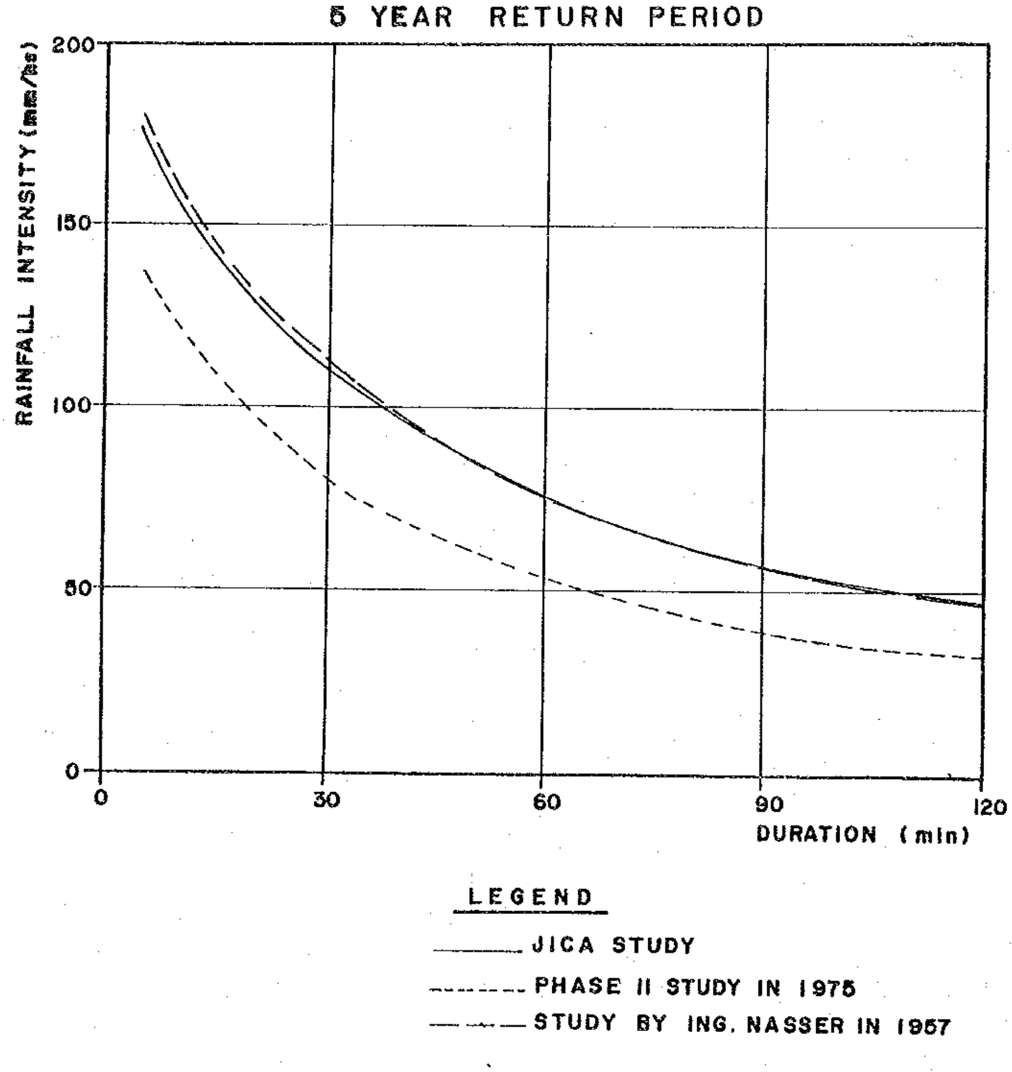


Fig. 1-8. (2/2) COMPARISON OF RAINFALL INTENSITY-DURATION CURVE

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

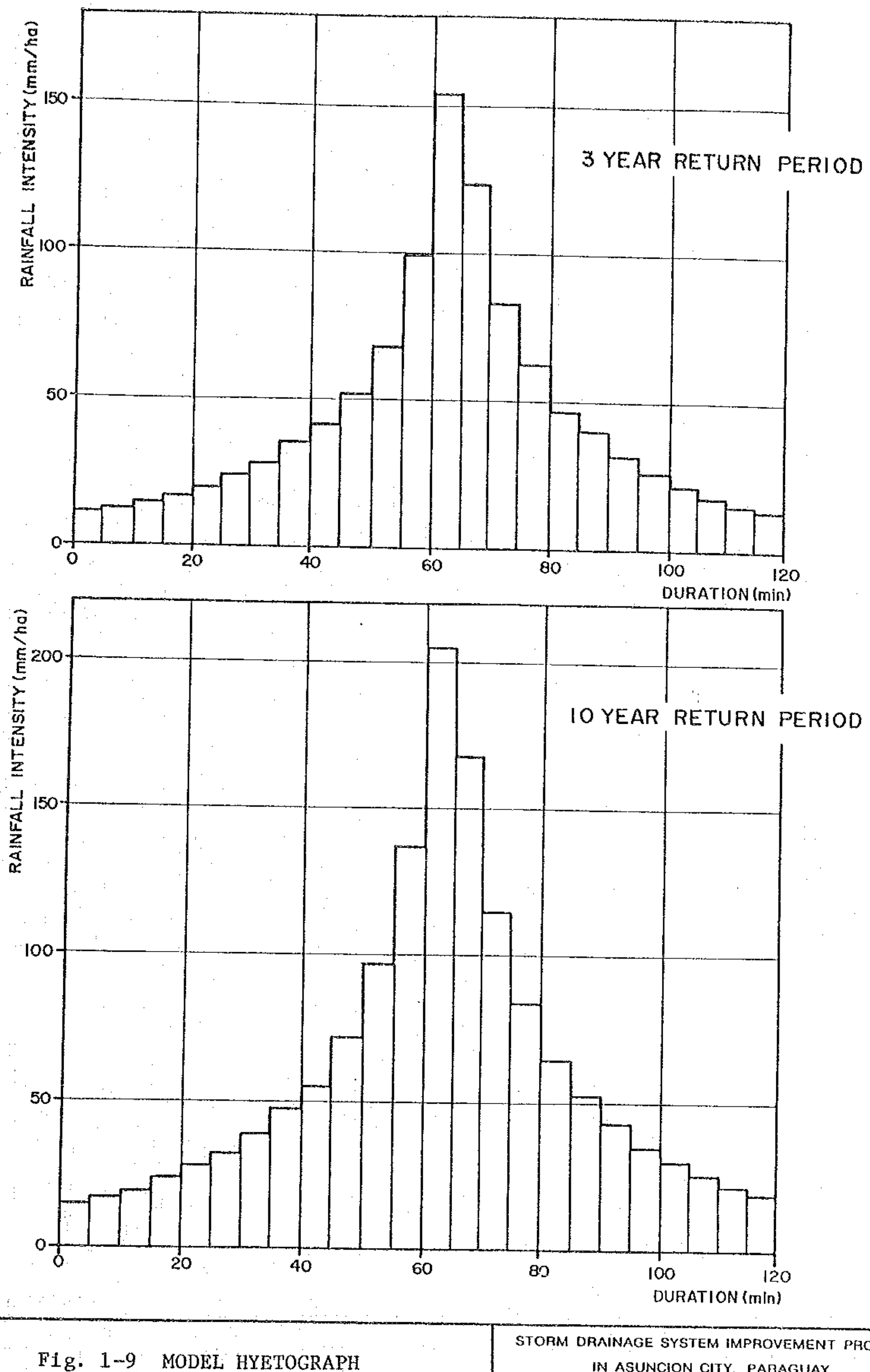


Fig. 1-9 MODEL HYETOGRAPH

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

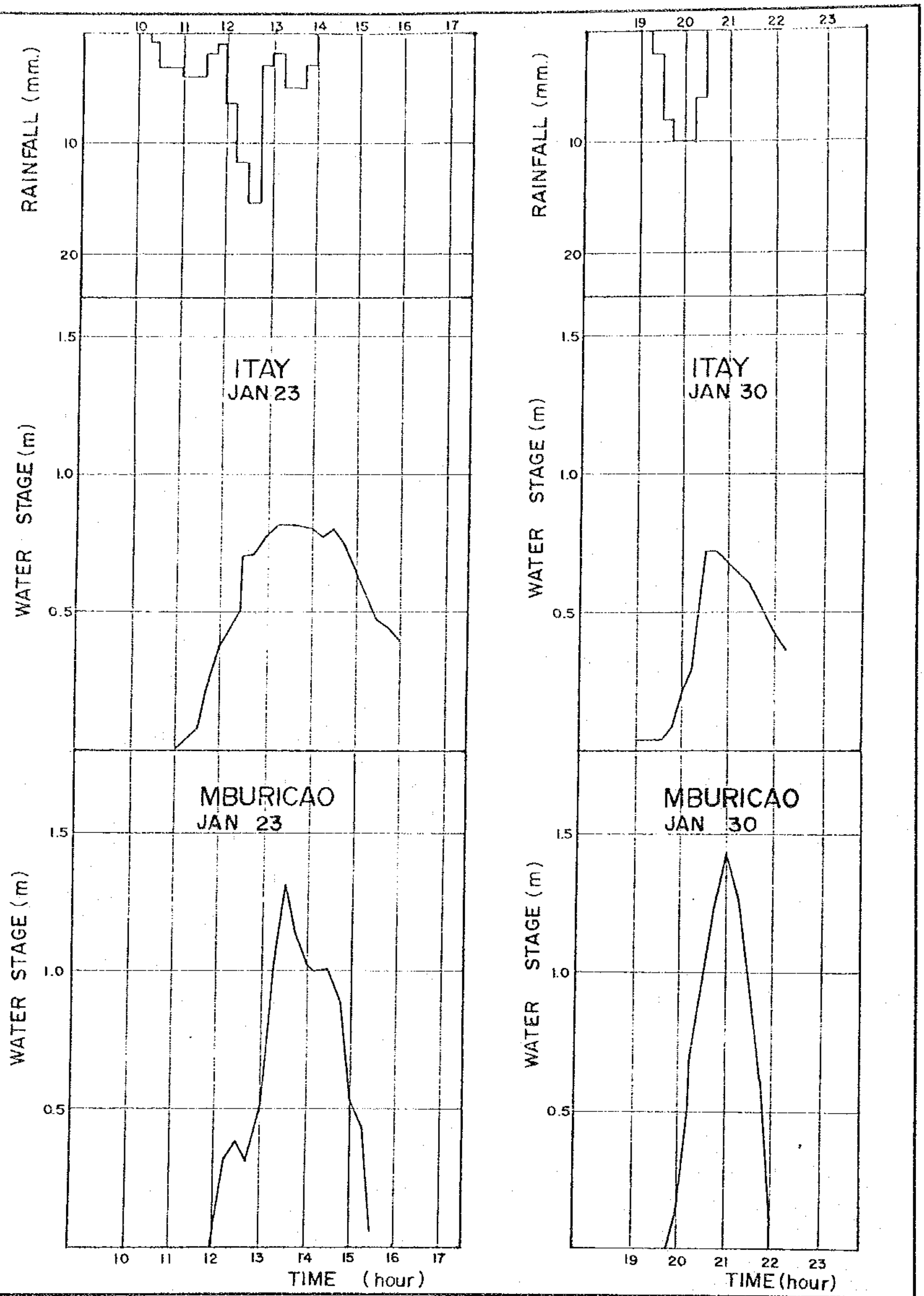


Fig. 1-10(1/4) HYETOGRAPH AND HYDROGRAPH OF STORM

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

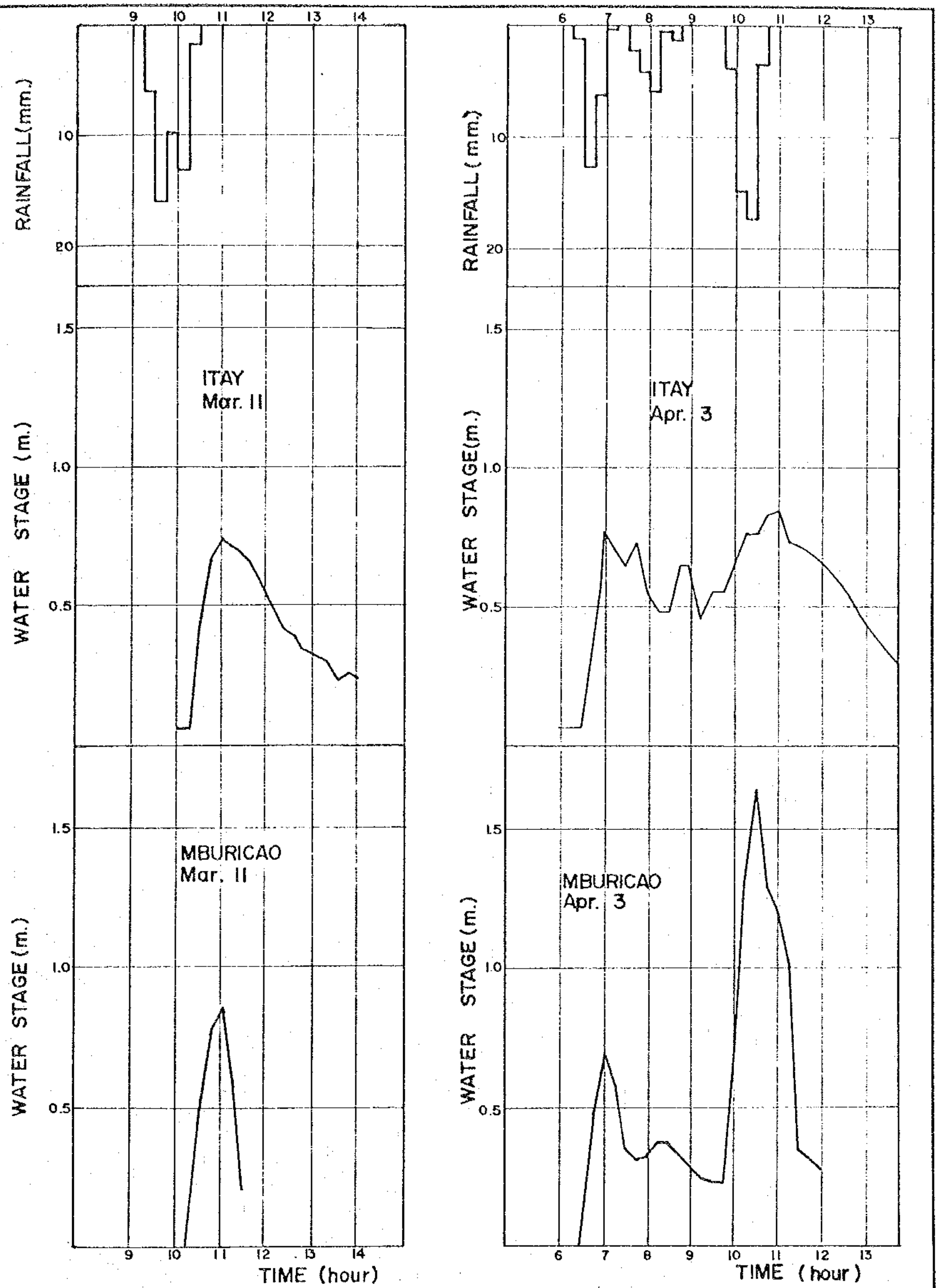


Fig. 1-10(2/4) HYETOGRAPH AND HYDROGRAPH OF STORM

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

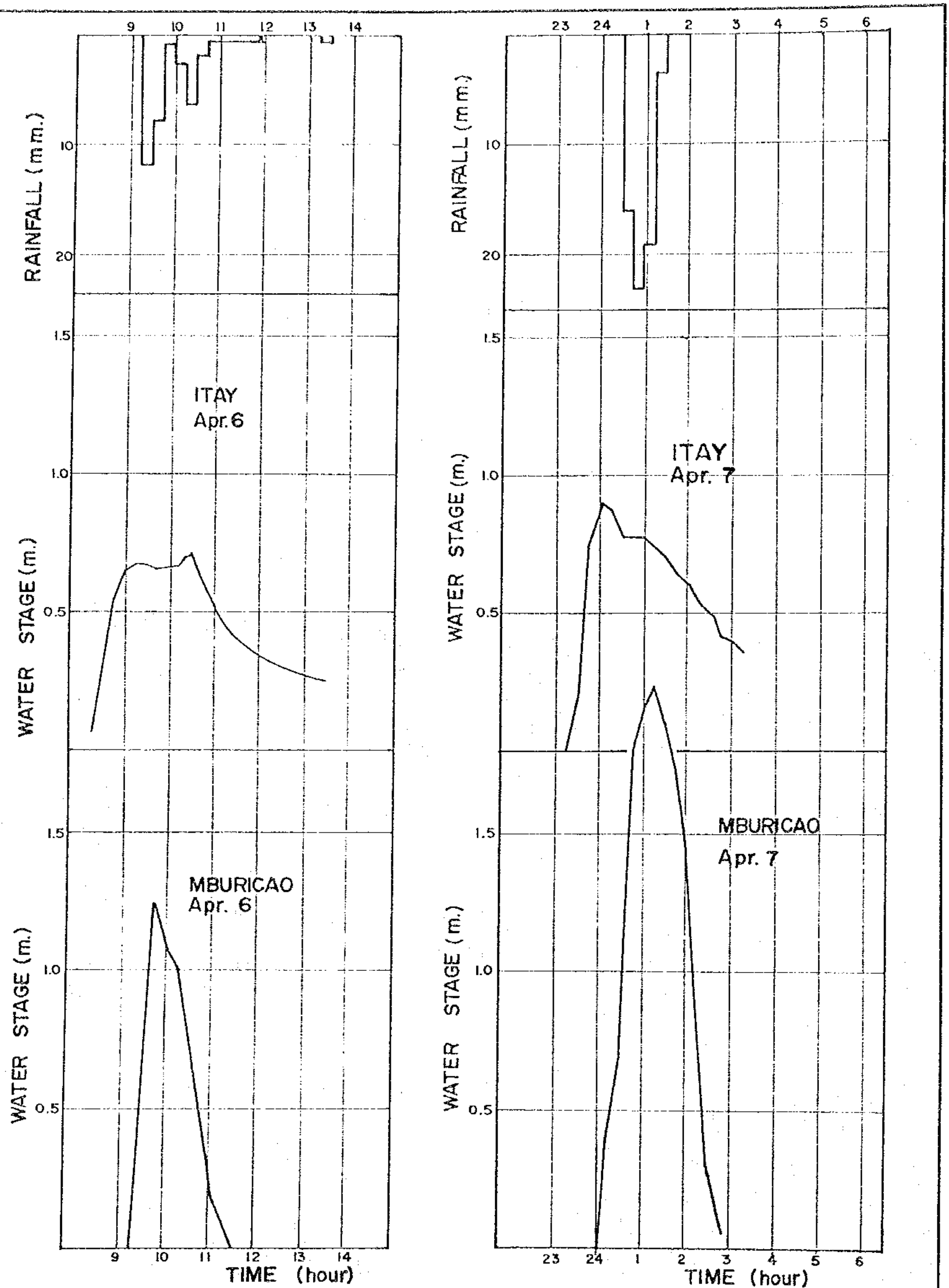


Fig. 1-10(3/4) HYETOGRAPH AND HYDROGRAPH OF STORM	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

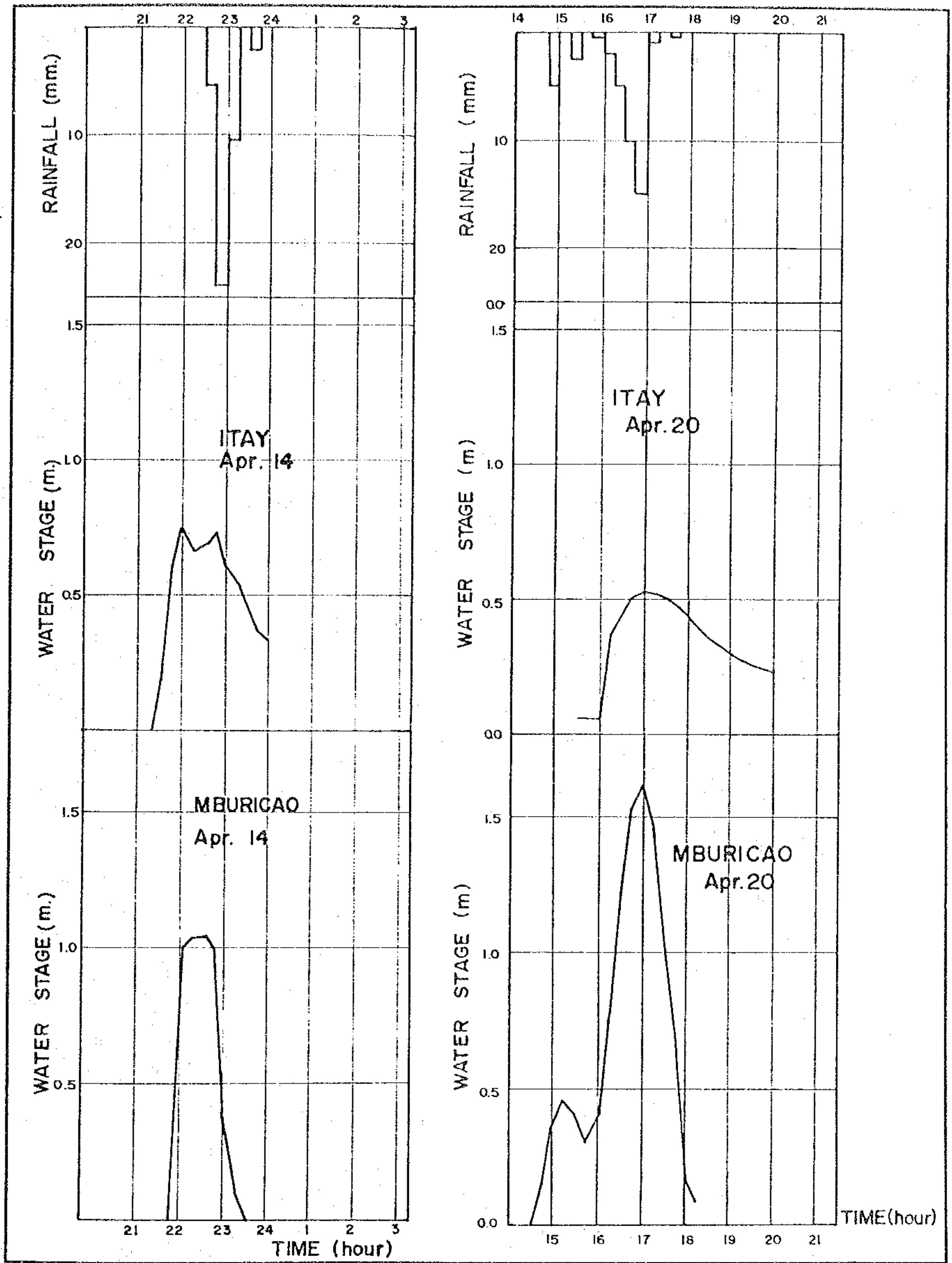


Fig. 1-10(4/4) HYETOGRAPH AND HYDROGRAPH OF STORM

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

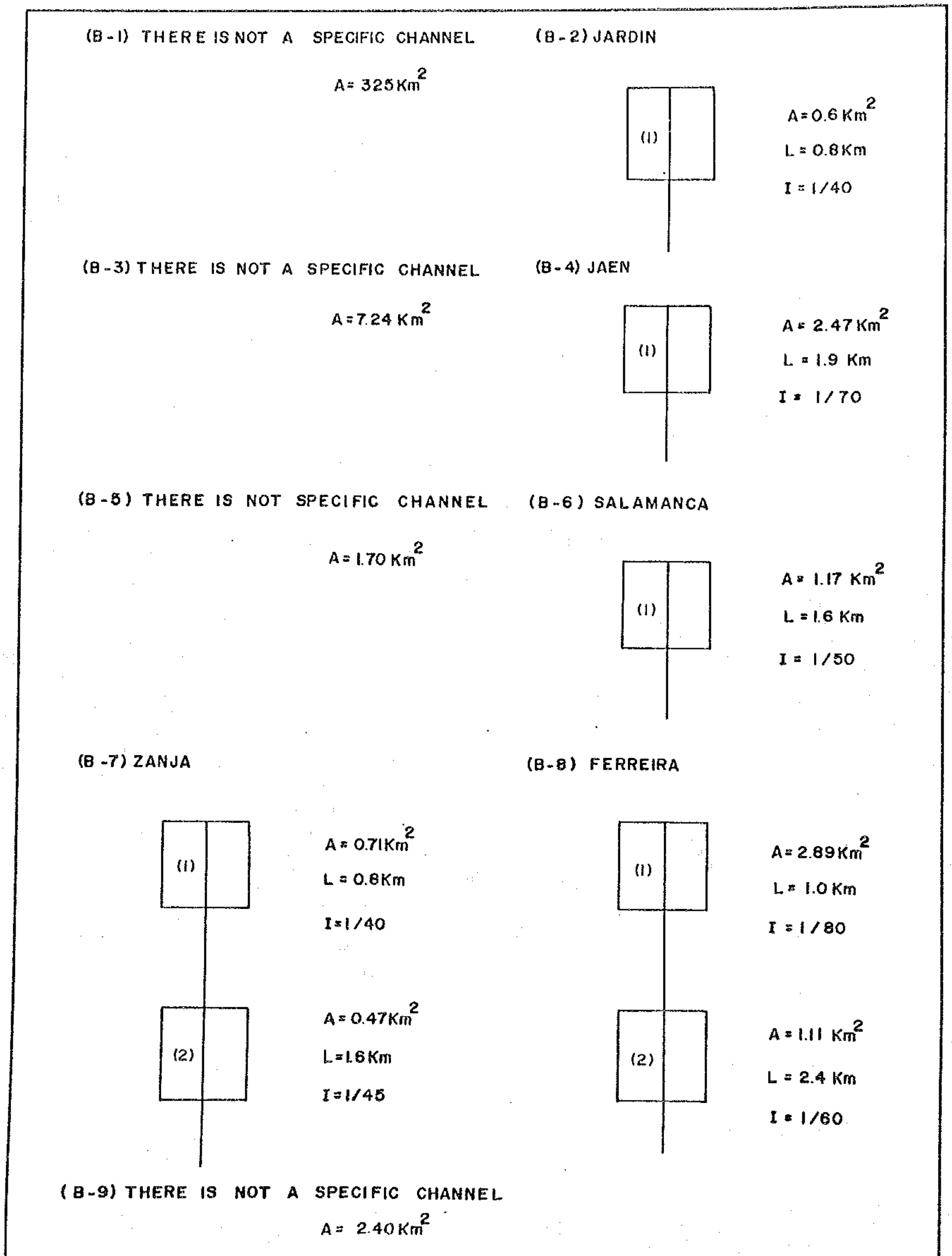


Fig. 1-11(1/6) RUNOFF CALCULATION MODEL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

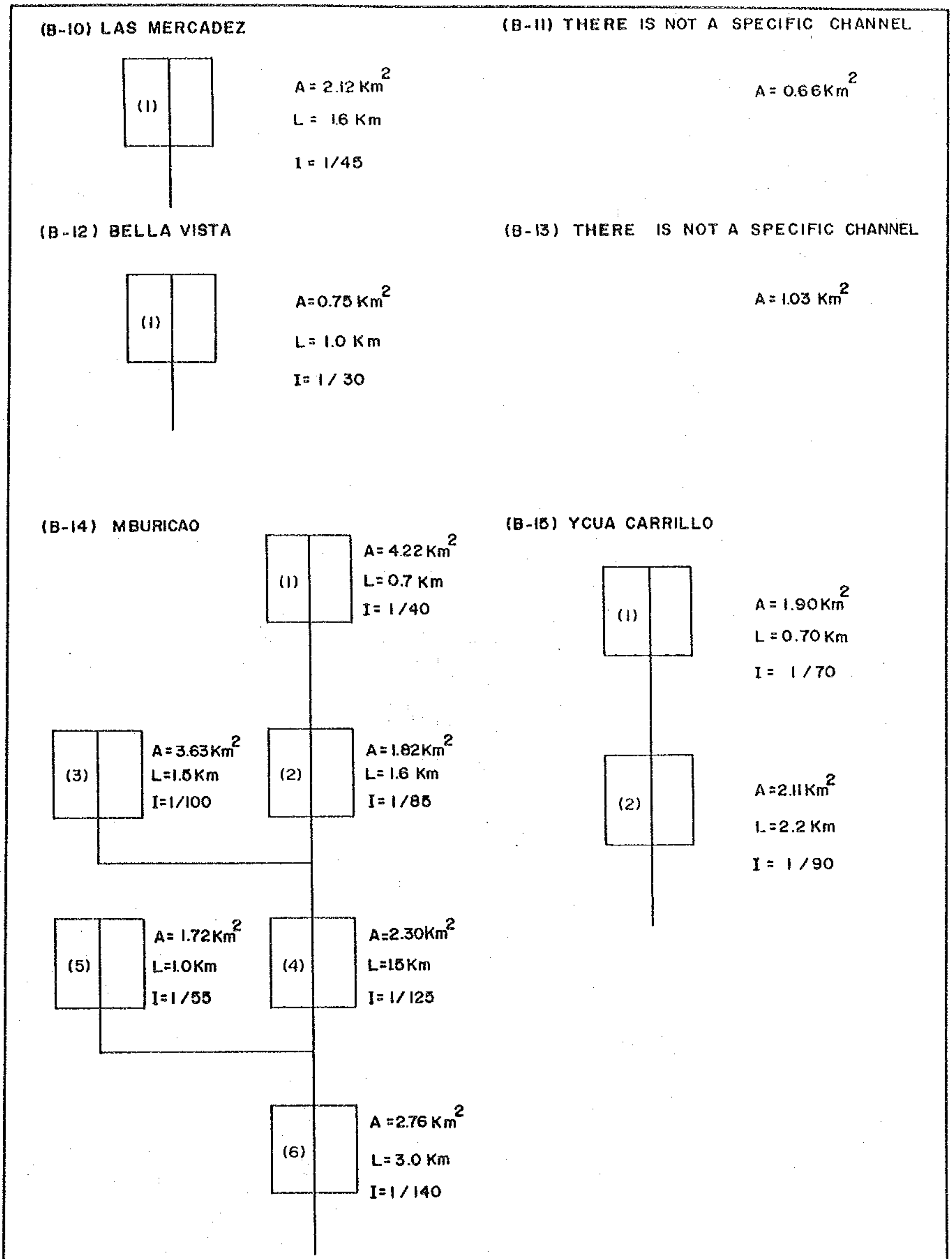


Fig. 1-11(2/6) RUNOFF CALCULATION MODEL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

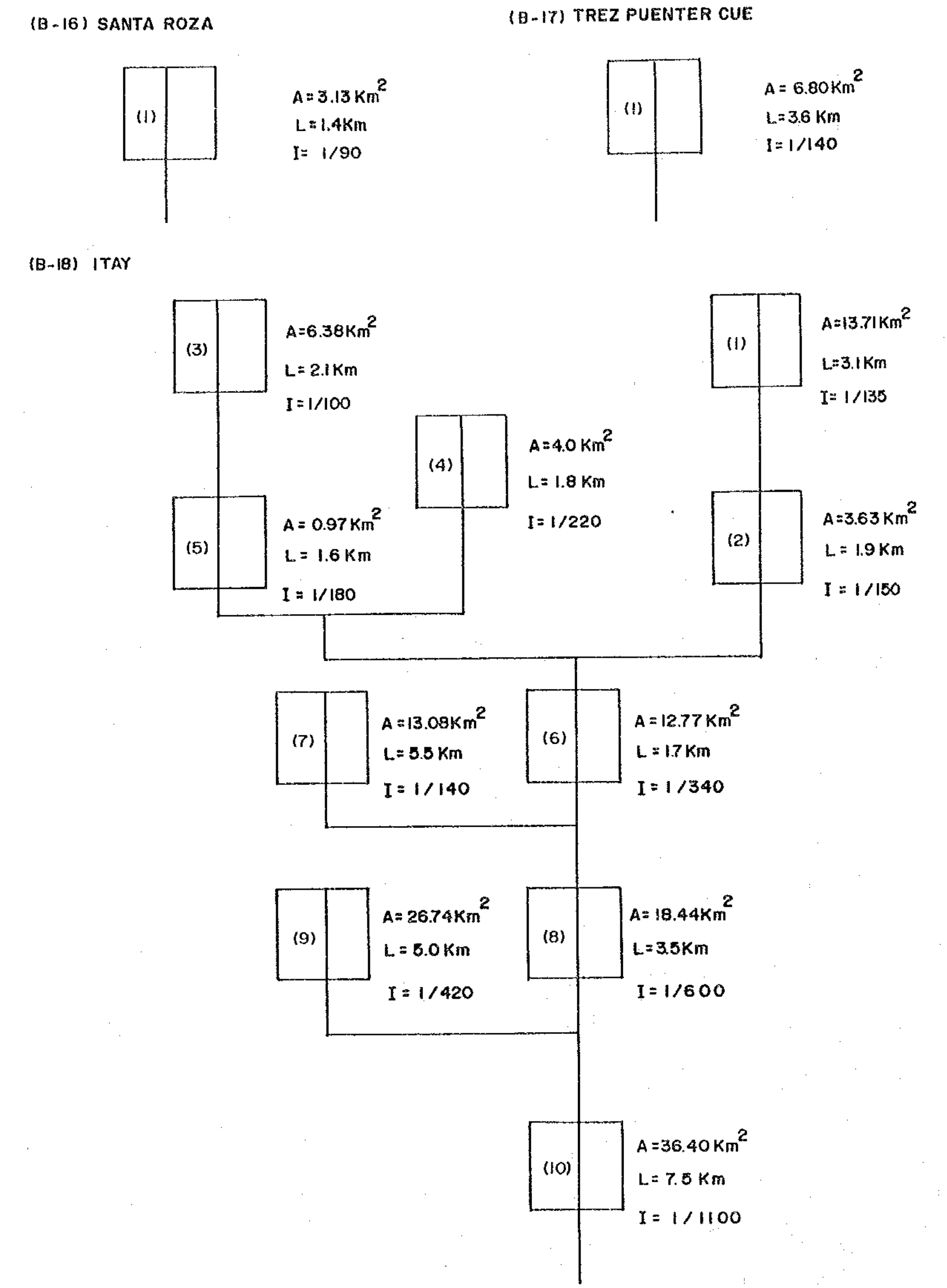


Fig. 1-11(3/6) RUNOFF CALCULATION MODEL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

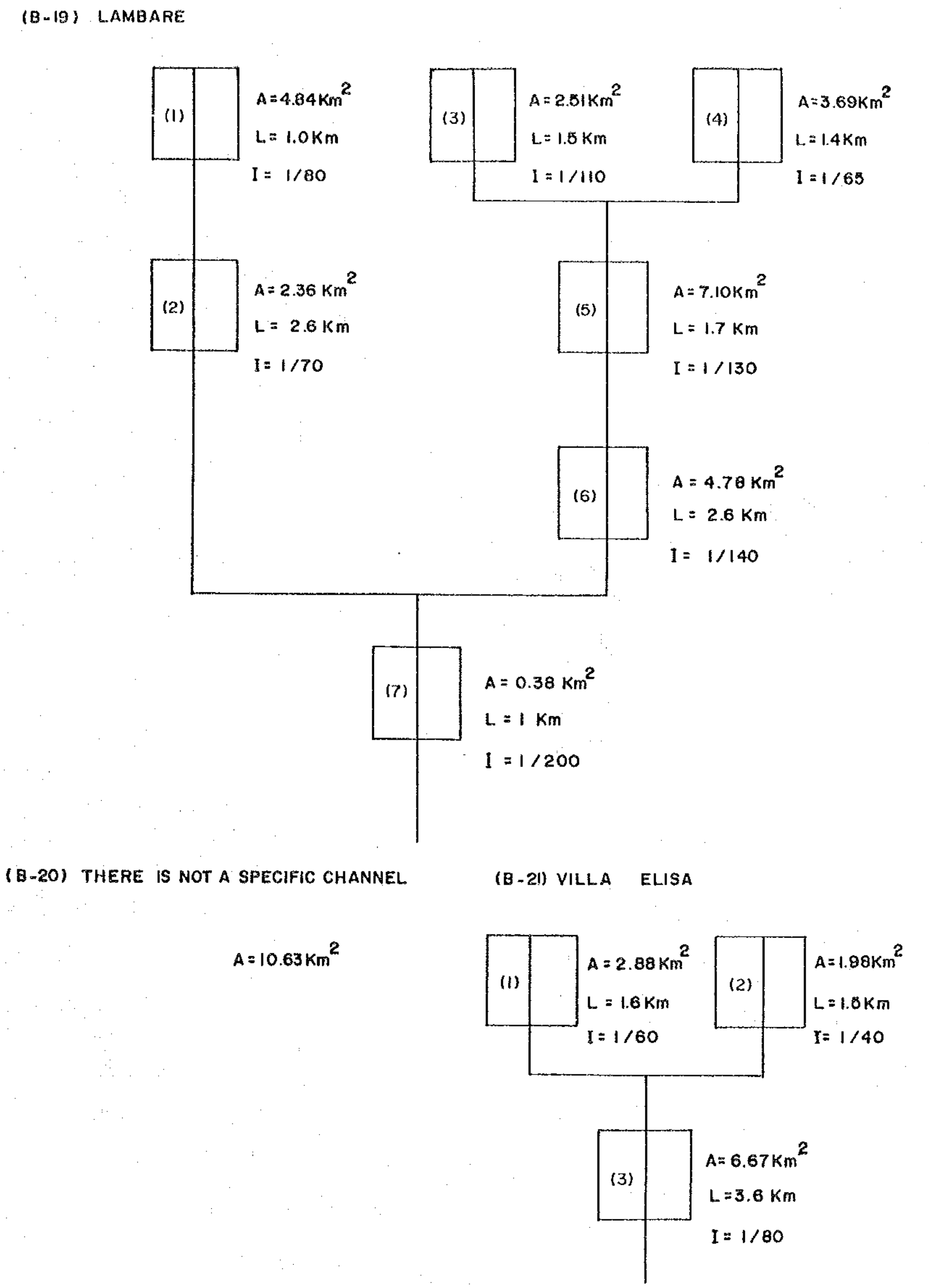
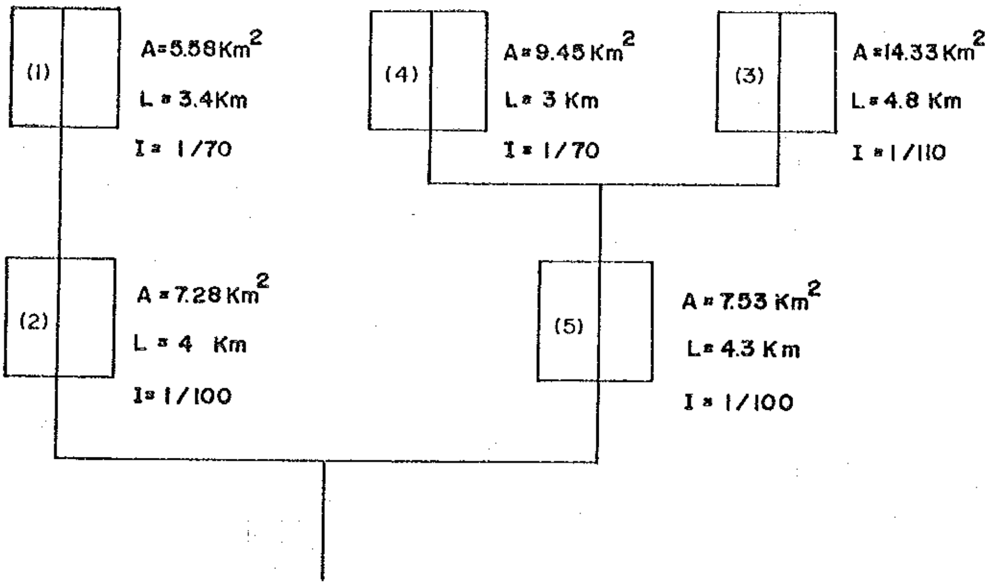


Fig. 1-11(4/6) RUNOFF CALCULATION MODEL

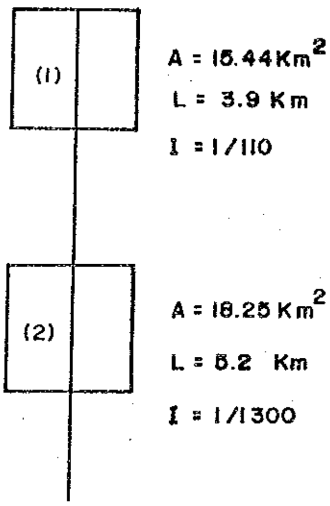
STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

(B-22) NEMBY



(B-23) SAN LORENZO



(B-24) TAYAZUAPE

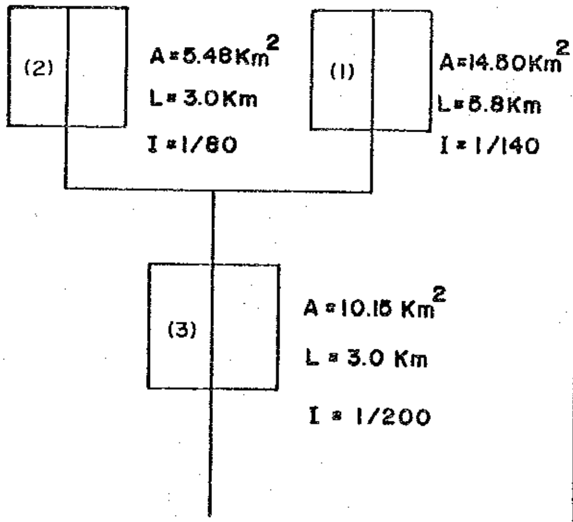
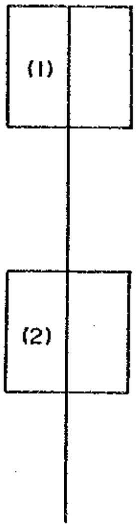


Fig. 1-11(5/6) RUNOFF CALCULATION MODEL

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

(B-25) YCUA DURE



$A = 6.75 Km^2$

$L = 2.2 Km$

$I = 1/80$

$A = 5.82 Km^2$

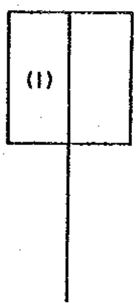
$L = 2.3 Km$

$I = 1/500$

(B-26) ZEBALLOZ CUE

$A = 2.13 Km^2$

(B-27) PASO CAI



$A = 5.49 Km^2$

$L = 3.6 Km$

$I = 1/40$

(B-28) THERE IS, NOT A SPECIFIC CHANNEL

$A = 15.65 Km^2$

Fig. 1-11(6/6) RUNOFF CALCULATION MODEL	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY
	JAPAN INTERNATIONAL COOPERATION AGENCY

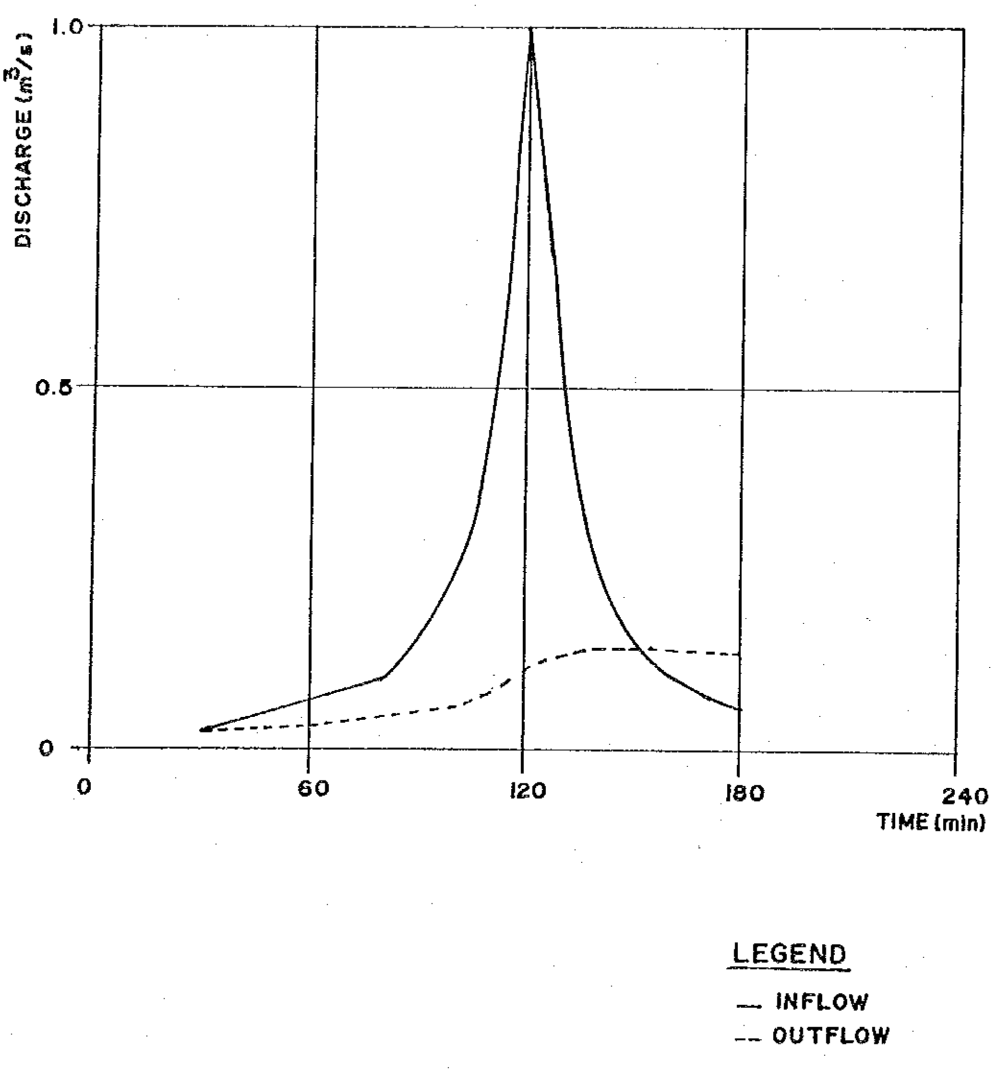


Fig. 1-12	EFFECT OF ONE UNIT OF DETENTION FACILITIES (STORAGE IN PUBLIC COMPOUND)	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

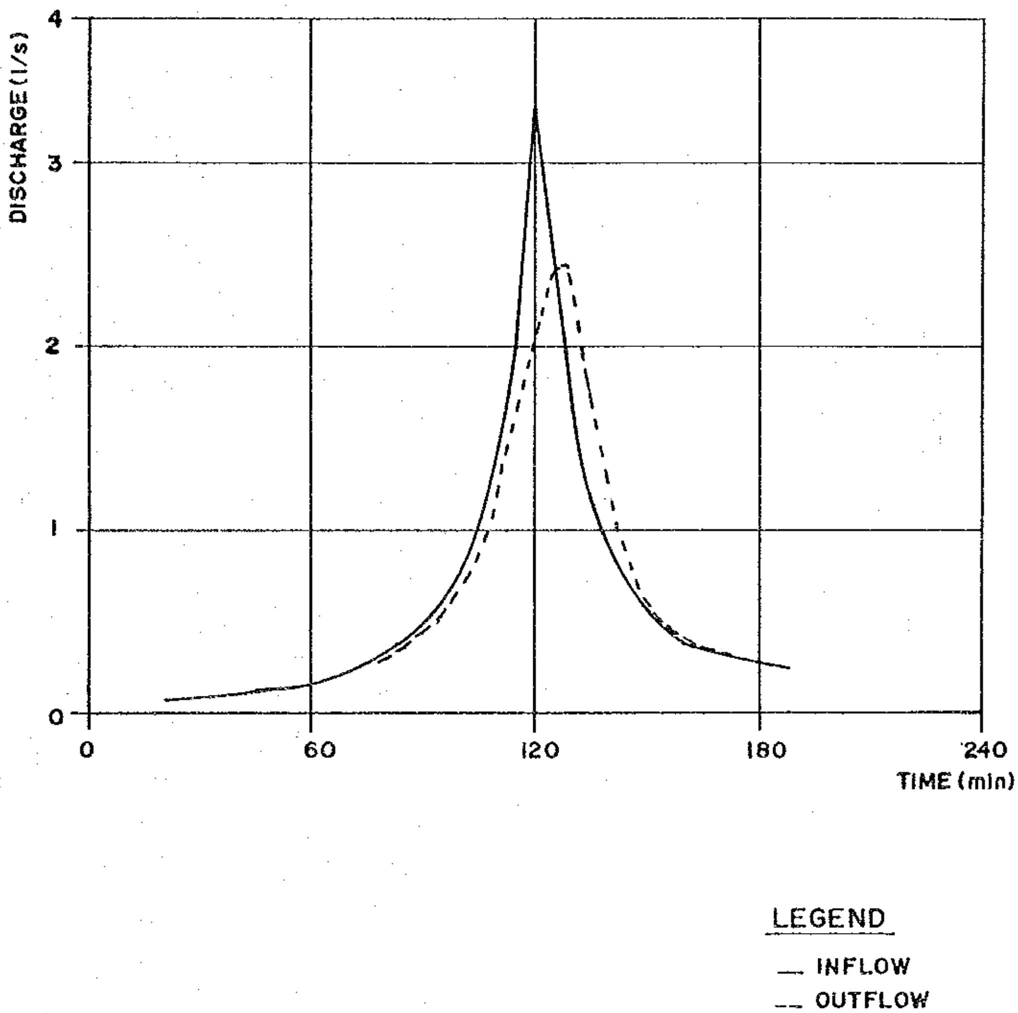


Fig. 1-13 EFFECT OF ONE UNIT OF DETENTION FACILITIES
(STORAGE IN A HOUSE LOT)

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

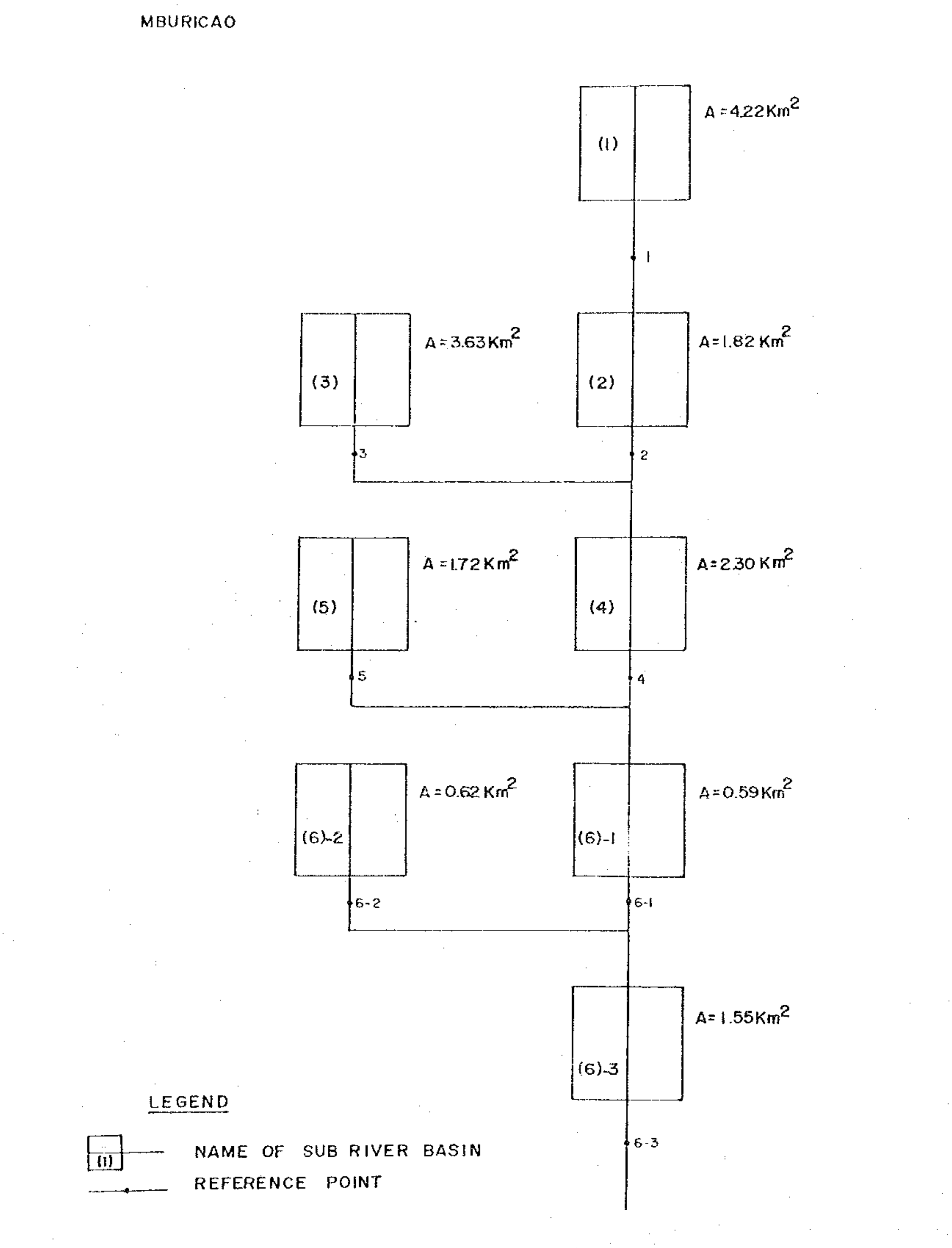


Fig. 1-14 RUNOFF CALCULATION MODEL OF MBURICAO RIVER BASIN

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

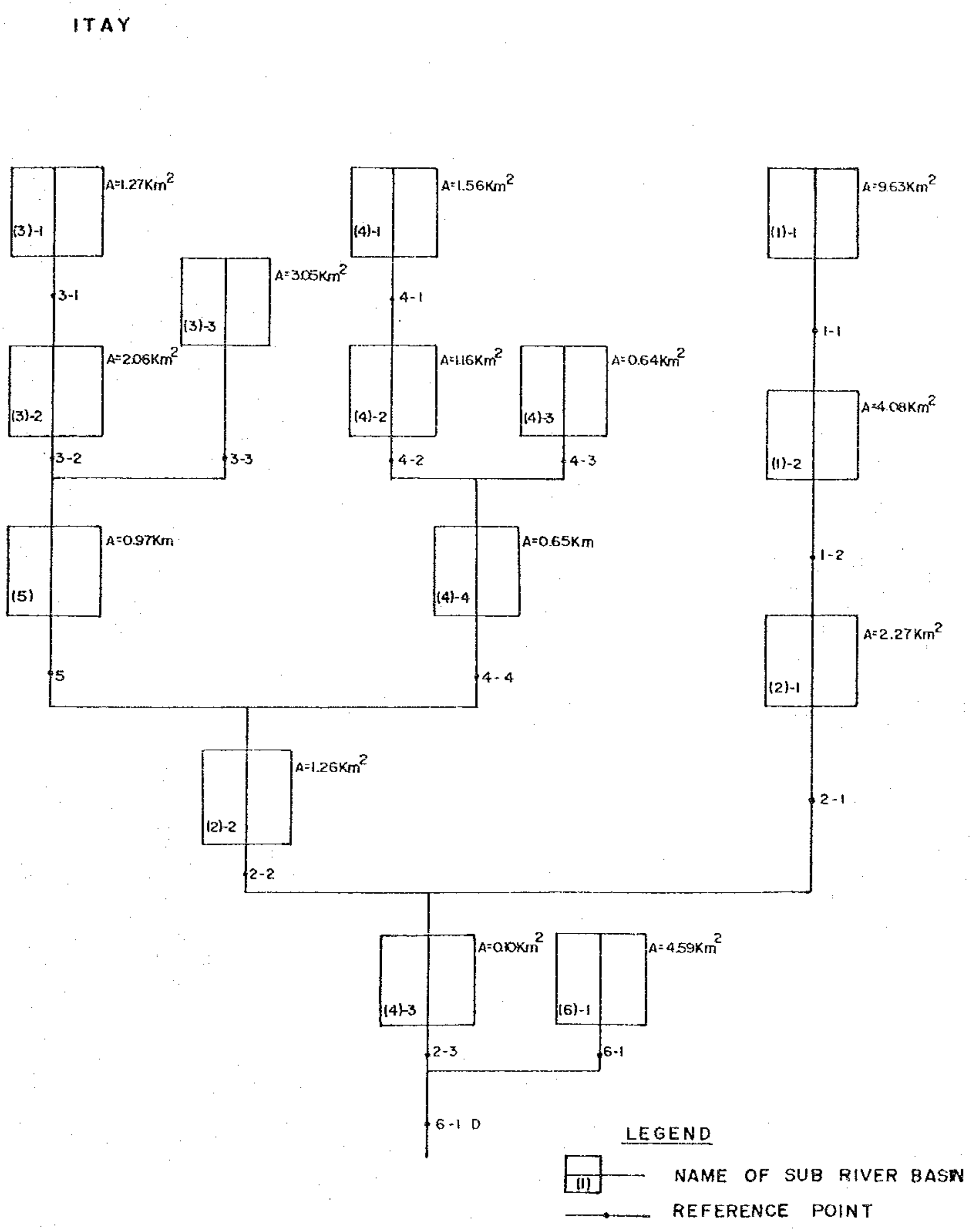


Fig. 1-15 RUNOFF CALCULATION MODEL OF ITAY RIVER BASIN

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

ANNEX

INSTALLATION OF HYDROLOGICAL OBSERVATION EQUIPMENT

1. Hydrological Equipment

For the precise study on the hydrological condition of the study area, the hydrological observation equipment listed hereunder were brought into Asuncion, Paraguay by the Study Team:

Automatic Rainfall Gauge	:	one (1) set
Automatic Water Level Gauge	:	two (2) sets
Manual Water Level Gauge	:	thirty (30) pieces

2. Site Selection

Automatic Rainfall Gauge

There is one rainfall station with a long observation record, i.e., the Airport Station, which is located outside of Asuncion City. The other station, Sajonia Station, recently resumed observation and is located at the western part of the city.

In view of the foregoing locations, it was necessary to install a rainfall station at a location in the center of Asuncion City to provide more precise data for the study on rainfall characteristics in the study area.

Rodo Point was selected as the site of the rainfall station since it is located at the center of the city and it is within the jurisdiction of CORPOSANA in connection with water supply. Its selection was based on the following:

- Rodo Point has an open space of more than 100 m^2 which is not affected by wind.

- The area is free from inundation.

- It is accessible and manpower for observation and maintenance is easily obtainable.

The location of the rainfall station is shown in Fig. A1-1.

Automatic Water Level Gauge

Automatic water level gauges were installed to precisely know runoff conditions.

Among the river basins in the study area, Mburicao and Itay river basins were selected for the installation of the gauges taking into account the following conditions:

- The basins are habitually suffering from serious flood damage.

- Much of their areas are urbanized and the urbanized areas are expected to expand in the future.

- They have sufficient catchment areas for the study on the relation between rainfall and runoff discharge.

The sites for the installation of equipment for the above river basins were decided on the basis of the following conditions:

- The steady flow on these sites are not affected by severe turbulent flow.

- There is a steady channel course and riverbed.

- The sites are accessible and manpower for observation and maintenance is easily obtainable.

The location of these stations is shown in Fig. A1-1.

Manual Water Level Gauge

Manual water level gauges were installed at several sites selected in the same manner as those for the automatic water level gauges.

Eleven (11) sites were selected as listed below:

(1) Mburicao (Jaen) River
(2) Salamanca River
(3) Moroti River
(4) Ferreira River
(5) Bella Vista River
(6) Mburicao River (Downstream Reaches)
(7) Mburicao River (Middle Reaches)
(8) Ycua Carrillo River
(9) Itay River
(10) Itay River
(11) Lambare River

The location of the gauges is shown in Fig. A1-1.

3. Equipment Installation

All the equipment were installed at the selected sites by CORPOSANA with instructions from the Study Team for the Storm Drainage System Improvement Project in Asuncion City, in cooperation with the counterpart personnel.

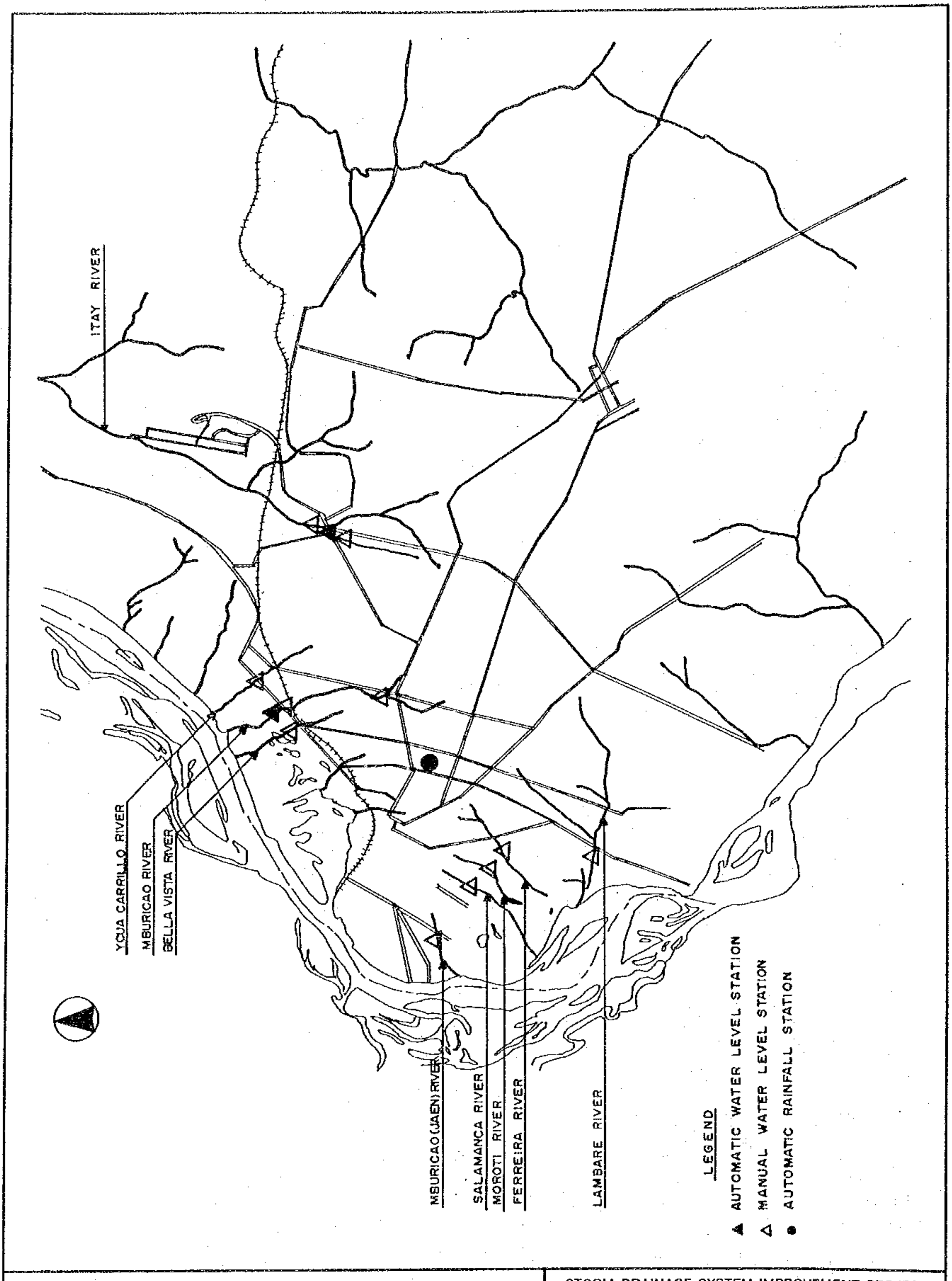


Fig. A1-1. LOCATION OF INSTALLED OF HYDROLOGICAL OBSERVATION EOUIPMENT	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

INFILTRATION TEST RESULTS IN THE BASIC AND MASTER PLAN STUDY STAGE

1. Purpose of the Test

Infiltration facilities are proposed in the project as one of the detention facilities for the effective regulation of flood discharge by promoting infiltration of storm water into the ground.

Since the possibility of providing infiltration facilities depends on soil conditions, infiltration tests were conducted to know the infiltration capacity of the ground for the study on the infiltration facility that is practicable in the area.

2. Test Procedure

The test includes the following works:

- Preliminary study on the permeable condition of the area.
- Selection of the location for the test.
- Execution of the test.
- Compilation of the test results.

The contents of these works are described hereunder.

3. Preliminary Study

For the effective execution of the infiltration test, preliminary study was performed to grasp soil permeability in the study area.

Available Data

There are few available data to define soil permeability in the study area except the results of the following tests:

(1) Boring Test
(2) Infiltration Test for Sewage

Boring Test Results

Boring tests were performed during the construction period of the second stage storm water drainage project by CORPOSANA. Since the area of the tests covered only 510 ha in Asuncion City, the permeability of soil in the whole study area can hardly be presumed.

However, as far as the area of 510 ha is concerned, it is mostly covered by soil of clayey-sand and, judging from this condition, soil permeability in this area is not so high.

Results of Infiltration Test for Sewage

This test was performed by CORPOSANA to study the possibility of sewage disposal into the ground. To distinguish this test by CORPOSANA from the test conducted by the Study Team for the Storm Drainage Improvement System Project in Asuncion City, the former test is hereinafter called the Simple Test for convenience and the latter, the Regular Test.

The Simple Test was executed at 38 points in the surrounding areas of Asuncion City where sewage system has not been adequately provided. The procedures were as follows:

- Digging a hole with the size of 60 cm to 100 cm in depth and 30 cm in diameter.
- Filling the hole with water and leaving it for one day.
- Pouring water up to 15 cm the following day and recording the time required for the water to recede by 1.0 cm.

From the results of the Simple Test, the permeability of 38 points were classified into three (3) categories in accordance with the required time, as follows:

(1) More than 20 minutes.
(2) Between 20 and 5 minutes.
(3) Less than 5 minutes.

In this classification, the permeability for the soil with the required time of 5 minutes corresponds to the quantity of infiltration water of 0.14 ltr/min and that for 20 minutes corresponds to 0.04 ltr/min as calculated by the following equation.

$$Q_t = \frac{3.14 \times r^2 \times h}{1000}$$

$$Q_5 = \frac{15 \times 15 \times 3.14 \times 1}{1000} /5 = 0.14$$

where;

r	:	radius (cm)
h	:	water depth (cm)
t	:	time (min)
Q_t	:	water quantity (ltr/min)

The said test points classified according to the soil permeability are shown in Fig. A2-2. Soil in the area surrounding Asuncion City may not have high permeability, judging from the fact that points of more than 5 minutes are predominant in the area.

Execution of Simple Test at Additional Points

There may be some differences in the results on infiltration capacity between the Simple Test and the Regular Test. However, the Simple Test which can be easily executed is effective to grasp the infiltration capacity of soil in the study area.

Since the Simple Test was executed on only the surrounding area of Asuncion City, tests at additional ten (10) points covering the core of Asuncion City were performed prior to the execution of the Regular Test.

The test points and the results are shown in Fig. A2-1, together with the previously executed points and results by CORPOSANA.

According to the results, the points with poor permeability are slightly predominant to those with high permeability even inside of city area.

4. Selection of the Points of the Regular Test

Based on the results of Simple Test, the points of the Regular Test were selected considering the following conditions:

- The test points are in the basin that has been rapidly urbanized, so that the necessity for providing storm control facilities is expected to be high.

- The points are in the area with relatively high permeability of soil with the time of less than 5 minutes in the Simple Test.

Five (5) points were eventually selected for the Regular Test, located as follows:

(1) Mburicao River Basin
(2) Lambare River Basin
(3) Ferreira River Basin
(4) Itay River Basin (upper reaches)
(5) Itay River Basin (middle reaches)

The location of the points is shown in Fig. A2-2.

5. Execution of the Test

For the said five (5) points, the test was executed on the basis of the following method and equipment.

<u>Test Method</u>

The method used in the test is briefly explained as follows:

- Digging a hole with the size of about 1.0 m in depth and 30 cm in diameter, more or less.

- Putting a pipe with the diameter of 30 cm into the hole and refilling the soil in the space outside of the pipe.

- Pouring water into the pipe and keeping the water level at 60 cm deep.

- Recording the quantity of water poured into the pipe which corresponds to infiltration water volume.

Necessary Equipment

The necessary equipment for this test are listed below:

- One tank filled with water.
- One tube with a valve to control the flow of water.
- One cumulative flow meter.
- One pipe of at least 1.0 m long and 30 cm in diameter.
- One scale, stopwatch and thermometer.
- One shovel and crushed stones.
- One field notebook and pencils.

6. Result Compilation

The test results are mainly expressed by the relation between the duration and the quantity of infiltration water. Fig. A2-3 shows the test results.

Based on the said relation, a coefficient which is expressed in the following equation was calculated:

$$Q_{15} = Qt \times \frac{Vt}{V_{15}}$$

$$a_{15} = \frac{Q15}{(H \times A)}$$

where;

Q_t, Q_{15}	:	Quantity of infiltration water at temperature t°C and 15°C (cm^3/hr)
V_t, V_{15}	:	Coefficient of viscosity at t°C and 15°C (cm^2/s)
A	:	Area of infiltration (= Bottom area of pipe; cm^2)
a_{15}	:	Coefficient of infiltration capacity of soil (ltr/hr)

The quantity of infiltration water (Q_t) observed at the 5 points was 115 ltr/hr on an average at the temperature of 25°C, and the coefficient (a_{15}) converted from the said quantity (Q_t) was 2.1 ltr/hr.

Since the infiltration capacity of soil was finally expressed by the following equation for the design of the infiltration facilities, the said coefficient (a_{15}) was also modified by several factors: K_1, K_2, K_3, K_4, and X.

$$Q_p = 10^3 \times a_p \times H_d \times A_p$$

$$a_p = K_1 \times K_2 \times K_3 \times K_4 \times a_{15} \times X$$

where;

Q_p	:	Quantity of infiltration water for the design of the infiltration facility (ltr/hr).
H_d	:	Design water depth of infiltration facility (m).
A_p	:	Design bottom area of infiltration facility (m^2).
a_p	:	Design coefficient of infiltration capacity (ltr/hr).
K_1	:	Coefficient based on the type of infiltration facility (K_4 = 0.5)
K_2	:	Coefficient for choking trash (= 0.85)
K_3	:	Coefficient for modification by the influence of underground water (= 0.9)
K_4	:	Coefficient for modification by the influence of previous rainfall (= 0.95).
X	:	Safety factor (= 1.0)

Similarly, the design coefficient of infiltration capacity (a_p) by a trench was obtained at 0.76 ltr/hr by the above equation.

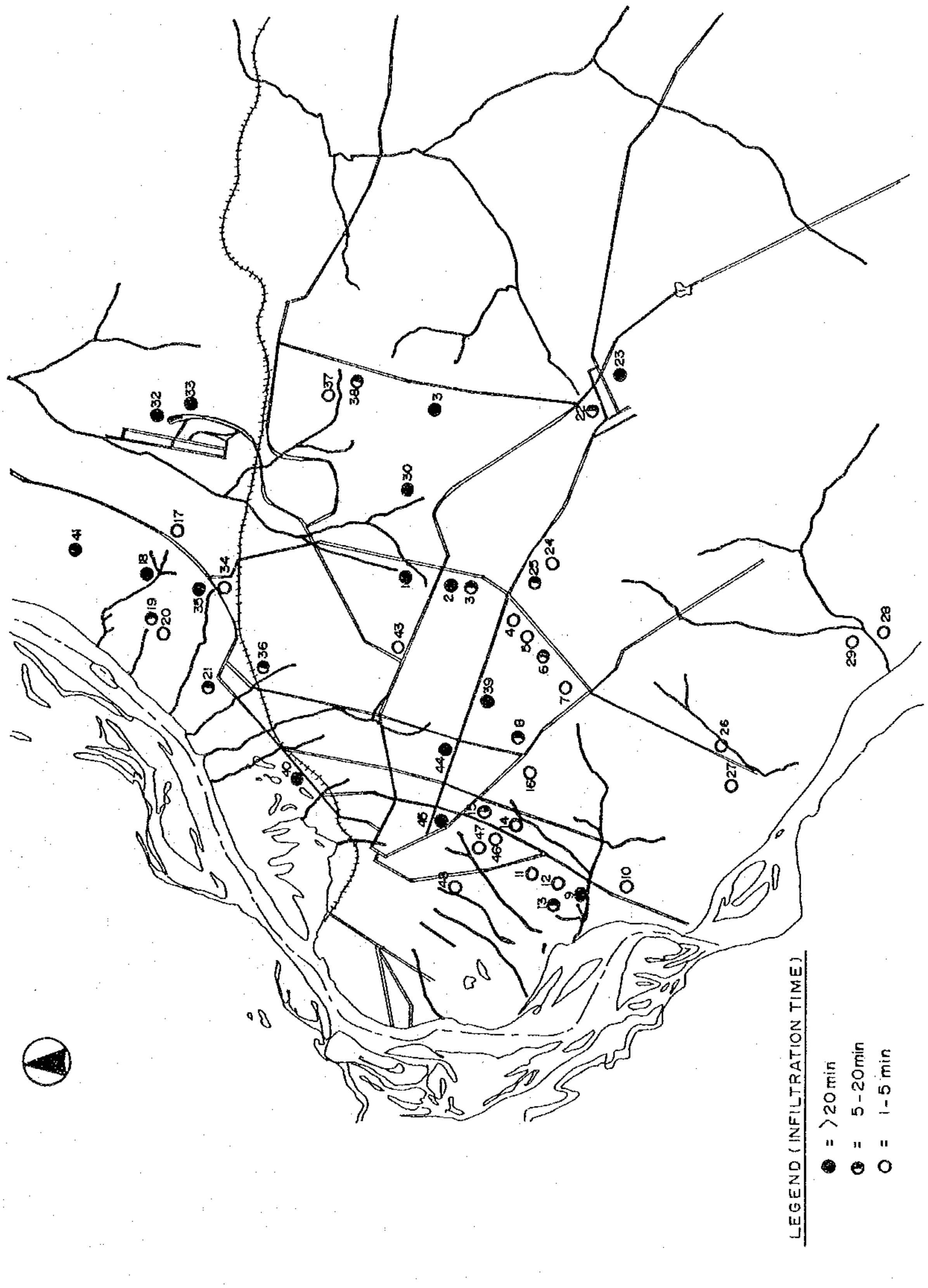


Fig. A2-1. LOCATION OF SIMPLE TEST

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

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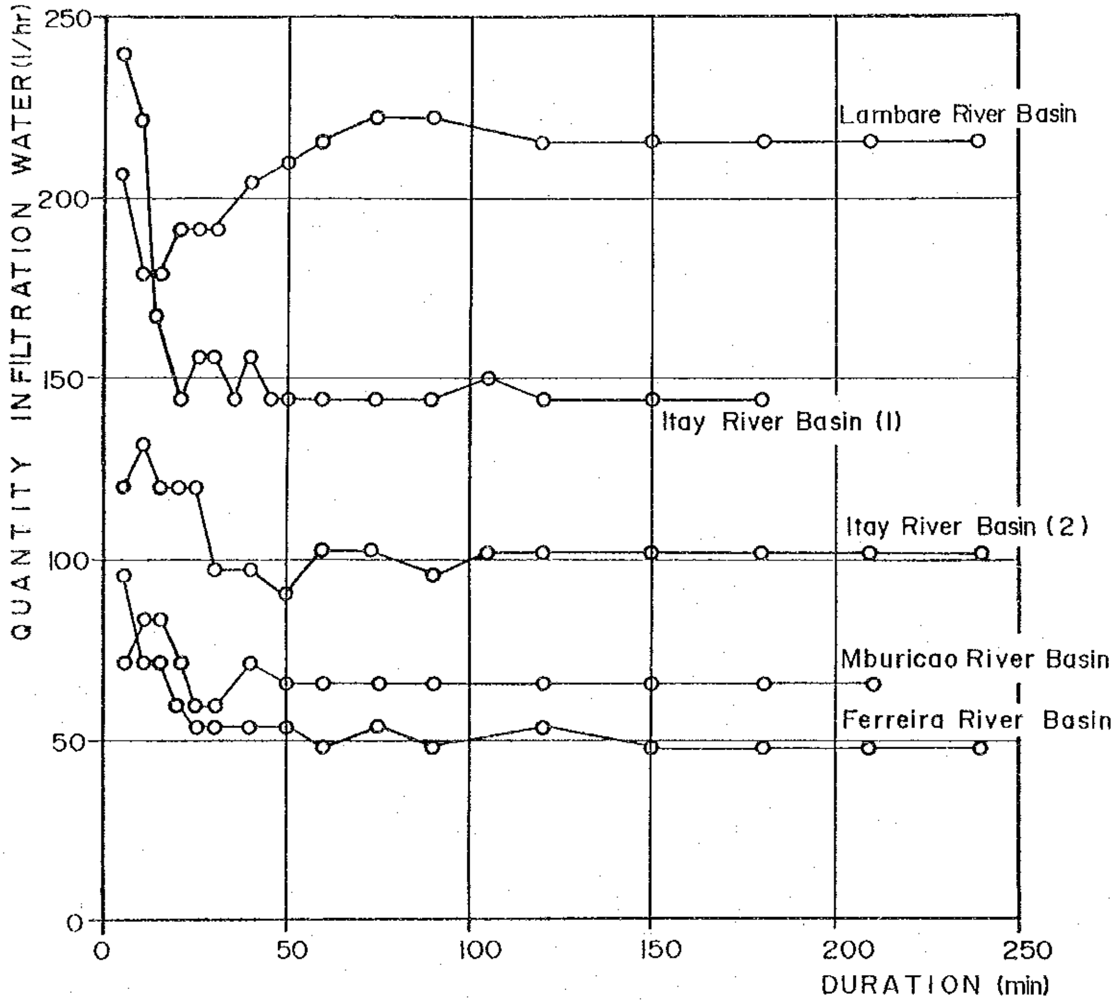

Fig. A2-2. RELATION BETWEEN THE DURATION AND QUANTITY OF INFILTRATION WATER

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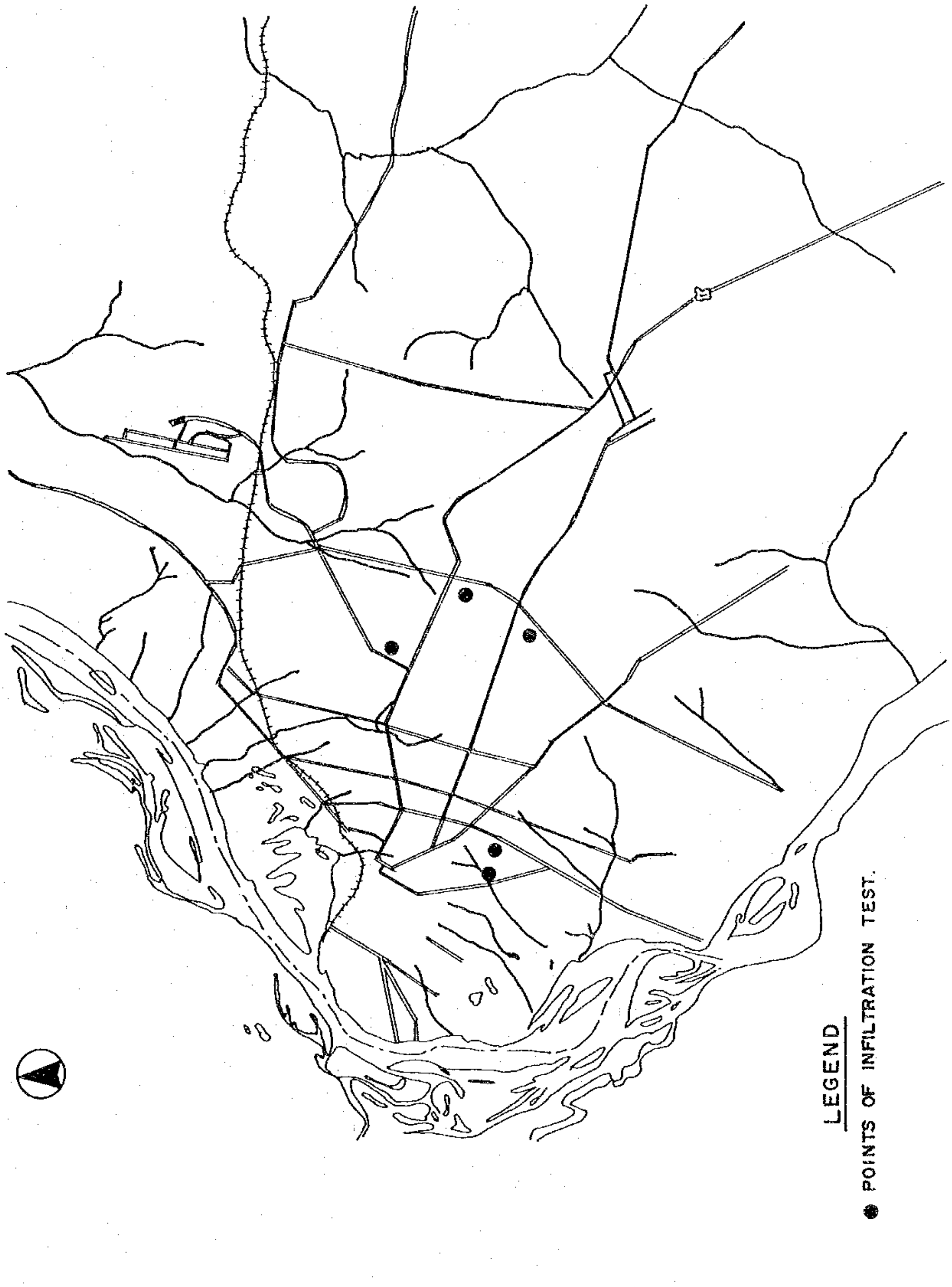

Fig. A2-3. LOCATION OF REGULAR TEST

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

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INFILTRATION TEST IN THE FIRST STAGE PROJECT STUDY

1. Purpose of the Test

In the Master Plan Study (Annex 2), the infiltration tests were conducted at several points to identify the infiltration capacity of the study area by using a vertical type of test facilities, as shown in Fig. A3-1. In the First Stage Project Study, the purpose of the test is to confirm the infiltration effects, as well as the infiltration capacity more precisely, by using a trench type of facilities that are expected to be installed in the study area in the future. These test facilities may also serve to demonstrate the functions to the inhabitants to make them understand and thus cooperate in the installation of similar facilities in their own housing lots.

2. Preliminary Study and Selection of Sites

A preliminary study, including site investigation, preliminary infiltration tests, etc., was carried out to select the suitable sites for the test. In accordance with CORPOSANA's recommendation, the four (4) points selected for the test are the yards of the CORPOSANA Office, the CORPOSANA's Sports Club, the Sajonia Meteorology Branch Office, and the Dr. C. A. Lopez Technical High School.

Tests on the infiltation capacities of the four sites were conducted by using the aforementioned vertical type of test facilities to narrow down the test points to two (2). The test results are summarized in terms of infiltration water volume (Qt) in the order of superiority, as follows:

(1) Sajonia Meteorology Branch Office : Qt = 150 ltr/hr

(2) CORPOSANA's Sports Club : Qt = 60 ltr/hr

(3) CORPOSANA Office : Qt = 52 ltr/hr

(4) Dr. C. A. Lopez High School : Qt = 40 ltr/hr

From the above results, the yard of the Sajonia Meteorology Branch Office, which has the superior infiltration capacity, was selected as a site to execute the infiltration test. The yard of the CORPOSANA Office, although third in superiority, was also selected for the convenience of CORPOSANA to introduce the facilities to the people concerned.

3. Dimensions and Design Drawings of the Facilities

The trench type of infiltration facilities proposed in the Master Plan Study was designed in the following dimensions: 10 m in trench length, 60 cm in width, and 1 m in depth. The facilities are designed to gather rain falling on rooftops having an area of 100 m^2.

Since the infiltration capacity and the rooftop area to collect rainwater at the sites mentioned above will not remarkably differ from the proposed conditions in the Master Plan Study, the same dimensions taken in the Master Plan were applied to the infiltration test in this stage. Fig. A3-2 and Table A3-1 show the draft drawings prepared on the basis of the said dimensions and the list of necessary materials and equipment, respectively.

4. Results of the Infiltration Test

4.1 Estimation of the Coefficient

The infiltration capacity applied to the design of the infiltration facilities is expressed in terms of design coefficient of infiltration capacity (a_p), which can be obtained from the results of the infiltration test using the vertical type of facilities, as follows:

$$a_p = K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot a_{15} \cdot X$$

$$a_{15} = Q_{15} / (H \cdot A)$$

$$Q_{15} = Q_t \cdot V_t / V_{15}$$

where,

a_p	:	Design coefficient of infiltration capacity (ltr/hr)
Q_t, Q_{15}	:	Quantity of infiltration water at temperature t°C and 15°C (cm^3/hr)
V_t, V_{15}	:	Coefficient of viscosity at t°C and 15°C ($cm2/_s$)
A	:	Area of infiltration (bottom area of pipe; cm^2)
a_{15}	:	Coefficient of infiltration capacity of soil (ltr/hr)
K_1	:	Coefficient based on the type of infiltration facility (0.5)
K_2	:	Coefficient for choking trash (0.85)
K_3	:	Coefficient for modification by the influence of underground water (0.9)
K_4	:	Coefficient for modification by the influence of previous rainfall (0.95)
X	:	Safety factor (1.0)

As the results of the infiltration tests carried out at the two (2) places of Sajonia Meteorology Branch Office and CORPOSANA Office, the calculated coefficients are as follows:

Sajonia Meteorology Branch Office : a_p = 0.98 ltr/hr
CORPOSANA Office : a_p = 0.34 ltr/hr

4.2 Study on Adequacy of the Coefficient

The adequacy of the calculated coefficients (a_p) which were used to estimate the effects of facilities can be confirmed through the comparison between the calculated infiltration water quantity (Q_1) and the observed one (Q_2).

(1) Estimated Infiltration Water Quantity (Q_1)

The infiltration water quantity in case of the trench type was estimated by the following experimental equation:

$$Q = 10^3 \cdot a_p \cdot H_d \cdot A_p$$

where,

Q	:	Infiltration water quantity (ltr/hr)
H_d	:	Water depth (m)
A_p	:	Bottom area of the facilities (m^2)

Since the facilities are designed with the dimensions of 10 m in trench length, 60 m in width and 60 cm in water depth, the infiltration water quantity (Q_1) was calculated as follows:

Sajonia Meteorology Branch Office	:	Q_1 = 3,600 ltr/hr
CORPOSANA Office	:	Q_1 = 1,300 "

(2) Observed Infiltration Water Quantity (Q_2)

The infiltration tests were conducted in water depths of 40, 50, 60 and 70 cm. The infiltration water quantity at the design water depth of 60 cm are as follows:

Sajonia Meterology Branch Office	:	Q2 = 4,300 ltr/hr
CORPOSANA Office	:	Q_2 = 1,500 "

Comparison shows that there is a difference between the estimated and the observed values, i.e., the estimated infiltration water volume is smaller than the observed one, though not so much, because the factor of choking trash was included in the formula to estimate a more accurate volume in consideration of the deterioration of function of the facilities in the future. Eventually, it is recognized that the coefficient (a_p) relatively reflects the effects of infiltration facilities, though further observation is necessary to confirm the adequacy.

In the case that the observed infiltration water volumes are evaluated in terms of rainfall depth in an area of 100 m^2, they are

43 mm/hr and 15 mm/hr for Sajonia and CORPOSANA, respectively, which correspond to 66% and 23% of 65 mm/hr of the probable rainfall depth on the 3-year return period basis.

Table A3-1 MATERIALS NECESSARY FOR INFILTRATION FACILITIES FOR ONE PLACE

Work Item	Type	Specifications	Unit	Quantity				
				Filter & Meter Box	Ditch	Cover	Filter	Total
Soil Works	Excavation	Manual	m^3	8.0	6.0			14.0
	Filling	Manual	m^3	4.9	1.8			6.7
	Disposal		m^3	2.8	4.2			7.0
Concrete		210kg/cm2	m3	1.86				1.86
Lean Concrete			m3	0.18				0.18
Mortar		1:2	m^3	0.004				0.004
Sand			m^3	0.20	.60			0.8
Form Works			m^2	14.20				14.20
Crushed Stone		Dia.20-30mm	m^3		2.98			3.0
Pipe	Steel Pipe	Dia.40mm	m	1.10				1.10
	Perforated Pipe	Dia.100-200mm	m		10.0			10.0
Permeable Sheet			m^2		25.0			25.0
Woods	Cover	t=10, 0.8x0.8m			1			1
	Filter		m^3				0.01	0.01
Iron	Reinforcement bar	Dia.12mm	kg	68.75				68.75
	Angle	50x50x6	m			5.58		5.58
	Plate	t=6	m2			2.74		2.74
Meter			set	1				1
Wire mesh sheet		mesh of 1mm	m^2				5.54	5.54
Lift						2		2
Plastic Tube		Dia.50mm	m		0.55			0.55
Gutter			set				1	1

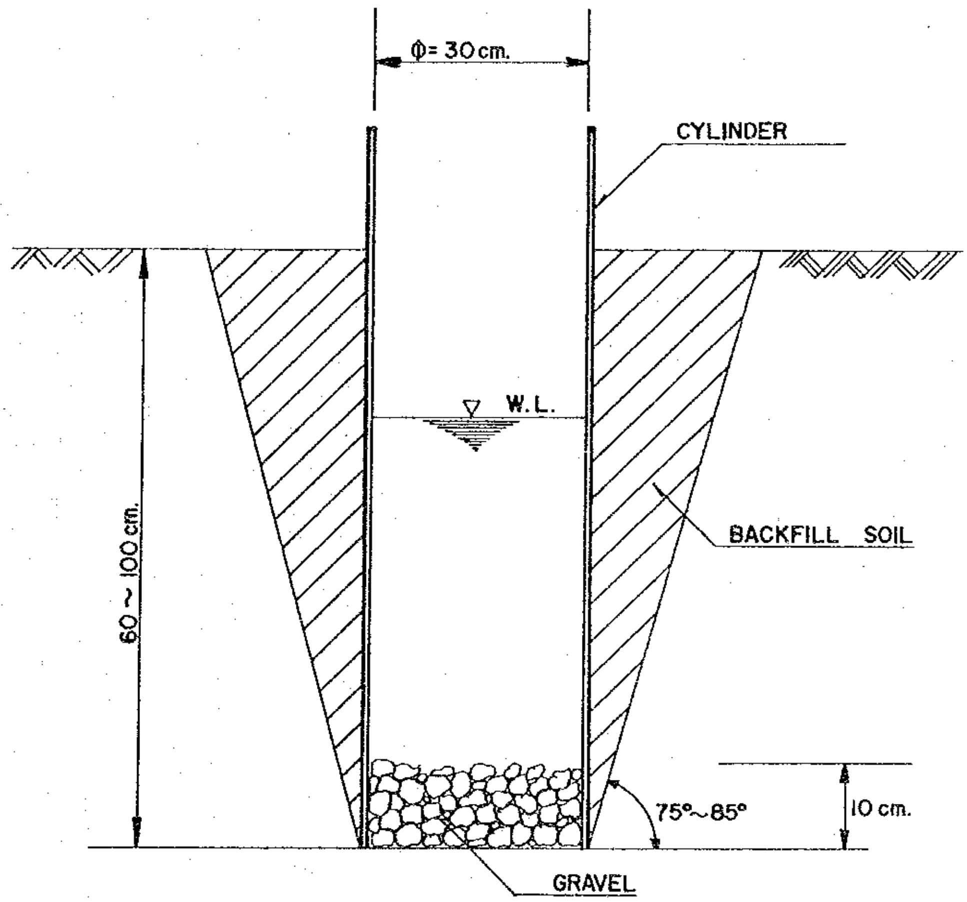

Fig. A3-1 FACILITIES FOR INFILTRATION TEST

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

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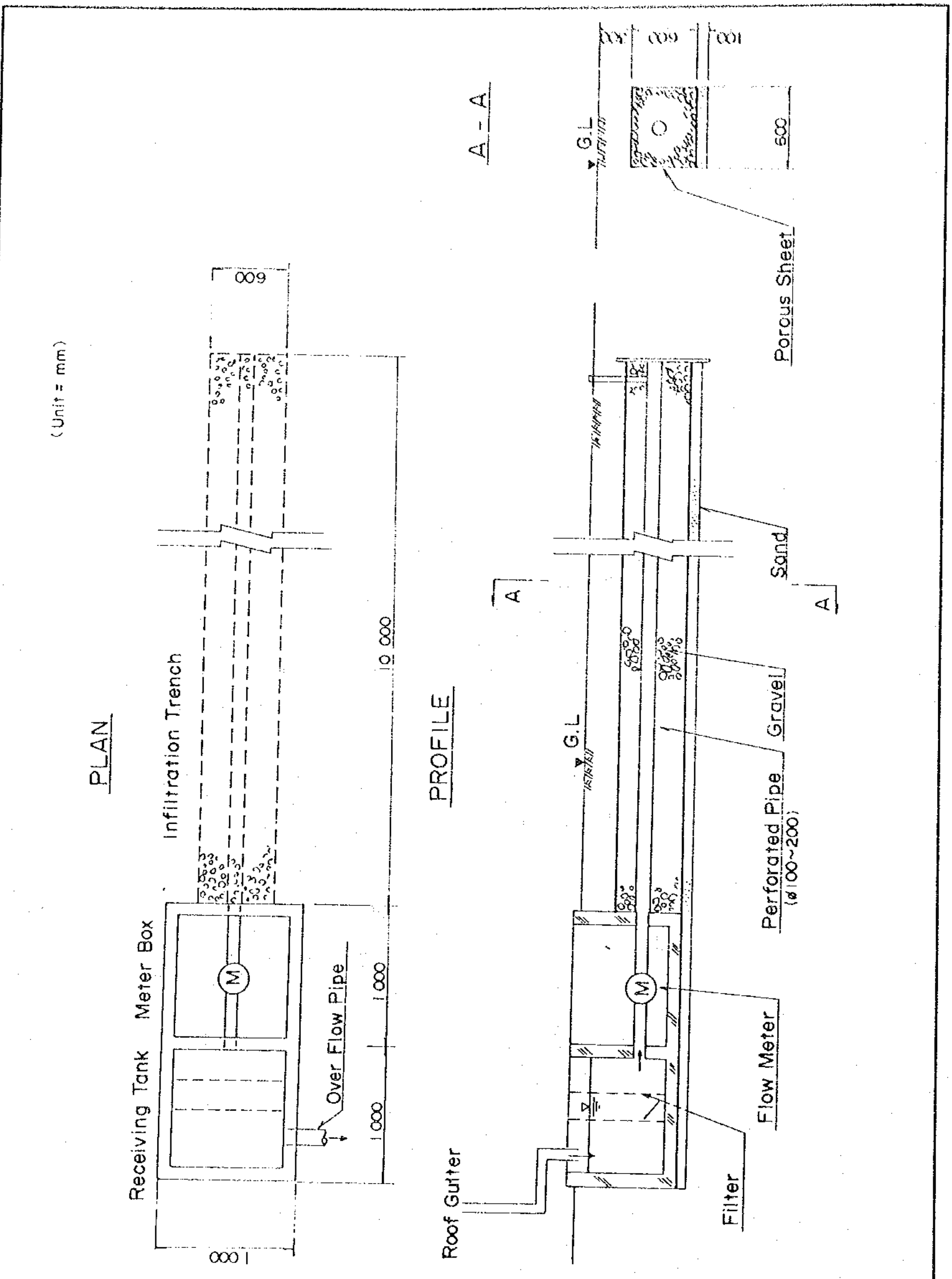

Fig. A3-2 DRAFT DRAWINGS OF INFILTRATION FACILITIES	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

2. GEOLOGY

SUPPORTING REPORT
ON
GEOLOGY

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SUPPORTING REPORT
ON
GEOLOGY

1. General

The geological survey which covered the whole study area was conducted to grasp the general geological condition on the basis of the previous geological studies and field investigations for the formulation of the Basic and the Master plans. Furthermore, in the sites of the proposed facilities for the First Stage Project such as drainage pipes, bridges, etc., detailed geological investigations, together with the excavation by hand auger, were carried out at 16 points along the course of the Mburicao River (tentatively designated M-1 to M-16) and at 16 points in Itay River (I-1 to I-16), as shown in Fig. 2-1.

2. Geology of the Study Area

The geology of the study area is composed of red sandstone (Misiones Sandstone), gravel bed, dolerite (intrusive or flow), and alluvium and backfill, as shown in Fig. 2-2.

The basement of the study area consists of the Misiones Sandstone of the Triassic to the Cretaceous. Its lithofacies consist mainly of soft massive aeolian sandstone and partially of river deposits. The sandstone is medium-red, and sometimes includes subangular gravel made of Silurian Sandstone and well-rounded gravel made of Pre-Cambrian Quartzite.

In the middle reaches of the Tacumbu and the Mburicao river basins, dolerites intrude into the sandstones, and alluvium sediments and backfills cover the basement. Outcrops are well exposed along the rivers except in the upper reaches of the basin. The exposures can also be seen on cuttings of the roads.

Sandstone

The lithofacies are made of red or reddish brown medium aeolian sandstone, and the matrix is clayey. Sometimes, subangular gravel made of sandstone or well-rounded gravel made of quartzite is contained lenticularly. The sandstone is generally massive, therefore, bedding place is not found. However, judging from the form of gravel bed, the orientation seems almost horizontal.

Fresh sandstone is red or reddish brown, but when it is weathered white dots are generated and these dots expand with the progress of weathering until the sandstone becomes entirely white. Besides white sandstone, gray, yellowish gray, and yellowish brown sandstones are seen. This breaching is generally found, but in some cases, the sandstone remains red even if it is weathered.

The change in color tone showing the progress of weathering relates well to the lowering in hardness. Fresh sandstone has sufficient sand, but when weathered, it has more clay and less sand. Weathered sandstone becomes hard if it is dry, and shrinks and cracks may occur.

In the sandstone area, the ground water level is shallow because the rock component of the weathered sandstone is rich in clay and permeability is not good. When alluvium is deposited on the weathered sandstone, little ground water is seen in the weathered sandstone. The water stays in the lower part of the alluvium in many cases. No fault was found in the survey, but thin limonite veins were seen.

Gravel Bed

The gravel bed in the sandstone is distributed in the upper stream area (MG-9) from the crossing of the Mburicao River and Chavez Avenue, and in the downstream area from the bridge of the crossing (MX-2) of Artigas Avenue. The gravel bed in the former area is at most 0.5 m thick, and is almost a horizontal lenticular bed extending about 4 m. In the latter case, the gravel is distributed in the riverbed between the bridge and a place 70 m downstream, and is assumed to be lenticular. Subangular sandstone gravel and well-rounded quartzite gravel are found. The gravel ranges in size from 1 to 8 cm in diameter. The amount of gravel is less than that of the matrix, so that there may be not trouble in excavation or other construction works.

Dolerite

Dolerite is distributed for about 30 m along the left bank of the upper stream area (mX-15) from the crossing of the Mburicao River and Chavez Avenue. It seems that dolerite have intruded into the sandstone irregularly, because the boundary was observed like an inter-finger form with the sandstone at Tacumbu. The lithofacies are black, hard and compact. The outcrop is subject to weathering and shows an onion structure.

In this area, excavation by concrete breaker seems to be possible from the surface to a depth of 1 to 1.5 m. In the survey area, the distribution of the dolerite is extremely limited, therefore, it can be neglected in the construction plan of rainwater drainage.

Alluvial Deposit and Backfill

Alluvium and backfill are deposited on the basement. Sand is deposited in the riverbed of the lower reaches of each river. In the lower reaches, the alluvium reaches 4 to 7 m thick, and in the upper and middle reaches, it is almost 0.3 to 1.5 m thick.

The grain size of the sediment is rich in clay and poor in sand; therefore, when the sediment is dry, it becomes hard. It may not flow even if it contains water. However, washed sand of yellow-gray color was found in some places. The sand is unconsolidated and may flow once it contains water (M-2, M-3 points).

Backfill is seen in reclaimed areas of rivers and swamps, and the soil is sandy clay made by excavation of weathered sandstone. Sometimes, rubbish and portions of retaining walls and revetments of old rivers are found. In the factory area in the lower reaches of Mburicao River, iron plates/scraps or vinyl pieces are piled up in large quantities on the riversides.

3. Geology of Mburicao and Itay River Basins

From the geological field investigations and hand auger excavation in the Mburicao and the Itay river basins, alluvium, backfill, weathered sandstone and dolerite were classified into four (4) classes, i.e, Low (L), Middle (M), High (H) and Higher High (HH), according to the order of increasing hardness, as shown in Table 2-1. The proposed river channel is located along the river, and the proposed location of the drainage channel is over the flat land between the two rivers. The major structures are bridges across the rivers.

The results of the geological investigation are shown in Table 2-4, and the columnar section of hand auger excavations are in Fig. 2-3.

3.1 River Channel

In the upper and middle reaches of the Mburicao and the Itay rivers, weathered sandstones are exposed in many places on the riverbed or the riverside. In the lower reaches, sand and mud are deposited on the riverbed, and weathered sandstone is less exposed. The average thickness of the alluvium and backfill is 1.0 m and 0.7 m in the Itay and the Mburicao river basins, respectively. The depth of the lower limit of weathered sandstone (M Class) is 3.5 m in the Itay river basin and 4.3 m in the Maburicao river basin on the average. The thickness of the alluvium and backfill, and the depth of the lower limit of weathered sandstone (M Class) are shown in Table 2-2.

The sandstone exposed along the river is subject to heavy weathering. It is, therefore, assumed that softening is expedited rapidly. Since the sandstone above the river water level is soft and that under the water level is hard, the latter has been classified as solid sandstone (H Class).

In accordance with the engineering observation, excavation can be done with back hoe in the area of weathered sandstone (M Class).

As for the solid sandstone (H Class), excavation by buckhoe is also possible but the efficiency may be lower.

The gradient of the river cliff is vertical, so that vertical cutting may not bring collapse. However, corners of the alluvium surface are found to be collapsed, so that collapses may occur in the alluvium deposit and backfill when they contain water. Therefore, preventive measures such as sheeting and cutting the gradient to as gentle as 1:0.3 should be taken, especially when the washed sand contains water.

As for excavation soil, both the alluvium sediment and weathered sandstone has much clay and may flow when it contains much water, drastically reducing the working efficiency. Therefore, sufficient drainage should be provided, and work in the rainy season should be avoided.

Fresh rock is desirable, instead of weathered sandstone, for the structure foundations. The weathered sandstone exposed on the present riverbed is subject to weathering and may become soft and weak, so that it is desirable to excavate to a depth of 0.5 to 0.8 m from the riverbed surface where fresh rock can be found.

As regards erosion, softening of the weathered sandstone is rapid. The root portion of the retaining wall is eroded by the flowing water and the retaining wall itself has collapsed in some portions. Therefore, measures to prevent erosion must be taken into account.

3.2 Drainage Channel

The average thickness of the alluvium and backfill in the riverbed is 0.6 m both for the Itay and the Mburicao river basins, which is thinner than that at the riverside. At the hand auger drilled points of M-2 and M-3 shown in Fig. 2-2, the depth of 2.0 to 3.0 m from the ground surface consists of washed sand that contains water and is unconsolidated. The average depth of the weathered sandstone (M Class) lower limit is 3.5 m in the Itay river basin, and 3.8 m in the Mburicao river basin. The thickness of the alluvium and backfill, and the depth of the weathered sandstone (M Class) lower limit are shown in Table 2-3.

Since the thickness of the alluvium and backfill is thin in the riverbed portion, the weathered sandstone exists just below the ground surface. Observations on the excavation and the characteristics of excavated soil are as described in Subsection 3.1.

The foundation consisting of weathered sandstone is seen at the depth of 3.0 to 4.5 m under the ground surface. The drainage channels may be installed directly on the base, judging from its scale and continuous structure.

In the portion where the water containing clayey stratum and washed sand are distributed thickly, flowing of the stratum may occur and dewatering may be needed for the deep excavation to prevent the inclination and sinking of adjacent structures when the drainage channel is constructed along the roads in the urban area. In this connection, a detailed investigation may be required prior to work execution.

3.3 Major Structure Site

Bridges are anticipated as major structures in this project. Three (3) bridges are proposed on Artigas, Mariscal Lopez and Espana avenues in the Mburicao river basin; while, in the Itay river basin, one (1) bridge is planned on Aviadores del Chaco Avenue.

Since bridges are subject to heavy load and strong vibration, their foundations need to be constructed on solid sandstone. In places where solid sandstone (H Class) is distributed deeply, pile foundations may be selected. In this case, a detailed investigation should be conducted to determine the length of piles to be used.

TABLES

Hardness Class	Consolidation	Depth (m)	Rock/Soil	Color	Weathering	Depth of Concavity when Tapped by Hammer (cm)	Relative Difficulty of Excavation by Hand Auger	Notes in Designing and Work	Remarks
L	Unconsolidated	-0.3 to -1.5	Alluvium: Sandy Clay, Sand Backfill: Sandy Clay, Concrete Lumps	Gray-brown; Gray; Gray-red	-	Ave. 5	Easy	° When excavated contains water, fluidity is increased. ° When washed sand contains water, it collapses.	
M	Unconsolidated to medial solid	-0.3 to -1.5 under	Weathered Sandstone: Sandy Clay, Clayey Sand	Gray/Yellow-Gray	Strong	5 to 7	Easy	° Consider drainage as fluidity is increased when excavation soil contains water. ° Cut soil face will remain vertical. ° Excavation is possible by manpower and power shovel.	When dry, hardness increases, but shrinks and cracks may occur.
	Medial Solid to Solid	-1.0 to -4.0		White/Yellow-gray spots in red/red-brown color	Medium	3 to 5	Medial hard		
H	Solid	-3.0 under	Sandstone	Red/Red-brown (white dots may appear	Weak	3 or less	Impossible	° The deep portion can be a bearing bed for structures. ° Working efficiency becomes low but excavation by power shovel is possible. ° It can be a bearing bed for structures.	
HH	Solid	0 below	Weathered Dolerite	Black-dark green-gray color	Medium	1 to 2	Impossible (estimate)	° Excavation is possible by concrete breaker. ° If the surface bed is removed, this dolerite can be a bearing bed for structures.	

Table 2-2. GEOLOGICAL FEATURE OF RIVER CHANNELS

(Unit: m)

ITAY RIVER BASIN			MBURICAO RIVER BASIN		
Location No.	Thickness of Alluvium and Backfill	Lower Limit of Weathered Sandstone	Location No.	Thickness of Alluvium and Backfill	Lower Limit of Weathered Sandstone
1-1	0.6	3.0	1	0.8	5.0
1-2	-	-	2	1.3	5.0
1-3	1.0	3.0	3	1.4	3.8
2-1	0.6	3.0	4	2.7	5.0
2-2	0.7	3.0	5	1.3	4.0
2-3	2.4	4.0	6-1	1.7	3.0
3-1	0.3	3.0	6-2	-	-
3-2	0.5	3.0	6-3	1.4	4.0
3-3	0.4	3.5			
4-1	0.5	4.0			
4-2	0.5	4.0			
4-3	0.5	4.3			
4-4	1.0	3.0			
4-5	2.0	4.5			
5	1.6	2.5			
6-1	1.6	4.5			

Note: Refer to Fig. 2-1.

Table 2-3. GEOLOGICAL FEATURE OF DRAINAGE CHANNELS

(Unit: m)

ITAY RIVER BASIN			MBURICAO RIVER BASIN		
Location No.	Thickness of Alluvium and Backfill	Lower Limit of Weathered Sandstone	Location No.	Thickness of Alluvium and Backfill	Lower Limit of Weathered Sandstone
1-1	0.3	4.0	1	0.3	4.0
1-2	0.4	3.0	2	0.6	4.0
1-3	0.2	4.5	3	0.5	4.5
2-1	0.3	3.0	4	0.2	3.0
2-2	0.5	3.0	5	0.4	5.0
2-3	1.5	3.0	6-1	0.5	2.0
3-1	0.3	3.0	6-2	1.6	4.0
3-2	0.4	2.5	6-3	-	-
3-3	0.3	4.0			
4-1	0.4	4.0			
4-2	0.4	4.0			
4-3	0.3	4.0			
4-4	1.0	3.0			
4-5	0.3	4.0			
5	1.6	2.5			
6-1	1.3	3.5			

Note: Refer to Fig. 2-1.

Table 2-4(1/5). GEOLOGICAL SURVEY DATA

Inves-tiga-tion Point	Refer-ence Point	Depth (m)	Classification of Rock and Soil L (m)	M (m)	H (m)	Water Level (m)	Photo No.	Remarks
MBURICAO RIVER BASIN								
M-1	road	4.5	0 to -4.5	-	-	-	-	
M-2	road	4.2	0 to -4.5	-	-	-1.5	-	
M-3	road	4.5	0 to -4.5	-	-	-2.0	-	
M-4	road	4.5	-	0 to -4.5	-	-4.5	-	
M-5	road	5.0	0 to -2.0	-2.0 to -5.0	-	-	-	
M-6	road	4.1	0 to -2.0	-2.0 to -4.0	-4.0 to -4.1	-2.8	-	
M-7	road	2.2	0 to -0.9	-0.9 to -1.9	-1.9 under	-	-	Hard: give up
M-8	gate	4.5	-2.0 to -4.5	0 to -2.0	-	-2.0		
M-9	road	4.65	0 to -4.65	-	-	-2.0		
M-10	road	2.4	-	0 to -2.1	-2.1 under	-1.0	-	Limonite vein
M-11	road	3.3	0 to -1.7	-1.7 to -3.2	-3.2 under	-2.0	-	Hard: give up
M-12	road	4.5	0 to -1.7 -2.8 to -4.5	-1.7 to -2.8	-	-1.0	-	
M-13	road	4.5	0 to -3.7	-3.7 to -4.5	-	-1.0	-	
M-14	bridge	4.1	0 to -2.5	-2.5 to -4.0	-4.0 to -4.1	-2.0	-	Hard: give up
M-15	road	2.6	0 to -1.8	-1.8 to -2.4	-2.4 under	-	-	Hard: give up
M-16	bridge	3.8	0 to -3.7	-	-3.7 under	-	-	Hard: give up
Total		63.35						
MG-1	road	1.0	0 to -1.0	-1.0 to -7.4	-	-7.4	-	
MG-2	road	0.8	0 to -0.8	-0.8 to -5.9	-	-6.5	10	Strike: NS, Dip 8°W
MG-3	hilltop	4.75	0 to -4.75	-4.75 to -7.75	-	-7.75	-	
MG-4	center pipe	1.25	0 to -2.0	-2.0 under	-	-2.0	11	
MG-5	bridge	2.0	0 to -2.6	-2.6 to -3.2	-	-	-	
MG-6	surface	1.6	0 to -1.6	-1.6 to -6.0	-	-6.0	12	
MG-7	bridge		0 to -3.45	-3.45 to -4.55	-	-4.35	-	

Table 2-4(2/5). GEOLOGICAL SURVEY DATA

Investigation Point	Reference Point	Depth (m)	Classification of Rock and Soil: L (m)	M (m)	H (m)	Water Level (m)	Photo No.	Remarks
MG-8	surface	4.0	0 to -4.0	-4.0 to -6.3	-	-6.3	-	
MG-9	surface	2.55	0 to -2.0	-2.0 to -4.1	-	-3.9	-	-3.6 gravel bed thickness = 0.3m
MG-10	bridge	3.0	0 to -3.8	-3.8 to -4.95	-	-4.95	-	Brdige, Generalisino Franco Avenue
MG-11	surface	3.7	0 to -3.7	-	-3.7 to -6.2	-6.2	-	
MG-12	bridge	0.36	0 to -2.8	-2.8 to -5.2	-5.2 to -7.2	-6.4	13	
MG-13	bridge	2.1	0 to -2.1	-2.1 to -5.6	-	-5.6	-	
MG-14	bridge		0 to -4.2	-4.2 to -5.0	-	-4.5	-	
MG-15	road	0.95	0 to -1.75	-1.75 to -4.45	-	-4.25	-	
MG-16	bridge	3.0	0 to -0.6	-0.6 to -2.2	-	-	-	
MG-17	road	1.65	0 to -1.65	-1.65 to -2.1	-	-1.9	-	
MG-18	surface	1.05	0 to -0.85	-0.85 to -2.7	-	-2.6	-	
MG-19	bridge	1.4	0 to -1.9	-1.9 to -2.85	-	-2.85	-	
MX-1	surface	1.0	0 to -1.0	-1.0 to -1.2	-	-1.2	-	
MX-2	surface	3.5	0 to -3.5	-3.5 to -4.0	-	-	-	
MX-3	surface	0.7	0 to -7.0	-	-	-	-	
MX-4	bridge		0 to -4.7	-4.7 under	-	-	-	With gravel bed Thickness = 0.3m
MX-5	surface	0.6	0 to -3.5	-3.5 to -5.8	-	-5.8	-	
MX-6	surface	2.0	0 to -2.0	-	-	-2.0	-	
MX-7	surface	1.5	0 to -1.5	-1.5 to -5.5	-	-5.5	-	
MX-8	surface	1.5	0 to -1.5	-1.5 to -6.0	-	-6.0	14	
MX-9	bridge	1.2	0 to -1.2	-1.2 to -3.4	-3.4 to -3.8	-3.6	-	
MX-10	surface		0 to -3.9	-3.9 to -4.1	-	-4.1	-	
MX-11	surface	0.8	0 to -0.8	-0.8 to -3.2	-3.2 to -4.4	-4.3	-	
MX-12	surface	1.2	0 to -3.2	-3.2 to -4.4	-4.4 to -4.7	-4.5	-	

Table 2-4(3/5). GEOLOGICAL SURVEY DATA

Investigation Point	Reference Point	Depth (m)	Classification of Rock and Soil L (m)	M (m)	H (m)	Water Level (m)	Photo No.	Remarks
MX-13	surface	1.2	0 to -1.2	-1.2 to -3.7	-	-3.7	-	
MX-14	surface	2.2	0 to -2.2	-2.2 to -4.4	-	-4.2	-	
MX-15	bridge	2.3	0 to -2.3	-	-2.3 to -4.1	-4.1	-	Weathered dolerite bed; Right Bank, Length = 30m
MX-16	bridge	1.65	0 to -1.65	-1.65 to -5.5	-5.5 under	-3.6	-	Bridge; Generalisino Franco Avenue, -6.5m
MX-17	surface	0.5	0 to -0.5	-0.5 to -4.2	-	-	-	
MX-18	surface		-	0 to -4.4	-	-	-	
MX-19	bridge		-	0 to -4.8	-	-4.8	-	
MX-20	wall top		-	0 to -7.4	-	-7.4	-	Banking upstream
MX-21	surface	0.6	0 to -0.6	-0.6 to -2.0	-2.0 to -2.2	-2.0	-	
MX-22	bridge	0.6	0 to -0.6	-0.6 to -1.6	-1.6 to -1.8	-1.6	-	
MX-23	surface	1.2	0 to -1.2	-1.2 to -2.95	-	-2.75	-	
MX-24	surface	2.0	0 to -2.0	-2.0 to -2.8	-	-2.7	-	
MX-25	surface		0 to -2.7	-2.7 to -3.9	-	-3.7	-	
MX-26	bridge		0 to -2.7	-2.7 to -3.8	-	-3.7	-	
ITAY RIVER BASIN								
I-1	road	4.50	0 to -4.5	-	-	-1.75	-	
I-2	road	3.28	0 to -1.5	-1.5 to -3.2	-3.2 under	-1.5	-	Hard: give up
I-3	road	2.55	0 to -1.5	-1.5 to -2.4	-2.4 under	-	-	Hard: give up
I-4	bridge	3.80	0 to -3.6	-3.6 to -3.8	-	-	-	Limonite vein
I-5	road	3.10	0 to -2.0	-2.0 to -3.0	-3.0 under	-1.0	-	Hard: give up
I-6	bridge	3.20	0 to -2.5	-2.5 to -3.2	-	-0.3	-	Bottom Level: -4.56m
I-7	road	4.45	0 to -2.5	-2.5 to -4.35	-4.35 to -4.45	0.85	-	
I-8	road	4.50	0 to -1.5	-1.5 to -4.5	-	-1.5	-	

Table 2-4(4/5). GEOLOGICAL SURVEY DATA

Investigation Point	Reference Point	Depth (m)	Classification of Rock and Soil L (m)	M (m)	H (m)	Water Level (m)	Photo No.	Remarks
I-9	road	4.50	0 to -4.0	-4.0 to -4.5	-	-	-	
I-10	road	4.50	0 to -3.5	-3.5 to -4.5	-	-1.5	-	
I-11	road	4.50	0 to -4.5	-	-	-1.4	-	
I-12	road	4.30	0 to -0.7	-0.7 to -4.2	-4.2 to -4.3	-2.0	-	
I-13	road	4.30	0 to -3.0	-3.0 to -3.5	-3.5 to -4.3	-1.6	-	
I-14	road	4.50	0 to -4.5	-	-	-	-	
I-15	road	2.50	0 to -1.0	-1.0 to -2.45	-2.45 under	-	-	Hard: give up
I-16	stake	4.50	0 to -0.3	-0.3 to -4.0	-4.0 to -4.5	-	-	
Total		62.98						
IG-1	bridge	2.45	0 to -2.45	-2.45 to -3.75	-3.75 under	-3.75	1	
IG-2	bridge	0.1	0 to -0.5	-0.5 under	-	-	-	
IG-3	bridge		0 to -0.75	-0.75 under	-	-	-	
IG-4	surface		0 to -2.0	-2.0 under	-	-	-	
IG-5	road	0.9	0 to -0.9	-0.9 to -2.3	-	-1.8	2	
IG-6	road	1.0	0 to -1.0	-1.0 to -2.5	-2.5 to -2.8	-2.5	3	
IG-7	road	1.1	0 to -1.1	-1.1 to -2.8	-2.8 to -3.2	-3.0	-	
IG-8	bridge	1.7	0 to -1.7	-1.7 to -3.6	-3.6 to -3.8	-3.6	4	
IG-9	road		0 to -1.0	-1.0 to -1.3	-	-1.0	-	
IG-10	road	0.7	0 to -1.55	-1.55 under	-	-	-	
IG-11	road	0.55	0 to -1.7	-	-1.7 to -2.4	-2.3	-	
IG-12	road	0.6	0 to -2.0	-2.0 to -3.5	-3.5 to -4.0	-3.8	5	
IG-13	road		0 to -2.0	-	-3.8 under	-	6	
IG-14	road	0.6	-	-	-3.8 under	-	7	
IX-1	surface		0 to -2.0	-	-2.0 under	-	-	
IX-2	road		0 to -1.65	-1.65 under	-	-	-	
IX-3	road		0 to -1.8	-	-	-1.7	-	

Table 2-4(5/5). GEOLOGICAL SURVEY DATA

Investigation Point	Reference Point	Depth (m)	Classification of Rock and Soil L (m)	M (m)	H (m)	Water Level (m)	Photo No.	Remarks
IX-4	road		0 to -1.2	-1.2 to -2.8	-	-	-	
IX-5	road	0.6	-	-0.6 to -1.2	-	-	-	
IX-6	bridge		0 to -0.6	-0.6 to -1.5	-	-1.5	8	
IX-7	bridge	3.5	0 to -1.9	-	-	-1.8	9	
IX-8	surface		0 to -1.8	-1.8 under	-	-	-	
IX-9	road	1.1	0 to -1.1	-1.1 to -1.45	-	-1.40	-	

FIGURES

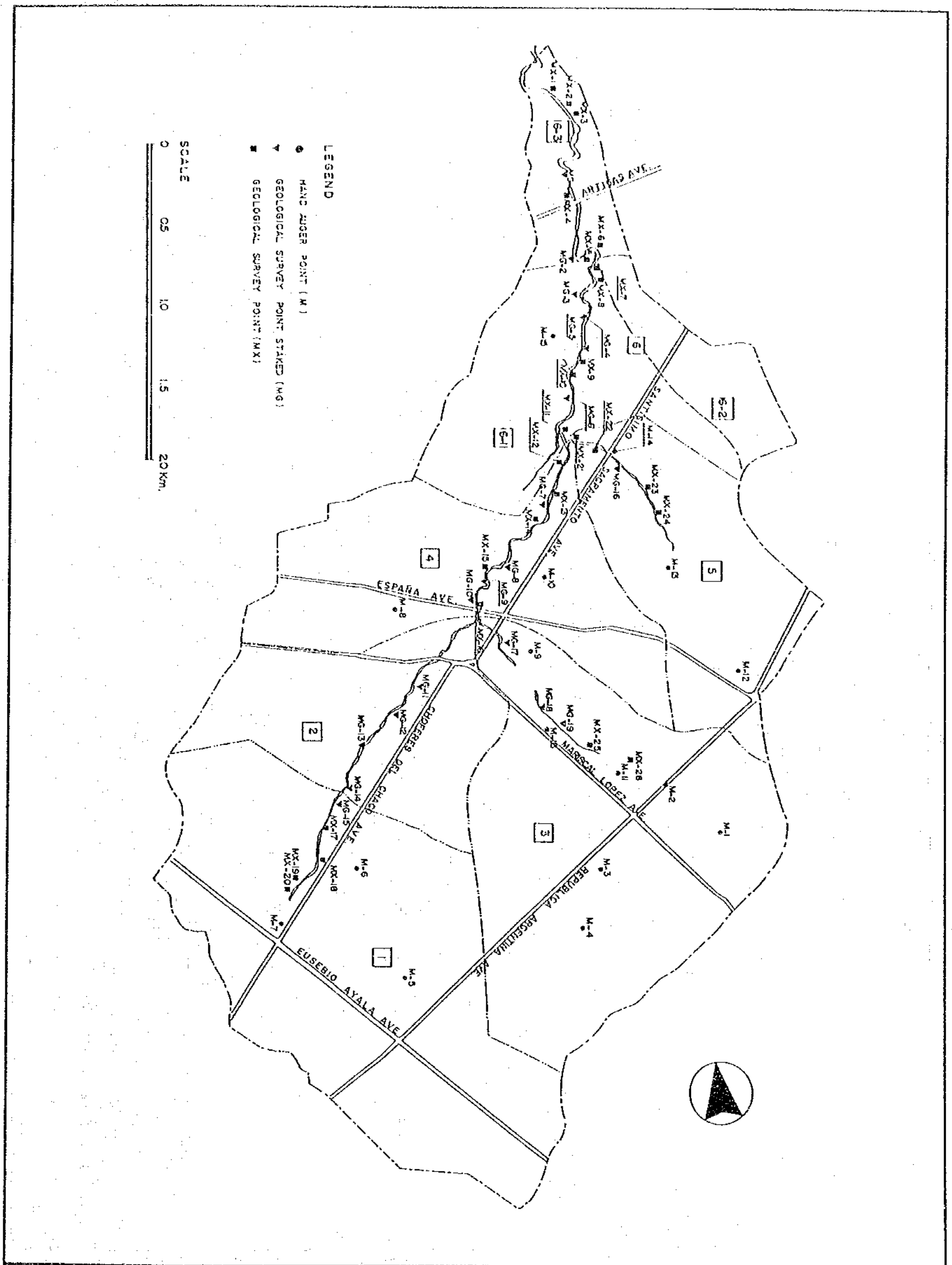


Fig. 2-1(1/2) GEOLOGICAL SURVEY POINT (MBURICAO RIVER BASIN)	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

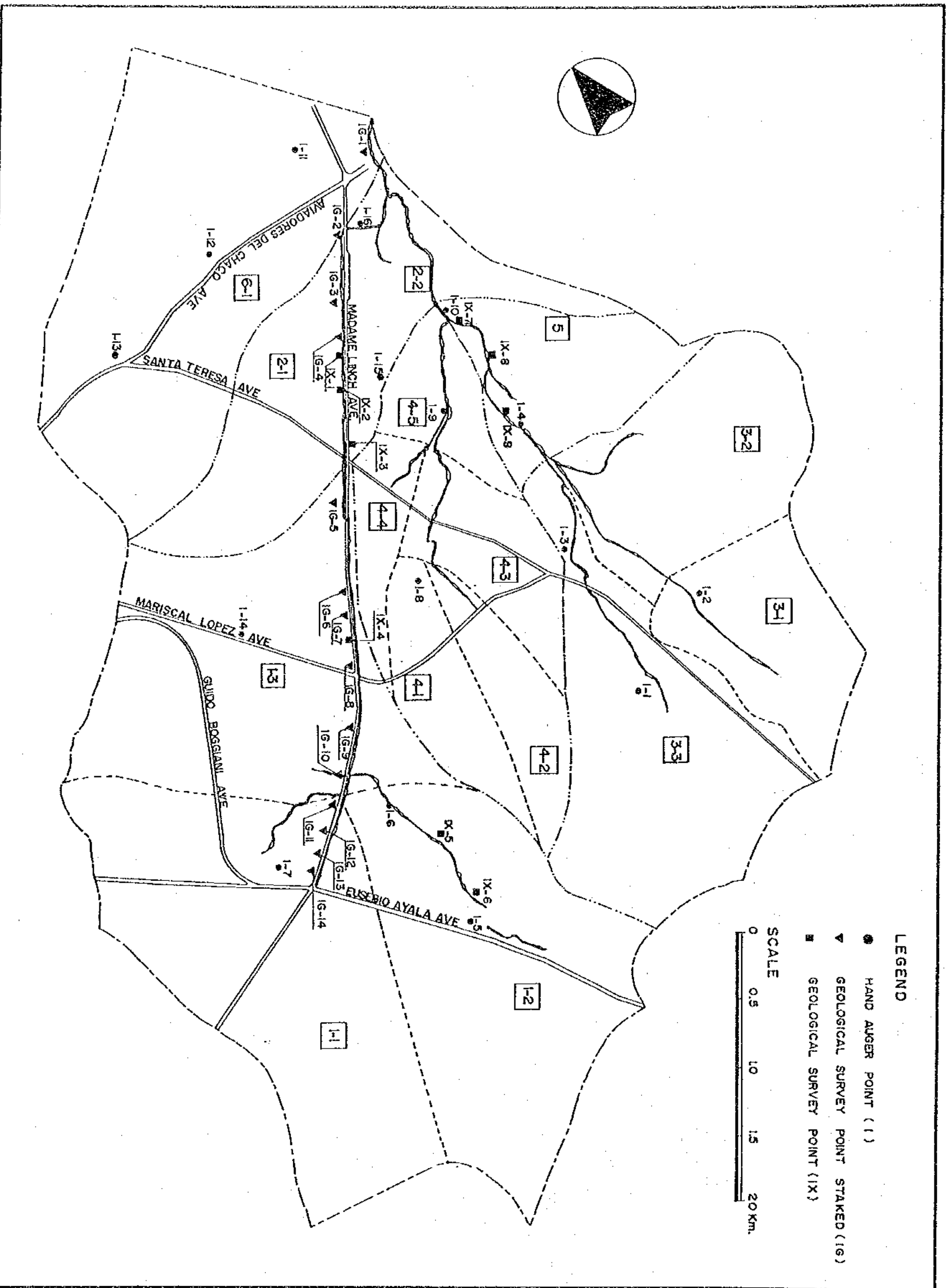


Fig. 2-1(2/2) GEOLOGICAL SURVEY POINT (ITAY RIVER BASIN)	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

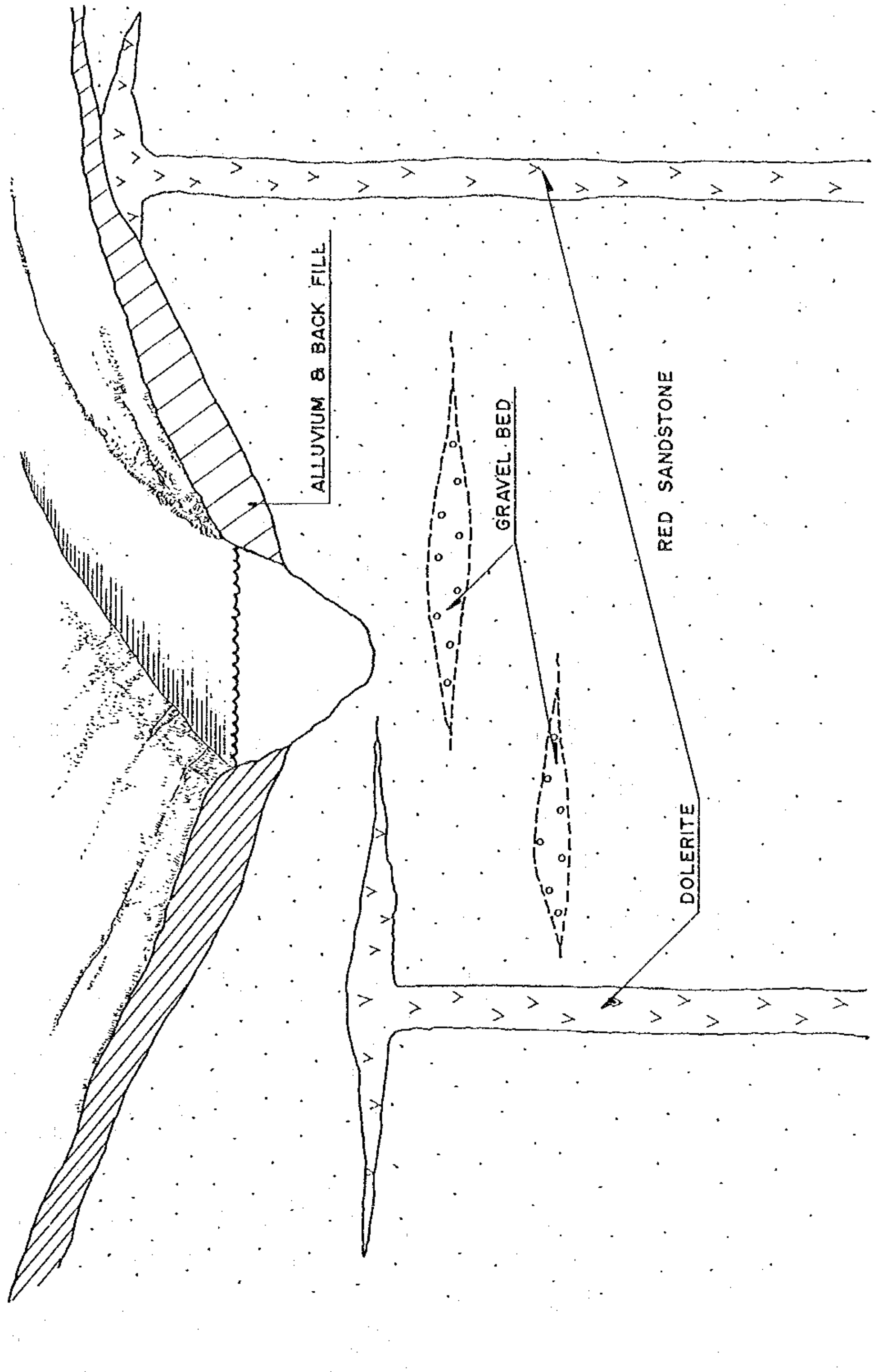


Fig. 2-2 SCHEMATIC GEOLOGICAL SECTION OF STUDY AREA	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY
	JAPAN INTERNATIONAL COOPERATION AGENCY

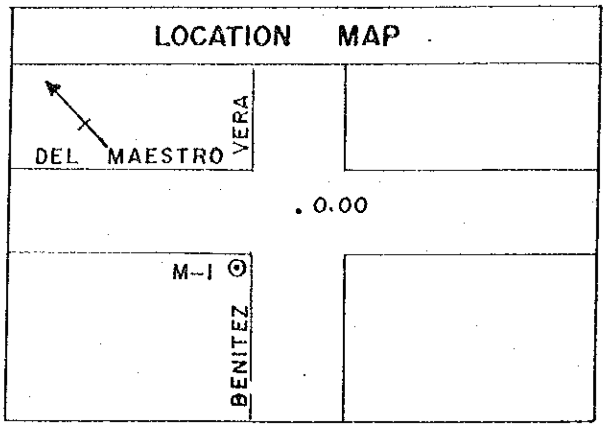
COLUMNAR SECTION (HOLE: M-I)

LOCATION:

GROUND HIGHT: 120.1 M.

DATE: 19-VI-86

DEPTH: 4.50 M.



0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.55	119.55		STREAM SEDIMENT WITH TRAVEL(L)
2.50	117.6		BROWN CLAYEY SAND (L)
4.50	115.6		GRAY CLAYEY SAND (L)
			LAKE DEPOSITE

1:50

Fig. 2-3(1/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

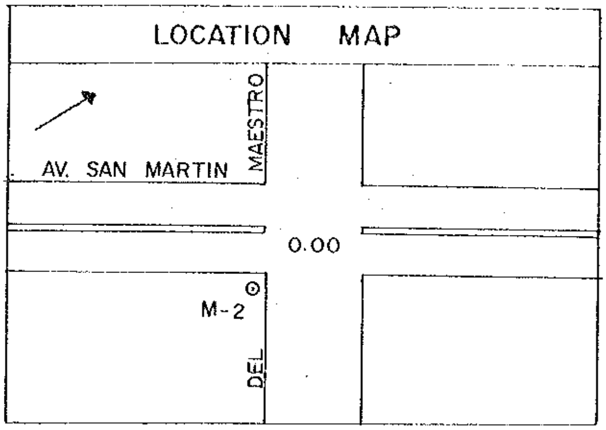
COLUMNAR SECTION (HOLE: M-2)

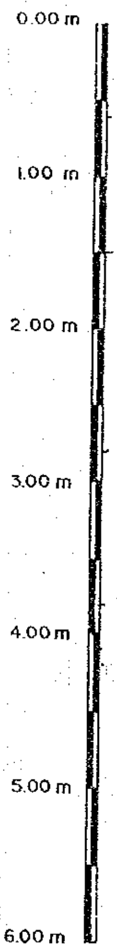
LOCATION:

GROUND HIGHT: 116.7 M.

DATE: 20-VI-86

DEPTH : 4.20 M.





DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.60	116.1		BACKFILL WITH GRAVEL (L)
1.50	115.2		YELLOWISH SAND (WASHED) (L)
2.80	113.9		
3.80	112.9		YELLOWISH GRAY SAND WASHED (L)
4.10	112.6		YELLOWISH GRAY CLAYEY SAND (L)
4.20	112.5		RED SAND (L)
			LAKE DEPOSITE BECAUSE OF THE MIX OF SAND AND WATER, IMPOSIBLE TO EXCAVATE.

1:50

Fig. 2-3(2/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M— 3)

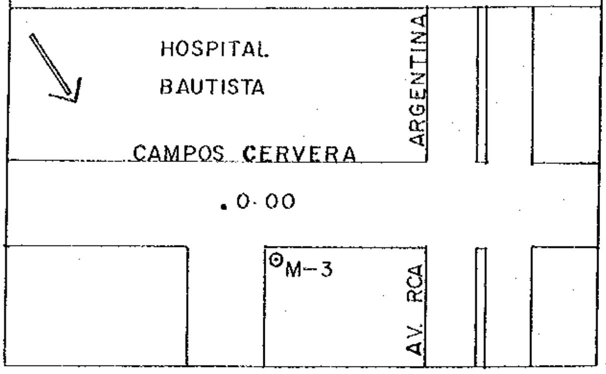
LOCATION:

GROUND HIGHT: 121.8 M.

DATE: 20—VI—86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	121.3		STREAM SEDIMENT (L)
2.00	119.8		BROWN WITH SLIGHTLY RED SAND (L)
4.30	117.5		RED SANDY CLAY (L)
4.50	117.3		RED CLAY WITH SLIGHTLY SAND (L)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(3/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M—4)

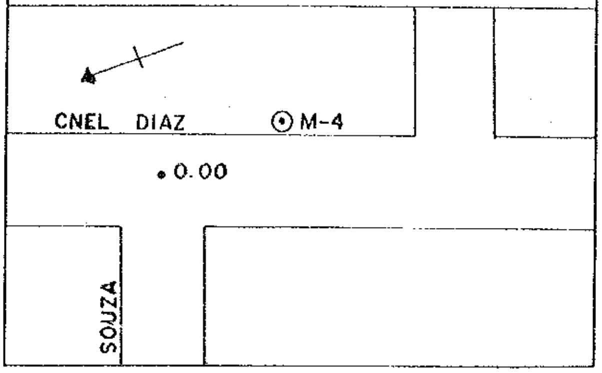
LOCATION:

GROUND HIGHT: 129.3 M.

DATE: 20-VI-86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	128.8		STREAM SEDIMENT (L)
2.30	127.0		RED CLAYEY SAND (M)
3.80	125.5		RED WITH GRAY AND YELLOW CLAYEY SAND (M)
4.50	124.8		RED CLAYEY SAND (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(4/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-5)

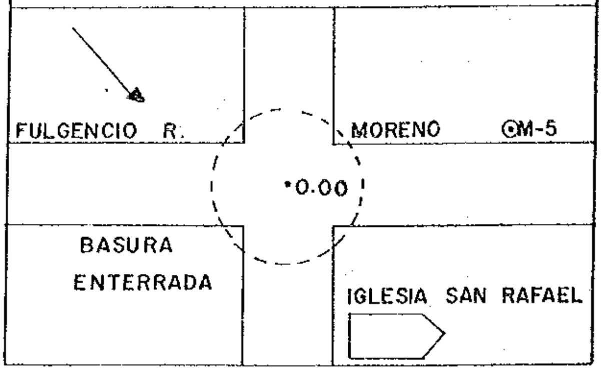
LOCATION:

GROUND HIGHT: 133.7 M.

DATE: 21-VI-86

DEPTH : 5.00 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
2.00	131.7		GRAY SANDY CLAY (L)
3.00	130.7		RED WITH GRAY SANDY CLAY (M)
3.50	130.2		RED SANDY CLAY (M)
5.00	128.7		RED WITH SLIGHTLY GRAY SANDY CLAY (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

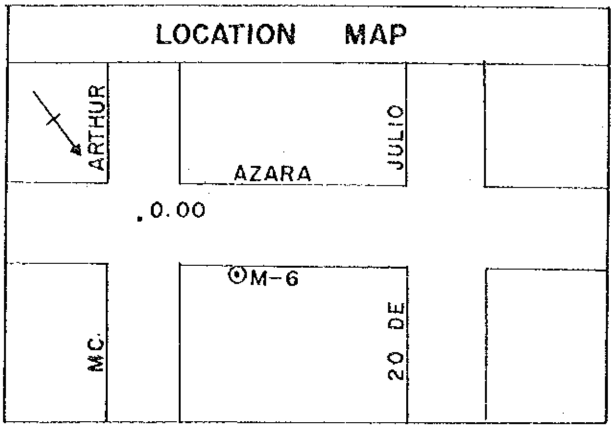
1:50

Fig. 2-3(5/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-6)



LOCATION:

GROUND HIGHT: 120.5 M.

DATE: 21-VI-86

DEPTH : 4. 10 M.

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.60	119.9		STREAM SEDIMENT (L)
2.00	118.5		GRAY CLAYEY SAND (L)
2.80	117.7		RED CLAYEY SAND (M)
4.00	116.5		RED CLAYEY SAND (M)
4.10	116.4		DITTO (H)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

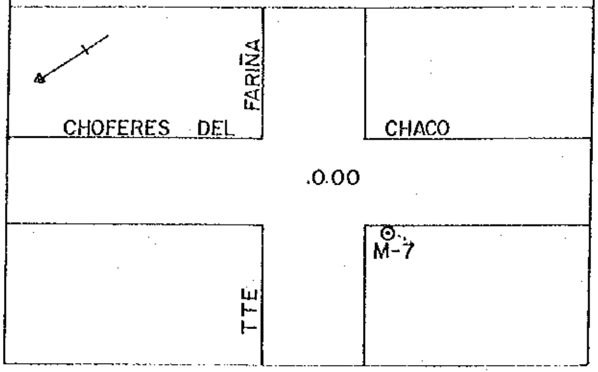
Fig. 2-3(6/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-7)

LOCATION MAP



LOCATION:

GROUND HIGHT: 131.5 M.

DATE: 23-VI-86

DEPTH : 2.20 M.

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.25	131.25		STREAM SEDIMENT (L)
0.90	130.6		RED COARSE SAND (L)
1.90	129.6		BROWN SANDY CLAY (M)
2.20	129.3		RED CLAY IS VERY HARD THEN CAN (M)
			NOT PENETRATE THE BIT

1:50

Fig. 2-3(7/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

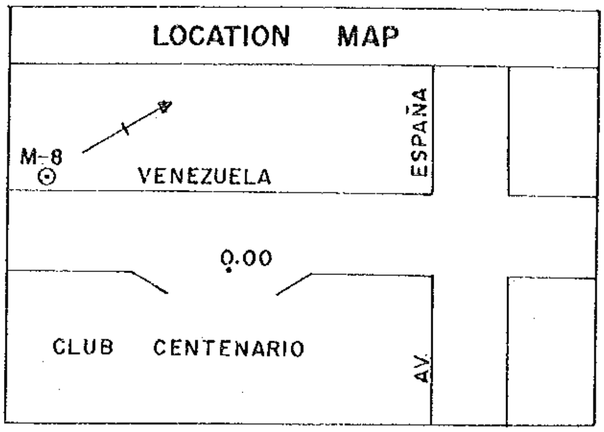
COLUMNAR SECTION (HOLE: M–8)

LOCATION:

GROUND HIGHT: 102.3 M.

DATE: 23–VI–86

DEPTH : 4.50 M.



0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.25	102.5		STREAM SEDIMENT (L)
			(M)
2.00	100.3		▽ RED CLAYEY SAND
			(L)
4.50	97.8		

1:50

Fig. 2-3(8/32) COLUMNAR SECTION	STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-9)

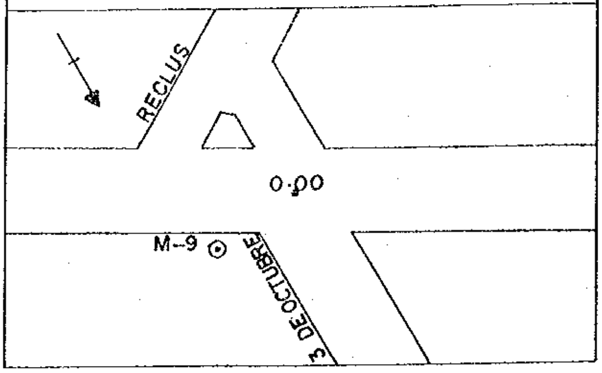
LOCATION:

GROUND HIGHT: 95.8 M.

DATE: 23-VI-86

DEPTH : 4.65 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	95.3		STREAM SEDIMENT (L)
1.50	94.3		LIGHTE GRAY CLAY (L)
2.00	93.8		GRAY WITH YELLOW AND RED CLAY (L)
2.50	93.3		
4.65	91.15		LIGHT BROWN SANDY CLAY(L)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

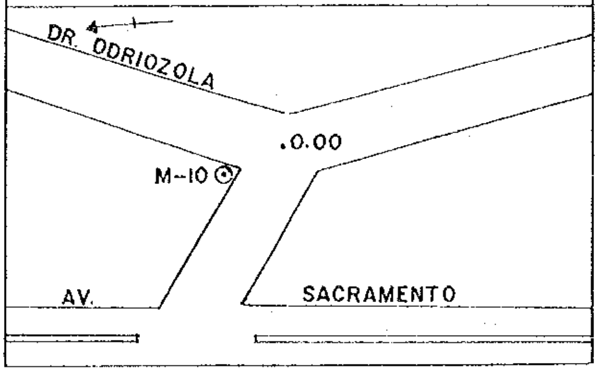
1:50

Fig. 2-3(9/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M—10)



LOCATION:

GROUND HIGHT: 92.2 M.

DATE: 23-VI-86

DEPTH : 2.40 M.

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.80	91.4		DARK GRAY SAND (M)
1.00	91.2		GRAY YELLOW AND RED SANDY CLAY (M)
2.10	90.1		GRAY AND BLACK CLAY WITH LIMONITE THIN VEIN IS VERY HARD THEN CAN NOT PENETRATE THE BIT (H)
2.40	89.8		

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(10/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

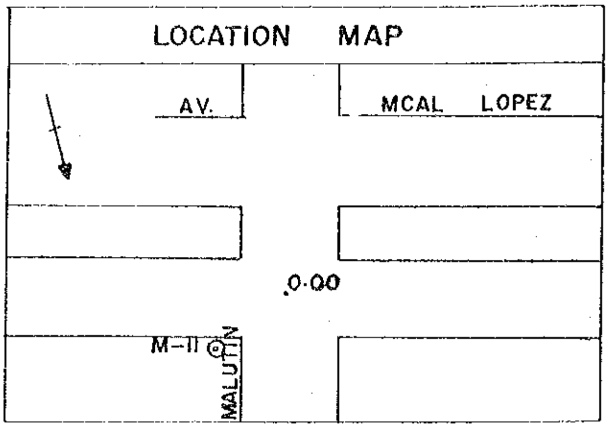
COLUMNAR SECTION (HOLE: M−II)

LOCATION:

GROUND HIGHT: 109.5 M.

DATE: 25−VI−86

DEPTH : 3.30 M.



0.00 m · 1.00 m · 2.00 m · 3.00 m · 4.00 m · 5.00 m · 6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	109		STREAM SEDIMENT (L)
1.70	107.8		DARKISH RED MEDIUM GRAIN SAND WITH BLACK LIMONITE (L)
2.00	107.5		▽ DARK GRAY WITH RED CLAY (M)
2.50	107.0		
3.20	106.3		REDDISH BROWN WITH GRAY SANDY CLAY (H)
3.30	106.2		CLAY INCLUDE WITH SAND AND GLOBULAR GRAVELS REDDISH BROWN WITH GRAY I'TS VERY HARD CAN NOT PENETRATE THE BIT

1:50

Fig. 2-3(11/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M—12)

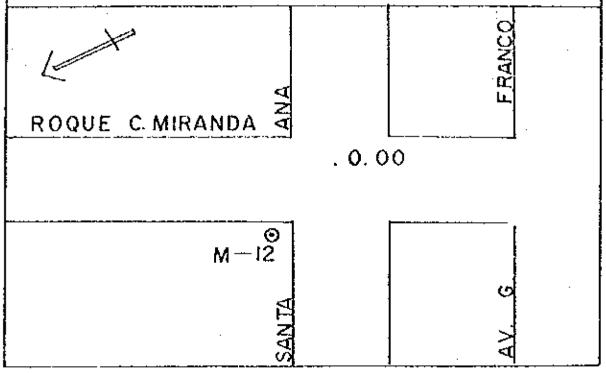
LOCATION:

GROUND HIGHT: 109.9 M.

DATE: 25—VI—86

DEPTH: 4.50 M.

LOCATION MAP



0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.40	109.5		STREAM SEDIMENT
1.00	108.9		▽ YELLOWISH MEDIAM GRAIN CLAYEY SAND (L)
1.70	108.2		
2.80	107.1		DARKISH RED SANDY CLAY (M)
4.00	105.9		RED WITH GRAY CLAYEY SAND (L)
4.50	105.4		RED WITH WHITE SAND (L)

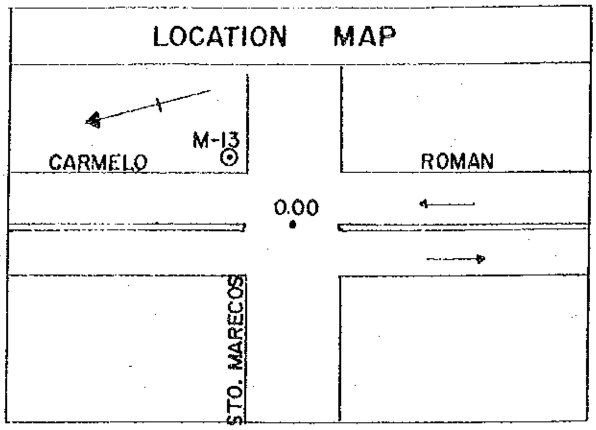
1:50

Fig. 2-3(12/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-13)

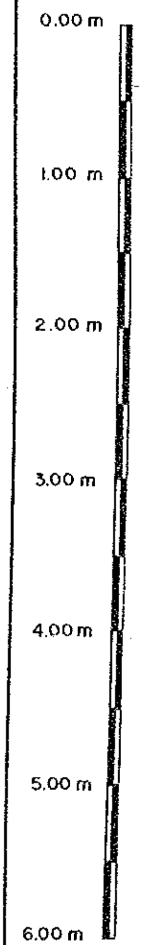


LOCATION:

GROUND HIGHT: 100.1 M.

DATE: 26-VI-86

DEPTH : 4.50 M.



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.40	99.7		STREAM SEDIMENT (L)
1.00	99.1		
2.60	97.5		GREY CLAYEY SAND (L)
3.70	96.4		GREY WITH YELLOW AND RED SANDY CLAY (L)
4.50	95.6		GREY AND RED CLAYEY SAND(M)

1:50

Fig. 2-3(13/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-14)

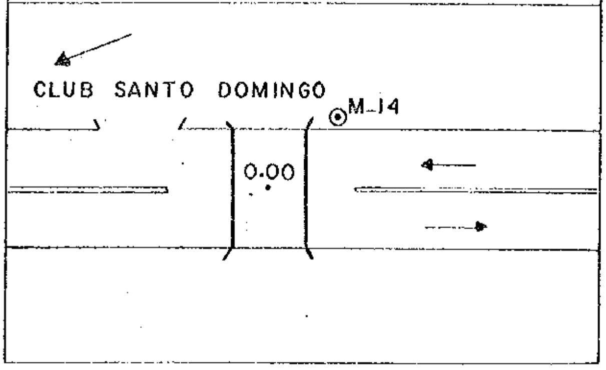
LOCATION:

GROUND HIGHT: 86.8 M.

DATE: 26-VI-86

DEPTH : 4.10 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.80	86.0		STREAM SEDIMENT (L)
1.30	85.5		GREENISH GREY MEDIUM GRAIM CLA- -YEY SAND (L)
2.00	84.8		
2.50	84.3		DARKISH YELLOW MEDIUM GRAIM CLAYEY SAND (L)
4.00	82.8		DARKISH RED MEDIUM GRAIM CLAYEY SAND (M)
4.10	82.7		DARKISH RED SANDY CLAY (H)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

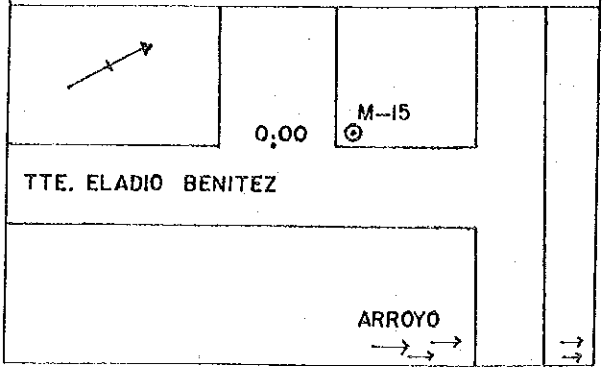
Fig. 2-3(14/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M–15)

LOCATION MAP



LOCATION:

GROUND HIGHT: 84.0 M.

DATE: 27–VI–86

DEPTH : 2.60 M.

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	83.5		BACKFILL WITH GRAVEL (L)
1.80	82.2		REDDISH BROWN CLAYEY SAND(L)
2.40	81.6		GREEN AND DARKISH YELLOW SANDY CLAY (M)
2.60	81.4		SANDY CLAY
			CAN NOT PENETRATE THE BIT BECAUSE IT'S VERY HARD (H)
			*LAKE DEPOSITE WITH GRAVELS

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

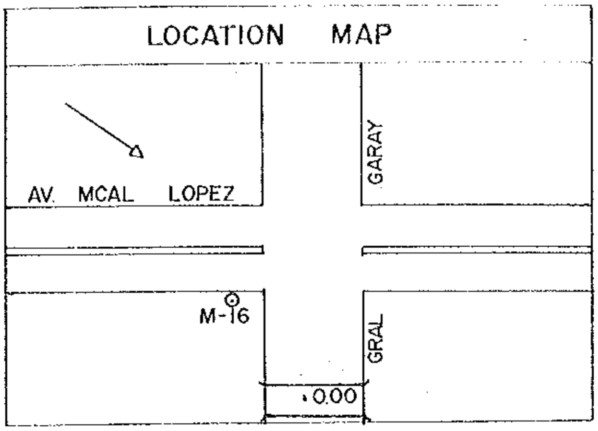
1:50

Fig. 2-3(15/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: M-16)



LOCATION:

GROUND HIGHT: 102.7 M.

DATE: 27-VI-86

DEPTH: 3.80 M.

DEPTH	ELEVATION	PROFILE	DESCRIPTION
1.70	101.0		REDDISH BROWN MEDIUM GREIN SAND (L)
3.70	99.0		DARK GREY COARSE SAND (L)
3.80	98.9		GRAY WITH WHITE AND BROWNISH RED (H)
			SANDY CLAY CAN NOT PENETRATE THE BIT

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(16/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

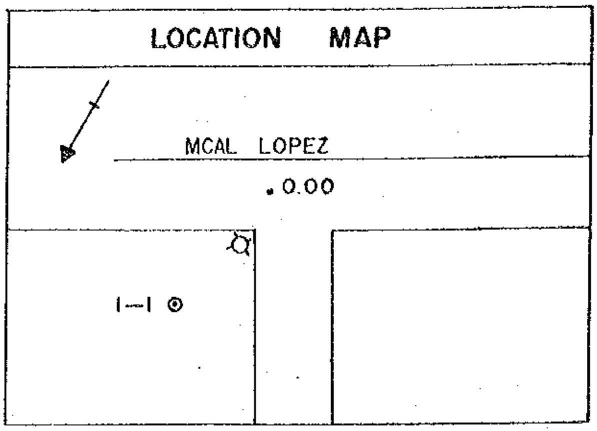
COLUMNAR SECTION (HOLE: I—I)

LOCATION:

GROUND HIGHT: +132.1 M.

DATE: .II-VI-86

DEPTH : 4.50 M



DEPTH	ELEVATION	PROFILE	DESCRIPTION
1.75	130.35		REDDISH SANDY CLAY (L)
2.00	130.1		
4.50	127.6		DARKKISH RED SANDY CLAY (L)

0.00 m 1.00 m 2.00 m 3.00 m 4.00 m 5.00 m 6.00 m

1:50

Fig. 2-3(17/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I−2)

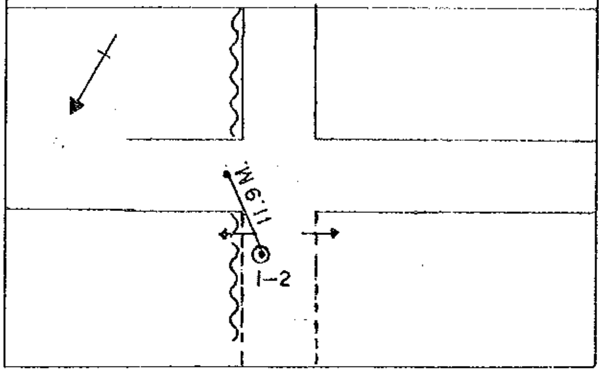
LOCATION:

GROUND HIGHT: 126.7 M.

DATE: 12−VI−86

DEPTH : 3. 28 M.

LOCATION MAP



0.00 m · 1.00 m · 2.00 m · 3.00 m · 4.00 m · 5.00 m · 6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
1.50	125.2		RED BUT WITH GRAY SANDY CLAY (L)
1.65	125.05		
3.28	123.42		RED AND WHITE CLAYEY SAND. HARD (M)

1:50

Fig. 2-3(18/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I−3)

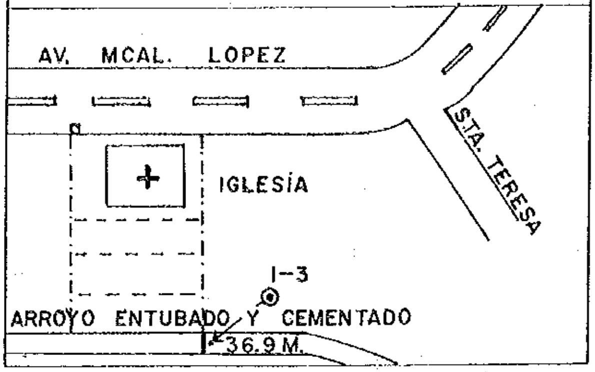
LOCATION:

GROUND HIGHT: 115.0 M.

DATE: 13−VI−86

DEPTH : 2.55 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
1.50	113.5		GRAY CLAYEY SAND (L)
2.10	112.9		RED AND GRAY CLAYEY SAND (M)
2.55	112.45		GRAY AND YELLOW SAND. HARD (M)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

Fig. 2-3(19/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-4)

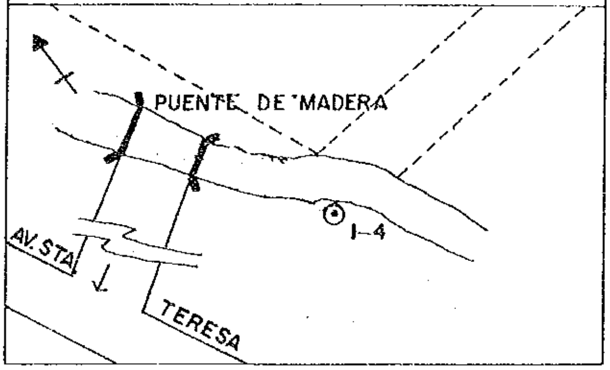
LOCATION:

GROUND HIGHT: 101.8 M.

DATE: 13-VI-84

DEPTH : 3.80 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.40	101.4		STREAM SEDIMENT (L)
1.50	100.3		GRAY CLAYEY SAND (L)
2.60	99.2		DITTO, GRAY WHITE (L)
3.60	98.2		DITTO, GRAY YELLOW (L)
3.80	98.0		BLACK LIMONITE, Ø=0.5~1 CM (M)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

Fig. 2-3(20/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-5)

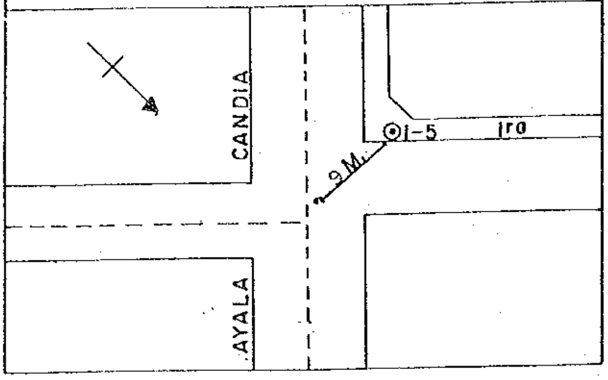
LOCATION:

GROUND HIGHT: 138.6 M.

DATE: 14-VI-86

DEPTH: 3.10 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.30	138.3		STREAM SEDIMENT (L)
1.00	137.6		GRAY SANDY CLAY (L)
2.00	136.6		
3.00	135.6		YELLOW BROWN CLAYEY SAND (M)
3.10	135.5		DITTO, HARD (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(21/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-6)

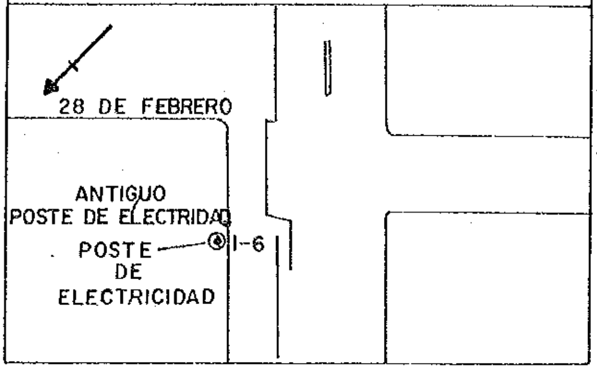
LOCATION:

GROUND HIGHT: 124.5 M.

DATE: 14-VI-86

DEPTH : 3.20 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.30	124.2		STREAM SEDIMENT (L)
2.50	122.0		GRAY BROWN YELLOW CLAYEY SAND (L)
3.20	121.3		RED AND GRAY CLAY (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(22/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

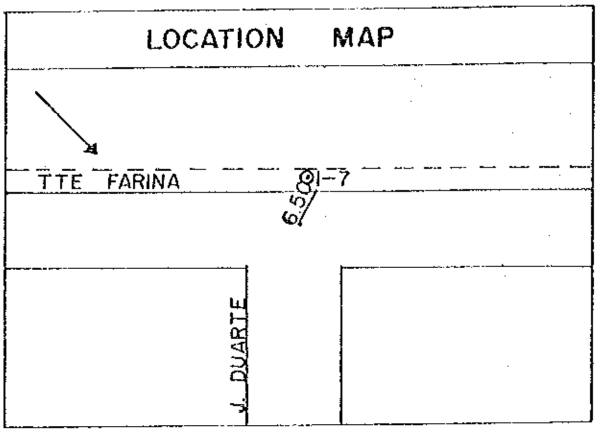
COLUMNAR SECTION (HOLE: I-7)

LOCATION:

GROUND HIGHT: 131.5 M.

DATE: 14—VI—86

DEPTH : 4.45 M.



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.25	131.25		STREAM SEDIMENT (L)
0.60	130.9		DITTO, WASHED YELLOWISH SAND(L)
0.85	130.65		
2.50	129.0		RED SAND (L)
3.50	128.0		RED AND GRAY SANDY CLAY (M)
4.35	127.15		RED CLAY (M)
4.45	127.05		RED CLAYEY SAND (H)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(23/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I–8)

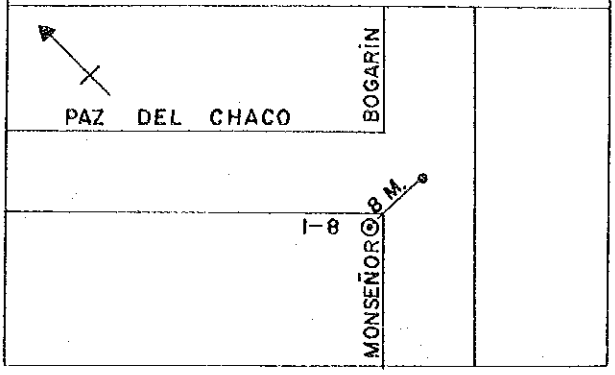
LOCATION:

GROUND HIGHT: 106.4 M.

DATE: 16–VI–86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	105.9		GRAYISH STREAM SEDIMENT (L)
1.50	104.9		GRAY SANDY CLAY (L)
4.00	102.4		LIGHT GRAY CLAY (M)
4.50	101.9		GRAY BUT WITH GREEN AND YELLOW (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(24/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-9)

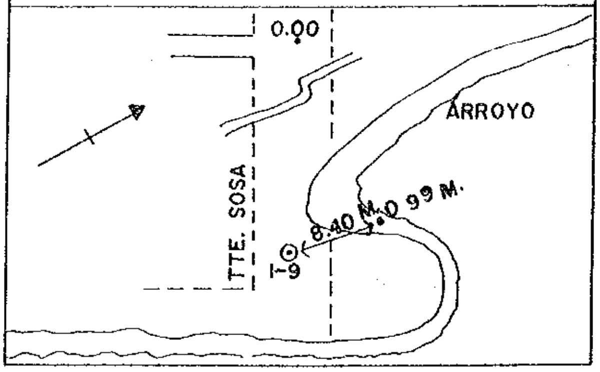
LOCATION:

GROUND HIGHT: 94.4 M.

DATE: 16-VI-86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	93.9		STREAM SEDIMENT (L)
3.20	91.2		STREAM SEDIMENT, YELLOWISH WHITE WASHED SAND (L)
4.00	90.4		WHITE BROWN CLAYEY SAND (L)
4.50	89.9		WHITE AND RED SANDY CLAY (M)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

Fig. 2-3(25/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-10)

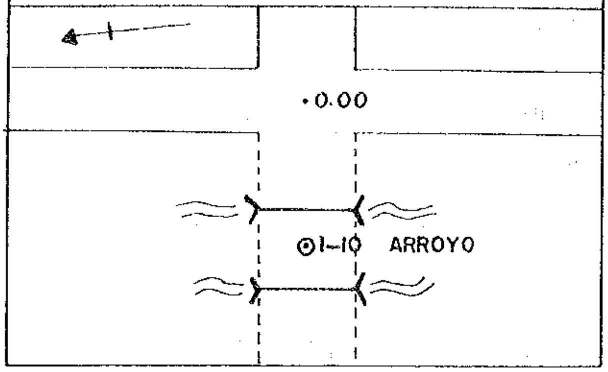
LOCATION:

GROUND HIGHT: 93.9 M.

DATE: 17-VI-86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.30	93.9		STREAM SEDIMENT (BROWN SAND)(L)
1.50	92.4		DITTO, GRAY CLAY (L)
3.50	90.4		GRAY BUT WITH GREEN AND YELLOW SANDY CLAY (L)
4.50	89.4		LIGHT BROWN SANDY CLAY (M)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(26/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-11)

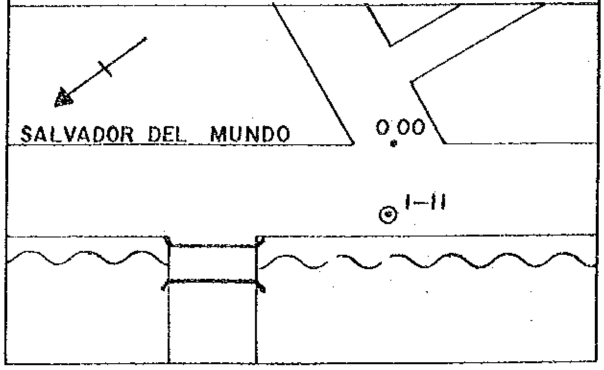
LOCATION:

GROUND HIGHT: 90.7 M.

DATE: 17-VI-86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.30	90.4		STREAM SEDIMENT (GRAY CLAY) (L)
1.40	89.3		
4.50	86.2		GRAY CLAYEY SAND (L)

0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

1:50

Fig. 2-3(27/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I—12)

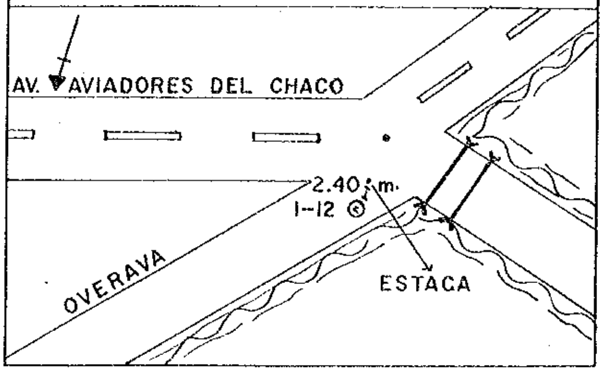
LOCATION:

GROUND HIGHT: 99.2 M.

DATE: 18-VI-86

DEPTH : 4.30 M

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.70	98.5		STREAM SEDIMENT (BLAK CLAY)(L)
2.00	97.2		YELLOWISH GRAY WITH BROWN CLAYEY SAND (M)
4.20	95.0		
4.30	94.9		RED GRAY SAND SLIGHTLY WITH CLAY (H)

0.00 m 1.00 m 2.00 m 3.00 m 4.00 m 5.00 m 6.00 m

1:50

Fig. 2-3(28/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-13)

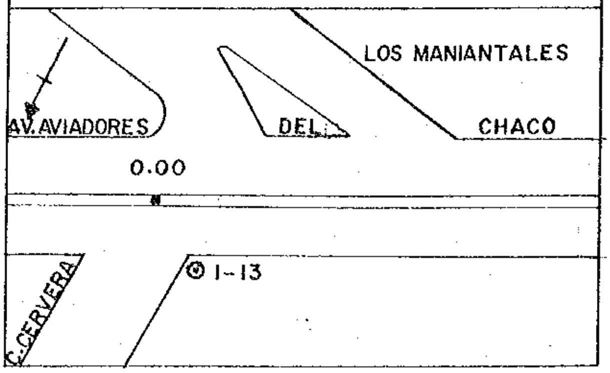
LOCATION:

GROUND HIGHT: 108.8 M.

DATE: 18-VI-86

DEPTH : 4.30 M.

LOCATION MAP



0.00 m
1.00 m
2.00 m
3.00 m
4.00 m
5.00 m
6.00 m

DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.50	108.3		BROWN BACKFILL (L)
1.60	107.2		STREAM SEDIMENT (GRAY BUT WITH BROWN CLAY)(L)
3.00	105.8		
3.50	105.3		GRAY BUT WITH RED CLAYEY SAND(M)
4.30	104.5		DITTO,YELLOWISH GRAY WITH RED(H)

1:50

Fig. 2-3(29/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT
IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

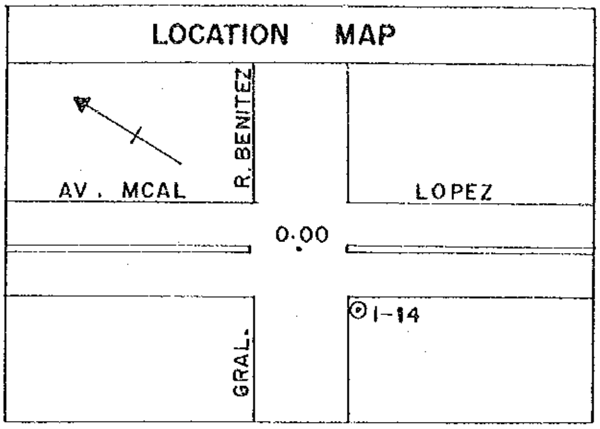
COLUMNAR SECTION (HOLE: I-14)

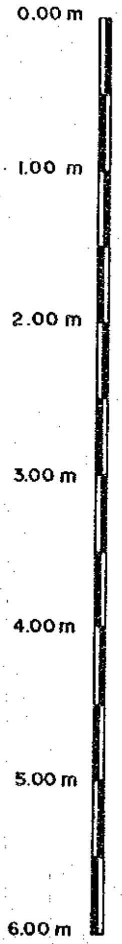
LOCATION:

GROUND HIGHT: 124.5 M.

DATE: 19-VI-86

DEPTH : 4.50 M





DEPTH	ELEVATION	PROFILE	DESCRIPTION
4.50	120.0		RED CLAYEY SAND (L)

1:50

Fig. 2-3(30/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

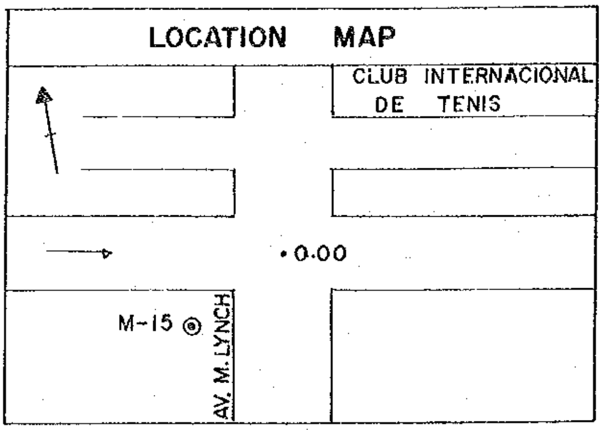
COLUMNAR SECTION (HOLE: I–15)

LOCATION:

GROUND HIGHT: 98.6 M.

DATE: 27-VI-86

DEPTH : 2.50 M



DEPTH	ELEVATION	PROFILE	DESCRIPTION
1.00	97.6		STREAM SEDIMENT (L) (DARK GRAY SAND)
2.50	96.1		YELLOWISH GRAY CLAYEY SAND(M) (CAN NOT PENETRATE THE BIT BE-CAUSE IT'S VERY HARD)

0.00 m, 1.00 m, 2.00 m, 3.00 m, 4.00 m, 5.00 m, 6.00 m

1:50

Fig. 2-3(31/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY

COLUMNAR SECTION (HOLE: I-16)

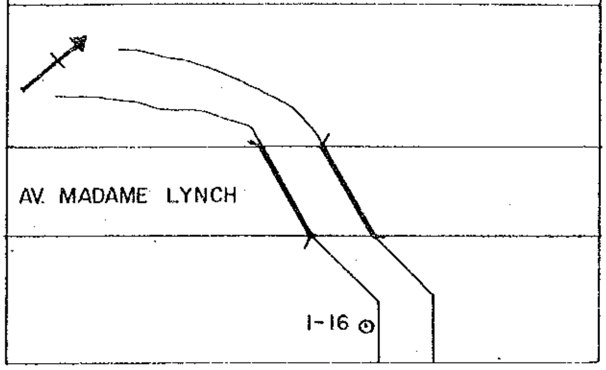
LOCATION:

GROUND HIGHT: 91.4 M.

DATE: 28-VI-86

DEPTH : 4.50 M.

LOCATION MAP



DEPTH	ELEVATION	PROFILE	DESCRIPTION
0.30	91.1		BACKILL RED (L)
1.70	89.7		DARK GRAY SANDY CLAY (M)
2.50	88.9		YELLOWISH GRAY SANDY CLAY (M)
4.00	87.4		SANDY CLAY (M)
4.50	86.9		WHITE SANDY CLAY (H)

1:50

Fig. 2-3(32/32) COLUMNAR SECTION

STORM DRAINAGE SYSTEM IMPROVEMENT PROJECT IN ASUNCION CITY, PARAGUAY

JAPAN INTERNATIONAL COOPERATION AGENCY